Advancing Foundations: Augered Cast-in-Place Bridge Foundations

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ABSTRACT: Augered Cast-in-Place (ACIP) piles are prevalent in vertical structures while bridge and transportation structures have been slow to adopt, with AASHTO not addressing their design. The OhioDOT used 176 ACIP piles in a 217' long pier footing requiring battered, reduced & low headroom, and phased installations within a highly charged aquifer. This is an overview of the project, geotechnical design, structural design, installation process, testing, and lessons learned from this first-of-its kind foundation for OhioDOT.

PROJECT INTRODUCTION

A typical Ohio Department of Transportation (ODOT) District 4 (D4) re-decking project morphed into a complex bridge replacement because a 50-foot-tall wall abutment on six rows of piles was sliding laterally atop two highly charged aquifers. One inch of lateral movement of the forward abutment was measured in four years of monitoring.

The project overcame many challenges, such as extreme skew, stability during construction, hydraulics, 60-inch sanitary sewer impacts, and maintaining interstate traffic on a bridge that acted strut restarining the abutment movement. A difficult subsurface resulted in installation difficulties of shafts and an alternate solution was investigate. Steel piles or cased shafts were not considered due to the vertical movement of fines along the smooth pile face.

As difficulty was encountered progressing deep drilled shafts on an adjacent ramp bridge, a long and skewed pier footing for bridge STA-77-0936 was re-designed to utilize ACIP piles; the first use of ACIP piles as a bridge foundation for ODOT. This bridge was a two-span (190 feet – 293 feet) steel girder bridge that was 90 feet wide and skewed 65 degrees (right forward) and crossed the West Branch of Nimishillen Creek in Canton, Ohio. It carries Interstate Route 77 just north of the US-30 interchange and shares its forward abutment with an adjacent ramp bridge.

This paper and corresponding presentation will focus on the reasons the project found ACIP piles suitable for both temporary shoring and a large foundation originally designed using drilled shafts.

Photo 1: Phase 3 (Initial) ACIP Installation

ACIP PILE OVERVIEW

Augered Cast-in-Place Piles (ACIP) are known by several names including Continuous Flight Augered Piles (CFA), Screw Piles, and Drilled Displacement Piles (DD). The technology was first developed by Intrusion Prepakt, Inc. A patent was applied for by Raymond Paterson in 1951 and was granted in 1956. Intrusion Prepackt granted licenses to Berkel & Company Contractor, Inc and Lee Turzillo Contracting Company. Berkel is in Kansas City and Lee Turzillo was in Ohio. In 1973 the patent expired, and more contractors entered the field.

An Augered Cast-in-Place Pile is a deep foundation element that consists of grout or concrete pumped into place in augered holes through the hollow stem of the auger. This allows the placement of the grout/concrete material during the withdraw of the auger from the hole. This eliminates the need for temporary casing or slurry to keep the hole open until grout/concrete can be installed. The installation method often allows for faster, easier, and less expensive installation than drilled shafts. ACIP piles were and are most often installed on crane mounted rigs that are, unlike drilled shafts, able to be installed on a batter, thus reducing the need of the vertical reinforcements to transfer the lateral loads. Since the reinforcement acts axially it is does not have to be as robust as that resisting flexure.

ACIP piles initially suffered from a bad reputation for poor quality control as typical methods of inspecting the shaft and reinforcement placement prior to pouring concrete were not possible and monitoring grout placement relied on manually counting pump strokes at the surface during installation. Another problem in the early years of use was controlling the placement and cover of steel reinforcement, which needed to be placed after the pile was installed while the grout/concrete was wet.

Compounding these early problems was the limited torque of early installation rigs, the use of lower pressure grout pumps, and the availability of only lower strength mixes of grout. Together these problems limited the diameter of the piles, and typical piles diameters rarely exceeded 16 to 18 inches. The depth was also limited and piles rarely exceeded 60 feet in length. Early rigs were unable to penetrate sufficiently into rock to allow confidence in their development of end bearing resistance. These limitations caused resistance to their widespread use in the transportation industry.

Despite the lack of wide use in the transportation industry, the advantages of the ACIP in speed of installation, low vibration, and affordability allowed them to enjoy popularity in the US commercial market. Their use in this market led to continued development in the technology. Today, modern rigs can provide up to 10 times the torque of the early rigs allowing larger diameter, greater depth, and even penetration into bedrock. Computer monitoring systems allows greater performance monitoring and quality controls. High strength grout and grout additives that allow easier installation of reinforcement has allowed for increased capacity of ACIP.

PROJECT GEOTECHNICAL CHALLENGES

The STA-77-0936 bridge faces many of its difficulties due to the problematic geological conditions underlying the structure. The most significant geological condition impacting the design of the project is the groundwater.

Photo 2: Constantly flowing forward abutment weep holes. Weeps are 3 feet above creek level and shown after 13 months of low rainfall.

In short, there is a deep buried aquifer that is a product of the preglacial drainage systems of northeast Ohio. The ancient drainage system of preglacial Ohio consisted of many major stream systems that deeply incised the bedrock through erosion. These systems were altered by glaciation. The glaciers cut valleys down to bedrock, scoured new valleys, and filled preglacial valleys with a variety of sediments that may often contain aquifers. The result was that the preglacial valleys were overlain by relatively flat plains or even rolling glacial hills; thus, these valleys are referred to as "buried valleys" and the presence, depth, and extent of the preglacial valleys are disguised by the current topography. This makes understanding the impact of the buried valley aquifer difficult to interpret without an in-depth subsurface exploration plan. For example, it is very common for a post-glacial stream to be flowing atop a buried valley but in the buried valley. It also makes understanding the impact of the buried valley aquifer difficult to interpret without an in-depth subsurface exploration plan.

At the STA-77-0936 bridge, multiple geotechnical explorations were performed between 2013 and 2017. The scope and breadth of each of explorations increased, eventually including conventional borings, sonic borings, Cone Penetration Testing with piezocone readings (CPTu), and the installation of an inclinometers and piezometers. The instruments were continuously read through remote reading devices until the surface conditions became clear. The observations were further supported by geological research and field reconnaissance for springs or artesian conditions.

After completion of the subsurface explorations, the effect of the Nimishillen Creek (aka West Branch, West Branch Nimishillen Creek) drainage system and its "charged" aquifer became clear. While the presence of buried valleys is not uncommon, the quantity of water in the aquifer and differential in head between the site and upstream are significant. It was found by review of the existing bridge plans that the original ground elevation near the IR77/US30 interchange is about El. 1000.0. This elevation was modified during the bridge construction and the relocation of Nimishillen Creek that occurred during the bridge construction. A review of bedrock topography mapping indicates that the bottom of the buried valley is at or below El. 800 and is about 1-mile wide in the interchange area. Over the broader study area, the valley is up to 350 feet deeper than the ground surface.

The charged conditions result from the differential in the pressure head between the head waters at an elevation of approximately

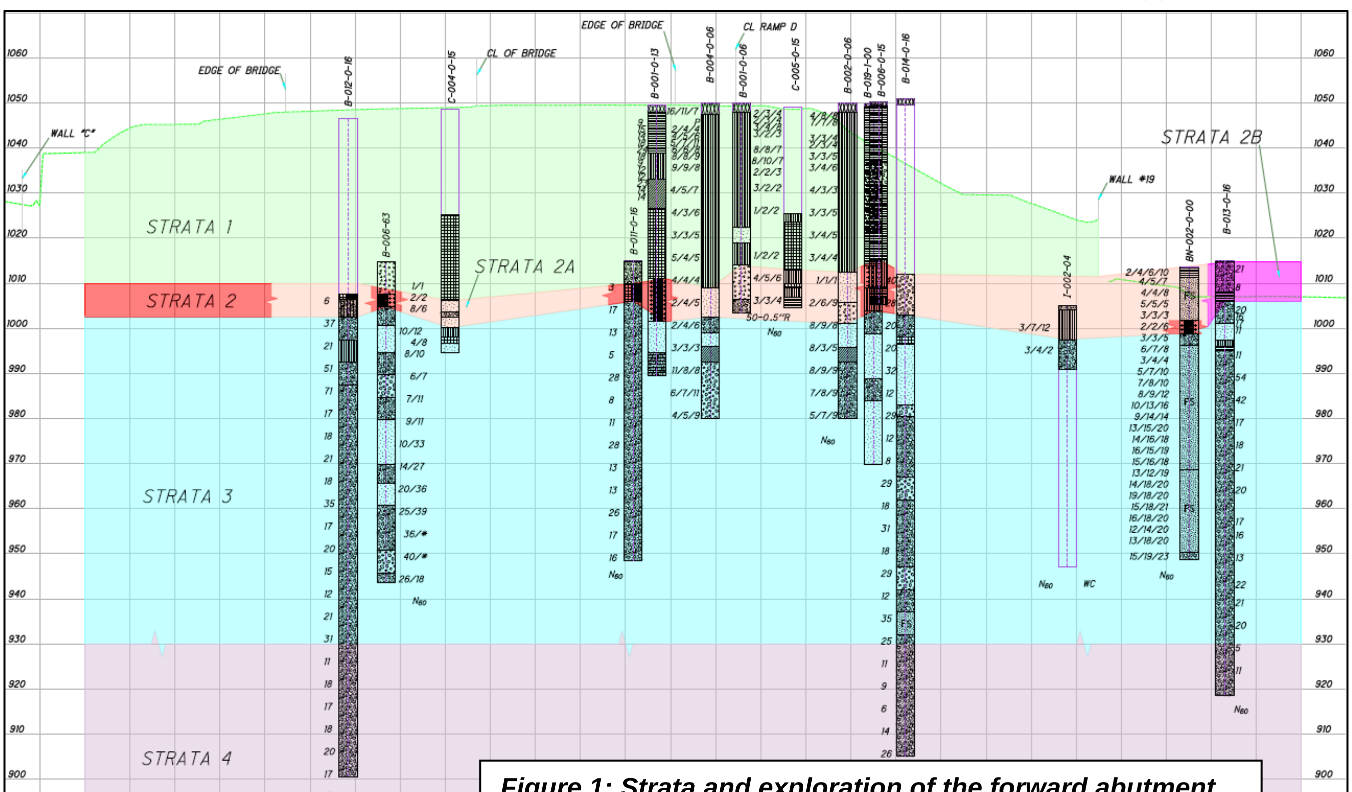

Figure 1: Strata and exploration of the forward abutment

1100 feet and the local valley bottoms of approximately 800 feet and a confining layer of fill from the bridge construction starting at around elevation 990 to 1000. Based on the piezometer information the ground water has a spatially variable head. The maximum elevations found in the piezometers range between 1014.3 and 1017.1. It is important to understand that the near surface soil and embankment soils acts a confining layer in respect to the underlying aquifer. The confining layer can be called an aquitard or aquiclude. A confining layer is defined as a soil layer that allows much less water volume passage than the aquifer it overlays. When the confining layer is penetrated water is released up, under pressure, through the confining layer even though the confining layer itself is saturated. Thus, we see when the soil is saturated is at a greater elevation than the confined aquifer's potentiometric surface, the saturated soil prevents piping. In other areas, where the ground surface elevation is lower than the potentiometric surface elevation, potential for piping exists. These areas are the stream bed, the stream banks, beneath the footings of the structures, and to some extent behind the abutment wall. Perhaps most significantly, at the footings, penetrations from driven piles provide a pathway for the piping and seeps to develop. The groundwater works to reduce the frictional resistance of the steel piles by transporting water and fine material away from the piles, reducing the available geotechnical resistance of the frictional piles. This destabilizes the bridge foundation and allows it to move over time. This is consistent with the observation of many artesian seeps at the site, in and around the stream and substructures, including water piping vertically out of the pier foundation several feet above the stream surface.

Compounding the problem is the existence of a second less well-defined aquitard near elevation 950. The geotechnical exploration had difficulty identifying this layer, but it was witnessed by the many holes that collapsed, and difficult drilling that began at near elevation 950. Local industry has used this near surface aquifer since the 1940s without significant drawdowns. It is clearly documented in Ohio Department of Natural Resources (ODNR) references that many pump fields are used to connect the deep aquifer below elevation 950 and to recharge the upper aquifer in times of extreme drought or demand. This is possible because the potentiometric surface the lower aquifer is similar or slightly greater than the upper aquifer and deposits of till within the valleys are generally highly permeable and porous, allowing for a large quantity of water to pass through the main valley at a significant transmissivity.

Not only has the groundwater contributed to the movement of the bridge, it has made site work and installation of the new proposed foundation more difficult. The foundations of the pier footings, and major utility improvements are installed below elevation 1114 and require extensive dewatering. Excavations into the granular soils of the aquifer are extremely unstable.

The transportation industry in Ohio has long preferred driven steel piles. However, due to the failure in the long-term performance of the driven piles, the design team felt strongly that their use would be inappropriate as it would result in additional piping. Frictional drilled shafts were proposed to but became difficult and costly to install below elevation 950 due to continued hole collapsing. It became evident during installation that they would not be a practical way to maintain the schedule.

ACIP AS TEMPORARY WORKS

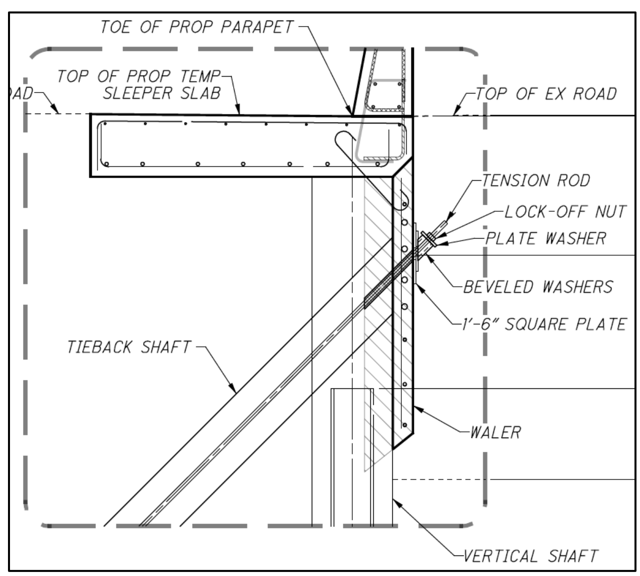

Figure 2: ACIP Shoring

THE NEED: Despite an innovative approach to the maintenance of traffic for the projected four-year duration of the project, significant shoring was required. Multiple lanes of traffic needed to be maintained during construction as the bridge could not be taken completely out of service. The sliding failure of the existing bridge made a low vibration shoring system necessary to support the 20 feet of excavation for foundation work. However, there were many constraints to this excavation. The excavation is offset only two feet from traffic and is constructed around the battered counterforts of the existing highly skewed abutment. A further complication arose due the skewed geometry of the bridge; the proximity to the corners of the abutment and the existing backwall to the excavation. In short, the skewed counterforts prevented the use of conventional (long small diameter) soil tiebacks that are typically required for a cantilevered wall with a twenty-foot height. Compounding these geometric problems was the subsurface difficulties resulting from the migration of the abutment backfill. The backfill migrated over the decades, creating an unreliable stratum for common short anchored shoring approaches such as soil nails. For these reasons, vibratory installations and typical soil nails were removed from consideration.

SHORING DETAILS: Ultimately ACIP pile walls with ACIP pile tiebacks were chosen for their low vibration installation, ability to probe and stop at the counterforts for obstructions, and ability to be installed as large diameter tiebacks coupled with grout pressures that could facilitate the filling of any encountered voids. The vertical 24-inch diameter ACIP piles were installed as a tangent wall with a W12 pile centered ing each. Every 4th pile saw an 18-inch diameter ACIP tieback installed at a 45-degree angle downward. The tie back used a center 1.375-inch diameter, high yield strength, thread-bar as the tension element. The large circumference of the ACIP tieback allowed adequate resistance of the tension forces in the variable strata. The tiebacks were locked off on a cast-in-place waler poured on the excavated face of the wall; the CIP waler produced uniform bearing against the uneven surfaces of the round ACIP piles. Further, the lateral movement of all piles was locked together by a sleeper slab cast to the top of the piles, allowing the toe of barrier to be only 18 inches from the wall. In areas near the abutment or conflicting with future phases of drilled shaft construction, a double row of tiebacks were fixed at their heads using a sleeper slab.

A plan for de-tensioning tiebacks during the backfilling of the excavation was required to ensure the second phase of excavation did not encounter tiebacks under tension. The first phase of backfill had self-supporting backfill placed against the vertical ACIP elements, reducing the need for tiebacks in the subsequent phase. A combination of geosynthetic reinforced soil (GRS) and cellular concrete fill was used as the self-supporting fill.

TIEBACK TESTING: The ACIP tiebacks were proof loaded per ODOT Supplemental Specification 866.06.C (Ground Anchors). The proof testing of each anchor was performed on a steel reaction frame prior to the cast-in-place waler being poured. The proof testing was loaded in increments of 20% of the maximum factored load, with a 10-minute dwell time at the factored design load. The displacements were as expected for the 15-foot unbonded length, and the 10-minute dwell loading displaced less than 0.04 inches (the maximum allowed by the specification).

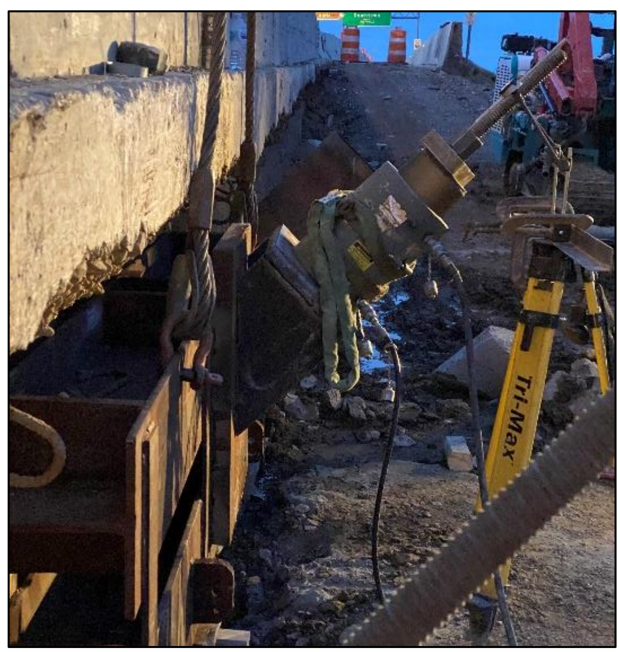

Photo 3: ACIP Tieback Proof Loading

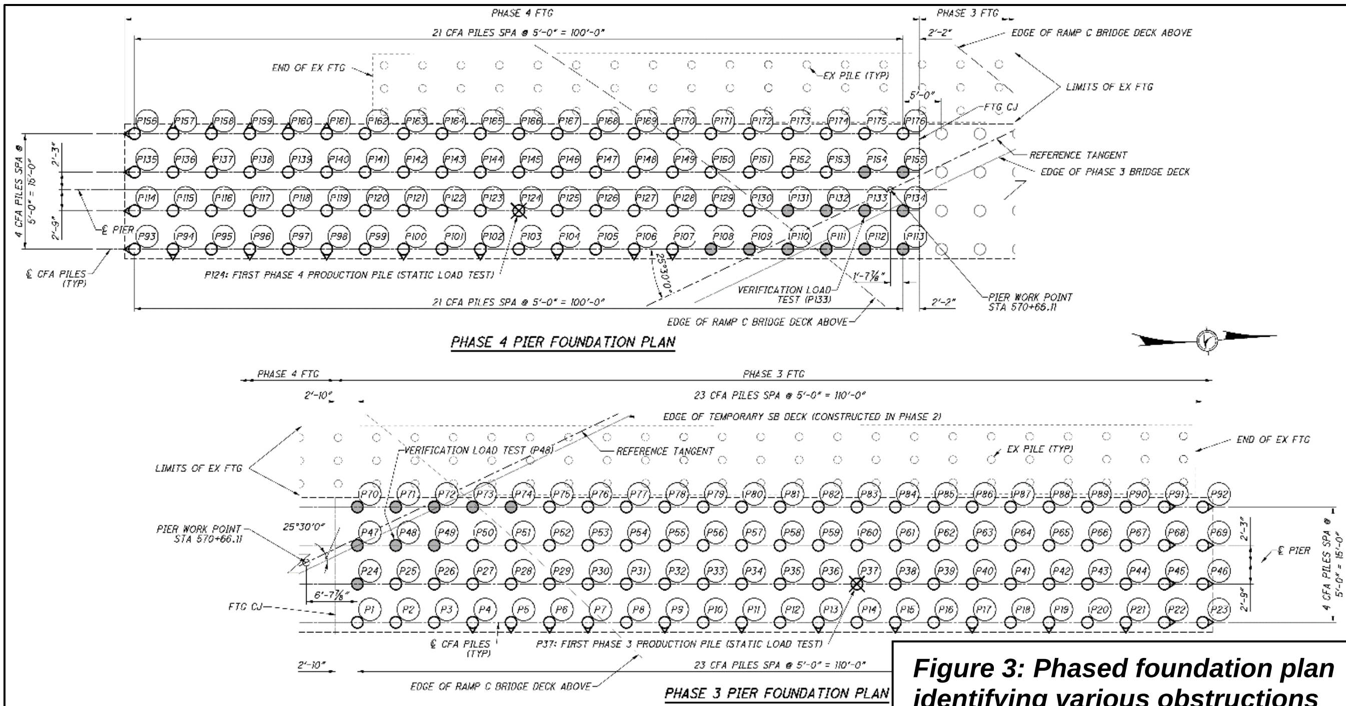

Figure 3: Phased foundation plan identifying various obstructions

FOUNDATION DESIGN

ORIGINAL SHAFT CONSTRAINTS: Beyond the difficult subsurface, the drilled shaft pier foundation was complicated by the overhead obstructions of the in-service portions of the bridge, and an overhead ramp bridge. This resulted in phase-line shafts being offset 26 feet from the edge of the first phase of footing. It also resulted in a large corner of the first phase of footing being installed without a deep foundation element for support and thus the footing had to be cantilevered over the shafts and several shafts had very high loads and required depths. This cantilevered footing made drilled shaft installation difficult, costly and time consuming.

ACIP IMPROVEMENTS: ACIP piles can be battered to resist lateral loads. Most of the forward ACIP piles could not be battered forward due to the existing pier piles. However, the design of the initial phase of the bridge able to accommodate a few forward battered piles along with piles battered to the rear. As a result, the first phase did gain the advantage battered ACIP piles that work in tension. This coupled with the successful tension testing of piles on the shoring walls, gave confidence that pile layout could more efficiently resist lateral loads than larger diameter shafts.

The 24 column by 4 row array of 18-inch diameter piles (at five foot spacing in each direction) was achievable through the use of reduced headroom (under the overhead ramp bridge) and low headroom (under the remaining in-service bridge) ACIP installations. By normalizing foundation loads across many more elements, the length of the resulting ACIP piles were reduced to 45 feet (from the original 95 foot long 42-inch diameter shafts). The reduced length allowed the foundations to not pierce the lower aquitard, further reducing installation complications due to differential water pressures that can occur at the interface of the aquitard.

Ultimately, the required factored resistance of the highest loaded ACIP pile was reduced to 132-kip, from the 690-kip maximum shaft reaction required of the less ideal drilled shaft foundation.

REINFORCING: Most ACIP piles acted in compression, with the few aforementioned piles potentially being required to resist tension during the first phase only (but at completion all battered piles were in compression only). Typically, a central reinforcing bar placed in the center of the ACIP pile and would be used as primary reinforcing for both tension and compression. In some cases supplemented with a reinforcing cage in the top portion of the pile. A full-length cage was designed for this project due to the subsurface conditions and novel use of the foundation type. Since this was a new foundation type for ODOT on a critical large bridge, additional integrity testing was warranted along with additional protections against unknown movements or unbraced length that may occur over decades of potential movement of fines in the aquifer.

The full-length cage allows implementation of Thermal Integrity Profiling (T.I.P.) wires in addition to providing reserve capacity to the pile. The cage was a simple array of 4-#8 reinforcing bars with a tied hoop cage. The hoops allowed for ease of splicing shorter cages in reduced and low-headroom conditions. The T.I.P. wires would be tied to each #8 bar on 30% of production piles and 50% of low headroom piles. The final specifications also dictated one static load test per construction phase, and 3 verification load tests per construction phase.

ACIP GEOTECHNICAL DESIGN

As this paper is being written, construction nears completion on the foundations of the STA-77-0936 bridge. A FHWA approach was used to design the ACIP pile foundations. This section will discuss design theory and approach without presenting detailed calculations.

It was clear that any system of deep foundations would need to be frictional rather than end bearing, as the depth to bedrock and the ability to drill through the charged till material prevented end bearing. The effect on the ground and frictional resistance of the ACIP falls somewhere in between a driven pile and a drilled shaft. A driven pile displaces soil and generally increases the stress in the soil surrounding it. The installations of drilled shafts loosen soil as the auger or casing is removed. Typical ACIP piles increase the stress of the soil while the auger advances but can experience a reduction of pressure as the auger is withdrawn. This is offset somewhat by pumping grout immediately during with draw. Some ACIP, called displacement piles (DD) are specifically designed to rely on the increase of stress developed during drilling.

The commercial market has developed many methods for determining the static capacity of the ACIP; however, at the time of the design, the ODOT *Bridge Design Manual* and the AASHTO LRFD 9th *Bridge Design Specifications* had not presented a preferred design method for ACIP piles.

The FHWA *Geotechnical Circular No 8 Design and Construction of Continuous Flight Auger Piles* has adopted design procedures based on Reese and O'Neill drilled shaft design. This design methodology largely ignores the displacement of soils in the ACIP installation and the effect of pumping the grout under pressure on the side resistance of the piles. Early in the design development, it was recognized that pumping under pressure would be advantageous, given the groundwater conditions encountered in the exploration and during the installation of drilled shafts. While perhaps slightly conservative, the method of design as presented in the HWA GEC 8 was adopted and was judged to be appropriately conservative due to this being a "maiden use" of the ACIP piles for ODOT. Therefore, a minimum pumping pressure of 300 PSI of grout was stipulated in the contract specifications but was not considered in the design.

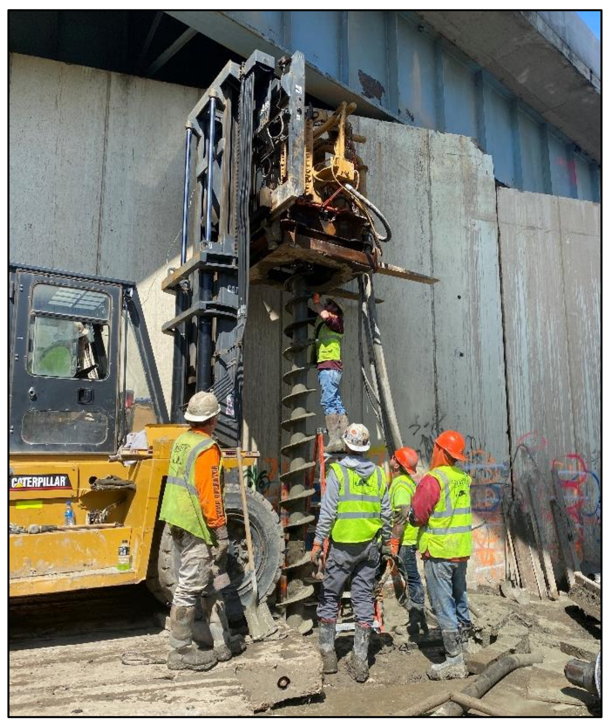

Photo 4: Low Headroom ACIP Installation

Another concern during design was penetrating the lower aquitard. When penetrated, the drilling became much more difficult due to groundwater infiltration and regions of less dense soil caused a drop in the side resistance. While the borings did not allow the team to determine the exact depth of the aquitard, the aquitard was estimated to be at elevation ±950 based on the depth where conventional borings began to experience excessive sand blow in the augers and auger refusal. Like the upper aquitard and the potentiometric surface elevation, the elevation of the lower aquitard was thought to vary across the site. It was surmised that minimizing the number of piles that penetrated below elevation 950 would increase the success of the ACIP installation.

ACIP FOUNDATION INSTALLATION

The advantage to the ACIP installation was that in a highly permeable and pressurized aquifer condition, the deep foundation installation did not require keeping a hole open. Once proof of concept testing was completed on mix designs that incorporated anti-washout admixtures and fluidifier, the pressurized grout was effective at maintaining a stable column of grout while also allowing the full-length reinforcing cage to be pushed into the grout column after auger removal.

A demonstration pile (non-production pile) was required at the start of each phase to ensure the mix design was still working properly and that the onsite crew could both complete the augering and installation of the full-length cage.

The mix design required the contractor to submit a design that satisfied the 4,500-psi compressive strength to meet ODOT's requirements for a drilled shaft concrete. The admixtures included fluidifier to aid in pumping. It was found during the demonstration pile installation that the most important admixture was "Intrusion-Aid™MAX" which allowed setting the cage.

ACIP pile installation on the project used two methods. ACIP installed in area with limited headroom and areas without limited headroom. In the areas without headroom constraints typical ACIP were installed on a set of leads attached to cranes with hydraulic motor traveling on the rails of the leads. Using this method, the Pile Installation Recorder (PIR) was able to provide grout quantities and pressure at depth. In the low head areas, a forklift was used to hold the hydraulic motor and smaller auger segments of five to ten feet were connected and disconnected. This allow installation with as little as 20 feet of headroom; however, it prevent the full utilization of the PIR with this set up. The PIR provided grout pressure and volume but could not provide depth information since it set not programed to be used with smaller augers.

The low overhead piles required 10- or 20-foot segmental reinforcement cage, depending on head room, that were mechanically spliced with 16-inch connectors. This was a laborious process and in the areas of low headroom production was about half the unrestricted head room areas.

ACIP TEST PROGRAM

Being the first installation of ACIP piles as a bearing foundation, and on a complex structure with a difficult subsurface, it was desirable to have a substantial pile capacity and integrity testing program. This testing program would not only demonstrate that the installed piles were satisfactory for this project, but would also evaluate the applied geotechnical design method. The robust testing program was established during the redesign from drilled shafts to ACIP piles and included both integrity testing and load testing.

Photo 5: TIP instrumentation of rebar cage taper at bottom

THERMAL INTEGRITY PROFILING: Along with monitoring using an electronic PIR system during installation, the main evaluation of pile integrity would be through Thermal Integrity Profiling (TIP). The TIP would be performed according to ASTM D7948 method B (ASTM, 2014). This method includes thermal wire cables attached to the rebar cage. Because a full-length rebar cage was part of the design and evaluating grout cover for the rebar cage was of specific interest, each tested pile included four thermal wire cables. Collected data from four wires allows for evaluation of the alignment of the rebar cage. (Belardo et al. 2021)

Evaluation of the TIP data was complicated by multiple factors. First, the rebar cage was bundled at the bottom to taper the cage and facilitate installation through a fluid grout column (Photo 5). Second, the piles were extended to the top of the template, which was eight feet above the final top of pile elevation, while only the data over the production length of the piles needed integrity evaluation.

To account for the extended length at the top, data above the cut-off elevation was removed from the analysis within the TIP-Reporter software. To account for the taper, an assumed typical temperature “roll-off” was applied. The temperature roll-off at the bottom is due to heat escaping both along the pile edge and bottom and is standard practice in evaluating TIP data. However, because the TIP sensors were physically closer to the center of the pile near the bottom, where the temperature is expected to increase, a mid-pile adjustment was applied to the data along with the roll-off correction. Because this required some assumptions, the roll-off and mid-pile adjustments for every pile were recorded and compared to evaluate consistency.

56 ACIP piles were tested using TIP, including both battered piles and piles installed using low clearance methods. Of those tested, only one pile indicated an integrity deficiency near the final top of pile elevation. Upon removal of the added length, this pile was chipped down to acceptable grout and formed back to pile cut-off elevation. Evaluation of the rebar cage alignment indicated that vertical piles maintained excellent alignment. The evaluation of the battered pile data indicated that the cage was only slightly shifted to the lower side of the pile; however, sufficient grout coverage was maintained along the length of the piles.

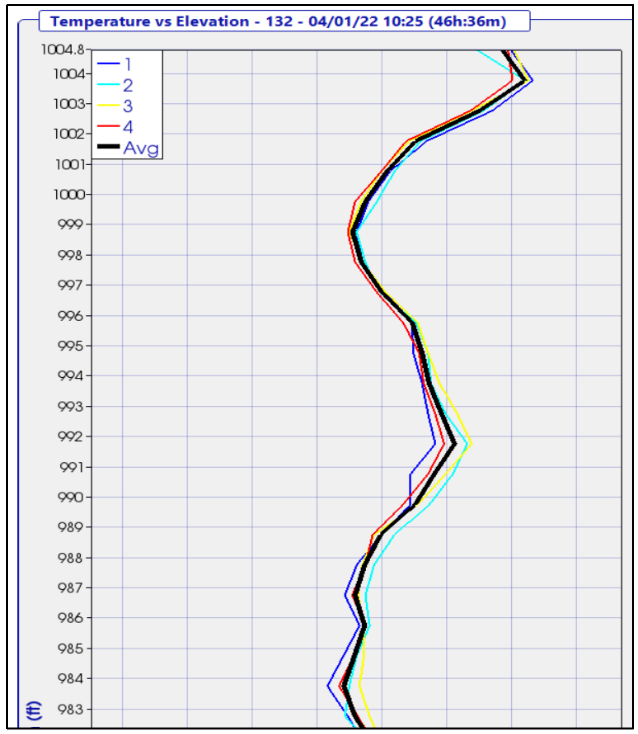

Figure 4: Example TIP Results, Wire temperature vs. depth at a specific time

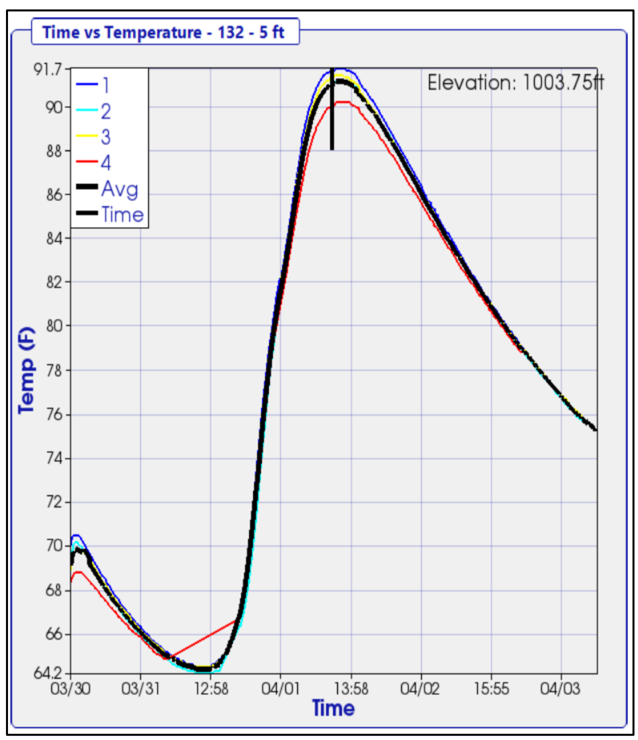

Figure 5: Example TIP Results, Wire temperature vs. time at a specific depth

ACIP CAPACITY TESTING

DESIGN VALIDATION LOAD TESTING: the project did a static load test on 5/28/2020 to validate the initial design, prior to fully converting the foundation to ACIP foundation elements. The tested pile and reaction piles were laid out in the same proposed pattern of the foundation, so that the frame and results would correspond to the requirements of the production foundation.

Ultimately, the static load test, administered and documented by the project team onsite, resulted in very good results. The maximum load tested was 380 kips (which included some overburden resistance); the maximum factored resistance required of each pile was 132 kips. The test resulted in less than 0.4 inches of total axial movement of the pile at the max test load. During a 60-minute hold time at the max test load, the pile creeped only 0.02 inches and rebounded to a total deformation of 0.25 inches, 10 minutes after the static test was completed.

Two production static load tests, one in each phase of construction, indicated very similar results. The maximum production test loads were 317 kips and resulted in less than 0.4 inches of axial movement, and a final total deformation of 0.2 inches. The consistency of the static load test results was reassuring of the adequacy of the design.

Photo 6: Static Load Test Set-up

VERIFICATION LOAD TESTING: The production verification load testing was selected to be performed using dynamic load testing according to ASTM D4945. A total of six production piles were selected for verification testing, three in each phase. The dynamic testing was performed using a drop hammer with a four ton drop weight. Drop heights ranged from 0.5 feet to 2.0 feet during testing.

The method of evaluation of the dynamic testing data per the project specifications was to generate a simulated load vs. displacement curve by superimposing curves from multiple impacts. The resulting simulated load vs. displacement curve was then evaluated using an appropriate method. For piles with significant displacement, the Brinch-Hansen 90% criteria was used, other results with less displacement used the maximum applied load as the nominal pile capacity. Dynamic test results indicated nominal pile capacities ranging from 215 kips to greater than 340 kips.

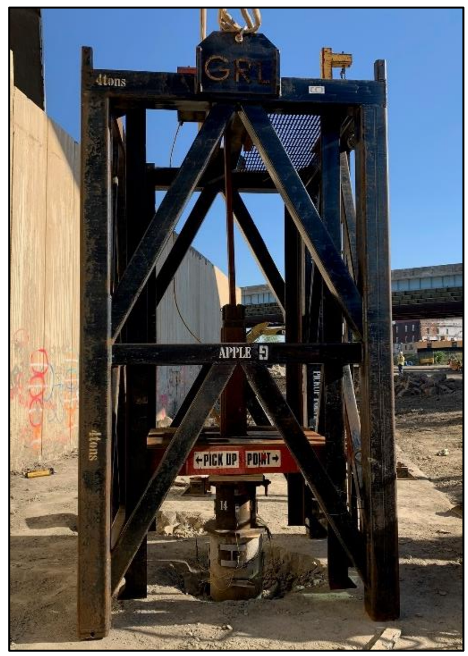

Photo 7: Drop hammer for dynamic load testing

Ultimately, all integrity and load testing of the ACIP piles indicated acceptable load carrying capability and integrity. The extensive load test program concluded that the ACIP piles that were selected to replace the potentially problematic drilled shafts performed sufficiently and the redesign was a successful venture for all parties on the project.

ACKNOWLEDGEMENT

Once the unstable abutment was found, this project became infinitely more challenging. Everything from geotechnical explorations, interim stabilization projects, and ultimately construction had unique challenges due to geometry and the difficult subsurface.

The authors would like to acknowledge the outstanding collaboration with ODOT District 4 Staff, including the District Deputy Director Gery Noirot, Project Manager Laura Beese, Area Engineer Bulent Bilgin, and Project Engineer John Roberts. The ability of the D4 team to make quick decisions and accept new methods of design and construction enabled the many hurdles to be overcome. Further, the collaboration with the ODOT Offices of Structures, Geotechnical Engineering, and Construction were instrumental in delivering a unique solution to a difficult project.

Additionally, the project benefited from having a prime contractor, Beaver Excavating Company, that was experienced in ACIP installations in the private sector. Their ability to modify their construction techniques and implement ACIP piles allowed the redesign to become reality.

REFERENCES

ASTM Standard D7949-14 (2014), "Standard Test Methods for Thermal Integrity Profiling of Concrete Deep Foundations," ASTM International, PA, 10.1520/D7949-14, www.astm.org.

ASTM Standard D4945-17 (2017), "Standard Test Methods for High-Strain Dynamic Testing of Deep Foundations," ASTM International, PA, 10.1520/D4945-17, www.astm.org.

Belardo, D., Robertson, S., Coleman, T., (2021), "Interpretation and evaluation of thermal integrity profiling measurements," *DFI 46th Annual Conference on Deep Foundations,* Las Vegas, NV, October 2021

W.M. NeSmith, Jr. P.E. (2015), "Recent Developments in ACIP and DD piling", Minnesota Geotechnical Society Annual Conference, February 2015