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ABSTRACT

The City of Grand Forks, North Dakota obtains drinking water from both the Red and Red Lake Rivers using a system of river intakes, shallow pipelines, and pump stations. However, during flood events, the City often loses access to the system, and persistent landslides along the riverbanks easily damage shallow components. As a result, the City decided to construct a gravity system with new pipelines leading to the base of a 65-foot deep pump station. Several construction techniques were evaluated for the pump station, including a self-sinking caisson, which was chosen for the project. Because the proposed 60-foot-diameter caisson was larger and would be sunk deeper than any previous caisson in the area, and would penetrate through a highly plastic, weak and brittle clay, FLAC soil-structure interaction analyses were completed using a strain-softening soil model. The paper discusses the results of the modeling and the subsequent design steps taken to avoid bottom instability. The paper also describes the construction process.

INTRODUCTION

The City of Grand Forks, North Dakota is located in the Red River Valley, adjacent to the Red River of the North (Red River), near its intersection with the Red Lake River. The City of Grand Forks (City) obtains its raw water supply from both these rivers through the use of three intake facilities. Two of the facilities draw raw water from the Red River, while the remaining facility draws water from the Red Lake River. New river intake structures for both river sources had been planned, but were accelerated as a result of a record flood in the spring of 1997. During this event, floodwaters overtopped the levees, rendered the existing intake pump house structures inoperable, and left the City without a potable water supply. The high river levels eventually flooded nearly three-fourths of the City and caused the evacuation of the entire community of 50,000 people.

PROPOSED RAW WATER SYSTEM AND COMPONENTS

In the aftermath of the Flood of 1997, the replacement of the raw water intake system became more important as the City realized the existing system was highly vulnerable to flood damage. Also, as a result of a proposed U.S. Army Corps of Engineers (COE) flood protection project to raise the elevation of the City's levee system, the existing intake pump house structures needed to be protected from future flood damage or a new raw water intake pump house constructed on the "protected" side of the levee. Because the existing intakes were located near the banks of the Red and Red Lake River channels, flood protection was not feasible without substantial modifications to the existing structures. Based on these factors, the City initiated the design of a new raw water intake system in May of 1998.

GEOLOGY AND GEOLOGIC HAZARDS

Geologic Setting

The geology influencing the Red River Valley is the legacy of Glacial Lake Agassiz and recent fluvial/alluvial processes of the Red River and its tributaries. During the glacial period, the entire watershed of the present day Red River was covered by a continental glacier. Periodically, as the glacial ice melted and retreated northward, huge ice dams were formed which blocked the natural northerly drainage pattern. Glacial Lake Agassiz, which covered approximately 200,000 square miles, resulted from the ice damming and subsequent ponding of meltwaters. At its maximum extent, Lake Agassiz is believed to have been approximately 400 feet deep at Grand Forks, and over time was infilled with generally fine-grained sediments.

Stratigraphy

Subsurface conditions underlying the site were explored by drilling and sampling nine borings and conducting three cone penetrometer tests. In general, the borings indicated that the proposed inland pump station was underlain by about 29 to 39 feet of medium stiff to stiff, clayey silt/silty clay (Sherack Formation) down to elevation 791 to 797 feet, overlying very soft to soft clay (Upper and Lower Brenna Formations) to an elevation of about 750 feet. The Brenna Formation is a uniform glaciolacustrine clay, with little or no visible structure. The major source of sediment for this formation was eroded Pierre Shale bedrock. Slickensides are commonly observed on shear planes in freshly broken samples. The Brenna Formation is notoriously unstable as a foundation material throughout the Red River Valley. The Brenna Formation clay was underlain by medium stiff, trace to slightly sandy and/or silty clay with a trace of gravel (Falconer Formation) down to about elevation 702 feet. The Falconer Formation was underlain by about 10 feet of medium stiff clay (Wylie Formation) to elevation 692 feet and then the Red Lake Falls Formation, a dense, slightly silty and clayey sand and hard, silty, sandy clay till with varying percentages of gravel. The thicknesses and geologic units encountered in the borings were similar to the regional geologic profile of the Grand Forks area, as presented on Fig. 1.

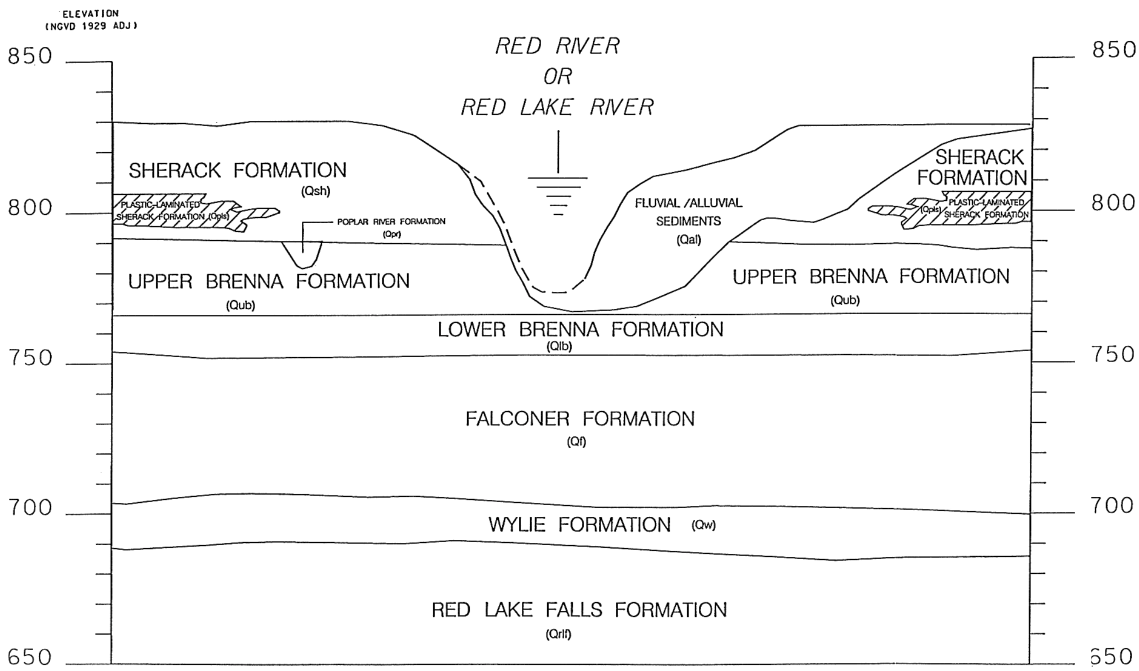

Fig. 1. Generalized Geologic Profile (after U.S. Army Corps of Engineers, 1998)

<u>Groundwater</u>

Historically, the groundwater levels in the project vicinity have been difficult to determine, mainly due to the low permeability of the soils, but also because little instrumentation existed prior to the 1997 flood. As part of a U.S. Army Corps of Engineers (1998) levee design study, vibrating wire piezometers were installed along the riverbanks in 1998 and 1999. New piezometers were also installed as part of the exploration program for this project. Data collected from these instruments indicated that groundwater was present about 5 to 20 feet below the ground surface (approximately elevation 810 to 825 feet) and that groundwater levels near the river directly correspond to changes in the river level.

<u>Geotechnical Laboratory Testing</u>

Laboratory tests were performed on selected samples retrieved from the borings to determine index and engineering properties of the soils encountered in the explorations. The tests completed included visual classification, natural water content determination (American Society for Testing and Materials [ASTM] Designation: D 2216), grain-size distribution (ASTM Designation: D 422), liquid and plastic Atterberg limits (ASTM Designation: D 4318), hydrometer analyses (ASTM Designation: D 422), consolidated-undrained triaxial compression tests (ASTM Designation: D 4767), unconsolidated-undrained triaxial compression tests (ASTM Designation: D 2850), and one-dimensional consolidation tests (ASTM Designation: D 2435). To evaluate the corrosion potential on the proposed transmission lines and foundation elements, selected soil samples also were tested for resistivity, redox, pH, chlorides, and sulfides.

PUMP STATION ALTERNATIVES

The new pump station was required to be a structure with inside dimensions of approximately 60 feet and a finished floor elevation of 763 feet, which is about 65 feet below the existing ground surface. Transmission lines were to penetrate the walls of the pump station at about elevation 765 feet (invert elevation). The 65-foot depth was determined based on required gravity flow conditions from each of the river sources via the transmission pipelines. The 60-foot inside diameter was determined by the size and configuration of interior process piping and space requirements for other amenities such as a stair tower.

The design team evaluated various types of substructures and square, rectangular, and circular shapes. In general, the pump station could be constructed to its proposed depth by using either a temporary, flexible excavation support system or a permanent, rigid excavation support system. Feasible construction methods included open excavation, sheeting and benching with tie-backs, in-situ secant wall construction, high pressure grout injection and soil stabilization, soil freezing, and caisson pouring and sinking methods. Usually, in excavations in competent soils that extend to only moderate depths (up to about 40 feet), experience indicates that temporary, flexible support systems are more feasible and economical, even when the cost associated with constructing the final structure walls within the excavation is considered. Under these circumstances, permanent, rigid systems typically are more expensive because of the large initial construction cost. However, based on the depths required and weak soil conditions, it was determined that the most cost effective and feasible pump house substructure would be a circular concrete caisson installed by pouring and sinking methods of construction. A caisson was also selected based on the local experience in sinking caissons in the Grand Forks area (albeit previous caissons were smaller and shallower).

GEOTECHNICAL ANALYSES AND MODELING

Selection of Engineering Properties

The selection of geotechnical engineering properties was based on the consideration of many factors, including the use of moisture contents and Atterberg limits to delineate geologic units, a review of previous test results and recommended properties by the U.S. Army Corps of Engineers (1998) as compared to recently obtained data, and the determination of shear strengths from cone penetrometer test results. In addition, we considered the "brittle", softening behavior observed in laboratory test results, as described below.

The soils in this region, especially the Brenna Formations, reach peak shear strength in laboratory tests at low axial strains (often 1 to 2 percent). With further axial strain, a significant reduction in shear strength then occurs.

To account for strain softening behavior, ultimate shear strengths were used for design instead of peak shear strengths. These ultimate shear strengths were determined from triaxial compression tests by recording the deviator stress at an axial strain of 15 percent. For a majority of the soils, this strain occurred after peak strength had been reached and hence the ultimate strengths were referred to as "post-peak shear strengths." The post-peak shear strengths were horizontally projected from the descending limb of the stress-strain curve to the ascending limb (Fig. 2). This mechanism provided a means to assure that the soil would not reach peak shear strengths and undergo potentially significant strength loss.

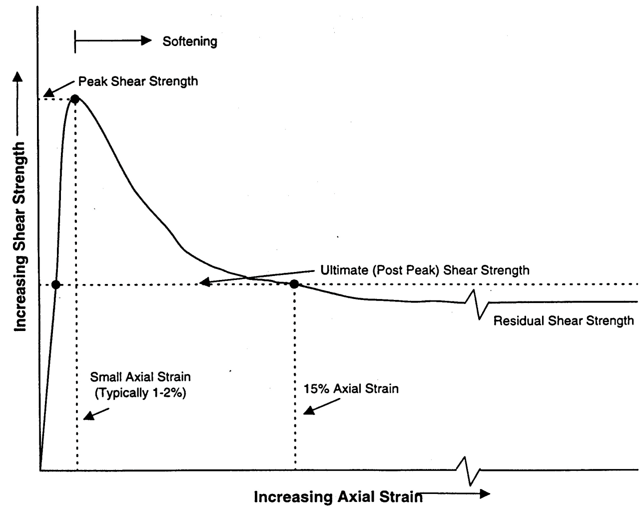

Fig. 2. Typical Softening Behavior (Shannon & Wilson, 2000)

Because of the unique brittle, softening behavior of the soils, the use of post peak shear strengths was appropriate when large strains were possible and/or when designing against potential instability. However, the peak undrained shear strength was also an important property in some instances, such as in overcoming the bearing capacity in sinking the caisson and estimating the difficulty in boring for the transmission lines. Hence, both peak and post-peak shear strengths were considered, depending on the analysis. Tables 1, 2, and 3 present engineering properties for the soil units encountered beneath the site.

Sinking Considerations

To sink the proposed caisson, the following issues needed to be resolved: (1) overcoming the side friction associated with sinking the caisson, especially through the medium stiff to stiff upper clay (Sherack Formation); (2) controlling the caisson weight to skin friction ratio; (3) devising a method of excavation that results in reliable controlled vertical sinking of the caisson while not allowing the bottom of the excavation to heave; (4) excavating to the design elevation, possibly in the wet, to place the concrete tremie seal; and (5) providing a means that allows the transmission lines to pass though the pump station's walls without allowing soil to squeeze inward. These considerations are addressed below.

Table 1. Recommended Drained Soil Properties (after the U.S. Army Corps of Engineers [1998])

Geologic Unit	Total Unit Weight - γ (pcf)	Buoyant Unit Weight - γ' (pcf)	Peak		Post Peak		Residual	
			ϕ' (deg)	c' (psf)	ϕ' (deg)	c' (psf)	ϕ' (deg)	c' (psf)
Sherack	122	60	26	320	30	0	27	0
Upper Brenna	100	38	16	0	13	0	7	0
Lower Brenna	110	48	21	0	20	0	10	0
Falconer	126	64	24	300	25	0	25	0
Wylie	126	64	32	0	25	0	21	0

Table 2. Recommended Undrained Soil Properties

Geologic Unit	Total Unit Weight - γ* (pcf)	Peak Undrained Shear Strength (psf)	Post Peak Undrained Shear Strength (psf)
Sherack	122	1,400	1,000
Upper Brenna	100	1,000	680*
Lower Brenna	110	1,000	680*
Falconer	126	1,400 – 1,600**	1,000 – 1,100**
Wylie	126	1,600	1,050*

* Properties obtained from U.S. Army Corps of Engineers (1998) data and report and project testing.
**Lower strengths applicable above elevation 730 feet; higher strengths below this elevation.

Table 3. Recommended Deformation Soil Properties

Geologic Unit	Shear Modulus, G (psf)	Compression Index, Cc	Recompression Index, Cr
Sherack	1.3×10^5	0.32	0.06
Upper Brenna	6.7×10^4	0.97	0.22
Lower Brenna	6.7×10^4	0.66	0.09
Falconer	1.7×10^5	0.23	0.04
Wylie	1.7×10^5	0.70	0.10

The upper 29 to 39 feet of soil consisted of a medium stiff to stiff, clayey silt to silty clay with peak undrained shear strengths on the order of 1,000 to 2,000 psf. Sinking the caisson through these medium stiff to stiff soils, without forming a significant height of the caisson, was estimated to be difficult. As an aid to sinking the caisson, we recommended overexcavating the site to a depth of 20 to 25 feet before sinking. This depth was determined to be the maximum depth possible, taking into account adjacent facilities (roadways and utilities), groundwater considerations, and slope stability concerns. An additional benefit of performing this excavation was a reduction of the driving force tending to cause bottom instability, and thus an increased factor of safety (FS) as compared to the case where no excavation occurred (as discussed later).

To aid the sinking process, the caisson substructure was designed with special provisions. The bottom section of the caisson was designed to include a cutting shoe to provide an inclined surface with a small, 12-inch wide bearing area. The cutting shoe was formed using plywood forms set at a 45 degree angle with the angled face towards the interior of the structure. This angled face and associated 12-inch bearing face would essentially cut through the soils thus improving the sinking process. The cutting shoe was also formed 6-inches wider than the 3-foot thick concrete caisson wall, with the extra width provided at the exterior of the caisson wall. As the caisson sinking procedure progressed, the 6-inch gap between the face of the caisson wall and the soil would minimize side friction, again, improving the sinking process. In addition to the oversized cutting shoe, 2-inch PVC slurry injection pipes were installed at 5-foot on center from the top of the cutting shoe to the top of the caisson wall. During the caisson sinking process, the PVC pipes were available for injection of a bentonite slurry mixture into the 6-inch oversized cutting shoe annulus to aid caisson sinking and also provide downhole pressure to minimize soil squeeze against the caisson wall surface (the pipes were not needed during construction, but were used for grouting following sinking). A 6-inch deep bearing surface was also cast into the interior of the cutting shoe. The 6-inch deep by 5 foot tall depression in the wall surface would allow the caisson base slab to be poured "under" the wall ledge, thus connecting the caisson base slab and associated deep foundation system to the caisson wall.

Bottom Stability

The total excavation depth of the caisson was about 75 feet, which included allowances of about 10 feet for the tremie seal and final concrete floor. Given the typically soft to medium stiff strength of the subsurface soils, the potential for bottom heave into the excavation was evaluated. Considering the high plasticity of the soils and relatively short duration of construction typically involved in sinking a caisson, we concluded that undrained strengths would control stability.

Initially, we evaluated the base stability using theoretical methods (Terzaghi et al., 1996). These methods implicitly assume elastic-perfectly plastic soils; i.e., they do not account for the strain softening effect. Considering the uncertainty associated with the softening effect, we used post-peak, undrained shear strengths of the soils (Table 2) to estimate a range of FS against base instability. As shown on Fig. 3, the FS against bottom heave was less than the minimum recommended value of 1.25 below about elevation 795 to 775 feet. A 1.25 safety factor was recommended based on a consideration of the short-term nature of the sinking and the use of post-peak shear strengths.

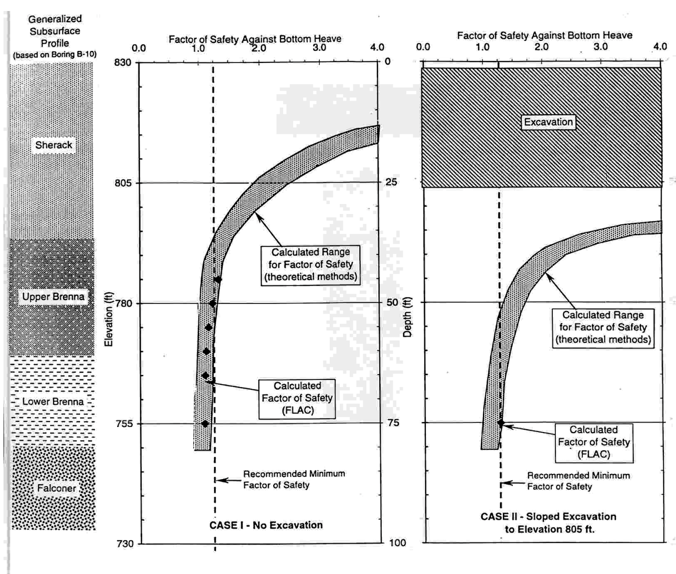

Fig. 3. FS Against Bottom Heave (Shannon & Wilson, 2000)

As an alternative to the theoretical methods, the numerical analysis method is more appropriate for the subject case as it has the capability of modeling the softening behavior of the soil. A numerical analysis was performed using the computer program Fast Lagrangian Analysis of Continua (FLAC) in two dimensions developed by Itasca Consulting Group, Inc. (1998). The two-dimensional FLAC program has a special feature to model three-dimensional cylindrical structures, such as the subject caisson.

To verify the applicability of the FLAC analyses for the subject case, we performed a base stability analysis using both the theoretical methods and FLAC. The case consisted of a 60-foot-diameter, 50-foot-deep excavation into a cohesive soil with undrained shear strength of 700 pounds per square foot (psf). The soil model used in FLAC for this case was an elastic-perfectly plastic model (i.e., no softening). The FS computed from the theoretical methods (Terzaghi et al., 1996) and FLAC were 1.1 and 1.0, respectively. The similarity in FS values for this simplified case indicated that the FLAC model was reasonable.

For the subject excavation model, the soil stratigraphy and strengths were based on the subsurface explorations performed in the vicinity of the site. Table 4 presents the relevant soil properties used in the FLAC analyses.

All the soils units were assigned a strain softening model. Because laboratory strength tests typically underestimate soil stiffness, empirical correlations between undrained shear strength, overconsolidation ratio, plasticity index and shear modulus were used to estimate the shear moduli (Ladd et al., 1977). The preliminary runs of the FLAC model indicated that some soil elements would axially strain beyond 15 percent. To account for the effect of straining beyond the post-peak strains, residual strengths and strains were approximately estimated from the laboratory tests.

In the FLAC model (Fig. 4), the construction method consisted of 15 sequential stages, each being five feet deep. To obtain the FS against base instability, the strength and stiffness of the subsurface soils were reduced until unrealistically large and increasing base deformations occurred (defined as failure condition). The reduction factor corresponding to this instability represents the FS. The FLAC results indicated that the FS against base instability for the 75-foot-deep excavation was about 1.1 (Fig. 3 left plot). The potential failure mode was a rotational-type failure into the base of the excavation, with large strains at the base.

The computed FS from the FLAC analyses was less than the minimum recommended value of 1.25. Similarly, as shown on Figure 3, the FLAC FS remains below 1.25 at any elevation below 785 feet. Of significance was the steepness of the FS curve below elevation 785 feet. Because of this steepness, small deviations in the undrained shear strength could result in significant changes in the FS. As such, the minimum FS of 1.25 was adhered to. Because the FS was less than 1.25, preventive measures were considered to increase the base stability. Remedial measures studied are discussed below.

Table 4. Soil Properties For FLAC Analyses (after Shannon & Wilson, 2000)

Geologic Unit	Unit Weight (pcf)	Undrained Shear Strength (psf)			Shear Modulus (psf)	Ko
		Peak	Post Peak @ 15% axial strain	Residual @ 30% axial strain		
Sherack	122	1,400	1,000	500	1.3×10^5	0.8
Upper & Lower Brenna	105	1,000	680	350	6.7×10^4	0.8
Falconer (above el. 730 feet)		1,400	1,000	500		
Falconer (below el. 730 feet) and Wylie	126	1,600	1,100	500	1.7×10^5	0.8

Ko = at-rest earth pressure coefficient

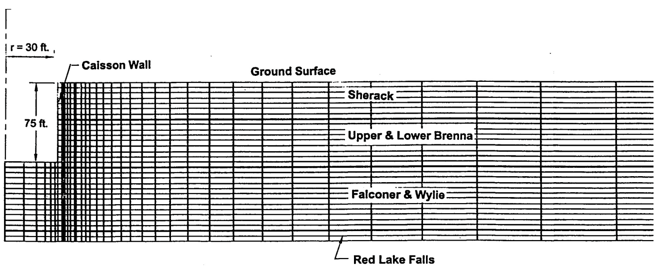

Fig. 4. FLAC Model

Unloading. Excavating soil from behind the caisson would reduce the driving force and thus, increase the FS. The results of our FLAC analyses indicated that a 25-foot-deep, 25-foot-wide excavation around the perimeter of the caisson would increase the FS against base instability to 1.25 (Fig. 3 right plot). The excavation width of 25 feet used in the FLAC analyses represents an average of the excavation width, approximately representing an excavation that is 10 feet wide at the bottom and 35 feet wide at the ground surface (slope of 1H:1V).

Increasing Shear Strength. Increasing the strength of the underlying soil properties would provide the most positive benefit to bottom stability; however, such improvement would be difficult because of the fine-grained nature and high plasticity of the soils. In addition, the soils that would be improved would still need to be penetrated by the caisson. As such, improvement would mainly be limited to beyond the edges of the caisson. Thus, it was our opinion that other alternatives to improving bottom stability should be considered.

Wet Excavation. Filling a caisson with water during sinking is often used to balance heave forces. However, excavating in the wet would increase the duration required to construct the caisson and make further controlled sinking of the caisson difficult. Factors that complicate excavation in the wet include: (1) difficulty in precisely placing a clamshell through murky water, (2) difficulty in inducing a bearing capacity failure of the soil beneath the caisson as it would be difficult for the clamshell bucket to excavate beneath the cutting shoe, and (3) difficulty in obtaining a good bond between the foundation uplift elements and the concrete tremie seal due to suspended bentonite in the water.

Balancing Force-Soil Weight. As an alternative, it would be possible to design the caisson walls and cutting shoe so that the caisson sinks through the Brenna Formations to its final elevation near the contact with the Falconer Formation, while leaving soil inside the caisson. This technique was used on a caisson sunk through soft soils in Detroit (ENR, 1990).

Piles/Shafts. Finally, it may be possible to install the Augered Cast-In-Place (ACIP) piles required for uplift resistance earlier in the construction sequence and use the strength of the piles to resist bottom heave.

To address the bottom stability concern, two construction sequences were considered in more detail. The first sequence included wet construction of the caisson. During sinking of the first 15 feet of the caisson, the soil would be excavated and removed to unload the site. At a depth of approximately 50 feet below grade, the caisson would be flooded to counteract basal instability tendencies. Caisson construction would then continue in-the-wet until the bottom of the caisson reached the required depth. Once in-place, the ACIP piles would be installed and the caisson base pad tremmied into place.

The second sequence involved installing the ACIP piles prior to sinking. The portion of the pile from the pile tip to the base of the caisson would be constructed with high strength grout, and the portion of the pile from the base of the caisson to the existing ground surface would be constructed with a low strength material. A minimum strength of 500 psi was required for this low strength material in order to provide soil stability while constructing the caisson. Once the ACIP piles were installed, caisson construction and excavation of the soil and low strength pile material would continue until the bottom of the caisson reached the required depth. Under sequence two, the ACIP piles would resist base instability by providing additional shear strength in the soil mass. Once the caisson was sunk to the desired depth, the ACIP piles would be trimmed to the required length, base slab reinforcement would be placed and tied to the ACIP piles, and the base slab would be poured. The second construction sequence was ultimately chosen because of ease of caisson construction.

In addition to the analyses completed specific to the proposed pump station, we analyzed a 24-foot-diameter, 50-foot-deep caisson using the geology at the proposed pump station site. Because the geology at the proposed pump station is similar to typical Grand Forks stratigraphy (see Fig. 1), this analysis was used to better understand the performance of past caissons that have been successfully sunk in the Grand Forks area. The results of the numerical analyses indicated that a 24-foot-diameter, 50-foot-deep caisson would have a FS of at least 1.4 against bottom instability. This FS is higher than the FS computed for the proposed caisson at the same depth (Fig. 3, elevation 780 feet) and illustrates the influence that three-dimensional effects have on increasing the FS of a small-diameter caisson, as compared to the proposed 60-foot caisson. It confirmed that small-diameter caissons could be excavated without inducing bottom instability (as was previously successfully completed in Grand Forks). Thus, it was important for contractors to appreciate the scale effects associated with this larger diameter caisson and the City spent considerable effort in attempting to educate local contractors about the significance of this caisson compared to previous caissons in the Grand Forks area, through a pre-qualification process, a mandatory pre-bid meeting, and requiring contractors to have a geotechnical consultant on their team.

CONSTRUCTION

Construction of the caisson and associated foundation system began on October 8, 2001 and was successfully completed on May 30, 2002. The contractor utilized the following general construction sequence:

- The construction site was prepared including tree and topsoil removal, and the erection of construction fencing.
- Once the site was prepared, a subcontractor specializing in the installation of deep ACIP piles was mobilized. Before beginning production pile construction, a test pile was installed near the caisson similar to those to be used for the caisson foundation. The test pile had a diameter of 24-inches and was installed to a depth of 119 feet below existing grade. A 1-1/4 inch diameter Dywidag anchor bar was installed the full length of the test pile. The lower 52 feet of the test pile was then constructed using 4,500 psi concrete grout, while the upper 67 feet was unfilled. Two reaction piles, one on each side of the test pile were also constructed. A static pile load test was then completed on the pile per ASTM Designation: D 3689 to the required 152-kip load.
- Following successful test pile loading, a total of 37, 24-inch diameter ACIP production piles were constructed from existing grade to a terminal tip elevation approximately 120 feet below grade. The piles were spaced in a grid pattern at 7-feet on center. As each pile was drilled, a 1-1/4 inch diameter double corrosion protected Dywidag anchor bar was inserted from the tip elevation to approximately 3 feet above grade. High-pressure grout injection was then used to construct the bottom 52 feet (approx.) of each pile with 4,500 pounds per square inch (psi) concrete grout. As the grout was injected, the auger was slowly withdrawn while rotating counterclockwise to force the grout downward. Upon completion of the high strength grouting and after letting the structural grout cure, the upper portion of each pile was filled with a weak, 500 psi grout mixture to grade.
- With the ACIP piles in place, the top 15 feet of soil overburden was excavated and removed from the caisson construction area to unload the site. At this point, the contractor removed the upper portion of the Dywidag anchor bars from the piles to clear the area for soil excavation during the caisson sinking process.
- A 3-inch thick by 5-foot wide non-structural concrete ring slab was poured as a working surface for erection of form work for the caisson cutting shoe. This working slab was intended to be sacrificial and provided a level, smooth surface to begin caisson construction.
- The caisson cutting shoe was formed and poured including the first 9-feet of the 3-foot thick caisson wall section. As explained previously, the cutting shoe contained a 45 degree bearing surface at the interior of the wall section, was constructed 6–inches wider than the wall section to form an annular space to aid the sinking process, and included a 6-inch bench at the interior wall surface to provide bearing for the base slab.
- The first 17-foot high wall section was formed and poured above the cutting shoe. At this point, the caisson broke through the ring slab and began sinking under it own weight, and stopped near the top of the angled portion of cutting shoe at a depth of approximately 2 feet below grade.
- The second 17-foot wall section was then formed and poured. During pour, the caisson again began to sink under its own weight. As the pour progressed, the caisson was observed to tilt (Fig. 5). Upon investigating the issue, an isolated area of frost under the cutting shoe was observed. The frost apparently was caused by insufficient insulation that allowed the January cold to penetrate the soils near the caisson wall. To correct the tilt, the contactor first excavated and removed the piece of frost from beneath the cutting shoe, and began excavating within the caisson by removing soils from near the frost location. The

decrease in soil mass resulted in lower friction forces at the interior wall of the caisson, thus prompting the "high" side of the caisson to sink faster than the "low" side. This process was continued and monitored until the tilt was corrected.

Fig. 5. Photo showing early tilt of caisson

- Soil excavation and removal from inside the caisson continued using a small backhoe near the walls and a crane with a 3-yard clam bucket. This process continued until the top of each caisson section was sunk to within 2 feet above existing grade (Fig. 6).

Fig. 6. Photo showing excavation inside caisson.

- The third caisson wall lift was then formed and poured and the excavation and removal of interior soils continued until the caisson was sunk to the required depth. At this point in the construction process, the contractor had to develop a means of stopping the caisson at or slightly below the target base slab elevation of 763 feet. To accomplish this task, the contractor used the PVC pipe system embedded in the caisson walls to inject a cement grout mixture to fill the void between the exterior caisson wall surface and the surrounding soil mass. Upon completion of the grouting process, it was determined that the caisson over-sunk by approximately 15 inches.
- The 37 ACIP piles were then trimmed to the desired length and non-destructive ACIP pile integrity testing was performed using sonic/ultrasonic pulse echo measurements. This testing verified the length of each pile and indicated that no weak zones were present in any of the 37 piles tested.
- Caisson base slab reinforcement was then placed, and the 6-foot-thick concrete base was poured with an integral sump and associated floor drains.
- With the base slab poured, the fourth and final wall section was then formed and poured. This final wall section was an additional 15 inches high to correct for the over sinking of the caisson, and to level the top of the caisson wall and address any minor tilt that remained from the sinking process.

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