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CUSTOM HELICAL PILE USE FOR REFINERY REVAMP: A CASE STUDY

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ABSTRACT

A refinery expansion project for Marathon Petroleum Company in Southern Louisiana required existing medium to large pipe racks to be expanded for additional capacity. Foundation modifications were necessary in order to add levels to the existing pipe racks. New piles were added and existing pile cap foundations were expanded. Several different pile types were considered. Custom designed helical piles were chosen, because they can be installed quickly with low overhead restrictions, produce minimal soil spoils, and are cost effective. The load capacity required for piles was higher than typical, off-the-shelf helical piles. In particular, the lateral load capacity demand was high due to wind loads of a hurricane prone region. Methods used to design and size helical piles are summarized including lateral pile analysis, bearing capacity calculations, and theoretical capacity to torque ratio estimation. Pile load testing was performed in order to confirm design capacities of the helical piles under axial and lateral loading. Measured axial and lateral deflections compared well with predicted values. This case history evaluates the effectiveness of modern helical pile design methods within the context of a real and practical example.

KEYWORDS

Helical Piles, Deep Foundations, Pipe Caps, Revamp, Non-building structures

INTRODUCTION

One of the many challenges for engineering and construction of refinery revamp projects is determining how to effectively expand the capacity of existing foundations for additional equipment, piping and other facilities.

This paper provides a case study of a recently completed refinery revamp project in which custom designed helical piles provided an effective solution to the challenges faced when expanding the capacity of existing pipe racks. The recommendations from the authors based on the project completion results are provided at the end of the paper.

PROJECT OVERVIEW

This Case study is based on a pipe rack modification project for the Marathon Petroleum Company refinery facility in Garyville, Louisiana. The governing code specified design lateral loads are hurricane winds due to the proximity to the Gulf coast. A robust lateral load capacity was required of the additional piles because of the hurricane wind loads.

PILE SELECTION

There were three different pile types initially considered in the project; 16-inch diameter auger-cast piles, 16-inch square pre-stressed concrete driven piles, and helical piles. Low-overhead auger-cast piles are very labor intensive and produce a large quantity of spoils for disposal. A photograph of a typical low-overhead auger-cast pile drilling machine is shown in Figure 1. Driven piles require high headroom for installation with additional risk of disturbing or damaging the existing pipes, duct banks, or foundations.

The selected pile type was base upon constructability and cost effectiveness. The modification to the existing pipe racks required the pile to be installed in locations with low overhead obstructions. The obstructions were typically about 15 feet above grade; however, some obstructions were as low as 10 feet above grade. Minimizing the number of new pile rows in order to mitigate the group pile shear reduction factor was a challenge for the engineers because of the existing conditions including existing battered precast piles. Underground obstructions were carefully avoided by studying existing underground drawings, exploratory trenching, and hydro-excavation pilot holes at the location of piles.

Helical piles were chosen for the speed of installation in low-overhead conditions which played a role in being the most economical solution. Helical piles minimized spoils while providing the optimum pile layout with the aid of high torque, low RPM motors, which allow advancement with minimal soil disturbance. A photograph of a typical low-headroom helical pile installation is shown in Figure 2.

Figure 1- Low Overhead Auger Cast Pile Installation (Courtesy of ...)

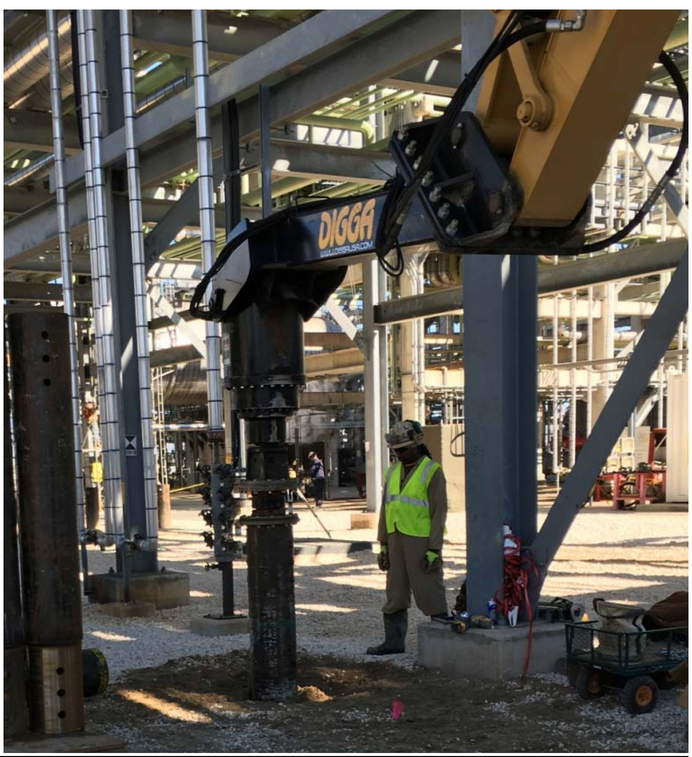

Figure 2- Low Overhead Helical Pile Installation (Courtesy of Cajun Deep Foundations, LLC)

PILE DESIGN

Table 1 shows the pile design parameters used for this study.   The pile capacity was based upon the capacity of a 16-inch diameter auger cast pile. The custom designed helical pile was required to have a large lateral capacity to match that of an auger cast pile. A schematic diagram of the final helical pile design is shown in Figure 3.

TABLE 1- Pile Design Capacities

Type of Loading	Axial Compression	Axial Tension	Lateral (horizontal) load (based on free-head) [5/8-in deflection]
Sustained (dead and Live)	60 kips	45 kips	9 kips
Transient (wind or seismic)	80 kips	60 kips	12 kips

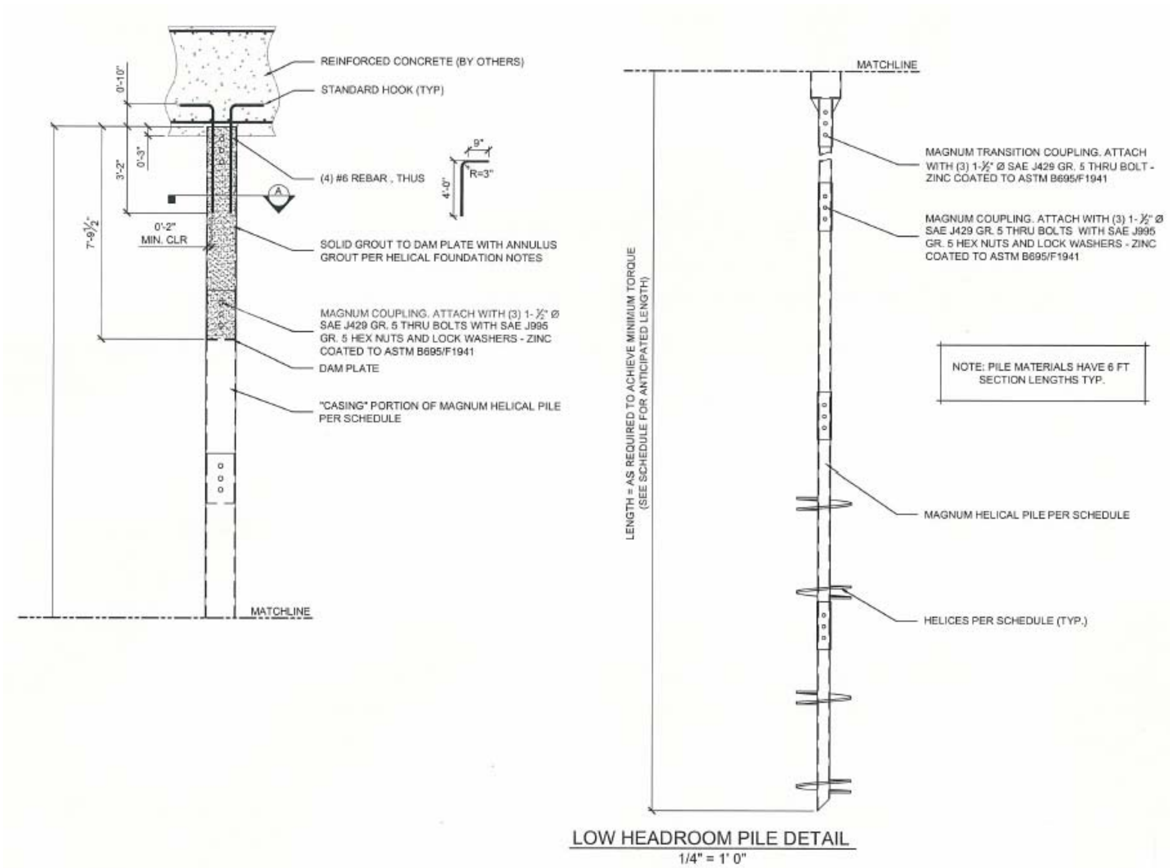

Figure 3- Pile Schematic Details (Courtesy of Magnum Geo-Solutions, LLC)

The generalized soil profile in this area consist of recent deposits of medium stiff clays and sands with an average unconfined compressive strength of 0.5 tsf overlying a stiff clay Pleistocene beginning at a depth of 34 to 42 feet below the planned top of pile. The Pleistocene had a lower bound unconfined compressive strength of 1.2 tsf. In order to provide better economy, a composite helical pile was chosen consisting of an upper 16 feet long, 13-3/8" diameter by 3/8" wall steel casing coupled to a lower section consisting of 5.5" diameter by 0.47" wall high-strength steel shaft with four 24" diameter helical bearing elements. Each helix had a 6" pitch. Helical pile properties and installation criteria are summarized in Table 2.

TABLE 2- Helical Pile Installation Criteria

Shaft size	Helix	Pitch	Installation Torque	Anticipated Length	Minimum Casing Length	Batter Angle
13.38" x 3/8 5.5" x 0.47"	4qty 24"dia.	6"	72,000 ft-lbs	46-ft to 54-ft	16-ft	0

Helical bearing elements were sized to provide the required bearing area in the Pleistocene layer using traditional individual bearing and cylindrical shear methods described in Perko (2009) [3]. Adhesion along the larger diameter upper steel casing was accounted for by taking the undrained shear strength as determined by unconfined compression tests and multiplying by 2/3 to account for soil-to-steel interaction. It was estimated that the helices would generate 88% of the required capacity of the pile and adhesion along the upper casing would produce the remaining 12% plus some additional resistance.

Since the foundation modifications on this project are to support an existing pipe rack with new loads, the structure was sensitive to total pile head movement. One of the challenging aspects of helical pile design is to estimate the total pile head deflection under design and maximum test loads. Theoretical methods for helical pile head deflection estimation are not widely known. Helical pile head deflections were estimated based on a model derived from the author's experience, published empirical test data, and well-known geotechnical relationships. The postulates describing the deflection model are contained in Table 3. The first two model points are based on the well-known Davisson offset method [4] which essentially states that pile tip deflection on the order of 2% of the average pile shaft diameter is required to mobilize the ultimate adhesion strength of a friction pile. The forth model point is based on several hundred measurements of pile soil deflection for various helical piles in different soil conditions by Cherry and Perko (2013) [5]. The final model point is based on the Modified Davisson offset method presented in ICC-ES Document AC358 [2].

The theoretical deflection curve based on these model points for the Marathon project is contained in Figure 4. In general, the theoretical pile head deflection curve developed during the design phase correlated well to actual test pile results. Pile testing is discussed in a subsequent section.

TABLE 3- Helical Pile Deflection Model

Model Point	Soil Deflection	Elastic Deflection	Load, P	Reference
1	0	0	0	Start Point
2	0	50% PL/AE	50% F_a	Davisson [4]
3	Δ_2 = 2% D_s	PL/AE	F_a + (Δ_2/0.375) 50% P_H	Davisson [4]
4	3/8"	PL/AE	F_a + 50% P_H	Cherry [5]
5	10% D_H	PL/AE	Fa + 100% P_H	AC358 [2]

D_s = Average Shaft Diameter (in)
F_a = Estimated Ultimate Shaft Adhesion Strength (t)
P_H = Estimated Ultimate Helix-Soil Capacity (t)
D_H = Average Helix Diameter (in)
L = Shaft Length (in)
A = Shaft Gross Area (in^2)
E = Elastic Modulus of Steel (tsi)

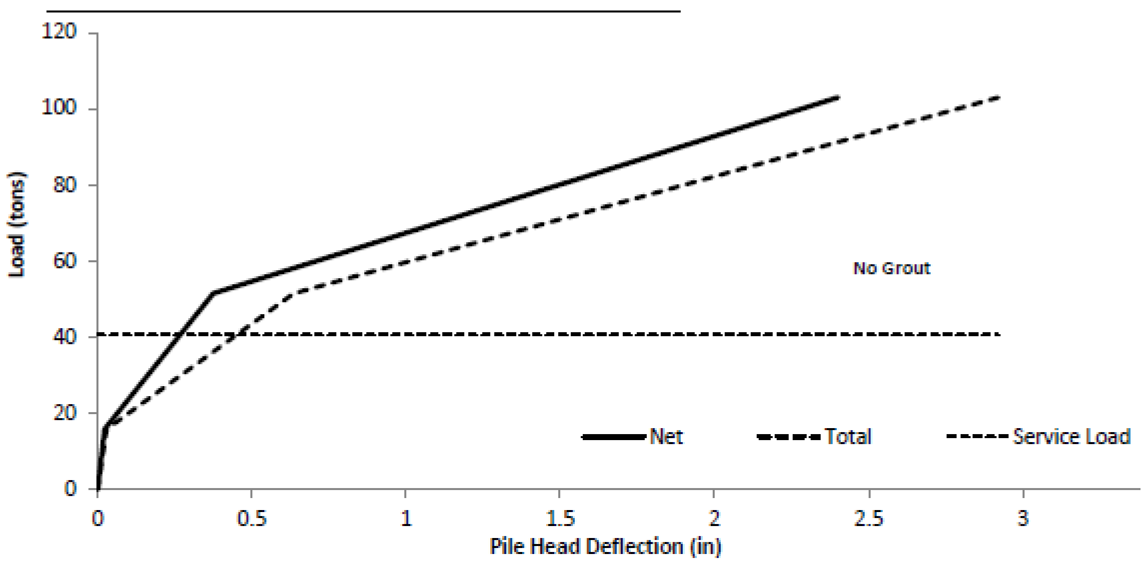

Figure 4- Predicted Vertical Pile Head Deflections (Courtesy of Magnum Geo-Solutions, LLC)

Another important aspect of helical pile design is lateral pile analysis. Project contract documents required a maximum total pile head deflection of 5/8" at design lateral loads for the free head condition. L-Pile Software by Ensoft, Inc. was used to estimate pile deflection under lateral loads. Auto generated p-y curves were applied. Structural properties of pile shaft materials were calculated for reduced sections after taking a 75 year corrosion allowance following methods contained in AC358[2]. A plot of L-Pile results is contained in Figure 5. In general lateral pile head deflections based on L-Pile analysis for the composite helical pile compared well with actual load test results. Lateral load testing is discussed in a subsequent section.

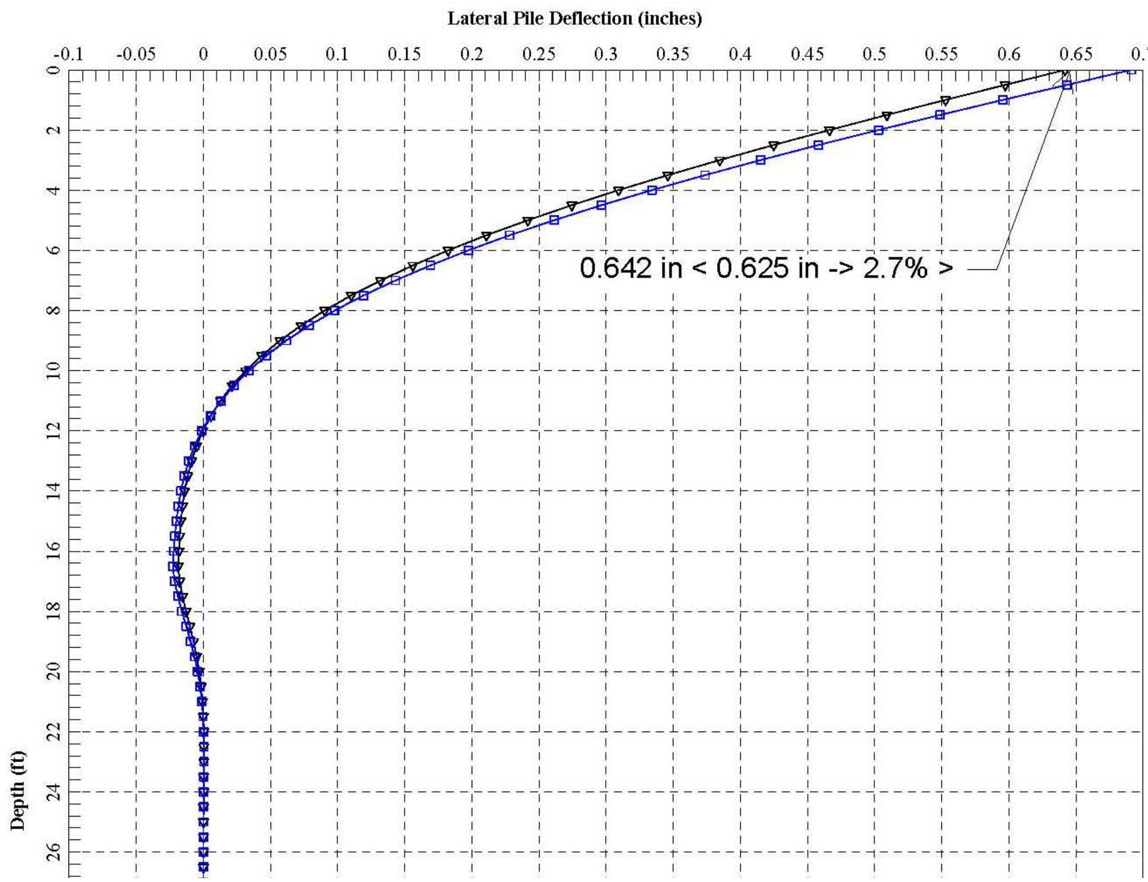

Figure 5- Predicted Lateral Pile Head Deflections (Courtesy of Magnum Geo-Solutions, LLC)

A final aspect of helical pile design was the determination of theoretical minimum installation torque for pile acceptance and field verification. The relationship between installation torque and helical pile capacity is well known for uniform diameter shafts [2,3]. Methods of predicting required torque for composite piles are less well-known. For this project, the minimum torque required was calculated using known methods [3] and a capacity to torque ratio, K_t, equal to 4.7 for the 5.5" diameter lead section and adding this torque to the torque predicted for the upper casing section. The torque required to overcome adhesion of the upper casing section was determined by calculating the total adhesion and multiplying by the radius of the casing.

PILE INSTALLATION

Helical piles were installed by Cajun Deep Foundations, LLC of Baton Rouge, LA with a CAT 323F excavator equipped with a two speed, 110,000 ft-lb Digga torque motor and a Digga jib attachment for added reach. Despite challenging conditions, overhead constraints, and access restrictions, the contractor was able to install an average of 15 piles per work day. Helical pile sections ranged from 15 feet with no overhead restrictions to as short as 4 feet where low overhead conditions prevailed. Helical pile sections were bolted together. All piling materials were manufactured by Magnum Piering, Inc. of Cincinnati, Ohio and trucked to the site in weekly recurring shipments. Approximately 1,200 piles were installed.

Photos showing examples of the challenging installation conditions are contained in Figures 6 through 8. The image in Figure 6 shows the installation machine set outside of the pipe rack area with jib arm reaching between existing braces to the pile location below. The image in Figure 7 shows the same installation machine reaching over an existing low pipe rack and alongside the existing taller pipe rack. The image in Figure 8 shows the hydraulic machine parked under the existing pipe rack and installing a pile within inches of an array of vertical pipes.

<u>Figure 6- Limited Access Helical Pile Installation (Courtesy of Cajun Deep Foundations, LLC)</u>

<u>Figure 7- Cross Equipment Helical Pile Installation (Courtesy of Cajun Deep Foundations, LLC)</u>

Installation torque was measured using a redundant system consisting of a wireless in-line torque sensor and differential hydraulic pressure. Torque and depth readings were obtained every 3 feet during installation.

Figure 8- Confined Area Helical Pile Installation (Courtesy of Cajun Deep Foundations, LLC)

PILE TESTING

Axial and lateral tests were performed on helical piles. Pile load tests were conducted on site with the sacrificial helical piles matching the size and configuration used for the final design on the project. After load testing, the reaction piles and helical test piles were removed by unscrewing the piles and backing them out of the ground. Remaining holes were backfilled from the surface.

COMPRESSION LOAD TEST

The test was conducted in accordance with ASTM Spec. D-1143-07 “Maintained Test” Method. The test pile platform was supported laterally to prevent sway, and isolated from the pile itself. The load was applied by a calibrated hydraulic jack. The tests were performed with an anticipated failure load of 162 kips.

The total test load was 200% of the anticipated pile design load on individual piles and was applied in increments of 25% of design load. Each load increment was maintained until the rate of settlement was not greater than 0.01 in. (0.25mm)/hour or until 2 hours have elapsed, whichever occurred first. After the required holding time the test load was removed in decrements of 25% of the total test load with 1 hour between decrements.

Settlement and rebound readings were taken to an accuracy of 0.01 in (0.25mm) before and after the application of each new load increment or the removal of a load increment. Additionally, not less than six immediate settlement and rebound readings were taken during each increment to define properly the shape of the time-load curves.

The test set-up is shown in Figure 9 and consisted of helical reaction piles and a load frame. Secondary deflection readings were taken using a ruler and optical transit.

<u>Figure 9- Compression Load Test (Courtesy of Magnum Piering, Inc.)</u>

TENSION LOAD TEST

The test was conducted in accordance with ASTM D-3689 "Quick Test" Method. The tensile load was applied by hydraulic jack supported on test beam. The ends of the test beam were supported by cribbing. The tests were performed with an anticipated failure load of 114 kips.

The test load was applied in increments of 5 % of the anticipated failure load. Each load increment was added in a continuous fashion and immediately following the completion of movement readings for the previous load interval. Load increments were added until reaching the anticipated design load. During each load interval, the load was kept constant for a time interval of not less than 4 min, using the same time interval for all loading increments throughout the test. The load was removed in five equal decrements, keeping the load constant for a time interval of not less than 4 min, using the same time interval for all unloading decrements. Time interval for the failure load was longer to assess creep behavior and for the final zero load to assess rebound behavior.

Test readings taken were recorded at 0.5, 1, 2 and 4 min after completing the application of each load increment, and at 8 and 15 min when permitted by longer load intervals. Record test readings taken at 1 and 4 min after completing each load decrement, and at 8 and 15 min when permitted by a longer unload intervals. Readings were taken at 1, 4, 8 and 15 min after all removing all loads.

The test set-up is shown in Figure 10 and consisted of twin load beams with web stiffeners and cross plates. The load beams rested on stacks of timber dunnage resting on the ground surface. Secondary deflection readings were taken with ruler affixed to the pile and optical transit.

Figure 10- Tension Load Test (Courtesy of Magnum Piering, Inc.)

LATERAL LOAD TEST

The lateral load test was conducted in accordance with ASTM D-3966 "Standard Loading Method". The load was applied by hydraulic jack acting between two test piles. The load was applied at a depth below ground surface corresponding approximately to the planned top of pile location. The test was performed with 1 test trial and anticipated failure load of 23.8 kips. A total test load equal to 200 % of the proposed lateral design load of the pile group were applied. A photograph of the lateral test set-up is shown in Figure 11. The test piles were spaced approximately 7 feet on-center. Deflections of both piles were measured simultaneously thus providing two lateral tests with one set-up.

Figure 11- Lateral Load Test (Courtesy of Magnum Piering, Inc.)

RESULTS AND DISCUSSION

Figures 12, 13 and 14 provide the results of the pile compression, tension, and lateral load tests, respectively. It can be seen from the test results that the custom designed helical piles performed well with respect to the design capacity. Total pile deflection at the design vertical loads for compression and tension and at the design lateral loads were approximately 0.4 inch. The piles held the maximum test load of 200% for the long duration hold period.

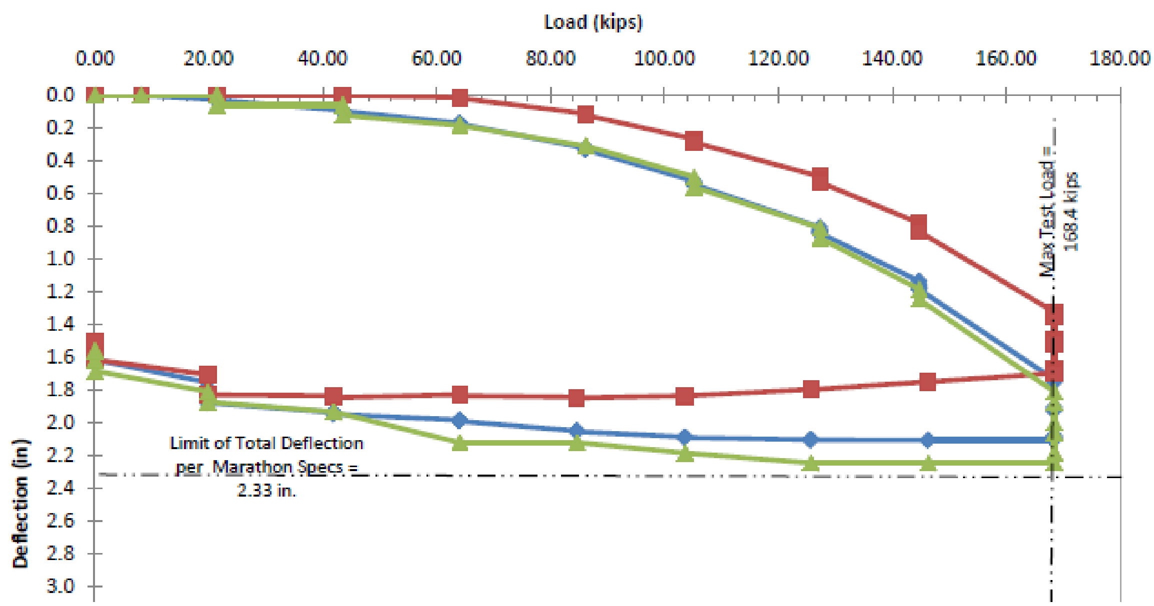

Figure 12- Typical Compression Test Results (Courtesy of Magnum Geo-Solutions, LLC)

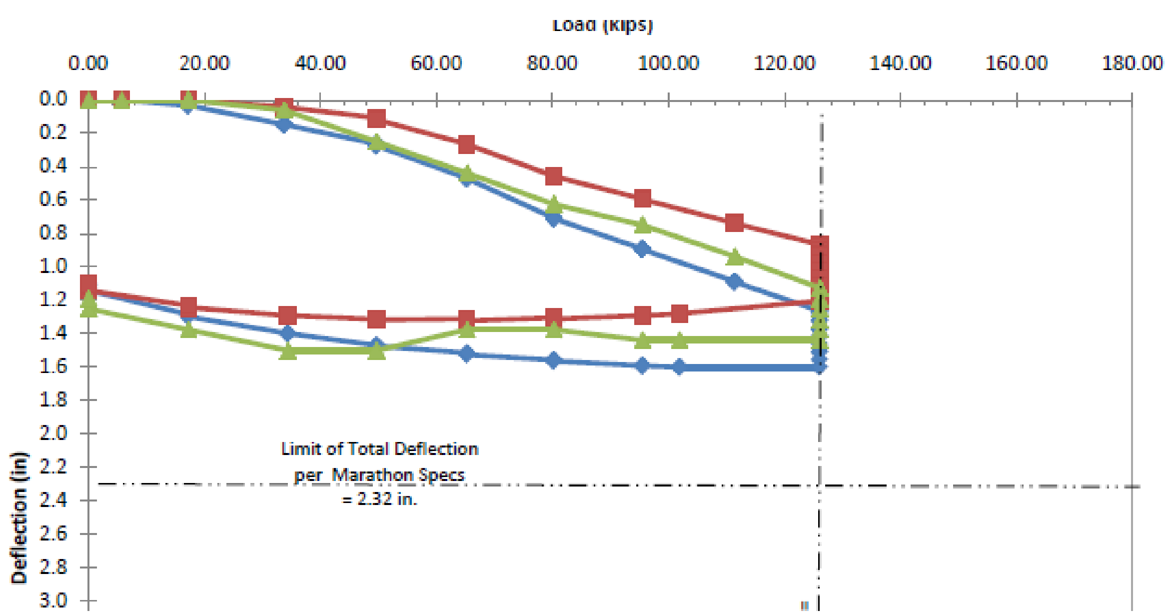

Figure 13- Typical Tension Test Results (Courtesy of Magnum Geo-Solutions, LLC)

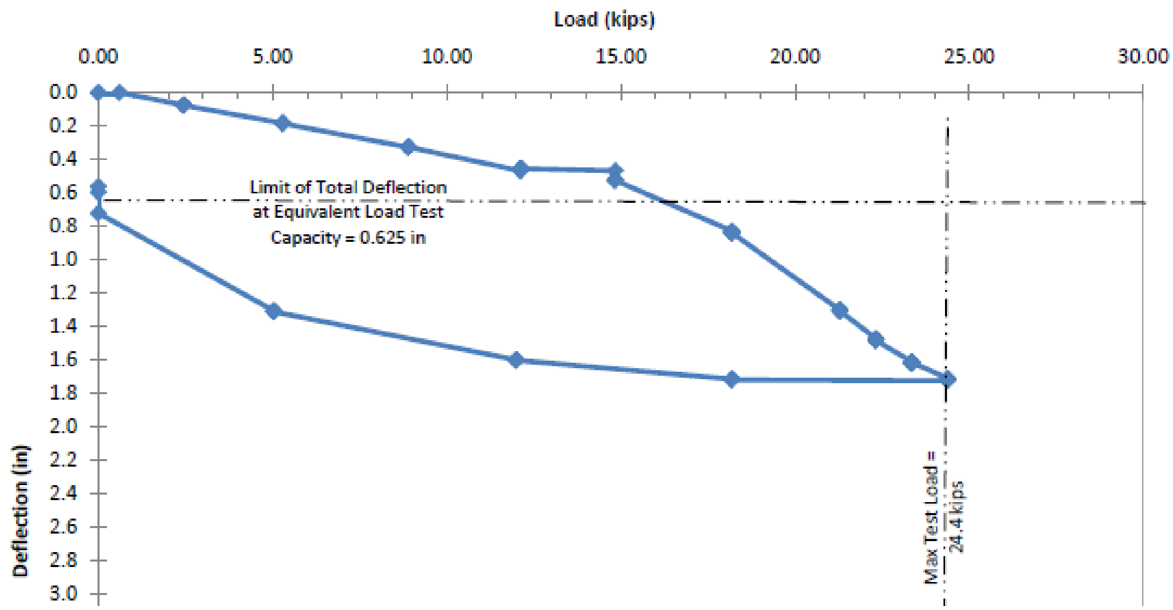

Figure 14- Typical Lateral Test Results (Courtesy of Magnum Geo-Solutions, LLC)

RECOMMENDATIONS AND CONCLUSIONS

In this paper, the authors presented a case study of a recently completed onshore pipe rack revamp project in the United States in a hurricane prone region. The following presents our recommendations regarding custom designed helical piles for revamp projects:

1. The "Comparability of Pile Types" from Different Pile Types need to include many factors in order to make a fair comparison. Pile types need to include all of the project specific requirements in order to determine the correct pile type for the project. There is no such thing as a "one type fits all" for refinery revamp projects. Some of the factors to consider are low overhead restrictions, contaminated soils, construction schedule, and restrictions with importing materials.
2. Relying on quick tests for pile testing does not always provide the full picture of pile axial deflection behavior in clay soils. It is common for Petrochemical companies to have to ASTM quick tests for pile testing programs because they are faster and less expensive. However, the maintain test will provide a more complete picture of the long term behavior of the piles under sustained axial loads.
3. A Refinery Structural Design is often Settlement Controlled Rather than Load Capacity Design Controlled. Refinery structures supporting deflection sensitive equipment often are controlled by the amount of lateral of vertical settlement rather than the design strength of the foundation.
4. To Ensure a Consistent Level of Helical Design Methodology and Performance Requires a Thorough Understanding of Different Codes' Elements. Practicing engineers working projects with helical piles often use various national and local codes/standards. Some may attempt to combine elements of different codes when specifying helical piles.

5. Clarifications on Performance Expectations of Helical Piles are needed. The authors realize that performance expectations of helical piles are not clearly documented in the standards and codes.
6. Helical piles have an advantage of the weldability of the head. In other projects where water is scarce, structure and support columns are directly welded to the helical piles hereby minimizing, if not, eliminating the need for concrete.

FUTURE STUDIES AND RESEARCH

The use of helical piles to support large refinery equipment has be slow to acceptance because of the lack of history and experience. The custom design of large helical piles for refinery work can benefit from experiences such as documented in this case study in order to help support the use for larger facilities. A greater understanding of the expected settlements under operating loads and long term durability and resistance to corrosion will help decision makers feel more comfortable with the choice of helical piles for refinery work.

ACKNOWLEDGEMENTS

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