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Case History - Hyperbolic Cooling Tower Foundation

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SYNOPSIS
This paper summarizes the design considerations and construction techniques employed during the construction of two hyperbolic cooling tower foundations. Included are descriptions of the site conditions, foundation loading requirements, drilled pier design techniques, construction coordination, and foundation installation.

INTRODUCTION
Two 640 MW fossil fueled electric generating units have recently been added to the Crystal River Generating Station near the Gulf of Mexico in Florida. These units use natural draft hyperbolic towers to fulfill their cooling water requirements. The foundation design required unusual considerations due to the Karst topography on which these units were constructed. The foundations for these towers employed cast-in-place drilled pier foundations to support the heavy loadings from the veil, and utilized the Vibroflotation technique to stabilize and densify the subgrade for the support of the basin and fill portions of the cooling towers.

SITE CONDITIONS
Crystal River Units 4 and 5 are located near the Gulf of Mexico adjacent to the town of Crystal River in Citrus County, Florida. The structures for the station are built on a typical Floridian Karst topography. Previous to design and construction, subsurface investigations at the site identified numerous solution cavities and loose sand-filled pockets usually encountered in formations of this type.

The subsurface stratigraphy consisted of a thin surficial overburden overlying lime rock of the Inglis Formation. This formation contained the majority of the solution cavities and is prone to the formation of sinkholes due to construction activities, increased loading, and changes in ground water levels. Underlying the Inglis Formation is a more competent dolomitic limestone of the Avon Park Formation, located approximately 40 to 70 feet below grade. The Avon Park Formation also contains solution cavities and soft seams; however, they are not as extensive and the Avon Park Formation is not as prone to developing sinkholes. The ground water level at the site is approximately 5 feet below grade.

FOUNDATION LOADING
The hyperbolic cooling towers at the Crystal River Station are 428 feet high and 350 feet in diameter. The veil is supported by precast columns at 40 support points or plinths. The basin within each of the cooling towers contains a constant 8-foot deep pool of water and provides the foundation for the columns supporting the tower fill.

The size of the cooling tower and the design wind velocities required to withstand hurricanes common to Florida resulted in substantial foundation loadings. The maximum compressive loading at the 40 plinths is approximately 1,800 tons with a corresponding tensile loading across the diameter of the tower of 900 tons. The basin loadings are 8 feet of water from the pool and maximum additional loadings of 3,000 psf over a 10-foot square area from individual fill support columns.

FOUNDATION DESIGN
The foundation loadings for these structures were supported by a combination of two techniques.

1. Drilled cast-in-place piers at plinth locations for support of the heavy veil loadings.
2. Deep compaction and stabilization of the overburden soil and the upper 30 feet of the Inglis Formation to support the basin and fill column support loadings.

Drilled Piers
The 4-foot diameter cast-in-place drilled piers were selected as the primary load-carrying elements for heavy foundation loads. The foundation system was selected as the result of a study in which foundation grouting of the Inglis Formation and drilled piers were evaluated as...
to cost and suitability to the subsurface conditions. Because of the numerous solution cavities in the Inglis Formation, the piers were designed as rock sockets deriving all their support in the Avon Park Formation. A total of 248 piers were installed at this site for both cooling towers.

The allowable pier design capacity was based on the results of unconfined testing of representative core samples recovered during the subsurface investigation program. The allowable bearing pressure used was one seventh of the average unconfined compressive strength of all the rock samples. No sample tested had a strength lower than this value. The allowable frictional resistance of the rock socket selected was one fifth of the allowable bearing pressure. The maximum allowable compressive loading was 600 tons with a corresponding tensile load of 300 tons per pier.

**Vibroflotation**

The Vibroflotation technique was selected to perform the in-situ stabilization within each tower. The Vibroflotation technique utilizes a vibration source—an electric motor driving eccentric weights—at the tip of a solid pipe to perform the deep compaction. The probe is advanced by water jetting and, in this situation, by the hammering or digging action of the vibrating tip, to penetrate the sand and lime rock and collapse and fill the solution cavities. The use of this technique instead of drilled piers to support the basin and fill support columns reduced the foundation costs by eliminating drilled piers in the tower interior, except for support of the erection crane in the tower center. This allowed the basin to be designed as a mat on grade instead of spanning pier caps.

The Vibroflotation technique was selected over other methods of deep compaction for the following reasons:

1. The technique allowed continuous monitoring of the compaction.
2. The technique had minimal interference with other construction activities in the area.
3. The technique allowed collapse, fill, and stabilization of all solution cavities within the depth of concern below the fill support columns.

**Foundation Installation**

The foundation installation was continuously monitored by a Black & Veatch geotechnical engineer during construction. The foundation construction was organized to avoid interference between pier installation and basin stabilization. The foundation operation and final site grading were in progress simultaneously.

**Drilled Piers**

To determine the location and length of the rock socket within the competent dolomitic limestone of the Avon Park Formation, and to verify that no caverns or unsuitable bearing material existed below the socket of the drilled piers, pilot holes were drilled within one or more piers at every plinth support group. These pilot holes were advanced using the rotary wash drilling method until the top of the Avon Park Formation was encountered (identified by drilling rate). The pilot hole was then advanced using a NWM core barrel with a maximum run length of 5 feet. The cores were retrieved and examined before termination of the pilot hole to ensure that the pilot hole was drilled deeply enough to establish the proper socket length on location.

The piers were drilled using crane-mounted drill attachments with telescoping kelly bars capable of drilling to 140 feet. The piers were advanced through the Inglis Formation with a 50-inch diameter auger equipped with replaceable steel cutting teeth. The oversized hole was advanced to the top of the Avon Park Formation where permanent steel casings were set as required, preventing sloughing of the excavation walls and loss of concrete in the solution cavities in the Inglis Formation. The rock sockets were drilled using a 42-inch diameter core bucket equipped with replaceable carbide-faced cutting teeth. The rock removed during construction of the rock socket was examined by the geotechnical engineer prior to concreting to verify the results of the pilot hole.

Concreting was performed using the tremie method. No attempt was made to dewater the pier holes to minimize disturbing the surrounding area. Immediately before concreting, after cleanout, the water in the base of the pier was agitated by pumping water to the base of the pier to suspend any remaining sediment.

During construction of the piers, numerous sinkholes resulted due to collapse of solution cavities in the Inglis Formation. These sinkholes extended a maximum surface radius of up to 30 feet, with cracks in the surface extending to a radius of up to 70 feet. The sinkholes were temporarily stabilized by backfilling with sand to grade before basin stabilization.

**Vibroflotation**

The Vibroflotation technique was employed at the location of all fill support columns, at pipe support locations, surrounding all piers, and in the location of all sinkholes that developed during the drilling of the piers or pipe holes. Vibroflotation probe patterns were established in a test program to provide stabilization of the tributary area below all load points and were extended to enclose all subsidence areas.

The Vibroflotation technique was successful; however, some damage to the probes did occur and some probes were replaced. Larger capacity probes were used during the basin stabilization of Unit 5 cooling tower (28 ton instead of the 18 ton used in Unit 4), reducing the rate of probe damage.
Compaction and densification were achieved by backfilling with a cohesionless material (pit run sand from a nearby borrow pit), and withdrawing the probe. The bottom water jets were shut off during withdrawal and top flushing water was introduced. The combination of the flushing water and the vibrating tip formed a very dense column of compacted sand at the probe location and increased the density of the native sand between probe locations.

Monitoring of the compaction was performed by two methods. All compaction probes were equipped with continuous strip chart recorders attached to ammeters to monitor the current draw of the electric motor in the probe tip. The current draw correlates with the increasing density of the backfill and surrounding native sand. Depending on the probe type, a minimum current draw was required before the probe could be withdrawn. The probe was withdrawn in 1-foot increments. Verification borings were drilled, taking Standard Penetration Tests continuously on treated areas and in all suspect areas, to verify that an acceptable level of compaction had been achieved. The basin was brought to final grade using conventional earth fill and compaction techniques.

PERFORMANCE

The cooling tower veil for Unit 4 was completed in August 1981. The tower has since been completed, and the unit was placed in commercial operation in December 1982. The maximum settlement noted at any plinth has been 1/4 inch, with differential of 1/8 inch between consecutive plinths.

ACKNOWLEDGMENT

The successful completion of the foundations for these structures in difficult foundation conditions was greatly facilitated by the experience and support provided by the joint venture of the Case International and Millgard Corporation during pier installation, and by the Vibroflotation Company during basin stabilization.