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DESIGN, CONSTRUCTION AND PERFORMANCE
OF A DEEP CIRCULAR DIAPHRAGM WALL,
STATEN ISLAND, NEW YORK

by

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1. BACKGROUND

The new Richmond Avenue Pump Station on Staten Island, New York, is an integral feature of the Borough's Oakwood Beach Water Pollution Control Project (City of New York, Department of Environmental Protection Contract T-7A Foundation). It was to be built within a circular structure, approximately 53' in diameter, with a base about 90' below normal ground elevation. The shaft was designed to tie into new 48" diameter influent and effluent sewer lines (Figure 1). By specification, the primary shaft forming method was not permitted to act as the permanent exterior structural wall of the Pumping Station. The shaft is located within two blocks of the ocean, and so the fine grained sediments through which it was to be constructed were saturated to within 15' of the existing ground surface.

The hydrogeology and the heavily urbanized nature of the neighborhood basically precluded the use of areal dewatering as a safe, reliable or economic means of constructing the shaft with conventional beams and lagging "in the dry". Following detailed review of several other construction options, including driven sheet piles and freezing, the Contractor, John P. Picone, Inc., selected a scheme proposed by Nicholson Construction featuring the use of a structural diaphragm wall to be constructed by the slurry trench method. Given the need to avoid "blow in" of the silty fine sands at the designed subgrade Elevation of -71*, and generally reduce seepage into the shaft excavation, options were examined to extend a cut off below subgrade

* Footnote: Elevations refer to Borough of Staten Island sewer datum which is 3.19 ft. above USGS Mean Sea Level (Sandy Hook, NJ).

elevation into a relatively impermeable hard silty clay, at Elevation -155'. Again, various schemes, including different types of grouting, were examined, but the most suitable option in this instance was simply to continue the diaphragm wall - unreinforced - down into the target stratum. This decision was further influenced by the dewatering difficulties experienced by other contractors working on adjacent contracts within the scheme, who had not opted for this positive deep cut-off solution for shaft construction.

As a consequence, the diaphragm wall ultimately executed by Nicholson Construction was 52' 6" in internal diameter, 165' deep, and 2' thick, making it, at the time, the deepest such structure installed by conventional diaphragm wall equipment in the United States. To assist with the interpretation of the geotechnical aspects and with the structural design, Nicholson Construction employed the services of Mueser Rutledge Consulting Engineers.

This paper summarizes the design, construction and performance of the wall and the subsequent excavation. This case history is presented as an encouragement to others to consider the technique in similar work on this scheme and in other applications in comparable hydrogeological conditions.

2. SITE CONDITIONS

2.1. Surface Access

As shown in Figure 1, the site was extremely restricted for work of this type, bearing in mind the space needed for the equipment and materials of construction, including reinforcing steel storage and cage fabrication areas. In addition, strict environmental controls were placed on the handling, transport and disposal of excavated debris and waste bentonite slurry.

To facilitate excavation operations, the ground level immediately around the shaft was excavated from the normal Elevation of +19' to about +12.5', and crushed rock placed as an operating platform. This level was consistent with

the requirement to have a final top of wall elevation, after removal of guide walls and bentonite contaminated concrete, of +10'.

2.2. Subsurface Conditions

The interstratum boundaries dipped very gently across the site but the following major units were recognized in the site investigation. The uppermost stratum (G-1) is Recent glacial, whereas the bulk of the sequence comprised Cretaceous alluvials and lacustrines. STP values relate to a 140 lb. hammer and a drop of 30".

El +10' to av +5' (G1) - Medium compact red-brown silty fine to medium sand, trace gravel, roots (SM). SPT 13-28 (Av 19), Water content 18%.

El Av +5' to -95' (C2) - Loose to very compact yellow, gray, tan, red and brown, fine to medium sand (SP, SM) trace silt, occasional silty clay layers (ML), stiff, grey, trace lignite. Very occasional micaceous, hard, black, white, silty clay (CL). Very occasional soft grey silty clay (CM).
SPT for sands 28-114 (Av 74), for others 5-22 (Av 13), water content 17-29%.

El -95' to -120' (C1) - Compact to very compact, silty or clayey yellow, brown and grey, fine, medium sand (SP, SC, SM), occasional silty clay layers, medium sand (SM). SPT values 57 to 114 (Av 81), water content 20-22%.

El -120' to Av -130' (C2) - Light grey, fine, medium sands, trace silt, trace clay (SP, SM, CL). SPT from 46-70 (Av 53), water content 22%.

El Av -130' to Av -155' (C1) - Grey, fine-medium sands, trace silt, clay (SP, SC, SM). Occasional seams of dark grey clay and silty clay (CL). Overall SPT values 36-85 (Av 56), water content 24-29%.

Below Av -155' (C3) - Hard to very hard, dark grey, silty clay (CL), trace silt, water content 24-26%.

The wall was designed to toe 2-5' into the hard, impermeable C3 stratum. During excavation, the ground water level averaged about El -15' but with a 2-3' variation, perhaps tidal, perhaps due to the adjacent dewatering efforts of another shaft contractor. The silty sands generally had in situ permeabilities (vertical) in the range of 8×10^{-4} to 3×10^{-6} cm/sec, based on laboratory testing of Pitcher barrel "undisturbed" samples. Values of 2×10^{-3} to 2×10^{-4} cm/sec were calculated from field falling head tests in piezometers (horizontal), and 2×10^{-2} cm/sec was estimated from an analysis of drawdown data on adjacent contracts. However, the piezometer test data were affected by dewatering within 200', while the dewatering analysis was equally influenced by the proximity of the ocean.

3. DESIGN CONSIDERATIONS

3.1. General Principles

The structural design considered the behavior of the shaft in two cases:

- as a perfect cylindrical shell
- as a discrete segment cylinder as a result of misalignment of up to 6" at the interpanel joints.

Longitudinal (vertical) and transverse (horizontal) bending were examined.

In the first case, the wall was assumed as constructed perfectly aligned and without ring beams for internal bracing. The structure was analyzed as a cylindrical shell under axisymmetric loading (triangular distribution). Both ends of the shell were assumed free and K_0 was used to compute soil pressure. ($K_0 = 0.5$ to a depth of 70' and 1.0 below 70'). The maximum displacement was computed as 0.1", too small to mobilize the soil into a K_a situation. The longitudinal bending moment associated with this case was small, as expected, since arch action in the shell resists load in ring compression. This case did not govern.

The second case considered the possibility of significant panel misalignments and the need to resort to ring beams for supplemental bracing. A panel was assumed to be misaligned to the extent that arch action was no longer sufficient to take the load in ring compression. In this analysis, the panel was analyzed as a continuous beam with ring beams as internal supports. Contrasting with the first case, K_a was assumed since displacements would be larger. This case governed the design but did not have to be constructed.

In the transverse analysis, a 6" maximum horizontal misalignment of adjacent panels was allowed. The section was designed as a column with the ring compression being the axial force and bending moment induced by the 6" eccentricity. Reinforcement was, therefore, designed to withstand the horizontal bending from the misalignment at joints and the vertical bending of panels with ring beams (conservatively). Since external pressure, and so ring compression and moment, varied linearly with depth to subgrade, the density of horizontal reinforcing varied accordingly. As a result of these reinforcing designs, based on reasonable magnitudes of misalignment, ring beams were not required, except for the one to stabilize the very top of the shaft.

Two dimensional flow net analyses were initially conducted to estimate ground water pressures and gradients acting on the shaft during construction. These indicated that for adequate safety against base upheaval, boils or piping, the toe of the wall had to reach El -130' to eliminate the need for continuous dewatering. However, the anticipated presence of cleaner, more permeable sands at El -120' to -135', and the proximity to the ocean, later caused concern that higher groundwater pressures than predicted by the general analysis would, in fact, occur. For practical security and economic reasons, it was therefore decided to continue the diaphragm wall cut-off to approximate El -155' and so into the effectively impermeable C3 material.

An uplift stability analysis was also conducted on the base slab. Prior to the base slab concrete reaching an Unconfined Compressive Strength of 3000 psi, the groundwater within the shaft had to be below the gravel drainage layer (El -73'). Thereafter, hydrostatic uplift forces corresponding to a

water level of El -55' could be resisted before a factor of safety of 1.05 on the weight of the base slab would be exceeded. Hence a standpipe exiting at this elevation was requested: overflow from this pipe would merely add weight to the structure and so raise overall stability. Full hydrostatic pressure could be resisted with a factor of safety of 1.1 when the outside walls reached El -11' and interior construction reached El -46'. Upon completion of the whole structure, the factor of safety against uplift is 1.4 on structural weight alone, but 2.9 assuming side friction.

3.2. Details of the Panels

The layout of the wall is shown in Figure 2. It consisted of seven panels, interconnected by 165' long W 24 interlock beams. Each panel was heavily reinforced to a depth of 107' with steel cages comprising #8 horizontal bars and #6, 7, and 8 vertical bars.

4. CONSTRUCTION

4.1. Foreseen General Sequence of Shaft Construction

This was divided into six stages:

I. Preparation

1. Excavate to site working level (approx El +12.5') and place crushed stone as a working platform.
2. Layout center of shaft and guide walls.
3. Excavate and install guide walls.

II. Construct Diaphragm Wall

1. Excavate Primary panel.

2. Maintain bentonite slurry in excavation with top of slurry at El +7' or higher.
3. Desand slurry at completion of panel construction.
4. Fabricate reinforcing cage and connect to interlock beams.
5. Install reinforcing cage and interlock beams as a unit.
6. Displace bentonite slurry continuously with concrete tremied from bottom of panel, with tremie outlet at least 8' below slurry interface.
7. Excavate and cast opposite and remaining two Primaries in sequence indicated in Figure 2.
8. Construct Secondary panels (4 each), allowing adjacent Primaries to cure a minimum of two days before excavation. Install cage to a "right fit" between existing interlock beams. Secondary panel S4 requires one interlock beam installed with the cage, however.

III. Control Groundwater

1. Install and develop deep well, minimum 5" i.d., to El -180', at position shown in Figure 1. Install submersible pump.
2. Install piezometer (NCC 3) within shaft as shown in Figure 1 to El -90'. (Note: Both of these to be sealed through subsequent shaft base.)
3. Test effectiveness of the wall as a cut-off by activating the pump and monitoring drawdown in the shaft. Determine if daytime pumping can maintain groundwater in shaft continuously below target piezometer level. If not, increase number of wells and pump capacity. (Note: This step may be executed as part of shaft excavation, Step IV, 3, below.)

IV. Excavate Shaft

1. Remove interior guide wall.
2. Excavate soil within shaft, taking care to preserve both well and piezometer casings to surface.
3. Continuously maintain groundwater within the shaft below excavation level.

V. Construction of Base Slab

1. Continue excavation to subgrade at El -73'.
2. Place filter fabric on subgrade and install clean porous crushed aggregate drainage layer on fabric and adjacent to well screen. Place plastic vapor barrier membrane over drainage layer and cast concrete mudmat.
3. Cast 7' thick base slab of permanent structure with the thickened perimeter keyed over two successive intervals by 4' into the diaphragm wall. Allow slab to cure to a minimum of 3000 psi while maintaining groundwater below El -73. Extend 5" well casing standpipe up to El -55', above slab, and 2" riser pipe for piezometer to El -45'.

VI. Construct Permanent Structure

1. Limit uplift pressures on base slab by allowing water to rise in standpipe and flow into shaft if appropriate. Dewater shaft as needed, but not less than daily. Operate deep well during the day to maintain water pressure on the slab to a pressure no higher than a head at El -55', as measured by riser pipe.
2. Construct pump station walls, followed by interior, while continuing Step VI, 1, above.
3. After outside walls of pump station reach at least El +11', and interior construction has reached at least El -46', the piezometer, riser pipe, well and standpipe may be sealed, allowing full hydrostatic pressure to develop on the base of the slab.

4.2. Details of the Diaphragm Wall Construction

The guide walls were of standard design and composition, being individually 3' deep, 18" wide and reinforced (Photograph 1). They were internally spaced to permit close tolerance for the 2' wide wall.

A 150 ton capacity 4000 Manitowoc crane was used to operate the Keller mechanical excavating grab (Photograph 2). This grab was extended by guides

to a length of 32' to aid verticality control and weighed about 14 tons. It was capable of a 9' 1" bite in one pass. Measurements were made every 10' of excavation depth of the position of the suspension cables relative to reference points on the guidewalls as a control over possible panel deviation. In addition, at least twice in each panel, any tendency for panel deviation or twisting was more closely measured using piano wires attached from the ends of the grab. A tolerance of 1% of verticality and 6" horizontal misalignment was permitted. Bentonite solution was prepared in a jet mixer before being pumped to storage tanks or ponds. Typically the specific gravity was about 1.02, the pH 8, and the Marsh Cone reading about 42 seconds. Panels were excavated in three bites per panel, with the middle section being about 30' above the edge bites. Throughout excavation, production rates were considerably lower than what had been anticipated on the basis of the geological logs, with the sands resisting removal more like a very dense glacial till. Average excavation rates in the upper 80' were reduced by a factor of 5 from 80-120' and by 2 below 120'. The grab's teeth were modified to improve "bite" efficiency. The more significant clay seams encountered only at 145' and 157' proved relatively easy to excavate. There was minor loss of bentonite slurry throughout the job, felt to be due to hydraulic gradients caused by adjacent local dewatering. Tendencies for panels to deviate were readily corrected by use of a specially designed chisel.

Desanding of each panel took about 10-25 hours. This was accomplished at three plan positions per panel and featured a double tube pipe system providing vigorous agitation and scouring of the base and removal of the suspended sands (from 16% by slurry volume to less than 2%) before concreting.

The interlock beams, 165' long, were WF 24x68 sections in three pieces, bolted or welded together. They conformed to ASTM-A36. The steel cages, 117' long, were fabricated out of Grade 60 rebar in two sections on site. A total of 192 tons of reinforcing steel was used, conforming to ASTM-A615. A special lifting bed was designed and built to handle the cages on account of their complex three dimensional shapes. Wood and styrofoam blockouts were attached to the cages to form the subsequent keyways in the exposed wall (Photograph 3).

The concrete was supplied by ready mix trucks and had a maximum aggregate size of 3/4", a target 28-day U.C.S. of 4000 psi, a slump of 7" and an air entrainment of 4%. Typical mixes had about 700 lbs. of cement, 1300-1400 lbs. fine aggregate (natural sand), 1650-1675 lbs. coarse aggregate (crushed rock) and 33-36 gallons of water per cubic yard. Various types and proportions of additives were used to entrain air and to retard/plasticize, although subsequent mix properties were very sensitive to minor variations in additive dosage. Seven-day tests gave cube strengths of 4500-5500 psi, increasing to 5500-7200 psi at 28 days. It was tremied via two 10" pipes in each panel at an average rate of 40 yds³/hour (Photograph 4). The overpour on the job was around 15%.

After completion of the wall, and when excavation had progressed 6' down, a 2' 6" deep compression ring beam was cast to secure the stability of the upper part of the excavation.

5. PERFORMANCE

Prior to excavation, the dewatering pump was activated. However, this had run for only a day or two before it "ran dry", highlighting the watertightness of the structure.

Excavation proceeded smoothly to subgrade and the gravel drain, mudmat and overlying structures were placed without incident. Pumping was occasionally done during excavation for about 1-2 hours every few days. The soil at subgrade was reported dry without noticeable seepage through the soil (with the well inoperative).

The Primary panels were found to be completely vertical with the Secondaries out by 4-6", as measured at the panel centers. The ends of the Secondaries were, on average, slightly further out: the beams were square to the Primaries, and this must have made the keying of the Secondaries much more difficult during excavation.

The keyways lined up well except at the P2-S5 joint where there was a 2' difference in elevation. Two small water seepages were noted due to local joint misalignment (S5 and P2) and minor concrete segregation (P1) respectively,. These totalled 2-5 gpm and proved no problem during subsequent construction. After excavation, the dewatering well was operated about once per week, for a few hours, until it was effectively "dry". Photographs 5 and 6 show the completed excavation.

6. FINAL REMARKS

For this 53' diameter shaft in difficult ground conditions, the decision was made to use a circular, 2' thick, cast-in-place, reinforced concrete wall constructed by the slurry trench method. Design requirements related to base stability and water inflow dictated the extension of the wall to over 80' below the subgrade elevation of the permanent structure to a total depth of 165'.

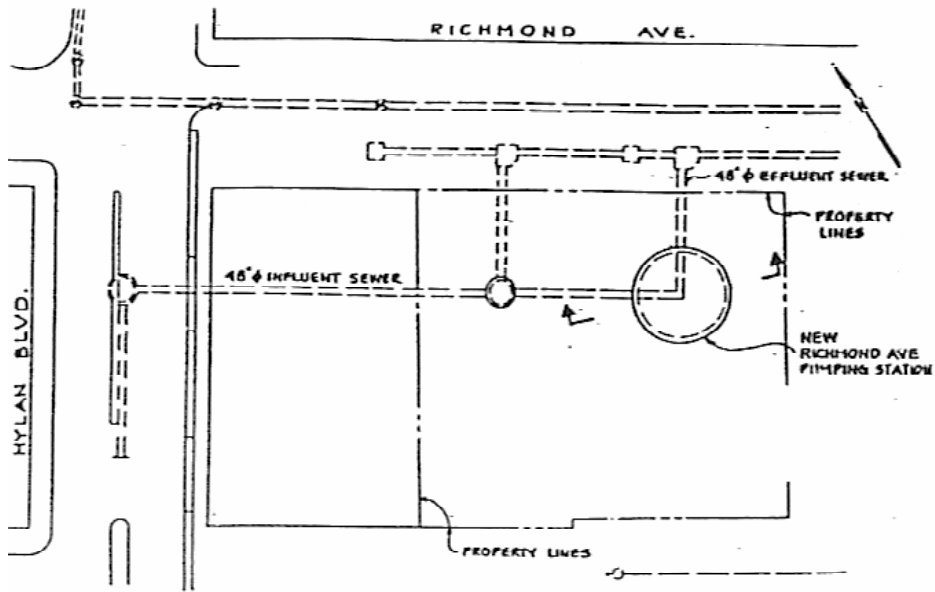
The shaft was completed despite adverse soil and hydrodynamic conditions, and the tight restrictions of panel geometry and site accessibility. Subsequent excavation confirmed the acceptability and quality of the structure apart from two minor seeps related to joints. The permanent structure was then built without incident within the excavation without the need for additional internal bracing.

The success of this project was further emphasized in comparison with the severe difficulties encountered by other contractors on similar jobs in the vicinity. This contract confirms the viability of deep diaphragm wall construction to significant depths in such soil conditions.

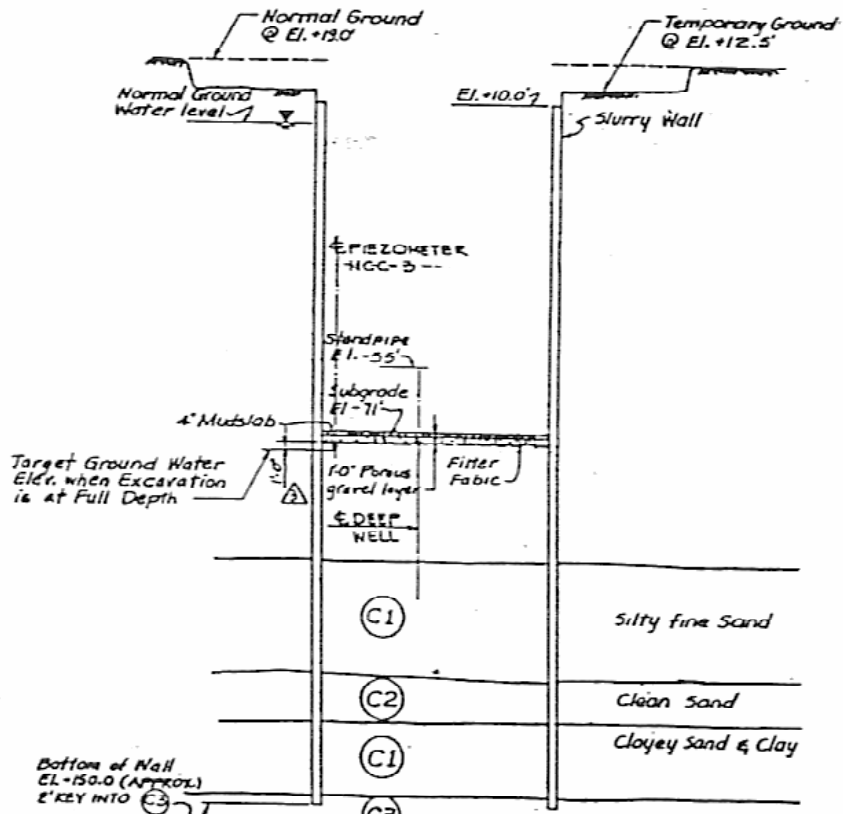
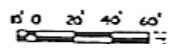
ACKNOWLEDGEMENTS

The authors are pleased to acknowledge the assistance of the following organizations, who also gave permission to publish this paper:

- City of New York Department of Environmental Protection
- John P. Picone, Inc.
- Mueser Rutledge Consulting Engineers
- Nicholson Construction



PLAN



GENERAL STRATA DESCRIPTIONS

- (G1) MEDIUM COMPACT RED-BROWN SILTY FINE TO MEDIUM SAND, TRACE GRAVEL, ROOTS.
- (G1) STIFF RED-BROWN CLAYEY SILT, SOME FINE MEDIUM SAND, TRACE GRAVEL.
- (C1) COMPACT TO VERY COMPACT SILTY OR CLAYEY YELLOW, BROWN AND GRAY FINE SAND, OCCASIONAL SILTY CLAY LAYERS, MEDIUM SAND.
- (C2) LOOSE TO VERY COMPACT YELLOW, GRAY, RED AND BROWN FINE TO MEDIUM SAND, TRACE SILT, OCCASIONAL SILTY CLAY LAYERS.
- (C1) HARD DARK GRAY SILTY CLAY.

GROUND WATER CONTROL METHODS
NO SCALE

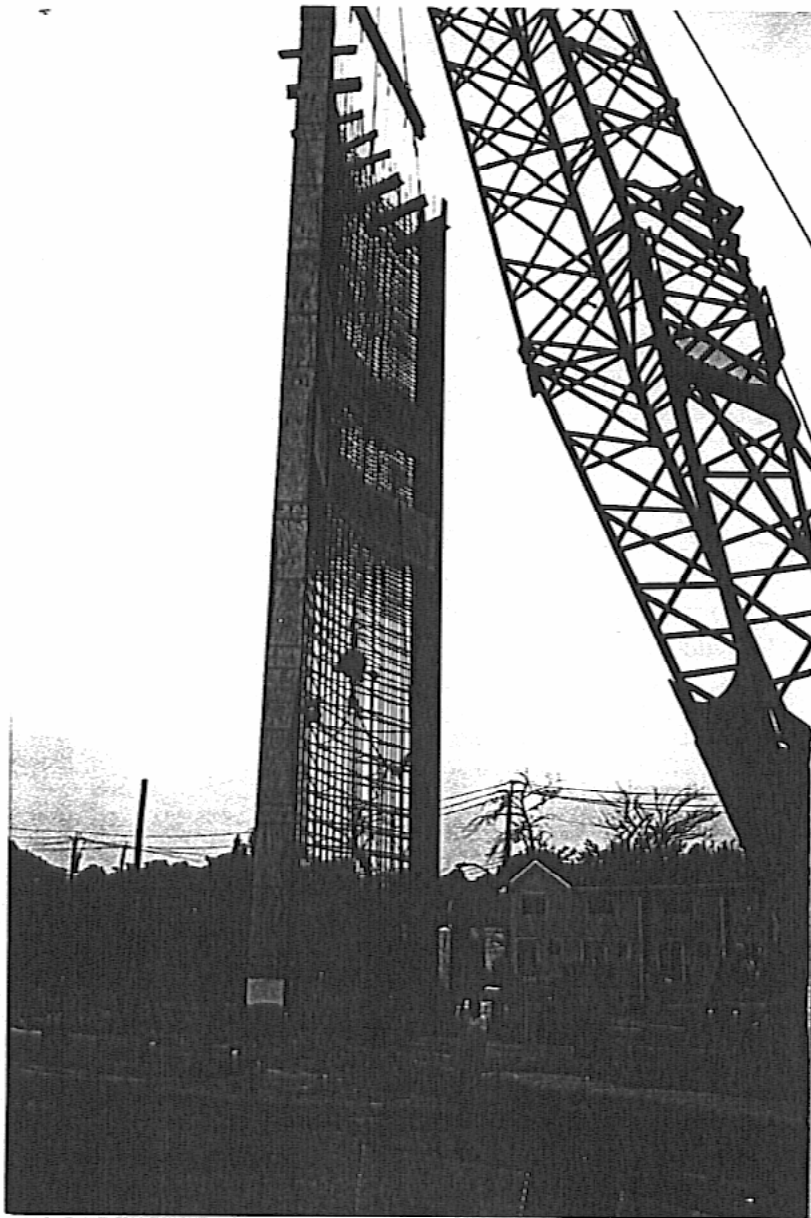
FIGURE 1



Photograph 1. Guide walls, prior to slurry wall excavation



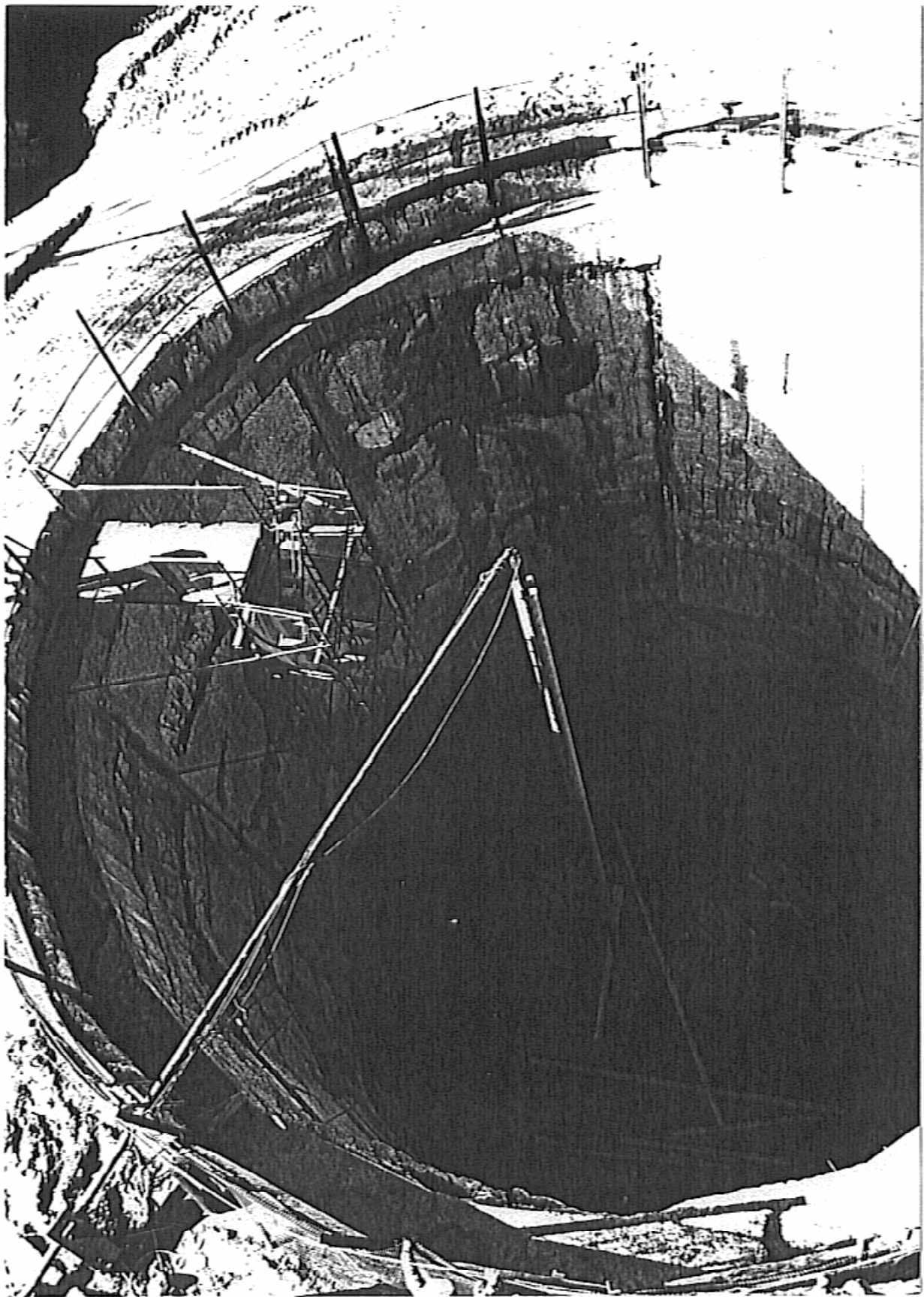
Photograph 2. Cable suspended excavating grab, showing long frame extension.



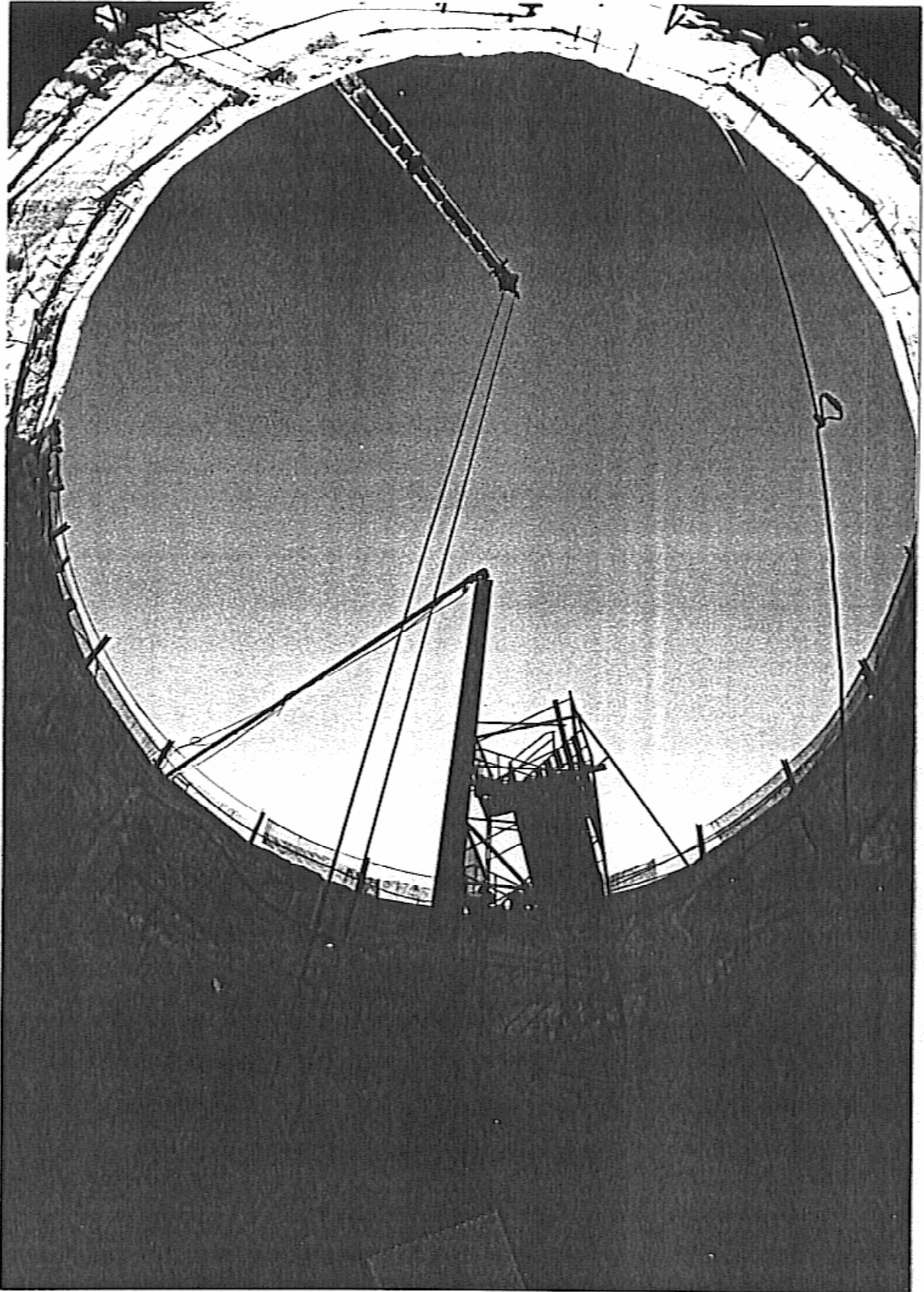
Photograph 3. (Left)
Handling the reinforcing
cage and transverse beams,
wood and styrofoam for
blockouts.



Photograph 4. (Below)
Tremie concreting a
Primary panel using two
lines.



Photograph 5. View of shaft after excavation showing dewatering well.



Photograph 6. Reverse view of shaft
after completion.