International Conference on Case Histories in Geotechnical Engineering
(2004) - Fifth International Conference on Case Histories in Geotechnical Engineering

14 Apr 2004, 4:30 pm - 6:30 pm

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Proceedings: Fifth International Conference on Case Histories in Geotechnical Engineering
New York, NY, April 13-17, 2004

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ABSTRACT

The replacement of a steam generator at a nuclear power plant was accomplished with the use of a massive crane, believed to be one of the largest available cranes in the world market. Four steam generators were replaced, with each generator weighing 3.2 MN. The crane used for the job had a circular support that required a ring foundation. The crane support imposed huge loads on the ring foundation. Because of the presence of soft to medium stiff clayey fill on one side of the ring foundation, the load-sensitive nature of existing underground safety-related duct banks, and the nature of the loads applied to the foundation, it was necessary to support the ring foundation on piles. In order to minimize vibrations next to the containment building (the plant was operational while the piles were being installed), micropiles were selected. A total of 80 micropiles were installed, most of them at a batter of 10^{o} from vertical, to provide the required vertical and lateral load resistance. Despite significant challenges imposed by space constraints, subsurface conditions, buried utilities, and design changes, the micropiles were installed within schedule, and performed as designed.

INTRODUCTION

Steam generator replacement at nuclear power plants is a regulatory-mandated activity. The replacement can be accomplished in different ways. In this case history, the old generators were picked up and removed through holes opened in the top of the containment building, and the new generators were lowered into place through the same holes. This approach requires the use of very large cranes positioned in close proximity to the containment building for lifting the old and new generators. Four steam generators were replaced, with each generator weighing 3,200 kN. The crane used for the job was believed to be one of the largest available cranes in the world, and the height to the tip of the fully extended boom was over 100 m. The crane had a circular support on 36 jack stands that required a ring foundation. This support imposed huge vertical (compressive), tangential (torsional), and transverse (lateral) loads on the ring foundation. The crane loading resulted in compressive design loads of up to 1,600 kN on individual piles under static conditions and as much as 3,200 kN when seismic loading was included.

This case history includes descriptions of the site conditions, geology and seismicity, and subsurface and ground water conditions at the ring foundation location. A discussion of the foundation system selection process is presented along with details of the selected foundation system (micropile). Two micropile installation procedures are described, one for micropiles extending beneath existing, operational, underground utility lines, and the other for micropiles that did not extend beneath such lines. Layout modifications arising from installation difficulties are described, and a comparison of installed micropile lengths and anticipated micropile lengths is presented.

RING FOUNDATION AREA CONDITIONS

Site Conditions

The ring foundation area was located adjacent to and north of the auxiliary building, and to the northwest of the Unit 1 reactor building, as shown on Figure 1. At the time the borings were drilled, this area was asphalt covered and formed the approach to a loading dock located on the east end of the service building (west side of the ring foundation). The area had a slight slope toward the loading dock, from about El. 214.6 m to El. 214 m. A refueling water tank (reactor water storage tank [RWST]) was located adjacent to and to the east of the ring foundation area. The RWST foundation was higher than the ring foundation area, and a reinforced concrete retaining wall about 2 m high separated the ring foundation area from the RWST. A RWST duct bank crossed the area in a northeast-southwest direction on the southeast quadrant of the area. An essential raw cooling water (ERCW) duct bank also crossed the area in a north-south direction near the center of the area. These two duct banks remained in place and operational during micropile installation, while all nonessential duct banks and pipes were removed and/or relocated before the installation. Following the removal/ relocation of nonessential duct banks and pipes, the ring foundation area was graded to about El. 213.8 m, from which all micropiles were installed. The ring foundation had to be designed

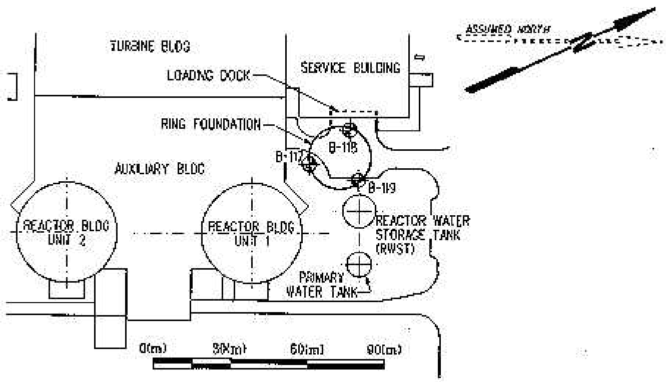

Fig. 1. Ring foundation and boring locations.

to bridge both the ERCW and the RWST duct banks at two locations each.

Site Geology and Seismicity

Bedrock beneath the power plant site belongs to the Middle Cambrian age, and is comprised of interbedded limestone and shale. Where it is unaltered by weathering, the shale is dark gray, banded, and somewhat fissile. The limestone is predominantly light gray, medium-grained to coarsely crystalline to oolitic, with many shaly partings. As a result of the presence of a thrust fault and a major overturned anticline beneath the site, the limestone and shale bedrock rests on dolomite bedrock, which normally overlies it. Movement of the fault and fold has caused the limestone and shale bedrock to become highly folded, complexly contorted, and cut by many small subsidiary faults and shears. The general strike of the beds comprising the limestone and shale bedrock is N30^{o}E and their overall dip is to the southeast, but the distortions created by the folding and faulting have resulted in many local variations to this normal trend.

Unconsolidated deposits overlying bedrock in the plant site region generally consist of residuum derived from weathering of the underlying bedrock, high-level alluvial terrace deposits, and recent floodplain alluvium.

Based on the information provided in the Updated Final Safety Analysis Report (UFSAR), an initial Safe Shutdown Earthquake (SSE) producing a maximum horizontal ground acceleration of 0.18g and a maximum vertical acceleration of 0.12g was determined as the seismic design basis for the plant. At a later date, the need to develop a site-specific response spectrum resulted in the consideration of an SSE with a peak horizontal ground acceleration of 0.22g for reevaluation of the plant design. The Operating Basis Earthquake (OBE) acceleration is 0.09g. This acceleration was initially based on the regulatory requirement of one-half the design basis SSE acceleration of 0.18g. This OBE acceleration value was subsequently justified for the present site-specific SSE acceleration of 0.22g.

Site Subsurface Conditions

The locations of three borings drilled in the ring foundation area (B-117, B-118, and B-119) are shown in Figure 1. These three borings disclosed subsurface material layers that can be classified as clay fill, residual soil (clays and silts), weathered bedrock and bedrock, as illustrated by the subsurface profile shown in Figure 2. The thickness of the clay fill ranges from zero to about 5.5 m. The fill thickness increases towards the auxiliary and reactor buildings, and reflects backfilling against the below-grade walls of the auxiliary and reactor buildings. The clay fill is generally a high plasticity, soft to medium stiff material, probably originating from on-site excavations performed during the plant construction. The fill is underlain by about 7.6 m of residual, stiff to very stiff clay and loose to medium dense silt. Slightly weathered, strong limestone was encountered beneath the residual soils. The highly folded nature of the bedrock suggested that the top of bedrock could be highly variable over short distances.

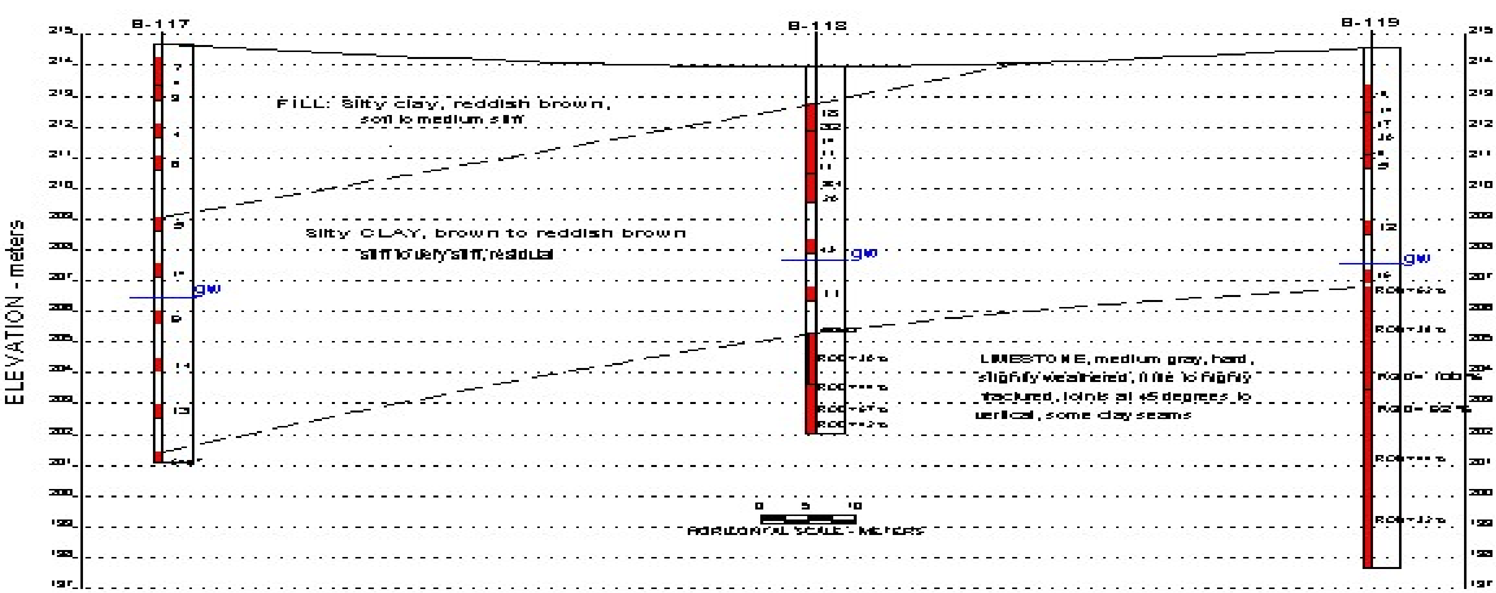

Fig. 2. Subsurface profile .

Ground Water Conditions

The ground water elevation in the Unit 1 reactor building area, based on the data provided in the UFSAR, was at about El. 210.3 m. This level results in a depth of about 4.6 m below the plant grade elevation (El. 214.9 m), or about 3.5 m below grade in the ring foundation area during micropile installation (El. 213.8 m). Ground water level measured in an observation well installed in boring B-118 was El. 208.3 m.

Ground water observations made inside micropile casings were consistent with these measurements.

FOUNDATION TYPE SELECTION AND DETAILS

Bearing capacity and settlement analyses using the crane loading and the subsurface data disclosed by the borings indicated that the ring foundation would require pile support. Driven piles were ruled out because of vibration concerns, and also because of lack of space for the large pile driving equipment that would be required. Thus, a system of drilled piles socketed into the bedrock was recommended. The piles could be of typical diameter (a 380-mm diameter pile was recommended), or of a small diameter with longer rock socket (a 203-mm diameter micropile was recommended).

A Drilled-in-Pile Specification was issued, which included the ring foundation size and loads (compression, tension, lateral, and torsion) to be used by the successful subcontractor in designing the piles. Engineering design parameters of subsurface materials were also provided to the subcontractor for their use in designing the drilled pile system. The successful subcontractor recommended the use of small diameter piles, i.e., micropiles.

The subcontractor developed a micropile layout consisting of 71 piles to be installed at a 10^{o} batter (68 micropiles) and vertically (3 micropiles) to resist the specified foundation loads. The micropiles were laid out along three concentric circumferences of radii equal to 10.2 m (inner ring), 10.8 m (middle ring), and 11.4 m (outer ring). Analyses performed by the subcontractor disclosed maximum compressive loads on the order of 1,600 kN per micropile under static conditions and 3,200 kN per micropile under seismic conditions.

Based on the anticipated maximum design loads, each micropile was designed to consist of a 245-mm outside diameter, 14-mm thick, carbide-toothed, steel casing drilled to competent bedrock. A 203-mm diameter socket would be opened from the bottom of the casing at least 6 m into competent rock using a downhole compressed air hammer. The length of rock socket was calculated using an ultimate bond stress of about 1,400 kPa between grout and rock. A 12-m long, 64-mm diameter, 1,030-MPa ultimate strength steel rebar was grouted into the socket using a 28-MPa-strength cement grout. The rebar extended up to 6 m into the casing, which was also filled with the 28-MPa-strength grout. The design included the assumption that the micropile resistance to the foundation loads was derived from the rock socket, and the casing acted as a load transfer element with negligible side resistance. The micropile design incorporated a minimum factor of safety of 2 for static loading, and a minimum factor of safety of 1 for seismic loading.

Figure 3 shows the as-built micropile locations and includes nine micropiles that were added during the installation phase.

MICROPILE INSTALLATION

Micropiles were installed using a Casagrande C8 rig with a 16 kN-m maximum torque. Micropile installation started on October 11, 2002 and ended on November 18, 2002. The subcontractor worked 30 days installing the micropiles, resulting in an average of about 3 installed micropiles per day.

General Installation Procedure

The general installation procedure described below was followed for all micropiles that did not extend beneath the existing duct banks.

a) Drill to refusal into the rock using the casing. Water was pumped inside the casing to wash the spoils to the surface.

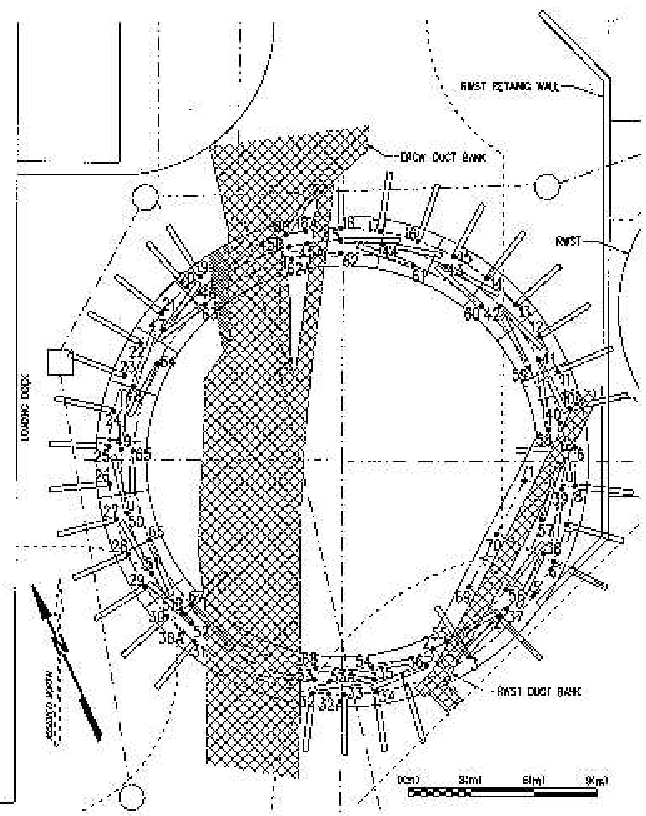

Fig. 3. As-built micropile locations.

The water and spoils were returned to the surface through an annulus created between the outside wall of the casing and the surrounding soil. This drilling procedure allowed no control of the return water and spoils. However, the driller continuously monitored the return water and spoils, and no fluid loss was observed during drilling of any micropile in the soil.

b) Extend the socket at least 6 m into competent rock using a pneumatic hammer that also uses a water jet. Hammering would reduce the rock to pieces generally no larger than about 6 mm that would be returned to the top of the casing by action of the compressed air and water in the hammer. The rock response to hammering was also used as a guideline for terminating the casing. The casing would often be drilled below the level achieved in Step a) above to the top of rock determined to be competent based on how the rock responded to hammering. In cases where seams of soft/broken rock were encountered below the casing refusal depth, the socket was extended beyond 6 m to compensate for the length of the soft/broken rock seam. The driller continuously monitored the hammering of the socket, and no voids were detected in the bedrock during hammering of the socket of any micropile. However, it was occasionally observed during hammering and/or drilling that water and spoils would return to the surface through the annulus of neighboring micropiles, indicating possible cracks in the rock and/or pervious seams in the soil.

c) Tremie grout the micropile to the top of the casing, and place the socket rebar. The socket rebar was kept centralized in the hole by means of three equally spaced centralizers. A measure of the volume of grout was provided for each micropile. Because a flow meter was not used, the volume of grout was estimated based on the number of cement bags used to grout each micropile. Grout cubes were collected periodically for strength testing.

Installation Procedure for Micropiles Extending Beneath Duct Banks

Step a) of the foregoing installation procedure was modified for micropiles that extended beneath the existing duct banks. A few versions of the modified procedure, called duplex drilling, were attempted until the procedure described below was selected.

- Drill to refusal into the rock using the casing. The casing was drilled dry and a drag bit was then used inside the casing to pump water and wash the spoils to the surface. The water and spoils were returned to the surface inside the casing, thus keeping the return water and spoils controlled throughout the drilling process. The driller continuously monitored the return water and spoils to verify that no fluid loss was occurring during drilling. This was deemed necessary as an additional step to ensure that no undermining of the critical safety-related duct banks would occur.
- Same as before.
- Same as before.

The vibrations caused by hammering the socket were barely perceptible to someone standing a few feet from the hole.

Micropile Layout Modifications

A few modifications were made to the originally intended micropile layout during the course of micropile installation. The modifications were as follows in chronological order of installation:

- An obstruction was encountered by Micropile 30 (refer to Figure 3 for as-built locations) at a depth of about 4.5 m. The obstruction could not be exposed by excavation for identification and was not shown on existing drawings. There was some evidence that the obstruction could be a piece of concrete or rock in the backfill placed against the below-grade wall of the auxiliary building. Boring B-117 drilled in the same general area had to be relocated slightly after hitting an obstruction at a depth of about 5.5 m. Micropile 30 was eventually installed within its location tolerance (about 125 mm) and at a steeper batter (13.5°) than originally planned.
- A shallow obstruction was encountered by Micropile 19 (refer to Figure 3 for as-built locations). The obstruction was exposed by excavation and was identified as a lateral extension of the ERCW duct bank that was not shown on existing drawings. Micropiles 19, 46, and 63 were then relocated to avoid this obstruction.
- The relocation of Micropiles 19, 46, and 63 caused the ring foundation northern bridge over the ERCW duct bank to become wider and require additional micropiles. Five piles (18A, 18B, 45A, 45B, and 62A) were installed through the gap in the ERCW duct bank. Micropile 18B was relocated slightly and installed at a 10^o batter away from the center of the ring foundation to avoid the ERCW duct bank (several attempts were made at installing this micropile vertically at its originally proposed location). Micropile 62A was installed at a slight batter toward the center of the ring foundation to avoid interference with Micropile 62. Another four micropiles were installed for the ring foundation southern bridge over the ERCW duct bank (Micropiles 30A, 32A, 51A, and 53A). Micropile 30A was installed vertically, Micropile 51A was installed at a 10^o batter toward the center of the foundation, and Micropiles 32A and 53A were installed at a slight batter to avoid interferences with previously installed micropiles. The total number of as-built micropiles was increased from 71 to 80 as a result of adding these piles.
- Micropile 9 was located in the field, as recommended on the drawing. This micropile was installed at a 10° batter away from the center of the ring foundation, as opposed to the 10° batter toward the center of the OLS foundation indicated on the original drawing.
- Rock was encountered at its shallowest depth at the locations of Micropiles 10 and 11. Because these micropiles were located near the RWST retaining wall and the RWST duct bank, it was necessary to ensure that the casing had indeed hit bedrock, and not some man-made obstruction. Based on top of rock profiles developed from previously installed micropiles, it was believed that the casing was

- indeed hitting bedrock. In order to obtain further evidence, the hammer was used to break into a few inches of the hard material and provide samples for visual identification. Both micropiles were then installed as planned after the samples collected from hammering were visually identified as rock.

It is noted that the events described in the previous bullets were handled in a timely fashion. The subcontractor was able to temporarily move the drill rig to other micropile locations and proceed without delays.

<u>Installation Records</u>

The subcontractor recorded installation logs detailing each micropile. Information contained in the logs included micropile number, installation date, grouting date, volume of grout, depth to and quality of bedrock, length of casing, and length of socket. Micropile installation profiles derived from these logs are shown on Figures 4 though 6. The profiles on Figures 4 through 6 indicate the following:

- The top of rock elevation was consistent with the information disclosed by the borings, except on the northern side of the foundation where the top of rock elevation was lower than could have been predicted by interpolation between Borings B-118 and B-119. This is evidenced by the plot on Figure 4 where information from Borings B-117, B-118, and B-119 is included. The rock elevation in Borings B-118 and B-119 is very close to the top of rock disclosed by the micropile rig. No rock was cored in Boring B-117, and thus a direct comparison with the micropile rig may not be very accurate.
- The rock sockets were consistently formed below a top layer of soft/broken rock. This top layer tended to be thicker on the southern side of the foundation. This could reflect the effects of rock blasting for construction of the auxiliary and reactor buildings.

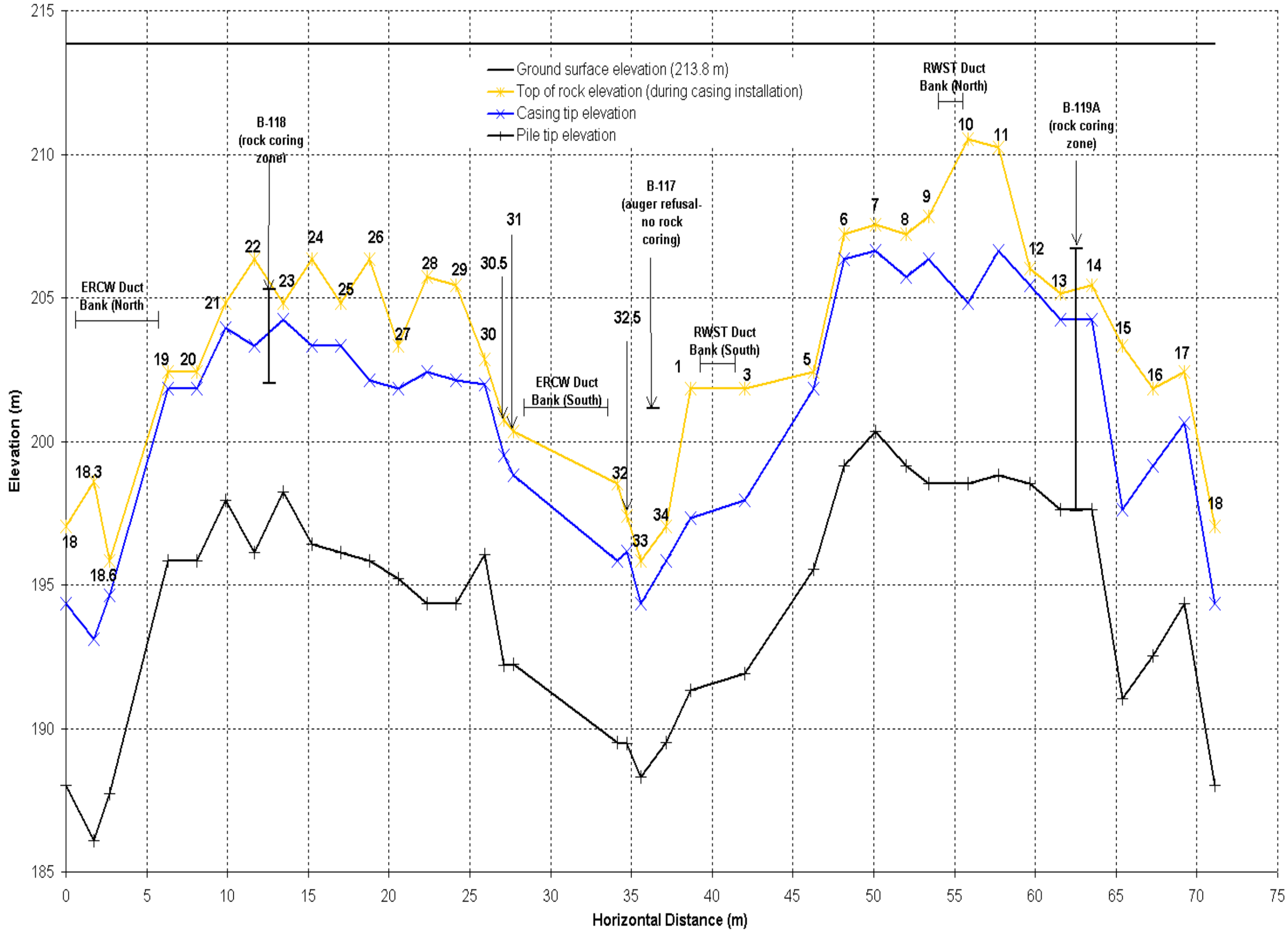

Fig. 4. Micropile profile along the outer ring.

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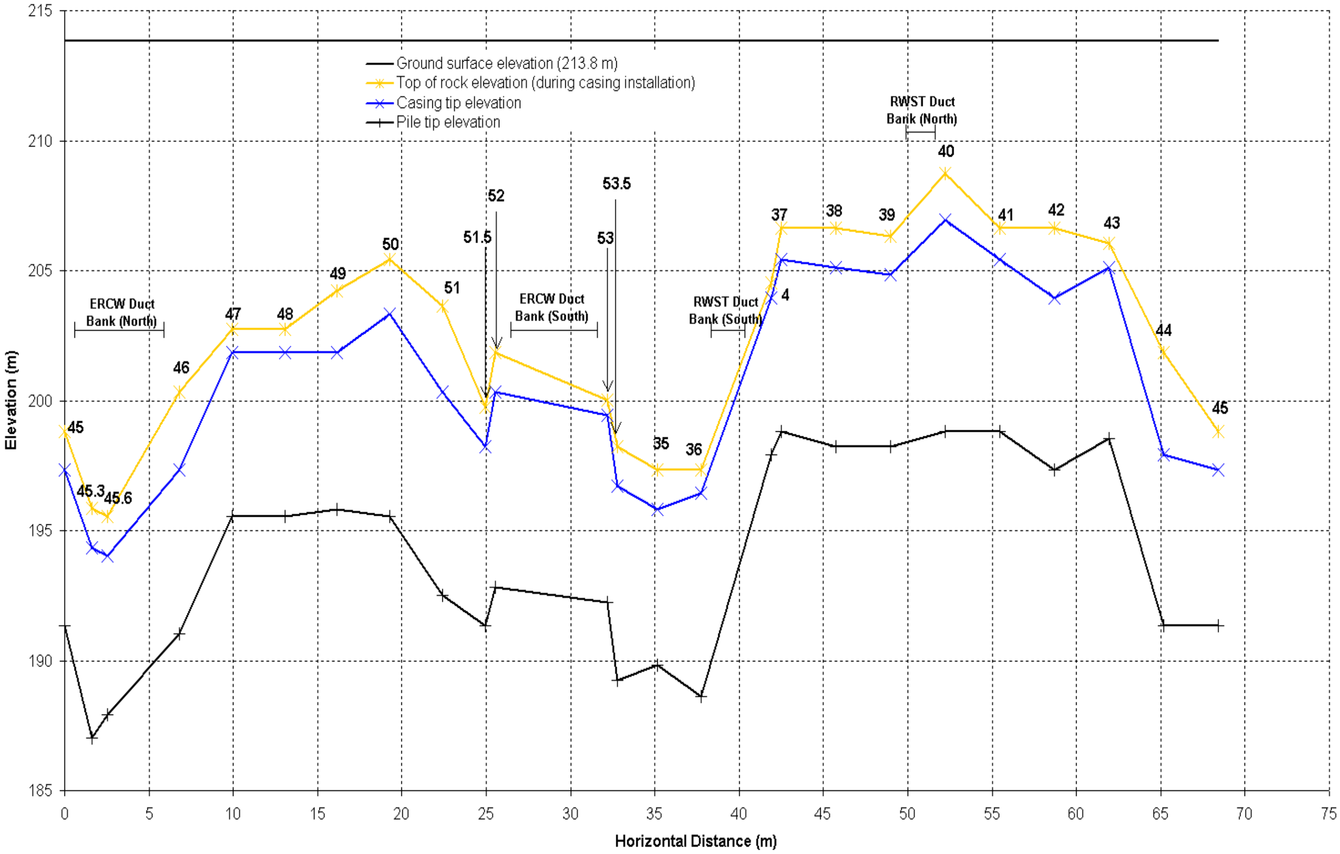

Fig. 5. Micropile profile along the middle ring.

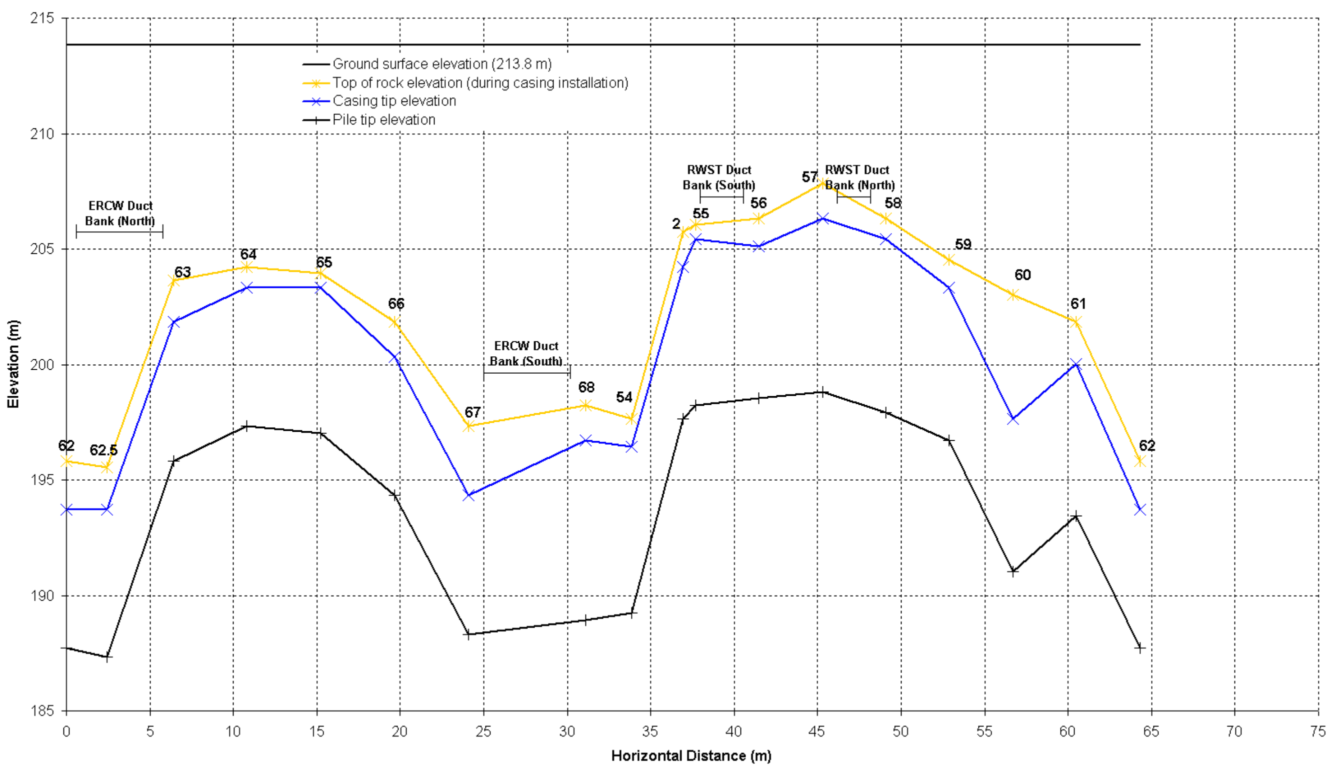

Fig. 6. Micropile profile along the inner ring.

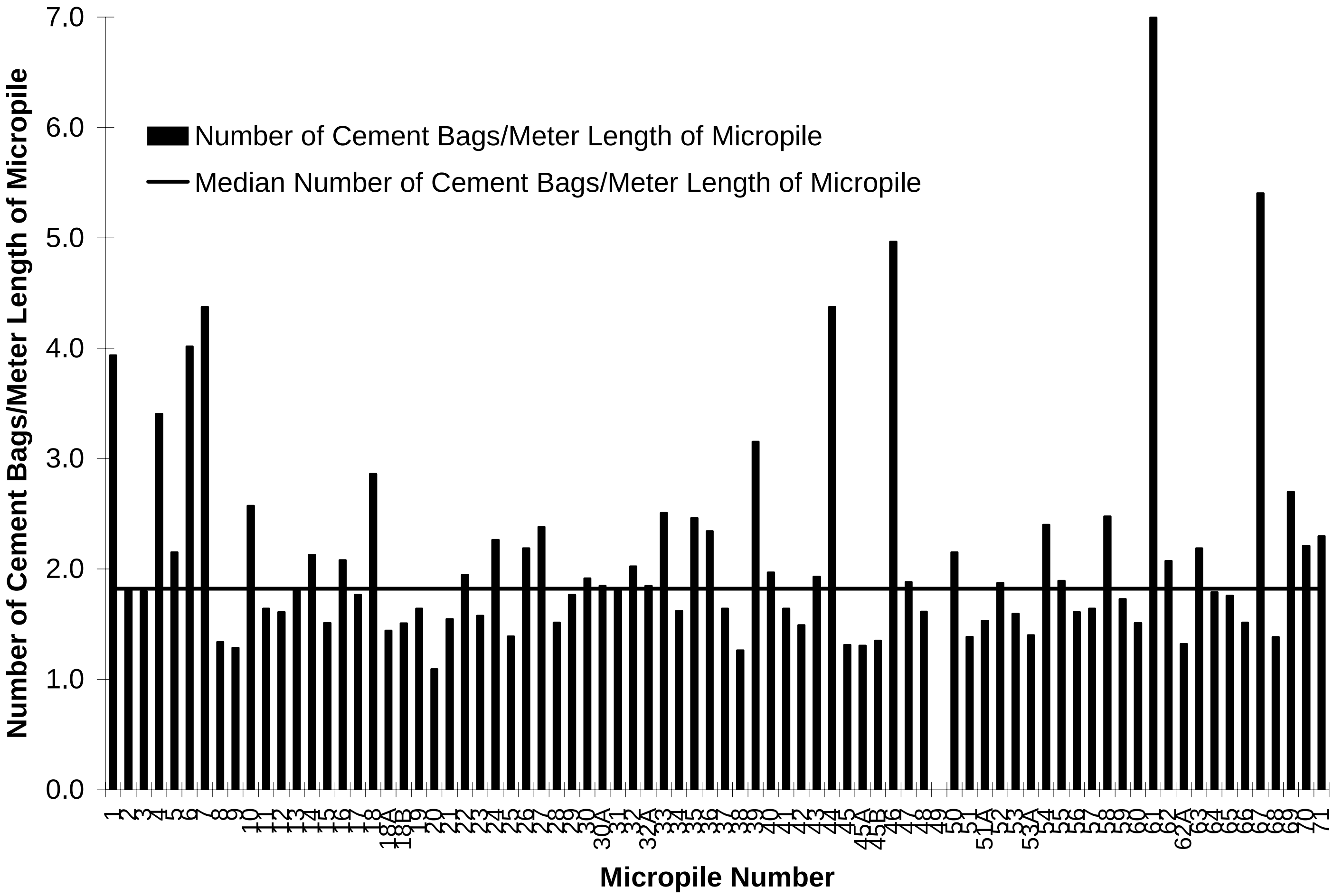

Fig. 7. Micropile grout take.

A summary of grout takes for each micropile is shown in Figure 7. It is noted that the grout take for Micropile 49 was not recorded. The graph in Figure 7 shows a few micropiles that required a volume of grout/foot length of micropile much greater than the median value shown. These micropiles tended to be those installed in areas where significant broken rock was observed, or where water and spoils were observed returning to the surface through the annulus of neighboring micropiles. An illustration of large grout take is provided by Micropile 39. Water in the casing of Micropile 57 was observed to flow out of the casing as Micropile 39 was grouted. This clearly indicated some connection between these two micropiles. Micropile 57 had to be redrilled before grouting, as it was observed that its socket had been completely filled with grout from Micropile 39.

Grout cube breaks disclosed grout strength of over 56 MPa, i.e., more than twice the design grout strength.

SUMMARY AND CONCLUSIONS

The micropiles were installed successfully within schedule, in accordance with the specification and design assumptions, and using industry-wide acceptable equipment and methodology. Work progressed smoothly with minimal stoppages or breakdowns despite the installation and design change challenges described previously and the fact that both nuclear reactors remained operational during micropile installation. The steam generator replacement was successfully accomplished in March-April 2003.