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SYNOPSIS: As part of a new cogeneration plant being built in Jacksonville, Florida, a 31-foot deep excavation was required to install a 173-foot by 53-foot coal unloading structure in loose to medium-dense fine sands with shallow ground water. A conventional system of excavation support would typically consist of installing and maintaining a dewatering system and driving sheet piles. However, due to the potential for shallow contaminated ground water at the site and a restricted amount of inflow treatment capacity, a nearly complete cutoff or "bathtub" structure was required. A system that is relatively new to the United States was designed and installed to meet the difficult needs of the site. The system consisted of a sheet pile perimeter wall placed in a cement-bentonite slurry trench, tied back with soil anchors, in conjunction with an anchored six to eight-foot thick soilcrete base mat installed using jet-grouting techniques.

This case history provides details regarding design and installation of the anchored cutoff structure. Specifically, design assumptions regarding lateral earth pressures are presented along with predicted versus actual anchor loads for various construction stages. In addition, the results of finite element seepage analysis of the soilcrete base cutoff, and a unique hydrostatic uplift analysis are also presented.

INTRODUCTION

A new cogeneration plant is being built adjacent to an existing paper mill. This plant is to furnish power to the paper mill, as well as the surrounding area, and is being designed to burn coal transported in by rail.

The plant is located adjacent to a tributary of the St. Johns River near the port of Jacksonville. The Coal Unloading Facility is within 100 feet of the river, and groundwater investigations indicated contamination is present that would necessitate treatment if dewatering was attempted.

The design engineers made every effort to found all buildings, equipment, and utilities above the groundwater level, but the Coal Unloading Facility required a location 31 feet beneath the subsurface due to the railroad requirements and other logistical elements. A structure of this size (173' x 53') would typically require a substantial dewatering system in the permeable soils at this site, and the existing treatment system could handle only an additional 100 GPM.

The fast track nature of the design and construction, combined with the logistics required for construction management led to a request for suggestions from a specialty geotechnical contractor. This led to a design/construct contract which was based on meeting schedule commitments and performance of the structure for a lump sum price.

The first part of the project involved a detailed review of the subsurface investigations, specifically to evaluate the existence of an aquiclude within a reasonable depth. Since an aquiclude did not exist, the next course was to review the construction choices and economic impacts as well as the potential risks involved.

All systems considered had to conform to the following criteria imposed by the owner:

- No greater than 100 GPM groundwater withdrawal with owner furnished treatment.
- No re-injection of groundwater permitted.
- The plan dimensions of the temporary pit had to be five feet greater than the design dimensions of the finished structure.

These restrictions, combined with the site conditions, severely limited potential methods of construction, and led this design team to look into new technologies. The sidewalls of the pit would need to be strong, as well as of low permeability. With the soils at this site and the size of the pit, the bottom must also exhibit these characteristics.

SUBSURFACE CONDITIONS

Based on the results of a series of soil test borings (including standard penetration tests) and cone penetrometer tests, the subsoils consisted of medium dense to very dense fine sand to slightly silty fine sand. The upper sand interval was generally poorly graded and a trend of increase in density with depth was observed. A profile of the subsurface conditions in the project area is included in Figure 1.

A few anomalous or discontinuous zones were discovered in the site subsoils including loose sand zones, very dense cemented sand zones, and loose clayey sand/clayey silt zones. Specifically, a loose sand zone was encountered from elevation -5 to -10 feet, MSL in a number of the borings. In addition, as will be described in more detail in the construction portion of this paper, a very dense cemented sand zone was encountered beneath a portion of the site from elevation -15 to -20 feet, MSL. This zone had Qc values as high as 400 tsf. Finally, a well-graded loose clayey sand to clayey silt zone was found beneath a portion of the site about elevation -35 to -40 feet, MSL.

A very high groundwater table was observed during exploration and from subsequent piezometric readings. In general, the groundwater ranged from elevation 6 feet to 1 foot, MSL.

ENGINEERING DESIGN AND ANALYSES

As shown on Figure 1, the construction of the pit called for a combination of a variety of specialty construction technologies, including slurry wall construction, sheet-pile installation, jet grouting, and soil anchors. The material parameters used in the design and analyses of the support system are also listed on Figure 1. Because the excavation and installation of the temporary retaining structure were to be performed in stages, different methods of analyses were selected to best model each stage. Computer programs CSHTWAL (George, 1981) and CBEAMC (Dawkins, 1982) were used to analyze the retaining walls.

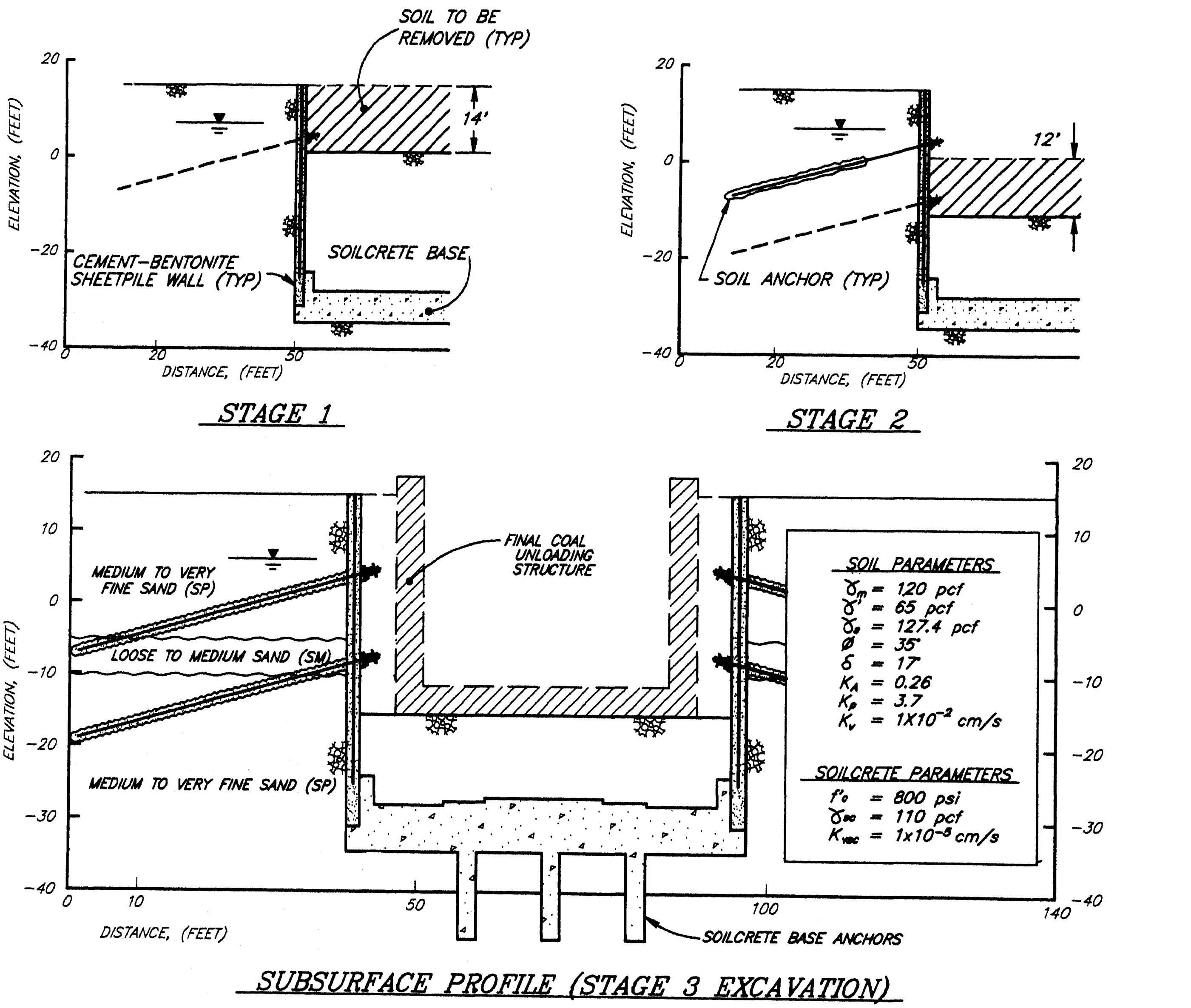

Figure 1. Construction sequence and subsurface profile.

A summary of the methods of analyses and description of the various stages is included in Table 1. The program results included estimated anchor loads, deflections along the wall, and maximum shear and bending moments.

TABLE 1. Summary of Methods of Analyses and Description of Stages

STAGE DESCRIPTION	PROGRAM/METHOD OF ANALYSES
Initial 14 ft. excavation	CSHTWAL-cantilever wall
Upper row anchors in place, second excavation to 26 ft.	CSHTWAL-anchored bulkhead
Two rows of anchors in place, excavation to 30.5 ft.	CBEAMC-multiple support (anchor) beam analyses

Based on the results of these analyses, we selected anchor spacings and capacity, sheet pile sections, and all other structural components (walers, braces, welds, etc.). In addition, the effects of a 100-ton crane surcharge were modeled using Newmark's 1942 lateral earth pressure charts.

Other analyses for the cutoff structure included seepage analyses and uplift analyses to design the soilcrete base mat. Seepage analyses were performed with computer program PCSEEP (Geo-Slope Int., 1987) which utilizes a finite element method to estimate inflow rates. Because the maximum allowable flow into the cutoff structure was 100 gpm, the program was used to estimate whether the proposed thickness and target hydraulic conductivity of the base mat could achieve this requirement. Once the model was established, permeabilities and thickness of the soilcrete could easily be varied. Using a minimum factor of safety of 10, the soilcrete properties and thickness were selected. To test the model, we analyzed the cutoff structure without the soilcrete mat, which resulted in an estimated inflow of approximately 800 gpm.

The final design and analyses performed for the cutoff structure involved estimating the ability of the soilcrete base mat to resist uplift forces. We modeled the mat as a uniformly loaded, one-way slab and estimated the moments along the span between the retaining walls. We then compared the maximum tensile bending stresses to the estimated tensile capacity of the soilcrete mass.

Due to the required safety factors for uplift and conservatively selected tensile capacity, our eventual design included a variably thick soilcrete mat (6 feet to 7.5 feet at the center of the span). In addition, to reduce the moments across the maximum span distance (53 feet) a series of interior mat anchors were installed which consisted of individual soilcrete columns extended below the level of the mat. The anchors were each reinforced with a No. 8 rebar.

SUPPORT SYSTEM DESCRIPTION AND INSTALLATION

The sequence of construction for the system shown in Figure 1 was as follows:

- A cement-bentonite (CB) slurry wall was constructed in panels to a depth of 46 feet, and interlocking steel sheet-piling was set to a depth of 40 feet prior to gel (initial set) of the slurry. This provided assurance of a continuous interlocked structural wall of very low permeability (less than 1×10^{-5} cm/sec based on core and wet samples).

- From original grade, a horizontal seepage barrier was constructed by triple-rod jet grouting across the entire pit base at a depth of 53 feet. In plan (Figure 2), this base slab consisted of an overlapping grid of soilcrete columns designed to be of very low permeability (less than 1×10^{-5} cm/sec) and high strength (average 800 psi unconfined compressive strength) based on actual core and wet samples, and had to be capable of resisting the uplift forces of the groundwater. To assist with this, uplift resistance was enhanced by the addition of vertical Soilcrete anchors (80 kips capacity), uniformly spaced to tie down the slab (see Figure 1). Along the edge of the pit, the Soilcrete mat extended to connect to the sheet-piling to offer additional lateral toe support to the wall.

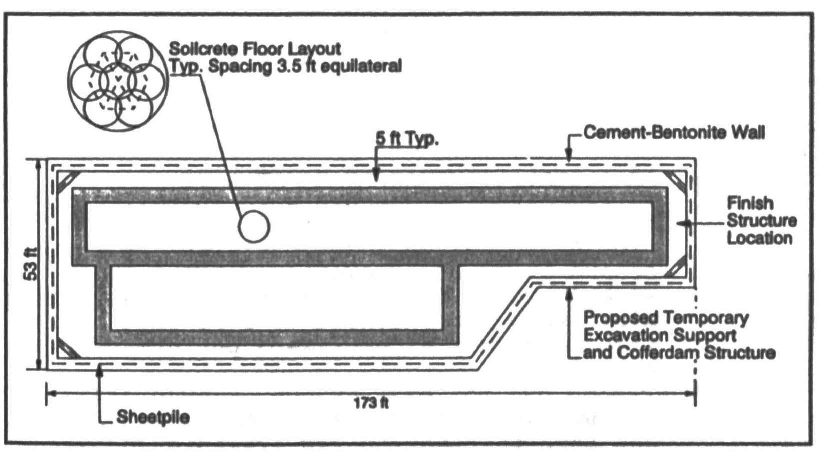

Figure 2. Site plan showing cutoff structure.

Figure 3. Installation of slurry trench and sheet-piles.

- Excavation of the pit then proceeded, stopping at two levels to install 142-kip soil anchors and wales to provide lateral support to the walls.

- Upon reaching the final excavation grade, a perimeter collection system of piping was installed to control any seepage into the excavation and maintain dry conditions for the final structure installation.

QUALITY ASSURANCE/QUALITY CONTROL (QA/QC)

The QA/QC program for the project considered details to assess adherence to design in all phases of the construction. A brief description of this effort follows:

1. <u>Cement-bentonite Wall Construction</u>
An automated, metered batching system was set up to prepare uniform and consistent slurry. The design mix consisted of 35 percent cement and 2.5 percent Hydrated Bentonite (by weight). The cement-bentonite grout had an average unconfined compressive strength of 80 psi in 28 days (see Figure 4). The design intent here was to achieve a strength equal to or greater than the adjacent soil with a permeability not to exceed 1 x 10^{-5} cm/sec.

   C-B slurry sampling was performed daily and cylinders were tested at frequent intervals early in the project to gain assurance that the design assumptions were met.

2. <u>Sheet-pile Installation</u>
Certificates of compliance were provided to attest to the material properties of the steel furnished. Interlocks were visually inspected to ensure that no flaws existed. Sheet-piling was installed in pairs and aligned before welding in place. Verticality and alignment are imperative when constructing any right-angle cofferdam.

3. <u>Soilcrete Construction</u>
Prior to any production jet grouting, a series of test sections were constructed to attest to the ability to construct the design geometry and quality. Since the Soilcrete was constructed at least 42 feet below working grade, excavation and observation were not feasible. In lieu of this, groups of three columns were constructed and a core was retrieved at the centroid of each group of Soilcrete columns. In any one group, the columns used identical jet grouting parameters and grout mix, with each column constructed on consecutive days (no two columns in any one group constructed during any single day).

   Based on visual observation of the cores retrieved and tested, the production jet grouting parameters were established. Due to the metered batching system, neat grout consistency was excellent, averaging 3750 psi in 28 days, with Soilcrete averaging over 1000 psi (See Figure 5).

   The drill rig used was specially built for jet grouting and had hydraulic controls and visual LED readouts for consistent real-time lift speed, rotation speed, and depth.

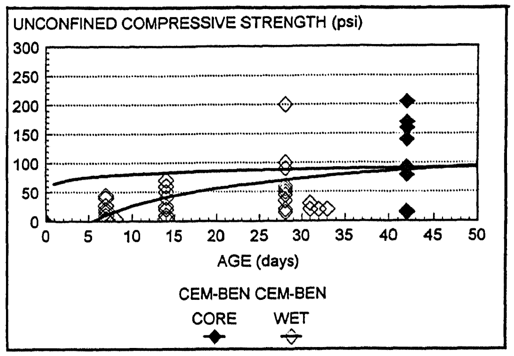

Figure 4. Cement-Bentonite strength data.

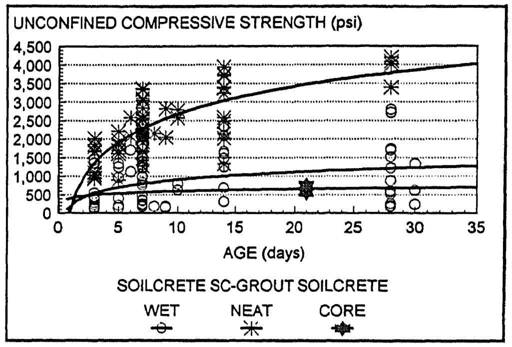

Figure 5. Soilcrete strength data.

   In order to assess Soilcrete quality during construction, a specially developed sampling device was used to retrieve wet samples immediately after column construction from any depth. Wet samples were then cast into cylinder molds for strength and permeability testing after curing. Additionally, in-situ piezometers were cast into the Soilcrete, and falling and rising head permeability tests were performed. Lastly, selected Soilcrete cores were retrieved from the interstice of production columns and tested for strength and permeability.

   Every Soilcrete column was documented for material quantities, time, and depth. Column layout was checked every day using permanently fixed batter boards and conventional surveying techniques. Since the working platform elevation varied from day to day, a laser level was used to calibrate the drill rods for each column.

4. <u>Soil Anchors/Movement Monitoring</u>
Thirty-six soil anchors were installed on this project after the first stage of excavation and an additional 104 after the second stage of excavation. All anchors were designed for a capacity of about 146 kips, were proof-tested to a minimum of 190 kips, and four anchors were performance-tested.

   Of the 140 anchors installed, only two did not pass the testing requirements and were replaced. In addition, over 26 lift-off tests were performed on select anchors along each face of the structure to make sure the soil anchors were not being overstressed. Comparisons of the measured anchor loads to the corresponding predicted values and lock-off values are included on Figure 6.

Flexible walls such as anchored bulkheads are expected to experience movement in order to develop active and passive pressure zones. Therefore, throughout the various stages of construction, movement monitoring of the anchored sheet piles was performed. Measured movements were variable, but generally did not exceed 1.0 to 1.5 inches maximum.

Finally, the measured seepage rates into the exposed excavation were very small (i.e., less than 5 gpm), and were primarily from water leaking through the sheet piles at the anchor locations.

wall excavation showed a high shell content and calcification. The 11-ton clamshell could not excavate this layer efficiently, causing the C-B slurry to partially gel by the time excavation was completed. This caused very long work shifts and short panel lengths. To increase the digging efficiency through this stratum, several claws were welded to the clamshell to serve as rippers on closing and were very effective.

Another problem, this one anticipated, was overcoming the water pressure when installing the bottom row of anchors. This lower row was 15 feet below existing groundwater and located in a strata of uniform, fine "running" sands. The problem was solved by using a pinch valve attached to the sheet-pile, through which all drilling and anchor grouting was performed. Upon completion, the valve was shut, and grouting behind the valve stopped any inflow of groundwater on removal of the valve a few hours later.

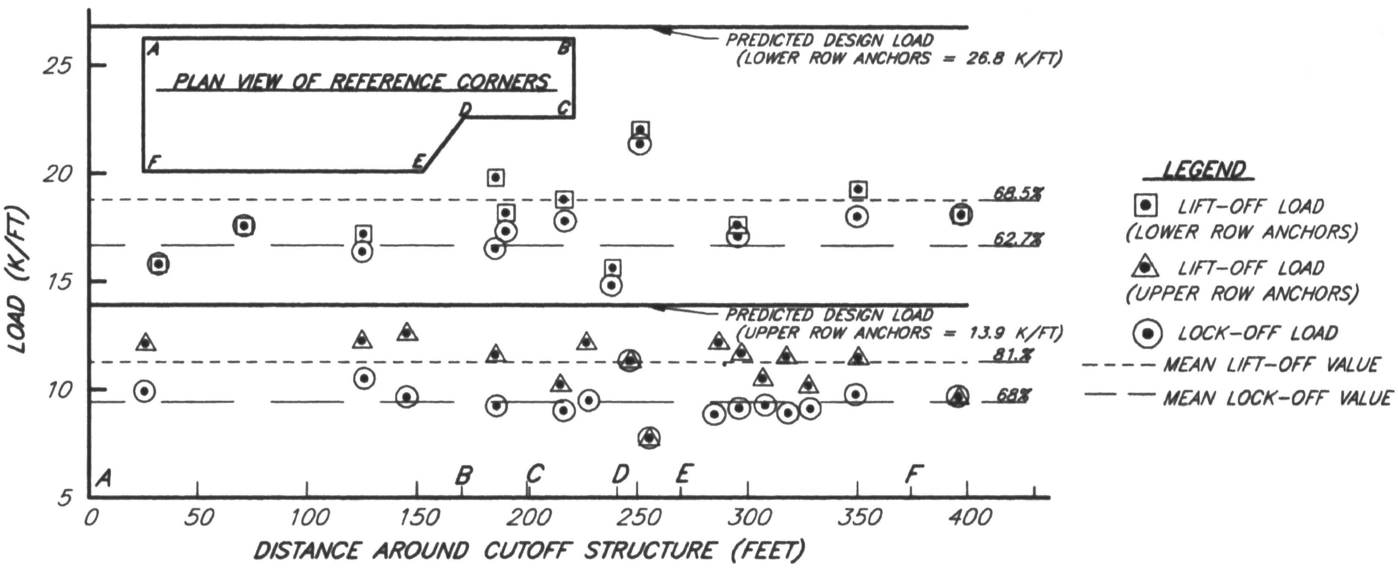

Figure 6. Summary of anchor testing.

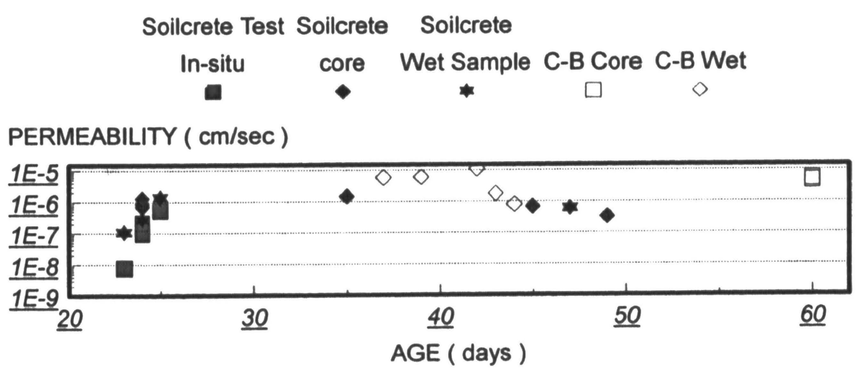

Figure 7. Summary of permeability testing.

Figure 8. Soilcrete cores.

CONSTRUCTION DIFFICULTIES

The first problem encountered was the existence of a discontinuous cemented sand stratum at a depth of about 40 feet. The field data interpreted this to be a dense sand layer, but

CONCLUSIONS

1. The soilcrete base mat in combination with the C-B sheet-piles provided a nearly complete groundwater cutoff.

2. The base mat could have potentially been redesigned for greater economy if less conservative parameters were selected for the uplift analyses or if an arch configuration was used to decrease bending moments in the span.

3. Using Rankine lateral earth pressures and modeling the anchored sheet-pile walls as simply supported beams provided a reasonably accurate estimation of anchor loads. This estimation became more conservative with depth, indicating that the use of apparent pressure diagrams may have been more reasonable for the lower row of anchors.

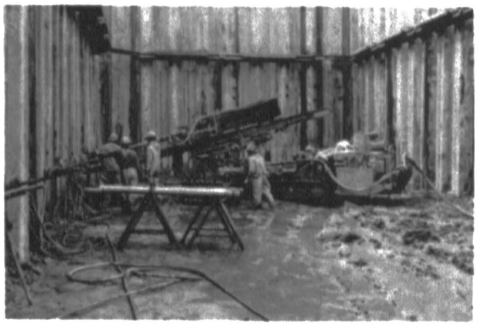

Figure 9. Drilling soil anchors below the water level.

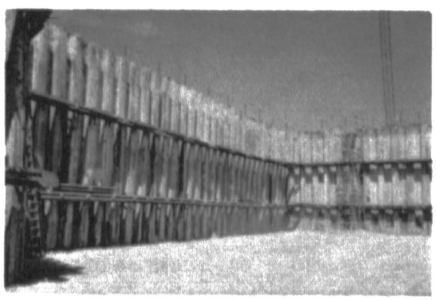

Figure 10. Interior of completed cutoff structure.

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CONVERSIONS

1 inch = 2.54 centimeters
1 foot = 0.3048 meters
1 pound = 0.4536 kilograms
1 gpm = 6.309×10^{-5} m^3/s