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ABSTRACT

Deflection analysis of piles under lateral live loads in various soil conditions is presented herein. Field testing of lateral capacity was conducted at four test sites in California and Nevada, where weak surface soil provides insufficient lateral capacity for helical piles. In these areas of weak surface soil, as defined by field or laboratory testing, the most feasible solution for a foundation system may be the implementation of a deep foundation system such as a pier and grade beam or helical pile (HP) foundation system. Helical pile diameters that normally range from 1-1/2 to 4 inches provide minimal support when subject to lateral loads. An alternate structural member introduced as a Lateral Restraint Device (LRD), has been developed which increases the lateral capacity of the helical pile foundation system by increasing the soil-structure contact bearing area of the laterally loaded soil near the ground surface.

Data was compiled at four testing locations during the load testing of various length and diameter Lateral Restraint Devices. Helical pile and Lateral Restraint Device systems have limited published data for methods to determine the capacity of the system based on variable soil conditions. In addition to providing data collected during field testing that verifies the capacity of an LRD per unit area, a correlation of capacity at 1/2-inch deflection to Standard Penetration Test blow count data was established. This research demonstrates that lateral capacities of helical piles increased substantially with the implementation of an LRD, which can be addressed early in a site investigation with correlation to blow count data and laboratory testing programs.

INTRODUCTION

Helical pile (HP) deep foundation systems are often suitable for axial loads, but due to the high length-to-diameter ratio, provide limited lateral resistance, especially near the ground surface. Research provided herein briefly describes an HP deep foundation system, and discusses in depth, lateral support provided to the HP by Lateral Restraint Devices (LRDs). A typical plan and section schematic of the structural system is presented in Fig. 1.

An HP is comprised of a single helix or a series of helices structurally connected to a square or cylindrical shaft, generally varying in length from 7 to 30 feet, or greater. The HP is installed by applying torque to the shaft to advance the helices into the subsurface soil to the required depth and torque value. Vertical foundation loads are transferred from the foundation, through the shaft of the HP to the load bearing helices, creating a deep foundation system. The torque value is closely related to soil strength parameters and recorded at 1-foot increments during installation for the sites evaluated in this study.

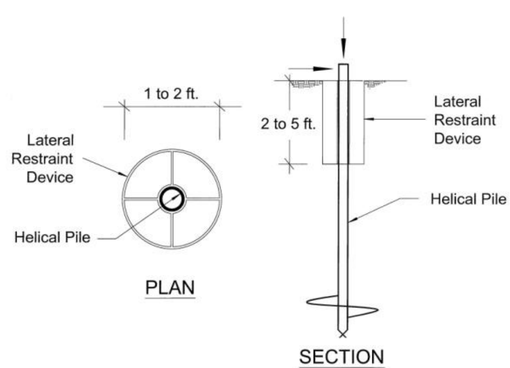

Fig. 1. Helical Pile with Lateral Restraint Device.

At sites where the HP shaft diameter is less than 8 inches, an LRD may be installed to develop lateral resistance for wind, seismic, and soil lateral pressures. The diameter of the LRD is designed from soil properties during the site exploration phase of the project inclusive of blow count data and field and laboratory testing. A system of LRDs is designed for installation on specific HPs within the foundation system and designed during the site exploration phase based on loading requirements. When the diameter of an HP shaft is equal to or greater than 8 inches, a lateral restraint system may not be necessary.

The LRD is generally constructed of steel, concrete, or polyvinyl chloride (PVC). If corrosive soil properties are present, steel may require galvanization, epoxy coating per ASTM 153, or cathodic protection.

LATERAL RESTRAINT DEVICE CONFIGURATION

The basis for design of an LRD system is dependent on site soil conditions and loading requirements. Typical LRD diameters range from 1 to 2 feet and extend to depths of 2 to 5 feet below the ground surface. The following Photos 1 and 2 show the vibratory installation of a 2 by 4 foot LRD laterally supporting a previously installed HP, see also Fig. 1:

Photo 1. Vibratory Installation of Lateral Restraint Device.

The LRD length and diameter are used to calculate the projected bearing area. Projected bearing area, described below, is considered to be the load transferring area and is defined as the diameter of the LRD multiplied by the length.

Sites that contain sandy soil with low cohesion properties benefit economically from the use of a steel or PVC member, eliminating the need to case the hole during excavation for a concrete LRD, or from the use of a more conventional concrete collar. This is also true for sites with groundwater levels in the upper 5 feet.

INSTALLATION AND EQUIPMENT

The most common methods for installation of an LRD are vibratory and excavation. The following shows the installed foundation system by vibratory methods:

Photo 2. Lateral Restraint Device and Helical Pier. Vibratory Installation.

INSTALLATION METHODS

Installation is not limited to the following methods of installation; however these are the most common.

Vibratory Installation

Vibratory installation, Photo 2, of the device is similar to installation methods used to install sheet piles. Generally, 40 to 50 units are installed per eight hours which is more productive than the excavation method used for concrete LRDs, mentioned later. There is no off-haul generated from this method of installation. A disadvantage of using vibratory installation is potential disturbance from settlement to surrounding structures caused from the vibrations.

Excavation

Excavation is necessary when installing a concrete LRD. The soil is excavated and used as a form for the concrete. This method requires off-haul and equipment capable of excavating the required LRD diameter around the shaft of the HP. Sandy soil with low cohesion properties may slough, not providing adequate formwork for the concrete LRD, and may be more adverse to high groundwater table conditions, as previously stated. Depending on loading conditions, steel reinforcement for the concrete may be required to prevent concrete cracking.

Testing completed during research reflects results from a steel framed LRD installed by vibratory methods, as shown in Fig. 1, and no concrete LRD has been tested for strength and performance in this research.

TESTING AND EQUIPMENT

Testing of lateral displacement during loading was performed with reference to ASTM D 3966, Standard Test Method for Piles under Lateral Loads. Incremental loads were applied to each unit until a minimum 1/2-inch deflection was measured. Results reflect a structural system with a free-end condition which allows rotation. A partial fixed-end condition resulting from embedding the top of the HP in a concrete grade beam provides additional stiffness against, and consequently more resistance, to lateral loading.

For Test Sites 1 through 4, loading measurements were recorded from either a strain gauge with readout device, or a hydraulic jack with data recorder. Load-versus-deflection was measured from a reference line installed 1 to 2 inches above the finished grade at a distance to eliminate influence from the lateral loading. Following the application of the ultimate load, ranging from 9 to 30 kips, loads were retracted and final deflections were recorded at zero load. Tables 1 through 4 present the field test data.

SITE DESCRIPTIONS AND TEST RESULTS

Several sites in California and Nevada were chosen for testing. All sites were tested to determine the load that resulted in a deflection of 1/2-inch. Results were correlated using the Standard Penetration Test (SPT) defined by ASTM D 1586, Standard Test Method for Penetration Test and Split-Barrel Testing of Soil. Data was corrected for a 140-pound hammer falling 30 inches-per-blow, and related to a standard split-spoon sampler with a 2-inch outside diameter. Correction factors [Robertson and Wride, 1997] were used to correlate all data to the corrected SPT, or SPT N_{60}.

The assumed bearing area, or LRD projected area, is the diameter of the unit multiplied by the depth of installation. Capacity will vary depending on the properties of the soil, such as soil arching, phi angle, cohesion, gradation, etc. Field test results, including SPT and torque data, were available for all sites; however limited laboratory data was available. A correlation between SPT N_{60} and LRD capacity at 1/2-inch deflection was derived based upon the data available.

Following the installation of helical piers, a hydraulic vibratory apparatus was used for each lateral device installation at Test Sites 1 through 4.

<u>Test Site 1. Williams, California</u>

Strength tests yielded SPT N_{60} = 8 with a range of N_{60} from 4 to 11, resulting in soil properties in the upper 5 feet classified per Unified Soil Classification System (USCS) as soft to stiff, sandy lean clay (CL). The in-situ dry density and moisture content were 105 pounds-per-cubic-foot (pcf) at 9 percent, respectively, with a Liquid Limit of 41 and Plasticity Index of 23.

Helical piers consisted of a 3 1/2-inch diameter central column installed to approximately 30 feet for Test 1 and 20 feet for Tests 2 and 3. LRDs at this site were 2 feet in diameter by 4 feet in installed length, providing a projected area of 8 square-feet.

Three tests were conducted with a correlation of the average load at the measured 1/2-inch deflection interpolated as 18.7 kips, with a range of 16.2 to 20.0 kips. Results for the three load tests are presented in Table 1:

Table 1. Load-versus-Deflection
Williams, California

Test Cycle	Load (kips)	Deflection (in.)
1	7	1/8
	10	3/16
	20	11/16
unloading	0	1/8
2	6	1/8
	10	5/32
	20	1/2
unloading	0	3/16
3	10	9/32
	15	7/16
	20	1/2
unloading	0	9/32

<u>Test Site 2. San Jose, California</u>

Strength tests yielded SPT N_{60} = 11, in the upper 3 feet. Classification from USCS resulted in stiff silt to clayey silt (ML to CL-ML).

The dry density and moisture content of the in place soil, as determined under laboratory conditions, were 108 pcf at 11 percent. A direct shear test indicated a phi angle of 19 degrees and cohesion of 480 psf.

Helical piers consisted of a 3 1/2 inch diameter central column installed to approximately 20 feet below existing grade. LRDs having a diameter of 1 foot were installed to 4 feet resulting in a projected bearing contact area of 4 square-feet.

Three tests were conducted and the correlated average load at 1/2-inch deflection was 11.3 kips. Results for Test Site 2 are shown in Table 2:

Table 2. Load-versus-Deflection
San Jose, California.

Test Cycle	Load (kips)	Deflection (in.)
1	7	1/16
	10	3/8
	11	1/2
unloading	0	1/8
2	5	1/4
	8	5/16
	9	1/2
unloading	0	1/4
3	7	1/8
	11	3/16
	13	7/16
	15	9/16
unloading	0	1/16

Test Site 3. North Las Vegas, Nevada

Test Site 3 resulted in SPT N_{60} = 25, with USCS yielding very stiff, sandy lean clay (CL). SPT N_{60} was correlated from the known installation torque at the location of the LRD to known installation torque and SPT N_{60} values onsite, with the correlation verified at several locations. The correlation was performed due to the limited laboratory testing data available at this site.

Following the installation of a 1 3/4 inch square HP to 13 feet, a 1-foot diameter steel LRD was installed to a total depth of 2 feet 4 inches. The LRD projected area was 2.3 square-feet. The interpolated load at 1/2-inch deflection was 19.5 kips.

Table 3. Load-versus-Deflection
North Las Vegas, Nevada.

Test Cycle	Load (kips)	Deflection (in.)
1	10	3/32
	18	7/16
	21	9/16
	22	15/16
unloading	0	1/8

Test Site 4. Pahrump, Nevada

Strength tests yielded SPT N_{60} = 13 with soil at this site classified as stiff silt (ML). Similar to Test Site 3, SPT N_{60} at this test site was derived from the known installation torque recorded during the installation of the HP and several locations of known installation torque and SPT N_{60}.

Following the installation of a 3 1/2-inch diameter HP to approximately 16 feet, the 1-foot diameter LRD was installed to a depth of 3 feet, resulting in a projected area of 3 square-feet. The interpolated load at 1/2-inch deflection was 17 kips. The following data, also shown on Fig. 2, resulted from testing:

Table 4. Load-versus-Deflection
Pahrump, Nevada.

Test Cycle	Load (kips)	Deflection (in.)
1	10	1/4
	20	5/8
	30	1-1/4
unloading	0	1/8

DISCUSSION

Data for Test Sites (TS) 1 through 4 were reduced to the Standard Penetration Test Number (SPT N) and further correlated to the corrected SPT, or SPT N_{60} [Robertson and Wride, 1997]. TS 1 and 2 had field testing values of SPT N_{60} in the upper 5 feet with TS 3 and 4 correlated to SPT N_{60} using torque-versus-depth readings recorded during HP installation as mentioned above. Torque-versus-depth was recorded in the field at each 1-foot increment from a data readout device connected to a hydraulic torque converter.

During testing at TS 1 through 4, lateral deflection tolerance was set at 1/2-inch based on excessive permanent deflection anticipated beyond 1/2-inch. SPT N_{60} multiplied by the LRD projected area, versus the load at 1/2-inch deflection is plotted on Fig. 2.

A trendline is fit through the data points for the four test sites. The trendline demonstrates increased capacity of the LRD related to increased soil bearing area as a function of the SPT N_{60}. Variation in the data is expected and may result from the following: (1) SPT N_{60} was taken as an average of the range in the upper 5 feet for TS 1 and 2, (2) SPT N_{60} was correlated from torque-versus-depth data during the installation of the helical pier to SPT N_{60} and the torque-versus-depth values onsite at several locations, and/or (3) use of SPT N_{60} rather than more accurate strength tests i.e. unconfined compression test, triaxial test, direct shear test, etc. Without extensive field and laboratory testing in close proximity of each test location, the correlation between LRD capacity and soil strength is expected to vary slightly from the trendline.

Several correction factors of SPT N to SPT N_{60} are based on factors which may include some or all of the following: (1) The energy ratio, which will differ from an automatic hammer, rope or pulley safety hammer, or manual hammer, (2) Rod length during sampling, (3) Sampler type i.e. 1 1/2 inch to 2 1/2 inch inside-diameter sampler, (4) Bore hole diameter, and (4) Anvil size. These correction factors were applied to all SPT data as applicable.

Figure 2 provides a method to determine lateral capacity of an LRD from its geometry and known SPT N_{60} values, with the following procedure:

1. Determine the SPT number, N, in the field during the site exploration phase and convert to SPT N_{60}.
2. Determine the required capacity from wind, seismic, and soil lateral pressure, with a factor of safety.
3. On the ‘X’ axis, find the required capacity from “2”, and find the corresponding SPT N_{60} multiplied by the LRD projected area value, on the ‘Y’ axis, from the trendline.
4. Divide SPT N_{60} * LRD projected area by SPT N_{60} to determine the required LRD projected area (LRD diameter and depth) in square feet.

Steps 1 through 4 result in the required projected bearing area of the LRD.

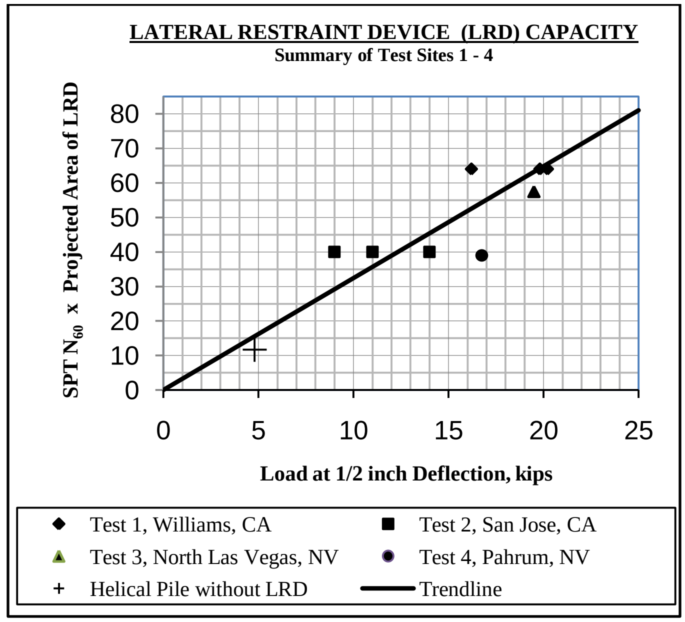

Fig. 2. LRD Capacity-versus-LRD Geometry x SPT N_{60}

The helical pier foundation system alone provides minimal lateral support for wind, seismic, and lateral soil pressure loads, when the diameter of the HP is less than 8 inches. Thus, the LRD was developed to transfer lateral loads to a larger soil area reducing lateral movement.

Due to the redundancy of the entire structural system when all HPs and LRDs are interconnected, the total resistance for the system is anticipated to be greater than the sum of the capacities of each individual member. Structural systems tested have a free-end condition. It is anticipated that a substantial increased lateral capacity will develop with a fixed-end condition developed during construction of concrete grade beams or a structural flooring system.

The data on Fig. 2 shows that at Test Site 1, the lateral capacity of a helical pile, laterally tested without a Lateral Restraint Device, produced a resistance of 4.8 kips at 1/2-inch deflection, compared to a 16 to 20-kip resistance at 1/2-inch deflection of a lateral restraint device and helical pier structural system.

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