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Pile Driving Adjacent to Municipal Drinking Water Storage Facility

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PILE DRIVING ADJACENT TO MUNICIPAL DRINKING WATER STORAGE FACILITY

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ABSTRACT

An unexpected response occurred as piles were driven within 3 feet of the west wall of an existing municipal drinking water storage reservoir. Being located in a confined urban space, the expansion of the parking garage at a facility on the south end of Lake Michigan required the installation of 122 steel H-piles as close as 3 feet to the reservoir. Historically, structures on the site were supported on either shallow spread footings or H-piles driven to bedrock. At the contractor's suggestion, considerable project savings were achieved by driving the H-piles to an extremely hard clay layer ("Chicago hardpan") above the bedrock. Pressuremeter testing, and static and dynamic load testing of the H-piles were completed as part of the project testing program. Both the horizontal and vertical movements of the reservoir wall were monitored during pile driving.

The paper presents the design parameter changes, static and dynamic pile testing, and vibration monitoring for construction of the multi-level parking structure adjacent to the 8 million gallon drinking water storage facility. The vertical movements of the tank's west wall and the corrective actions taken after water began seeping from pre-existing cracks in the tanks wall are the focus of the case study.

INTRODUCTION

Expansion of a parking facility on the south shore of Lake Michigan required driving 122 steel H-piles as close as 3 feet away from an 80+ year old water storage reservoir. The response of the West Wall of the reservoir to pile driving was unexpected; the wall moved up rather than down.

All of the H-piles were to be driven to either the underlying dolomitic "limestone" or "Chicago hardpan." Existing structures at the site were supported on either spread footings or H-piles driven to the bedrock. The geotechnical engineer's recommendation was to support the new parking facility on H piles driven to bedrock.

At the pile driving contract's suggestion, considerable savings were achieved by driving the majority of the H-piles to bear in the extremely hard clay layer (Chicago hardpan) located above the bedrock. This reduced the length of piles by approximately 20 feet each and eliminated the need for pile rock tips, along with reducing the amount of very difficult driving.

Site layout

This site is located in an older urban/ industrial area in Indiana. Figure 1 shows the site, including the outlines of the old and new parking garages, as well as the existing water storage reservoir. This paper focuses on the area where the water storage reservoir and the new parking garage meet (Fig. 1).

The existing parking garage and office building are supported on HP14X89 H-piles driven to bedrock. These piles were chosen over drilled piers because of the possible construction difficulties associated with sand layers encountered in the clay and hard pan, and the proximity of the site to nearby steel mills. The site's historic geotechnical data indicated that there were discontinuities in the hardpan, although the project soil borings for the new parking garage did not reflect this. The previously installed piles on the South and East sides of the existing parking garage were located approximately 15 feet from the edge of the water reservoir walls; no adverse impacts had been observed after driving the existing parking garage H-piles to bedrock.

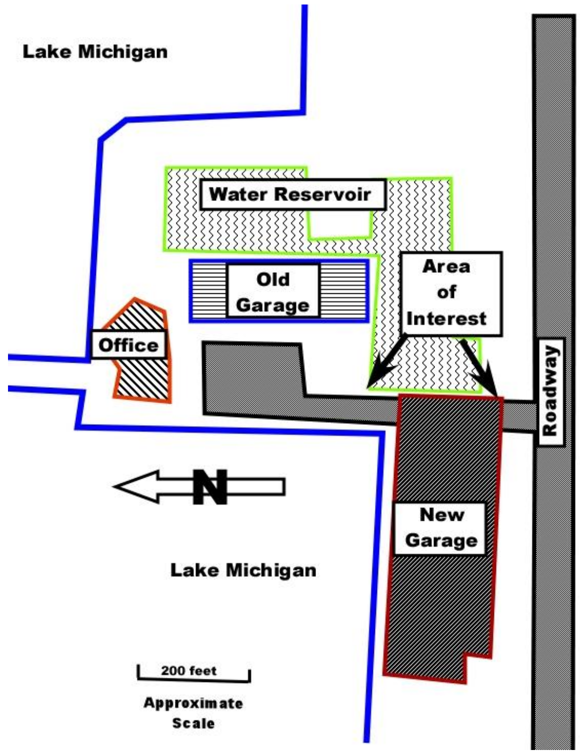

Fig. 1: Site Location.

Geotechnical Engineering

The site geotechnical conditions were well understood based on previous projects on and around the site, as well as regional work (Peck and Reed, 1954). Figure 2 shows a generalized cross-section of the site, including the relationship between the water reservoir and adjacent H-piles. The 96-foot deep soil profile includes 9 feet of fill, overlying 21 feet of fine silty sand (N = 9 to 26), 30 feet of very soft to stiff gray silty clay (Q_p<0.5 to 2.0 tsf, w_c = 20 to 34 %, N = WoH to 9), 25 feet of stiff to very hard silty clay (Q_p> 4.5 tsf , w_c = 8 to 14%, N = 39 to 115), and 5 feet of silty sand and sandy silt (w_c = 17, N = 100), and terminating in the Niagara Dolomite (RQD = 95 to 100 %).

As the new parking garage project grew in size and scope, settlement tolerances were reduced to 1/4 inch total and 1/8 inch differential, with settlement tolerances of 2 inches total and a one-inch differential originally specified, respectively. With column loads of up to 750 kips, together with the initial settlement tolerances, foundation support for the new garage was originally planned for spread footings. Column loads were later increased to 1600 to 2900 kips. Based on the higher loads, more stringent settlement tolerances, and the clients need for performance above cost savings, the foundation recommendations were modified to include driving H-piles to bedrock. However, in the "area of interest" adjacent to the water reservoir (Fig. 1), micropiles were recommended rather than the driven piles.

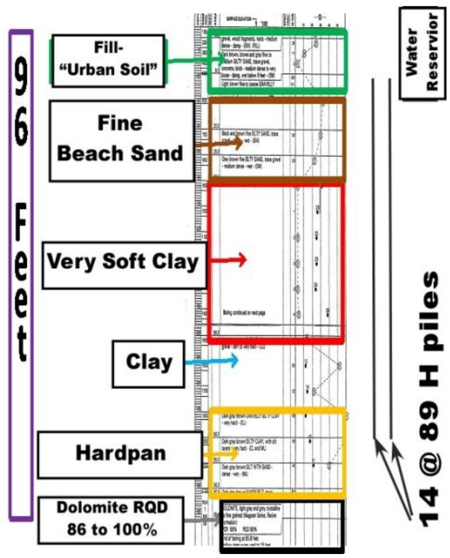

Fig. 2: Site Soil Cross-Section.

The successful foundation contractor's proposal was based on supporting the new garage on H-piles driven to the hardpan, rather than bedrock. As part of their proposal, the contractor proposed both static and dynamic pile load tests. The piles were to have a design capacity of 300 kips per pile, which, assuming a factor of safety of 2, would require an ultimate capacity of 600 kips per pile. If the test results were unsatisfactory, the contract dictated that the contractor drive the piles to bedrock for the same price per pile.

Test Piles

Installation. The piles were driven with a Delmag D46-32 open ended diesel hammer which had a ram weight of 10.14 kips and a manufacturer's maximum rated energy of 113 k-ft. The intent was not to drive the piles to bedrock but rather to bear the piles within the hard pan. Figure 3 presents a plot of penetration resistance with depth for four test piles.

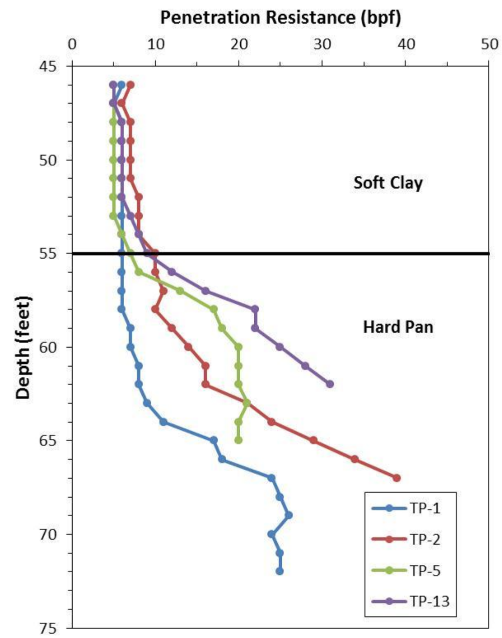

Fig.3-. Driving logs from four of the test piles.

As shown in Fig. 3, the piles experienced relatively easy driving within the soft clay, with penetration resistances on the order of 5 to 9 blows per foot, with a 6 to 6½-foot hammer stroke. The penetration resistances increased in the hard pan, but were not excessive. The test piles terminated at penetration resistances of 20 to 38 blows per foot, with hammer strokes on the order of 8 to 9 feet.

Pile Testing. Testing consisted of high strain dynamic testing with a Pile Driving Analyzer (PDA), and two axial compression static load tests. Signal matching analyses were also performed on the dynamic test data using the Case Pile Wave Analysis Program (CAPWAP). The dynamic testing was performed during initial driving and also during restrike. Table 1 presents the results of the restrike CAPWAP analyses for the different pile penetration depths into the hardpan. The restrikes were performed 15 days after the end of initial driving.

It is generally accepted that a set of at least 0.10 inch per blow is required to fully mobilize the pile capacity. As can be seen in Table 1, three of the four test piles had a set greater than 0.10 inch per blow; therefore, the predicted capacities of 580 to 611 kips represents a fully mobilized ultimate capacity at the time of the restrike testing.

Table 1: Dynamic test result from the beginning of restrike.

Pile Penetration Depth (feet)	Restrike Results	
	Ave. Set per blow (inches)	Total Predicted Pile Capacity (kips)
62	0.33	580
65	0.09	870
67	0.19	579
72	0.13	611

The pile driven to 65 feet, however, had a high restrike penetration resistance, and the full capacity may not have been realized, even though the CAPWAP analysis predicted a total ultimate capacity of 870 kips. This pile was one of the piles also tested by a static load test. The static load test was performed first, after which the pile was restruck the same day and monitored with the PDA. Apart from predicting the pile resistance, the CAPWAP analysis also produces a simulated pile top force versus pile top movement loading test graph. Figure 4 presents the results of both the static load test load-displacement curve and the CAPWAP produced curve.

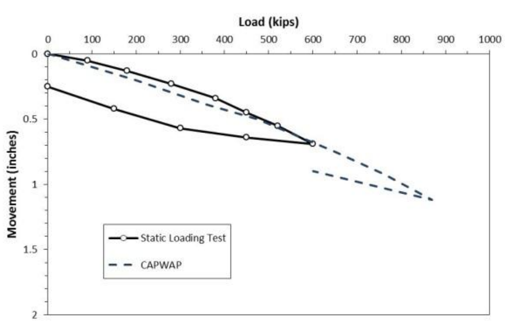

Fig. 4: Comparison between the static loading test results and the CAPWAP simulated load-movement curve from beginning of restrike.

During the static load test, the pile was only loaded to twice the required allowable load, therefore the test was terminated at a load of 600 kips. Superimposing the CAPWAP simulated graph shows very good agreement, and it also shows that the pile could have been loaded much higher.

<u>Driven pile - soil displacement</u>

As shown in Fig. 2 the bottom of the water storage reservoir is located on the naturally-occurring fine beach sand at or below the water table. The geotechnical data indicated that this sand varied from loose to dense. As most of the parking garage piles were driven, the surface settled and sand was imported to maintain grade. This confirmed the general understanding of soil behavior during pile driving as stated by Peck and Hagerty (1971) and others:

> Saturated, insensitive clay will behave incompressibly during pile driving, and,
>
> Soil settlement is likely to occur when piles are driven into clean granular soils.

During pile driving the behavior of sand is a function of the pre-driving density. Loose to medium dense sand will undergo densification, thus volume reduction. Extremely dense sand will expand because the particles must move over each other as sand particles are displaced by the pile. A threefold difference in the void volume occurs in different idealized sand packing configurations (Mitchell and Soga, 2005**)**.

Thus, the displaced volume of clay will be approximately equal to the volume of the driven pile, and extremely dense sand will have a displaced volume greater than the volume of the driven pile.

<u>Pile driving vibration energy</u>

The final project plans called for driving the piles within 3 feet of the West reservoir wall, rather than using micropiles. The vibratory energy that would be imparted to the West reservoir wall was unknown. Dowding (2000) states that a reinforced concrete structure, such as the West reservoir wall, could be expected to withstand a velocity of 10 in./sec without structural damage. Thus, it was agreed with the water reservoirs structural engineer that 5 in./sec would be the maximum horizontal velocity acceptable during pile driving activities.

The particle velocity in the sand 3 feet from the pile while driving was unknown. A stable platform for measuring the particle velocity in the soil was developed by burying a 28 pound lead block with securely mounted Geophones 3 feet beneath the surface and 3 feet from the pile. Measured soil particle velocity readings were in the range of 3.9 to 4.3

in./sec. This was acceptable to both the client and the reservoir owner's structural engineer. However, additional reductions in the energy imparted from the pile were achieved by lowering the diesel hammer fuel setting when driving through the sand.

Six monitoring locations were established on the West reservoir wall using bricks, with Geophone receptors epoxied to the reservoir wall as shown in Fig. 5. At the center of the reservoir wall, two monitoring blocks are shown (Fig. 5), with Station 3 on the left and Station 4 on the right, one on each side of the construction joint. Also, note the two Avongard strain gauges that span the construction joint. A Geophone is installed on monitoring Station 3 (the left side (North) of the construction joint).

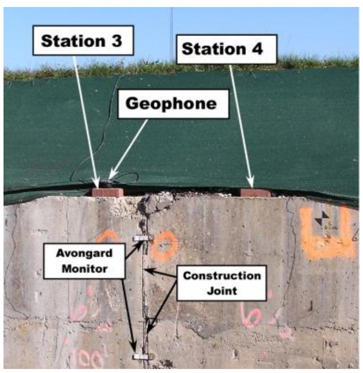

Fig. 5: Geophone monitoring blocks and Avongard strain gauges at the West reservoir wall construction joint (center).

Figure 6 shows the locations of the 122 H-piles, and six monitoring stations. The piles were labeled beginning with 1 at the North end and ending with 122 at the South end. The center pile, number 66, is located approximately 30 feet south of the construction joint.

Figure 7 shows the driven 122 H-piles and the West wall and South West corner of the water reservoir. The old parking garage is visible behind the reservoir; the office building is behind the piles. It should also be noted that the top of the water tank has approximately 3 feet of soil for frost protection.

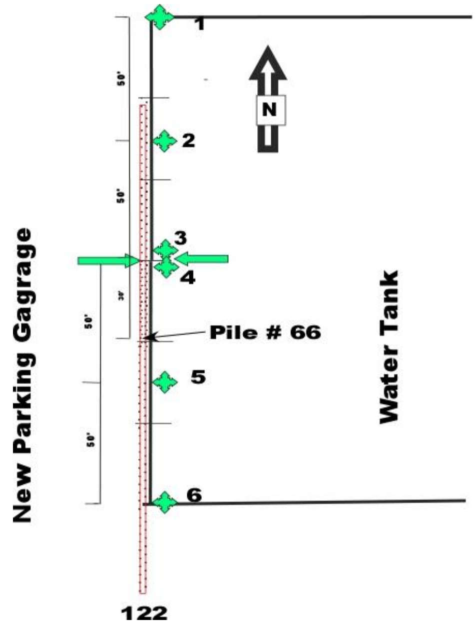

Fig. 6: Pile locations and vibration monitoring stations.

Fig. 7: Driven piles and West wall of Water Reservoir.

Figure 8 shows the vehicle barrier wall and garage support columns supported on a grade beam cast over the top of the H-piles shown in Fig. 7. Note the top of the reservoir wall and soil cover on the right hand side.

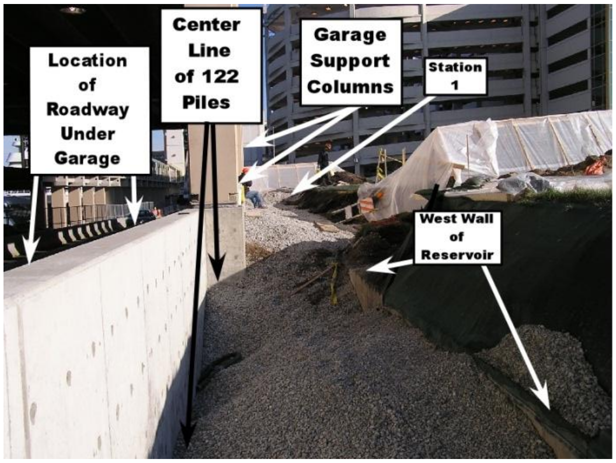

Fig. 8: Completed Garage Barrier Wall.

Figure 9 shows Station 5 at the center of the south half the West reservoir wall. Three pre-existing cracks with crack monitors are highlighted with white paint; these cracks are spanned by Avongard crack monitors. Water is seeping from the cracks and ponded adjacent to the wall (driven piles are visible in the foreground).

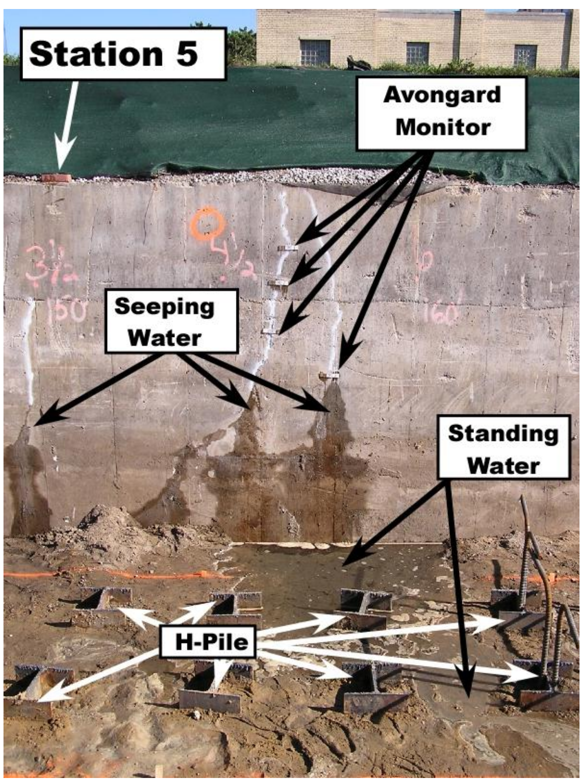

Fig. 9: Monitoring Station Number 5, 150 feet South of North end.

The monitoring stations shown in Fig. 5, 6, and 9 were established to provide vibration monitoring for energy input into the wall, as well as explicit survey reference points during pile driving. Prior to any pile driving along the tank, baseline readings were taken for both vertical and horizontal control.

Horizontal Wall Movements

A maximum recorded east-west wall horizontal movement of 0.03 inches was recorded during pile driving, with the majority of readings around 0.01 inches. Thus, horizontal wall movements were judged to be insignificant and of no concern.

Vertical Wall Movements

Figure 10 shows changes in vertical elevations of the six monitoring stations during pile driving. Note that five explicit dates from September 4 through September 17 are highlighted on Fig. 10. These response dates are used in Fig. 12 to show the changes in the structures response that were achieved.

At first there was doubt amongst the project team that the tank was being raised, rather than settling. Conventional thinking and experience with the majority of the site piles predisposed the engineers to assume settlement would occur. Before the project began, the concern was that densification of sands beneath the reservoir would occur and cause the reservoir to settle, resulting in a loss of water, and in the worst-case scenario a collapse of the tank.

There are several things to note in Fig. 10. First, note that all stations on the West reservoir wall were raised as the result of the pile driving. Second, Station 1, at the far North end remained at essentially the same elevation after September 9th. Third, Stations 3 and 4(also shown in Fig. 5) moved in tandem until September 10, when they begin to show approximately 1/10 of an inch height differential that continued to the end of the project. Next, Stations 5 and 6 on the South half of the reservoir wall lagged behind the movement of Stations 3 and 4. Last, the North end of the wall was raised first, and after September 11 essentially remained unchanged.

At no point was the differential movement between any of the points greater than 1 inch which means the structure of the tank had a Δ/L ratio of < 1/1200 which was acceptable to the owner's structural engineer.

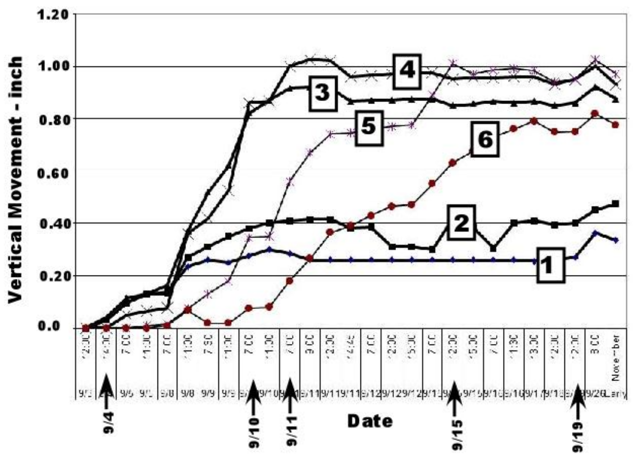

Fig. 10: Changes in vertical elevations of the six Monitoring Stations on West Wall of water storage reservoir.

The uplifting of the West Wall appeared to be within tolerable limits, with no adverse impacts noted until September 10, when water was observed seeping from one of the cracks at Station 5 (Fig.9). It was also noted that there was approximately 1/10 of an inch vertical and horizontal (North-South) differential movement across the construction joint (Stations 3 and Station 4).

With both the differential elevation changes and water seeping from the West Wall of the reservoir, discussions were held with the owners engineer representative and eventually the owner. Based on these discussions it was decided that the most pragmatic approach to correcting the problem was to alter the pile driving sequence so that the middle and South end of the southern half of the reservoir would be raised more uniformly.

Figure 11 shows the pile location number versus the driving sequence. The nearly linear line on the left side of Fig. 11 for piles 1 through 60 represents the effort prior to September 10. The scattered driving sequence in a much less linear fashion on the right side of Fig. 11 for piles 60 through 122 shows the corrective driving sequence.

As pile driving progressed to the South, one of the major concerns was the differential elevation between Stations 3 and 4, which were located approximately 3 feet apart as shown in Fig. 5.

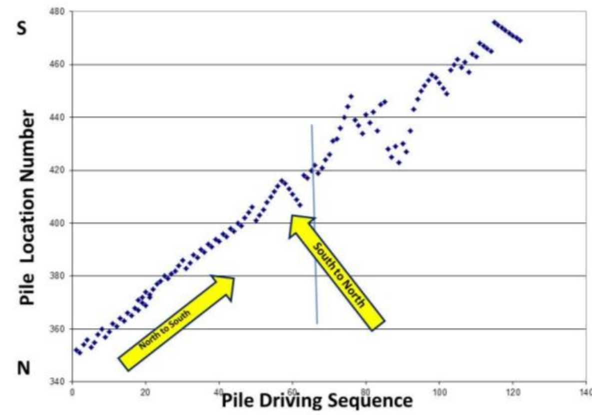

Fig. 11: Pile location versus driving sequence.

The effects of altering the driving sequence shown in Fig. 11 are shown in Fig. 12 where the monitoring station locations are plotted against the wall elevations for five dates. Stations 3 and 4 are approximately 3 feet apart, while the other stations are each 50 feet apart. At the beginning of the project, as shown in the bottom line for September 4, the three monitoring stations on the North half of the reservoir wall had risen, while none on the South half had risen. Station 3 was approximately 1/20 of an inch higher than Station 4. On September 10, when the seepage began, Station 1 had achieved its maximum height change. Station 2 was very close to its maximum height change, and Station 3 has a greater elevation change than Station 4.

After the altered pile driving sequence began on September 10 the impacts can be seen in the upper September 10 line shown in Fig. 12. Station 2 has reached its maximum elevation change, Stations 3 and 4 are nearly equal in elevation change. Stations 5 and 6 are both moving up, and the sharpness of the curvature of the representative line is less than it was before the pile driving sequence was revised. On September 11, Station 4 has reached its maximum elevation change, and the three stations to the North remained constant. Both Station 5 and Station 6 moved upward and the line between Stations 3, 4, and 5 is increasingly straight.

On September 15, Stations 1, 2, 3, and 4 show no change, while both Stations 5 and 6 have nearly 4/10 of an inch of elevation change. The curvature of the line between the stations on the South end of the wall is now upward. At the completion of the pile driving on September 17, Station 4 and Station 5 both have approximately 1 inch of elevation change. Station 6 has an elevation change of slightly more than 8/10 of an inch. At the construction joint, there is a differential elevation change of approximately 1/10 of an inch between Station 3 and Station 4.

As shown in Fig. 7 and Fig. 9, not all seepage was eliminated from the wall. However, seepage was reduced in the majority of the cracks. At the construction joint in the center of the wall there is both vertical and north-south differential movement between Stations 3 and 4.

The as-built details of this joint were unknown and there was a concern for failure along the joint as a result of excessive movement. The joint did not fail, and the subsequent scuba divers inspection of the interior of the wall showed that the precipitation and sediments within the tank tended to cover over the cracks and provide some sealing.

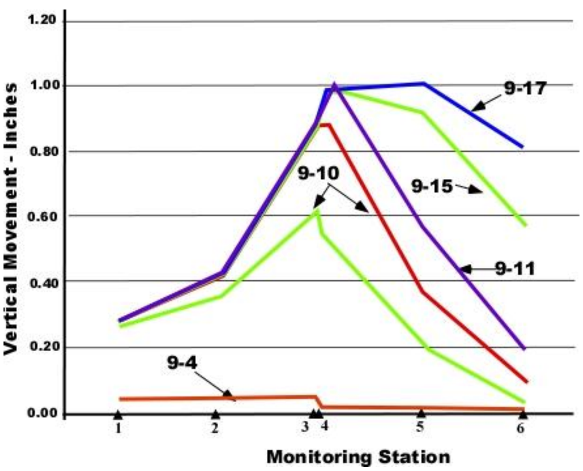

Figure 12: Wall monitoring station elevations at the beginning, middle and end of project.

The North end of the tank was raised approximately 3/10 of an inch. There is nearly 7/10 of an inch differential between the North and South Ends of the wall. A differential is to be expected since, the North 30 feet of the tank had no piles driven adjacent to it, and the piles extended approximately 30 feet beyond the South End of the wall. A more interesting question is: Why was the southern end of the tank, Station 6, only raised 4/10 of an inch, when the piles extended 30 beyond the South End?

<u>Sand Density</u>

The structure responded to pile driving differently than expected. Normally small displacement H-piles driven into sand, densify the sand. On the other hand insensitive saturated clays can be expected to have a displacement equal to the volume of the piles. Given the relative amount of sand and clay it was expected that minor settlement would occur at the surface.

It is concluded based on the behavior of the reservoir that the change in elevation of the West wall occurred because of the combined displacement of the sand and clay beneath the tank. The reinforced concrete water retention reservoir had been built on the site more than 80 years before the project at the approximate level of Lake Michigan. The beach sand beneath the tank is generally rounded with virtually no clay content and relatively small silt content. During the lifetime of the reservoir, sand beneath the reservoir has been saturated and subjected to continual vibrations from the constant movement of the water in the reservoir. These vibrations were felt on the side of the reservoir wall. In addition, the area periodically experiences minor earthquakes. Therefore, it is concluded that the earthquakes along with the constant vibrations compacted the confined sands to an extremely dense state.

Conclusions:

1. The sequencing of the piles driven after September 10 were individually selected based on the changes in the crack gauges and the authors “feel” for the structure. Altering the driving sequence resulted in preventing a possible rupture of the tank wall or creating a significant gap for water flow.
2. Preconceived notions of a materials behavior can result in responses which seem to be not possible.
3. Monitoring and surveillance of potentially impacted structures can alert an engineer to potential adverse consequences
4. Working closely with other engineers, and contractors and contractor personnel, can allow corrective measures to be taken to prevent adverse events.
5. Understanding both the soil and the type and response of the structure impacted, allowed the corrective actions to be taken.

   What did we learn?

   - Subsequent to completing the pile driving the interior of the water reservoir was examined by a licensed structural engineer/certified scuba diver who observed that the deposits on the interior of the tank remained essentially unchanged through the pile driving and there was no obvious distress at the locations of the observed exterior cracks and the construction joint, both vertical and horizontal.

- The reservoir has since operated for over 10 years with no reported problems either to the reservoir or the garage support.
- The mechanical response of the reservoir to the change in elevation was not anticipated. The North 100-foot long half of the wall individually responded in a very smooth serpentine manner. However, with no structural reinforcing across the construction joint the relative uniform response of the structure did not continue to the southern half of the wall.
- Once the structure behaved differently than anticipated (September 4) more attention should have been given to the anticipated response.
- By paying attention to the monitors and the survey data, as well as the physical events (water seepage) a relatively simple altered driving sequence allowed the project to be successfully completed.
- The author was on site during implementation of the altered driving sequence, evaluating the response of the individual gauges and selecting the next pile location to drive. This was done in close conjunction with the pile driving crew to elicit their opinions and support which proved to be an invaluable.

Lesson learned

It would have been impossible from a pragmatic standpoint to sample the sand beneath the water tank.

Initially, we should have believed "our instruments" and re-evaluated our expected response from the reservoir.

How many times do we hear "We had the data, but just did not look at it"? Had the curve from the September 4 readings been available for review on September 5, we might have altered our pile driving sequence then rather than one week later when the seepage occurred.

The data was available to pinpoint the problem with the construction joint early in the pile driving. The data was not reviewed and is not known if it would have been correctly interpreted. On the other hand, a rapid response to the seepage of the water allowed the driving sequence to be altered and the curvature of the bottom of the reservoir to be smoothed.

The northern half of the structure behaved beautifully, in a serpentine manner that lulled the author into complacency. The discontinuity associated with a construction joint interrupted the smooth flow of the stress.

The time associated with altering the driving sequence of the piles, in retrospect, does not seem to be a significant concern given the potential consequences. The pile driving sequence should have been given more attention before the project began. It was assumed that the decision to drive the piles North to the South was okay, and well thought out.

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