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ABSTRACT

In June of 1994 a 20-m section of 1.4-m diameter, restrained-joint, ductile iron pipe failed during construction of a new section of water pipeline for the city of Cairo in the Arab Republic of Egypt. The failure occurred in an area where the pipe was supported on piles, and compacted silica sand was used as side support for the pipe. Soil above the crown of the failed section of pipe was 6 m or more in thickness. Results of a detailed review of the failure revealed that a number of unique and related factors apparently caused the failure. The most significant of these causes was the native soil surrounding the pipeline, which was formed from an accumulation of 800 years of building and construction debris. At the location of the failure the debris was in excess of 15-m thick. When subjected to water at this location, this debris underwent significant settlement, which eventually led to loss in side support for the pipeline. To repair the pipeline and to avoid future similar failures, a utilidor was used to protect the pipeline in areas where overburden thickness was greater than 4.5 m, and a pipe encasement was used where the overburden thickness was less.

KEYWORDS

Pipeline, Piles, Failure, Egypt, Debris-fill, Sand, Settlement, Earth Pressures, Construction, Utilidor, Post-Repair Performance

INTRODUCTION

In June of 1994 a 20-m section of a 1.4-m diameter water pipeline for the Rod El Farag Water Distribution System failed during water pressure testing. The failure occurred less than 3 months before final commissioning of this \$100± million (US) water system upgrade for the city of Cairo in the Arab Republic of Egypt. The failure was attributed to unique soil conditions existing at the site. Implications of the failure were serious: it brought into question the entire design of the pipeline system, as well as the foundation support system for three 80-m diameter, cast-in-place, post-tensioned concrete water reservoirs, which had recently been constructed near the area of failure. This paper describes geotechnical investigations that were carried out to investigate the cause of the pipeline failure, the suitability of pile-supported structures at the site, and the repair procedures that were completed to put the pipeline back into operation and to prevent future similar failures.

Project Description

The Rod El Farag Water Distribution System Upgrade Project involved installation of 18 km of restrained-joint, ductile iron pipe in the mid and north sections of the East Bank of Cairo (Fig. 1). The project was constructed to provide drinking water to this heavily populated and rapidly growing area. Funding for the project was from the governments of the Arab Republic of Egypt and the United States; project administration was being handled by the General Organization for Greater Cairo Water Supply (GOGCWS) and the United States Agency for International Development (USAID); CH2M HILL International, in association with two Egyptian firms, Dr. Ahmed Abdel Warith and United Consultants, designed the upgrade and were providing engineering and construction management services at the time of the failure. The construction contractor was Morrison Knudsen.

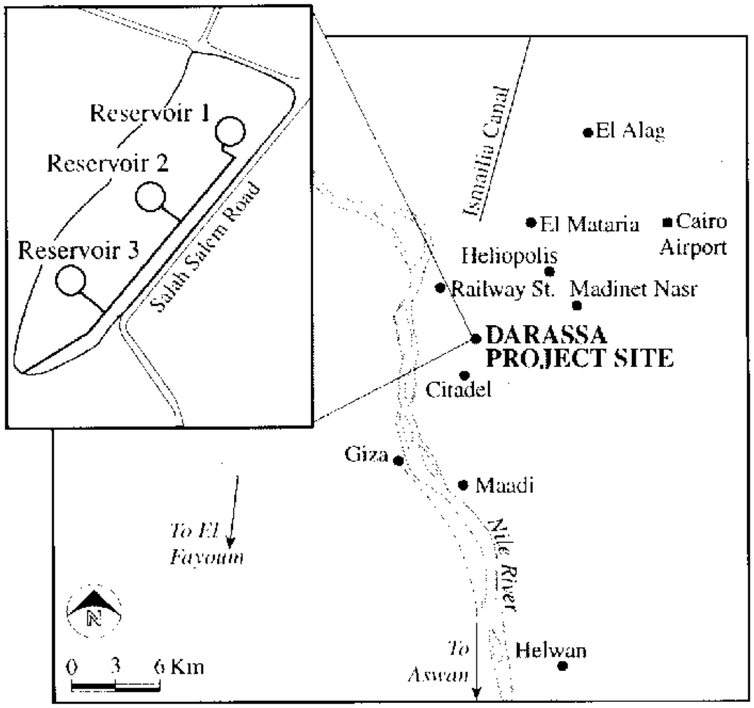

Fig. 1 Map of greater Cairo.

The pipeline failure occurred near one of the newly constructed 80-m diameter reservoirs. Each reservoir has a 1.4-m diameter pipeline that connects it to a 1.4-m diameter transmission main. The distance from the reservoirs to the transmission main ranged from 70 to 130 m. A 10-m by 15-m altitude valve vault (AVV) is located between each reservoir and the transmission main to adjust water line pressures. The pipe failure occurred along a 20-m section at Reservoir 1 between the AVV and the reservoir, where approximately 6.5 m of fill were located above the crown of the pipeline.

Geotechnical Conditions. Geotechnical conditions in the area of the failure were reported by United Engineers as consisting of 15 to 25 m of dry fill over a dense sand layer. Locally, the dry fill is referred to as the Darassa fill. It consists of gravel, sand, silt, crushed brick, pottery, bones, and traces of organic material. The Darassa fill is thought to be "building debris" that was discarded over an 800-year period just outside of the Old Wall of Cairo, after the wall was constructed in the 12th century. Blowcounts from standard penetration tests (SPTs) in the fill typically range from 15 to 20 blows per 30 cm and are up to 40 blows per 30 cm near the bottom of the fill. Direct shear test values for the fill were reported to range from 37 to 43°. The sand below the Darassa fill is dense to very dense in consistency with SPT blowcounts in excess of 50.

Groundwater is located 12 to 15 m below the top of the dense sand. The material in the upper 2 to 4 m of Darassa fill appears to be very dry becoming somewhat moist at depth with moisture contents of 30% or less.

Foundation Systems. Recognizing the heterogeneous characteristics of the Darassa fill, the designers required that all major structures, such as the AVVs and the 80-m diameter reservoirs, be on cast-in-place deep foundations supported in the dense sand below the Darassa fill. The piles were located in the dense sand layer to avoid localized settlement, which was expected to occur when voids and loose areas within the Darassa fill settled, shifted, or collapsed.

In the case of the pipelines, piles were required where fill thickness above the crown of the pipe was in excess of 3 to 4 m. The pipeline support system consisted of a 2-m wide by 0.4-m thick pile cap located on 0.4-m diameter Delta piles, a proprietary driven/cast-in-place concrete pile commonly used in southern Europe and north Africa. The Delta piles were located in a two-bent configuration with center-to-center spacing of 1.2 m and bent spacing of 4.5 m. The piles were designed as end bearing piles. Allowances were included in design for downdrag from the settlement of the Darassa fill. The capacity of the Delta pile was confirmed during the early phases of construction by conducting High Strain Dynamic Tests (HSDT) with an energy of approximately 20 KJ and a static pile-load test. Ultimate geotechnical capacities of the Delta piles were estimated from these tests to be greater than 3 MN which exceeded the original design capacity.

The pipeline was located on a 30-cm thick sand bedding placed on the pile cap. The width of the pipe trench was typically two pipe diameters beyond the edge of the pile cap; trench walls were normally excavated at near vertical to heights in excess of 7 m. Backfill placed in the trench around the pipeline was an imported clean silica sand compacted to 95% of its maximum dry density determined by standard Proctor methods.

The reservoirs and AVVs were supported on 0.6-m diameter drilled shafts with the toe of the shafts located at least 3 m into the dense sand bearing layer. HSDTs and static pile-load tests were conducted on a limited number of these drilled shafts during the early phases of construction. The mobilization energy of the HSDT was approximately 40 KJ. Ultimate geotechnical capacities of the shafts were estimated from these tests to be greater than 5.5 MN, exceeding the original design capacity.

A rigorous inspection program was followed during construction to assure that a high-quality upgrade project would result. Construction monitoring included continuous on-site inspection by the designer's engineers, frequent laboratory testing of concrete and sand products, and close conformance to the design drawings and specifications.

Pipeline Failure

During inspection of AVV 1 in June of 1994, water was observed to be trickling through a pipe penetration in the wall of the AVV. In an effort to determine the source of the water, the 1.4-m pipeline between Reservoir 1 and AVV 1 was drained and inspected. During that inspection, the engineering staff observed that the diameter of the pipe had deformed downward from 10 to 20%, and the diameter at the springline of the pipe had increased in a similar amount. These measurements indicated that the pipe had ovalled, the extent of which was sufficient to break the seal at the pipe joints.

POST-FAILURE EVALUATIONS

Following discussions with USAID and GOGCWS representatives, the pipelines between the three reservoirs and the AVVs and between the AVVs and the transmission main were exposed (1) to determine the extent and potential cause of the failure between Reservoir 1 and AVV 1 and (2) to investigate conditions of the other pipelines that had been constructed under similar conditions. Test pits were dug below the bottom of the pile cap at the failure location to inspect conditions immediately below the pile cap. The interiors of the pipes for Reservoirs 2 and 3 were also inspected to determine if they had deflected in a manner similar to what was observed at Reservoir 1.

Observations after Excavations

The ovalling of the pipe at Reservoir 1 was clearly visible when the backfill was removed (Fig. 2). The deformations of the ovalled pipe sections were sufficient to cause the concrete lining of some of the most heavily deformed areas to crack and fall away. The pile cap was also extensively damaged over a 23-m distance, with the Delta piles typically punching up through the pile cap in a number of locations (Fig. 3). The sand fill that was removed in this area was very wet (e.g., moisture contents in excess of 50%), relative to when it was placed and relative to the surrounding Darassa fill.

Perhaps the most interesting observation made after the pipeline and pile cap were exposed in the area of failure was the nearly 30 cm void that existed between the bottom of the pile cap and the top of the original fill. This void occurred after construction, as the pile cap had been cast on a mud slab

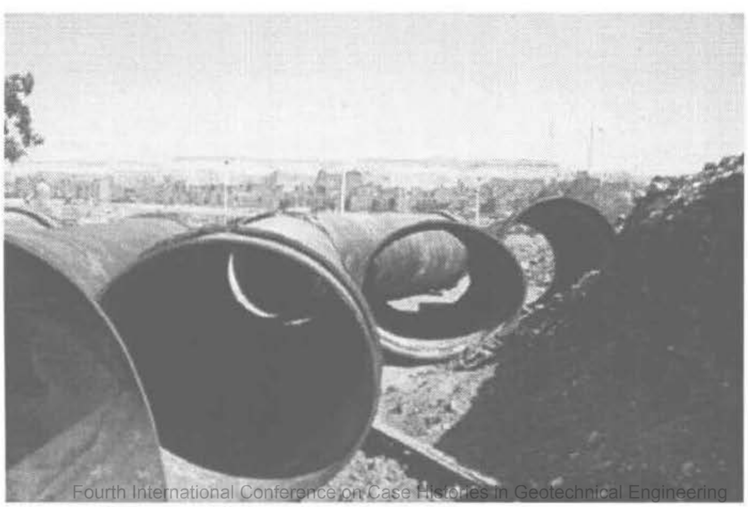

Fig. 2 Damaged 1.4-m diameter pipes.

Fig. 3 Punching shear failure of Delta pile through pile cap.

that had been poured on the fill. The settlement was not directly related to the height of overburden. For example, next to Reservoir 1 where the height of overburden was nearly 7.5 m, the void was less than a few centimeters. There was also no damage to the pipe at this location. The sand fill surrounding the pipeline at this location was relatively dry, similar in moisture content to its original placement condition.

A second revealing observation was also made in the area of the pipe failure. Along with the settlement, there was as much as 17 cm of silica sand between the bottom of the pile cap and the top of the mud slab. This silica sand was the same sand that had originally been placed as trench backfill. Laminations of irregular thickness were visible in the sand, suggesting that the sand had flowed from the trench into the void on more than one occasion. It was later hypothesized that the sand had been washed into the void by water that had accumulated in the trench when a connector pipeline leaked (Fig. 4).

Inspections between AVV 1 and the transmission main, where the height of soil above the crown of the pipe ranged from 4.5 m to 1.5 m, found no obvious signs of damage to the pipeline or pile cap, although from 30 to 47 cm of settlement had occurred beneath the pile cap at one location. Results of the inspection of the other pipelines are summarized in Table 1. The maximum deflection of the pipeline at these other locations was less than 5%.

As was also observed in the area of pipeline failure, backfill conditions were generally wet in areas of maximum settlement, relative to their placement condition. A review of field records determined that numerous cases of water leakage had been recorded by the construction inspection staff in the vicinity of the settled areas. The cause of the leakage ranged from leaking utility lines for construction support facilities to leaky connectors on the pipelines. For example, at Reservoir 1 the source of the leak was a faulty saddle connection between the 1.4-m pipeline and a 100-mm drain pipe; at Reservoir 3 the area of maximum settlement coincided with one of the contractor's temporary utility lines.

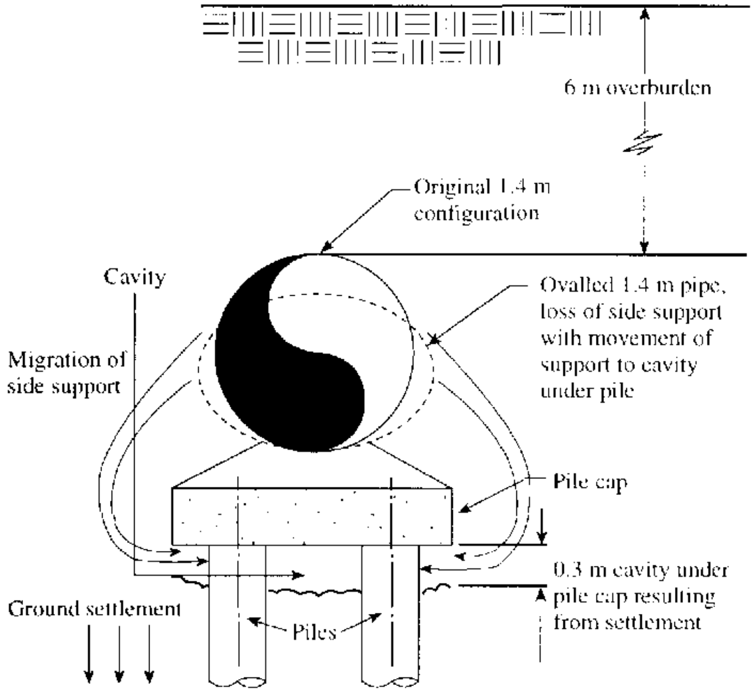

Fig. 4 Hypothesized Pipe Failure Mechanism.

With the exception of the actual area of failure, the amount of silica sand found beneath the pile cap in other areas where settlement had occurred was minimal, suggesting that the amount of water flow at these other locations had not been sufficient to wash the sand from the trench into the void beneath the pile cap.

An elevation survey was also conducted on the pile cap after the pipelines were exposed. Results of this survey indicated that the settlement of the pile cap for the undamaged sections of the pile cap was negligible. On-the-other-hand, there was clear evidence of settlement of the pile cap in the areas that had failed. Although the movement of the damaged section was thought to be due to the punching shear failure resulting in the pile cap moving downward relative to the piles and not due to pile movement, the movement did introduce the possibility that the Delta piles might have plunged downward some unknown amount during the pipeline failure.

Geotechnical Assessment

The post-failure geotechnical assessment included geotechnical explorations, laboratory testing, pile integrity testing, pipeline capacity evaluations, and a reanalysis of pile capacities for all pile-supported structures.

The geotechnical explorations included five borings drilled by hand-auger bucket methods (Fig. 5) through the Darassa fill approximately 1 to 2 m into the underlying dense sand. Modified penetration tests using a 100-mm outer diameter by 83-mm inner diameter split-spoon sampler driven by a 64 kg hammer were conducted at approximately 1-m intervals in the Darassa fill; SPTs were conducted in the dense sand underlying the Darassa fill. The objectives of these explorations were (1) to determine the consistency of the fill and sand in areas where maximum settlement had occurred and (2) to collect representative samples of soil for laboratory classification testing. Undisturbed block samples of the soil were also obtained from the test pits dug below the pile cap.

In general results of these explorations were very consistent with those performed as part of the original design. Blowcounts from the modified penetration tests and the SPTs were similar to those recorded previously. Laboratory tests confirmed that the Darassa fill was a mixture of gravel, sand, silt, and clay-size materials with from 50 to 80% sand-size or coarser. Water contents for samples ranged from less than 10 to as much as 50%, with the highest values being recorded in the vicinity of the pipeline break. Consolidation tests conducted on specimens trimmed from the block samples suggested that vertical strains would be up to 3% when water was added after consolidating the samples to the estimated overburden pressure.

After removal of the damaged pile cap, pile integrity testing was conducted on the 0.4-m diameter Delta piles by Pile Testing of Egypt to determine if the piles between Reservoir 1

Table 1. Summary of settlement measurements and damage survey.

Location	Overburden Height (m)	Maximum Settlement beneath Pile Cap (cm)	Damage
Reservoir 1			
Reservoir to AVV	7.5 to 5.0	30	Failed pipe and pile cap
AVV to Transmission Main	4.0 to 1.5	47	None
Reservoir 2			
Reservoir to AVV	7.5 to 5.0	< 1 to 2	No damage
AVV to Transmission Main	4.0 to 2.0	30	No damage
Reservoir 3			
Reservoir to AVV	7.5 to 1.6	40	Minor hairline cracks to pile cap
AVV to Transmission Main	1.6	25	No damage

Fig. 5 Hand-auger bucket soil boring.

and AVV 1 had been damage by the failure. Results showed that all but one pile was intact. The damaged pile had compressive and tensile cracking in the upper 1 m. Subsequently, the upper 1 m of this pile was removed and replaced.

Axial capacity analyses were conducted to confirm that the plunging, downdrag, and structural capacities were sufficient, in light of the overburden loads and the potential for large downdrag loads. The Unified Method of Design described in the Canadian Foundation Engineering Manual (CFEM, 1985) was used in performing these analyses, similar to the original design. With the Unified Method, downdrag is assumed to be mobilized under very little relative movement between the pile and soil, and undoubtedly much less than would occur at the Darassa site. The only question in these analyses was the Beta value to use for the pile or shaft resistance factor. After review of the available pile-load test data and discussions with Professor Bengt Fellenius, a consultant to CH2M HILL on the project, an average Beta of 0.2 with a range of 0.15 to 0.3 was selected for the Delta pile. For drilled shafts a Beta value of 0.3 with a range of 0.25 to 0.35 was used. The toe resistance factors, N_t, were 60 and 30 for the Delta pile and drilled shaft, respectively. The different Beta and toe resistance factors for the Delta piles and the drilled shafts reflected the different construction methods.

For the pipeline and pile capacity reanalysis, overburden load computations accounted for positive and negative trench configurations recommended by Spangler and Handy (1982) and within the U.S. Army Corps of Engineers (1978). A $C_p = 2$ was used, which was consistent with the upper-bound value that was necessary to cause failure for a fixed-base condition with the outside soil settling. Results of these analyses confirmed that the factor of safety for plunging of the Delta piles exceeded 3, and the load factor for downdrag was greater than 1.2. The structural capacity to load ratio exceeded 1.2, where the load was defined as the dead load plus the downdrag load.

For the reservoirs, which were supported on 0.6-m diameter drilled shafts, load-deformation analyses were conducted using results of previously performed pile-loads tests and HSDTs to confirm that the capacities of the drilled shafts were consistent with the original design. These reanalyses determined that factors of safety for plunging and downdrag of the drilled shafts were greater than 3 and 1.7, respectively. The structural capacity had a load ratio of greater than 1.7.

It was concluded from these analyses that the design of the Delta piles and the drilled shafts met normal requirements for the safe design of important structures.

Structural Assessment

The structural assessment included review of the load carrying capacity of the pile cap, as well as structural inspections and testing. Particular focus was placed on the allowable overburden for the pipeline.

Results of this review determined that, if the soil loads above the pile cap were based on twice the weight of the prism of soil over the pile cap (a conservative assumption of $C_p = 2$) and the pipe was filled with water, the pile cap, by itself, was suspect to support 7 m cover over the pipe. According to American Concrete Instituted Building Code Requirements for Structural Concrete (ACI, 1995), the maximum overburden load for punching shear with $C_p = 2$ would have been only 3.5 m, rather than the 6.5 m observed over the pipe. Observations at Reservoirs 2 and 3 indicated no damage. The discrepancy between computed and observed behavior suggests either that a $C_p = 2$ was too high or that the stiffness of the 1.4-m diameter ductile iron pipe relative to the pile cap influenced the capacity computation. In this latter case, with the pipe being roughly 15 times stiffer than the pile cap, loads could have been redistributed. In all likelihood some combination of the two explanations probably occurred.

Results of pipe coupon tests determined that the load carrying capacity of the ductile iron pipe, which was required to exceed a 60-42-10 criteria (60 ksi tensile strength; 42 ksi yield strength; and 10% minimum elongation), met or exceeded requirements. It was also concluded that if the soil surrounding the pipe exceeded 90% relative compaction according to the standard Proctor test, the pipe could support 9 m of overburden using a conservative assumption of soil load (C_p = 2), but that if the pipe lost side support, it could only support approximately 3 m of overburden.

Failure Mechanisms and Implications

From the observations made after the pipeline excavations, the geotechnical assessments, and the structural analyses, it was concluded that the failure was the result of a combination of high overburden pressures, soil settlement, and water leakage.

The combination was apparently a unique occurrence from the standpoint that other sections of the pipe that involved similar combinations of overburden pressures, equal or greater soil settlement, and water leakage did not exhibit the failure.

The specific cause of failure was postulated to be as follows:

- A water leak at the saddle connection caused significant settlement in the Darassa fill below the base of the pipe trench, resulting in a cavity forming beneath the pile cap.
- The water saturated sand fill surrounding the pipe was slowly washed into the cavity below the pile cap, resulting in a progressive loss of side support along the springline of the pipe.
- As the side support was lost, the pipe deformed, eventually resulting in more water leakage from the water-tight seals at the pipe joints and further flowing of the sand fill into the void.
- The pile cap failed in punching shear when the pile cap-pipeline-soil system adjusted to a new state of equilibrium under the soil and water loads.

The implications of the failure seemed to indicate that only pipelines supporting overburden heights in excess of 3 m were of concern, as the pipeline and pile cap could support at least 3 m of overburden without side support. This led to the conclusion that only the sections of pipeline between the reservoirs and the AVVs were at greatest risk. It also led to the conclusion that the failure mechanism did not directly involve the piles themselves, and therefore, repair should focus on protecting the pipeline from future occurrences of excessive settlement in the Darassa fill caused by water.

FACILITY REPAIR AND MODIFICATIONS

Following completion of the post-failure evaluations, methods for repairing and modifying the facilities to handle consequences of large settlements of the Darassa fill were established. These repairs were based on the understanding that the Darassa site surrounding the water supply facilities would be shared with the new El Azhar Park under development by the Aga Khan Trust for Culture.

Initially, the repair program focused on the pipeline between the reservoirs and transmission mains. However, in anticipation of potential surface water infiltration and underground irrigation pipeline leaks from the new park, the extent of the repair and modifications was expanded to include most of the water facilities installed at the Darassa site. The criteria used in development and selection of the options included long-term usage of the facility, reliability, constructibility, and schedule of completion. These where essential considerations, given the plans for the future park, the need for complete approval by the owner, and the fast-track schedule that would be required to minimize disruption to the start-of-service date of the water facilities.

Alternative Evaluation and Design

The section of 1.4-m diameter pipe between each of the three reservoirs and the associated AVVs was of primary concern because of the failure near Reservoir 1 and the high overburden loads above the pipe and pile cap. The site and pipeline areas to be modified and/or repaired are shown in Fig. 6. From the post-failure analysis it was concluded that the current pile cap and pipe support design could not handle the combination of large overburden pressures and loss of side support. Alternatives were identified for repairing the pipeline and pile cap conditions from Reservoir 1 and modifying conditions from Reservoirs 2 and 3 to the transmission main.

Alternatives were narrowed to two potential options: (1) a reinforced concrete utilidor and (2) a concrete pipe encasement. A third option involving dual steel, welded wire fabric walls (commonly referred to as a Hilfiker wall) with a reinforced concrete top slab spanning between the walls was also identified as a cost-effective variation to the concrete utilidor. However, construction of Hilfiker walls was unproved in Egypt; therefore, the owner was unwilling to accept this approach, despite potentially significant cost savings. After further review and discussions with the owner, Alternative 1 was selected for the pipeline section from the reservoirs to the AVVs, where the height of soil over the crown of the pipe was greater than 4.5 m; and Alternative 2 was selected for other areas where overburden heights were less than 4.5 m and greater than 3.0 m.

Utilidor Alternative. The proposed concrete utilidor, Alternative 1, is shown in Fig. 7. The design concept for the utilidor was to use the concrete structure to carry the trench overburden loads around the existing pipe, pile cap, and piles to new strip footings located on either side of the pile cap. The design concept was also to allow the utilidor structure above the pipe to settle independently and articulate according to the ground movement without affecting the pipeline.

The utilidor consisted of precast concrete "U"-shaped sections each approximately 3-m long and each spanning approximately 4.6 m with a height of 4.1 m. The utilidor sections were designed such that they were not connected to the footings nor to each other, allowing differential movement to accommodate ground settlement.

Each utilidor segment was supported by a relatively flexible, lightly reinforced concrete strip footing 2-m wide and 0.5-m thick. To prevent backfill materials from entering between the segments, a loosely laid nonwoven geotextile covered the exterior face of the utilidor. In addition, the utilidor was developed with a manhole entrance at either end for human access to permit long-term visual inspection and maintenance

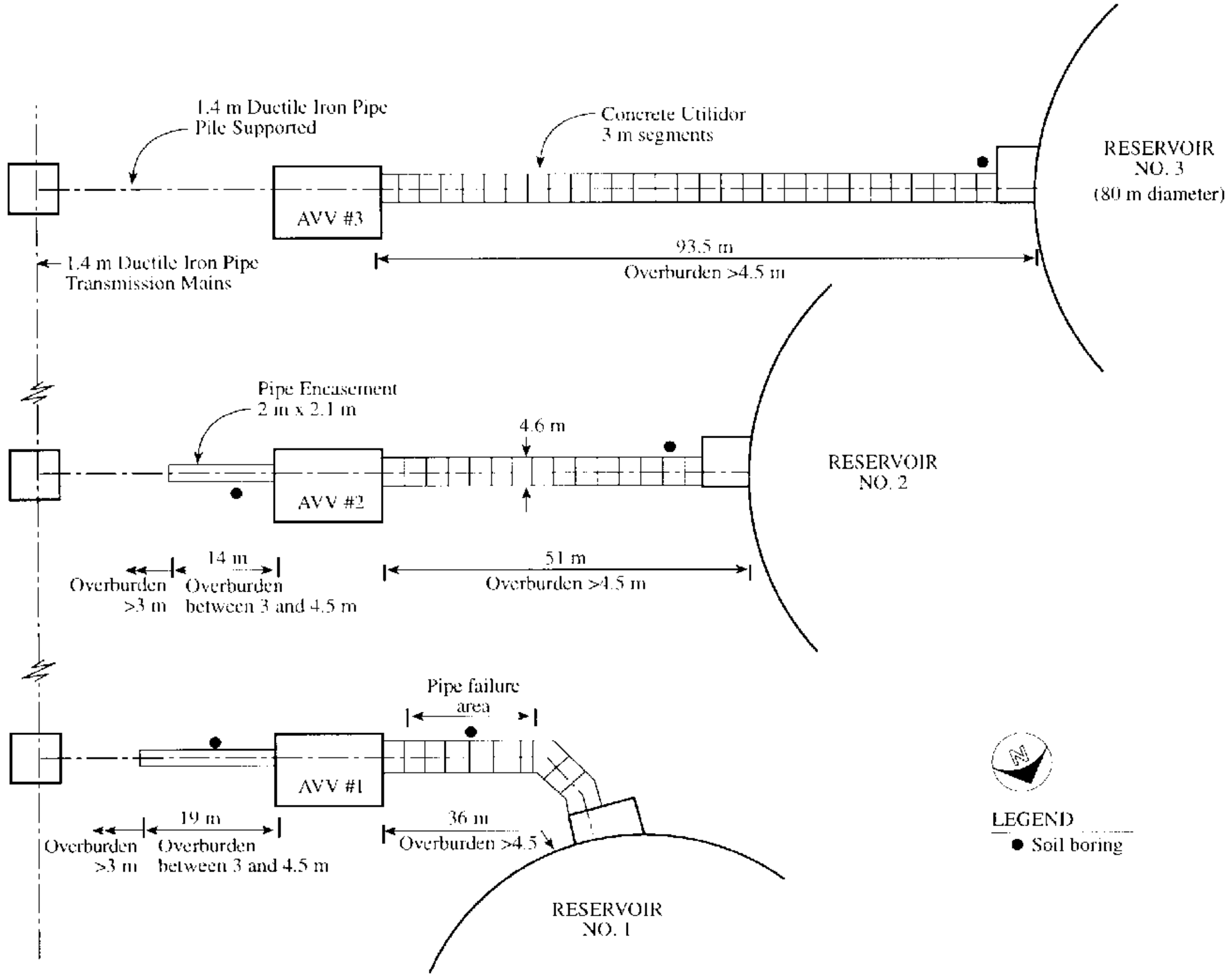

Fig. 6 Darassa site with three reservoirs, altitude valve vaults, pile-supported pipeline, and transmission mains.

of the pipeline. The owner viewed the ability to inspect the pipeline as a significant enhancement to the design.

Other appurtenances associated with the reservoir and piping system, including a 300-mm drainline, a 100-mm washdown pipeline, and instrumentation, control and electrical cables, were placed inside the utilidor. Again the owner viewed this location as beneficial from the standpoint of long-term maintenance. With this new design concept, the method of supporting the 1.4-m diameter pipe was changed from the original sand fill to a pipe cradle on top of the pile cap.

During review of the utilidor concept, concerns were expressed about the additional lateral loading to the Delta piles from the nearby utilidor footings. Concerns were also expressed about the potential for additional downdrag. Additional analyses were conducted to show that the pile system would perform adequately under these loads.

Pipe Encasement Alternative. The concrete pipe encasement alternative involved a reinforced concrete encasement to prevent ovalling of the pipe and provide beam rigidity and punching shear resistance to the pile cap. This alternative was determined to be economically feasible in areas where the soil thickness above the crown of the pipe was greater than 3.0 m and less than 4.5 m. In areas with higher overburden, additional piling would have been required to carry the combined weight of the soil, encasement, and water-filled pipeline. The cost of additional piles when added to the cost of the pipe encasement was estimated to be much more expensive than the utilidor concept.

The encasement was 2-m wide, matching the width of the existing pile cap and 40 cm above the top of the pipe. In addition, flexibility between the encased pipe and the pipe entering adjacent pile-supported structures was provided by leaving the last two pipe joints uncased and free to rotate if needed.

Other Considerations. Under each alternative, flooding of the trench adjacent to the pipeline system (i.e., pipe, pile cap, and piles) was recommended as an inexpensive method to induce subsurface ground settlement prior to reconstruction, which would reduce the amount of settlement occurring after

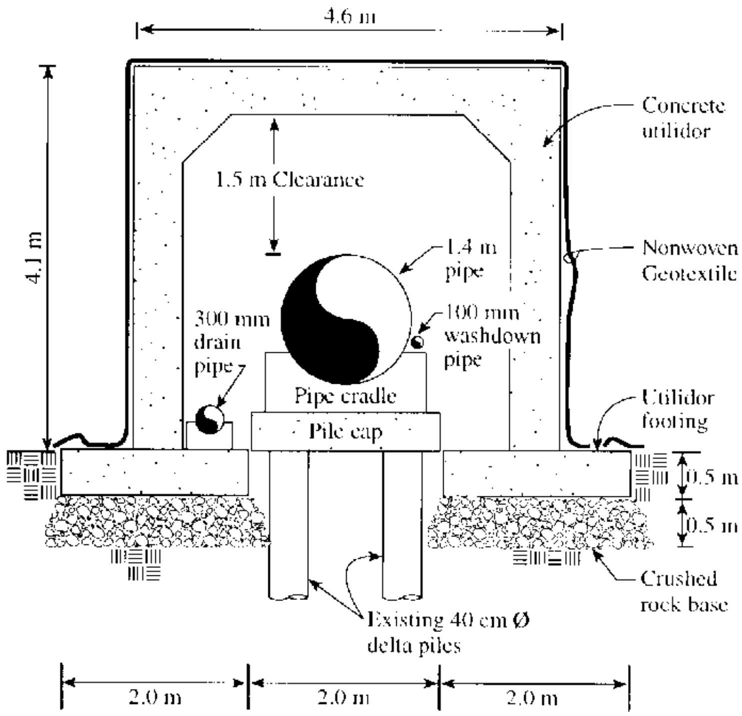

Fig. 7 Reinforced Concrete Utilidor Section.

construction. The amount of post-construction settlement was particularly important for the utilidor, as it was designed to accommodate a fixed amount of settlement before the utilidor would contact the pipeline.

The pipe cradle supports were constructed prior to flooding. Settlement of the trench ground surface, the pile cap, and adjacent pile-supported structures were monitored. After settlement of the flooded area had essentially stopped, the flooded area was dried and compacted, and construction of the utilidor and pipe encasement began.

Design Criteria

Design criteria for the repair and modifications were developed based on the alternatives selected for implementation. The criteria included the design site grading plan, as well as geotechnical and structural requirements for each alternative. In addition, criteria related to construction issues were established.

Overburden Loads. The design site grading plan, which was developed during the post-failure evaluation, was established to estimate the maximum expected soil loads that could be imposed to the utilidor or pipe encasement protecting the 1.4-m diameter pipe, pile cap, and complete system. This grading plan exceeded the actual contract grading plan by as much as 2.0 m.. It was set to accommodate the new park construction and any associated unforeseen fill and grading alterations that might occur in the future. At the highest point this plan suggested that soil overburdens could be as great as 7.3 m above the concrete utilidor, where the piping connected to the reservoir. In areas where the pipe encasement would be used, the maximum soil overburden used in design was 4.5 m. In areas in which the soil overburden was 3.0 m or less, no modifications to the pipeline system were required.

Geotechnical and Structural Criteria. Geotechnical and structural design criteria were established to determine the wall thickness and size of the utilidor and the pipe encasement. Analyses were also conducted to determine bearing stresses and settlements for the utilidor footings. These analyses were based on Darassa fill characteristics that had been evaluated during the post-failure evaluation and monitored over the past 4 years in the field during the site construction.

A maximum settlement of 1 m was estimated from (1) the maximum observed settlement at the ground surface from induced water during construction and (2) vertical strain measurements made during the laboratory consolidation tests. A considerable degree of engineering judgment was required in making this estimate, as the zone in which the observed settlements occurred was unknown and the representativeness of the laboratory data was uncertain. Given these uncertainties, a safety factor of 1.5 was established between CH2M HILL and the owner, resulting in the clearance between the bottom of the utilidor and the top of the pipe at 1.5 m.

Beneath the utilidor footing, a 0.5-m thick crushed rock base which is 0.5 m wider than the footing was placed to reduce the bearing stresses on the underlying fill, to maintain support as the fill beneath the cap settles, and to minimize the potential for squeezing of soil into the void beneath the pile cap in the event of future settlement.

Construction Criteria. Construction criteria included setting the sequence of construction. Pipes were to be supported by the pipe cradles on the pile cap prior to the flooding. Flooding duration at each location was to be monitored for settlement using settlement plates. Further construction was not permitted until the rate of settlement was negligible. Pile repairs were to occur for piles between Reservoir 1 and the AVV 1, as determined necessary by the pile integrity tests. This work was to occur on a fast-track schedule to minimize impacts to the start-up and turn-over of the facilities.

Construction

The construction work was awarded to an experienced local Egyptian construction firm, Sami Rizallah Contractors. Bids from the pre-design and final design ranged from a high of $6.6 million (US) by a U.S. based contractor, to the awarded bid of slightly under $600,000 by Sami Rizallah. The awarded bid was approximately $500,000 less than CH2M HILL's estimate. Skepticism on the completion and quality of work was apparent from the expatriate community, particularly relative to meeting an aggressive construction schedule and the low contract bid amount.

Notice to Proceed was given on February 1995. The actual construction duration was approximately 5 months, which was within the specified contract schedule. Work often proceeded in double shifts and 6 to 7 days a week.

The overall construction operation for the repairs and modifications was unique in Egypt, as the work was performed under a partnering agreement between CH2M HILL and the Egyptian contractor. Modification or rehabilitation construction work can result in premium associated costs due to unforeseen adjustments to the design and slowed construction progress. However, the project philosophy of partnering assured that the job was done as quickly and smoothly as possible. Day-to-day adjustments and revisions were made, as necessary, to the design by CH2M HILL with minimal impacts to the progress of the work and the Egyptian contractor. In many instances, design modifications were updated in Cairo during the day, electronically transmitted to the US for senior review during the night in Egypt (daytime in the US), and revised for construction the following day in Cairo. As the work progressed, the project developed many features similar to a design/build approach. CH2M HILL expedited the work by assisting the Egyptian contractor in the scheduling and administration of the daily work.

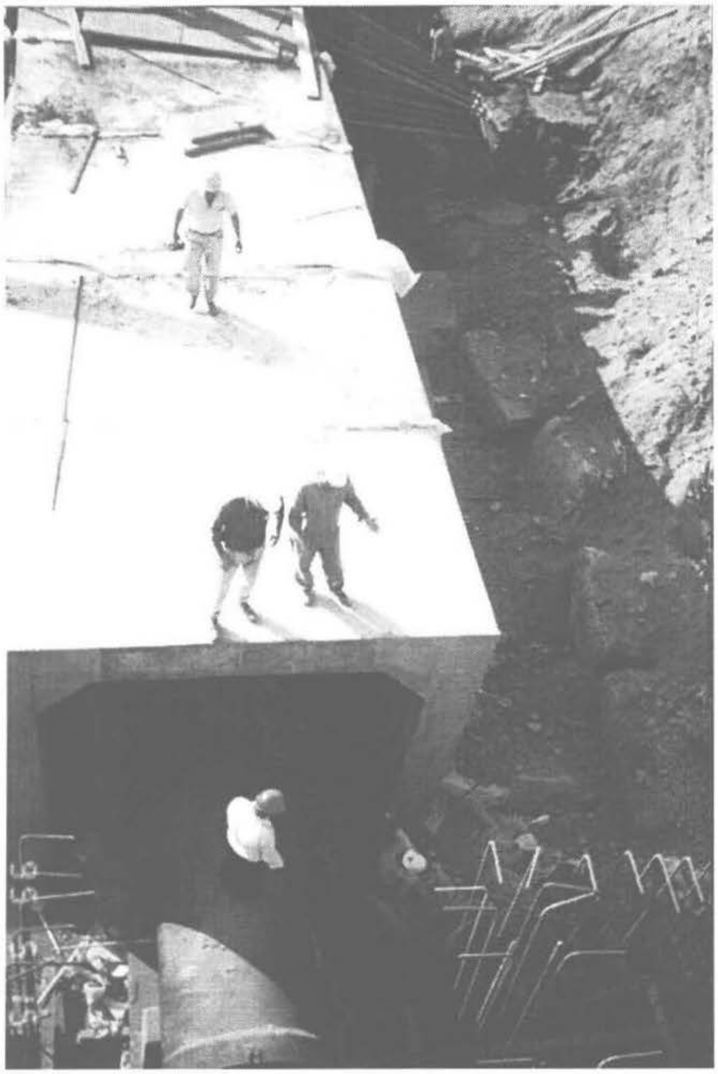

Fig. 8 Utilidor construction over 1.4-m diameter pipe.

During the work, the flooding and monitoring stage occurred within 7 to 10 days, with minor subsurface settlements occurring at Reservoirs 1 and 3 and no settlement at Reservoir 2. The utilidor and pipe encasement construction occurred in stages utilizing re-usable wooden gang forms (Fig. 8). Upon completion of the work and the proper concrete cure period, backfilling over the work took place, and the original construction grades were established. Settlement monitoring of the utilidor structures occurred throughout the backfilling operation.

As summer approached near the completion of the work, ambient air temperatures began to rise above 40°C during daylight hours. Concrete temperatures during placement where kept to a minimum by requiring that placement occur during night shifts and cooling of the aggregates and water prior to batching the concrete with engineering controls. Construction monitoring and quality control standards were maintained throughout the construction period. Independent laboratory testing was performed for the concrete and backfill placement operations.

The repair was completed within the 5-month construction schedule and within the $600,000 bid amount. Change orders requiring costs alterations to the contract were approximately 2% of the total contract value and no claims occurred.

Upon completion of the work, the pipelines and water supply system were tested and turned over to the owner. The completed water supply and storage system was put into service by the end of the summer of 1995 with all systems operating in accordance with the original design.

Post-Repair Performance

Since settlement of the utilidor was anticipated, a monitoring program was established within the utilidor structures. This monitoring program involved elevation surveys at multiple points on the utilidor and on the pile cap within the utilidor.

The first month after construction, the maximum settlements within the utilidors were as great as 26 cm, 6 cm, and 9 cm between the reservoirs and AVVs 3, 2, and 1, respectively. The post-construction settlements were typically greater at the higher fill areas (i.e., adjacent to the reservoirs with 5.5 m of fill over the top of the utilidor). However, settlements were variable at each location and along the pipeline length.

The settlements decreased rapidly within the next 6 months and after one year, virtually no measurable settlement was occurring. The total maximum accumulative settlement of a single utilidor segment was 30 cm for Reservoir 3, 7 cm for Reservoir 2, and 10 cm for Reservoir 1. Since practically all the post-repair performance settlements within the utilidors occurred rapidly, the settlement was attributed primarily to the adjustments of the fill material to the structural loads and construction activities, rather than settlement induced by water.

Over 1.2 m of clearance currently exists between the top of the pipe and the bottom of the utilidor. This clearance is thought to be sufficient for any future settlement that might be induced by water from irrigation of the park above the pipeline or from any future leaks in water lines.

CONCLUSIONS

Complex geotechnical subsurface conditions occur everywhere, the 800-year old debris fill at the Darassa site in Cairo, Egypt was no exception. In this case, the subsurface materials consisted of a heterogeneous material with an unique behavior when subjected to excessive water. For such conditions, efforts during the design process must be allocated for completing geotechnical explorations, material testing, and engineering evaluations, and for preparing design recommendations as were conducted.

However, unforeseen behavior can still occur, as did at the Darassa site. In this case the unforeseen behavior led to a pipe failure and repair process. Based on the results of a thorough review of the failure, a relative unique design solutions involving use of a flexible concrete utilidor was developed to protect the pipeline from future failures.

This case history demonstrates the importance of geotechnical explorations and design evaluations where complex geotechnical conditions occur. In addition, it demonstrates that careful and well-planned post-failure evaluations can lead to the use of innovative design alternatives. Also, use of cooperative construction techniques, such as the design/construction partnering arrangements for the utilidor construction, can be used in all parts of the world and can result in a successful conclusion for repair/modification or any type of design/construction work.

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