FOUNDATION SUPPORT OF THE LILLY MEDICINE FOUNDRY IN LEBANON IN

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ABSTRACT

Eli Lilly and Company are investing $4.5 billion to create the Lilly Medicine Foundry, a new center for advanced manufacturing and drug development, combining research and manufacturing in a single location. The Medicine Foundry is in Indiana's LEAP Research and Innovation District in Lebanon, Indiana and will expand the company's investment there to more than $13 billion.

Multiple new facilities are planned for the manufacture of a range of bioactive compounds for use in pharmaceutical therapeutics. Distinct compounds include Peptides (short chains of amino acids that are involved in many physiological processes and biochemical functions), Oligonucleotides (short chains of nucleotides that can be made of DNA or RNA) and Small Molecules (with a molecular weight of < 900 Da). A Central Utility Building (CUB) with an extensive array of pipe and utility racks is planned to support the manufacturing facilities. The facilities are being constructed on pads of structural fill approximately 5-ft to 15-ft above the natural ground surface. Native soils are generally up to 35 feet of soft to stiff (generally highly weathered) glacial deposits with stiff to very hard glacial deposits underlying the site. Column loads for manufacturing facilities typically ranged from 400 to 1000 tons with 4000 psf design bearing pressures for large floor areas in numerous facilities to accommodate the relatively heavy manufacturing equipment at each facility.

Project requirements include a maximum of 3/4 in total settlement of the structures. Given the size and overall weight of the structures and the large area loads of the manufacturing equipment within the structures, this specification became the primary design consideration for foundation support of most of the structures. The manufacturing facilities will be supported on a combination of Augered-Cast-in-Place (ACIP) piles under building columns and augered, Rigid Inclusions (RIs) under the floors. The CUB facility will be supported by a range of RIs under columns and floors depending on the required support across the facility. To date, approximately 6,500 ACIP piles and 4,700 RIs have been installed to support these facilities.

This paper addresses the design and construction of the referenced ACIP piles and RIs with particular regard to the behavior of the foundation elements in the glacial till deposits underlying the site. Geotechnical site characterization information included logs of several borings and the results of cone penetration tests (CPT) performed to up to 100-ft below grade across the entire project site. The boring logs included visual classifications and Standard Penetration Test (SPT) results. Results of laboratory tests performed on collected samples were also included in the report. The results of several seismic CPTs were also provided. The shear wave velocity profile information indicated stiffnesses in the till deposits that were significantly greater than might be indicated by the CPT and boring data. The anticipated soil stiffnesses were verified by the pile and RI performance testing program, allowing the project team to economize the foundation design for pile capacity, but more significantly for the estimation of overall structure settlement.

Keywords: Augercast, ACIP Piles, APG piles, Rigid Inclusions

BACKGROUND

In a landmark development for pharmaceutical manufacturing infrastructure, Eli Lilly and Company has committed $4.5 billion to establish the Lilly Medicine Foundry, an innovative center that seamlessly integrates advanced manufacturing capabilities with drug development operations. This strategic investment, situated within Indiana's LEAP Research and Innovation District in Lebanon, Indiana, represents a significant expansion of Lilly's presence in the region, bringing their total investment to more than $13 billion.

The ambitious scope of this project encompasses multiple specialized manufacturing facilities designed for the production of three distinct pharmaceutical components: Peptides (short chains of amino acids crucial to physiological processes and biochemical functions), Oligonucleotides (short nucleotide chains composed of either DNA or RNA), and Small Molecules (compounds with molecular weights less than 900 Da). Supporting these production facilities, a comprehensive Central Utility Building (CUB) and an extensive network of pipe and utility racks ensure seamless operations across the manufacturing complex. The construction sequence incorporated elevated building platforms, with facilities being erected on structural fill pads positioned 10 to 15 feet above the natural ground surface.

Berkel was initially contracted to provide Rigid Inclusion (RI) support for the CUB facility. Based on the effectiveness of this solution, the contract was subsequently expanded to include deep foundation and RI support for the additional manufacturing facilities. These additional facilities were in early phases of development when Berkel was integrated into the project team, and included unknown final utility locations, unknown final equipment loads and locations, and preliminary structure loads. The early involvement of the foundation contractor enabled a rapid response to accommodate facility design modifications as these, and other details were finalized.

LOCAL GEOLOGY

The project site is situated within the Tipton Till Plain section of the Central Till Plain physiographic region of Indiana. This area falls within the limits of Wisconsin and Pre-Wisconsin glacial depositions, which have significantly influenced the regional geology and soil characteristics. The Tipton Till Plain section is, in general, topographically homogeneous. However, several glacial features are common throughout the landscape which result in subtle variations in subsurface profiles and groundwater conditions observed across the site.

The predominant soil type in the project site is reported as silt or silty clay loams of the Crosby-Treaty Association. The parent materials of these soils are described as silty materials or loess over loamy till, which is consistent with the glacial deposition history of the region. These soil associations typically exhibit moderate drainage characteristics and are known for their relatively high clay content.

Regional bedrock mapping indicates that the project site is underlain primarily by the Devonian-Mississippian aged New Albany Shale formation. This formation consists predominantly of organic-rich black shale with minor amounts of greenish-gray shale and dolostone. Bedrock elevations in the project vicinity are reported as ranging from El. 700 to El. 750 ft, while the site surface elevation is approximately El. 920 ft. This indicates a significant overburden of glacial and post-glacial deposits above the bedrock surface. It should be noted that neither borings nor soundings conducted during the site investigation encountered any bedrock, as these investigations did not extend to the expected bedrock depths.

SITE-SPECIFIC PROFILE

Site preparation for the project involved the placement of engineered fill to reach design levels of El. 925 ft for the Central Utility Building (CUB) and El. 928 ft for the Peptide 1 facility. Prior to the placement of this fill, the existing ground level across the site was approximately El. 915 to El. 920 ft, with natural groundwater levels observed between El. 915 ft and El. 918 ft. Table 1 is a generalization of the subsurface profile including the engineered fill, a shallow cultivated zone and the underlying glacial till.

Glacial till deposits consisted predominantly of lean clays (CL), silty clays (CL-ML), and interbedded sands (SM and SP-SM) that extended to depths exceeding 100 ft below the exploration level. The glacial till was generally described as exhibiting varying shades of brown, brown and gray, or gray, with distinct weathering characteristics at different depths throughout the profile (noted in Table 1).

Interbedded in both the upper and lower till were deposits of silty sands (SM), occasionally containing gravel, and poorly graded sands with silt and gravel (SP-SM). These granular deposits likely represent glaciofluvial sediments deposited by meltwater during glacial retreat phases. SPT N-values in these materials ranged from 2 bpf to 45 bpf, indicating very loose to dense relative densities, with the density generally increasing with depth.

Table 1. General Subsurface Profile

Elevation [ft]	
925 to 928 (Final Grade)	ENGINEERED FILL: primarily on-site natural soils processed and compacted to meet project specifications.
915 to 920	CULTIVATED ZONE: (historic and active row cropping agricultural activities). Typically, dark brown lean clay containing variable amounts of sand, root hairs, and organic content (1.2 percent to 2.7 percent). **Removed** prior to the placement of the engineered fill mentioned above.
914 to 917	GLACIAL TILL: highly weathered, in soft or loose (to occasionally hard) conditions indicative of intense weathering processes, including freeze-thaw cycles, oxidation, and biological activity, resulting in reduced strength and increased compressibility compared to deeper materials. Low plasticity lean clays with varying amounts of sand and sandy lean clays (CL). Standard Penetration Test (SPT) N-values ranged from 0 to 80 blows per foot (bpf), reflecting the heterogeneous nature of glacial deposits. Natural moisture contents ranged from 8.1 percent to 26.2 percent, with the higher moisture contents generally corresponding to the softer consistency materials.
890	GLACIAL TILL: moderately weathered, firm/stiff or medium dense. Transition between the highly weathered near-surface materials and the less weathered deeper deposits with properties ranging from those described above and below. Low plasticity sandy silty clays (CL-ML). SPT N-values ranged from 3 to 60 bpf, indicating soft to hard consistency, though generally exhibiting greater stiffness than the upper till materials. Natural moisture contents ranging from 9.0 percent to 13.2 percent, consistent with their greater density and lower void ratio.
870	GLACIAL TILL: stiff/hard or dense/very dense, with generally increasing stiffness and density with depth reflecting the reduced weathering and larger overburden pressure.
850	GLACIAL TILL: very hard, minimally weathered with significant overconsolidation
800	Approximate Extent of Exploration

DESIGN REQUIREMENTS

General

The owner's primary objective was centered on achieving production readiness by a specific target date. To meet this critical timeline, the project team adopted a conservative approach to floor load support. This strategy included designing support for a broader potential support area to accommodate uncertainty in the final positioning of heavy manufacturing and production equipment. Additionally, due to the precise tolerances required by sensitive manufacturing equipment, a maximum total facility settlement of 3/4-inch was established as a design constraint.

Cub Facility

Project documentation specified design bearing pressures for spread footings in the CUB facility ranging from 4000 psf to 6500 psf (Fig. 1). Additional design floor loads were subsequently incorporated into the requirements:

- Column Line C-1 to C-4.1: up to 450 psf
- Column Line C-4.1 to C-12: between 1000 psf to 1900 psf as determined by the project team

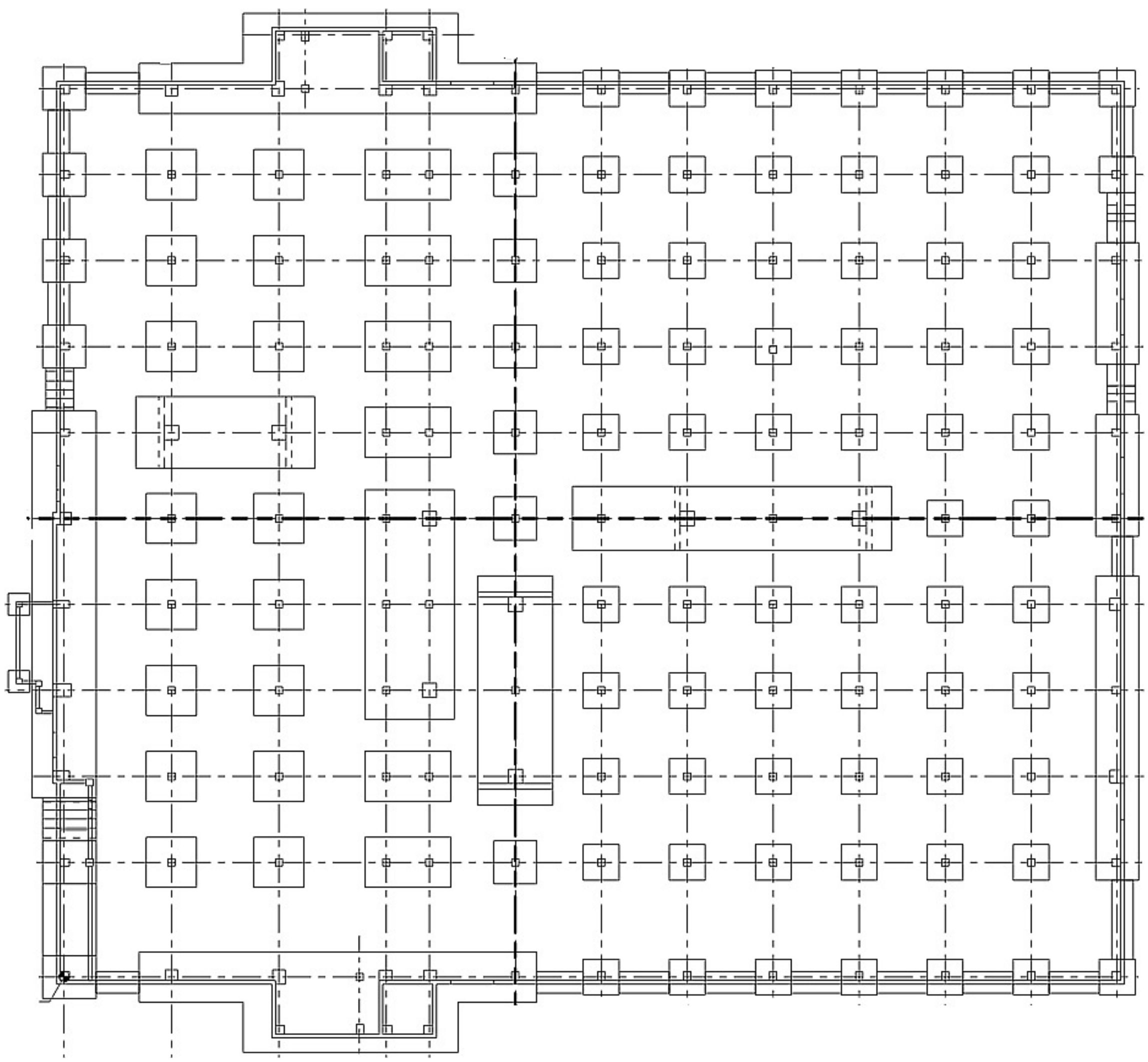


Fig. 1. CUB Facility Footings and Floor

Peptide 1 Facility

Design reactions for columns to be supported at the Peptide 1 facility were provided by the project team (Fig. 2). The evaluation of these reactions indicated that piles would be necessary for column support due to the relatively high lateral demands. Individual 20-in diameter ACIP piles were specified to provide resistance for up to 210 tons compression, 50 tons tension, and 11 tons lateral load. Additionally, the project team specified the following floor loads to be supported:

- Row P1-M to P1-C: up to 4000 psf
- Row P1-C to PO1-A: up to 1000 psf

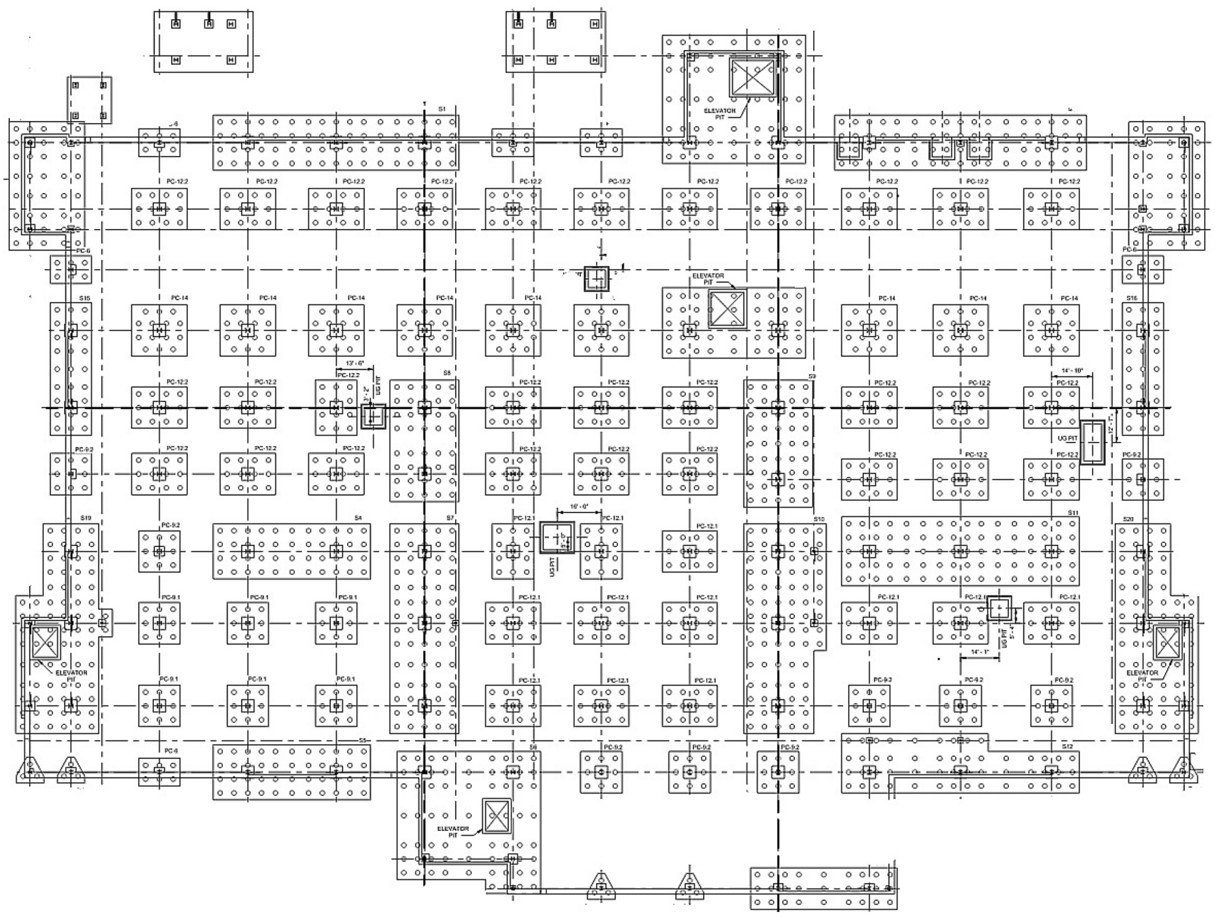


Fig. 2. Peptide 1 Facility Pile Caps and Floor

ENGINEERING/DESIGN

Axial Resistance of Piles and Rigid Inclusions

General

The design of compressive resistance for both ACIP RIs and piles involved application of recognized methodologies from geotechnical literature. Two principal approaches guided the design process:

- The methodology developed by Bustamante and Gianeselli (1982), which utilizes static penetrometer (CPT) data to predict pile bearing capacity through empirically validated correlations

- The evaluation framework outlined by Coleman and Arcement (2002), specifically developed for auger cast piles in heterogeneous soil conditions similar to those encountered at the site

These methods were refined using a proprietary database of performance of ACIP piles and RIs in comparable geologic settings, with particular attention to fine-grained till soil behavior. This calibration process ensured that theoretical predictions were adjusted to reflect actual field performance in similar conditions.

The designers applied these analytical approaches to both the boring data from the geotechnical report and the CPT data supplied separately. This analysis yielded a representative design profile for the RIs, providing a reliable basis for subsequent detailed design work across the various facility locations.

CUB Facility Rigid Inclusions

Figure 3 presents a representative axial capacity calculation for an individual foundation RI. An important design consideration was that while the soil profile documentation begins at elevation +920 ft, zero capacity was assumed from that level down to elevation +915 ft, representing the lowest anticipated top-of-RI elevation.

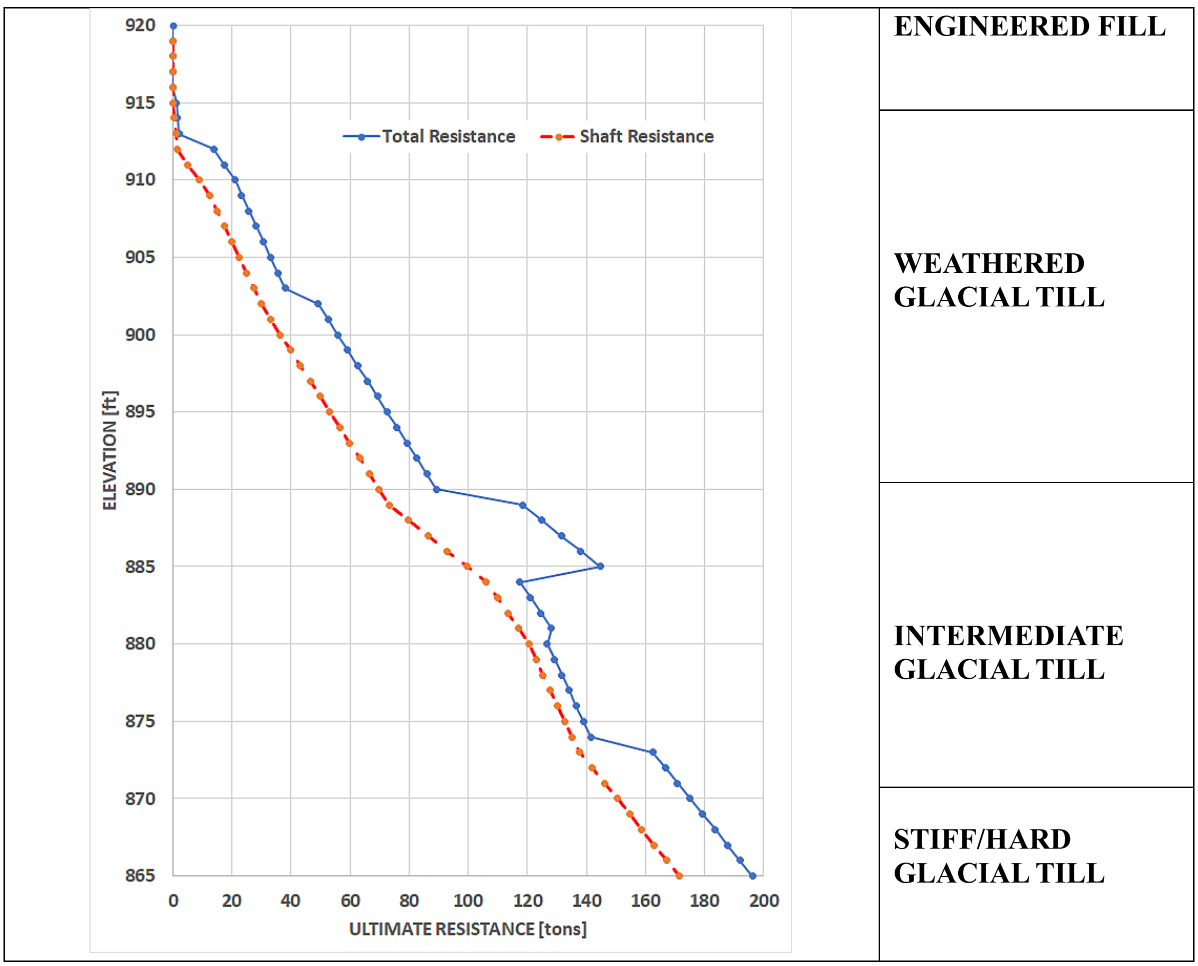


Fig. 3. Example Capacity Estimate 14-in Diameter Augered RI – CUB Facility

The final design specified 14-inch diameter ACIP RIs extending to toe elevation +865 ft, positioned according to the RI layout plan. This configuration was determined to adequately support the design bearing pressures for the spread footings. Rather than implementing a uniform grid, the design team strategically positioned RIs to address both bearing capacity concerns and settlement limitations as outlined in the project requirements. This non-uniform arrangement optimized material usage while ensuring performance criteria were met. Individual RI axial loading was capped at 95 tons to maintain appropriate safety factors.

For floor support within the CUB facility, the design incorporated 14-inch diameter ACIP RIs arranged in an approximate 10-ft by 10-ft square grid pattern. In areas with design floor pressures up to 450 psf, RIs extended to elevation +888 ft. For zones with higher design pressures (between 450 psf and 1900 psf), RIs reached deeper, to elevation +865 ft, to provide greater allowable bearing pressures.

Peptide 1 Facility ACIP Piles and Rigid Inclusions

An example calculation for axial capacity of a 20-inch ACIP pile is provided in Fig. 4. With the lowest top-of-pile elevation established at approximately +914 ft, the design specified that 20-inch diameter piles extend to elevation +841 ft to achieve the required axial resistance. Production pile lengths were standardized based on reaching this uniform toe elevation, regardless of variations in pile-head elevation (provided these remained above +914 ft). This approach maintained consistent performance characteristics while accommodating final structure levels.

The floor support system for the Peptide 1 facility employed two distinct configurations based on loading requirements. For areas with design pressures up to 4000 psf, 14-inch diameter RIs installed to elevation +876 ft on a relatively tight 7-ft by 7-ft grid were specified. In zones with lighter loading (up to 1000 psf), 14-inch RIs reaching elevation +885 ft on a wider 10-ft by 10-ft grid provided adequate support. The RI layout was carefully engineered to limit floor settlement within project tolerances rather than providing uniform bearing pressure. The design acknowledged that modifications to floor loading configurations might necessitate corresponding adjustments to RI quantity and placement. Under the specified spacing and loading conditions, individual RI axial loads would not exceed 100 tons for the 4000 psf zones or 50 tons for the 1000 psf zones.

Considerations for Underground Utilities

As previously noted, the final equipment layout and associated utility routes were not fully determined at the time of foundation support system design and installation. Furthermore, excavations for utilities were scheduled to take place after installation of the RI support system for building floors. To maximize system flexibility, RIs were installed in potential utility corridors with RI tops positioned below anticipated utility levels.

The project team incorporated a structural mat for the facility floors, and utility excavations were backfilled using the same engineered fill initially employed for building pad construction (see Site Specific Profile). Figure 5 illustrates an example of this configuration.

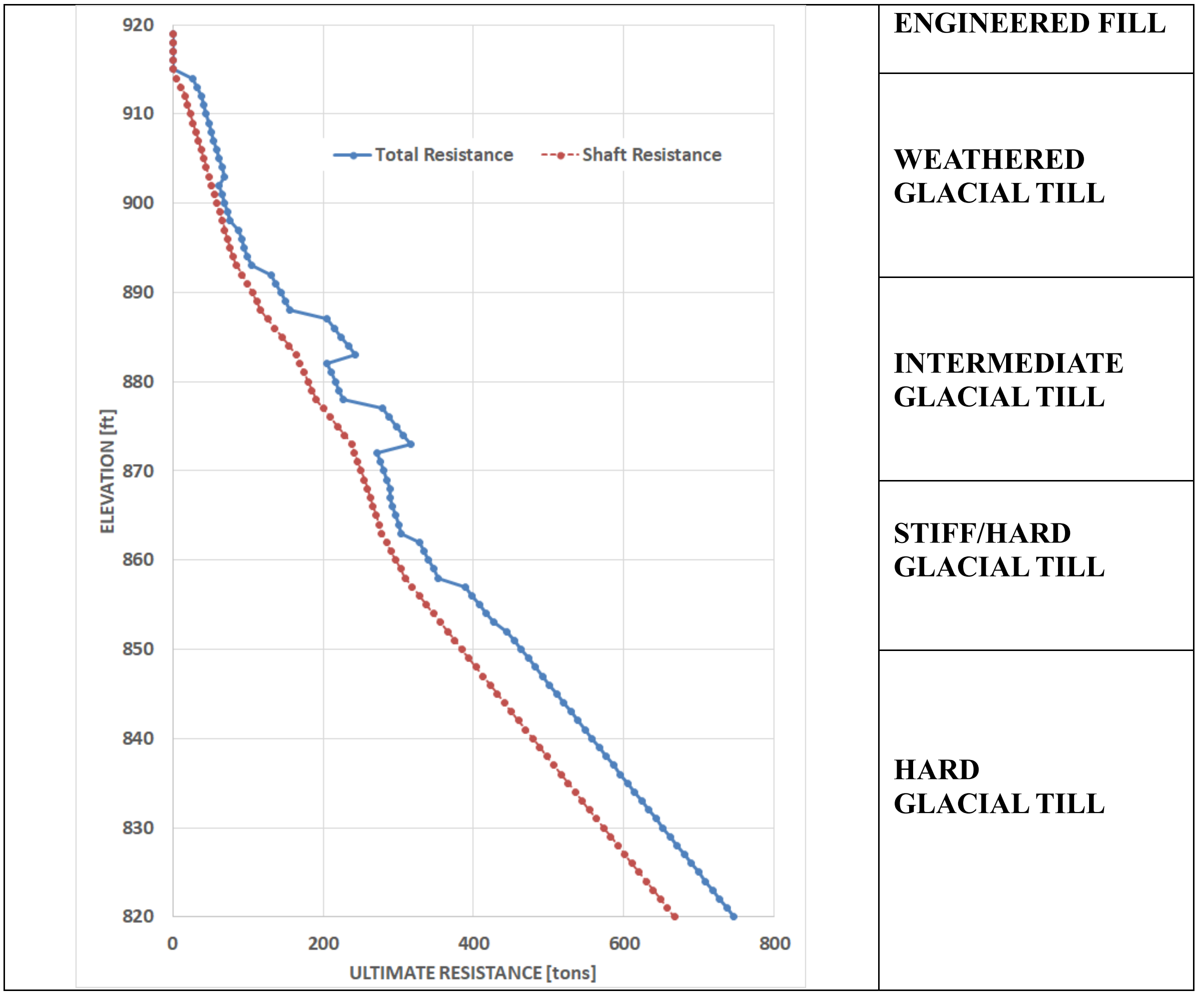


Fig. 4. Example Capacity Estimate 20-in Diameter ACIP Pile – Peptide 1 Facility

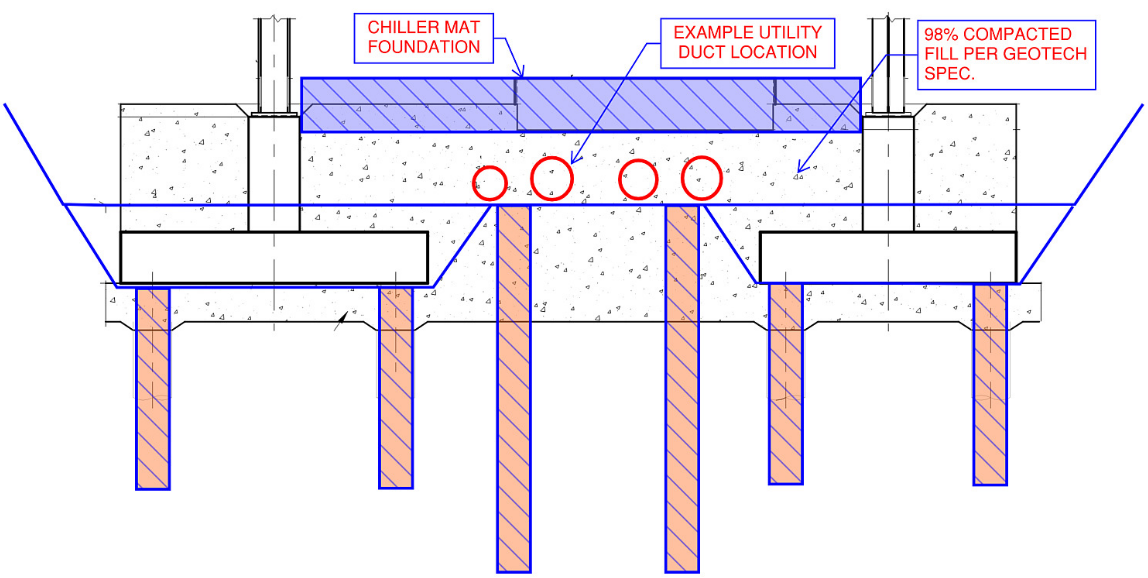


Fig. 5. Example Utility Corridor with Lower RI Levels and Additional Engineered Backfill

Lateral Resistance of Piles in Peptide 1 Facility

The evaluation of lateral resistance characteristics for the 20-inch diameter ACIP piles employed the industry-standard computational platform LPile 2022 (Ensoft, Inc.). The International Building Code (IBC) defines geotechnical allowable lateral load as one-half of the load that produces 1 inch of deflection under free-head conditions. Computational modeling for a single isolated pile indicated that a lateral load of 139 kips would generate 1-inch deflection, establishing the maximum geotechnical allowable lateral load at 69.5 kips per pile.

However, *the practical lateral load capability of ACIP piles is typically governed by structural considerations*, particularly the bending moments that develop within the pile shaft when subjected to lateral forces. The quantity and arrangement of the reinforcing steel extending from the piles to their caps was considered to produce fixed to semi-fixed conditions for the pile-heads. The structural analysis incorporated both fixed-head and free-head boundary conditions to bracket the range of potential pile behavior. Group effects were particularly significant for this project, as some configurations included more than five rows of piles at relatively close spacing (3 diameters center-to-center). To account for this pile-soil-pile interaction, a p-modifier of 0.3 was applied in the analytical model, representing the reduced efficiency of closely spaced piles in resisting lateral loads. The evaluation determined that the 20-inch diameter ACIP piles could reliably resist service lateral loads up to 22 kips while maintaining acceptable deflection characteristics and structural integrity. Figure 6 presents example analysis results which account for production pile arrangement and variations in top-of-pile elevation.

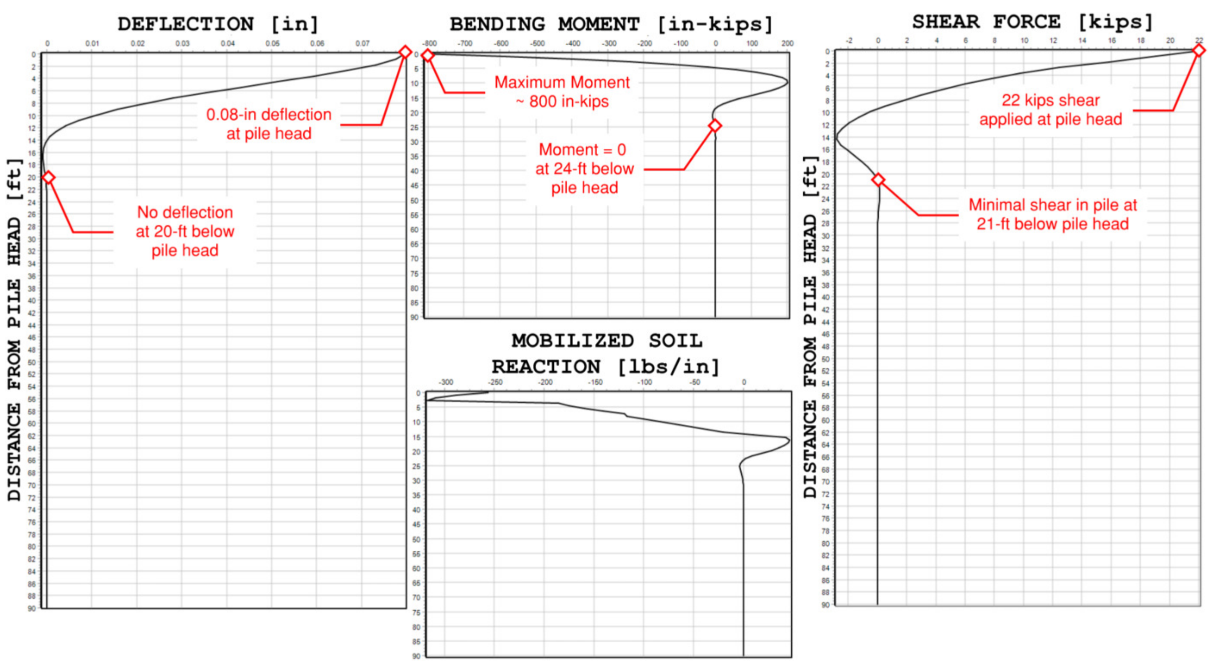


Fig. 6. Example Lateral Analysis Results for 20-in diameter ACIP Piles – Peptide 1 Facility

Settlement Analysis

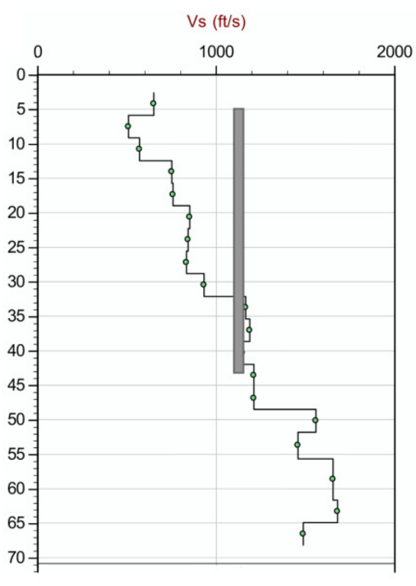
Equipment manufacturer specifications necessitated limiting total structural settlement to 3/4-inch. The project team initially anticipated that the very stiff till (detailed in the Site-Specific Profile) would provide sufficient resistance to maintain settlements within this limit. However, the large area loads required for floor support, combined with interactions between adjacent columns and specified floor loads, presented significant design challenges.

Initial soil stiffness and compressibility parameters were derived from correlations with SPT and CPT results along with laboratory test data. Settlement estimates based on these preliminary parameters exceeded the project limits. During this assessment period, Seismic CPTs were conducted at several proposed facility locations to enhance the initial site characterization. Stiffness parameters calculated from the shear wave velocities obtained through these tests were significantly higher than those indicated by correlations with CPT tip resistance, SPT N-values, or available laboratory data.

Settlement analyses using these enhanced parameters yielded values well below the 3/4-inch threshold. The load test program (discussed in Performance Verification) provided additional insights. While not directly applicable for structure settlement predictions, the test results demonstrated higher resistance values in the till than those estimated from CPT and SPT-based methodologies.

Young's modulus was also estimated from the axial load test results through a radial shear attenuation analysis (1978) which strongly favored the values estimated from shear wave velocities. The final soil stiffness parameters were implemented in various analytical models to verify compliance with project settlement requirements. Figure 7 presents a comparative example of Elastic Moduli estimated from SPT N-value (q_c) (Coduto, 2001) and those derived from shear wave velocities for the CUB facility, along with a schematic of the RIs in profile to their target depth with corresponding shear wave velocities. The difference in estimated moduli is approximately an order of magnitude.

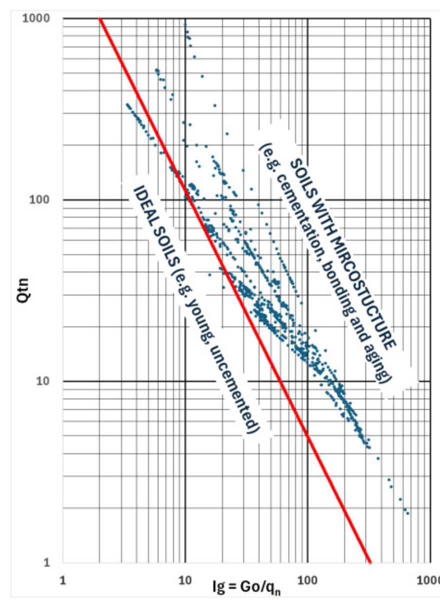
Additionally, soil microstructure was evaluated as described by Robertson (2016). The comparison of tip resistance data and Rigidity Index is also presented in Fig. 7 and further suggests age and bonding effects that would lead to an underprediction of soil stiffness using CPT tip resistance (and particularly SPT N-values). Although the Seismic CPTs were originally conducted to aid in Seismic Site Class determination and subsequent Seismic Design Category assignment, the shear wave velocity data proved invaluable in assuring the project team that total structural settlement would remain within acceptable limits.



(a)

	E' [psf]	
EL [ft]	**Vs**	**SPT N60**
919	1,568,131	401,067
911	1,568,131	401,067
910	1,615,647	103,619
905	1,615,647	103,619
904	2,384,332	205,576
895	2,384,332	205,576
894	2,930,909	301,333
888	2,930,909	301,333
887	5,328,253	721,778
882	5,328,253	721,778
881	5,152,125	473,600
872	5,152,125	473,600
871	9,237,119	876,445
865	9,237,119	876,445
864	9,472,450	1,020,121
855	9,472,450	1,020,121
854	11,331,788	1,041,778
820	11,331,788	1,041,778

(b)



(c)

Fig. 7. (a) Schematic of ACIP RI in Shear Wave Velocity Profile

(b) Comparison of Estimated Modulus, E', from Shear Wave Velocity and SPT results

(c) Evaluation of soil microstructure (Robertson, 2016)

PERFORMANCE VERIFICATION

Compression Test Results

CUB Facility Rigid Inclusions

To verify design assumptions and validate performance characteristics, a test RI was installed from ground surface elevation +924 ft to approximately 59 ft below grade (terminating at elevation +865 ft). Testing followed the procedures of ASTM D-1143-07 for static axial compressive load testing of deep foundations. The loading sequence consisted of approximately 8-ton increments applied at 5-minute intervals until reaching a maximum test load of approximately 200 tons. Unloading proceeded in five nearly uniform decrements, also at 5-minute intervals.

Figure 8 presents the load-displacement relationship observed during testing, including a hyperbolic curve-fitting analysis based on the method developed by Chin (1970). This hyperbolic projection provides an estimate of the ultimate load capacity beyond the maximum applied test load, offering additional insight into foundation performance at higher loading levels.

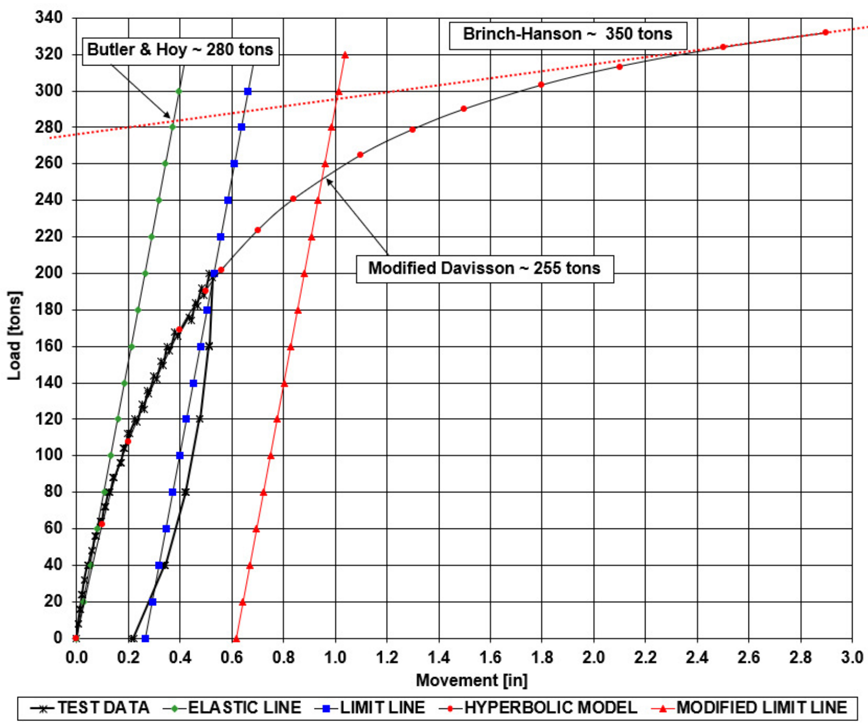


Fig. 8. Geotechnical Capacity Estimate of 14-in Diameter RI – CUB Facility

Ultimate load estimates were determined according to criteria specified in the applicable building code. The Davisson Offset Limit (DOL) approach was employed with a modifier of 4, consistent with Davisson's 1993 recommendations for cast-in-place pile systems.

Further refinements to the analysis included adjustments to the elastic line following methodologies proposed by Kulhawy and Chen (2005) and NeSmith and Siegel (2009). For ACIP piles not loaded to geotechnical failure, Studlein et al. (2014) recommend the Butler-Hoy method for ultimate load estimation. Based on these analytical approaches, the ultimate load of the test RI was established at 280 tons.

The load transfer mechanism along the RI shaft was evaluated through strain gauge instrumentation at various depths. Figure 9 illustrates the distribution of load with depth for each increment of the loading sequence up to maximum test load.

Peptide 1 Facility ACIP Piles

The Peptide 1 facility testing program included two dedicated test piles, designated CTP-PT and LTP-PT, both installed from surface elevation +924 ft to a depth of approximately 83 ft (reaching elevation +841 ft). Pile CTP-PT underwent compression testing following ASTM D-1143-07, with load applied in approximately 21-ton increments at 5-minute intervals. The test progressed to a maximum load of approximately 504 tons.

While attempting to increase the load to 525 tons, an apparent structural failure occurred within the test pile. This event was characterized by a sudden decrease in the sustainable load (dropping to approximately 370 tons) accompanied by excessive pile head movement. Following this event, the pile was unloaded in five nearly uniform decrements at 5-minute intervals. Figure 10 presents the load-displacement relationship recorded during testing, including a hyperbolic analysis following Chin's method (1970) to estimate ultimate capacity.

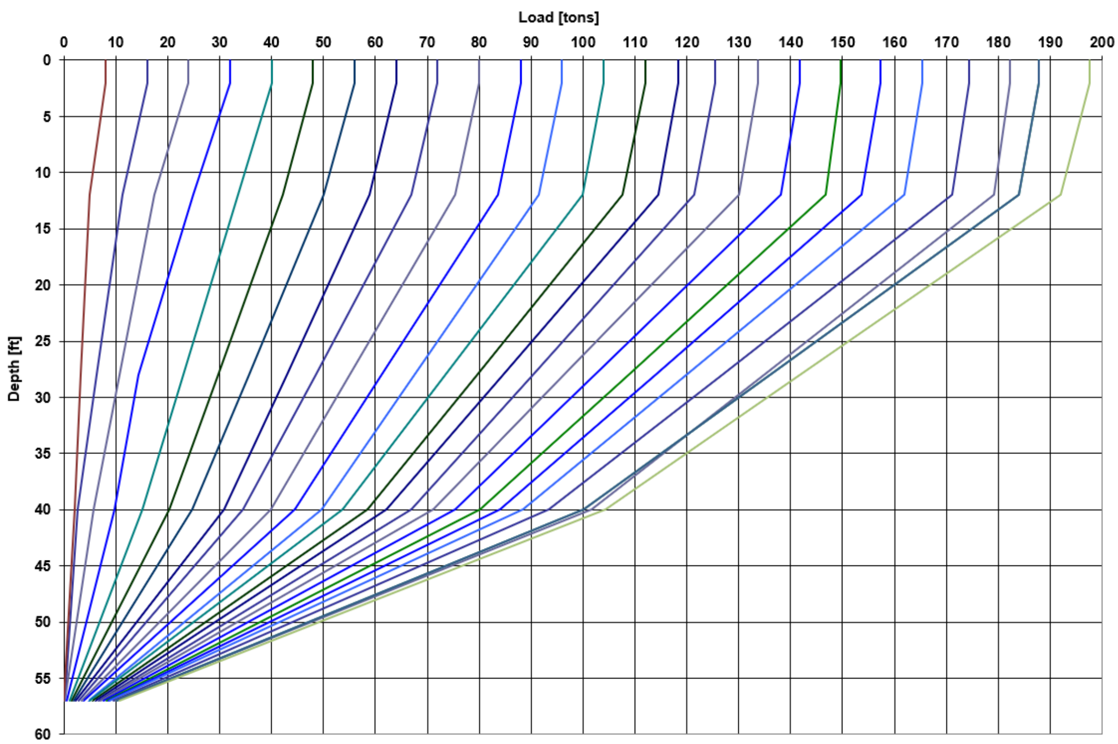


Fig. 9. Load Transfer with Depth – 14-in Diameter RI – CUB Facility

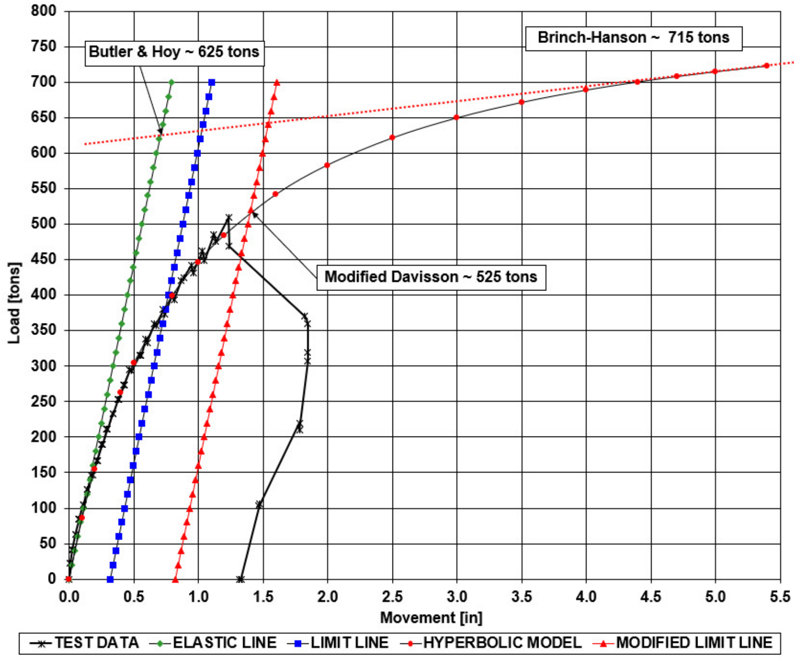


Fig. 10. Geotechnical Capacity Estimate of 20-in Diameter ACIP Pile – Peptide 1 Facility

The ultimate load analysis employed the same methodological approach described for the CUB facility testing, incorporating the Davisson Offset Limit with appropriate modifications, elastic line adjustments per Kulhawy and Chen (2005) and NeSmith and Siegel (2009), and consideration of the Butler-Hoy method as recommended by Studlein et al. (2014). Through this comprehensive analytical framework, the ultimate load capacity of test pile CTP-PT was determined to be 625 tons. Figure 11 depicts the load distribution with depth for the various loading increments. This was derived from strain gauge instrumentation using the analytical approach previously described.

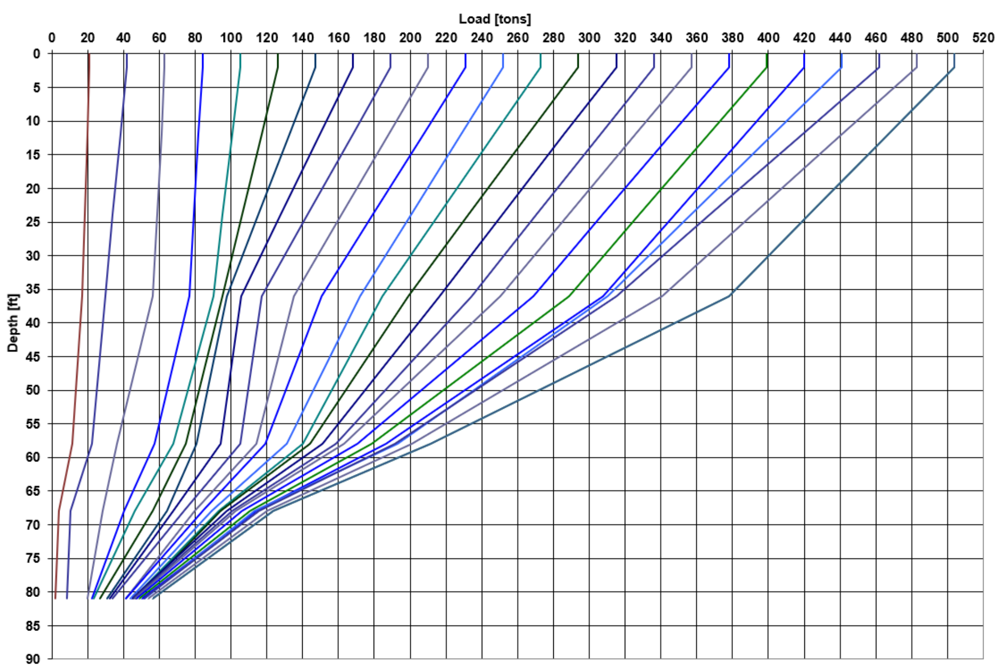


Fig. 11. Load Transfer with Depth – 20-in Diameter ACIP Pile – Peptide 1 Facility

__Lateral Load Test Results Peptide 1 Facility__

Lateral load testing was conducted on pile LTP-PT in accordance with ASTM D3966-07 standard procedures. The initial testing sequence involved loading the pile to a maximum lateral load of 22 tons while monitoring pile head displacement. These results are presented graphically in Figure 12. At the request of the design team, a supplementary loading sequence was performed on the same test pile, this time targeting a maximum pile head deflection of 1 inch rather than a predetermined load level. This approach directly assessed the pile's behavior at the deflection threshold specified in building code provisions for determining allowable lateral loads.

The load-displacement relationship from this supplementary test is illustrated in Figure 13. Analysis of the test data indicates that a lateral load of approximately 38 tons (76 kips) produced the benchmark 1-inch pile head deflection, validating the design assumptions employed in the analytical modeling.

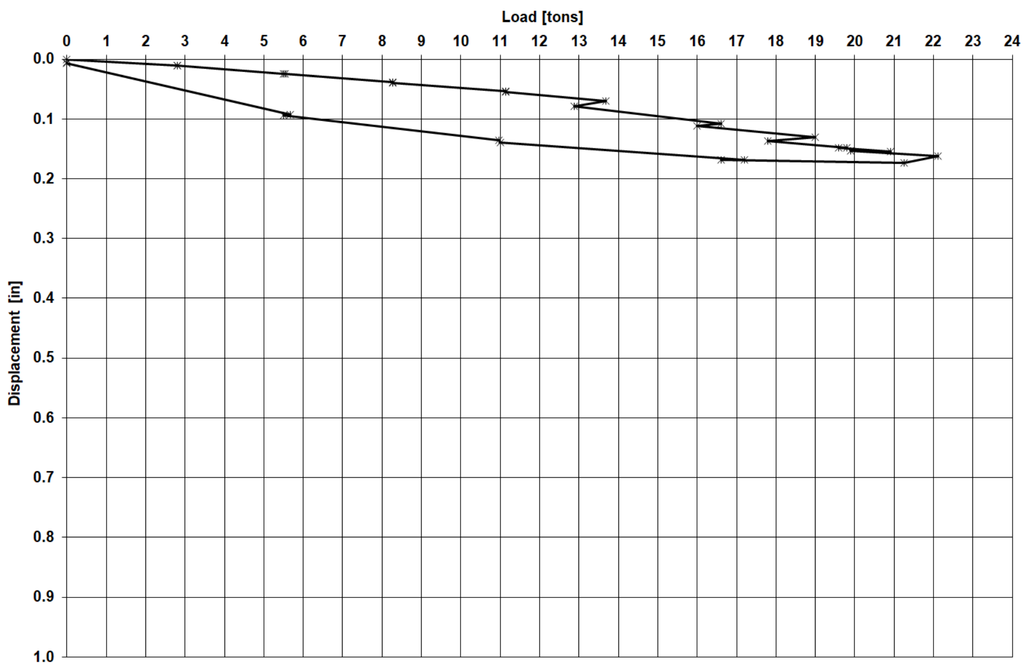


Fig. 12. Lateral Deflection with Load Applied – 20-in Diameter ACIP Pile – Peptide 1 Facility

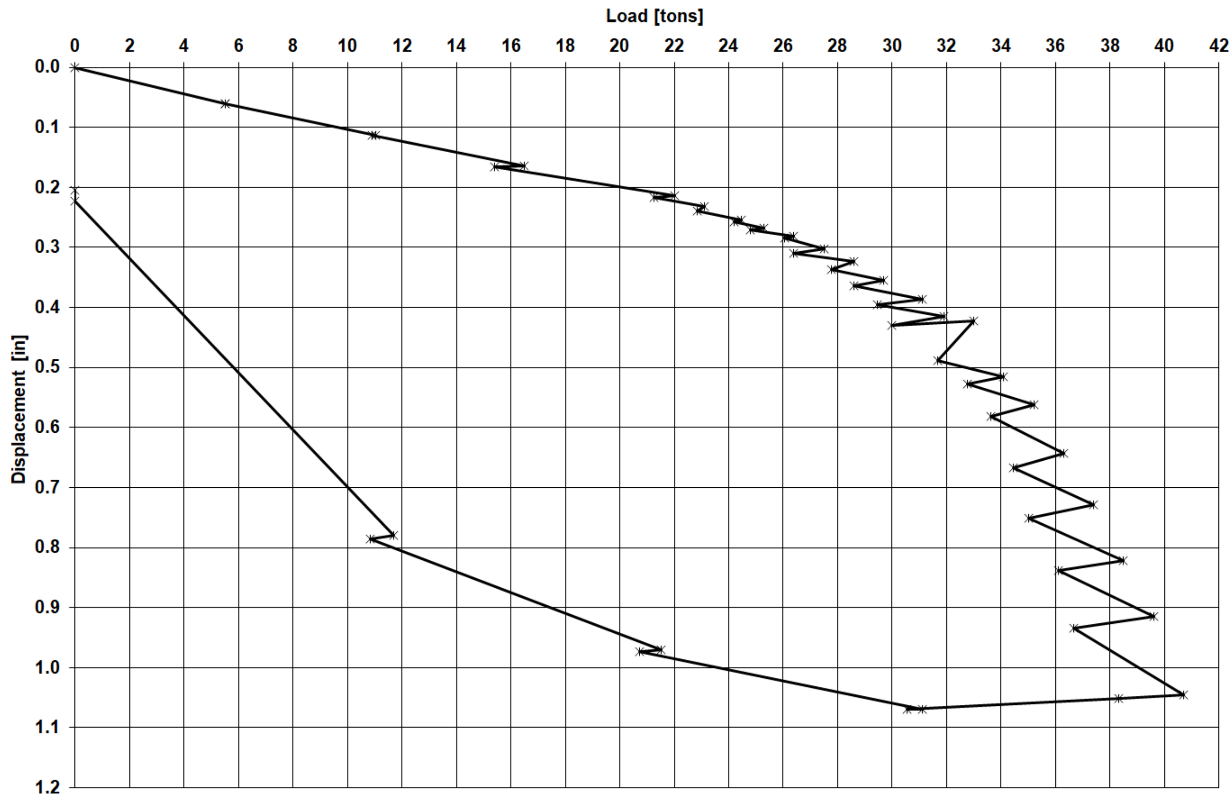


Fig. 13. Lateral Deflection with Load Applied to 1-inch Total Deflection – Peptide 1 Facility

FIELD COORDINATION AND DESIGN EXECUTION

General

Successful execution of the foundation support for the Lilly Medicine Foundry in Lebanon relied heavily on close coordination between field operations, engineering teams, and the construction manager, Fluor. From the outset, the field team had to navigate the unique challenge of beginning pile installation prior to the building being 100% designed. This meant that field operations were carried out based only on the approved-for-construction RIs of the design, while simultaneously staying agile and responsive to design updates as they became available.

Communication between the field and engineering was essential to maintaining schedule without sacrificing quality or safety. The team ensured that pile caps not yet cleared for construction were clearly identified and placed on hold until designs were finalized. This approach allowed work to progress efficiently in the areas that were ready, while maintaining flexibility for the remaining portions of the project.

Engineering also had to remain responsive to unexpected conditions encountered in the field. One such condition was refusal—where the drill rig could not penetrate further to reach the design toe elevation. When this occurred, the engineering team worked quickly to assess the load-carrying capacity of the achieved pile and, if necessary, identify a new location for a replacement pile to meet structural requirements. This dynamic collaboration between field personnel and engineering allowed for real-time problem solving and kept production moving without delay.

Constructability Challenges

Pile chipping proved to be one of the more logistically demanding phases of the project due to the depth of some pile cutoffs, which in certain caps exceeded 10 feet below working grade. Because these deep cutoffs could not be dipped to elevation at the time of installation, they were left long and required chipping down to final elevation after excavation (Fig. 14).

The chipping process involved multiple layers of coordination. After excavation and mud mat installation, surveyors would lay out the pile cutoff elevations and center bar locations. In cases where piles needed to be chipped multiple feet, a hydraulic pile crusher suspended from a track-mounted crane was employed. The crusher, composed of a donut-shaped frame equipped with hydraulic pistons, compresses the pile from all sides until the concrete fractures and breaks away. The concrete spoils are then collected in a skip pan and removed from the excavation area by crane.

Once piles were chipped and bars were trimmed to design levels, the pile cap construction could begin. The site works contractor would then form and pour the pile caps. This sequence required careful planning to ensure safety and efficiency, particularly given the number of trades and equipment operating in confined excavation zones.

Fig. 14. Excavated Piles to be Chipped to Final Cut-off Levels

Sequencing and Coordination Between Structures

The sequence of installation varied across the site. For example, in Peptide 1, most rigid inclusions (RIs) were installed prior to excavation. This allowed for installation of ACIP piles and RIs from the same working surface but introduced challenges later during excavation, as crews had to dig around already-installed rigid inclusions.

In contrast, at the Small Molecule 1 structure, approximately half the ACIP piles were installed, excavation began on the far west side of the building where there was sufficient separation from active drilling operations. This allowed excavation, pile chipping and pile-cap construction to run concurrently with pile installation. After pile caps were completed, drilling crews returned to install the 14-in diameter rigid inclusions in the backfilled zones between caps. Executing this safely and efficiently required careful coordination among all contractors involved, and the project team did an excellent job of maintaining clear communication and collaboration throughout.

Each execution strategy presented distinct advantages and challenges. The approach used at the Peptide 1 building (installing both ACIP piles and rigid inclusions prior to excavation) allowed for earlier completion of the foundation scope and reduced potential conflicts with other contractors during later phases. This method promoted schedule certainty and minimized remobilizations. However, it introduced logistical complications for the excavation contractor, particularly working around numerous rigid inclusions embedded within the building footprint, potentially slowing progress and requiring additional care during earthwork operations.

Conversely, the sequencing at the Small Molecule 1 structure, with delayed installation of rigid inclusions until after pile caps were completed and backfill operations concluded, allowed excavation and cap construction to proceed more rapidly without obstructions. While this phased approach offered advantages in terms of excavation efficiency, it introduced challenges for pile installation, including the need for re-establishing access, preparing stable working surfaces, and coordinating work in tighter spaces. In some instances, RI installation outpaced the availability of work areas, leading to idle time and reduced productivity.

From the authors' perspective, completing all deep foundation elements ahead of excavation remains the preferred strategy, as it allows the foundation contractor to operate continuously and exit the site prior to structural activities. However, the decision must be made with consideration for the overall project sequencing, site conditions, and trade coordination to ensure the safest and most efficient outcome for all stakeholders.

REFERENCES

- Chin, F.K., (1970), "Estimation of Unit Load on Piles not Carried to Failure", Proc. 2nd Southeast Asia Conference on Soil Engineering, 81-90.
- Coduto, D.P. (2001) Foundation Design: Principle and Practices. 2nd Edition, Prentice Hall, Upper Saddle River.
- Davisson, M.T. (1972) "High-capacity piles". Proceedings of the Lecture Series *Innovations in Foundation Construction*, ASCE, Illinois Section, 52 pp.
- Davisson, M.T. (1993) "Negative skin friction in piles and design decisions" *Proceedings*, Third International Conference on Case Histories in Geotechnical Engineering, St. Louis, 1793-1801.
- Kulhawy, F.H. and Chen, J-R. (2005) "Axial compression behavior of augered cast-in-place piles in cohesionless soils" *Advances in Deep Foundations, GSP* 132, ASCE.
- NeSmith, W.M. and Siegel, T. (2009). Shortcomings of the Davisson Offset Limit Applied to Axial Compressive Load Tests on Cast-in-Place Piles**.** *Contemporary Topics in Deep Foundations*. Selected Papers from the 2009 IFCEE. GSP 185**,** pp. 568 – 574. Orlando FL, 15 – 19 March 2009.
- Randolph, M. F. and Wroth, C. P. (1978) "Analysis of Deformation of Vertically Loaded Piles", Journal of the Geotechnical Engineering Division, ASCE, 104(12), 1465-1488.
- Robertson, P.K. (2016). "Cone penetration test (CPT)-based soil behaviour type (SBT) classification system — an update". Canadian Geotechnical Journal, 53, 1910-1927.
- Studlein, A.W., Reddy, S.C., Evans, T.M. (2014). "Interpretation of Augered Cast in Place Pile Capacity Using Static Loading Tests." The Journal of the Deep Foundations Institute. Vol 8, No. 1, pp. 39 – 47. 09 June 2014.