# DESIGN, CONSTRUCTION AND PERFORMANCE OF A DEEP CIRCULAR DIAPHRAGM WALL

by

Dr. Donald A. Bruce, Patrick H.C. Chan

and George J. Tamaro

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NICHOLSON CONSTRUCTION

P.O. Box 308

BRIDGEVILLE, PA 15017 (412) 221-7977

Donald A. Bruce, Patrick H. C. Chan and George J. Tamaro

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**ABSTRACT:** The new Richmond Avenue Pump Station in Staten Island NY was successfully constructed to a depth 27m below existing grade within a 16.5m circular, 50m deep slurry wall cofferdam. Construction was carried down through water bearing sands with minimal dewatering of the interior of the cofferdam and with no distress to the surrounding residential structures. This paper describes the design, construction and performance of the slurry wall and the general excavation.

**KEYWORDS:** Slurry Walls, Cofferdams, Pump Stations, Excavation, Dewatering

## INTRODUCTION

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). The permanent shaft was built within a circular reinforced concrete slurry wall structure, approximately 16m in diameter, with a base 27m below normal ground elevation. The shaft connects new 1.2m diameter influent and effluent sewer lines (Figure 1). The specifications did not permit the temporary slurry wall cofferdam to be utilized as part of the permanent structural wall of the Pump Station.

The shaft, located within two blocks of the ocean, was constructed through fine sandy soils with the natural ground water level found within 4.6m of the existing ground surface. Dewatering operations associated with other adjacent sewer construction provided some relief of water head during construction. The close proximity of a residential neighborhood and the effects and cost of deep prolonged pumping increased the risks associated with dewatering necessary for the construction of the shaft 'in the dry' using conventional soldier beams and lagging. Following detailed review of several construction options, including driven sheet piles and ground freezing, the General Contractor, John P. Picone, Inc., selected a scheme proposed by Nicholson Construction featuring the use of a structural diaphragm wall constructed by the slurry trench method. Given the need to avoid 'blow in' of the silty fine sands at the designed subgrade elevation of -21.6m (Elevations refer to Borough of Staten Island sewer datum which is 970mm above USGS Mean Sea Level at Sandy Hook, NJ), and to generally reduce seepage into the shaft excavation, options were examined to extend a cut off below subgrade

Donald A. Bruce is Technical Director at Nicholson Construction Company; Patrick H. C. Chan is a Design Engineer and George J. Tamaro is a Partner at Mueser Rutledge Consulting Engineers.

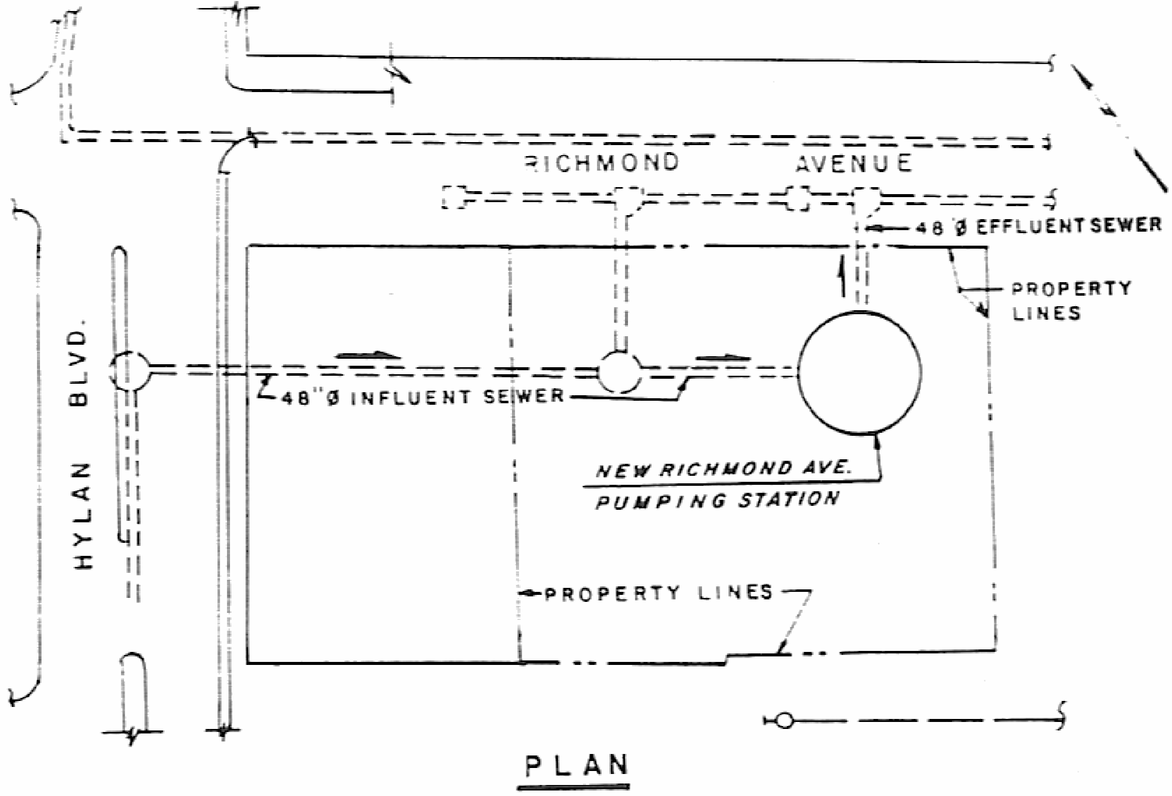


FIGURE 1

elevation into a relatively impermeable hard silty clay, at Elevation -47.3m. 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 to elevation -47.3. This decision was influenced by the dewatering difficulties experienced by other contractors working on adjacent shafts.

The diaphragm wall installed by Nicholson Construction was 16m in internal diameter, 50m deep, and 600mm thick, making it, at the time, the deepest such structure installed by conventional diaphragm wall equipment in the United States. Mueser Rutledge Consulting Engineers provided geotechnical services, designed the slurry wall and inspected installation of the wall.

This paper summarizes the subsurface conditions, design, construction and performance of the wall and the subsequent excavation.

## SITE CONDITIONS

### Surface Access

The site was 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 (see Figure 1). 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 preexcavated to Elevation +3.8m for installation of a crushed stone work platform. This level was slightly above the final top of wall (El +3.0m) obtained after removal of guide walls and bentonite contaminated concrete.

## Subsurface Conditions

The stratum boundaries dipped gently across the site. The following major units were identified by the additional site investigation undertaken after award of the contract. The uppermost stratum (G-1) is a Recent glacial deposit, whereas the bulk of the sequence comprised Cretaceous beach sand deposits. STP values relate to a 64 kg. hammer and a drop of 760mm (Figure 2).

El +3 to +1.5 (G1) - Medium compact red-brown silty fine to medium sand, trace gravel, roots (SM). SPT 13-28 (Avg 19), Water content 18%.

El +1.5 to -29m (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 (CL). SPT for sands 28-114 (Avg 74), for others 5-22 (Avg 13), water content 17-29%.

El -29m to -36.6m (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 (Avg 81), water content 20-22%.

El -36.6m to Avg -39.6m (C2) - Light grey, fine, medium sands, trace silt, trace clay (SP, SM, CL). SPT from 46-70 (Avg 53), water content 22%.

El Avg -39.6m to Avg -47.3m (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 (Avg 56), water content 24-29%.

Below Avg -47.3m (C3) - Hard to very hard, dark grey, silty clay (CL), trace silt, water content 24-26%.

## Permeability

The wall was designed to toe 0.6m to 1.5m into the hard, relatively impermeable C3 stratum. During excavation, the ground water level averaged about Elevation -4.6m, with a 0.6m to 0.9m variation, attributed to tidal effects and the adjacent dewatering efforts of another shaft contractor. The silty sands generally had in situ permeabilities (vertical) in the range of $8 \times 10^{-4}$ to $3 \times 10^{-6}$ cm/sec, based on laboratory testing of Pitcher barrel undisturbed samples. Values of $2 \times 10^{-3}$ to $2 \times 10^{-4}$ cm/sec were calculated from field falling head tests in piezometers (horizontal), and $2 \times 10^{-2}$ cm/sec was estimated from an analysis of drawdown data on adjacent contracts. The piezometer test data was affected by dewatering within 60m of the site and the dewatering analysis was influenced by the proximity of the ocean.

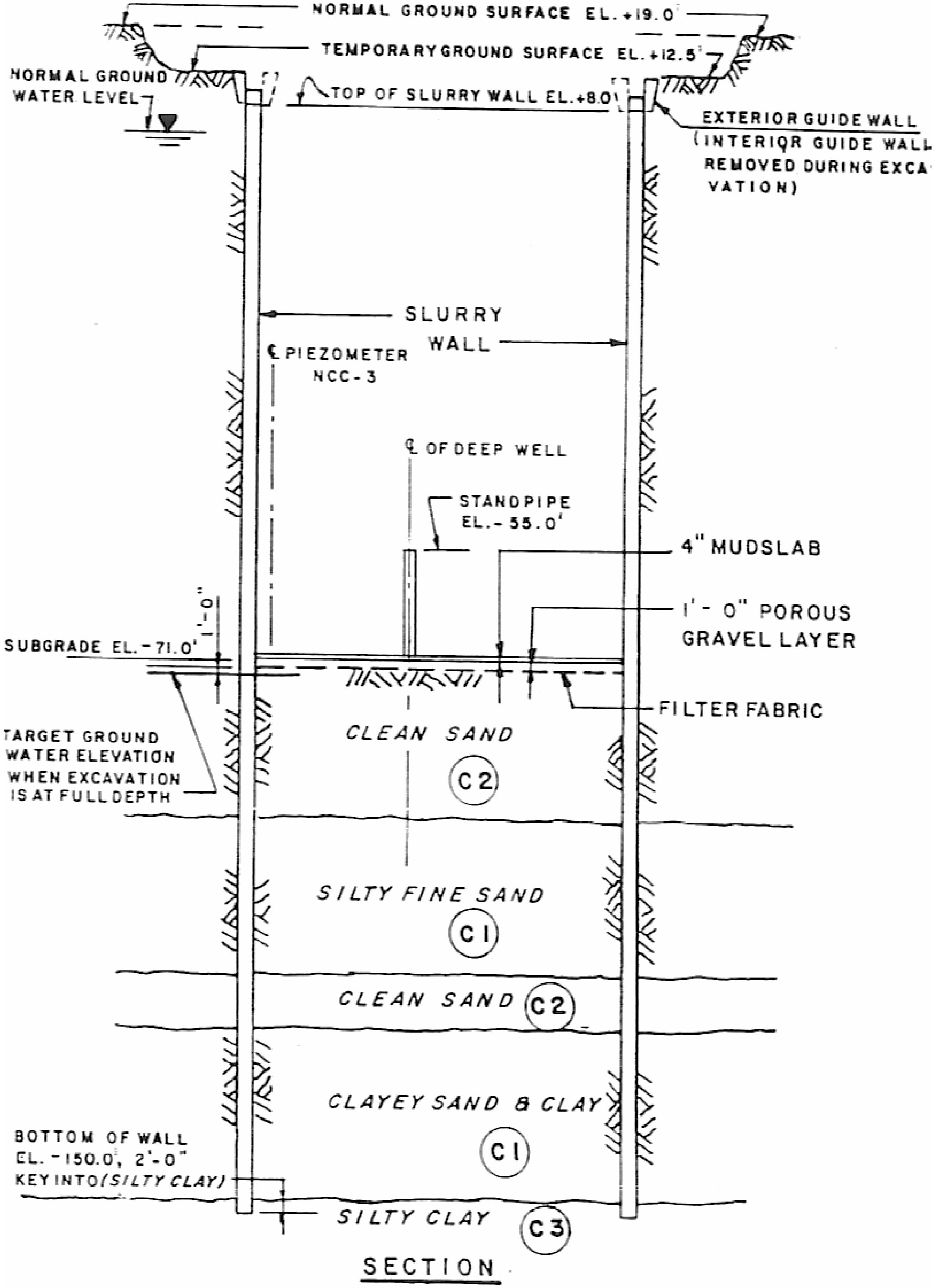

NORMAL GROUND SURFACE EL.+19.0'
TEMPORARY GROUND SURFACE EL.+12.5'
NORMAL GROUND WATER LEVEL
TOP OF SLURRY WALL EL.+8.0'
EXTERIOR GUIDE WALL (INTERIOR GUIDE WALL REMOVED DURING EXCAVATION)
SLURRY WALL
℄ PIEZOMETER NCC-3
℄ OF DEEP WELL
STANDPIPE EL.-55.0'
4" MUDSLAB
1'-0" POROUS GRAVEL LAYER
1'-0"
SUBGRADE EL.-71.0'
FILTER FABRIC
TARGET GROUND WATER ELEVATION WHEN EXCAVATION IS AT FULL DEPTH
CLEAN SAND
C2
SILTY FINE SAND
C1
CLEAN SAND
C2
CLAYEY SAND & CLAY
C1
BOTTOM OF WALL EL.-150.0', 2'-0" KEY INTO (SILTY CLAY)
SILTY CLAY
C3
SECTION

FIGURE 2

## DESIGN CONSIDERATIONS

### Water Cutoff

Two dimensional flow net analyses were conducted to estimate ground water flow during construction. These studies indicated that an adequate factor of safety against base heave, boils or piping could be obtained if the toe of the wall was established at Elevation -39.6, otherwise continuous dewatering would have been required. The anticipated presence of cleaner, more permeable sands at Elevation -36.6m to -39.6m, and the proximity to the ocean, caused concern that higher groundwater flows than predicted by the general analysis would, in fact, occur. For safety and economic reasons, it was therefore decided to continue the diaphragm wall cut-off to Elevation -47.3m where a more certain water cutoff could be obtained from the low permeability C3 material.

An uplift stability analysis was also conducted on the finished base slab. The groundwater within the shaft had to be maintained below the gravel drainage layer until such time as the 2.1m thick base slab concrete reached an Unconfined Compressive Strength of 21 kPa. Thereafter, hydrostatic uplift forces, corresponding to a water level of Elevation -16.8m, could be resisted with a factor of safety of 1.05. An open standpipe was installed to Elevation -16.8m to ensure that the pressure beneath the slab never exceeded that level: overflow from this pipe would merely add weight to the structure and increase overall stability. Full hydrostatic pressure could be resisted with a factor of safety of 1.1 when the outside walls reached Elevation -3.4m and interior concrete reached Elevation -14m. Upon completion of the whole structure, the factor of safety against uplift was calculated to be 1.4 for structural weight alone (including the weight of the slurry walls), and 2.9 assuming side friction on the slurry walls.

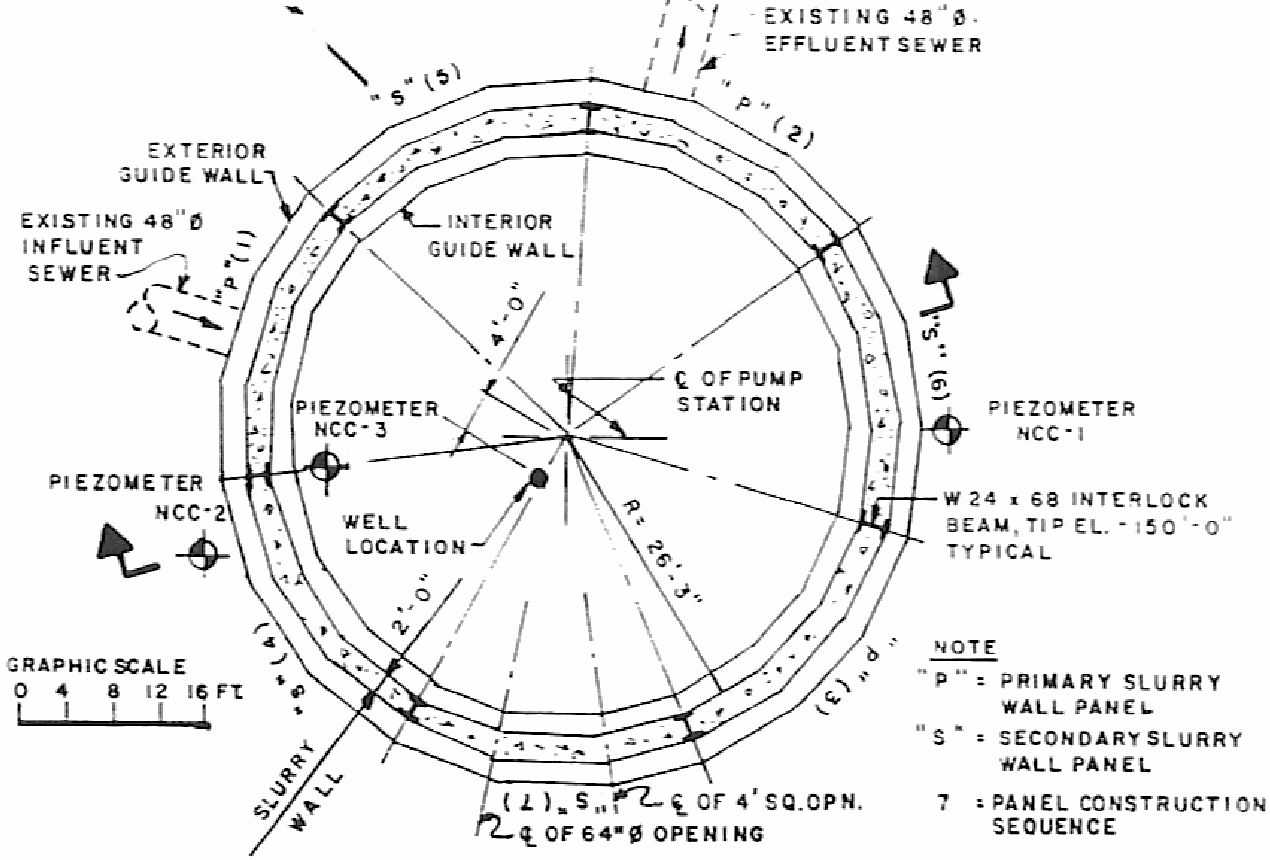
The interior walls of the pump station were connected to the slurry walls by keyways and dowels designed to transfer the weight of the slurry wall to the permanent structure in an uplift condition. The factor of safety against uplift was thus improved for both construction and the permanent structure, resulting in savings in the cost of pumping beneath the base slab during concreting of the permanent shaft since pumps could be shut down earlier.

### Slurry Wall Panel Geometry

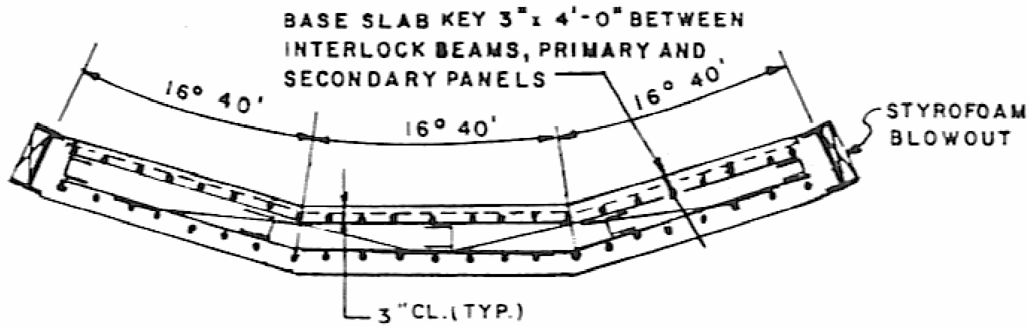
Available excavation equipment dictated that the wall be laid out in segments of seven panels, each consisting of three chords (Figure 3). The deflection angle at the joint between adjacent panels was sufficiently large to accommodate minor variation in plan geometry without fear of local buckling 'snap through'. Panel joints were formed by 50m long, 600mm deep wide flange beams. Panels were heavily reinforced to a depth of 32.6m with steel cages comprised of 25mm horizontal bars and 20mm, 22mm, and 25mm vertical bars.

### Analysis and Design

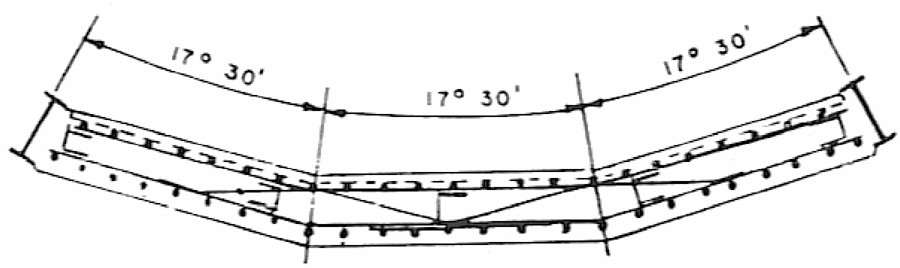
The structural design considered the behavior of the shaft as a cylindrical shell constructed in chord segments and then as a cylinder of discrete segments with misalignments of up to 150mm at the panel




GENERAL PLAN




SLURRY WALL PRIMARY PANEL
PLAN




SLURRY WALL SECONDARY PANEL
PLAN

FIGURE 3

joints. Longitudinal (vertical) and transverse (horizontal) bending was also examined.

In the first case, the wall was assumed to be constructed perfectly aligned, without ring beams or any other internal bracing. The structure was analyzed as a cylindrical shell under axisymmetric loading (triangular distribution). Both ends of the shell were assumed unrestrained. Soil pressures were calculated assuming at rest soil conditions ($K_o$ = 0.5 to a depth of 21.3m and 1.0 below 21.3m). The maximum radial displacement computed was 3mm, a movement too small to mobilize the soil into a $K_a$ situation. Ground movements associated with panel excavation were not considered to be sufficient to significantly reduce at rest radial soil pressures. The longitudinal bending moment associated with this case was small, as expected, since the shell resists load in direct compression. This case did not govern the design.

The second case considered the possibility of significant panel misalignments occurring during construction requiring the installation of ring beams for supplemental bracing. A panel was assumed to be misaligned to the extent that arch action was no longer sufficient to fully support the load. 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 associated with bending of the cylinder would be larger than displacements associated with ring compression.

For the transverse analysis, a 150mm maximum horizontal misalignment of adjacent panels was assumed. The wall section was designed as a column resisting ring compression axial force and a bending moment generated by the 150mm eccentricity. Reinforcement was provided to withstand the horizontal bending moments associated with misalignment of joints and the bending of panels between future stabilizing ring beams. External pressure, ring compression and moment varied linearly with depth to subgrade, with the density of horizontal reinforcing varying accordingly. This assumed construction defect governed the design but did not develop in the field. As a result, ring beams were not installed. A cap beam was installed at the very top of the shaft to finish the top of the wall and to provide stability to the individual panels at the top.

## CONSTRUCTION

The Sequence of Shaft Construction was divided into six stages,outlined as follows:

I. Site Preparation
   1. Excavate to site working level (approx El +3.8m) and place crushed stone as a working platform.
   2. Layout center of shaft and guide walls.
   3. Excavate and install guide walls.

II. Diaphragm Wall Construction
   1. Excavate Primary panel.
   2. Maintain bentonite slurry in excavation with top of slurry at El +2.1m or higher.
   3. Desand slurry at completion of panel construction.

4. Fabricate reinforcing cage and attach 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 2.4m below slurry interface.
7. Excavate and cast remaining two Primary panels in sequence indicated in Figure 2.
8. Excavate Secondary panels (4 each), allowing adjacent Primary panels to cure a minimum of two days before starting excavation. Install cage to a 'right fit' between existing interlock beams. Secondary panel S4 requires one interlock beam installed with the cage.

II. Control Groundwater

1. Install and develop deep well, minimum 125mm i.d., to El -54.9, at position shown in Figure 1. Install submersible pump.
2. Install piezometer (NCC 3) within shaft as shown in Figure 1 to El -27.4m. (Note: Both to be sealed through base slab when cast.)
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 maintain 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 -22.7m.
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 base slab of permanent structure and key into slurry wall. Allow slab to cure to minimum $f'_c$ of 21 kPa while maintaining groundwater below El -22.3m. Extend 125mm well casing standpipe up to El -16.8m and 50mm riser pipe for piezometer to El -13.7m.

VI. Construct Permanent Structure

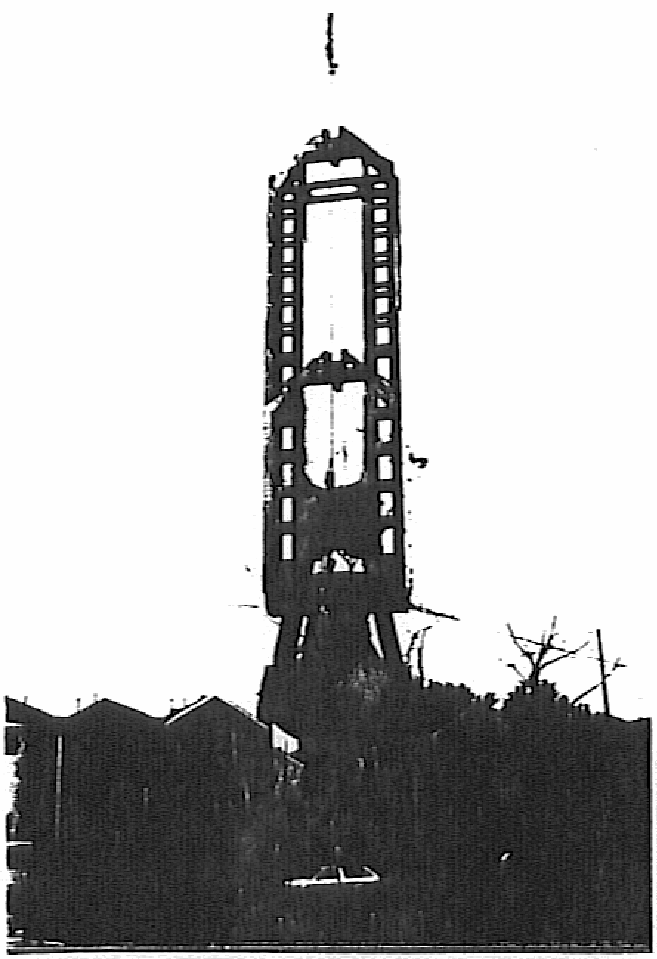
1. Limit uplift pressures on base slab by allowing water to rise in standpipe and flow into shaft if necessary. Dewater shaft as needed, but not less than daily. Operate deep well during the day to maintain water pressure on the slab to a pressure no higher than a head at El -16.8, as measured by riser 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 +3.4m, and interior construction has reached at least El -10.7m, the piezometer, riser pipe, well and standpipe may be sealed, allowing full hydrostatic pressure to develop on the base of the slab.

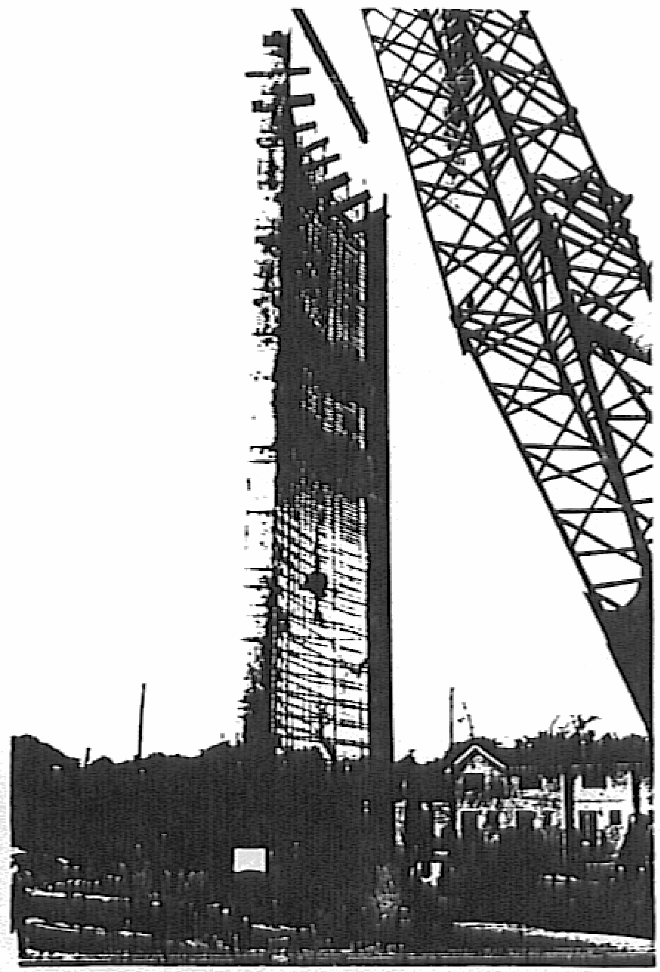
Diaphragm Wall Construction

The guide walls were of standard design and construction, 900mm deep by 450mm wide, reinforced longitudinally and with sufficient tolerance for the 600mm wide bucket to pass unencumbered.

A 140 tonne capacity Manitowoc 4000 crane operated the 13 tonne Keller mechanical excavating grab (Photograph 1). This grab was extended by guides to a length of 9.8m to aid verticality control and was capable of a 2.7m bite in one pass. Verticality measurements were made every 3.0m of excavation depth by checking the position of the lifting cables. In addition, at least twice in each panel, piano wires were attached to the grab to measure panel deviation or twisting. Panel deviation was readily corrected by use of a specially designed chisel. A tolerance of 1% of verticality and 150mm horizontal planar misalignment was permitted.



Photograph 1. Cable Suspended excavating grab, showing long frame extension.

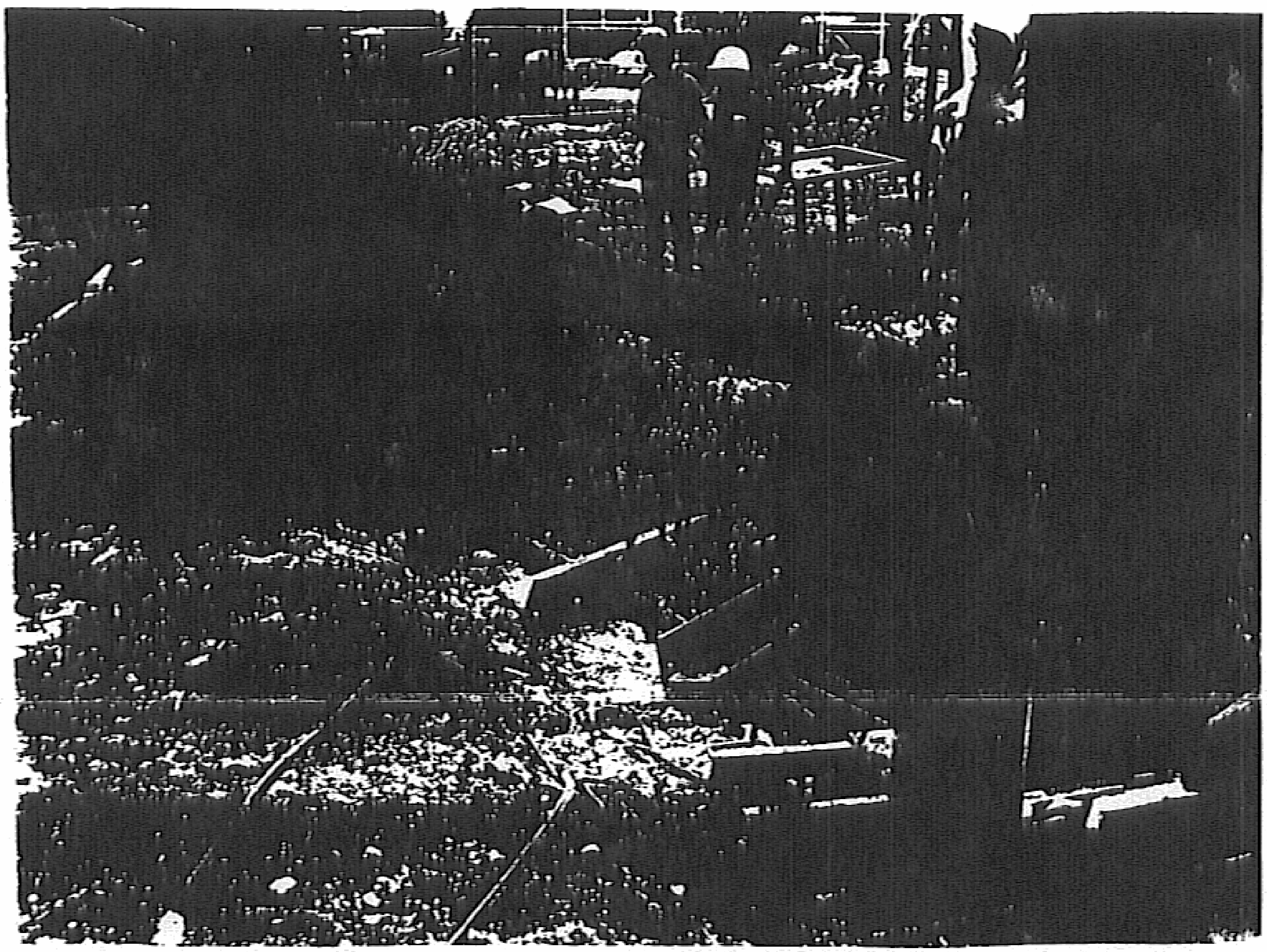


Photograph 2. Placement of reinforcing cage and transverse beams. Note wood and styrofoam for blockouts & joint keys.

Bentonite slurry 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 following about 9m behind the end bites. Production rates were considerably lower than what had been anticipated on the basis of the geological logs. Average excavation rates in the upper 24mwere reduced by a factor of 5 from 24m to 36m and by a factor of 2 below 36m. The grab's teeth were modified to improve 'bite' efficiency. The clay seams encountered at 44m and 48m proved relatively easy to excavate. There was minor loss of bentonite slurry throughout the job, attributable to drawdown from adjacent dewatering. 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% to less than 2%) before concreting.

The 50m long ASTM A36 interlock beams (WF 24x68) were delivered in three pieces and bolted or welded together. The 38m long steel ASTM Grade 60 rebar cages, were fabricated in two sections on site. A total of 175 tonne of ASTM A615 reinforcing steel was used. A special lifting bed was designed and built to handle the segmented long cage. Wood and styrofoam blockouts were attached to the cages to form keyways in the exposed slurry wall (Photograph 2).

Concrete was supplied by ready mix trucks and had a maximum aggregate size of 20mm, a target 28-day $f'_c$ of 28 kPa, a slump of 180mm and 4% air entrainment. Typical mixes contained 420 kg of cement, 780 to 830 kg of fine aggregate (natural sand), 980 to 1000 kg of coarse aggregate (crushed rock) and 165 l-180 l of water per cubic yard. Various types and



Photograph 3. Tremie concreting of Primary panel.

proportions of additives were used to entrain air and to retard/ plasticize. Seven-day strength tests indicated cube strengths of 31 kPa-38 kPa, increasing to 38 kPa-50 kPa at 28 days. Concrete was placed by tremie, via two 250mm diameter tremie pipes in each panel, at an average rate of $31m^3$ 3/hour (Photograph 3). The overpour on the job was around 15%.

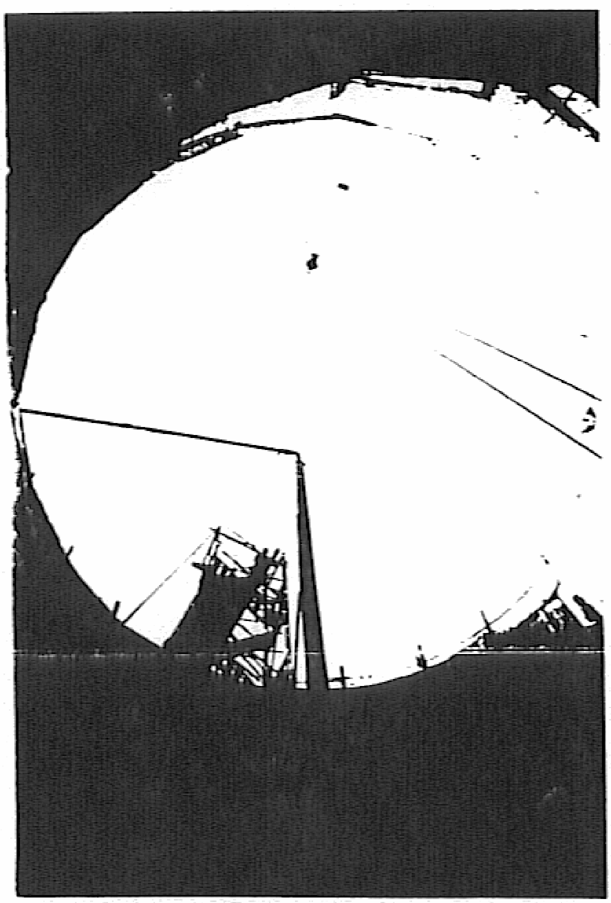
After completion of all panels the General Contractor removed the upper 3m of soil from within the shaft and installed a 750mm deep ring beam to secure and finish the upper part of the slurry wall.

**PERFORMANCE**

The dewatering pump was activated prior to the start of excavation and operated for only a day or two before it 'ran dry', highlighting the watertightness of the structure and supporting the decision to embed the wall in the low permeability clays.

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

Primary panels were found to be completely vertical with the Secondaries out by only 100 to 150mm, as measured at the panel centers. The ends of the Secondaries were, on average, slightly further out: the beams were square to the Primaries which possibly made the keying of the Secondaries much more difficult.



Photograph 4. View of shaft after excavation showing dewatering well.

Photograph 5. View from bottom after completion.

The keyways lined up well except at the P2-S5 joint where there the key was 600mm out of position. Small water seeps were noted at a local joint misalignment (S5 and P2) and at a point of minor concrete segregation (P1). These seeps produced about 8 to 20 lph prior to sealing and proved no problem during construction. After completion of excavation, the dewatering well was operated about once per week, and then for a only few hours, until it was effectively 'dry'. Photographs 4 and 5 show the completed excavation.

## CONCLUSIONS

The 16m diameter Oakwood Beach shaft was constructed within a tight site, under difficult ground and water conditions, using the slurry trench method of construction . Design requirements related to base stability and water inflow dictated that the wall be carried to a depth 24.4m below the subgrade elevation of the permanent structure, a total depth of 50m.

Excavation within the slurry wall cofferdam confirmed the suitability and quality of the structure. The permanent structure was built without incident within the excavation and without the need for internal bracing.

## ACKNOWLEDGEMENTS

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