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PROCEEDINGS

OHIO RIVER VALLEY SOILS SEMINAR

"LATERAL EARTH PRESSURES"

OCTOBER 27, 1972

Fort Mitchell, Kentucky

Volume 1 of 2

Division of Research

LEXINGTON

COMMONWEALTH OF KENTUCKY

CINCINNATI SECTION

Dept. of C.E.
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PROCEEDINGS 1972
OHIO RIVER VALLEY
SOIL SEMINAR

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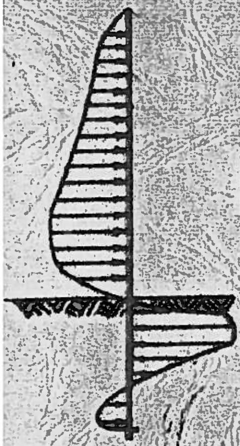
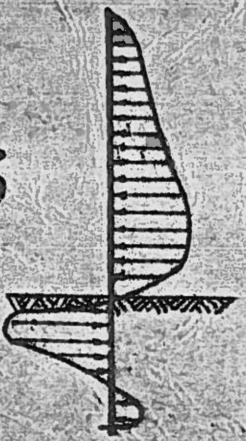


LATERAL EARTH PRESSURES



OCTOBER 27, 1972

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Proceedings

of

THIRD OHIO VALLEY SOILS SEMINAR

LATERAL EARTH PRESSURES -

CASE HISTORY STUDIES

October 27, 1972
Ft. Mitchell, Kentucky

Sponsored by

Cincinnati Section ASCE
Cincinnati-Dayton Soil Mechanics and Foundation Group

University of Cincinnati
Department of Civil and Environmental Engineering

and

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Soil Mechanics and Foundations Group

University of Kentucky
Department of Civil Engineering

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Department of Civil Engineering

1972 OHIO RIVER VALLEY SOIL SEMINAR

LATERAL EARTH PRESSURES

FOREWORD

The third Ohio River Valley Soil Seminar, held October 27, 1972 at the Rowntowner Inn in Ft. Mitchell, Kentucky, has as its theme: LATERAL EARTH PRESSURES.

The general purpose of the Seminar and this proceedings is to review existing knowledge and new techniques for the investigation, design and construction of facilities involving lateral earth pressures.

Presentations include field measurements, computations and analyses, construction case histories, and project critiques.

ACKNOWLEDGEMENTS

This symposium has been organized by:

Soils Group
Cincinnati Section of ASCE

University of Cincinnati
Department of Civil Engineering

It has been ably supported by:

Soils Group
Kentucky Section of ASCE

University of Kentucky
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We gratefully acknowledge the efforts of our Task Committee:

N. Schomaker, Environmental Protection Agency

R. Goettle, Goettle Construction Company

M. Nethero, H. C. Nutting Company

A. Bodocsi, University of Cincinnati

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Cincinnati, Ohio

October 1972

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INSTRUMENTATION FOR
STRESS AND STRAIN MEASUREMENTS IN SOILS

by

Stanley D. Wilson*)

INTRODUCTION

Although stress gages have been used for at least 50 years to measure the stress distribution in soils and the pressures produced on the surfaces of buried structures, the state-of-the-art today still does not permit such measurements to be made with a high degree of reliability. However, recent developments using pneumatic piezometers combined with oil-filled cells show promise of reasonable success. Details of such instruments and data from several typical installations are presented in this paper.

In contrast to earth pressure instrumentation, there now exists a variety of instruments for measuring small strains in soils with great precision and reliability. Typically these consist of some form of linear potentiometer coupled to an anchor some distance away. When the distance is short, say up to 15 feet, such a device is referred to as a strain meter; at greater distances it is called an extensometer. The basic requirements of such a device are that it be stable with time, i.e., have zero drift, be non-sensitive to temperature changes and have minimal hysteresis effects. Typical instruments and data are presented herein.

FACTORS INFLUENCING PRESSURE CELL DESIGN

The basic problem in designing a pressure gage is to make it have identical compressibility and stiffness characteristics to those of the surrounding soil. Thus a gage embedded in an embankment should be designed to change thickness exactly the same amount as the adjacent soil during completion of the embankment above the cell. Even if one knew in advance how compressible the soil was going to be after compaction, it would be nearly impossible to duplicate the effects of secondary compression and of

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changes in pore water pressure under constant total stress. Furthermore, consider the complexity of designing such a gage to be placed in a vertical plane to measure horizontal stresses perpendicular to the axis of a dam. Such embankments may develop extension strains in the upstream-downstream direction during construction even though horizontal stresses increase. To read correctly, such a gage might have to be designed to expand laterally as the vertical stresses increased.

Now consider a gage embedded in the face of a rigid concrete wall to measure the horizontal component of backfill against the face of the wall. This gage should not only be as rigid as the wall itself but also should not be affected by shear stresses along its face.

Other factors influencing gage design and performance are:

1. Diameter of gage relative to size of soil or rock particles (Brown, 1967)
2. Placement procedures
3. Type of sensing element
4. Frequency response
5. Density

The influences of gage stiffness and of thickness-diameter ratio (T/D) on the stress distribution around the gage are illustrated in Fig. 1. In general, gages for free-field measurements should have a T/D ratio less than 1/5 and a diameter-deflection ratio greater than 2000. When this is done the over or under registration will probably be within tolerable limits provided the five factors listed above are satisfactorily taken care of.

FREE FIELD STRESS GAGES

In evaluating the performance of embankments and of earth and rockfill dams, the most common need is for a stress gage that can be buried in the embankment and record the changes in stress that develop as the embankment is built and then put into usage. Such a gage must be fairly large in diameter so as not to be unduly influenced by rocks or gravel or local irregularities in the embankment. Its frequency response need be only for static loads (except in the case of measuring the stress under moving wheel loads). The one most significant requirement is that the sensing element be stable, i.e., to have zero drift and no change in calibration with time. Because of the hostile environment (moisture and corrosion) in which the gage is embedded, and the long leads to the terminal house, sensors which rely on electrical circuits are generally unsatisfactory. Therefore, one generally avoids SR-4 strain gages, LVDT sensors and the like.

Around 1950, A. Casagrande designed some pressure cells about 3 feet in diameter and less than 1" thick, consisting of a rigid rim with two flexible sheet metal diaphragms. These cells were filled with oil and connected to Bourdon pressure gages some distance away by means of copper tubing. Although these gages appeared to have ideal characteristics, the results were somewhat disappointing. The reasons for this were a combination of leaks at the welds, inability to remove all the air from the system, volume changes and leaks in the tubing and volume changes in the Bourdon gage.

Within the past decade there have been significant improvements in pneumatic and hydraulic piezometers which permit a low volume change pressure measuring device to be placed in close proximity to the flat cell embedded in the soil. The Glotzl cell was the first of such devices. The CFE pressure cell developed in Mexico by Marsal utilizes a pneumatic piezometer for the pressure sensing element and a similar cell developed by the Slope Indicator Company of Seattle is shown schematically in Fig. 2. Experience with these cells has been excellent and actual case history records are included in this paper. Fig. 3 shows a similar cell except that the sensor is actuated by a LVDT, thus it responds to dynamic stresses.

All soil pressure gages measure total pressure, i.e., effective stresses plus neutral stresses. In order to determine soil pressure only (effective stress), the neutral stress as determined by an adjacent piezometer, must be deducted from the total stress.

EXAMPLES OF PRESSURE GAGE MEASUREMENTS

Guadalupe Dam, located northwest of Mexico City, provides an interesting example of earth pressure measurements. This dam was first constructed in 1940 as a rockfill dam with upstream facing of concrete. It was on a compressible foundation and during first filling the concrete facing failed. Next an upstream berm of silt was placed, but this also failed by piping into the previously failed slab. Finally in 1966 the embankment was reconstructed as a sloping core rockfill dam utilizing highly plastic clay and was designed to accept large settlements and deformations. Extensive instrumentation was installed to monitor the performance of the structure. It was filled successfully in 1971 for the first time.

Fig. 4 shows typical pressure cell readings over a 4-year period covering construction of the embankment and subsequent controlled first fitting of the reservoir. The cluster of 4 CFE cells whose data are shown are located in the lower portion of the clay core (see inset Fig. 5). The response of the cells to first the embankment construction and then to successive cycles of reservoir filling and emptying is clearly indicated.

Figs. 5 and 6 show typical stress ellipses from 2 clusters of cells in the same section. During construction the weight of the rock above the clay caused the clay core to settle and to displace upstream. Note that in Fig. 5 the major principal stress is inclined more or less parallel to the inclined core whereas the upstream cluster (Fig. 6) reveals the major principal stress to be nearly horizontal, the result of the upstream displacement. In the opinion of the author, these measured stress values appear to be realistic. The stresses shown in Figs. 5 and 6 are those recorded by the pressure cells in November 1970 (Fig. 4).

During the construction of La Vallita Dam on the Balsas River in Mexico, CFE pressure cells were installed in both the clay core and the rockfill embankment at several sections. After completion of the embankment, a boring was being made in the core near one of the cell clusters, the data from which are shown in Fig. 7. Drilling mud was being used to keep the hole open and to remove the clay from the cutting head. At depth approximately 100 feet, the depth of the pressure cell, there was a sudden loss of drilling mud and it was subsequently found that some of this mud had contaminated other instruments more than 30 feet away in a direction along the axis. Note that at a depth of 100 feet, drilling mud weighing 68 lbs/cu. ft. would exert a pressure of 3.4 TSF, the exact value of the minor principal stress shown in Fig. 7. The evidence is substantial that hydraulic fracturing of the clay core had resulted from the drilling operation. In other words, the pressure in the fluid at the bottom of the hole resulting from the weight of the drilling mud and possibly excess pumping pressure exceeded the minor principal stress, thus cracking the clay and permitting the mud to escape through the crack. Such fluid losses are common whenever exploratory holes are drilled in cores of dams and, in fact, similar procedures are used to determine the in situ stress as described subsequently in this paper.

IN SITU STRESS MEASUREMENTS

The determinations of the in situ stress of natural soil and rock formations are even more difficult than of measuring earth pressures in embankments. In rocks, numerous techniques are available such as overcoring, stress-relief, flat jacks, etc., but such techniques are not generally applicable to soils. Recently, however, Bjerrum & Andersen (1972) have investigated a technique for determining the minor principal stress based on the principals of hydraulic fracturing. They found while performing ordinary in situ permeability tests in clay, that when establishing a water pressure higher than a certain critical value, the permeability apparently increased abruptly. The reason for this was that cracking occurs in the soil around the piezometer tip when the water pressure gets large enough to cause tensile stresses in the soil next to the piezometers, thus resulting in a sudden increase in flow of water into the soil. However, the pressure at which fracturing occurs is usually somewhat greater than the in situ stress and a more reliable indicator is the water pressure at which the crack closes. The technique developed consists of first forcing water into the soil at a rate which produces enough pressure to form a crack. Then the water pressure is allowed to decrease slowly, at the same time observing the rate of flow. An abrupt reduction in the rate of flow is observed when the crack closes. Fig. 8 is a sketch of the field apparatus, and Bjerrum and Andersen (1972) describe the test procedure as follows:

"The test is performed by forcing water out or the screw control at a constant rate large enough to cause cracking. As a standard, a rate pf 1.5 cm^3/min, corresponding to 12 turns per minute of the screw control, has been adopted. During this process the water pressure should be read at the mercury manometer for instance every half minute. As the pressure approaches a constant value, forcing water out of the screw control is stopped, and the pressure and the rate of flow are allowed to adjust freely to each other like in a "falling head" test. The pressure should now be read at certain time intervals, the intervals being small to begin with and becoming longer as the rate of the drop in water pressure decreases. A drop in the pressure in the mercury manometer corresponds to a certain volume of water flowing out of the manometer, and by means of the pressure readings at certain time intervals, a connection between pressure and rate of flow into the soil can be calculated; these data are presented as the curve shown in Fig. 9".

The method, although promising, is limited to the determination of the minor principal stress only in fine-grained impervious soils.

IN SITU STRESS-STRAIN MEASUREMENTS

The determination of the in situ stress-strain relationships of soils can be made in a number of ways as listed below:

- Plate bearing tests at ground surface
- Plate bearing tests at bottom of test pits
- Horizontal plate bearing tests in test pits, caissons or tunnel drifts

Large-scale loading tests
Various types of expanding bulbs
Goodman Borehole Jack

The Goodman Jack (Fig. 10) is a comparatively new development and consists of a hydraulic jack with curved bearing plates for use inside a standard NX (3-inch diameter) borehole. It is especially suitable for use in coherent rock and the degree of anisotrophy can be investigated in situ with this instrument by rotating the device in the hole. The instrument consists of two curved bearing plates which are forced against the wall of the borehole by hydraulic pistons. Pressure in the hydraulic line produces a uniform and unidirectional stress field at the bearing plate. A hand-operated pump can easily produce this pressure. After a test, the jack is retraced by means of two reversed pistons to provide clearance in the borehole.

Under pressure, the borehole deformation is measured very accurately by two self-contained LVDT's (Linear Variable Differential Transformers), which are read out on a portable indicator. The applied hydraulic pressure is measured with a standard bourdon-type pressure gauge.

Two models are available. For use in rock, multiple oval-shaped pistons are used with a pressure capability of 10,000 psi. With three circular pistons, the jack can be used in soft rock, soil and stiff clay.

STRAIN MEASUREMENTS IN SOILS

A variety of techniques and apparatus have been developed for measuring strains in soils. Such measurements are needed, for example, in the interior of earth and rockfill dams, below the foundations of structures, adjacent to cut slopes and beneath large excavations. Bore-hole extensometers are precision instruments designed to measure strain within earth and rock masses. They are capable of measuring large strains with a sensitivity of as little as 0.002 inch axial movement from six positions within a boring. An instrumented boring can be horizontal, inclined or vertical. All movements are referenced to a base point at the surface.

A typical extensometer assembly consists of a portable readout strain indicator, a reference base extensometer assembly and fixed anchors. The change in distance between the reference base extensometer assembly and each anchor in the boring is measured by balancing a Wheatstone Bridge circuit, the reading being indicated on a digital readout potentiometer. The electrical output may also be recorded automatically on a multi-channel, strip-chart recorder. These electrical readings are easily converted to strain measurements.

The six position extensometer assembly, shown in Fig. 11, utilizes six precision potentiometer sensing elements. These are actuated by flexible stranded stainless cables attached to solid, restraightened stainless steel spring wires leading to the anchors. The flexible cable passes over a pulley mounted on the potentiometer shaft. Each cable is terminated by a pair of bearing-mounted constant tension springs. The pulley diameter is such that two inches of wire movement equals slightly less than one turn of the shaft. There are no stops on the potentiometer, therefore movement of the cable may continue beyond two inches and data continuously obtained. Since the total travel of the springs is about 3-1/2 inches, deformations greater than this amount will require readjustment of the cable terminations on the springs. These springs provide a thirty-pound force over the spring extension range. The force is constant within 1% for either continuous extension or retraction.

A new type of strain meter (Fig. 12), consisting of multiple units in tandem which can be extended to any desired length, is finding increased usage in earth dams to measure strains along the crest and adjacent to abutments, for example. The instruments are designed for direct burial.

The sensing element is a linear potentiometer which has a resistance element of conductive plastic. The potentiometer and its electrical connections are potted and sealed within a protective enclosure of PVC pipe. A rod 1/4 inch in diameter connects the potentiometer shaft to the anchor point. The rod material is normally stainless steel. To eliminate temperature effects on long lengths, Invar rod is used. The rod is enclosed within PVC pipe or polyethylene tubing depending on the application.

The strain meter may also be used for measuring heave under excavations. Coiled electrical leads are enclosed in a riser pipe and can be extended more than six times the closed length. This allows the leads to be lowered into the pipe when not in use and they can be retrieved for reading. In case the riser pipes are damaged by excavation equipment, the leads are coverable.

The readout used with these strain meters is a portable, battery-powered unit with a digital indicator reading from 1 to 1000 dial units. It utilizes a Wheatstone-bridge circuit which is manually balanced until a null condition is achieved as indicated on the panel mounted galvanometer. At this point, the reading of the dial indicator is recorded. The dial reading is in percent of strain, which for a 10-foot gauge length and 6-inch range is 0.05% strain per dial unit. The strain meters are connected to the portable indicator by multi conductor cables either directly or through a terminal box.

The instruments are also used to measure settlements and rebound beneath excavations and structural foundations such as caissons, piers, and mats.

The displacement range of the sensors is optional and can be selected to meet requirements for measurements in rock or soil. The basic sensing unit is adaptable to other uses such as monitoring slope stability and structural joint movement.

Sometimes single-unit strain meters, consisting of an anchor, a connecting rod,say 10 feet long, enclosed in telescoping casing, and a sensing element are grouped together in clusters of 3 to 6 to measure components of strain in the same directions in which the earth pressures are measured. Often the vertical strain (resulting from compression of the material) is largely compared to the horizontal strains and is often determined by measuring the closing of the gaps between telescoping inclinometer casing which is also installed during construction.

CONCLUDING REMARKS

Recent developments in instrumentation now make it possible to measure with precision in situ stress and stains within earth and rockfill dams, and under foundations of structures. As more and more reliable data become available and the results analyzed and compared with theoretical studies using the finite element method of analyses, (see Duncan and Kulhawy, 1972) our ability to design safe structures is bound to improve. Further improvements in automation will make possible continuous surveillance of the safety of important dams.

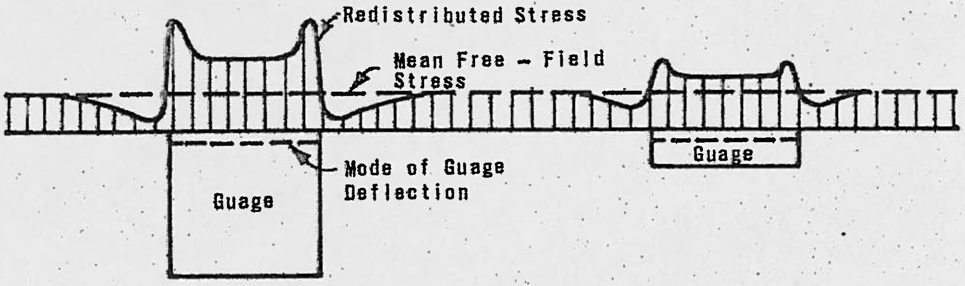
LIST OF REFERENCES

Brown, Colin C. (1967), "Pressure Meter Design in Fills with Rock Inclusions," Geotechnique, Vol. XVII, Number 3, September, 1967.

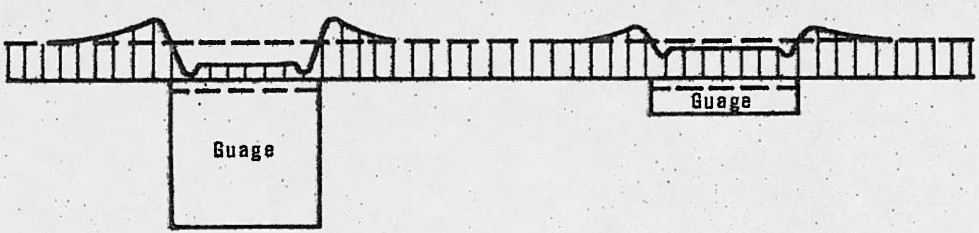
Bjerram, L. and Andersen, K.H. (1972), "In Situ Measurement of Lateral Pressures in Clay", Preprint of paper presented at 5th European Conference on Soil Mechanics and Foundation Engineering, Madrid, April, 1972.

Duncan, James M., and Kulhawy, Fred H. (1972), "Stresses and Movements in Oroville Dam"; Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 98. No. SM 7, Proc. Paper 9016, July 1972, pp. 653-665.

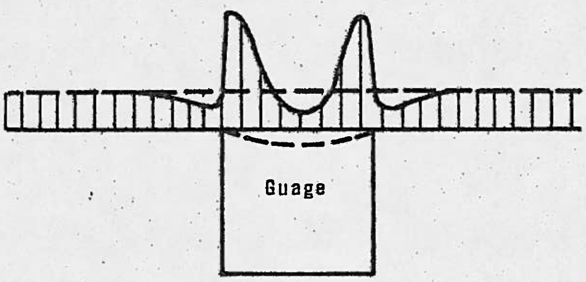
Selig, E.T. (1964), "A REview of Stress and Strain Measurements in Soil", Proceedings of the Symposium on Soil Structure Interaction, pp. 172-186, University of Arizona, September 1964.



(a) GUAGE MORE STIFF THAN SOIL



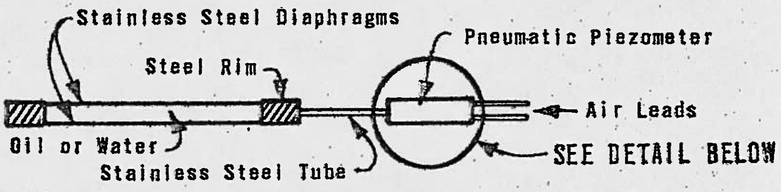
(b) GUAGE LESS STIFF THAN SOIL



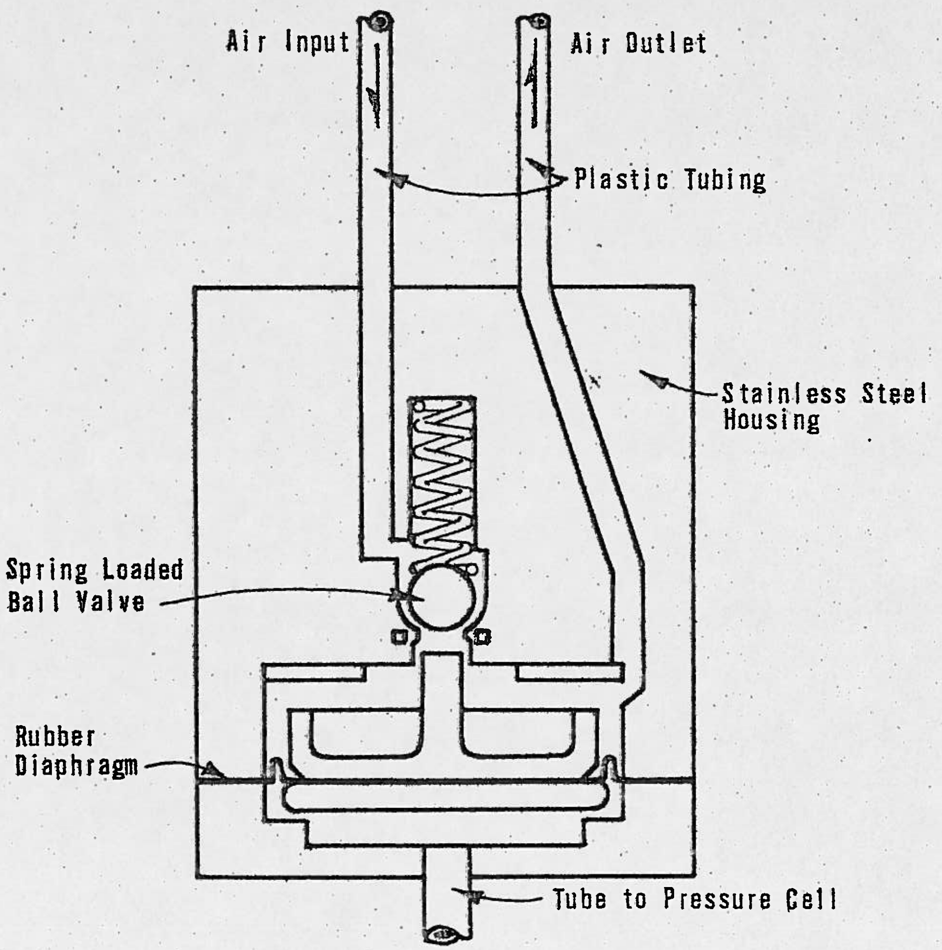
(c) STIFF GUAGE WITH FLEXIBLE DIAPHRAGM

(After Selig, 1964)

FIG. 1. STRESS DISTRIBUTION AROUND EMBEDDED GUAGES



PRESSURE CELL



SINCO PNEUMATIC PIEZOMETER

FIG. 2. SCHEMATIC DRAWING OF EARTH PRESSURE CELL WITH PNEUMATIC SENSOR

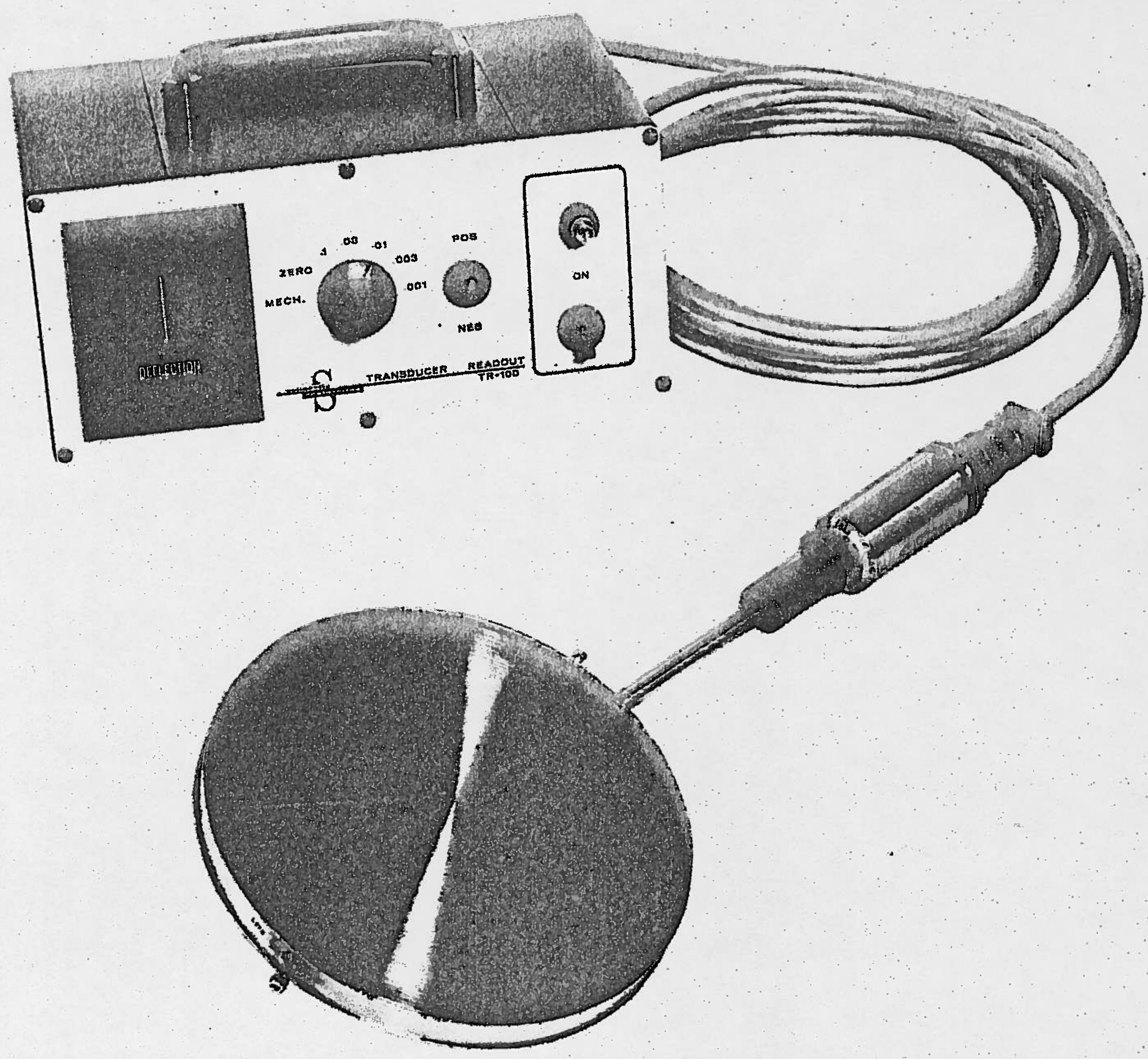


FIG. 3. OIL – FILLED EARTH PRESSURE CELL
WITH LVDT SENSING ELEMENT

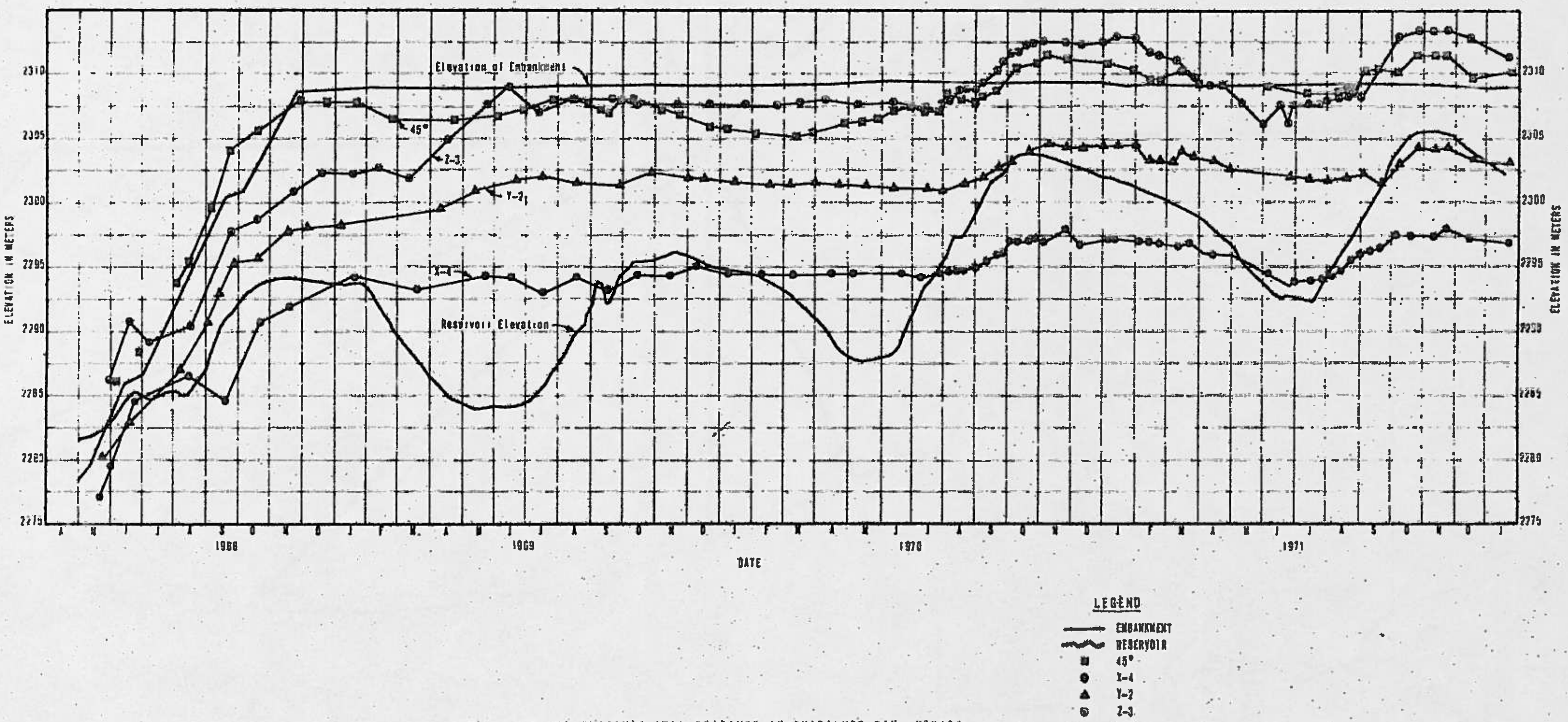
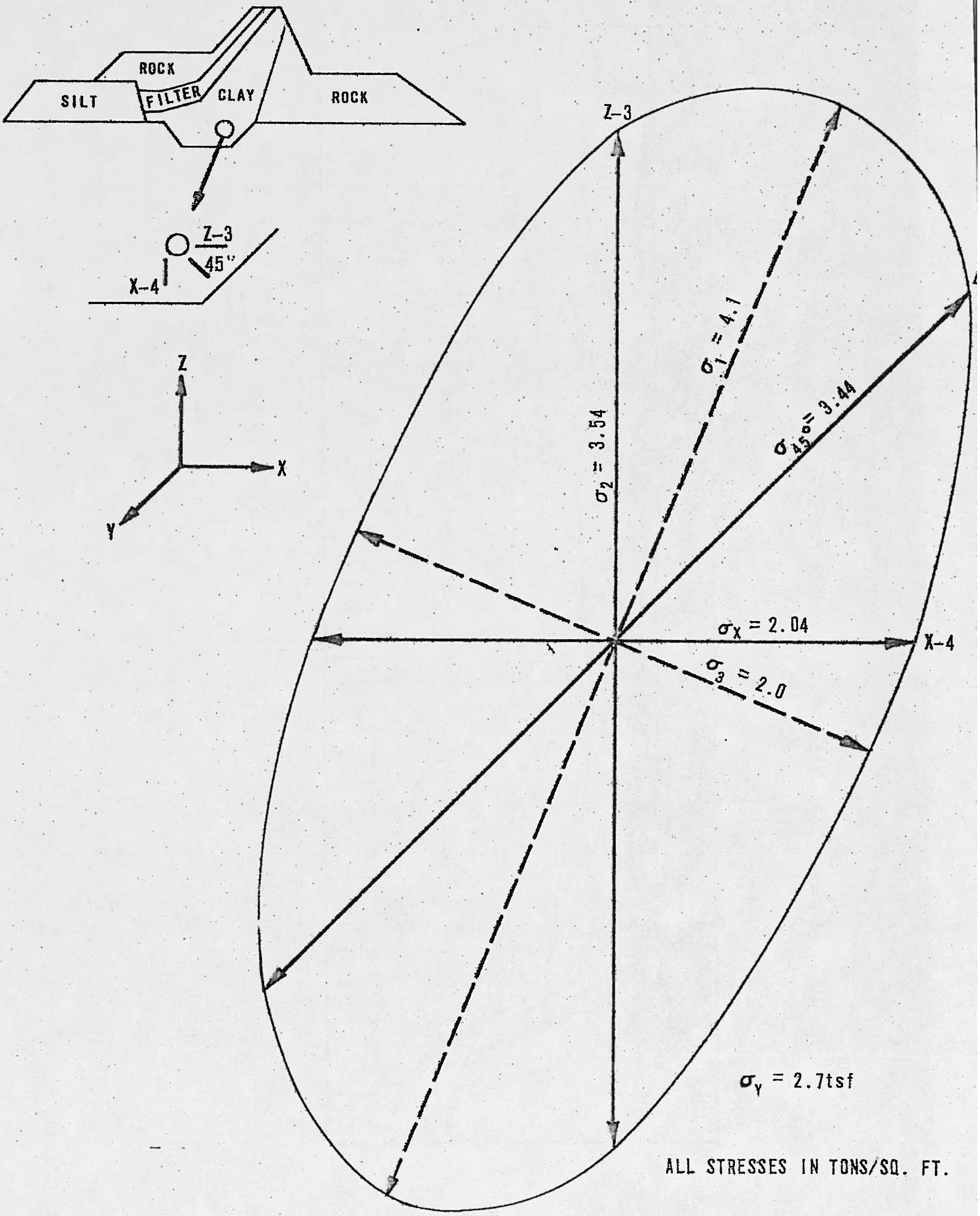
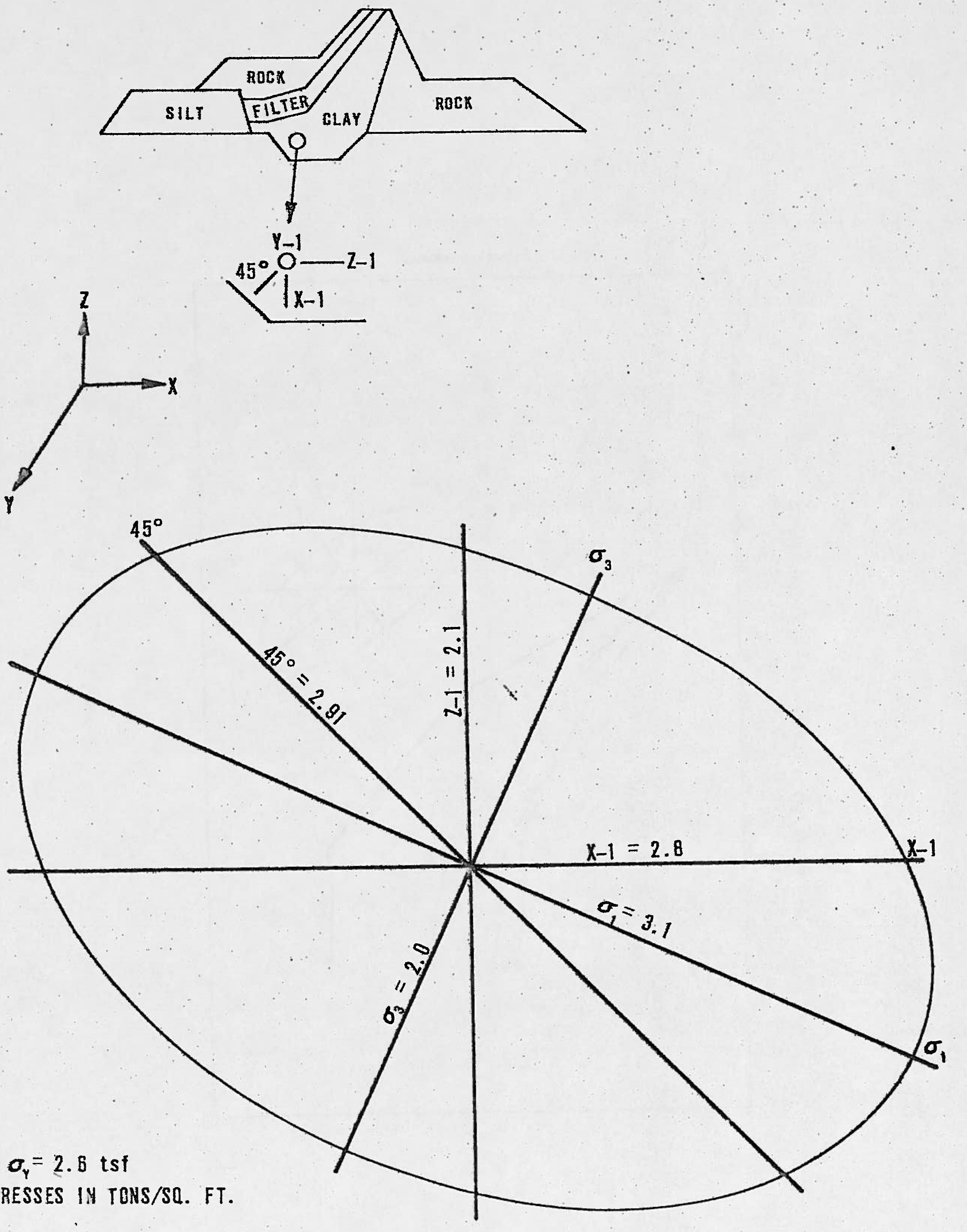


FIG. 4 TYPICAL PRESSURE CELL READINGS AT GUADALUPE DAM, MEXICO



ALL STRESSES IN TONS/SQ. FT.

FIG. 5. TYPICAL STRESS ELLIPSE, GUADALUPE DAM, MEXICO



NOTE: σ_Y = 2.6 tsf
ALL STRESSES IN TONS/SQ. FT.

FIG. 6. STRESS ELLIPSE, GUADALUPE DAM, MEXICO

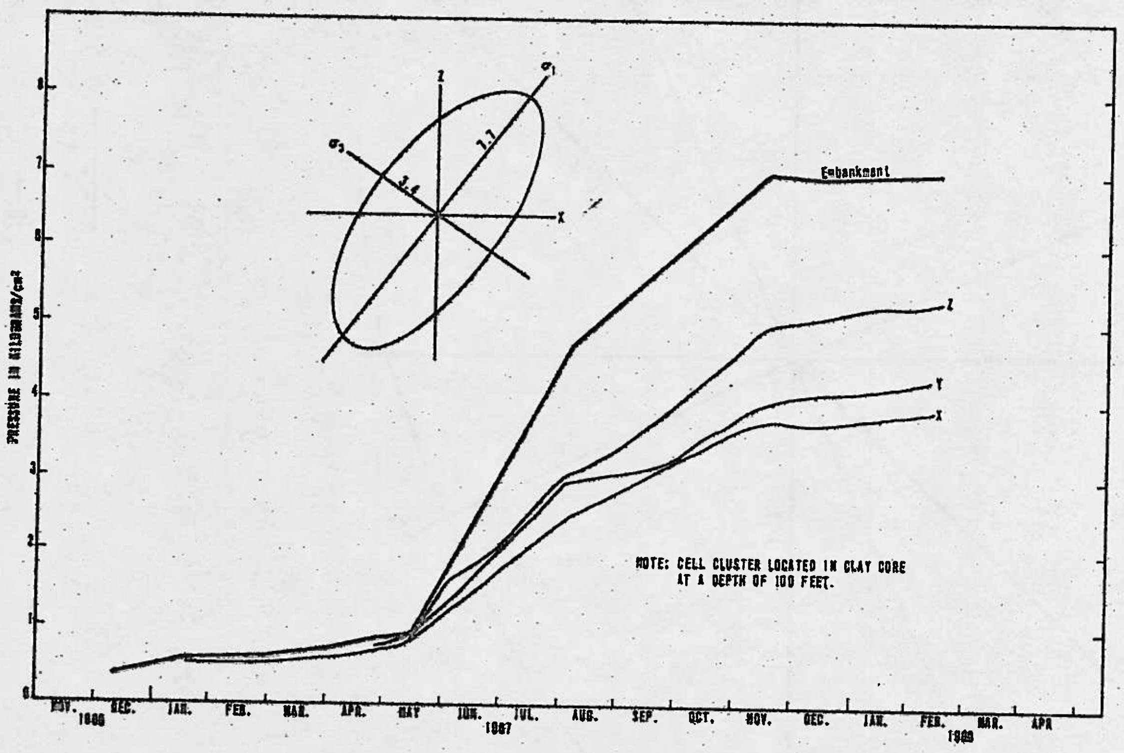
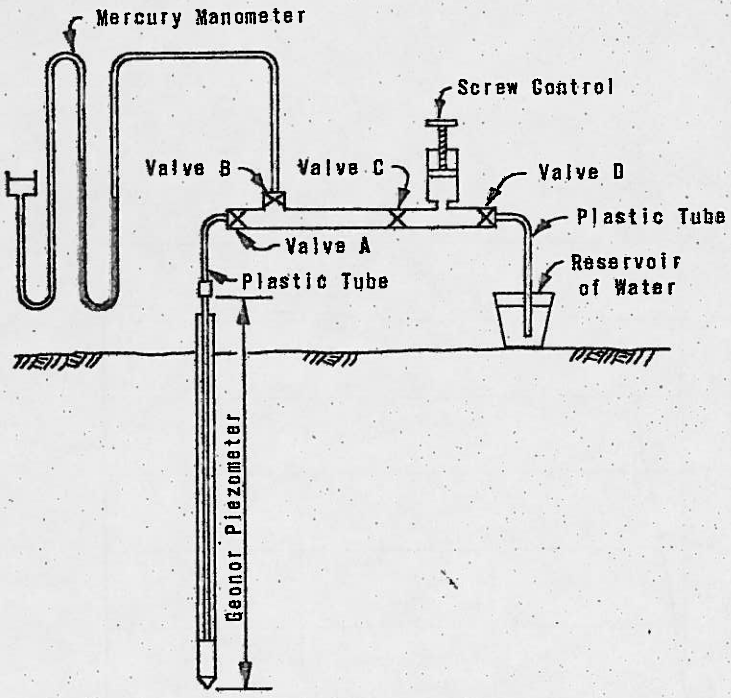
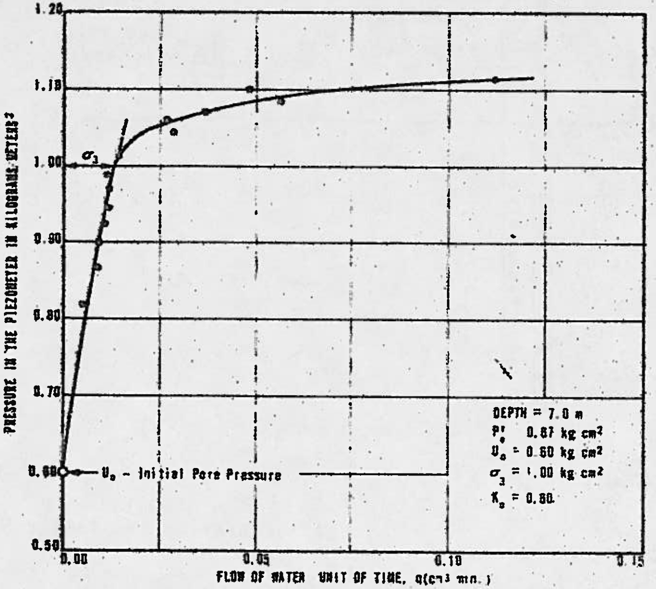


FIG. 7. TYPICAL PRESSURE CELL DATA FROM LA VALLITA DAM, MEXICO



(After Bjerrum and Anderson, 1972)

FIG. 8. SKETCH OF APPARATUS FOR DETERMINATION OF IN-SITU LATERAL STRESS IN CLAY



(After Bjerrum and Andersen 1972)

FIG. 9. TYPICAL DATA FROM HYDRAULIC FRACTURING TEST IN CLAY

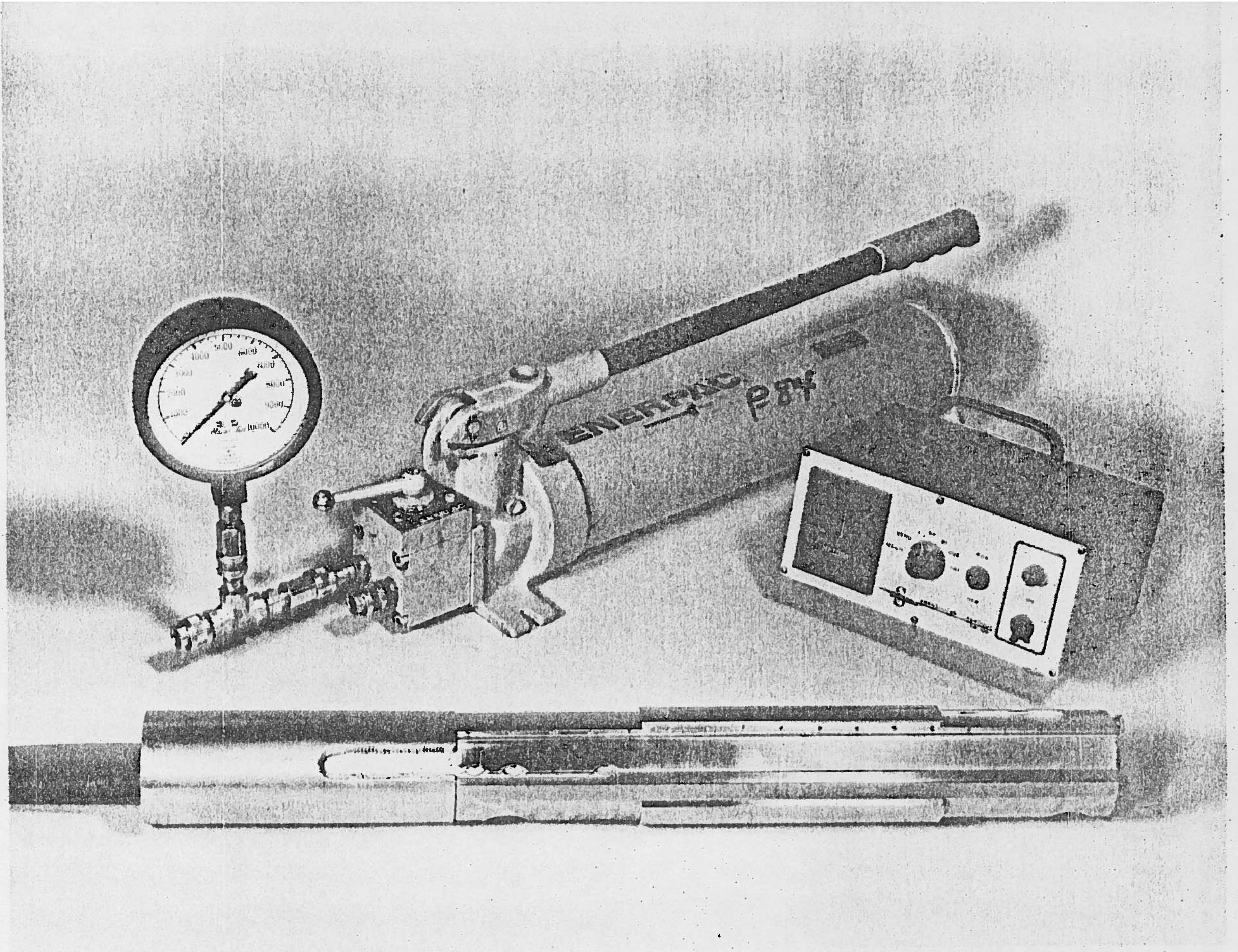


FIG. 10. GOODMAN BORE HOLE JACK

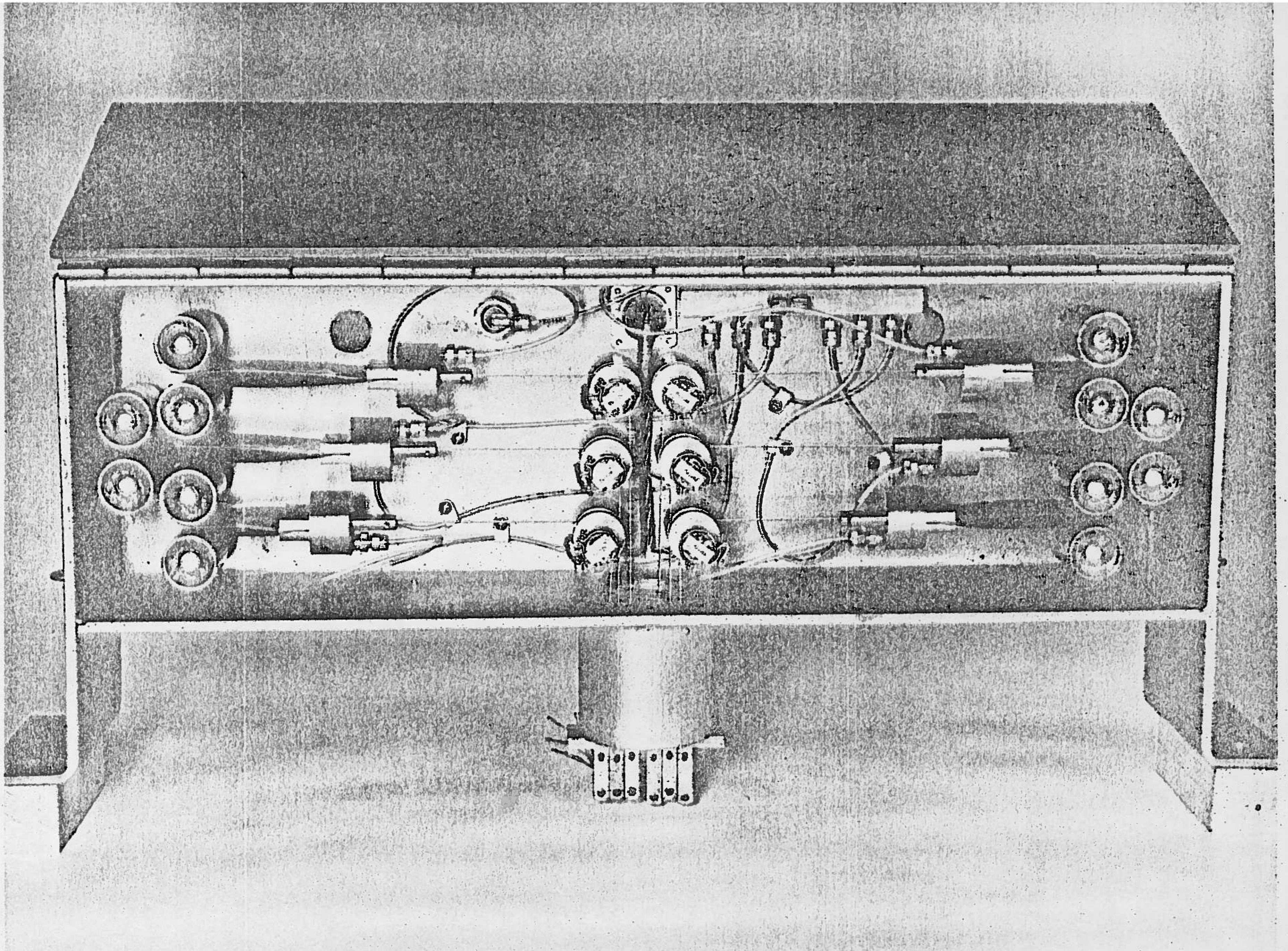


FIG. 11. SIX - ELEMENT RECTILINEAR EXTENSOMETER

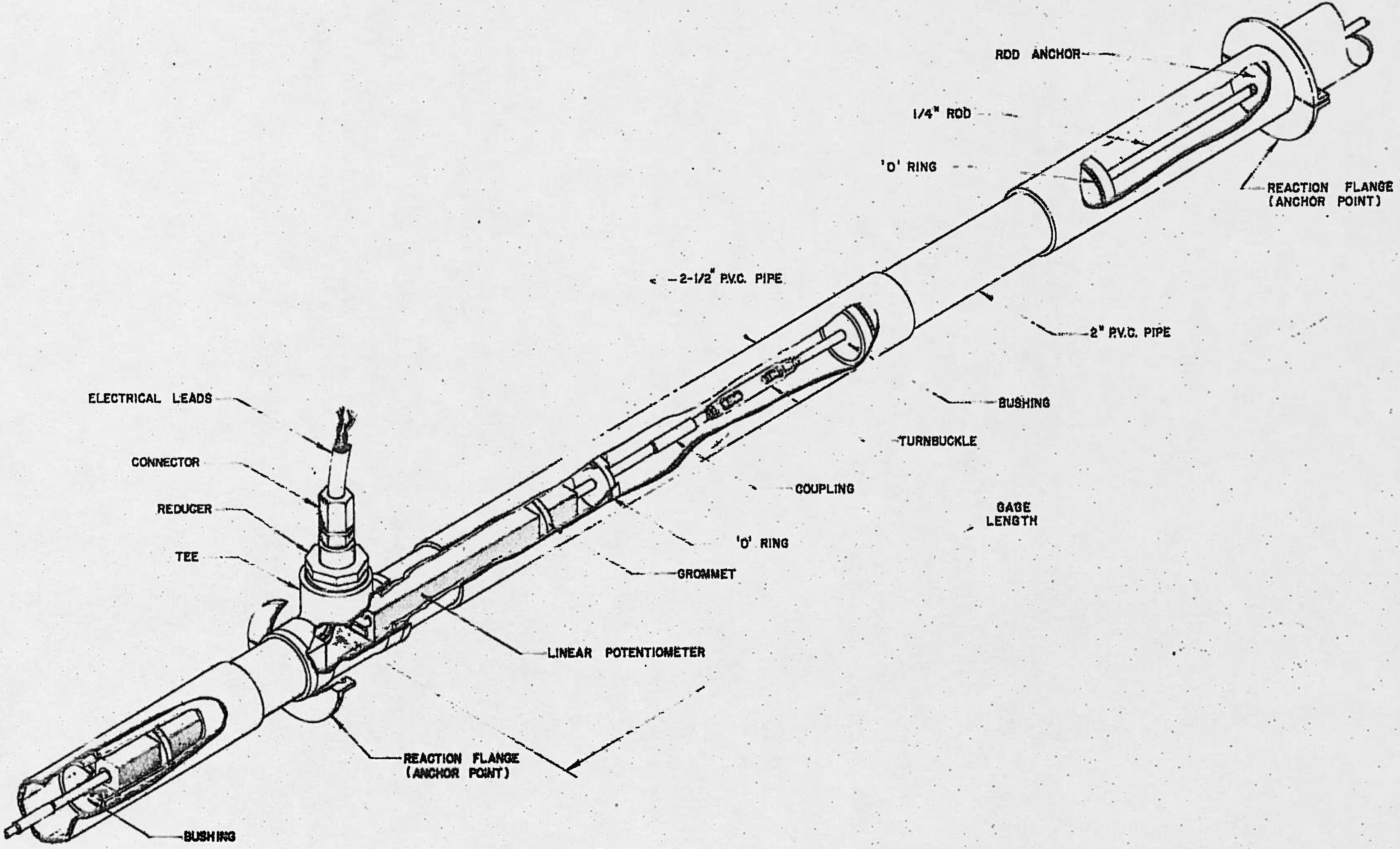


FIG. 12. MULTI – ELEMENT SOIL STRAIN METER

DEEP EXCAVATION AND BUILDINGS SUPPORTED BY TIE-BACK SYSTEM

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SECTION A

INTRODUCTION

We are going to present to you a rather thorough and detailed analysis of one specific hole in the ground, as viewed by, in order of appearance, the Contractor, the Structural Engineer, and the Soils Engineer.

In the fall of 1970, bids were asked for Block "H", an area in Cincinnati bounded by Walnut and Main Streets on the East and West and Sixth Street and Government Place on the North and South, approximately four hundred by two hundred feet in plan. The owner of this project is the Western and Southern Life Insurance Company.

The request for bids included a section relating to protection of streets, utilities, and abutting structures by whatever means the contractor chose with the condition that any damage would be the contractor's responsibility.

Protection of deep cuts in congested areas has undergone major changes in the last decade. Fifteen years ago this installation would have been done using sheet piling and cross lot bracing, a time consuming and congestion producing, sledge hammer operation.

Ten years ago we would have at least graduated to H beams and lagging, with a raked bracing system to footings previously installed behind berms. It is quite possible that a start on this project today would entail the construction of a slurry wall, combining the protection system and the exterior building wall.

The site has two encroachments, one temporary and one permanent. In the South-East corner is an existing building occupied by the Southern Ohio Bank which will be razed in the next few months when they move into their permanent quarters in the new structure. Temporary protection was necessary for this area. In the South-West corner is an eleven story bank building. Footing levels of the Southern Ohio building were normal basement levels ten to twelve feet below grade. Footing levels at the other bank were somewhat more complex. In the Northern half of the building footing elevation was fourteen feet below grade. In the Southern half of the building footings were twenty seven feet below grade.

Turner Construction Company, the General Contractor, asked for bids on either braced or tied-back excavation protection systems.

The use of tied as opposed to braced cuts is an economic matter. The tied cut must have advantages for the General Contractor, for the Excavator, and possibly for the Steel Erector over the braced installation or it simply won't be installed. Five or six years ago tied excavations were considerably higher in cost than were braced excavation and the economic advantages had to be considerable for the other people involved in the project before tied systems even became feasible. They have been frequently used in Europe for many years but Europe has different cost factors affecting construction.

As you may or may not know the basin area of Cincinnati is glacial out-wash, medium dense sand with some small gravel. One tie-back system using strands had been installed in Cincinnati prior to the bidding of this project. We felt that the installation techniques used on that project would have to be improved upon if we were to be competitive with braced hole excavations.

At this point in time, Burt Hartmann, representing the Bauer Anchor System, a German development, contacted us and declared unequivocally that the Bauer System would work. This system is designed to use a single bar with a pressure grout application to develop the stress in the bar. The grout earth interface is subjected to upwards of four hundred pounds per square inch pressure. The steel bars now in use have ultimate strengths of 200 kips each.

Bidding documents consisted of specifications and drawings EX1 and EX2. The bank building was detailed only to the extent that columns and pilasters were indicated and conventional pit underpinning was contemplated to support the wall columns, with restraint between pits to prevent loss of subgrade material. Efforts were made to obtain the construction drawings of the bank building, but the architectural firm had been out of business for sometime and no known plans existed.

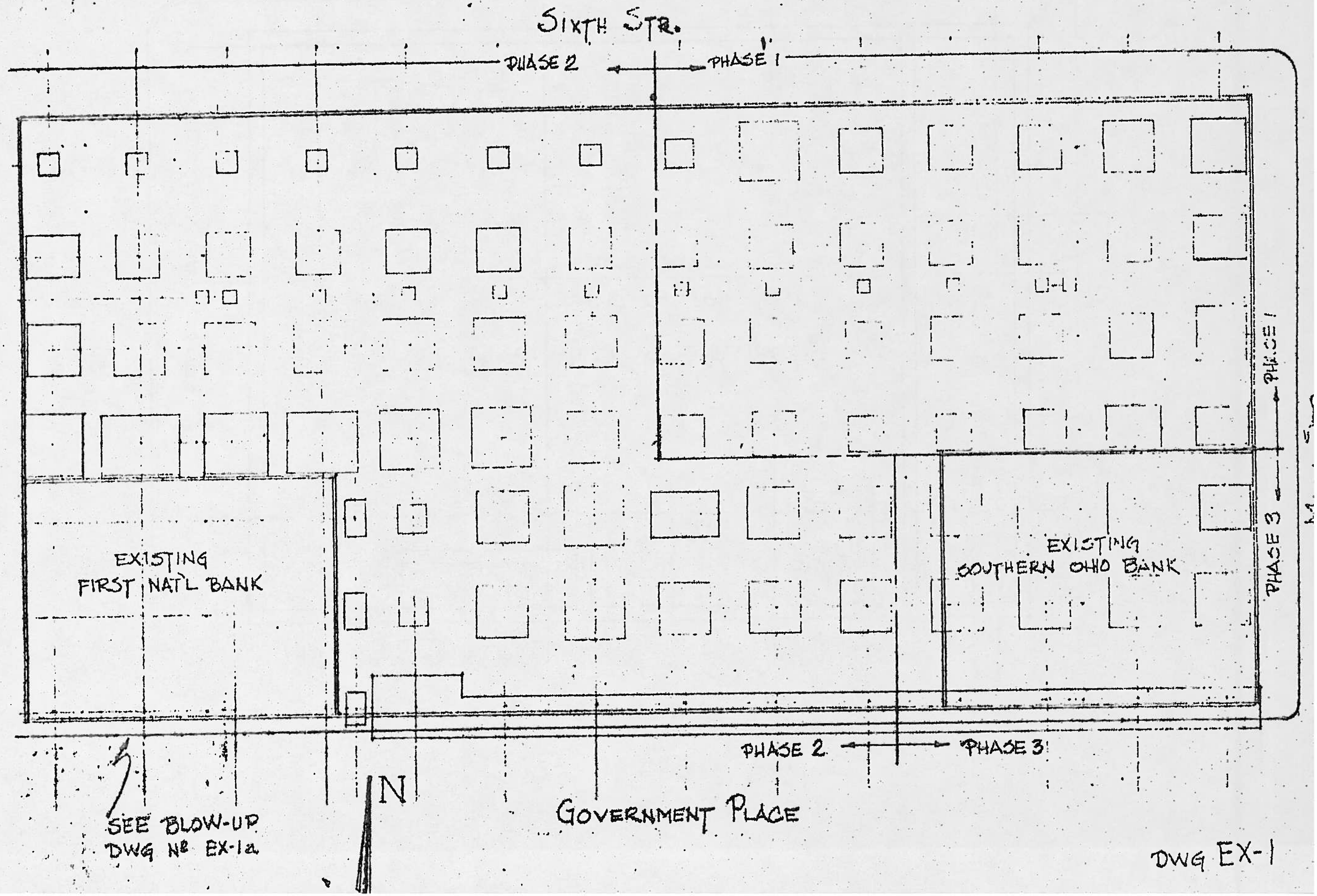
Temporary protection for the existing Western Southern Building was to be driven sheet piling on the north side to guarantee no loss of material and possible subsequent settlement and standard H beam and lagging, tied, on the west side, as we were well away from the building proper with the cut.

Our bid was submitted on both tied and braced types of construction for the same general price.

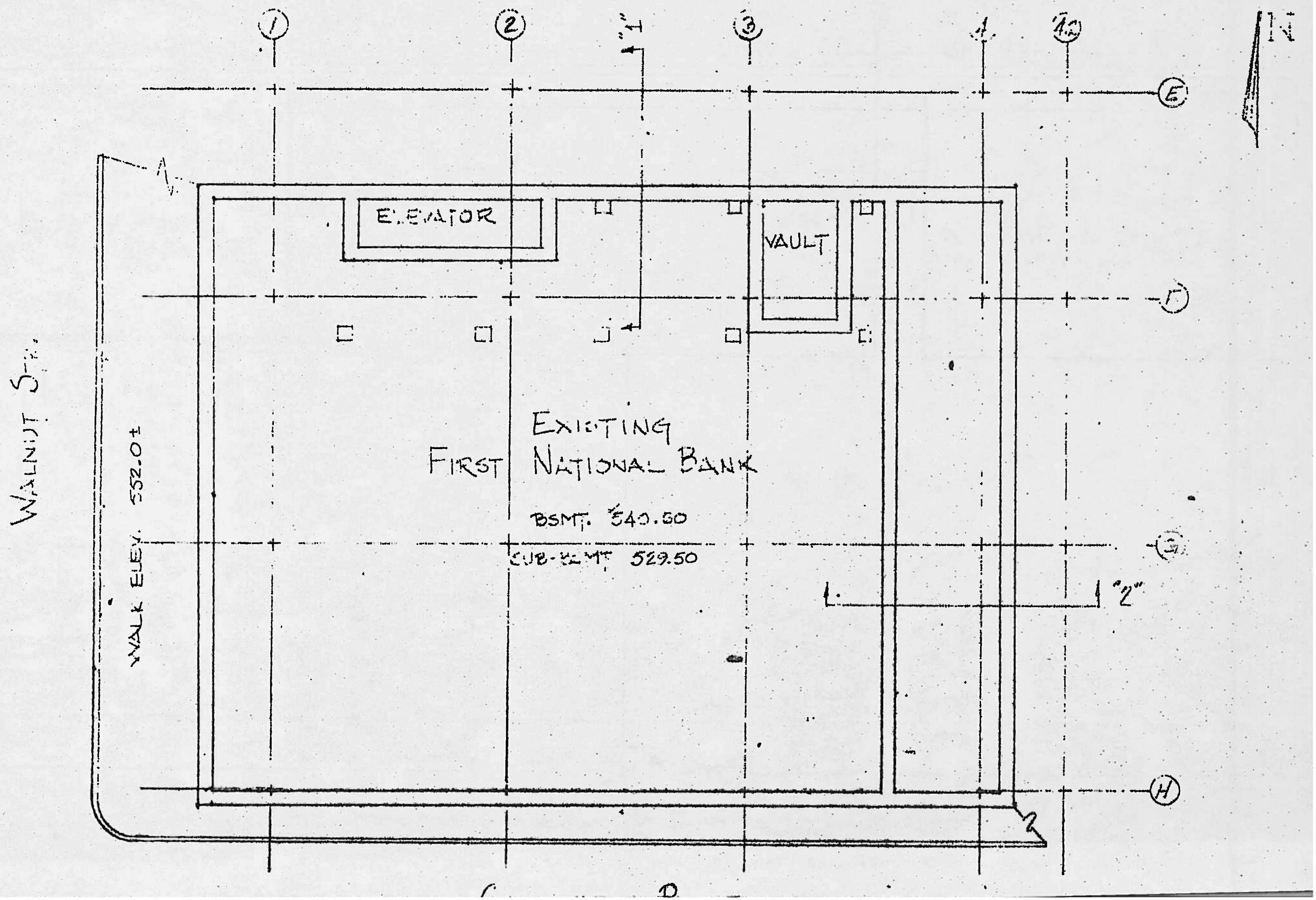
Shortly after an award was made the management at the bank building determined that a possible connection between the garage of the new building and their building might be useful. A set of plans then materialized showing the footing construction per plan sheet one, certainily a different configuration than contemplated, being a combined footing for the exterior and first interior column, strapped to the second interior column footing. This presented an extremely difficult underpinning operations, for determination of load distribution on this multiple footing can be little more than an educated guess. Any redistribution caused by the underpinning could have had serious structural side effects. Alternate schemes were then investigated. Mr. Miller will discuss this facet later.

Among these schemes we presented the tied lagging and H beam wall that was installed. Subsequent speakers will fully develop the structural and soils aspects of this installation.

I would like to comment briefly on one point, of paramount importance in my view. The use of anchors properly applied to the site conditions gives an almost foolproof solution. Every anchor is of known capacity upon installation, having been stressed beyond working load. The wall is often pulled back into the soil mass. K factors for active conditions have been used to design these systems, but the installation techniques used actually create K's more nearly related to "at rest" values. To duplicate this with prestressed bracing systems would be enormously exorbinant, either cross lot or raked, and it is impossible to restrain movement without prestressing.


SIXTH STR.
PHASE 2
PHASE 1
PHASE 1
PHASE 3
EXISTING FIRST NAT'L BANK
EXISTING SOUTHERN OHIO BANK
PHASE 2
PHASE 3
N
GOVERNMENT PLACE
SEE BLOW-UP DWG Nº EX-1a

DWG EX-1


1
2
"1"
3
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4.2
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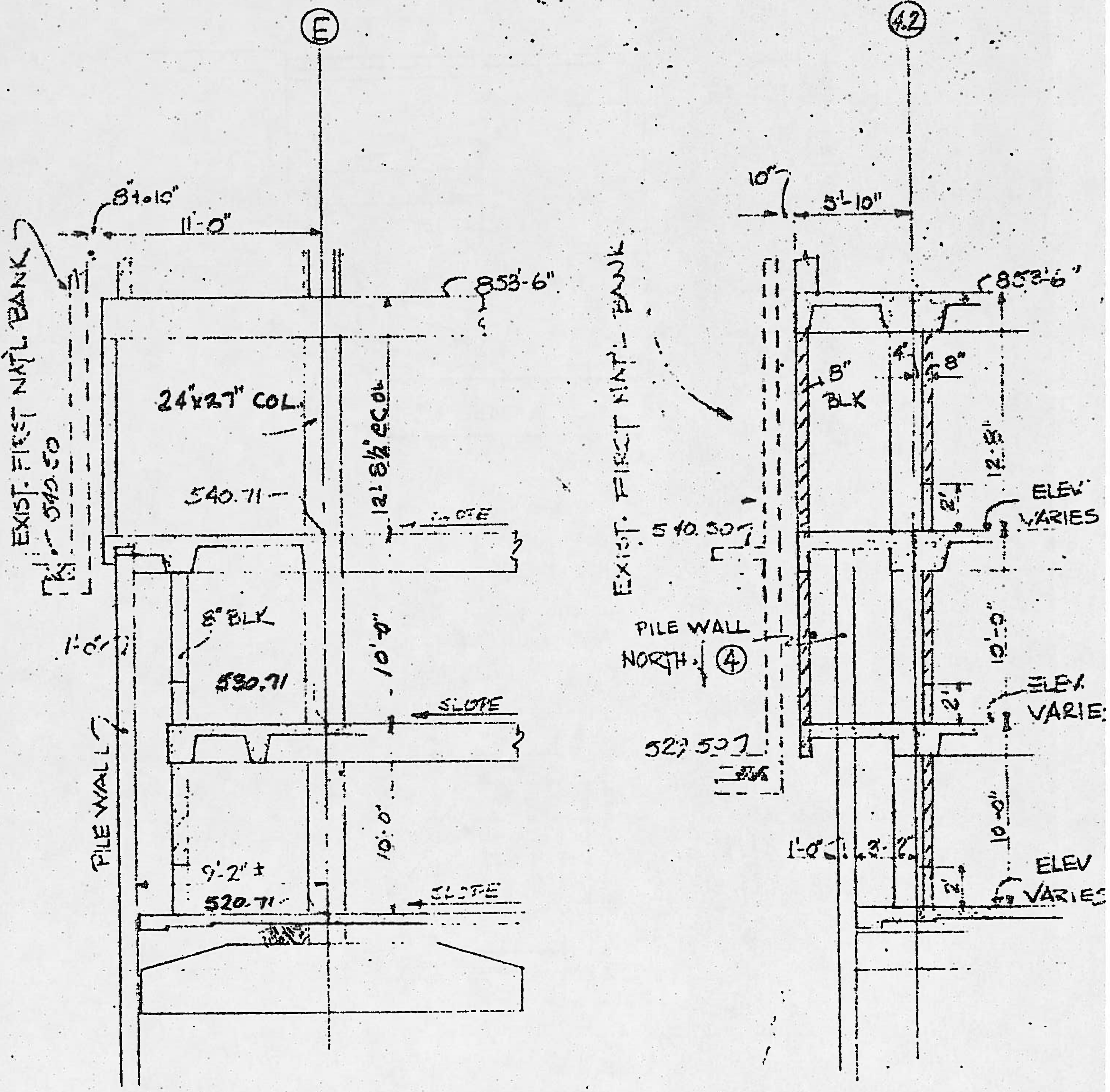
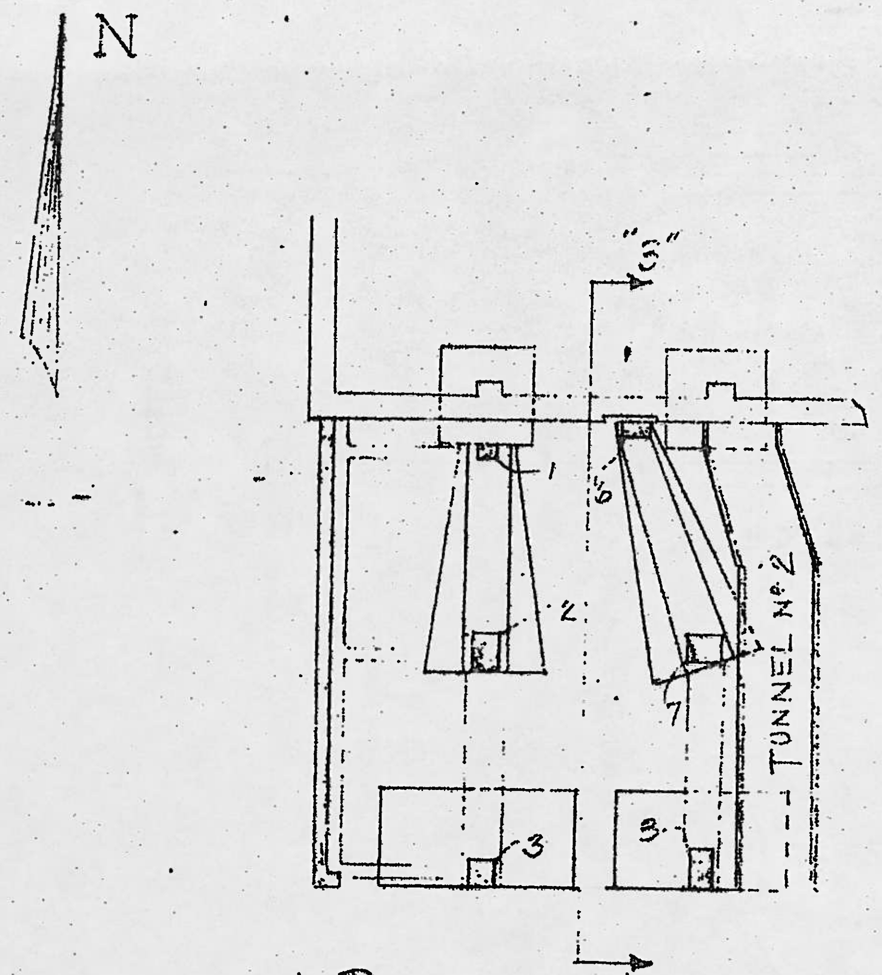
H
ELEVATOR
VAULT
EXISTING
FIRST NATIONAL BANK
BSMT. 540.50
SUB-BSMT 529.50
"2"
WALNUT ST.
WALK ELEV. 552.0±

SECTION "1"

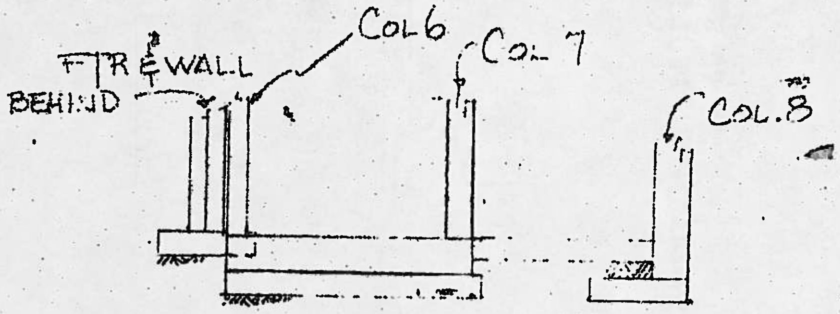
SECTION "2"



TYPICAL SECTIONS
DWG EX-2



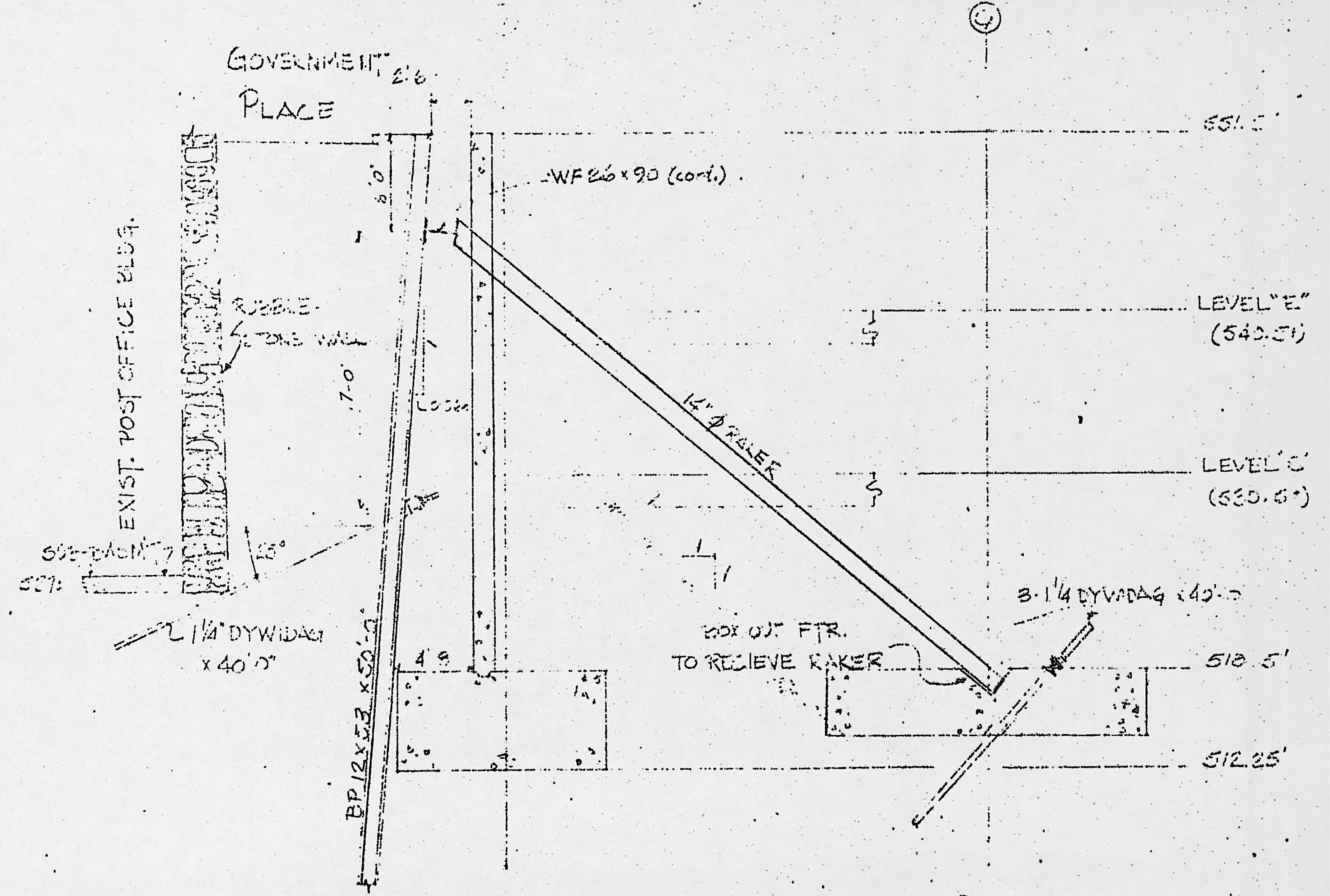
PORTION in N.W. CORNER
of EXISTING FIRST NAT'L BANK
(from TIETIG & LEE DRAWINGS,
SEPT. 1921.)



SECTION "3"

ACTUAL EXISTING FOUNDATION
at FIRST NAT'L BANK BLDG.

DWG 4



TYPICAL SECTION @ GOVERNMENT PLACE

Scale: ⅛" = 1'-0"

DWG 5

Section B

The Analysis

As indicated in the earlier discussion, the lack of available information on the adjacent 11 story bank building structure, led us to believe that a simple underpinning of an exterior bearing wall would be sufficient as a solution to the problem. Since this had been a party wall, with the adjacent building, simple symetrical footings were anticipated. It was later determined however, that the foundation of the adjacent Bank was very complicated with the exterior columns tied back on trapezoidal footings to the first interior column line and again strapped back to the second interior column line, so that actual determination of pressures under the building was almost impossible. Additionally, the foundation drawings obtained were incomplete and without key dimensions or reinforcing steel shown. The system of exterior containment suggested by the Goettle Construction Co., utilizing the Bauer Anchor tie back system seemed quite appropriate with the complicated foundation confronting us.

We explored the Bauer Anchor system in detail in our office and concurred with them that this seemed to be an appropriate solution. There were still many problems to be dealt with in the use of the system, particularly in this critical location. First, there was the question of possible short or long term creep in the anchors, which might permit settlement of the building. Second, the extremely high loading condition resulting from the surcharge pressure of an 11 story concrete building, required a considerably higher density of anchors than had been previously used. There was also some question relative to the capacity of the individual anchors, but perhaps the major question, was the integrity of the tie back system for such a critical task, that of supporting an 11 story building with a 40' vertical cut immediately adjacent, via horizontal containment.

Since we had the opportunity to utilize the system first, on the balance of the perimeter of the lot, we took this opportunity to make some tests of the anchors themselves, to verify their capacity. It was contended by the Bauer Anchor licensee that the capacity of the anchor was not based upon the pressure grouted length of the bar, but that the capacity was based only on the yield point of the bar. In order to check, three anchors were tested. Although the grout length on the typical production anchor was 15 ft., the three test installations were made with grout lengths of 10 feet on one anchor and with 5 feet on the two other anchors. Hydraulic jack loads were applied to each individual anchor, in approximately 10 kip increments. The load and strain readings were recorded and plotted. The strain was uniform up to the yield strength of the bar, with no indication of any slippage in the anchor itself. The strain in the bar recorded was only that which would be anticipated from the elongation of the bar, due to the applied load and length of the rod from the load point to the anchorage. Thus, the indication was that with the 15 feet grouted length we had a factor of safety of at least 3 on the yield strength of the bar relative to actually pulling loose or pulling out the anchorage itself, in this site material.

At this point we were satisfied that the system could be adequately designed to provide the necessary support for the adjacent building. However, the consultants for the adjacent bank felt otherwise and would not approve this solution. They sited a number of installations in which tie backs had exhibited substantial deflection. These installations were not necessarily using the Bauer tie back anchor system. Secondly, they insisted that the vertical load from the existing building should not be supported with, essentially, a horizontal load, transferred through the sheeting from the tie back anchors. They felt that the vertical load from the bank should be transferred by means of concrete piers or underpinning down to the elevation of the excavation for the Western Southern Life Insurance building. Admittedly we could not site a previous use of this method under similar circumstances.

The latter solution, in our opinion could be disastrous since with the combined footings, an attempt to support only the exterior, or one end of the footings could result in significantly unequal pressures, and in fact could cause shear failures in the combined footings. An alternate, of course, would be to underpin the footings all the way back to the first and second interior column lines, thus providing the same support under the entire footing. Such an installation however, would be _very_ extensive and costly.

To make it more challenging, both the City and General

Contractor had serious doubts as to the validity of this unique method. The owner and the Architect had been closely involved in each step of the problem and solution, and might well be considered as partners on our decisions.

In order to support or refute the proposed method of support for the adjacent bank we called in, with the concurrence of our client, the Western Southern Life Insurance Co., Dr. Paul Anderson, formerly professor of the soils engineering at the University of Minnesota, and presently consultant with the Johnston Saulman Company of Minneapolis. We asked him to review the problem in detail and to present us his recommendations.

Dr. Anderson generally confirmed our thinking on the design with certain welcomed refinements, but he stressed three potential problem areas. The first, was that of possible consolidation of the sand and gravel under the bank building due to the driving of H Piles immediately adjacent and the possibility of this contributing to settlement. Secondly, he was concerned about the possibility of settlement occuring under the bank building by the lateral movement of sand particles beneath the retaining structure and out into the unloaded excavated area below the proposed new construction. Third, he did not feel we could assume that creep would not occur in the tie backs on a long term basis. He was not concerned about creep during the short period of construction, but over a period of ten to twenty years. He did not feel there was sufficient data to assume that creep would not occur, thus relaxing the sheeting and permitting settlement to occur under the adjacent building.

After receiving Dr. Anderson's report, we again reviewed in detail the many potential alternate systems, including, auger cast piers instead of driven piles, drilled piers with metal shields, drilled metal piers, slurry wall, drilled lamella wall, and others. These were all reviewed relative to three areas of possible distress to the adjacent bank building. First, differential settlement due to consolidation of the strata from vibrations due to the installation of the support construction. Second, differential settlement due to a lack of containment of the granular mass below the bank, and third, differential settlement due to the changes in vertical pressures as excavation proceeded and the structural load was applied to the Western Southern property.

Although the non-vibrating type of pier installation, such as the auger cast piers would obviously cause less disturbance than would the driving of H piles, any of the methods indicated which utilized a grout or wet concrete mix in the construction gave us some concern. Since we would be dealing with lateral pressures of approximately 3000#/sq. ft., we were concerned that this pressure would actually cause considerable loss of ground to occur in the area due to the compressible wet concrete or grout. At the same time we have had a fair amount of experience in the downtown area with driving piles and have normally found the disturbance to adjacent structures to be negligible. Since the material through which we were driving was quite compact, as high as 73 blows per foot on boring samples, additional consolidation due to the driving has to be almost negligible. In a similar case a year earlier, we had to drive 14" pipe piles immediately adjacent to footings for a very heavy building. In this case the columns had strain gauges mounted on either side of the adjacent columns, and they were vertically monitored. We found in driving these piles that only a plus or minus 3 percent of differential stress occured on opposite column faces and no vertical movement was recorded. Thus we had to reject the possibility of distress occuring due to the driving of H Piles immediately adjacent to the bank building.

Secondly, again relying largely on Dr. Ralph Peck in the International Conference of Soil Mechanics and Foundation Engineering in Mexico City in 1969, we also had to reject the possibility of lateral flow of the sands from the area under the Bank Building below the cut off wall and into the area of excavation for Western Southern.

> "Cohesionless Sands
>
> Few records are available of the settlement of the ground surface adjacent to cuts in cohesionless sands. Projects of this type appear to fall into two categories. On one hand, if the sand is above water table or if the groundwater has been lowered and brought under complete control, adjacent settlement of dense sand appears generally to be inconsequential. The absence of settlement records at least suggests the absence of serious settlements."

We did concur that there was insufficient data available to say unquestionably that there could be no detrimental creep in the grouted Bauer tie back system. Logic indicated that there would be none, because the pressure applied to the sand and gravel should result in an instantaneous deflection of the sand and gravel and there was no reason, if non-vibrating type loading was present, that there should be any future creep. As a result of this particular point however, on consultation with the Western Southern Life Insurance Co., it was concluded that we should redesign the lower frame of the adjacent building such that if long term creep did occur in the tie backs, load from the sheeting system would be transferred to the frame of the new structure. The design thus provided, would permit .07 of an inch deflection in the H Piles from floor to floor of the new structure and a deflection of the adjacent frame of one tenth of an inch for a total maximum deflection of the wall at a single point of .17 inches. This occurring over an extended period of time, and only if the total load is transferred from the tie backs to our frame. Therefore, if creep did occur, we would anticipate only a part of this movement. We could envision no significant problem resulting in the adjacent bank due to this amount of displacement.

Again as Mr. Peck points out in the reference previously noted, if the total sheeting and support system could be installed prior to excavating for the new building, any movement of the soil would approach zero.

> "In principle, the movements could be prevented if the entire supporting structure including the sheeting, the wales, the struts or ties, and even the base slab for the completed structure could be constructed in their final positions before the removal of the enclosed soil. Subsequently, upon excavation of the enclosed earth, the settlements of the surrounding ground surface would correspond only to those associated with the deflections of the bracing system and floor slab. These movements would be extremely small compared to those that occur during excavation before the structural systems are complete."

Although ideal, this is an impossible task. Obviously, some excavation was required in order to install the tie back wall. However, the excavation of the interior lot site was kept some substantially higher than the elevation at the wall under construction.

Only sufficient sand and gravel was removed to permit concreting and tensioning immediately adjacent to the wall.

An item earlier mentioned which no one could clearly define was the effect of density of the tie back anchors and a solution to the maximum density question. Known installation in several cases of the Bauer anchor systems had utilized anchor densities up to 1 anchor per 40 square feet. In order to accomplish the tie back requirement for this installation, with the very high surcharge pressures, we required approximately one tie back for every 32 square feet. No one was actually sure that a theoretical determination was appropriate to obtain a limit for maximum density, although some theories were advanced. It was generally concluded however, that based upon intuition, opinion, or just engineering judgment, that 32 square feet per anchor should not be too great a density. Therefore, this was an assumption that was made, without real varification, for this installation. As will be discussed later, due to the contentions that had been raised and since this was really a first in this use of the Bauer Anchor system in this country, we did institute a number of very precise controls on the installation and the monitoring of both the tie back loads and the vertical and horizontal movements of the wall and the building. This will be discussed further at a later point in the paper. Again, our client, recognizing that monitoring could be most beneficial to the Engineering Profession for potential future use of the system, allocated a sum of money to us to provide this monitoring and collect data.

It should be made clear that these detailed considerations were being made not only to support academically a proposed system, but that there were significant cost and time differentials involved. Just to change from the permanent tie back system, to a temporary tie back system, involving strengthening the Block H structure, cost approximately $40,000, additional. It was estimated that not to use the tie back, but to provide temporary rakers within the excavation to support an H Pile wall and concrete lagging, would have cost an additional $150,000. Conventional concrete underpinning of the perimeter of the bank with rakers would be as high as $180,000 additional. Freezing the soil mass, while building conventional Block H structure, would have cost approximately $200,000, additional.

Freezing and then placing concrete underpinning of the perimeter and utilizing rakers would have increased the cost by $300,000. To jack the piles instead of driving them using concrete lagging and tie backs, would have been an additional $350,000. Lastly, to go into the interior of the adjacent bank, to underpin totally some of the combined footings inside, would certainly be in excess of a half million dollars, over the original system.

Faced with these magnitudes, in terms of additional dollars, plus significant time delays, certainly we could afford to spend a considerable amount of time and effort in order to ascertain that the proposed system could be successfully utilized to adequately support the adjacent Bank.

There was one change that took place prior to proceeding with the construction effort, and this was a shift in responsibility. The original support of the adjacent building, as was the support around the entire perimeter of the Block H excavation, was the responsibility of the general contractor. His proposed methods of course were to be reviewed and approved by the structural engineer. Because of the disputes which had arisen relative to the adjacent Bank, at the owner's request, we as structural engineers prepared the drawings and specifications for this portion of the perimeter support, and provided these drawings and specifications with our engineering seal to the contractor for his execution. We then supervised the construction throughout this procedure to assure that the construction complied with all requirements. Thus, there was a shift in responsibility for this support from the Contractor to the Engineer.

SECTION C

GEOLOGY

Geologically, the site is situated on a granular terrace within a deep valley of Teays age. During the Pliocene Epoch this valley contained the heavy flows of a northward flowing river which was later blocked by ice during the Pleistocene Era, creating extensive marginal lakes with deposition of fine-grained sediments over the bedrock formation.

The backed-up water subsequently cut its way through a ridge in western Cincinnati creating the present Ohio River Valley. The surface of the fine-grained sediments was eroded by melt water from the glacier and was in turn overlain by near-surface sands and gravels representing outwash deposited by the melting Wisconsin ice sheet.

All buildings in the downtown area of Cincinnati are founded in these Wisconsin outwash deposits well above the normal pool of the present Ohio River.

SUBSURFACE PROFILE AND SOIL PARAMETERS

Table C1 shows a typical soil column at the site. The upper 10 to 12-ft. within the test borings was miscellaneous fill generally consisting of brickbats and construction debris backfilling old basements.

The first natural soil is dense to very dense outwash sand and gravel to about elevation 516. The surface is in some cases medium dense. The gravel content decreases with depth from about 50% at the surface to 33% near the base. The maximum sizes in the split spoon sampler were 1-1/2" in diameter, however, particle sizes up to 3" were recovered by the augers. The sand content ranges from about 43% near the surface to 58% at the base. The silt content ranges between 7 and 9%. The uniformity coefficient for this zone is on the order of 25. The Unified Soil Classification ranges from GW-GM to SW-SM. The peak angle of internal friction from triaxial tests exceeds 40°. The dry unit weight of the material is on the order of 130-lbs. per cu. ft. Past experience indicates that there

TABLE C1

TYPICAL SOIL COLUMN

Elevation	Description	Standard Penetration Resistance, N	Angle of Internal Friction, ø	Dry Unit Weight, lbs./cu.ft.
547-539	Misc. Fill (man-placed materials)			
539-516	Fine to coarse sand & gravel (Wisconsin outwash)	11-130 typically more than 50	40°	130
516-496	Fine to med. sand (Wisconsin outwash)	20-58 typically more than 25	35°	110
496-482	Silty fine sand some sandy silt (Winconsin outwash)	17-47 typically more than 25	32°	115
482-441	Gray silt & clayey silt with clay seams (Lakebed deposits)	19-49		
441-432	Gray sandy silt (Lake deposits)	18-43		
432-415	Gray silt & clayey silt with clay seams (lake deposits)	18-35		
415-399	Gray silty fine sand (old glacial outwash)	43-48		
399	Gray limestone & shale layers (bedrock)			

is some slight cementation of this zone as evidenced by vertical cuts standing without shoring. This cannot, however, be considered in the design but does give an inherent immeasurable strength to the upper sand and gravel profile.

The very dense sand and gravel is underlain by compact outwash fine to medium sand to about elevation 496. Mechanical analysis indicate from 0 to 10% of fine gravel, 84 to 91% sand and 5 to 9% silt. The uniformity coefficient is on the order of 4. The Unified Soil Classification is SP-SM. The peak angle of internal friction from triaxial tests was found to be on the order of 39° with an ultimate of 35°. The dry unit weight is on the order of 110 lbs. per cu. ft.

Between elevation 496 and 482 compact silty fine sand with some sandy silt was encountered. The sand content is usually in the range of 68 to 76% with the silt content from 23 to 25%. From 0 to 7% fine gravel was found within the samples. The uniformity coefficient is on the order of 5. The Unified Soil Classification is SM. The peak angle of internal friction is on the order of 35° with a dry unit weight of 115 lbs. per cu. ft.

These upper soils represent Wisconsin outwash.

Below elevation 482 fine textured soils, representing sediments within glacial lakes, were encountered. Between elevation 482 and 415 the types of material were compact or medium stiff to stiff gray sandy silt, clayey silt and silt with clay seams.

This is underlain by 16-ft. of very dense silty fine sand which represents outwash from an earlier glacier. The silty fine sand is underlain by gray shale and limestone of Ordovician age, Cincinnati series.

The ground water table at the site was encountered at an elevation of about 490. Normal pool in the Ohio River which is some 8 to 10 blocks to the south is at 455. Little fluctuation of the ground water table has ever been shown for the downtown Cincinnati area.

LATERAL PRESSURES FOR DESIGN

The pressures determined for design of the anchorage wall, walers and tie-backs must be very conservative to insure safe support for the adjacent 11-story bank building. The anchorage wall must be designed rigidly and the supporting members including walers and tie-backs designed to insure limited movements due to surcharge from the existing building and the natural soil to be supported.

BASIS OF LATERAL PRESSURES FROM BUILDING SURCHARGE

As reported by Tschebotarioff (Reference 1), an experimental investigation of the effect of a concentrated surcharge load was performed by Spangler at the Iowa Engineering Experiment Station. A 6-ft. high model reinforced concrete wall was used for the purpose. It was backfilled with pit run gravel, 13% of which was from 1/2 to 1-1/2" in size and another 13% passing the 100 mesh sieve. The remainder was a sand of intermediate

(1) Tschebotarioff, Gregory P., Soil Mechanics, Foundations and Earth Structures, pp. 288-291

size. Concentrated loads were applied to the backfill surface with the wheels of a loaded truck which was backed to the desired position. No wall displacements were measured during this investigation, however, it was assumed that the wall was very rigid. The lateral unit pressures were plotted versus depth below the load. The pressures were also plotted for the Boussinesq equation for lateral pressures from a point load, P:

$$p_x = \frac{3P}{2\pi} x^2 z (x^2 + z^2)^{-5/2} \qquad \text{(Eq. C1)}$$

Where,

p_x = horizontal pressure due to point load P,

x = horizontal distance from point load to wall and

z = vertical distance from ground surface to point where pressure is computed.

The value of Poisson ratio, μ, was assumed to be 0.5 which is equivalent to the assumption that the soil is entirely incompressible. The distribution of lateral pressure by the Boussinesq equation was found by Spangler to be two to three times smaller than the actual measured values. According to Tschebotarioff, Spangler correctly attributed this circumstance to the restraint of lateral soil displacements which was imposed by the rigid wall. Under such conditions lateral pressures double the ones given by the Boussinesq equation are to be expected. This point was illustrated in Reference (1) wherein the application of a concentrated or linear load P to the surface of a large elastic body will cause an originally plane vertical surface 00 to deform and take the shape indicated by the curve ab in Figure C1.

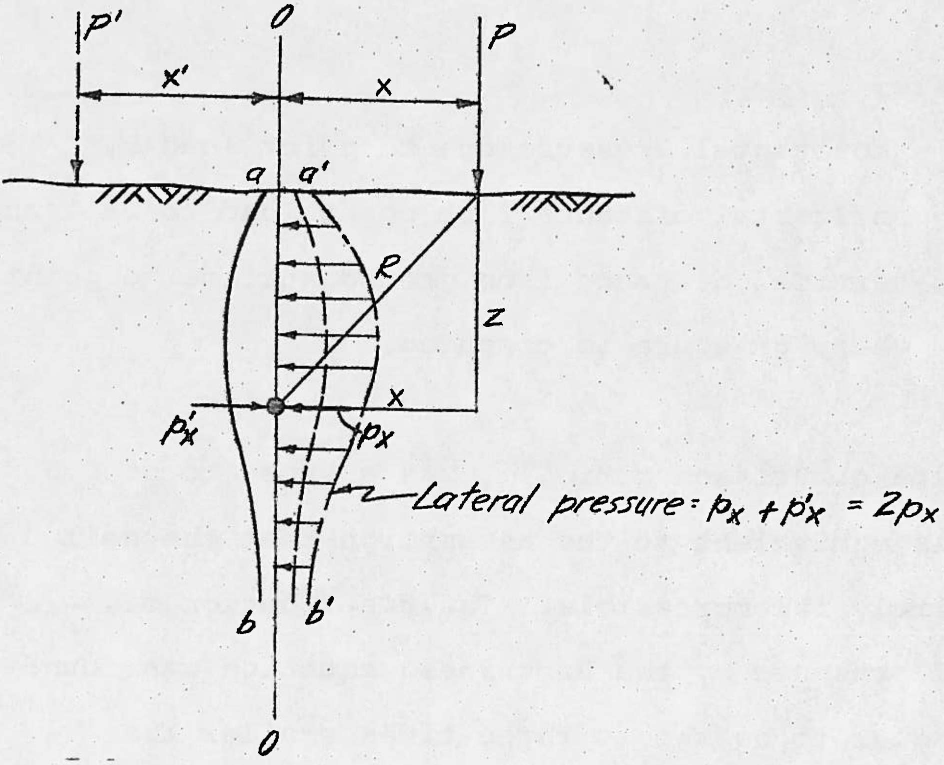


Fig. C1. Lateral pressure distribution from concentrated load P in the case of a rigid unyielding wall. (Mirror-image effect)

This deformation will be prevented if the face of a rigid and unyielding wall is located along the plane 00. Exactly the same condition of zero lateral displacement along the vertical plane 00 will be produced by the action of a so-called reflected load P'; i.e., of an imaginary load of exactly the same magnitude and distance from 00 as the load P. (Mirror effect). The imaginary load P' will induce lateral pressures p_x' against the vertical plane 00 which will be equal to the lateral pressure p_x exerted by the actual load P on that plane in the unrestrained mass of soil. Therefore the actual lateral pressure against an unyielding rigid wall would be equal to

$$p_x'' = p_x + p_x' = 2\ p_x \qquad \text{(Eq. C2)}$$

where p_x is defined by equation (C1). Tschebotarioff indicates that until more is known concerning the effects of such a displacement and the values of the Poisson ratio of soils, it appears advisable to use the results of Spangler's tests and the definitely conservative relationship given by equation (C2).

COMPUTATION OF LATERAL PRESSURES FROM BUILDING SURCHARGE

For determining potential lateral pressures on the rigid anchorage wall we first assumed the existing building loads to act as line loads parallel to the anchorage wall through each column line. These line loads were based on the dead load plus 50% of live load to each column divided by the column spacing. Line loads appeared reasonable for the first interior column located 17-ft. back of the wall and the second interior column located 35-ft. back of the wall because of the minimal distance between edges of footings. This assumption was questionable on the exterior column line immediately behind the anchorage wall as the distance between edges of footings was much greater; although, load from columns would be transferred by shear into foundation walls thus reducing soil pressures below column footings and increasing pressures below wall footings. It was thus assumed that a linear load was approached on the exterior column line. It should be noted that the exterior column and the first interior column are supported on trapezoidal footings, the footing width increasing from the exterior column to the interior.

The Boussinesq equation for uniform line load, P, of infinite length was used to compute p_x the horizontal pressure at various depths due to the assumed loading conditions:

$$p_x = \frac{2P}{\pi} x^2 z \quad (x^2 + z^2)^{-2} \qquad \text{(Eq. C3)}$$

where the variables are defined in equation C1.

A computer was used to determine lateral pressures at 1-ft. intervals of depth below the bottom of existing footing. These values were doubled as per Spangler's findings. The results are shown in Column 1 of Table C2.

Another run was made assuming that a line load parallel to the anchorage wall acts along the centroid of trapezoidal footings instead of along the lines of the exterior and first interior columns. Line loads were used for the other interior footings as before. The results of this run are shown in Column 2 of Table C2. The second run gave higher values to a depth of 16-ft. below bottom of footing and the first run higher results below that depth.

TABLE C2
LATERAL PRESSURES FOR NORTH WALL
(in lbs./sq. ft.)

	LATERAL PRESSURE FROM BUILDING SURCHARGE				LATERAL EARTH PRESSURE		MAX. LATERAL PRESSURE - BUILDING + EARTH
	(1)	(2)	(3)	(4)	(5)	(6)	(7)
	Line Loads at Each Col.	Line Loads at Centroid/ Trapezoid and Interior Columns	$0.5p_v$ 60^o Prism	Maximum Values from (1),(2) & (3)	Redistributed $0.65\gamma H\ K_a$	At Rest Triangular γHK_o	(4) + Max. Value from (5) & (6)
541					615	0	615
540					615	65	615
539					615	130	615
538					615	195	615
537					615	260	615
536					615	325	615
535	(Bot. of footing)		2350	2350	615	390	2965
534	434	484	2280	2280	615	455	2895
533	828	930	1900	1900	615	520	2515
532	1142	1305	1630	1630	615	585	2245
531	1471	1592	1425	1592	615	650	2242
530	1545	1783	1260	1783	615	715	2498
529	1625	1886		1886	615	780	2666
528	1713	1982		1982	615	845	2827
527	1758	2018		2018	615	910	2928
526	1772	2012		2012	615	970	2987
525	1766	1975		1975	615	1040	2015
524	1746	1921		1921	615	1105	3026
523	1719	1858		1858	615	1170	3028
522	1687	1790		1790	615	1235	3025
521	1653	1724		1724	615	1300	3024
520	1618	1660		1660	615	1365	3025
519	1583	1600		1600	615	1430	3030
518	1548	1544		1548	615	1495	3043
517	1514	1492		1514	615	1560	3074
516	1480	1444		1480	615	1625	3105
515	1447	1401		1447	615	1690	3137
514	1415	1360		1415	615	1755	3170
513	1384	1322		1384	615	1820	3204

Within the upper portion of the new anchorage wall at footing level and immediately below footing level studies were made to determine possible higher lateral pressures. The soil pressure at the bottom of existing column footing was taken as 5700 lbs. per sq. ft. The pressure is probably less due to load transfer through foundation walls. Because of the close proximity of the end of the footing to the anchorage wall (1'-3") we assumed that the 5700 lbs. per sq. ft. would be distributed within a prism having 60° sides to the horizontal and that the horizontal pressure is 0.5 times the vertical pressure; i.e., the at-rest condition. This resulted in higher pressures as shown in Column 3 of Table C2. The highest value at any given depth below bottom of footing from these results was taken as the pressure from building surcharge. This is shown in Column 4 of Table C2.

COMPUTATION OF LATERAL EARTH PRESSURE

The lateral earth pressure was determined based on the re-distributed pressure diagram as shown in Terzaghi and Peck. This assumes uniform straight line pressure distribution from the surface to the bottom of excavation equal to 0.65 γ HK. K was taken as 0.25, the active coefficient. The results are shown in Column 5 of Table C2.

The lateral earth pressure was also determined using triangular distribution with K = 0.5, the at-rest coefficient. The results are shown in Column 6 of Table C2. According to Jaky, the at-rest coefficient, K_o = 1-sin ϕ. With $\phi = 40^o$, K_o = 0.43. The maximum values from the building surchage (Column 4 of Table C2) were then added to the maximum earth pressures from either Column 5 or 6. These are shown in Column 7.

The Structural Engineer decided to use an average design pressure of 3000-lbs. per sq. ft. for design of the anchorage wall, walers, and tie-backs. This is considered to be a conservative value which would provide for an overall rigid design.

It is felt that the natural cementation of the sand and gravel provides additional safety factor of unknown magnitude because the lateral earth pressures would be something less than indicated by the analyses because of the increased shear strength of the sand and gravel.

STABILITY ANALYSIS OF ANCHORED EXCAVATION SUPPORT SYSTEMS

INTRODUCTION

Injection anchors are increasingly used in the construction industry for support of excavation walls during construction operations. This was recently investigated by Ranke & Ostermeyer (Reference 1) with the objective of developing procedures for designing earth anchor systems in an industry standard for harbor construction in Germany. A calculation procedure has been proposed for multiple anchored walls which is based on procedures in Kranz/s original work (Reference 2) because of the availability of data from a completed case history of a triple-anchored foundation excavation wall.

BASIS FOR CALCULATION

The important factors for determining the stability of a tied-back excavation support system are:

1. the strength of the individual structural elements

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(1) Armin Ranke/Helmut Ostermayer, Beitrag Zur Stabilitatsuntersuchung Mehrfach Verankerter Baugrubenumschlie Bungen (Translated to English by B. E. Hartmann - unpublished)

(2) Kranz, E.: Uber Die Verankerung Von Spundwanden, 2. Auflage Mitteilungen aus dem Gebiete des Wasserbaues und der Baugrundforschung, Heft 11. Berlin: Wilh. Ernst & Sohn 1953.

of the excavation walls including soldier beams, lagging, walers and connections.

2. The bearing capacity of the individual anchors or anchor groups.

3. The stability of the total system, shoring wall-soil-anchors.

Restraint capacity or load carrying ability of the excavation support wall is obtainable by static calculation using an assumed lateral earth pressure envelope. The anchorage capacity of the individual anchors can be estimated by analysis and determined by pull-out tests.

In the calculation of the stability of the overall system there are two basic failure conditions which must be investigated. These are designated by the concepts of "internal" and "external" stability. The determination of the external stability is based on the investigation of the failure of the wall toe and

- -

a monolithic circular failure along an assumed slip circle as shown in Figure C2. In contrast for the determination of the internal stability, we assume that the shear strength of the soil in the system (wall-anchors-soil) is exceeded so that shear surfaces are formed extending outward from the anchor end in the direction of the wall to be anchored (lower failure plane) whereby the wall can turn over as shown in Figure C3. For determination of the anchor length in the normal case, the slip circle failure is usually not the most critical. The stability in the lower failure plane is usually the critical case.

STABILITY IN THE LOWER FAILURE PLANE

The stability with respect to the lower failure plane is customarily analyzed by Kranz's method. We assume a line between the anchor mid-point and the rotation point of the excavation wall as the lower failure plane. An equivalent anchorage wall is assumed in the middle of the anchorage zone as shown in Figure C4a. The force vector polygon can be obtained from

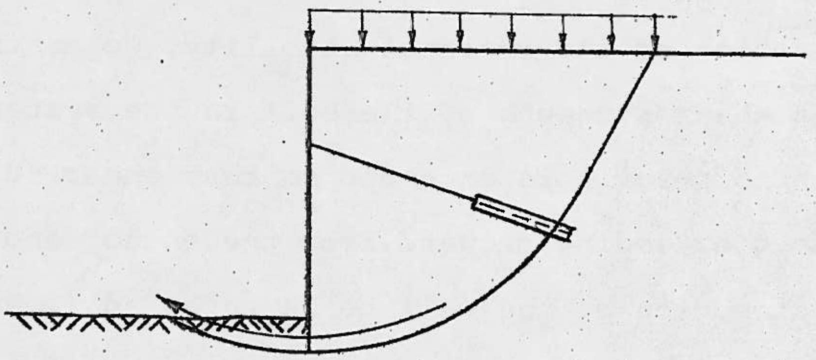


Fig. C2. Failure along slip circle - external stability.

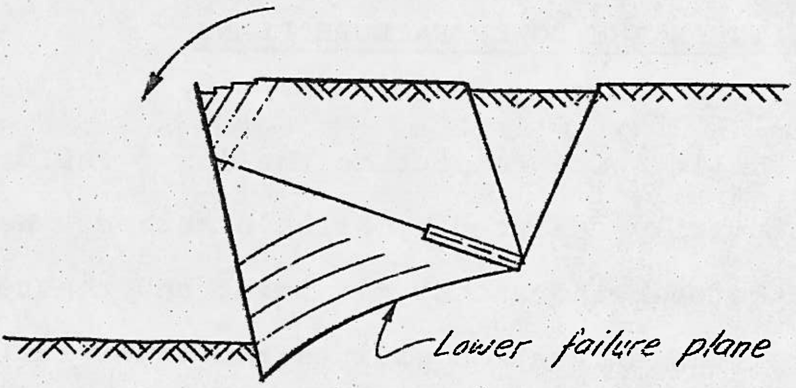


Fig. C3. Failure along lower failure plane - internal stability.

the equilibrium diagram and from this the maximum value of the possible anchor forces for the cross hatched soil zone A_h can be read off as shown in Figures C4b and C5. The safety factor in accordance with Kranz's method is defined as the ratio of the maximum possible anchor forces to existing anchor forces:

$$\eta = \frac{\text{Possible } A_h}{\text{Existing } A_h} \qquad \text{(Eq. C4)}$$

The existing anchor forces are determined by static methods. The maximum possible A_h is determined by the following equations:

$$E_{rh} = \left[G - (E_{ah} - E_{1h}) \tan \delta \right] \tan (\phi - \nu) \qquad \text{(Eq. C5)}$$

$$\text{Poss. } A_h = f_A (E_{ah} - E_{1h} + E_{rh}) \qquad \text{(Eq. C6)}$$

$$f_A = \frac{1}{1 + \tan \alpha \cdot \tan (\phi - \nu)} \qquad \text{(Eq. C7)}$$

where,

G = total weight of soil above the lower failure plane, E_{ah} = active earth pressure (without water

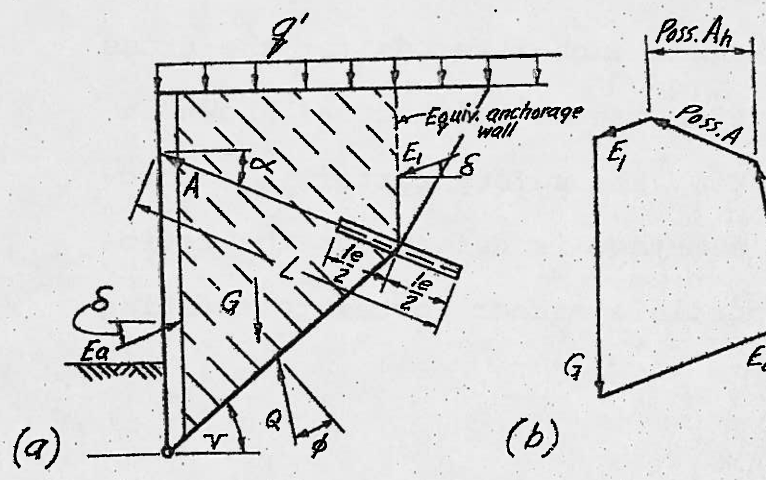


Fig. C4. Determination of stability for the lowest failure plane - injection anchor system.

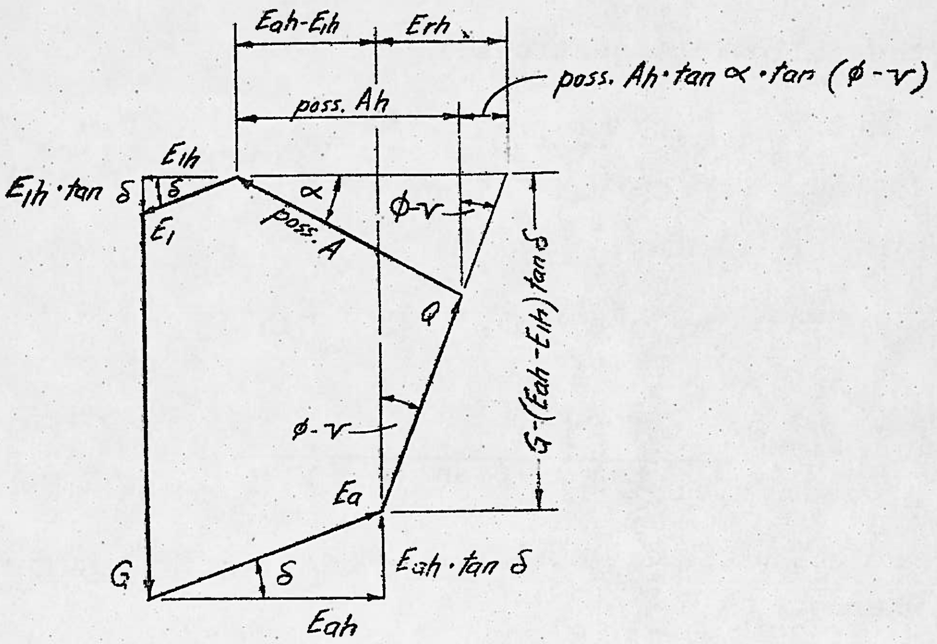


Fig. C5. Geometric relationships on a force-vector polygon.

pressure) on the excavation wall from the top down to the rotation point of the wall,

E_{th} = the active earth pressure of the equivalent anchorage wall,

ϕ = the internal angle of friction of the soil,

δ = wall friction angle and

ν = inclination angle of the lower failure plane.

CRITIQUE OF THIS PROCEDURE

It is possible to raise certain basic theoretical objections to the method of Kranz. For the assumption of a non-linear sliding surface a lower safety factor results than by the assumption of a straight line for the lower failure plane. During the failure or rupture of the system, an earth pressure is introduced which is larger than the active pressure as a result of the stressing between the anchorage and the supporting wall.

Tests have shown that the lower failure plane extends outward from the anchor end to the rotation point of the wall. This means that the assumption of the lower failure plane from the center of the anchorage zone to the rotation point of the wall produces conservative values.

MULTIPLE ANCHOR SYSTEMS

The basic considerations for the calculation of single level anchored walls generally apply to the case of multiple level anchored walls as shown in Figures C6 thru C9. In accordance with the arrangement of anchors between the middle point of the anchorage zone and the rotation point of the wall, various lower failure planes may be drawn in order to determine the required safety factor of $\eta = 1.5$.

ANCHORING FOR AT-REST EARTH PRESSURE

It is shown in the paper by Ranke and Ostermayer that walls designed for at-rest earth pressure require an investigation of the failure conditions to be completed where the earth pressure and anchor forces must be determined from the active limiting condition. It is only possible to determine the stability in the lower failure plane through this initial assumption. It is shown that the safety factor for the at-rest condition is related to that determined using active pressure assumptions as shown in the following equation.

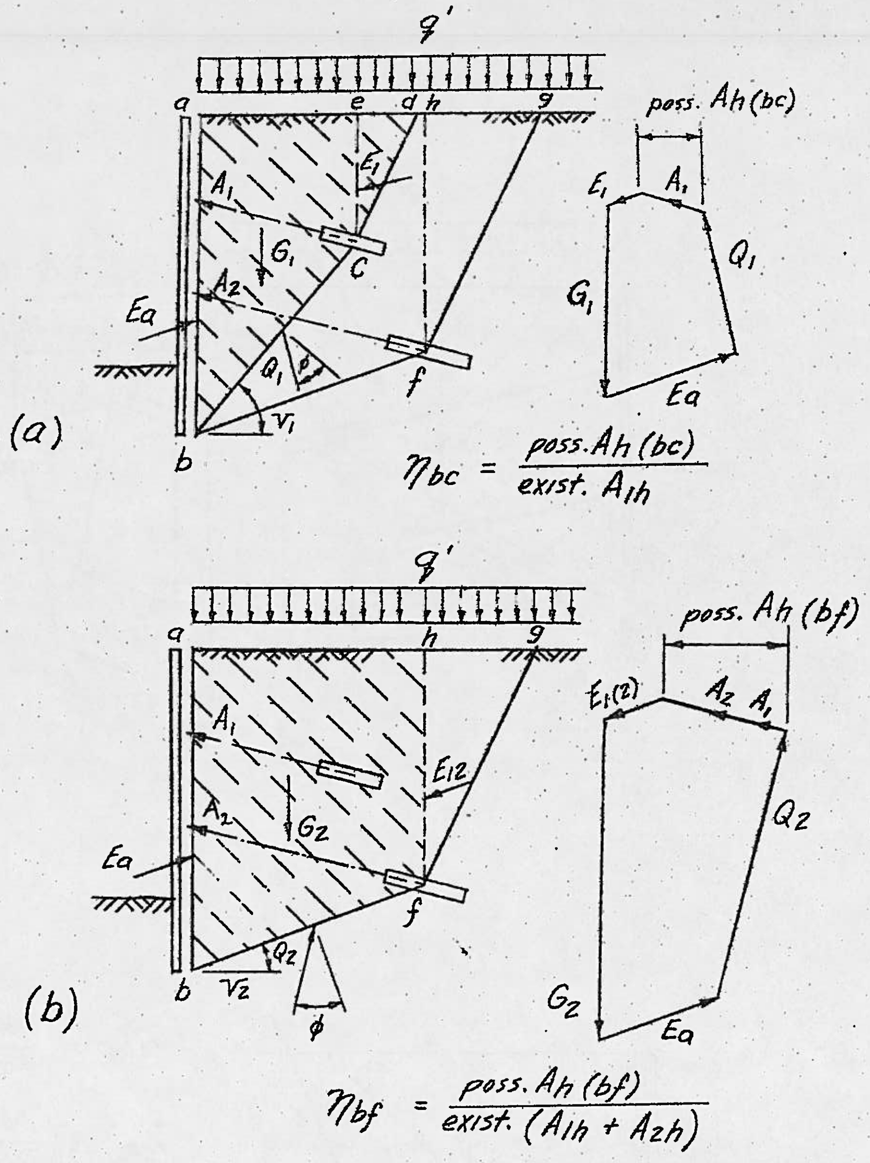


Fig. C6. Force applications and safety factor values for 2 anchor levels - Case 1.

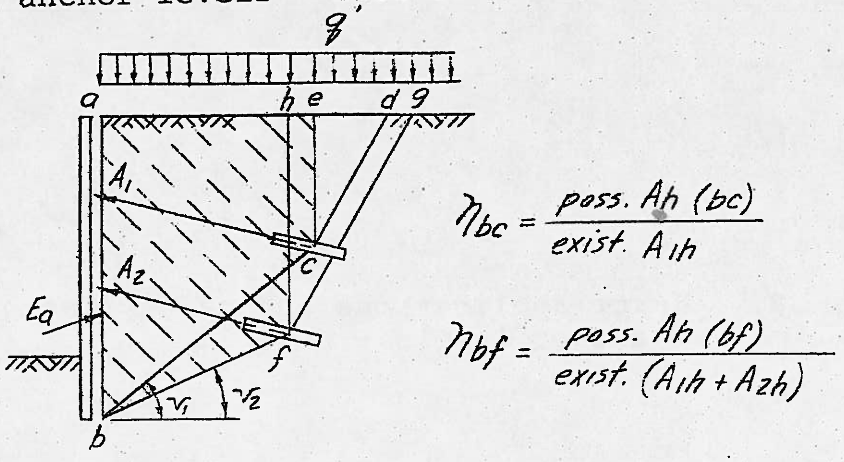
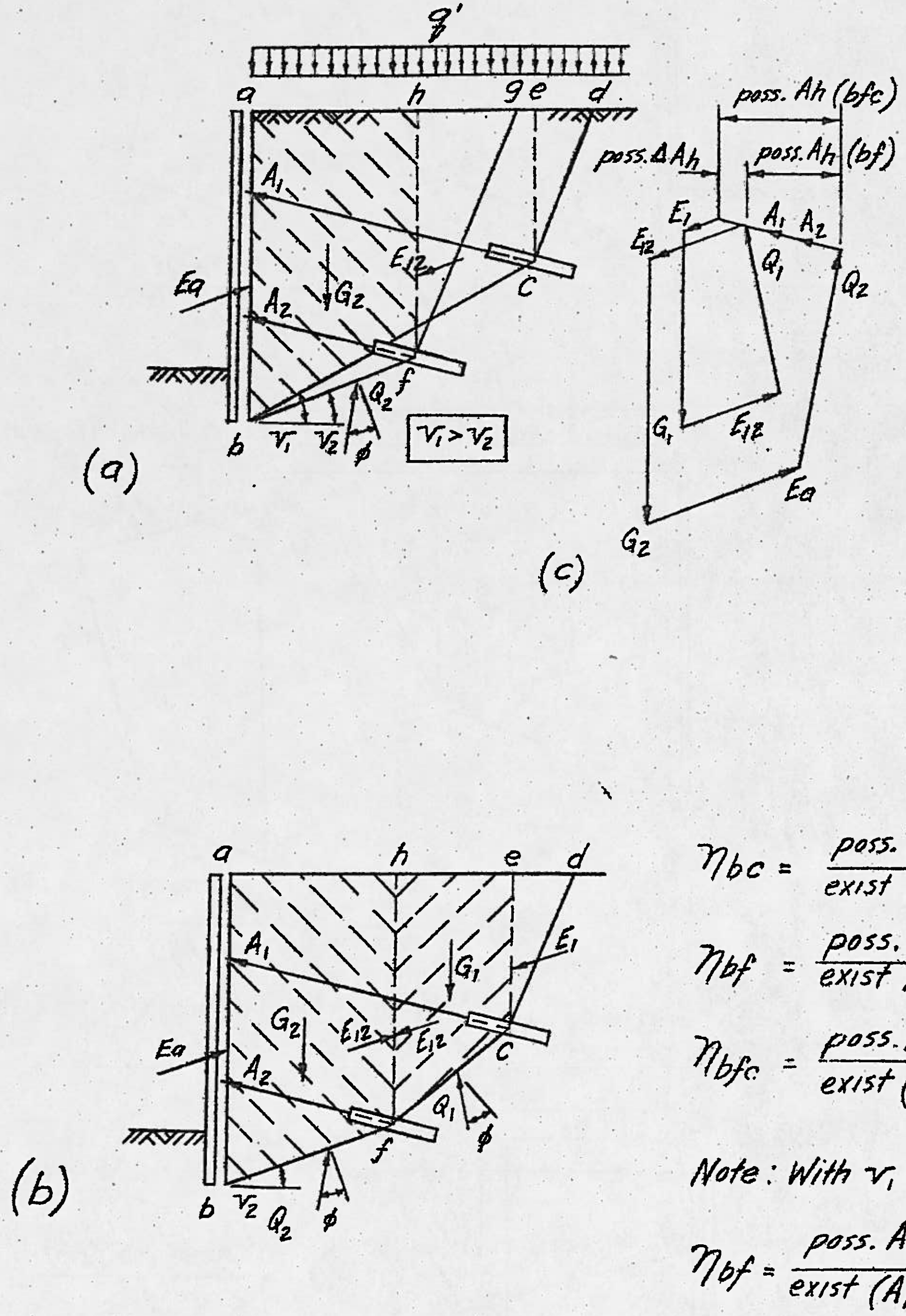


Fig. C7. Force applications and safety factor values for 2 anchor levels - Case 2.



$$\eta_{bc} = \frac{\text{poss. } A_h(bc)}{\text{exist } A_{1h}}$$

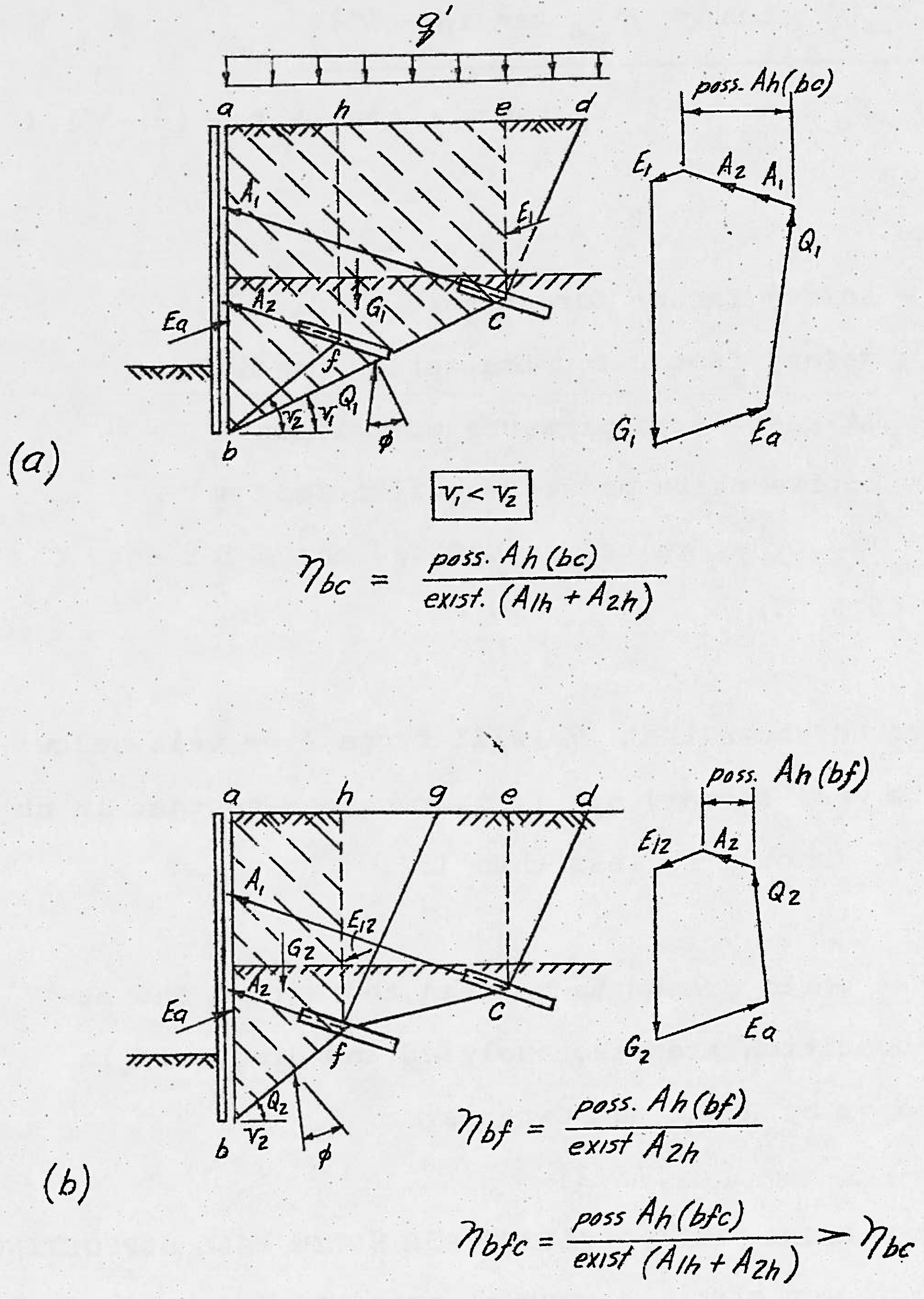
$$\eta_{bf} = \frac{\text{poss. } A_h(bf)}{\text{exist } A_{2h}}$$

$$\eta_{bfc} = \frac{\text{poss. } A_h(bfc)}{\text{exist } (A_{1h} + A_{2h})}$$

Note: With $\nu_1 > \nu_2$ use

$$\eta_{bf} = \frac{\text{poss. } A_h(bf)}{\text{exist } (A_{1h} + A_{2h})}$$

Fig. C8. Force applications and safety factor values for 2 anchor levels - Case 3.



$$\boxed{\nu_1 < \nu_2}$$

$$\eta_{bc} = \frac{\text{poss. } A_h(bc)}{\text{exist. } (A_{1h} + A_{2h})}$$

$$\eta_{bf} = \frac{\text{poss. } A_h(bf)}{\text{exist } A_{2h}}$$

$$\eta_{bfc} = \frac{\text{poss } A_h(bfc)}{\text{exist } (A_{1h} + A_{2h})} > \eta_{bc}$$

Fig. C9. Force applications and safety factor values for 2 anchor levels - Case 4.

$$\eta_o = \frac{\eta_a \left[K_a \tan \nu + \frac{K_a}{K_o} \tan (\phi - \nu) \right]}{K_a \tan \nu + (1 - K_a \tan \nu \cdot \tan \delta) \tan (\phi - \nu)} \quad \text{(Eq. C8)}$$

where,

η_o = safety factor for at-rest condition,

η_a = safety factor for the active condition

K_o = at-rest earth pressure coefficient

K_a = active earth pressure coefficient and

the other values as defined above for eqs. C5, C6 & C7.

It can be shown that η_o will range from well below 1.5 to well above 1.5. It is recommended that in no case η_o should be less than 1.2.

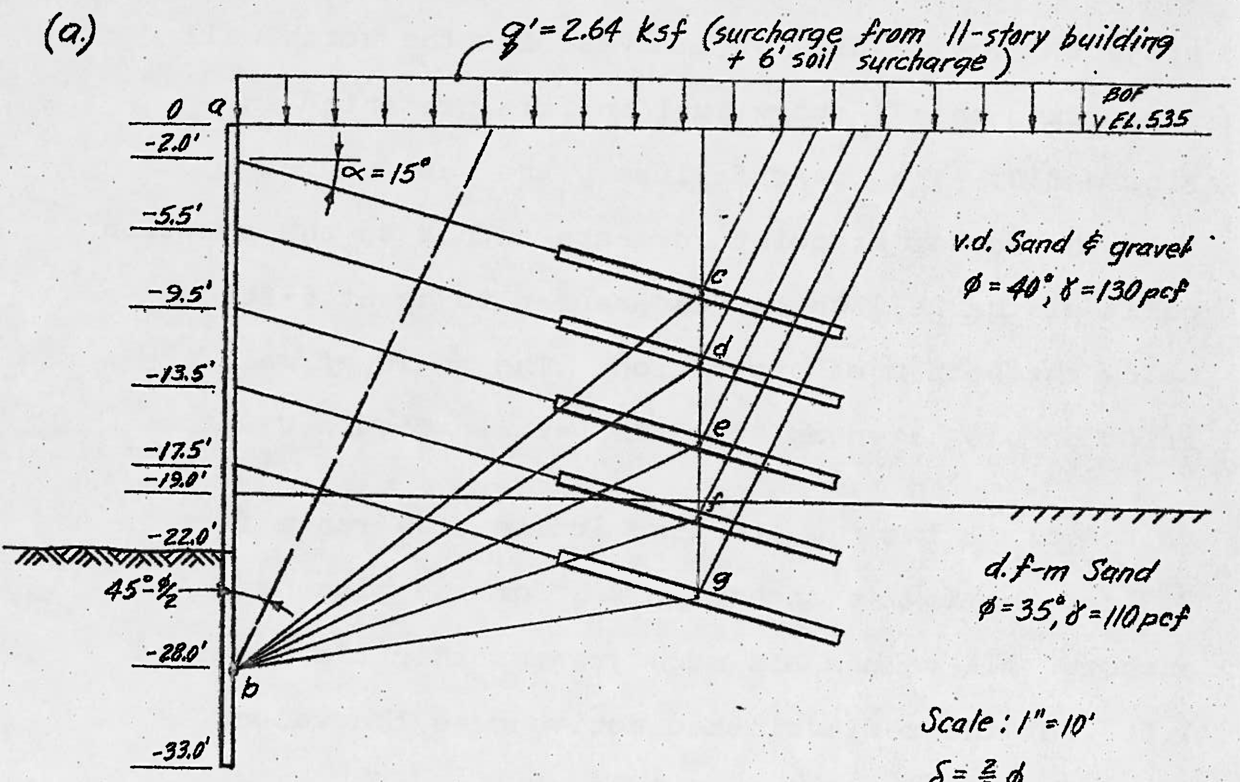
Lateral earth pressures between the active and at-rest condition are also analyzed and discussed in the paper by Ranke and Ostermayer.

LOWER FAILURE PLANE ANALYSIS FOR NORTH WALL SUPPORTING 11-STORY BUILDING - "INTERNAL STABILITY"

The method of Ranke and Ostermayer was used in the design of the wall-soil-anchor systems at the subject project. The results of analysis for the north wall supporting the 11-story building are presented in Figure C10. The lower failure plane was assumed from the anchor midpoint for each anchor to the rotation point of the wall which was assumed to be at 6-ft. below the bottom of excavation. The angle of wall friction was taken as $\frac{2}{3}\phi$. As seen in Figure C10 the values of η_a for the limiting active case range from 3.3 for the lowest anchor to 8.5 for the highest anchor. All values are much greater than the required 1.5. For the redistributed active case the values of η_a range from 3.3 to 6.0.

Also as seen in Figure C10 the values of η_o, the stability for the at-rest condition, range from 1.8 for the lowest anchor to 9.8 for the highest anchor. The computed values are greater than the required 1.20 and 1.75 for the lowest and highest anchors respectively. For a redistributed at-rest pressure distribution the values of η_o range from 1.8 to 6.9.

For the design of the north anchorage wall, a total lateral pressure was assumed which is 1.65 times greater



(b.)

Lower failure plane	Active limiting η_a	Active redistributed η_a	At-rest η_o	At-rest redistributed η_o	Uniform pressure = 3000 psf η
bc	8.5	6.0	9.8	6.9	–
bd	5.3	4.0	4.8	3.6	–
be	4.0	3.3	2.8	2.3	–
bf	3.8	3.3	2.4	2.1	–
bg	3.3	3.3	1.8 (2.3)	1.8 (2.3)	1.06 (1.24)

Value in ( ) indicates η for lower failure plane from end of anchor

Fig. C10. (a) Anchor arrangement. (b) Safety factor values for lower failure plane.

than the at-rest earth pressure. If such pressures are in fact possible then analysis by Ranke and Ostermayer's method using equation (C8) with $K_a = 0.22$ and $K = 0.83$ (instead of $K_o = 0.5$) yields a safety factor of 1.06 for the lowest anchor. It must be considered that a very minute wall movement will reduce the lateral earth pressure appreciably. According to Terzaghi a movement at the top of anchorage wall as little as 1/8" (0.0005 H), and assuming this movement is linear over the wall height, will reduce the total lateral pressure from the at-rest to the active.

It should further be considered in accordance with Spangler's experimental investigation that for loads at a distance of 0.83 H from an unyielding wall the pressures by the Bouissinesq equation were equal to the experimental data. For the north anchorage wall of the 11-story building a considerable part of the computed stress from the building surcharge was produced by footings located more than 0.83 H from the wall. These stresses were still assumed to be two times Bouissinesq values.

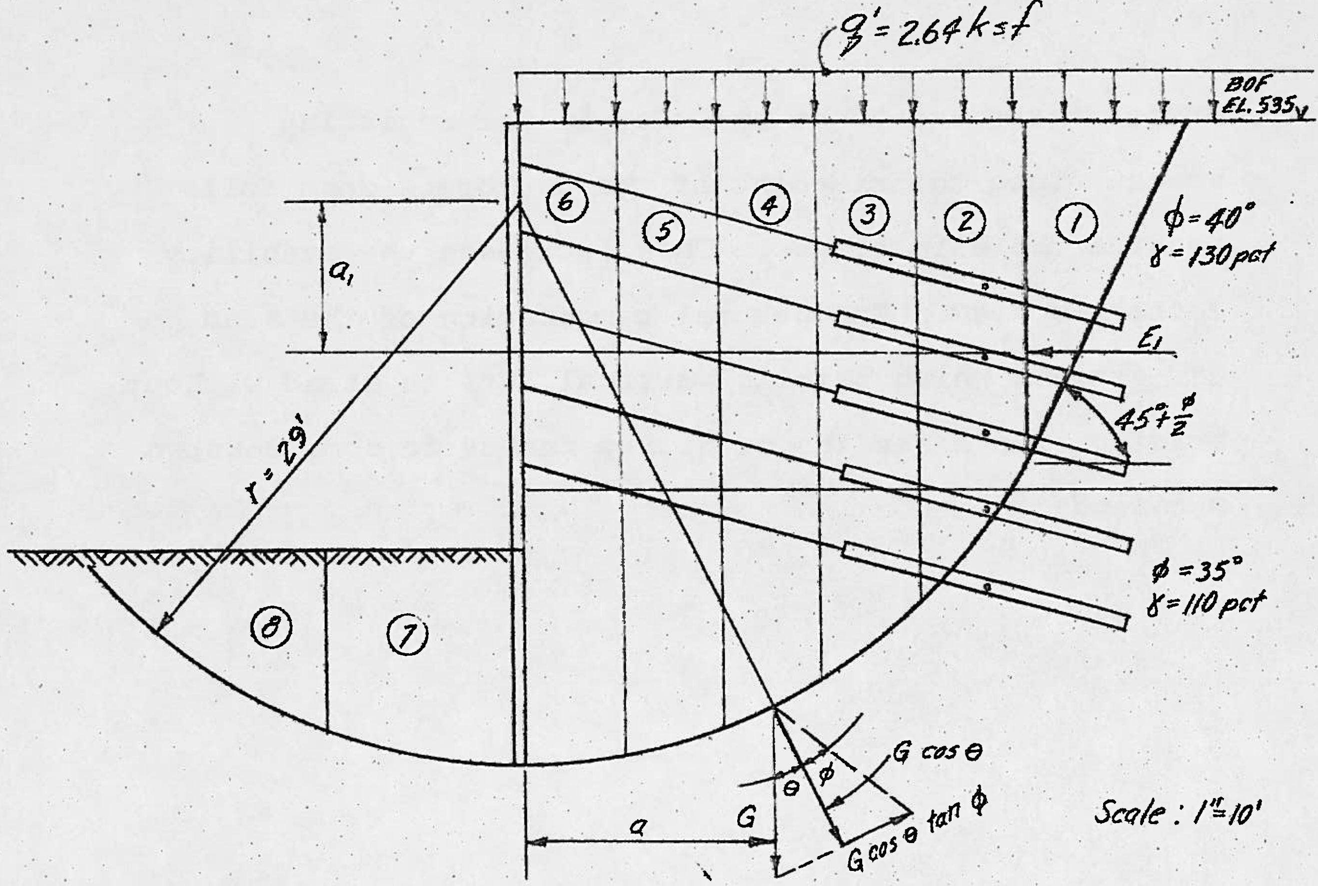
It should be noted that the force for the lowest anchor was determined assuming no passive resistance from soil in front of the wall.

For comparative purposes we have computed a safety factor for the lowest anchor for the at-rest pressure condition and assuming the lower failure plane extends from the end of anchor to the rotation point of wall. This results in $\eta_o = 2.3$. As expected this value is greater than 1.8 determined for the lower failure plane starting from the anchor midpoint. A similar analysis yields $\eta = 1.24$ for the 3000-psf uniform pressure distribution.

SLIP CIRCLE ANALYSIS FOR NORTH WALL SUPPORTING 11-STORY BUILDING - "EXTERNAL STABILITY"

The slip circle investigation gives a stability factor $\eta = 1.65$ which is greater than the required 1.4. The analysis for the critical circle is shown in Figure C11.

If lateral stresses were included on individual slices a larger stability factor would result. The lowest



Segment	h	b	G	a	$a \cdot G$	θ	$\cos\theta$	$G \cdot \cos\theta$
1	$E_1 \cdot a_1 = 5.2 \times 7.9 = 41.$				41.			
2	21.8	5.5	30.1	22.8	686.	52°	.615	18.5
3	27.0	5.0	30.7	17.5	538.	38	.787	24.1
4	30.0	5.0	32.7	12.5	409.	26	.899	29.6
5	31.9	5.0	33.9	7.5	254.	15	.965	32.7
6	33.0	5.0	34.6	2.5	86.	5	.995	34.4
7	10.5	10.0	11.5	−5.0	−58.	10	.984	11.3
8	6.0	13.0	8.6	−16.5	−142.	35	.820	7.0
				Σ	1814.			147.6

$$\eta = \frac{r \cdot \tan\phi \cdot \Sigma G \cos\theta}{E_1 a_1 + \Sigma G \cdot a} = \frac{29. \times 0.70 \times 147.6}{1814.} = 1.65 > 1.4$$

Fig. C11. Slip-circle analysis.

anchor force should be included in the resisting moment since the midpoint of the anchorage zone falls outside the slip circle. This increases the stability factor to 1.80. The natural cementation of the sand and gravel, which permits vertical cuts to stand without bracing, increases the stability factor to some unknown magnitude.

Section D

DESIGN OF TIED BACK RETAINING WALL

There are three locations at Western and Southern's Block H Development which have significantly different design conditions. They are the basic tied back retaining wall used at excavations adjacent to the street, the temporary wall around the Southern Ohio Bank, in the southeast corner and the permanent wall at the adjoining 11 story concrete building, in the southwest corner.

The tied back wall along the street retains cuts varying from 29 to 40 ft. (9.5 to 13 m.). It consists of H Piles driven at approximately 6 ft - 0 in. (2 m.) centers to 10 ft (3 m.) below the bottom of the excavation with wood lagging spanning between flanges. Back to back channels support each pair of H piles. The channels are then anchored at their midspan by the Bauer Anchor tieback. The wall was designed for a uniform lateral earth pressure as determined from Terzaghi and Peck's formula - $p = .65 K_a \gamma H$ where

p = uniform lateral pressure

K_a = Rankine Coefficient $\tan^2 (45^\circ - \phi/2)$

$\phi = 36^\circ$

H = height + 3 foot (1 m) surcharge to account for traffic loads

γ = weight of soil = 130pcf. (2040 kg/m^3)

FIGURE D1(a).

Another method of pressure determination was furnished by H. C. Nutting Co., in their report. This consists of a trapezoidal distribution rather than uniform. However, the total load on the wall differs only slightly with either method. FIGURE D1(b).

The anchors are threaded bars 1¼ in.(33 mm) in diameter with a yield point of 156^k at 121 ksi (70×10^3 kg at 85.1 kg/mm^2) and an ultimate strength of 198^k at 154 ksi (90×10^3 kg at 108.6 kg/mm^2). A safety factor of 1.25 (to yield point) was provided at those walls adjacent to the street. The passive restraint provided by the embedment of the piles below the bottom of the excavation was neglected, in computation of required anchors.

The anchors were made long enough so that the 15 ft. (5 m.) pressure grouted zone was beyond the active failure plane of the soil, although the contractor had arrived at the lengths by a different procedure.

The wall around the old Southern Ohio Bank consists of sheet piles restrained by walers and tie-backs. The design of this wall is similar to the walls at the street except that a surcharge of 7 ft. (2.4 m) was added to "H" in the lateral earth pressure formula to account for the load super-imposed by the four story timber, concrete, and masonry building. This wall is only temporary, since the building will be demolished, as soon as occupancy of the new facility is obtained. A safety factor of 1.5 was used in the anchor design on this wall. As an aside, the anchors for this area will be exposed during excavation for the replacement tower , and can be examined for shape and condition.

The third design condition is around the adjacent 11 story concrete building in the south west corner of the site. The excavation for Block "H" extends 22 ft. (7.2 m) below and is only 1 ft - 3 in. (3.8 m) out from the bottom of the adjoining buildings footings. The tied back wall used to retain the soil under the building consists of HP12X53 piles, cast in place reinforced concrete for lagging between piles, and walers with tie backs at midspan to support the piles. FIGURE D2 & D3.

The most difficult problem in the design of this wall was determining a conservative but realistic value for the lateral earth pressure to be restrained. It was assumed the pressure could be thought of as being the result of two actions;

1. The lateral load on a retaining wall due to the soil only.
2. The lateral load due to the surcharge of the adjacent building.

For the lateral load on a retaining wall due to the soil only, the at rest pressure of the soil had to be maintained. To obtain the active pressure the soil would have to deflect from the at rest condition which could damage the building. Thus, the at rest pressure at each elevation below the adjacent footings was calculated by $p_o = K_o\gamma H$ where

p_o = the at rest pressure (psf)
K_o = coefficient for at rest pressure = .5
γ = weight of soil = 130 pcf. (2040 kg/m^3)
H = height

The equation used at other locations $p = .65K_a\gamma H$ is not applicable because it assumes the wall will deflect enough to allow the lateral force to descend nearly to the active pressure condition.

The lateral pressure, p_s, due to the surcharge of the adjacent 11 story building was calculated and furnished by Mr. James Flaig of the H. C. Nutting Company. The pressures p_o and p_s were then added together for each elevation. The average total pressure at each elevation equaled 2,920 psf. (14200 kg/m^2).

The wall was then designed for 3000 psf. (14600 kg/m^2).
FIGURE D4.

The anchor and pile spacing was selected to provide a safety factor of 1.75 to the yield point of the bar. This was achieved with the pile spacing at 3 ft - 6 in. (1.15 m.) and typical waler spacing at 4 ft. - 0 in. (1.3 m.) vertically. Also, at the bottom row of anchors, to maintain this safety factor, the passive pressure against the embedded section of pile was included.

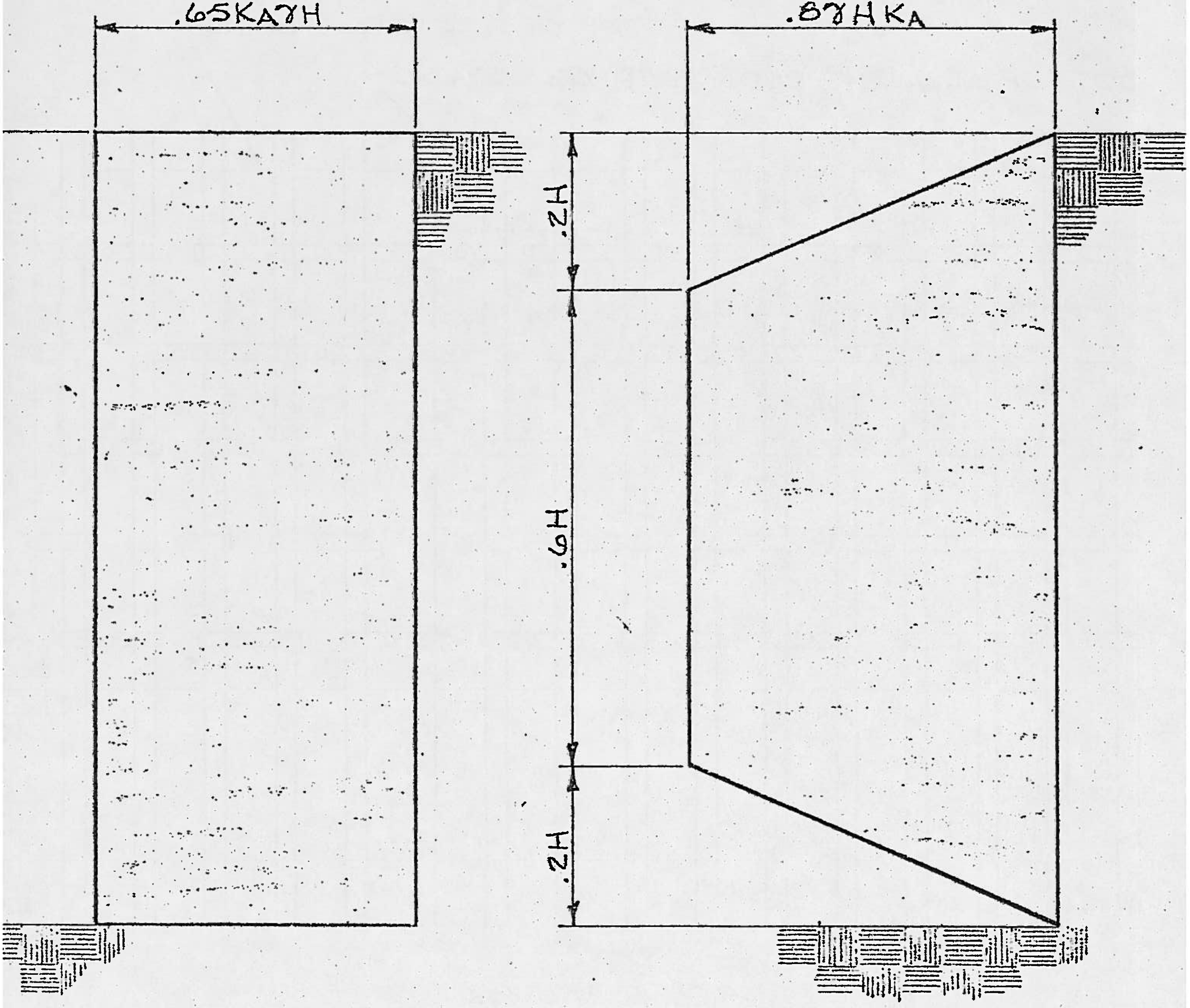
The top row of anchors is about 20 in. (0.5 m.) below the bottom of the footings. Much concern was given to these anchors, since there was the possibility that a slight settlement due to the concentrated loads from the above footings might tend to bend and fracture the tie back bar. Since these high strength bars are relatively brittle an abrupt strain might fracture the bar. To account for this, anchors were located at every second and third space between piles, instead of alternate spaces, an increase of 30% over the calculated number of anchors required. Continuous walers consisting of C12 x 20.7 channels 3 in. back to back were used.

The walers were designed so that if one anchor failed, the waler could distribute the load to adjacent anchors. Thus if there was a severe anchor overload or strain due to the irratic pressures under a footing, adjacent anchors could maintain the integrity of the wall. At the remaining three rows of walers C 8 x 13.5 channels were used as simple spans between alternate piles.

Around the corner of the adjacent building, anchors were not used between the last two piles. However, the walers are continuous around the corner, with a rigid connection. Thus each side of wall acts as a shear wall to brace the other side.

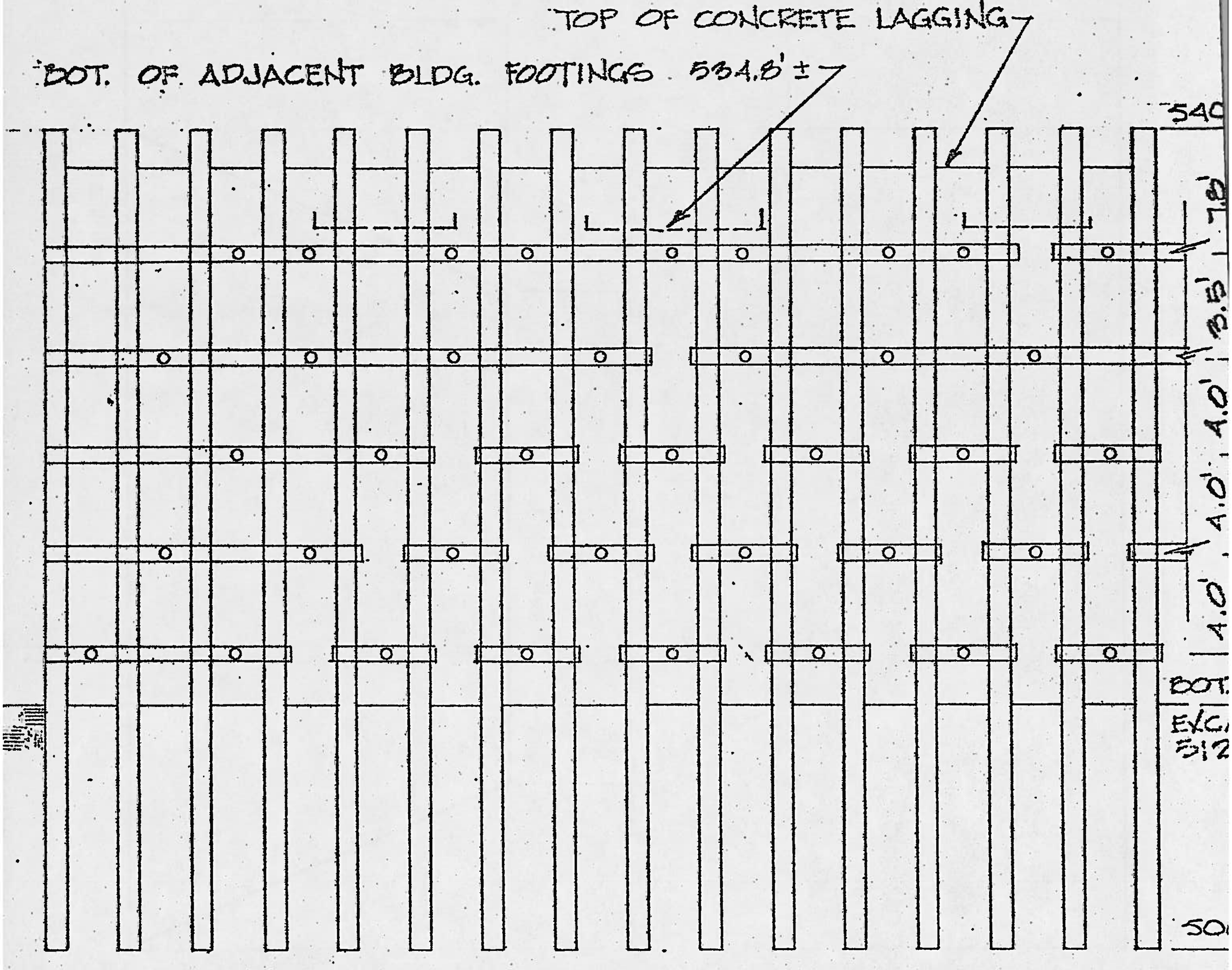
As an added safety precaution, the Block H column footings 60 feet (19.7 m.) north of the wall had a pocket formed in the side so that the 70 ft. (23.0 m.) long steel jumbo columns, for the building, could be placed as a strut from the footing to the adjacent building if any distress was noted. The footings are 15 ft (4.9 m.) square by 4 ft, (1.3 m.) thick. Tie back anchors were installed diagonally through the footings to counteract the horizontal force from the struts. The anchors in the footing were to be tensioned if and when the struts were needed. However, no problems were encountered, and the struts were not required. A similar installation was used at the Government Place retaining wall since the basement of the Post Office building on the other side of the street was below the elevation at which tie backs were required. Rakers were installed from column footings to the top of the wall. These footings required tie back anchors to resist the horizontal force from the rakers.

Since there was a question of whether creep would occur in the tied back soil, the Block "H" structure was designed to take the lateral earth loads if the anchors eventually relaxed. Since the new building was estimated to settle 3/4 in. (1.9 cm.) it had to be free from the tied back wall until most of this settlement occured, to keep from pulling the wall down also. A 1 in. to 2 in. (2.5 to 5. cm) gap was left between each floor and the wall. When all of the dead load was on the building this gap was filled in with grout. Now, if the tie backs exhibit long term creep the new building can resist the load from the wall.



(a) (b)

FIGURE D1



PILES @ 3'-6" c/c

FIGURE D2

CONSTRUCTION PROCEEDURES

Construction of most every major under grade facility leads to unique problems. This project was no different. The subgrade material in the core area is somewhat difficult for pile driving operations, and "drift" and "out of plumbness" occur. Fixed leads aid in installing the soldier piles to good lines without drift and therefore ease the problems of lagging and bracing.

Lagging, as you may know, does not compute. Three inch thick material, simple spanning six to eight feet will not support the back wall pressures used in designing the system. It doesn't compute - but it does work. Most experts subscribe to the theory that earth arching from H pile to H pile relieves the load going to the wood elements. In only one instance in Cincinnati have we had a lagging board break (not on this project) and that board had a large knot hole and shouldn't have been installed anyway.

Tie installations vary with the system used and the soil encountered. Anchors can be installed in augered holes, driven holes, rotary cut holes, and some installations have been made driving plate anchors. Cohesionless soils lend themselves readily to driven hole anchors. An air-trac drill with a percussion hammer is used to drive the drill, in the case of Bauer-Anchors a 76 mm hollow string.

The lead element is a conical point countersink to receive a Dywidag thread bar. When the point has advanced to its design location a bar is placed in the string and the point is driven off. An adaptor at the outer end of the string allows the introduction of grout, thru the string and around the bar to the annular space at the end. Grout is pumped into the system until a pressure of 150-350 psi is reached; the drill string is incrementally extracted, 10-14", and grout again pumped to the above pressures until the design length of anchor bulb has been installed. The string is then removed. Anchors can be stressed in three days. Cubes should be made periodically to determine grout strengths, but, again the true test of the system is the loading of the bar.

Other systems using cable or strand can be installed in a similiar manner, and some systems depend solely on a gravity grouted bulb for resistance to pullout. Augered hole ties have been installed as long as 140 feet. These are generally high capacity anchors depending on shear at the grout soil interface or on belled bottoms to resist pullout. The anchor assemblies sometimes have a plated bottom to transfer stress from the tension member to the grout.

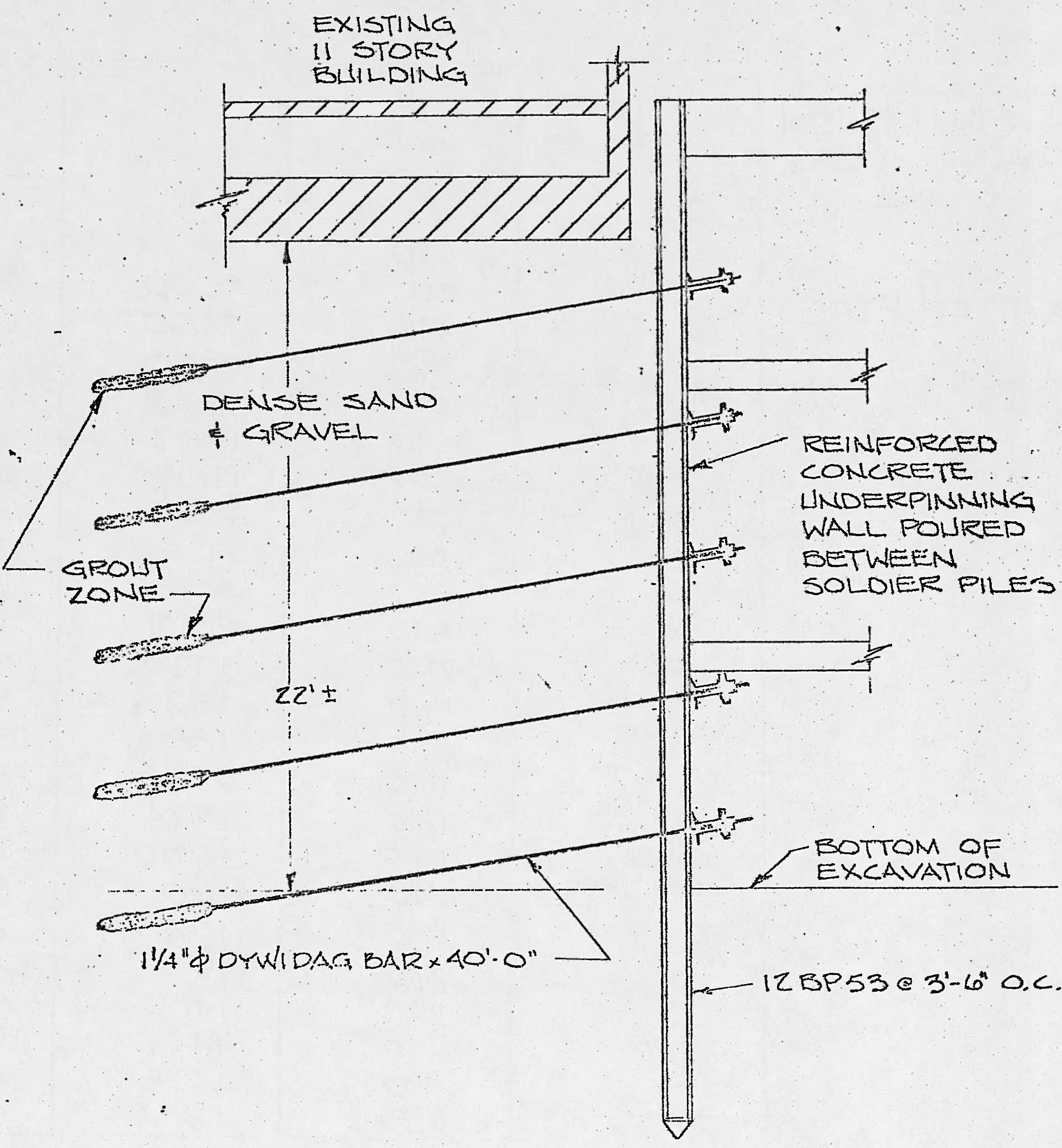

EXISTING
11 STORY
BUILDING
DENSE SAND
& GRAVEL
GROUT
ZONE
22' ±
REINFORCED
CONCRETE
UNDERPINNING
WALL POURED
BETWEEN
SOLDIER PILES
BOTTOM OF
EXCAVATION
1¼"ϕ DYWIDAG BAR × 40'-0"
12BP53 @ 3'-6" O.C.

FIGURE D3

ACKFILL BETWEEN
DJACENT BLDG.
& WALL
↓
OT. OF FTNG.

OT. OF EXCAVATION

ELEVATIONS	Po LATERAL PRESSU FROM SOIL ONLY	Pe LATERAL PRESURE FR BLDG. SURCHARGE	TO PRE
541	0		
540	60		
	130		
	180		
	250		
535	380	3000	338
	440	2400	284
	510	2000	251
	570	1720	229
	640	1592	223
530	710	1783	249
	770	1836	265
	830	1982	281
	900.	2018	291
	970	2012	298
525	1040	1975	301
	1100	1921	30
	1160	1858	30
	1230	1790	30
	1290	1724	30
520	1360	1660	30
	1420	1600	30
	1490	1548	30
	1550	1514	30
	1620	1480	31
[illegible]	1680	1417	31
	1750	1415	31
[illegible]	1820	1384	32

Σ = 66

AVG. = 29

FIGURE D4

Of all the facets of earth retaining systems, the installation of anchors has to be fraught with more unpredictability than any other.

We apparently were to blame for the complete evacuation of the LaNormandie Restaurant during the dinner hour, as well as upsetting the preparation operations at Gregory's Steak House. We were accused of grouting full a 75 year old sewer in Sixth Street. You should say to me that this is sloppy craftsmanship. The grout take should have indicated something was wrong, and to a certain extent I agree. But grout "takes" are occasionally far from uniform even in natural material. Two adjacent anchors in the sheet pile wall at Southern Ohio Bank took, respectively, 6½ sacks and 27 sacks. Each of these were driven at the same angle to the same tip elevation, six feet apart. We are going to excavate these anchors soon and hope to get some indication for the reasons behind this discrepancy.

One facet of high strength rods bears special mention. These bars are manufactured to very close chemical tolerances, cold worked by stretching and proof loaded, altogether an expensive process. These are <u>not</u> rebars. The point of interest, common to most all high strength steels, is that they all have high notch sensitivity. They should not be dropped or treated carelessly.

The saving grace is that defects reveal themselves when stressing. To my knowledge the only bar we broke on this project was one on which weld spatter had accidentally dropped. The bar snapped at the spatter mark and had to be replaced. You have had a thorough explanation of the treatment at the bank building. One other area deserves mention, the Government Place protection. Government Place is the service road for the Main Post Office and an absolutely essential piece of real estate. The present building was constructed (along the north line) inside the old foundation wall of the preexisting structure, and precluded driving anchors due to its close proximity. External bracing for the upper part of the protection system was a must. Our new building geometry limited the number of braces we could use and therefore the stress level of the raker was rather high. Permanent footing kick blocks did not, by analysis, give sufficient resistance to the braces to withstand these loads. A readily available solution was to tie the footings with Bauer Anchors to compensate for the inadequacy.

The lower anchors in this wall required a steeper pitch than had been anticipated. As usual we found this out the hard way. Five to six foot thick masonry construction a hundred years old does not lend itself to being drilled with the type equipment we were using.

An interesting side light on this wall concerns itself with the removal of the wale and raker system. We had been assured that the backfill to the waler had been thoroughly compacted, yet on burning the raker loose, the soldier piles jumped toward the hole some 4" to 5" causing a minor settlement of the street. With this kind of evidence there can be no doubt that a real load existed on the system, and this should serve to warn all concerned that backfilling operations inspection is of paramount importance if settlements are to be avoided.

Other variations using ties are available. Somewhat more expensive, but valuable where space limitations exist, is the tie installation thru the H beams, eliminating a waler assembly. With properly driven pile this type wall can serve as a back form for a concrete wall. As a precaution, the adjacent structure should be designed to accomodate full active and surcharge loading.

Section F

SUPERVISION AND MONITORING

In general, the initial form of supervision and monitoring on the tied back walls was during tensioning of the anchors. The anchors were stressed to 138^k (62,500 kg) then released to 115^k (52,000 kg). Five percent of the anchors were stressed to 160^k (72,500 kg), before releasing to 115^k (52,000 kg). The deflection in the bar was measured with a dial gauge and recorded at given intervals of load to verify that a proper load deflection relationship occured. The gauge was mounted in such a way that the change in distance between the waler and the anchor rod was recorded. In retrospect, it would have been better to have mounted the gauge to a tripod on the ground since the results we obtained were affected by deflection of the waler. The results, however, were still basically linear, which is the expected relation between the load and deflection.

For the wall around the adjacent building, a much more detailed program of monitoring was used. It basically consisted of two phases. First, was the effects on the building from the pile driving operation. This was monitored by strain gauges applied to some of the adjacent building's columns and a seizmograph located inside the building. The second phase consisted of load cells applied to 9 anchors and visual inspection for movements both during the construction of the wall, and on a continuing basis after construction was completed.

Before the pile driving began, reinforcing bars on four columns were exposed and strain gauges were applied to the bars. FIGURE F1. A limit of 1500 psi was set as the maximum change in stress which would be tolerated. Difficulty arose in reading the gauges when piles were driven along the east face of the adjacent building. The problem was corrected before driving piles on the north face. A pile was driven opposite each of the two columns on the north face. It was observed that the load in a column next to the pile being driven decreased while the load increased in the adjacent columns. Thus by driving successive piles at a different column or bay the change in stress was held well within the 1500 psi ($105 kg/cm^2$) limit. The average stress change in the seven bars monitored was 240 psi ($17 kg/cm^2$) compression.

A Sprengnether U.S.-4000 Blast Seismograph was located on the basement slab immediately adjacent to the exterior wall where piles were being driven. Readings were taken at 5 ft. intervals during driving, with a required safe vibration level of 2.0 inches/second. (4.0 inches/second is considered threshold, resulting in minor perceptible damage.) Values varied from 0.075 to 0.88, but most were in the range of 0.2 or 0.3. Therefore no problem was anticipated, due to the pile impact.

In the second phase of the monitoring, the load cells were installed. They had two purposes. First, to provide a check of the wall system to reveal any distress in the wall due to a significant change in load during construction and second, to maintain a long term record of the soil pressure which should show whether creep is a factor in this type of soil. Figures F2 and F3.

With somewhat limited resources and with delivery time required considerably less than available from commercial suppliers, we obtained the assistance of Professor Andrew Bodocsi and Gerry Roberto, to manufacture locally, strain gauge, and calibrate the 9 special load cells. Due to the lack of sophistication of the cell installation and severe exposure to construction operations, the results of readings were somewhat less than desirable. They have however, performed a valuable service as an indicator of any potential problem.

The #1 load cell began showing increasingly higher loads. This caused some concern as the loads continued to increase. However, since the remaining load cell results showed no real problem, and dimensionally there was no movement in the wall, or anchor at the #1 cell, and the cell was not seated squarely, it was assumed that the cell was malfunctioning. About eleven months after installation, the cell was reseated, but the cell continued to malfunction. When jacking the load off the cell, the force on the jack during reseating was 80 to 100 kips (36000 to 45400 kg) which verified our assumption that the anchor was performing as designed.

The remaining load cells performed adequately for the first few months; however, after a year, the results were scattered and not consistent with each other. At this time, cells #4, 5, and 6 were reset, to check on the reference condition.

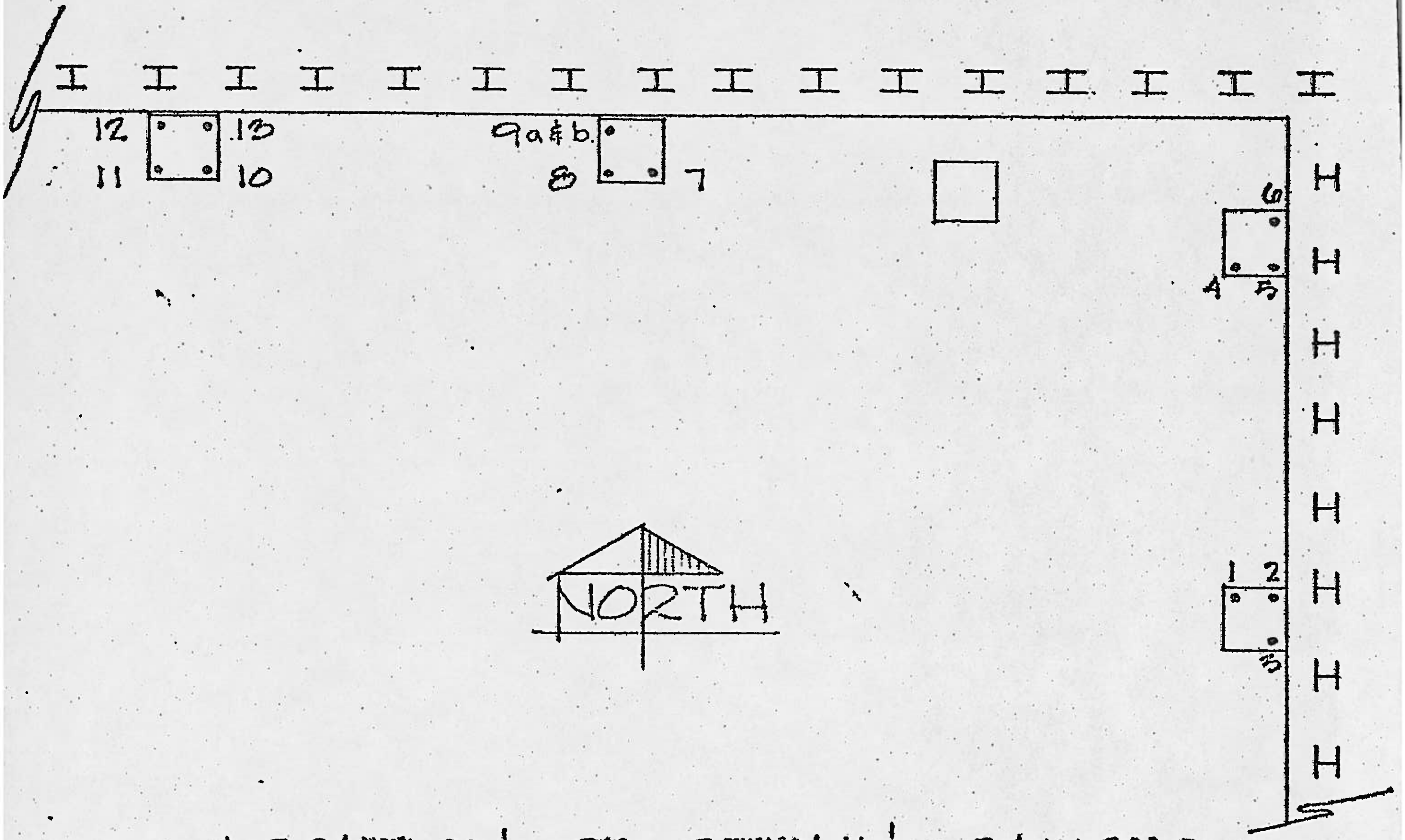
The results were very consistent with each other and the initial loads on the cells. In analyzing the reading of the load cells, it seems apparent that the load decreases as the anchors below are tensioned. As the time since installation increased, the readings became more eratic. However, with the resetting of cells #4, 5, and 6 fairly accurate comparisons could be made. At installation, the readings were 108.4, 109.4, and 95.4 kips (49.2 x 10^3, 49.6 x 10^3, 43.3 x 10^3 kg) for the respective cells. After tensioning of the two lower rows of anchors these loads decreased to 96.0, 97.3, and 91.3 kips. After a year, the loads determined when the cells were reset were 94.5, 94.4, and 83.6 kips (42.9 x 10^3, 42.8 x 10^3, 37.9 x 10^3 kg). This is a decrease of 1.5, 2.9, and 7.7 kips respectively (.6 x 10^3, 1.3 x 10^3, 3.5 x 10^3 kg), an average decrease of 4.2%. These results show that the wall is performing adequately and that there is a strong possibility that some slight amount of creep does occur.

With this magnitude, however, it is equally possible that changes in substrata settlement due to excavation and the new building load could create geometric changes in the wall and relax the load. If the cells will continue to function (Provision has been made for accessibility in the completed building), continued readings over several years may determine this with more certainty.

Surveying instruments were used to observe any vertical or horizontal movement in the building or in the ends of the anchors. Brackets with scales were attached at 16 places at the roof and at street level on the North and East sides of the adjacent building. Movements at 5 locations totaled 0.02 ft. (0.6 cm) over the past 14 months.

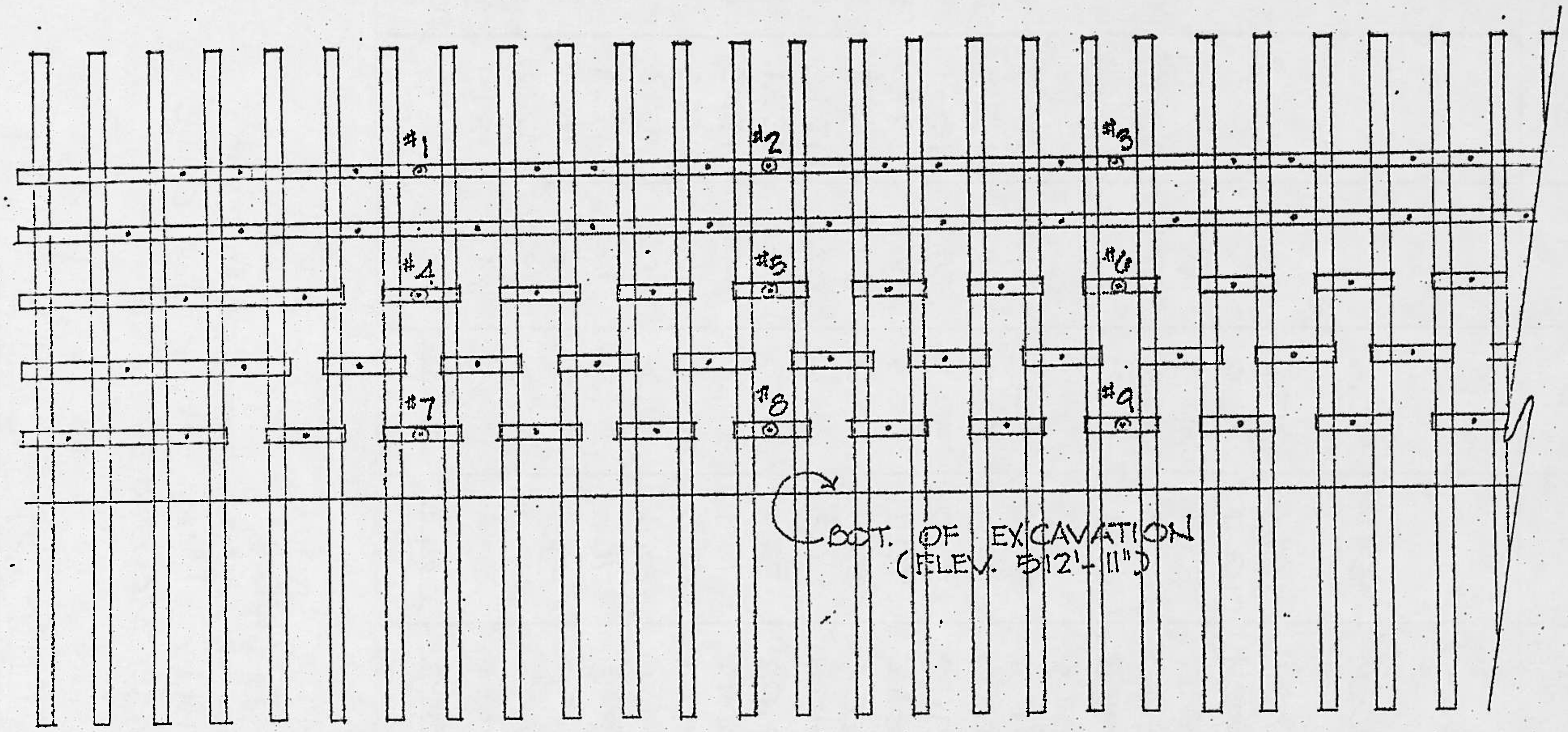
Since some of the movements are in opposite directions and the accurancy of the optical observation is 0.01 ft. (o.03 cm) at best, it is assumed no visible movement has occured in the bank. Vertically, no movement of the wall or adjacent building has occured, even though the new Block "H" tower has settled 3/8 in. (1.8 cm) as load was applied. Some of the top two rows of anchors were marked and their lateral movement also recorded. Seven out of 31 anchors surveyed appeared to move outward 1/8 in.

(0.6 cm), while others have not moved or shown a reverse deflection. Again the conclusion must be essentially zero movement.



LOCATION OF STRAIN GAUGES ON COLUMN BARS FOR MONITORING COLUMN LOADS DURING PILE DRIVING.

FIGURE F1



⊙ - DENOTES LOAD CELL

LOCATION OF LOAD CELLS

NORTH WALL OF ATLAS BANK BLDG.

FIGURE F2

	LOAD CELL NUMBERS							
DATE	#2	#3	#4	#5	#6	#7	#8	
/22/71		99.3*						
/2/71	84.7*	104.2						
/6/71	92.0	109.5						
/10/71	76.4	92.5	108.4*	104.4*	95.4*			
/11/71	88.2							
/12/71	84.0	106.4	108.6	107.4	97.2			
/18/71	75.5	93.6	102.4	100.1	96.0			
/23/71			98.7	97.4	92.9			
/24/71	96.0		96.0	97.3	91.3	101.2*	106.0*	107
/26/71	83.5	113.6	95.8	94.4	92.8			
/27/71	88.5	102.6	97.4					
/30/71	87.4	103.0	97.2	93.6	92.4	94.7	110.7	104
/7/71	81.5		92.3	81.9	93.3		127.8	106
/10/71	86.9	103.1	91.0	77.0	93.1	87.1	133.1	106
/20/71	93.9	110.1	89.2	78.7	92.3	85.6	134.6	10
/27/71	79.6	94.9	88.3	77.8	117.6	84.3	140.7	10
/4/71	86.5	92.8	87.9	76.8	91.6	82.3	134.8	10
/15/71	92.7	109.8	90.2	79.0	93.3	82.6	133.1	112
/25/71	82.9	99.6	89.6	77.9	94.7	80.7	136.5	10
/12/71		110.4	93.1	79.0	101.1	76.2	132.3	10
/26/71	112.3	130.6	96.8	81.1	111.6	75.0	129.0	101
/14/71	93.2	82.6	99.3	76.8	84.3	74.1	124.9	95
/10/72	106.6	89.2	119.9	82.0	86.2	73.1	136.6	98
/16/72	83.9	152.7	59.9		111.7	72.4	144.0	82
/9/72	77.9	51.6 COMP.	81.1		130.2	64.6	157.9	102
/15/72			94.5	94.4	83.6			

* - INSTALLATION.

CELL #1 MALFUNCTIONED.

ON 8/15/72 CELLS #4, #5, & #6 WERE RECALIBRATED, IT CAN BE ASSUMED THAT THE OTHER CELLS ARE NO LONGER ACCURATE.

LOAD CELL RESULTS (IN KIPS)

FIGURE E3

LATERAL PRESSURES AND MOVEMENTS CAUSED BY PILE DRIVING

By D. Joseph Hagerty,[1] A.M. ASCE

Driving piles is a commonplace method of foundation construction, and considerable study has been devoted to the analysis of the mechanics of pile driving. However, the emphasis in these studies and investigations has been on the insertion of the pile into various kinds of soil; e.g., how to maximize ease and rate of penetration and how to achieve maximum bearing capacity per pile in any given situation of soil and water conditions. Until recent years, little attention has been paid to the possible effects of pile driving on the area surrounding the pile foundation. Of course, in some instances the act of driving piles has produced damage in adjacent structures and facilities, and extensive litigation has resulted. Even so, no comprehensive program of research has been undertaken on the subject of the effects of pile driving on nearby facilities. Several individual case-history studies have been made which deal in part with detrimental effects of pile driving (Ireland, 1955; Klohn, 1963; Lambe and Horn, 1965; Hagerty, 1969; Hagerty and Peck, 1971; D'Appolonia, 1971; Hagerty and Garlanger, 1972), but in most of these studies the focal point of the research has been vertical movements of soil, piles and adjacent structures. Nevertheless, the lateral pressures and movements associated with pile driving can be most serious from the point of view of damage to existing facilities, and also from the point of interference with the installation of the pile foundation itself.

Lateral pressures and displacements caused by pile driving are significant: they are of considerable magnitude, in general; they occur at relatively rapid rates; and they can be seriously detrimental to adjacent facilities. The lateral pressures created during pile driving, particularly during pile driving in stiff cohesive soils, may easily exceed the design pressures which retaining structures such as braced cuts, anchored bulkheads and retaining walls are built to withstand. In several instances, lateral pressures as high as 2-3 times the total overburden pressure have been measured. See Figs. 1, 2A and 2B. The movements corresponding to such high pressures also can be of large magnitude; in one instance, lateral movements of in place pile caps of as much as twenty-four inches have been recorded as a result of later pile driving (Hagerty, 1969).

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[1] Assistant Professor of Civil Engineering, University of Louisville, Louisville, Kentucky.

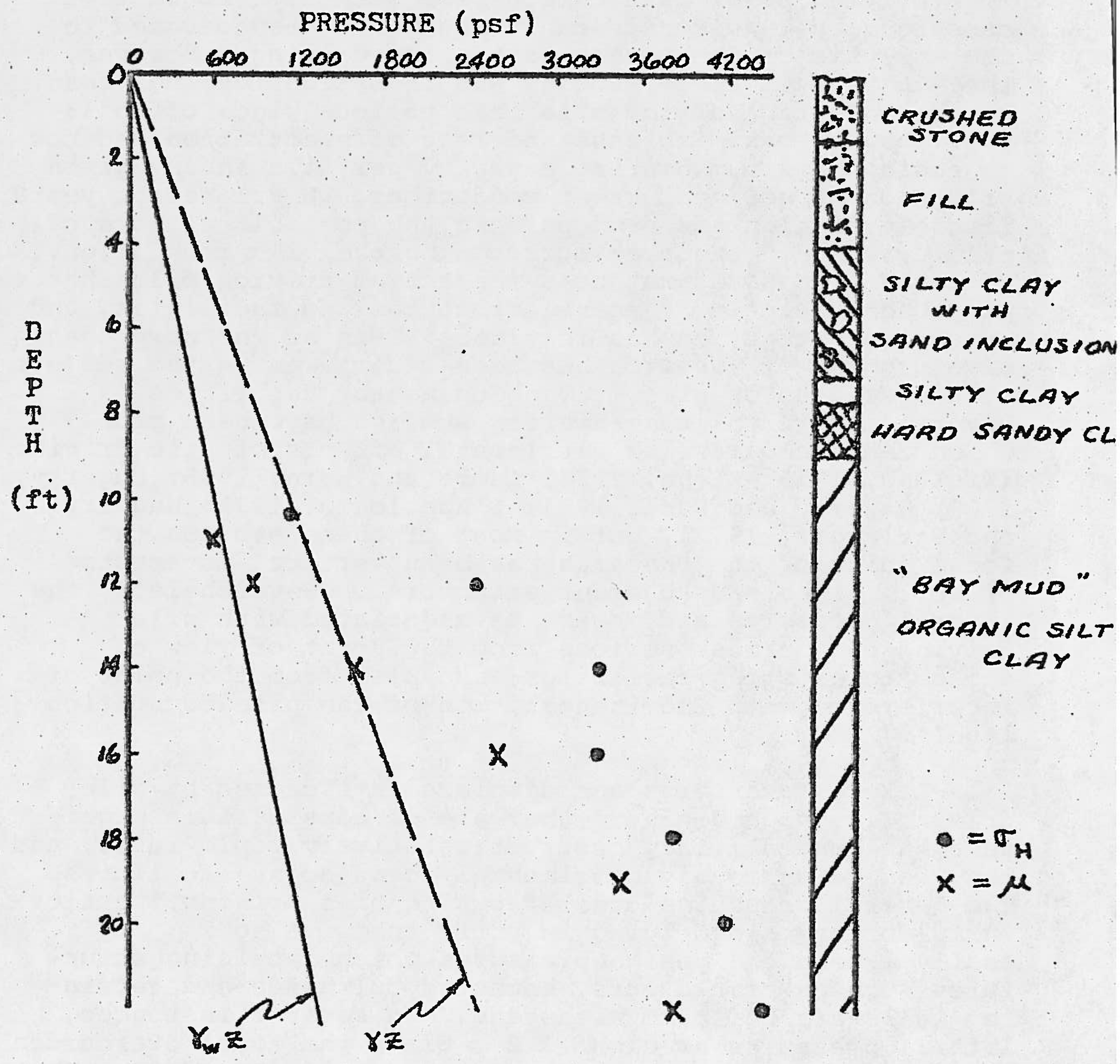
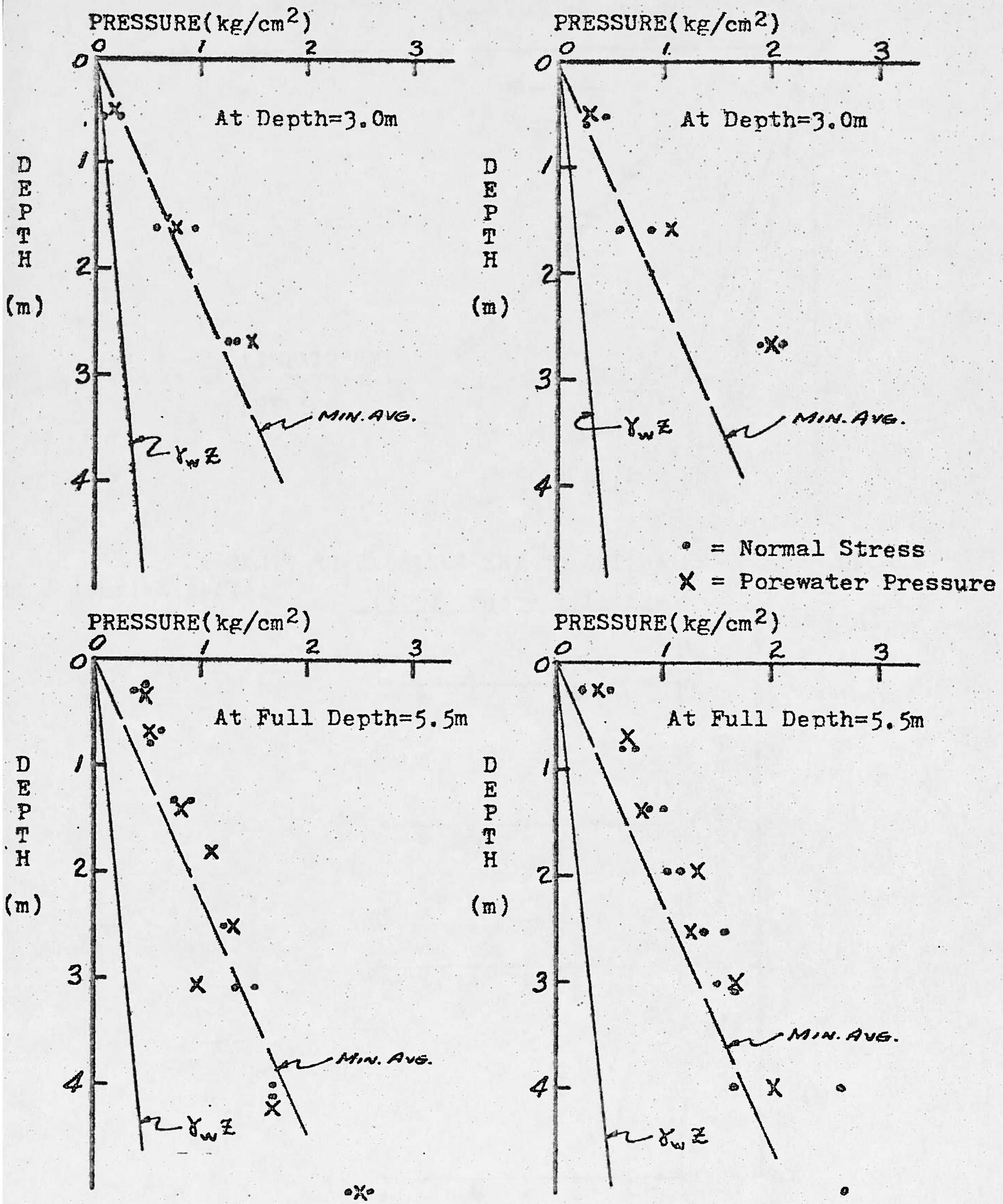


FIG. 1. PRESSURES GENERATED BY PILE DRIVING IN BAY MUD

(After Reese and Seed)



(a) Pile No. 1

(b) Pile No. 2

FIG. 2A. PRESSURES INDUCED DURING PILE DRIVING

(After Koizumi and Ito)

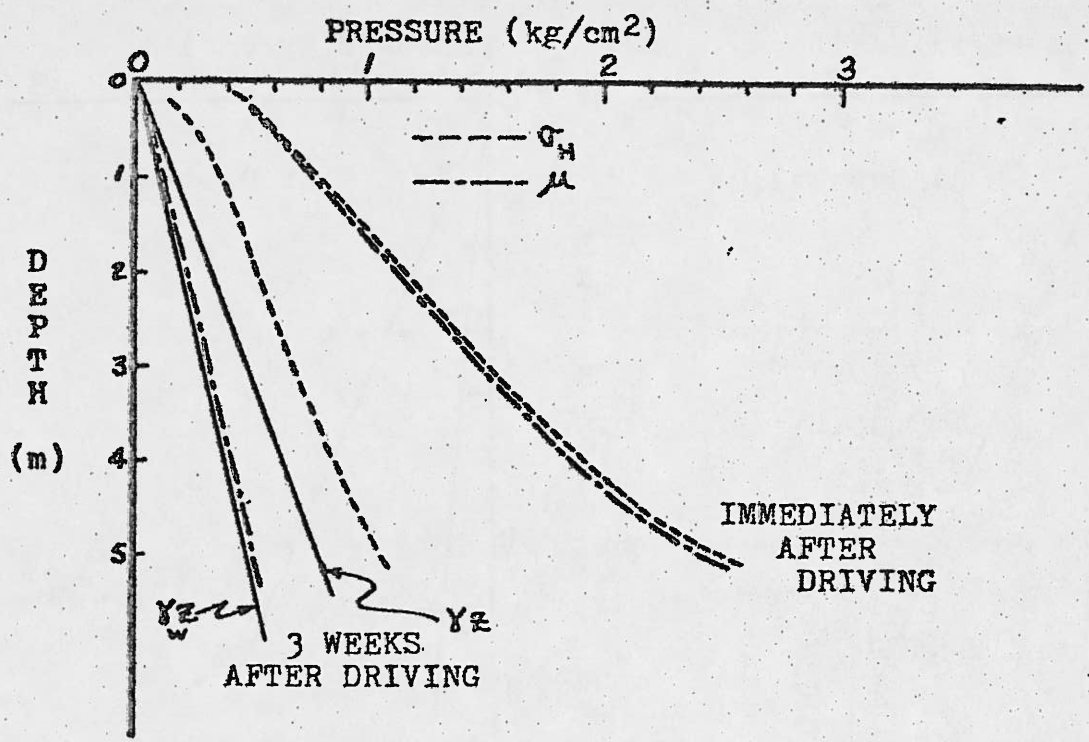


FIG.2C. PRESSURES ACTING ON THE SURFACES OF PILES

(After Koizumi & It

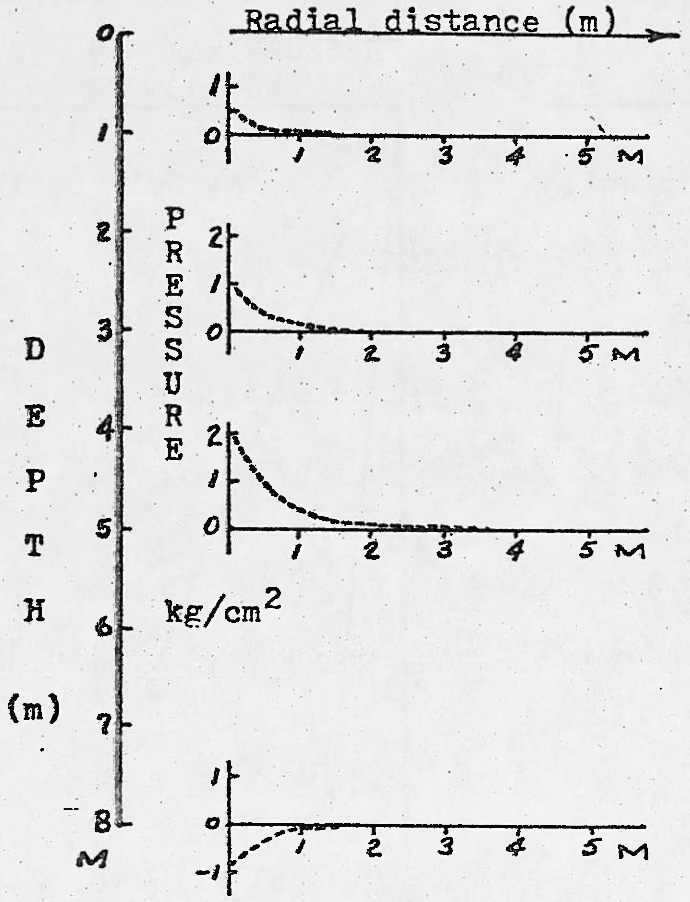


FIG.2B. RADIAL DISTRIBUTION OF INDUCED POREWATER PRESSURES

(After Koizumi and Ito)

The relatively rapid rate of pressure generation and the consequent rapid rate of displacement of deformable soils or structures is also significant. Many existing structures could withstand the lateral movements associated with pile driving if the movements occurred over long periods of time; however, the rapid distortion of building frames as a result of nearby pile driving will cause severe cracking and damage to the affected structures. An additional factor in the response of structures to movements caused by pile driving is that although the movements produced during pile driving occur very rapidly, the recovery of the structure (or rebound and reversal of movement) after pile driving generally occurs at a very slow rate because of the slow dissipation of generated pressures and slow stress-relief creep movements.

Finally, the lateral pressures and movements caused by pile driving are serious because most structures are not designed to withstand or tolerate lateral shifting of foundation elements. Lateral displacement of bearing walls as a result of nearby pile driving can create eccentricity of loading, overstressing of wall materials, and progressive failure of the support system. Of more immediate concern, the lateral displacement of foundation elements may actually remove all support from beneath beams or joists which carry the lower floors of a structure. Lateral displacement of driven piles or piers themselves obviously may cause eccentricity of loading, overloading of individual piles, and consequent failure of entire pile groups.

The general experience of movements of structures caused by pile driving is that of a sudden movement in a given direction with a long-term reversal of that movement in the opposite direction (Lambe and Horn, 1965). This slow reversal of movement can cause as serious problems as can the initial rapid movement. Remedial actions may be out of the question during the period of slow rebound or recovery of a damaged structure. Additionally, reversal of movement may open cracks and breaks in masonry elements which occurred as a result of the preceding rapid displacement during the pile driving operation.

In summary, lateral pressures and movements caused by pile driving can be of large magnitude, will occur relatively rapidly, and can cause serious damage to nearby structures or facilities. It behooves the foundation engineer to anticipate such movements; to expect such pressures, to attempt to predict magnitudes of pressures and movements, and to formulate preventive measures. In this case (lateral pressures and movements), prevention may be relatively easy, but remedy or cure may be impossible.

PREDICTION OF PRESSURES AND MOVEMENTS

The problem of predicting the magnitudes of lateral pressures and lateral movements caused by pile driving has been investigated in part in several past studies. However, no complete solution to the problem has been found, nor is one likely to be obtained in the near future.

A number of investigations have formulated expressions for preducting the excess porewater pressures generated during pile driving (Reese and Seed, 1955; Lo, 1961; Ladanyi, 19631963; Lo and Stermak, 1965; D'Appolonia, 1971; Hagerty and Garlanger, 1972). However, the theoretical expressions developed by these investigators are all based upon certain factors such as existing vertical stress, existing lateral stress, and undrained soil shear strength and angle of internal friction, which are subject to considerable change during pile driving. For example, the displacements of soil by a driven pile will remold the soil in a zone around the pile (Zeevaert, 1950; Legget, 1950). Thus, porewater pressures are generated simply because the soil is not consolidated, in its remolded state, at the stress levels existing in the soil prior to pile driving. Furthermore, the ambient stress levels are undoubtedly changed; the increase in lateral total stress during pile driving is generally recognized, but this increase may be accompanied by an increase in total vertical stress. A long-term increase in lateral effective stress certainly will occur. See Fig. 2C. Finally, the remolding action of pile driving decreases the undrained shear strength of the penetrated soil, the extent of the strength loss being dependent upon the sensitivity of the soil. For these reasons, no theoretical prediction of generated porewater pressures should be considered reliable. A preferable procedure is to estimate the generated excess porewater pressures on the basis of collected experience.

The estimation of generated excess porewater pressures can serve a double purpose; data from a number of case histories show that during and immediately after pile driving the total porewater pressure is virtually equal to the total lateral stress, with the lateral effective stress temporarily equal to zero. See Figs. 2A and 2C. Therefore, estimation of excess porewater pressures yields estimates of created total lateral pressures.

To facilitate the estimation of generated excess porewater pressures, data has been collected from a number of case records and is presented in Figure 3. As mentioned previously, in a number of instances the generated porewater pressures and total lateral pressures have been equal to the total overburden pressure (Reese and Seed, 1955; Milligan et al, 1962; Bjerrum and Johannessen, 1960), and in a few cases the generated pressures have exceeded the overburden pressure (Hanna, 1967; Reese and Seed, 1955; Koizumi and Ito, 1967).

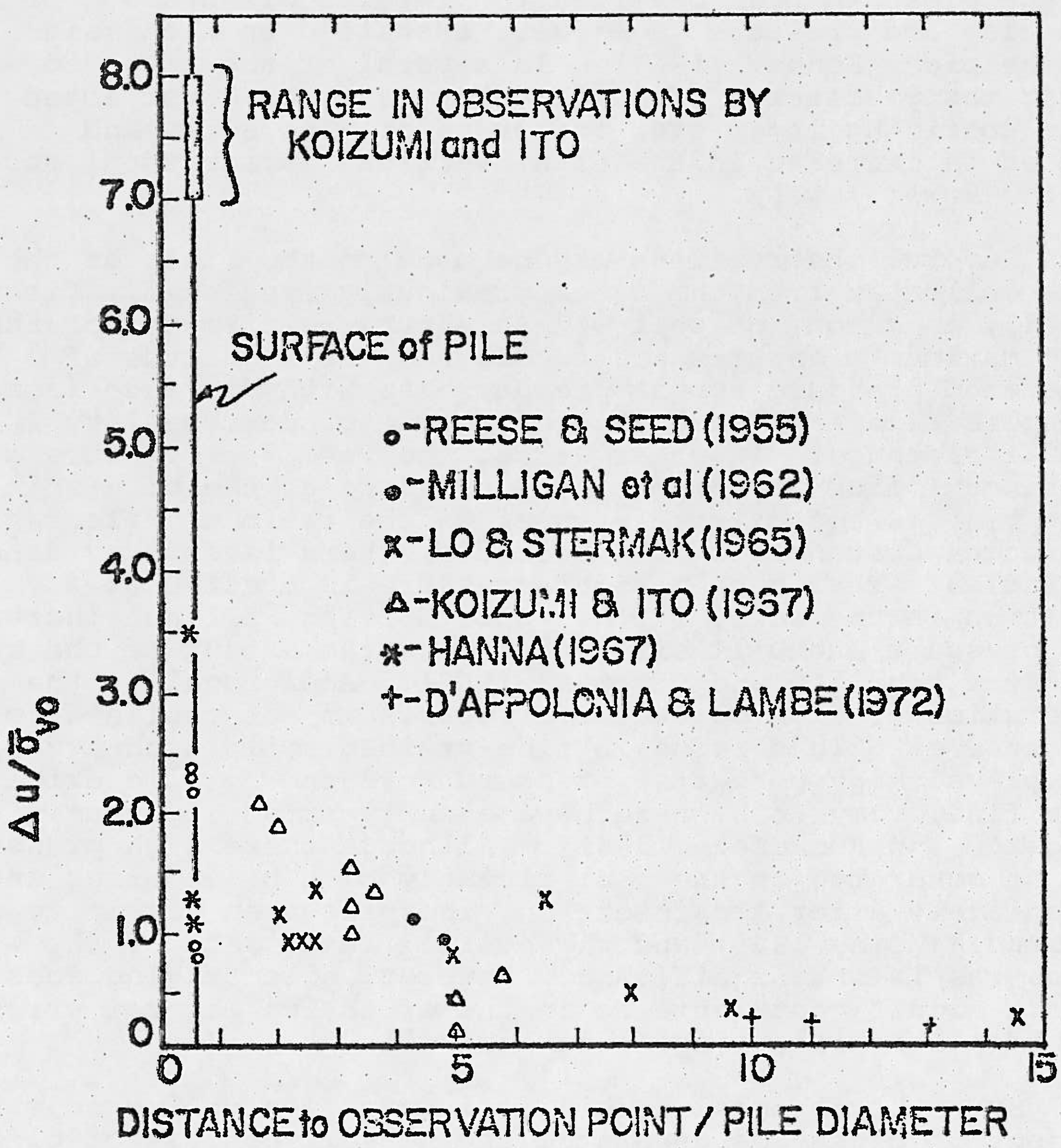


FIG.3-EXCESS PORE PRESSURES GENERATED BY DRIVING SINGLE PILES

In one particular case, porewater pressures equal to the overburden pressures were measured at the skin of a large-diameter pipe which was allowed to sink into a sensitive Norwegian clay under its own weight only; the bottom edge of the pipe had been bevelled to minimize disturbance of the clay and pressure gages were installed on the inside of the pipe (Kenney, 1967). In several of the recorded cases the presence of a stiff upper layer or crust acted as a confining layer over softer underlying soils and caused an increase in the total vertical (overburden) stress at the lower levels.

Several observations may be made on the basis of the data collected from the cases previously mentioned. First of all, in a zone of soil within about one diameter of the pile maximum pressures are generated; the magnitude of generated pressure appears to decrease with distance from the pile skin approximately in inverse proportionality to radial distance. In other words, the ratio of pressure at distance r from the pile to the pressure at the pile skin, u_r/u_{pile}, is approximately equal to the ratio of pile radius to radial distance, r_p/r. Secondly, there is some evidence to indicate that once maximum pressure is created at any location in the soil, further pile driving will not increase the pressure but will simply increase the extent of the high pressure zone (Lo and Stermak, 1965). Additionally, the generation of high porewater pressures is not confined to the case of pile driving in fine-grained soils such as silts and clays; high porewater pressures created by pile driving in a fine, very loose sand have been reported (Bjerrum, Kingstad and Kummeneze, 1961). Although these high pressures may be generated in the near vicinity of a pile during and immediately after its insertion, the pressures dissipate quickly in sands (Horn, 1966) and the overall importance of the water pressures is not significant. However, pile driving does produce a significant increase in the effective lateral stresses around piles driven into granular soils.

Even though considerable data have been collected on the subject of pressures generated in the soil because of pile driving, no comprehensive collection of information on lateral movements associated with pile driving has been made. The prediction of lateral movements to be anticipated as a result of pile driving cannot be done on any theoretical basis, and examination of the scant collected data on lateral movements will permit of only a gross empirical estimate of anticipated movements. A number of factors contribute to the complexity of the problem of predicting lateral movements (and lateral pressures, for that matter) caused by pile driving. These factors include: as mentioned previously, the characteristics of the soil into which the piles are driven; the sequence of pile driving operations; and the geometry of the entire foundation area.

FACTORS AFFECTING PRESSURES AND MOVEMENTS

Effects of Soil Characteristics

Driving piles into loose granular materials will cause densification of the loose soil and surficial settlement (Swiger, 1948; Lynch, 1960; Terzaghi and Peck, 1967; Feld, 1968). Apparently the zone of significant volume decrease is limited to radial distances of two to three diameters away from the driven pile (Plantema and Nolet, 1957; Kerisel, 1961), although some surficial settlement may be caused at distances of as much as 50 feet from the locus of pile driving (D'Appolonia, 1971).

Lateral stresses created in granular soil as a result of pile driving may be very high, as mentioned previously, but they are of short duration. Nevertheless, significant long-term changes in lateral effective stresses are produced by pile driving, even in sands and gravels. Increased in lateral stresses generally are confined to the soil within the immediate zone of pile driving, and widespread increases in lateral pressures (with consequent lateral movements of structures and adjacent facilities) have not been noted when piles were driven in sands.

When piles are driven into fine-grained soils with low values of permeability, the soil behaves essentially incompressibly. Practically no drainage occurs during the insertion of a pile. Because of this short-term incompressibility of clays during pile driving, the effects of driving in clays are much greater in areal extent than are the effects of driving in sands which readily densify under the vibrations associated with pile driving. Pile driving in clay soils has been discussed in a number of previous publications (Klohn, 1963; Casagrande, 1947; Cummings et al, 1950; Holtz and Lowitz, 1965; Horn, 1966; Lambe and Horn, 1965; Orrje and Broms, 1967; Zeevaert, 1950); in the following paragraphs only those aspects of pile driving in clay which pertain to lateral pressures and movements will be mentioned.

One additional comment should be made concerning the influence of soil characteristics on generated pressures; sensitive fine-grained soils tend to liquify around a pile during driving and the effects of the pile driving are localized within one or two diameters of the pile skin (Zeevaert, 1950; Legget, 1950; Tajime, personal communication).

In addition to the effects of soil properties, the overall geometry or arrangement of the pile foundation can have an important influence on generated pressures and movements.

Effects of Foundation Geometry

In the context of this discussion, the terms "foundation geometry" are used to indicate the physical arrangement of the foundation area--the changes in elevation from one part of the area to another, the presence of existing structures around or in the foundation area, etc. The general effect of differences in elevation within the foundation area is that of a weak zone where the elevation is lowest because of the obvious absence of soil there to resist lateral forces created by the pile driving. The overall influence of existing structures is a stiffening effect on the subsoils, which tends to reduce movements in the direction of the existing facility. In effect, the soil is prestressed at such a location and consequently is made stiffer. The effects of foundation geometry can be demonstrated best by a brief examination of several cases.

In one instance, flexible piles were driven behind a bulkhead into a soft clay deposit (Hagerty, 1969; Hagerty and Peck, 1971). Plan and profile views of the site are shown in Fig. 4A and 4B. Driving proceeded generally away from the anchored bulkhead and both driven piles and the bulkhead were displaced toward the river by later driving. Displacements measured shortly before the bulkhead was shored with additional cable anchorages are shown in Fig. 4C. In this case, the absence of soil on the river side of the bulkhead created a weaker area with respect to generated lateral pressures, and lateral movements occurred preferentially toward the river. This case also illustrates the fact previously mentioned, that earth supporting structures are not designed or built to withstand magnitudes of pressures which are generated during pile driving.

In a second instance, discussed in detail elsewhere (Hagerty, 1969), displacement piles were driven into soft and medium stiff clays to support columns of a hospital structure. Plan and profile views of the site are shown in Figs. 5A and 5B. Small-magnitude lateral movements of driven piles were noted during the driving of later piles within the same pile cluster, as shown for a typical cluster in Fig. 5C. However, the presence of a low spot at the proposed elevator pit location, and the difference in elevation between the basement and sub-basement levels of the structure caused large lateral movements of driven pile clusters. These movements continued for as long as sixty days after the conclusion of pile driving. The large magnitudes of these movements may be noted in Fig. 5D.

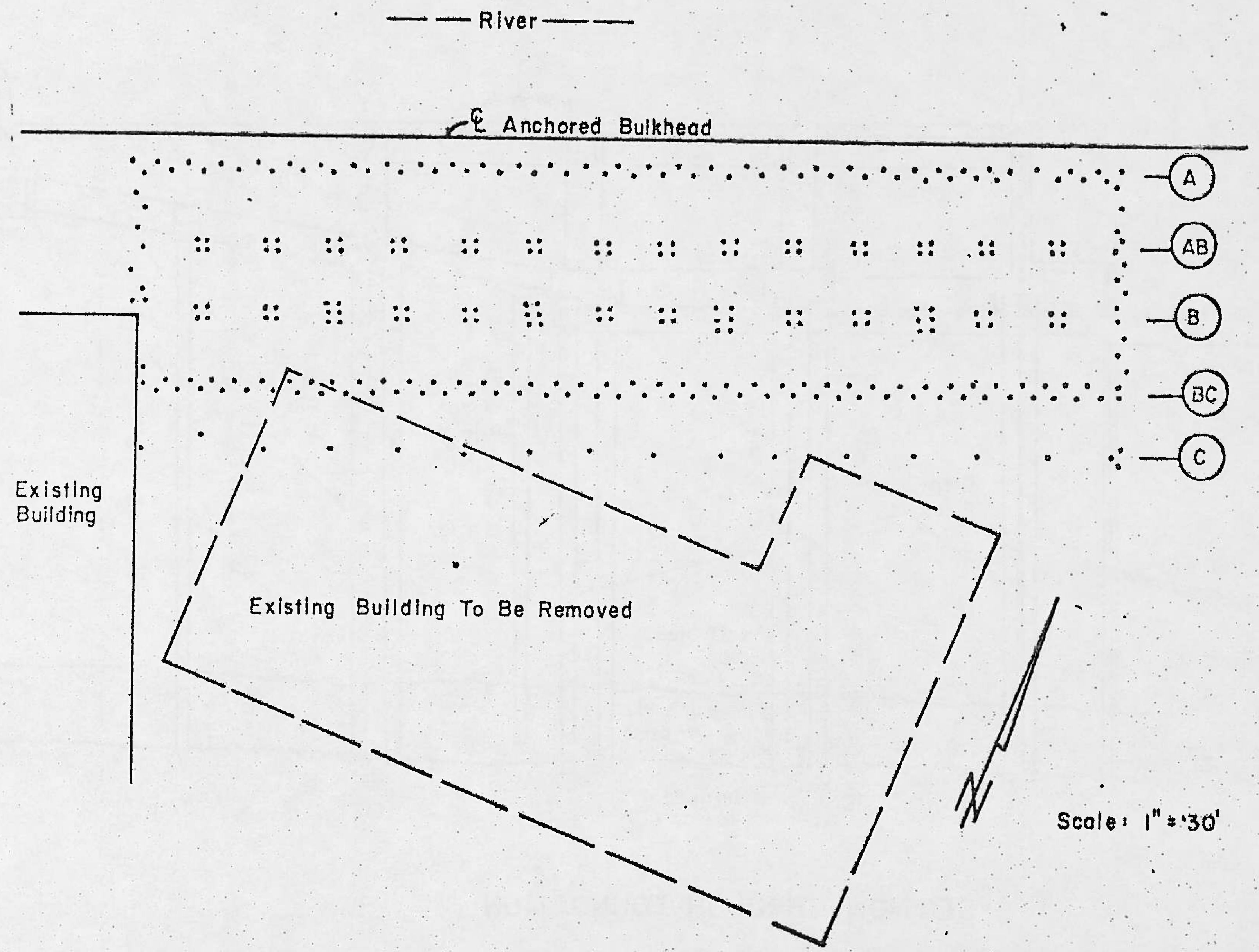
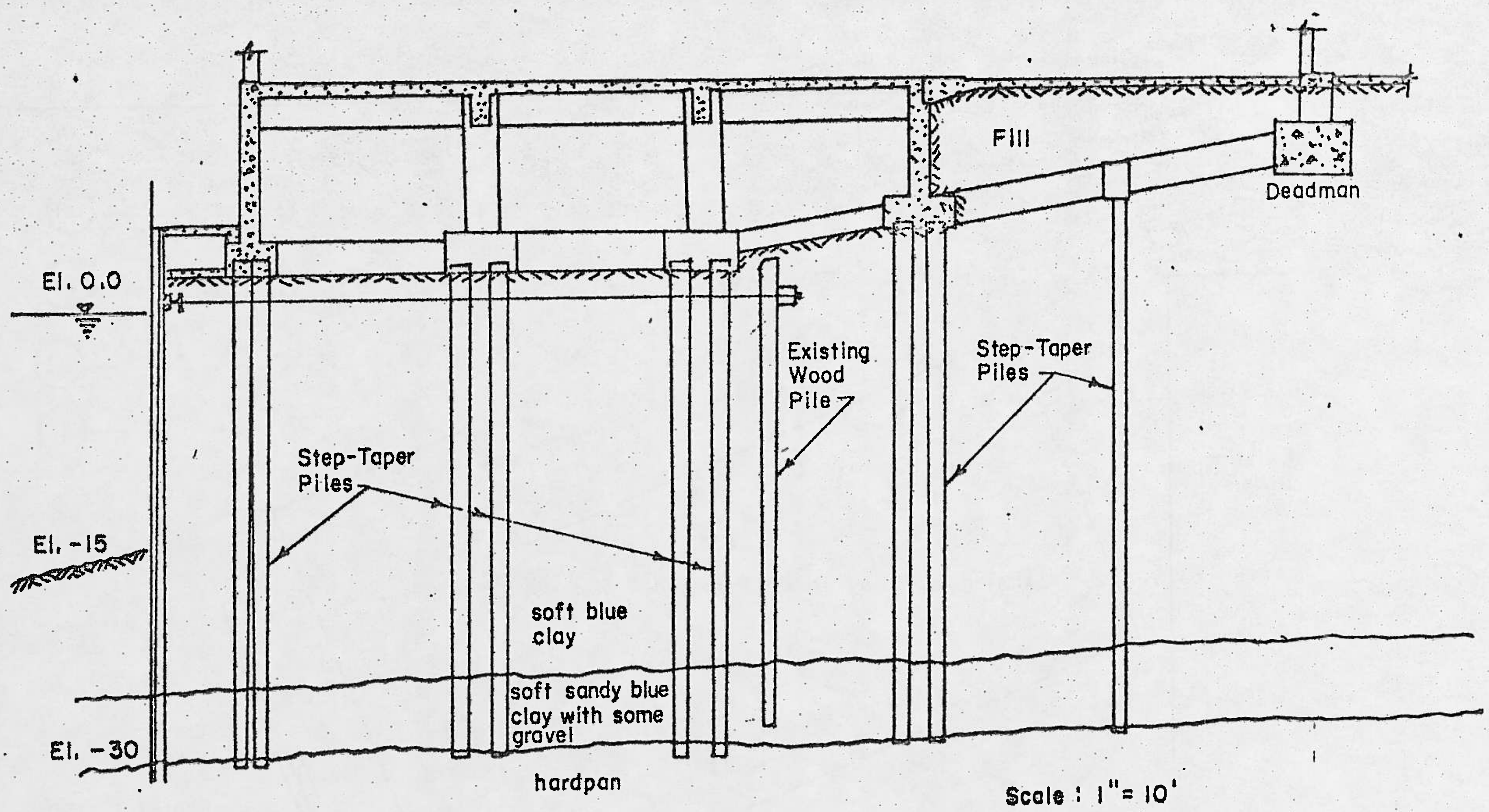
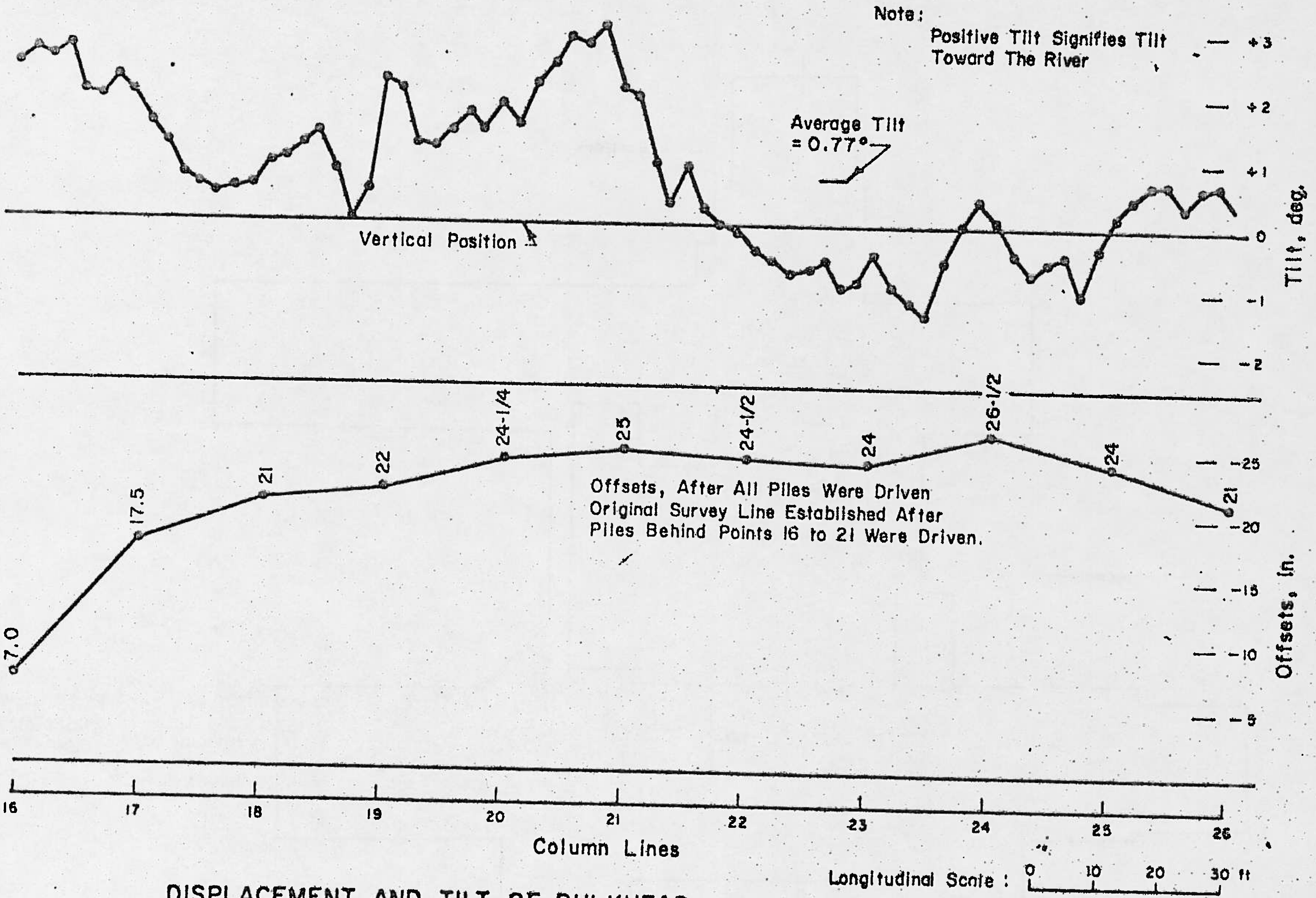


FIG. 4A. PLAN, PROPOSED WAREHOUSE ADDITION



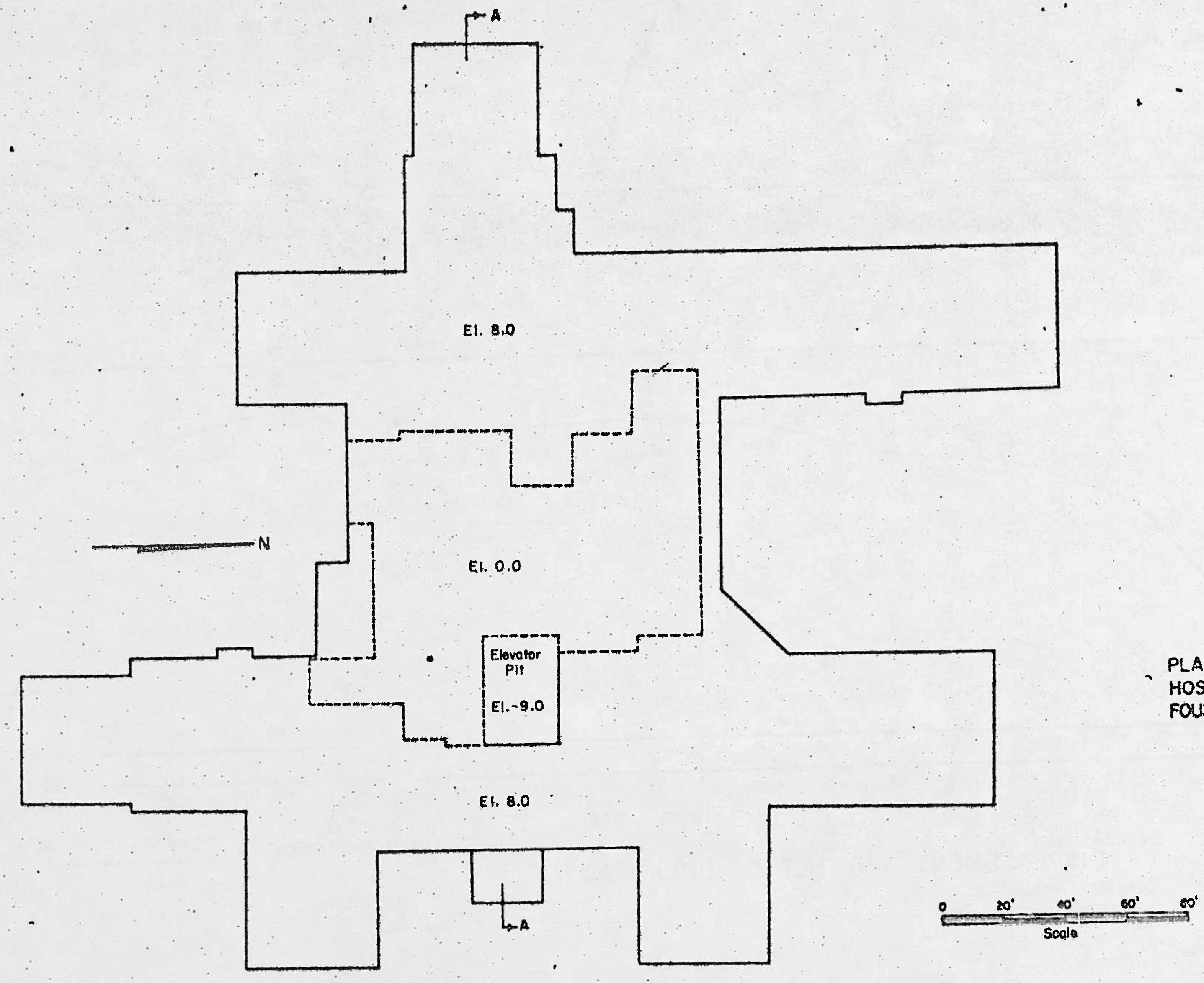
SECTION THROUGH FOUNDATION

FIG. 4B.



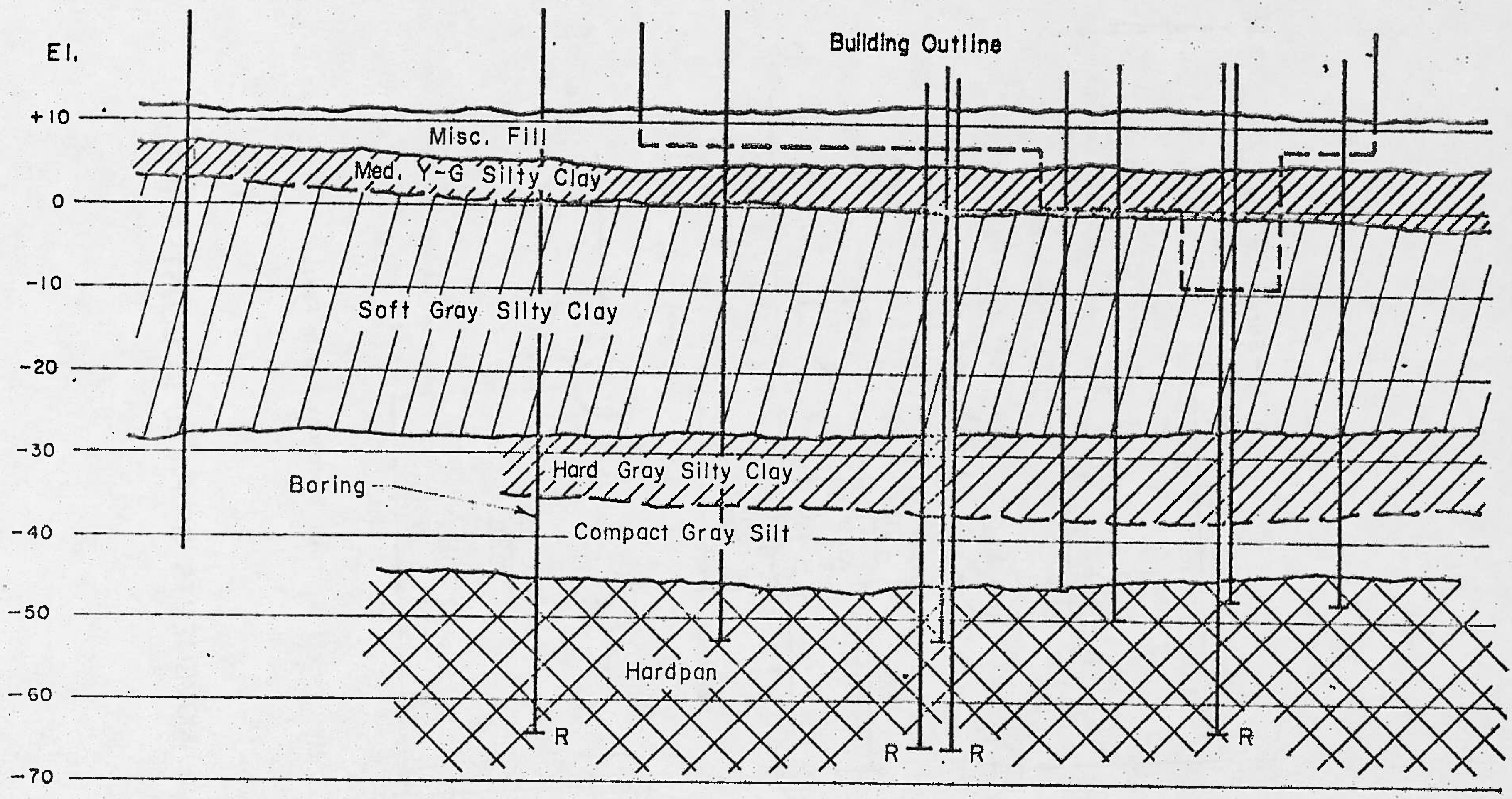
DISPLACEMENT AND TILT OF BULKHEAD

FIG. 4C.



PLAN VIEW OF
HOSPITAL
FOUNDATION,

FIG. 5A



Section A—A

FIG. 5B

SECTION THROUGH FOUNDATION

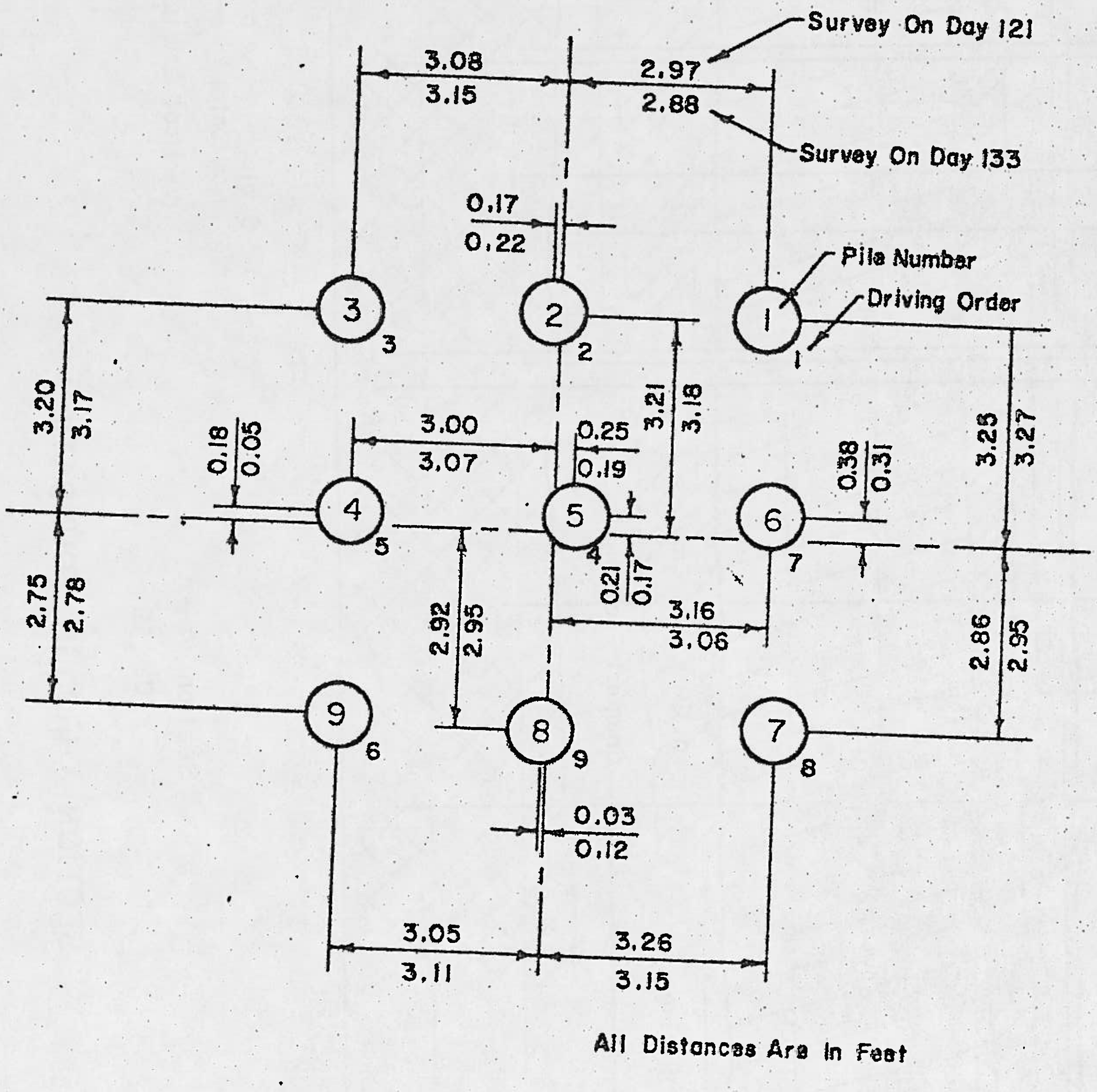
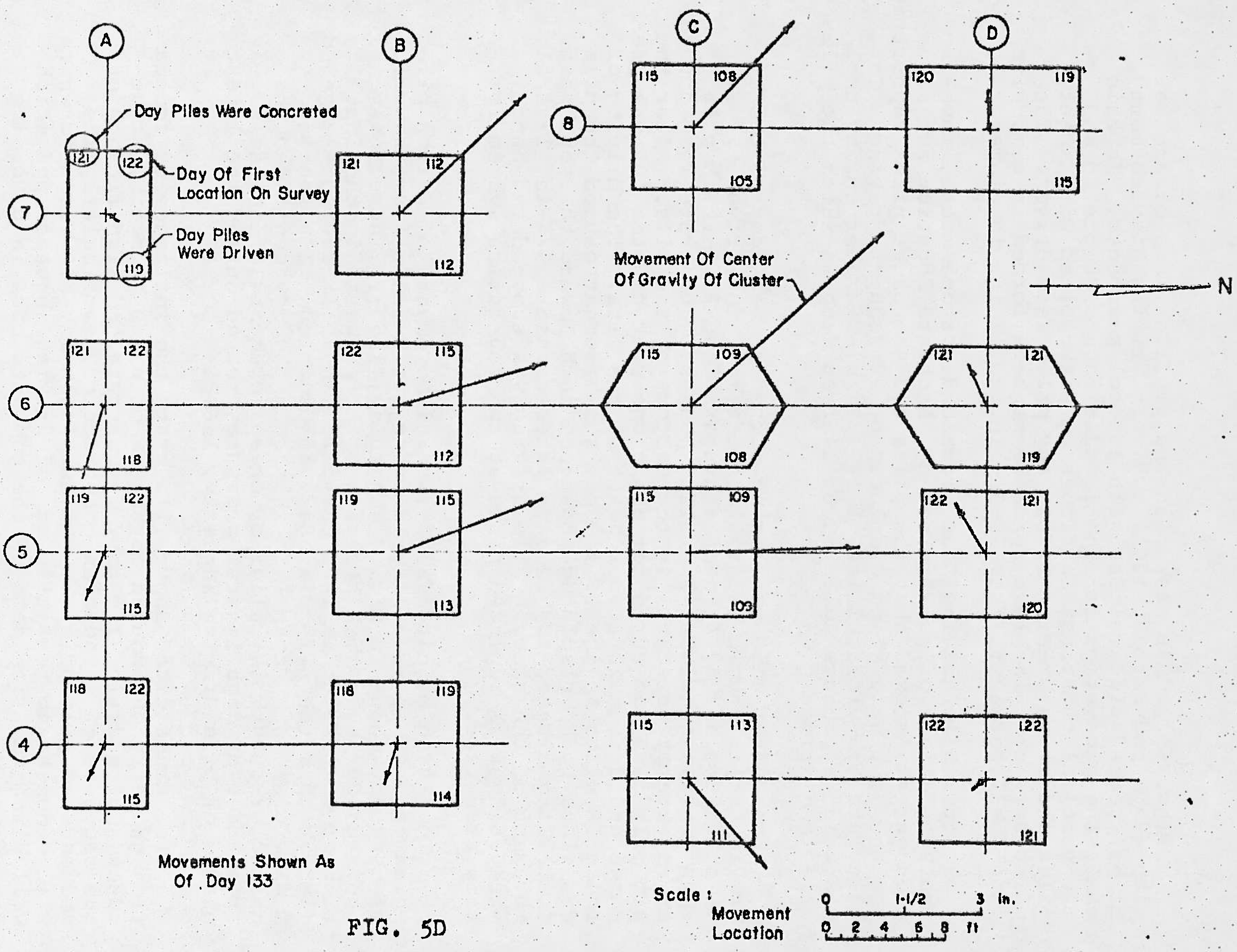


FIG. 5C LATERAL MOVEMENTS OF PILES IN CLUSTER D-5



Movements Shown As Of Day 133

FIG. 5D

LATERAL MOVEMENTS OF PILE CLUSTERS

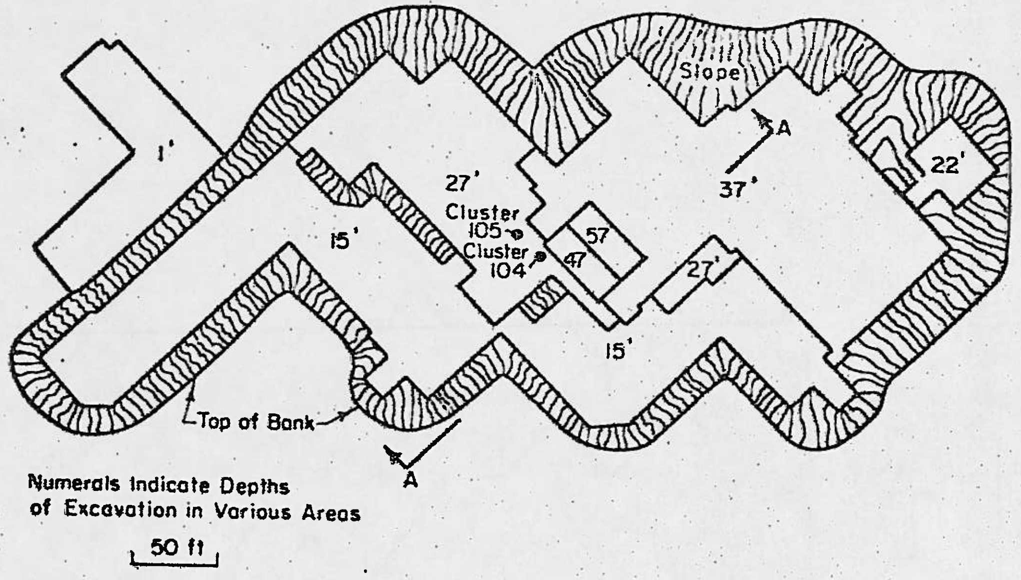
Figs. 6A and 6B show plan and profile views of another site (Peck and Berman, 1961), wherein driving displacement piles created difficulties with lateral movements. In this case, the excavation for the foundation was completed and the vertical construction slopes were supported with sheet piles and cross bracing before any piles were driven. The piles were cast-in-place concrete members formed in composite pipe-shell sections, from 10-3/4 in. to 12 in. in diameter.

After pile driving had proceeded for one month, movements of driven piles were noted. Accurate location and elevation surveys showed that the piles and the sheet piles supporting the deep cut were being displaced by subsequent driving. To alleviate the difficulties, a 14-1/2 in.-diameter coring tool was used to prebore holes about 14 feet deep in which the piles were then driven.

Even after this pre-coring procedure was adopted the driving still caused lateral displacements of driven piles. In the 47 ft and 57 ft-deep elevator pits, 8 ft-thick pile caps containing almost 3 million pounds of concrete were poured in an attempt to stabilize the bottoms of the pits. After the pit pile caps had cured for about one week cluster 105 was pre-cored and driven 20 ft northeast of the pits. The driving of these 18 piles in the 27 ft-deep sub-basement caused the pile cap slabs to be displaced about one inch toward the southwest. Installing cluster 104 12 days later caused a 3/4-in. lateral movement of the slabs. Pile driving was stopped and hand-excavated piers replaced piles as the foundation system for this structure.

The large magnitudes of lateral movements of driven piles which can result from later driving can be seen from the values of movement shown in Fig. 7 (Thornley, 1953). These movements occurred during a work stoppage of three weeks in the installation of composite displacement piles in 20 ft-deep pre-augered (larger than the pile diameter) holes through 110 feet of medium clay to hardpan. In this case the movements had the character of stress-relief or creep movements of the soil away from zones of high lateral pressure created during the latest phases of pile driving (Hagerty, 1969).

The cases cited above illustrate the influence of foundation geometry on induced movements of piles or nearby structures, but additionally they demonstrate the significance of the sequence of foundation construction, particularly the sequence of pile driving. Another case may be presented which will demonstrate the influence of driving sequence and which will also summarize some of the comments made in preceding paragraphs.



Numerals Indicate Depths of Excavation in Various Areas

FIG. 6A. EXCAVATION LEVELS

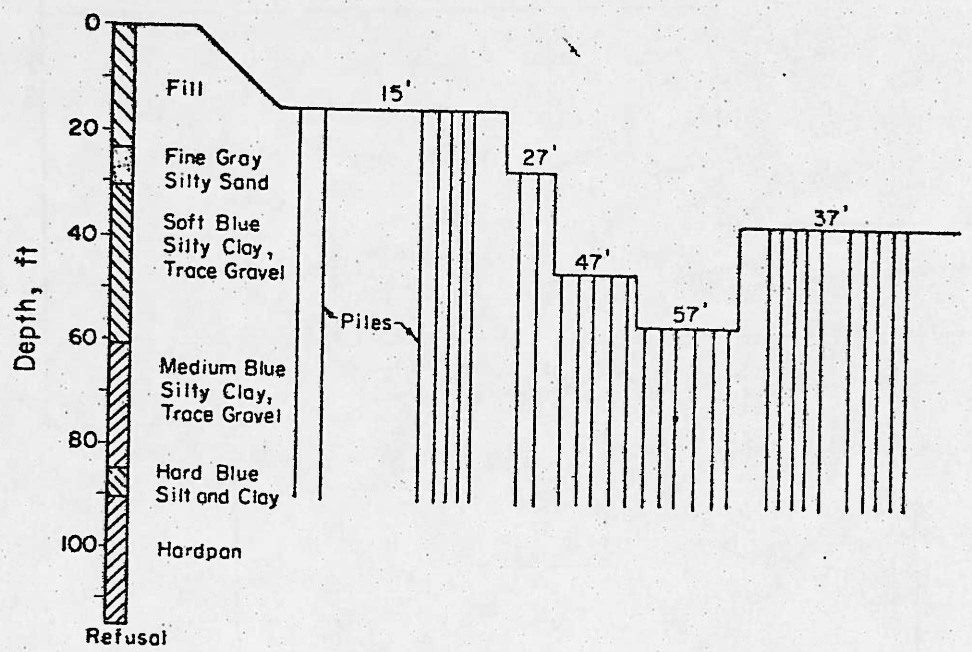


FIG. 6B SECTION A-A THROUGH FOUNDATION

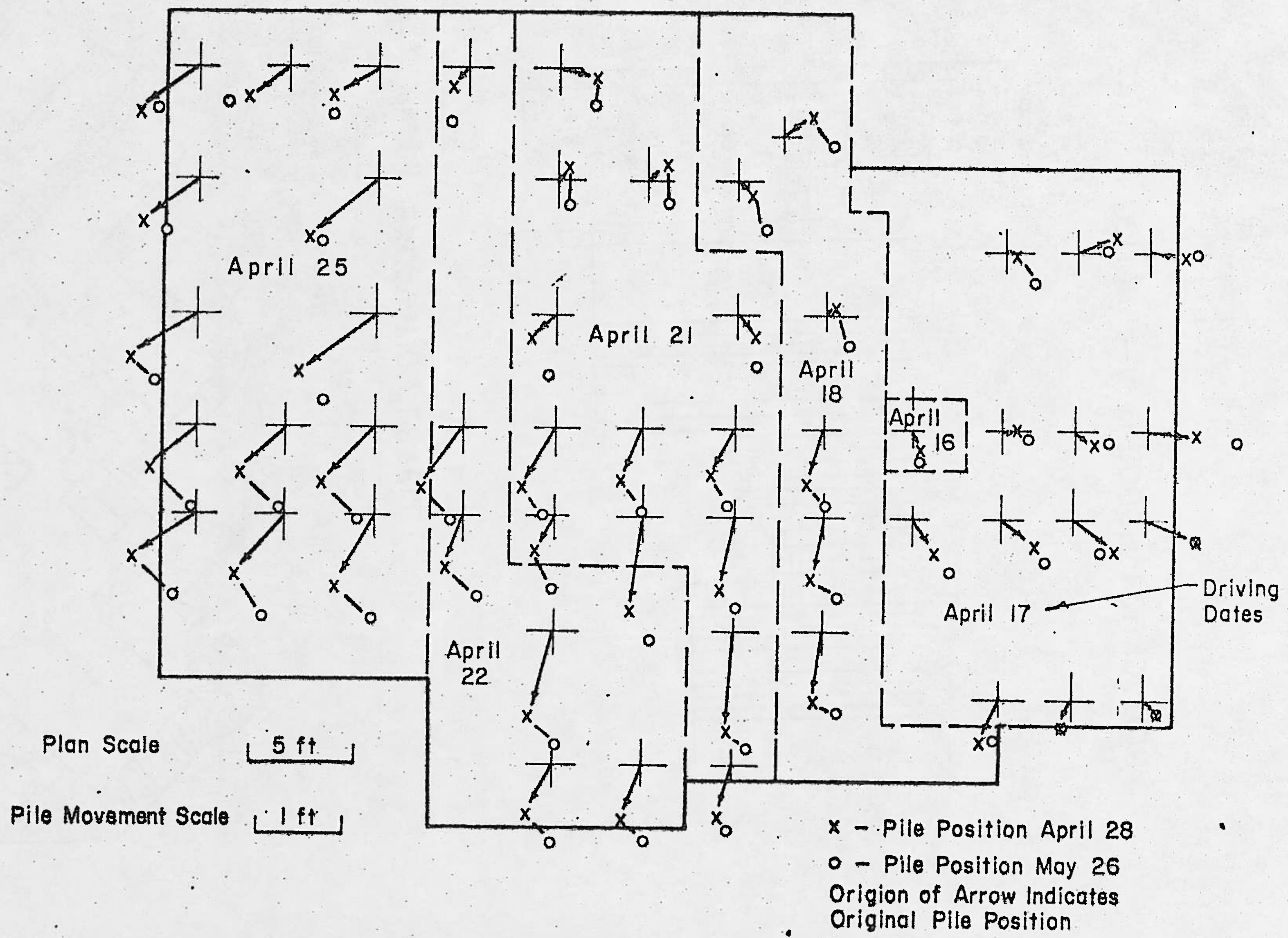


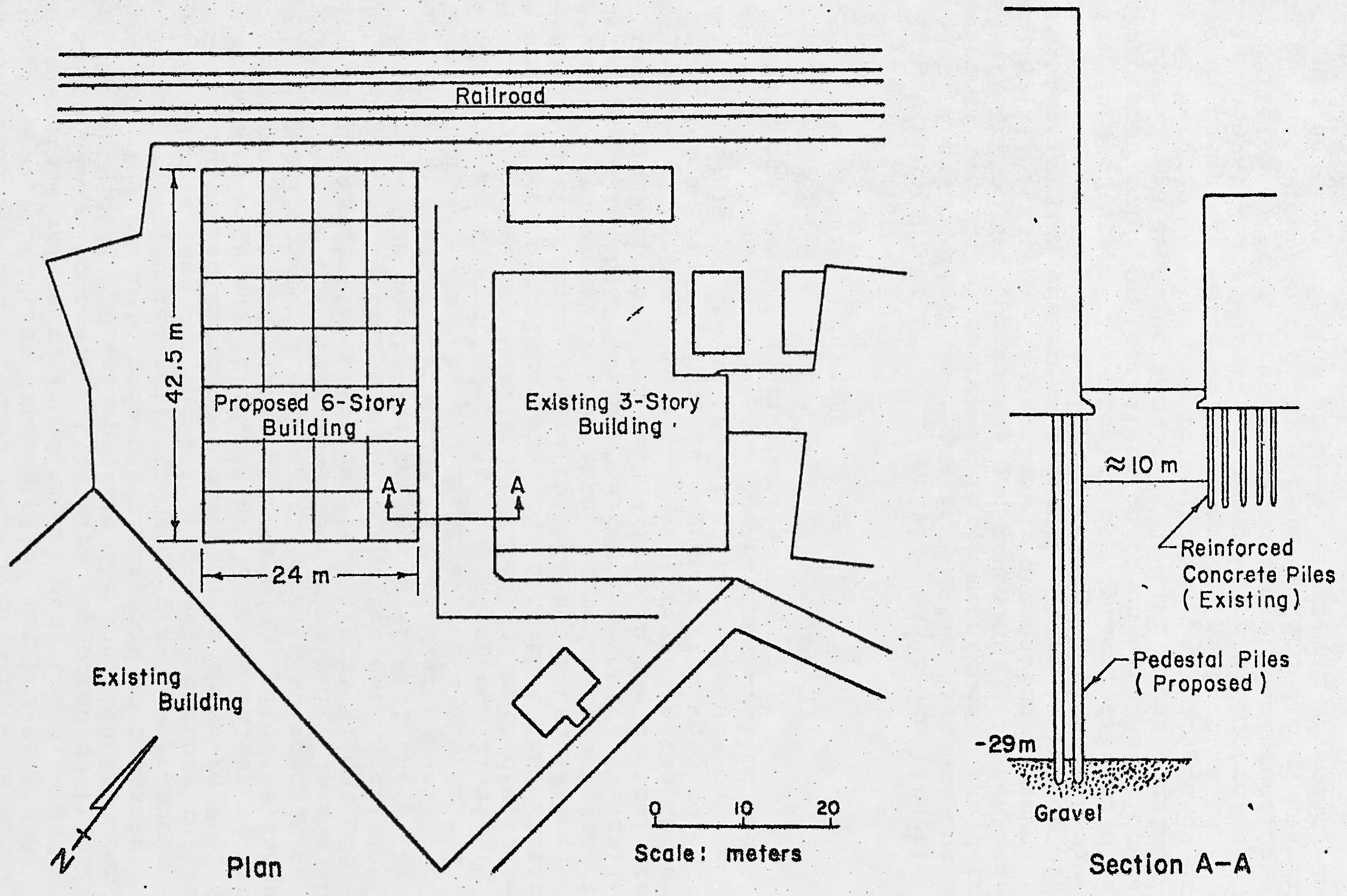
FIG. 7 LATERAL PILE MOVEMENTS

Effects of Construction Sequence

Fig. 8A shows a plan view of a site where careful observations of soil movements caused by pile driving were made because it was feared that driving piles for the proposed building would displace railroad tracks near the site and would cause damage to nearby structures supported on friction piles (Hokugo, 1964). A soil profile for the site is shown in Fig. 8B. The concrete pedestal piles used to transmit load through the alluvial soils to dense gravel were about 20 in. in diameter and are shown in Fig. 8C. Surface reference points were installed throughout the site as shown in Fig. 8E and piezometers were installed. Fig. 8D shows values of porewater pressure generated during pile driving. Fig. 8F shows observed lateral movements. The horizontal displacement of the points south of the piles occurred principally during the driving of the first line of piles (1-12); some slight additional movement of southside points occurred during driving of piles 26-62. The points on the north side of the piles moved away from driving operations in a regular manner. On the basis of these movements and observed heave movements, the design engineers concluded that the first two lines of piles acted as a shelter wall and reduced the soil displacements south of the pile locations, and also that the existing pile-supported structures tended to make the subsoils more resistant to displacements. Because the movement appeared to be taking place in the "flexible" direction toward the raiod tracks, no further piles were driven in the central area of the site. Forty-six more piles were driven in the southwest corner of the foundation, in sequence from west to east to minimize displacements of the existing buildings on the west, and in the remainder of the foundation, drilled piers were substituted for the undriven piles.

SUMMARY

It has been shown that pile driving can cause lateral movements of structures, soils and foundation elements and can generate lateral pressures of large magnitude. The detrimental effects of such pressures and movements has been mentioned and illustrated. As mentioned, prevention of such difficulties is much easier to accomplish than is remedy of a situation where they have already arisen. Prevention may take the form of use of low-displacement piles, driving piles in pre-augered holes or the selection of an alternate foundation system. Pre-augering should be done with care, using slurry techniques to minimize lost ground; partial pre-augering in several cases was shown to be somewhat ineffective in reducing generated pressures. However, careful control of pre-augering operations, prudent selection of construction sequence, and allowance for the effects of foundation geometry can do much to alleviate the problem of lateral movements and pressures created by pile driving.



Plan

Section A–A

PLAN OF TELEPHONE BUILDING (After Hokugo)

FIG. 8A

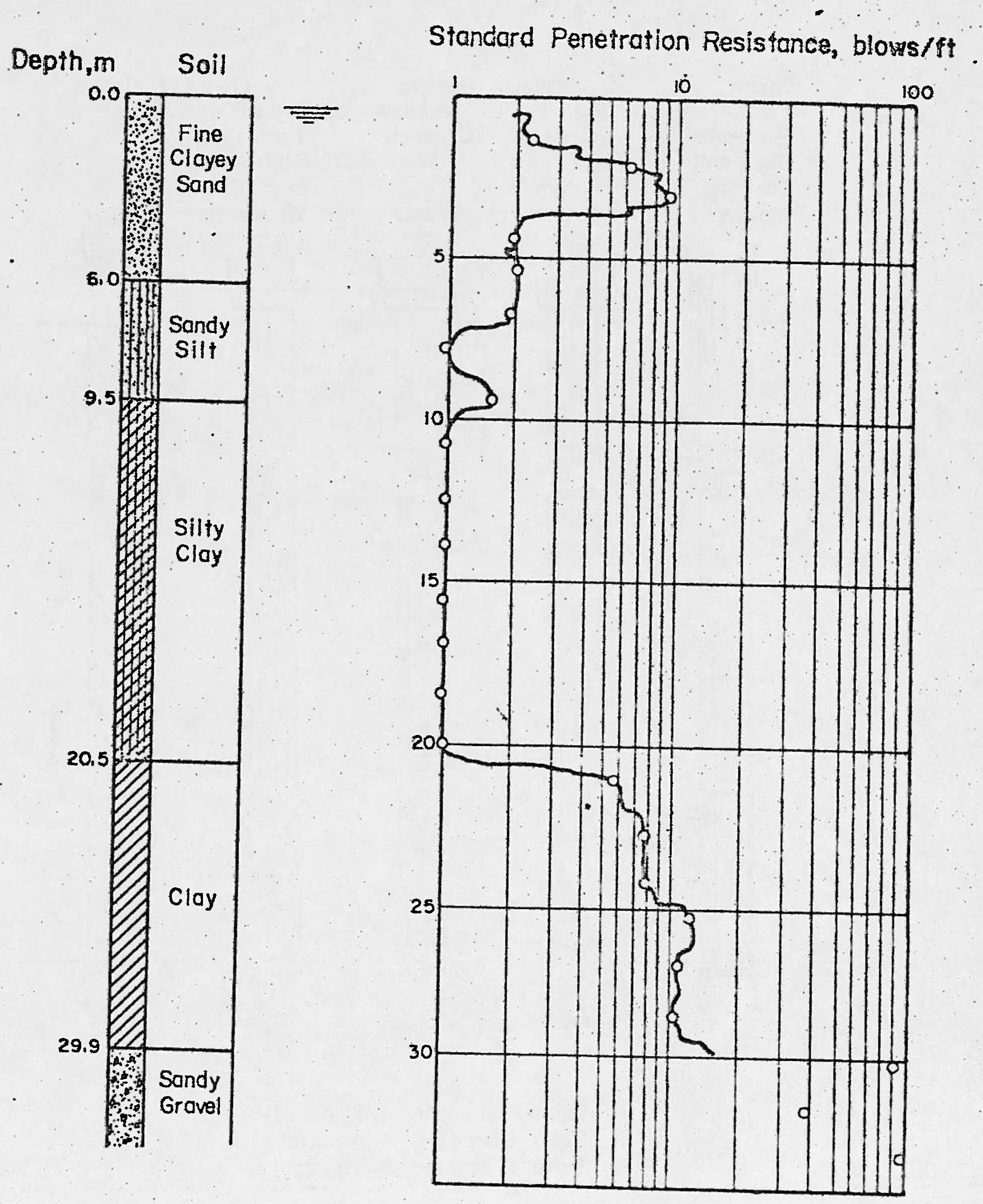
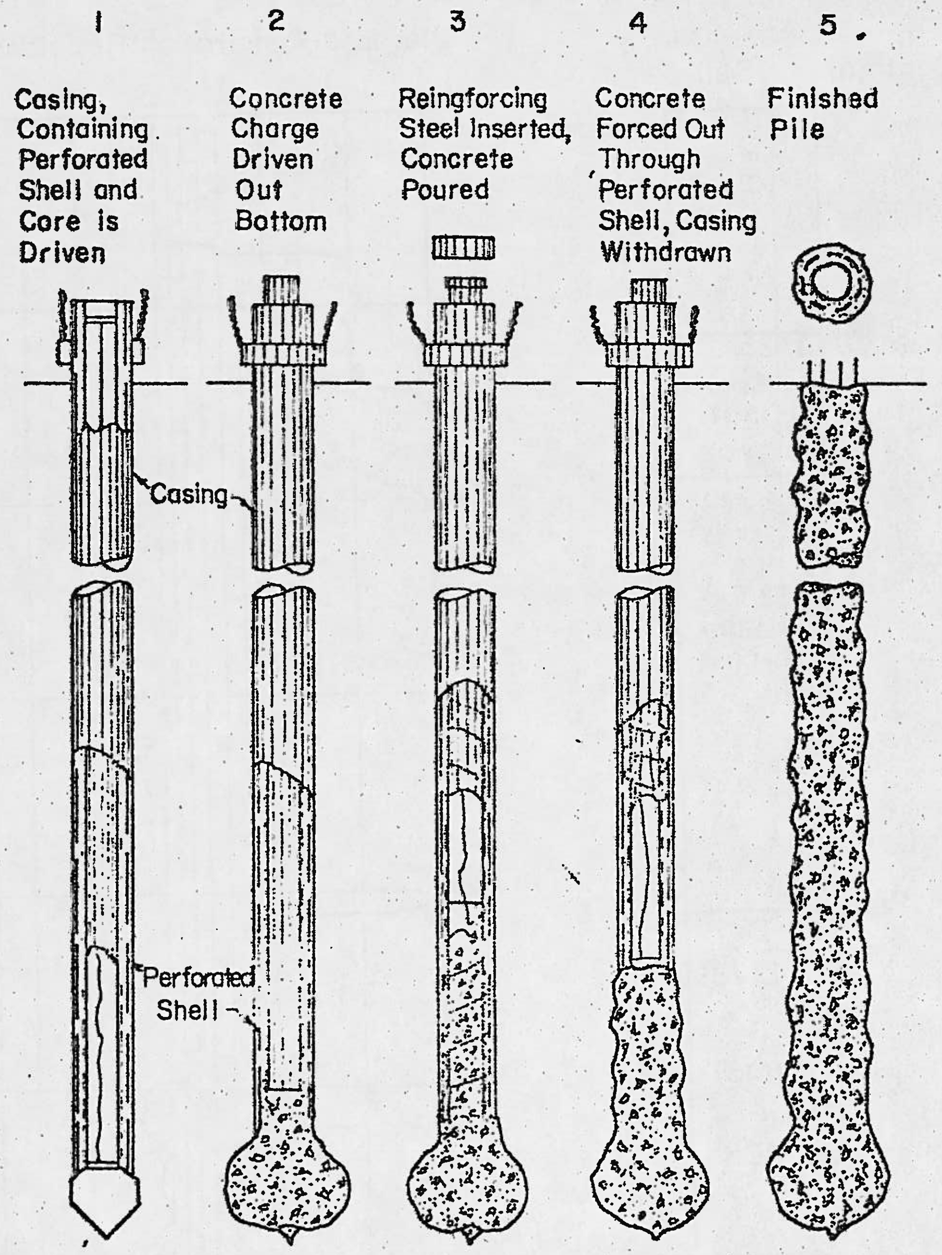


FIG. 8B SOIL PROFILE, TELEPHONE BUILDING
(After Hokugo)



Casing Diameter = 51 cm
Shell Diameter = 45 cm
Core Diameter = 35 cm

FIG. 8C SCHEMATIC DIAGRAM OF PEDESTAL PILE, CASE B8 (After Taiyo Corp.)

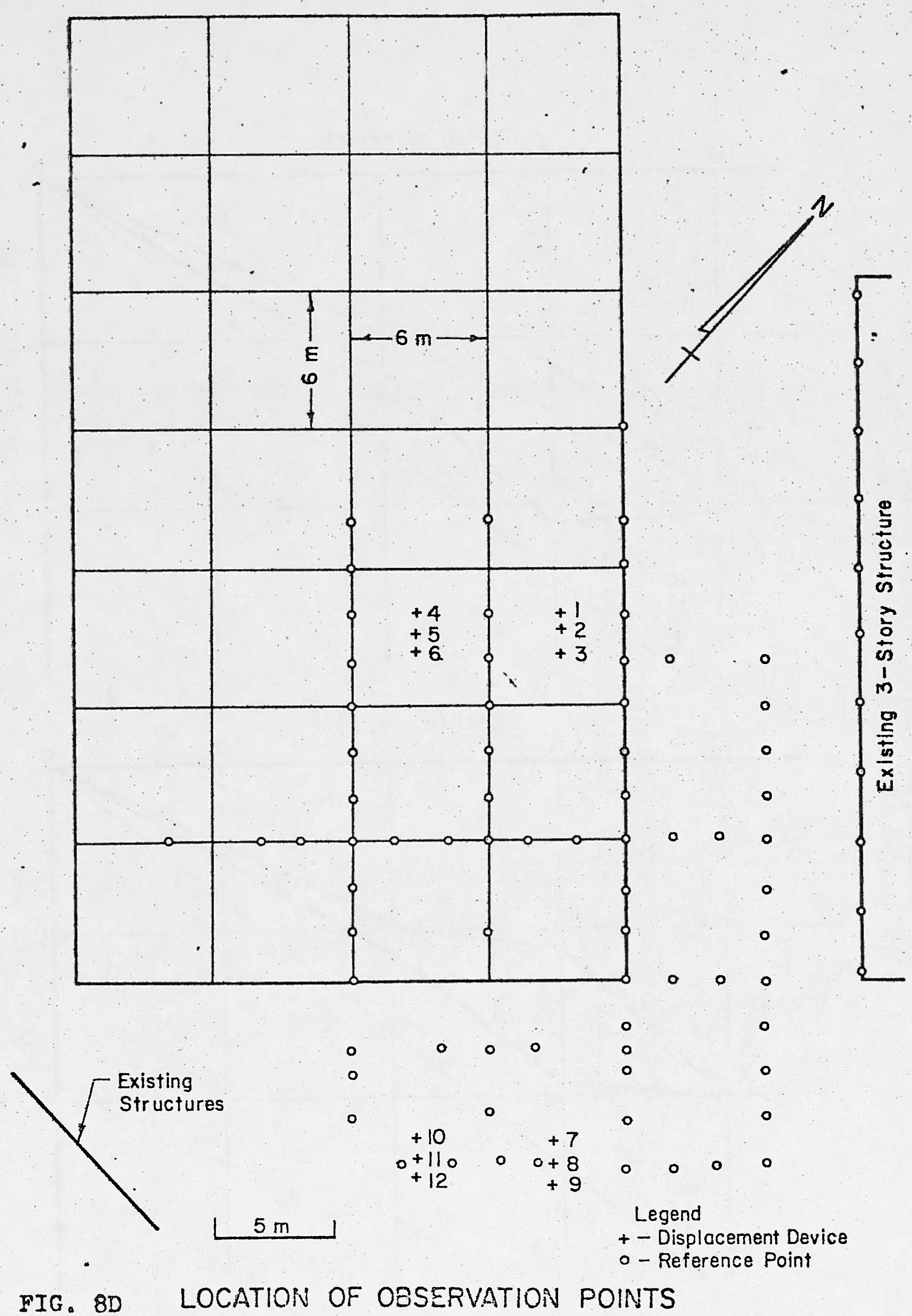


FIG. 8D LOCATION OF OBSERVATION POINTS
(After Hokugo)

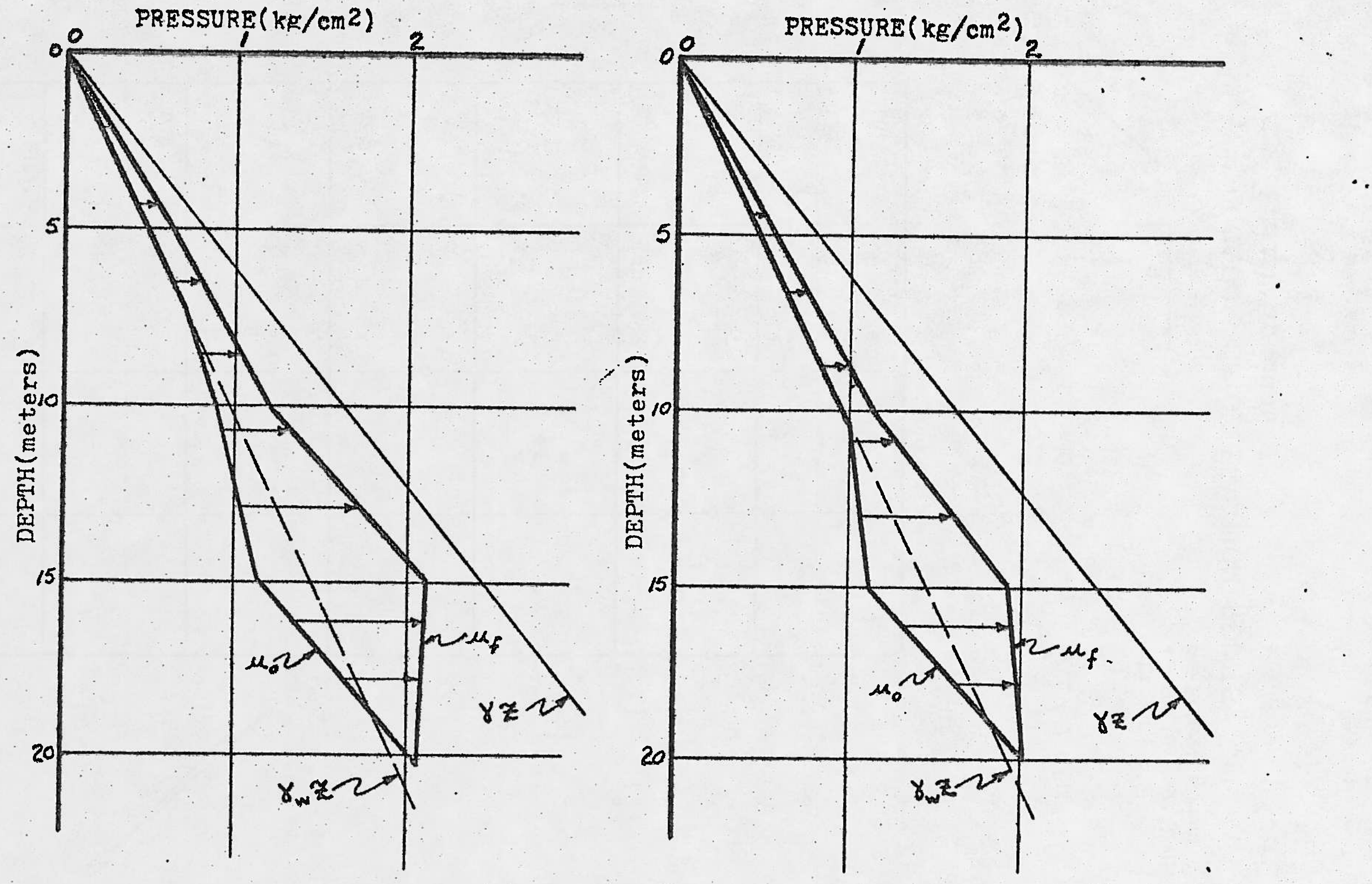


FIG. 8E. POREWATER PRESSURES GENERATED BY PILE DRIVING, TOKYO TELEPHONE BUILDING

(After Hokugo)

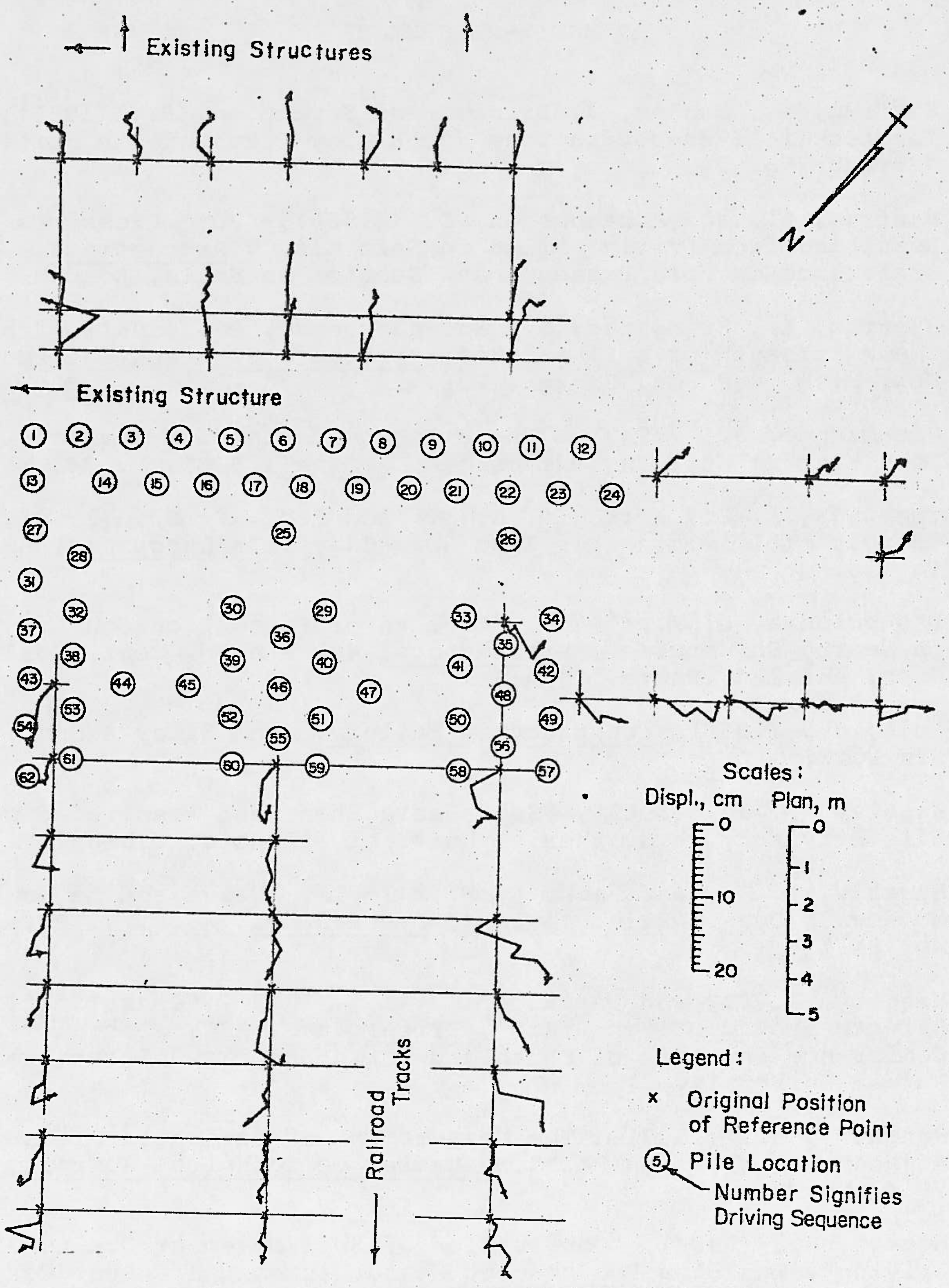


FIG. 8F HORIZONTAL DISPLACEMENT OF REFERENCE POINTS (After Hokugo)

APPENDIX-REFERENCES

Bjerrum, L., Hansen, J. Brinch, and Sevaldson, R., (1958), "Geotechnical Investigations for a Quay Structure in Horten", NGI Pub. No. 28.

Bjerrum, L., and Johannessen, I., (1960), "Pore Pressures Resulting from Driving Piles in Soft Clay", Proceedings, Conference on Pore Pressure and Suction in Soils, London.

Bjerrum, L., Kringstad, S., and Kummeneze, O., (1961), "The Shear Strength of a Fine Sand", Proceedings, Fifth Int. Conf. Soil Mech. and Fdn. Engrg., Vol. 1.

Casagrande, A., (1947), "The Pile Foundation for the New John Hancock Building in Boston", Journal, BSCE, V. 34, No. 4.

Cummings, A. E., Kerkhoff, G. O., and Peck, R. B., (1950), "Effect of Driving Piles Into Soft Clay", Transactions, ASCE, V. 115.

D'Appolonia, D. J.,(1971), "Effects of Foundation Construction on Nearby Structures", Proceedings, 4th Pan Am. Conf. Soil Mech. and Fdn. Engrg., San Juan.

Feld, J., (1968), Construction Failure, John Wiley & Sons, New York.

Hagerty, D. J., (1969), "Some Heave Phenomena Associated with Pile Driving", Ph.D. thesis, Univ. of Illinois, Urbana.

Hagerty, D. J., and Peck, R. B., (1971), "Heave and Lateral Movements Due to Pile Driving", Proceedings, ASCE, V. 97, No. SM 11.

Hagerty, D. J., and Garlanger, J. E., (1972), "Consolidation Effects Around Driven Piles", Proceedings, ASCE Specialty Conference on Perf. of Earth and Earth-Supported Structures, Purdue University.

Hanna, T. H., (1967), "The Measurement of Porewater Pressures Adjacent to a Driven Pile", Canadian Geotechnical Journal, V. 4, No. 3.

Hokugo, H., (1964), "Observation of Soil Movement Due to Pile Driving", Building Engineering News, Nippon T & T Pub. Corp., (in Japanese).

Holtz, W. G., and Lowitz, C. A., (1965), "Effects of Driving Piles in Lean Clay", Proceedings, ASCE, V. 91, No. SM 5.

Horn, H. M., (1966), "Influence of Pile Driving and Pile Characteristics on Pile Foundation Performance", Lecture Notes, Metropolitan Section Lecture Series, ASCE , New York.

Ireland, H. O., (1955), "Settlement Due to Building Construction in Chicago", Ph.D. thesis, Univ. of Illinois, Urbana.

Kenney, T. C., (1967), "Field Measurements of In Situ Stresses in Quick Clays", Proceedings, Geotechnical Conference, Oslo.

Kerisel, J., (1961), "Fondations profondes en milieu sableux", Proceedings, 5th Int. Conf. Soil Mech. and Fdn. Engrg., Vol. 2.

Klohn, E. J., (1963), "Pile Heave and Redriving", Transactions, ASCE, V. 128.

Koizumi, Y., and Ito, K., (1967), "Field Tests with Regard to Pile Driving and Bearing Capacity of Piled Foundations", Soil and Foundation, V. 3, No. 3.

Ladanyi, B., (1963), "Expansion of a Cavity in a Saturated Clay Medium", Proceedings, V. 89, SM 4.

Lambe, T. W., and Horn, H. M., (1965), "The Influence on an Adjacent Building of Pile Driving for the MIT Materials Center", Proceedings, 6th Int. Conf. Soil Mech. and Fdn. Engrg., V. 2.

Legget, R. F., (1950), Discussion-"Effect of Driving Piles Into Soft Clay", Transactions, ASCE, V. 115.

Lo, K. Y., (1961), "Stress-Strain Relationship and Porewater Pressures Characteristics of a Normally Consolidated Clay", Proceedings, 5th Int. Conf. Soil Mech. and Fdn. Engrg., V. 1.

Lo, K. Y., and Stermac, A. G., (1965), "Induced Pore Pressures During Pile Driving Operations", Proceedings 6th Int. Conf. Soil Mech. and Fdn. Engrg., V. 2.

Lynch, T. J., (1960), "Pile Driving Experiences at Port Everglades", Proceedings, ASCE, V. 86, No. SM 2.

Meyerhof, G. G., (1959), "Compaction of Sands and Bearing Capacity of Piles", Proceedings, ASCE, V. 85, No. SM 6.

Milligan, V., Soderman, L. G., and Rutka, A., "Experience with Canadian Varved Clays", Proceedings, ASCE, V. 88, No. SM 4.

Orrje, O., and Broms, B., (1967), "Effects of Pile Driving on Soil Properties", Proceedings, ASCE, V. 93, No. SM 5.

Peck, R. B., and Berman, S., (1961), "Recent Practice for Foundations of High Buildings in Chicago", The Design of High Buildings, Symposium, Univ. of Hong Kong.

Plantema, G., and Nolet, C. A., (1957), "Influence of Pile Driving on the Sounding Resistance in a Deep Sand Layer", Proceedings, 4th Int. Conf. Soil Mech. and Fdn. Engrg., V. 2.

Reese, L. C., and Seed, H. B., (1955), "Pressure Distribution Along Friction Piles", Proceedings, ASTM, V. 55.

Swiger, W. F., (1948), "Effect of Vibration on Piles in Loose Sand", Proceedings, 2nd Int. Conf. Soil Mech. and Fdn. Engrg., V. 2.

Terzaghi, K. T., and Peck, R. B., (1967), Soil Mechanics in Engineering Practice, John Wiley and Sons, New York.

Thornley, J. H., (1953), "The Mystery of the Restless Substratum", Engineering News-Record, New York, March 19.

Zeevaert, L., (1950), Discussion-"Effect of Driving Piles Into Soft Clay", Transactions, ASCE, V. 115.