Development of Foundation Systems for Solar Array at Owens Lake, California

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ABSTRACT

Owens Lake, located in Central California, has been a dry lakebed for almost a century, ever since the water rights in the Owens Valley were purchased by the City of Los Angeles and the water was diverted to the City. As a result of the diversion, the salt-laden lakebed sediments are picked up by the wind, creating major dust storms. Various dust mitigation projects have been implemented over the years. The most common method is the creation of shallow ponds. This method is effective but costly and highly water intensive. A large solar panel array has been proposed as a more sustainable method of dust control which would reduce water usage at the lake and also be a source of electrical power. However, the lake bed sediments are very poor soils, not generally considered viable for solar panel foundations. The poor quality of the soils made both tradition site investigation methods and traditional empirically based design methods inappropriate.

This paper describes an innovative testing system which was developed to allow for efficient field load testing of the foundation systems being considered. This project provides an example of how sustainability concerns have actually led to exploring viable foundation designs for a site previously considered to be unsuitable.

INTRODUCTION

Owens Lake has been a dry lakebed for almost a century, ever since the water rights in the Owens Valley were purchased by the City of Los Angeles, and the water was diverted to the City through the Los Angeles Aqueduct. The dry lakebed sediments at Owens Lake are exposed to wind erosion and as a result, dust emissions are a health concern. The Los Angeles Department of Water and Power (LADWP) has implemented various dust mitigation projects on over 10,000 hectares (39 square miles) within the lake bed. A variety of dust mitigation methods have been applied including shallow flooding, gravel cover, and managed vegetation. The vast majority of the dust mitigation has been accomplished by creating shallow ponds. This method is effective but costly because it is highly water intensive with significant losses to evaporation. In the 2011-2012 season over 91 million m^3 (74,000 acre-feet) of water was used for dust mitigation (LADWP, 2012, section 3.9).

To reduce the cost and water demand of dust mitigation, LADWP is considering the use of solar panels as a sustainable alternative to shallow ponds. The solar panels can help reduce wind shear on the ground surface and solar fields can be supplemented with a lesser amount of gravel cover to achieve the desired dust mitigation. Obviously, this approach can also generate significant revenue by generating electricity at the same time.

The feasibility of this concept depends largely on the cost of foundations required to support solar panels in Owens lakebed soils, which are generally considered unsuitable for foundations. Complicating the issue, these foundations will have to resist the lateral and uplift forces generated by high wind loads which are characteristic of the area. Typical foundations used for solar panels include spread footings, post piles, or helical anchors. These have been used successfully at many other sites, but how they might perform on Owens lake soils was unknown.

Due to the size of the project, it was recognized that even small improvements in foundation efficiency could be a significant factor in deciding if this site could be suitable for solar development. LADWP's long-range plans consider development of up to 2 GW of solar power generation in this vast, uninhabitable area. With the potential foundation quantities involved in a development of that size, even a slight variation in foundation efficiency might mean the difference between this site being feasible or not. Even though foundations at this site might be expensive compared to other potential solar generation sites, these costs might be more than offset by the benefit gained from reduced water usage.

With these issues in mind, and given the highly variable soil conditions across this vast site, it quickly became apparent that a specialized foundation load testing program would be the best way to gather data needed for decision making. Early evaluations by LADWP concluded that this project would likely be developed under a design competition approach. The development will likely utilize a design-build contract delivery system in order to incentivize the potential bidders to propose the most efficient foundation system. However, this approach also requires the establishment of data and other guidelines in order to establish a sound technical basis for comparison, and for meeting various design objectives. Hence, it was necessary to provide load test data for several different foundation types, to allow the market forces to work in identifying the optimum foundation system. This required acquiring a relatively broad spectrum of field load test data that can be used by future design-builders.

The preliminary project scoping identified a large number of variables at play (soil variability within a large site, different foundation types, different geometries for a given foundation type, different ground preparation methods, etc.). Considering the number of variables and field tests involved, it became obvious that a rapid field testing system was needed. Conventional load tests with jacking frames, and associated setup could not respond to the project's needs and budget. This paper presents the system devised to meet these needs and discusses its development and implementation. Details of the geotechnical data, analysis and interpretation is not a part of this paper, as it is anticipated to be covered elsewhere.

OWENS LAKE: GEOLOGIC CHARACTERISTICS

Owens Lake is a terminal lake located in Inyo County in eastern central California (Figure 1). The lake is bounded on the west by the Sierra Nevada range, on the northwest by the Inyo Mountains and on the southeast by the Coso Range. It lies at the western end of the Great Basin at the terminus of the Owens River.

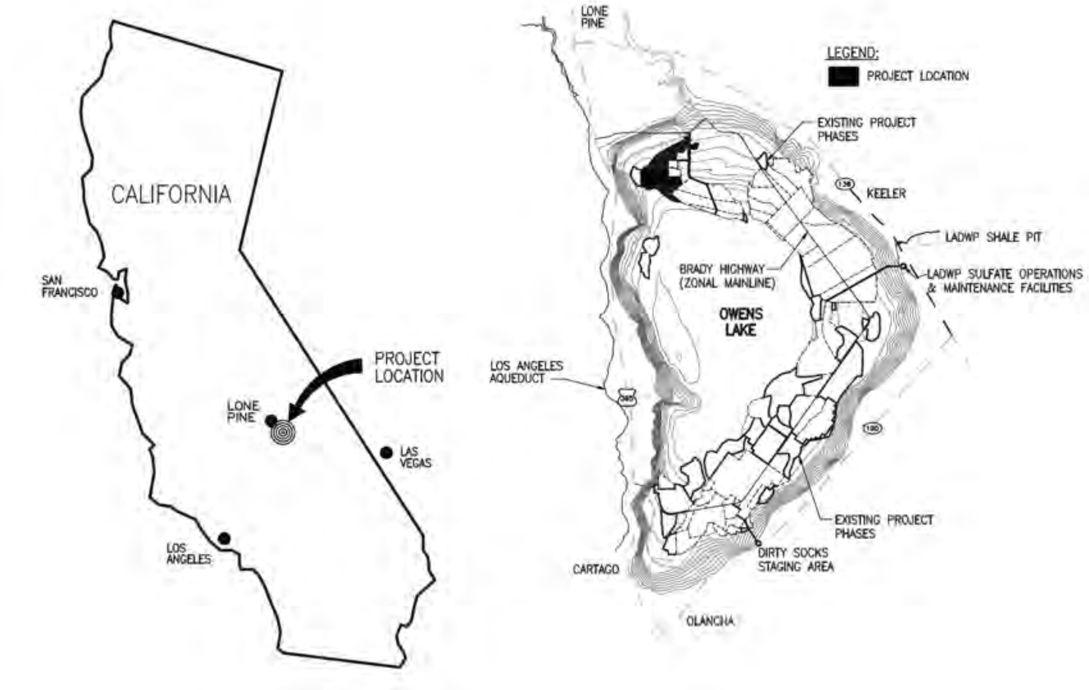

Figure 1. Site vicinity map

The sediments in Owens Lake consist of alternating layers of sand and gravel, silty-clay, and clay beds and lenses deposited through a combination of lacustrine, fluvial and alluvial fan mechanisms. Sediments extend to depths up to 2450 meters (8000 feet), (Hollett et al, 1991). There are some surface deposits of sand to depths of 0.5 to 3 meters (2 to 10 feet). The deposits consist of a thin layer of aeolian sand and clay intermixed with salt and fluvial deposits with varying thicknesses of sand, silty and clayey sand classifying as SP, SP-SM, SM, and SC in the Unified Soil Classification System (USCS). The sandy materials are underlain by fine-grained silt and clay lacustrine deposits of ancestral Owens Lake. The silt and clay typically classified as ML, MH, CL and CH.

These sediments have dry densities typically less than 13 kN/m^3 (80 lb/ft^3). As expected these sediments exhibit very low strength with undrained shear strength ranging from 5 to 10 kPa (100 to 2000 lb/ft^2) (CDM, 2008).

The field testing reported in this paper was conducted at two sites within Owens Lake pre-selected by LADWP for solar development. The sites were designated Foundation Study Site 1 (FS-1) and Geotechnical Study area 3 (GS-3). Both sites are located in the northern part of the lake as shown in Figure 2. These sites do not represent average conditions for soil deposits within the lake, but represent some of the more competent areas within the lakebed and were considered suitable for solar

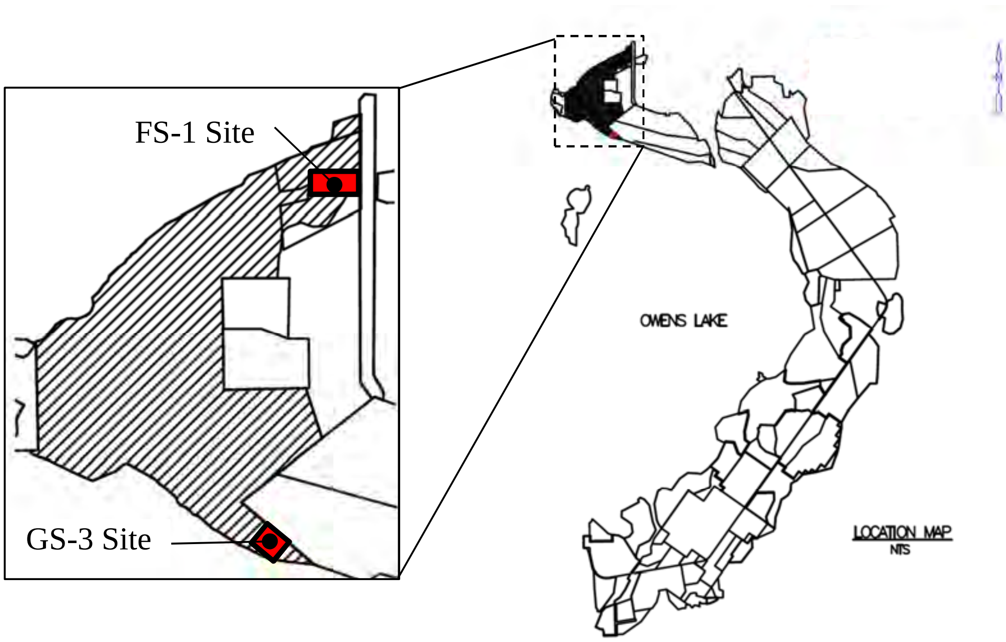

Figure 2. Locations of sites FS-1 and GS-3.

development, based on site reconnaissance, proximity to existing power lines, and accessibility.

The FS-1 site was selected as representing the average subsurface conditions within areas suitable for solar development. At this site the subsurface conditions consisted of a 2.5 cm (1 in) thick salt crust underlain by approximately 0.75 to 1.2 meter (2.5 to 4.0 feet) of sandy silt (ML). Below the layer of sandy silt was a sand/silty sand (SP, SM) layer approximately 0.75 m (2.5 ft) thick. This sandy layer was underlain by a deposit of interbedded silty sand (SM), sandy silt (ML), clayey silt (MH) and fat clay (CH) to a depth of 4.75 m (15.5 ft). Groundwater seeps were encountered at depths from 1.3 to 3.75 m (3.5 to 12 ft). The seeps generally occurred along the sandy seams.

The GS-3 site was selected as a marginal but potentially viable site for solar development. Similar to the FS-1 site, this location had a thin salt crust. This crust was underlain by approximately 0.3 m (1 ft) of silty sand. The silty sand layer was underlain by fat clay (CH) material approximately 0.3 m (1 ft) thick. Below the clay the deposit consisted of interbedded silty sand (SM), sandy silt (ML), clayey silt (MH) to a depth of 3 m (9.5 ft). The main difference between the GS-3 site and the FS-1 site was the relatively thin silty sand layer at the top of the GS-3 site compared with the thick silt and sand layers at the top of the FS-1 site. Table 1 summarizes the average soil properties at the two sites.

Table 1. Average soil properties at FS-1 and GS-3 sites (CDM Smith, 2012)

Area	Depth, m (ft)	USCS	w_c, %	γ_d, kN/m^3 (lb/ft^3)	c, kPa (lb/ft^2)	ϕ°	c_r, kPa (lb/ft^2)	ϕ_r°
FS-1	0 - 0.75 (0 - 2.5)	ML	31	13 (83)	8.4 (175)	30	6.0 (125)	30
	0.75 - 1.5 (2.5 - 5)	SP, SM, SP-SM	28	14 (89)	6.7 (140)	31	5.7 (120)	31
	1.5 - 2 (5 - 6.5)	CL, CH	46	11 (74)	-	-	-	-
	2 – 4.7 (6.5 – 15.5)	SP, SM, SP-SM	28	14 (89)	6.7 (140)	31	5.7 (120)	31
GS-3	0 - 0.3 (0 - 1)	SM	34	12 (77)	-	-	-	-
	0.3 - 0.6 (1 - 2)	CH	31	10 (63)	4.8 (100)	26	4.8 (100)	26
	0.6 - 3 (2 - 9.5)	SM with layers of ML, CL, SP-SM	30	14 (90)	9 (188)	33	7.2 (150)	31

PROJECT CHARACTERISTICS, DESIGN REQUIREMENTS, AND LIMITATIONS

The objective of the project was to determine the feasibility of using various foundations systems to carry the dead and live loads to support a large solar panel array and provide recommendations for foundation design. Due to the preliminary nature of the project, specific structural design loads were not available. Nonetheless, it was known that the vertical loads would be relatively small, approximately 5 kN (1 kip). Relatively large lateral and upward live loads were expected. It was assumed both lateral and uplift loads could be of the same order of magnitude as downward loads. It was assumed the solar array could sustain vertical displacements of 5 to 10 cm (2 to 4 inches) without significant performance problems. Based on the low loads, it was determined that viable foundations systems included: surface footings bearing directly on native soils, surface footings bearing on gravel pads underlain by a geosynthetic reinforcement, short pushed piles, and helical piles.

The site characteristics and loading conditions gave designers a number of concerns about the appropriateness of using traditional foundation design analyses to determine foundation capacities. The soil profile, consisting of a relatively strong crust underlain by very weak soils, combined with large lateral loads, were not conditions where traditional bearing capacity theory is particularly applicable. Similarly the short length and small diameter of the piles put these systems outside the database of most empirically derived pile capacity formulas. These concerns about the accuracy of traditional foundation design approaches combined with the relatively low expected design loads led to the decision to conduct full-scale field testing of the candidate foundation elements rather than relying on traditional empirical design procedures.

Given the challenging site and loading conditions, it was important to understand whether or not the site would hold promise for solar development without excessive foundation costs. In order to gather relevant technical data, it was decided to use field load tests rather than geotechnical explorations and correlations. It was also important to determine if a practical level of ground improvement could improve the performance of footings.

DEVELOPMENT OF FIELD TESTING EQUIPMENT

Requirements

Determining the feasibility of developing the site for solar power generation required testing of several combinations of variables including: different foundation types, different loading configurations, different ground improvement methods, and different potential sites. To meet the project requirements a foundation load testing system needed to be developed to satisfy the following criteria:

- Be self-contained to test in remote areas without commercial power supply
- Test combinations of loads and be portable to setup and dismantle in multiple locations
- Test foundations to failure to establish realistic design limits
- Apply and record biaxial loads
- Test under large foundation movements

The specific foundation systems to be tested included: spread footings, piles, and helical anchors. Additionally, the effect of including ground improvement below the spread footings was to be tested. The specific ground improvement tested included a gravel ballast placed below the footings with and without the use of a layer of geotextile.

Concept development

The key goal in developing the test equipment was to be able to evaluate a large number of test conditions in a practical manner. Using traditional foundation load test systems which rely on installing some form of a reaction frame was not feasible. The system designed uses an excavator as the reaction weight. This greatly increases portability compared to conventional load test setups.

Meeting the requirements to provide combined axial and lateral loads was challenging. Initial concepts used separate vertical and horizontal loading systems. However, the requirement to test under large displacement made it difficult to maintain two separate points of load contact during testing. Consequently, the concept was refined to a system which applied the load at a single point with the ability to vary angle of application of the force. Using a single load point proved to be a more effective way to meet the combined loading requirements.

There are some limitations in using an excavator as the load applicator. Some of these limitations are listed below with explanations of how these limitations were addressed and mitigated:

- The maximum test load that can be applied is limited. Beyond a certain load, the excavator will start to tip up. However, the loads achievable with such a system were considered adequate for the purposes of this study.
- The excavator arm and its linkages may have some play in them as loading progresses. This issue was mitigated by measuring the foundation movements with respect to a fixed ground reference.
- Long-term, steady and controlled loads cannot be maintained using an excavator. Hence, these tests are neither stress-controlled, nor strain-controlled in the traditional laboratory testing sense. However, ultimate foundation failure, even under relatively rapid loading provided valuable information for design. Even though some long-term behavior may remain unanswered using this approach, adequate safety factors can be used to account for such uncertainties.

Overview of test system

The testing system, shown in Figure 3, is made of four distinct components:

a) Excavator Attachment
b) Instrumented Swivel Unit
c) Foundation Attachment
d) Reference Frame with Position Sensors

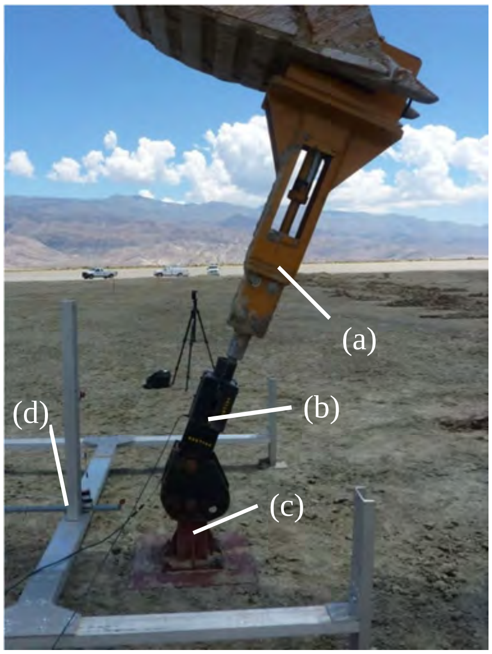

Figure 3. Test system showing (a) excavator attachment, (b) Instrumented swivel unit, (c) foundation attachment, (d) reference frame with position sensors

Movements of the system were measured from a stationary aluminum reference frame, isolated from the plate or pile test unit and soil movements. Movements were monitored with two inclinometers and two draw-wire sensors. The sensors measured the load, initial and final positions, and changes in position during testing.

Compression force was measured with a load cell housed within a guide in the swivel. Uplift force was measured with an S-beam load cell.

Excavator attachment unit

Two basic methods were devised to apply loads with an excavator. One method was to load using a hand pumped hydraulic piston mounted inside the excavator attachment as shown in Figure 3(a). The second method used the excavator arm pushing directly on the test piece, either without activating the hydraulic piston, or by using a simpler attachment unit as shown in Figure 4.

In order to transfer the applied foundation loads concentrically without inducing a moment, the tip of the ram was machined to a spherical shape. This tip fit loosely inside a chamber in the swivel unit, which had a matching concave shape. This attachment detail is shown in Figure 5.

Figure 4. Direct push attachment with pin grabber

(a) (b)

Figure 5. (a) Spherical tip of the ram and (b) interior of the swivel chamber instrumented swivel unit

The load measurement sensor for compression and lateral load tests consisted of 50-kip capacity bidirectional load cell. This load cell was mounted in a guide system, shown in Figure 6, which ensured all loads were transmitted through the load cell. This unit was fit to a pivot system with preset positions at angles of 0, 10, 20, 30, 40, 45 and 90 degrees from vertical, as shown in Figure 7.

Figure 6. Load cell held inside the swivel unit

(a) (b)

Figure 7. (a) Swivel unit bolted on the footing stem and (b) detail of preset positions to set the swivel unit angle to 0°, 10°, 20°, 30°, 40°, or 45°

FOUNDATION ATTACHMENT UNIT

The system was designed so it could quickly be changed to test footings, piles or anchors. The steel footing stem, shown in Figure 7(a), was fabricated to fit different adaptors. For footing tests, different sized plate adaptors were used. A typical 60 cm by 60 cm (2 feet by 2 feet) plate is attached to the stem in Figure 7(a). Another adaptor which was used to connect the top of the piles to the loading system is shown in Figure 8 (a), and a third adaptor fabricated to test screw anchors is shown in Figure 8 (b). The adaptors were designed to allow the application of axial downward loads, lateral loads, and uplift loads to piles and screw anchors.

(a) **(b)**

Figure 8. Foundation attachment unit (a) connected to a 9 cm (3.5") diameter pile (b) attached to the helical pile

MEASUREMENT SYSTEM

In order to study the load-deformation behavior of test foundations, it was necessary to continuously measure the position of the foundation over a large displacement range. This was accomplished by tethering a point on the foundation to two separate potentiometers on a fixed reference frame and measuring displacements with a pull line. From these measurements, the vertical and horizontal position changes of the foundation were calculated continuously. A tiltmeter mounted on the foundation stem recorded the foundation. This measurement system is shown in Figure 9. Potentiometers used in the system were able to record up to 30-inches of movement. The data acquisition system used a battery operated six channel data logger that could operate either independently or connected to a PC.

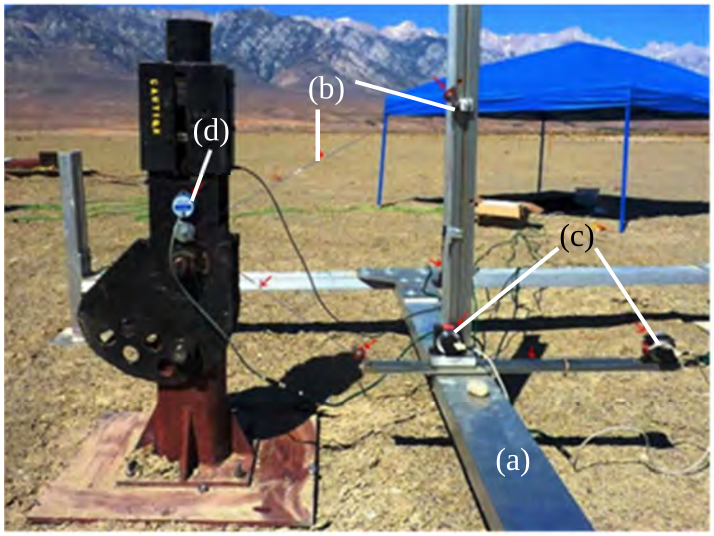

Figure 9. Measurement system showing (a) reference, (b) tether lines and pulleys, (c) potentiometers, and (d) tiltmeter

FOUNDATION TEST PROGRAM

Footing tests

Simulated test footings consisted of steel plates of various sizes. They were square or rectangular shaped, with the largest dimension not exceeding 1.2 m (4 feet). The footing sizes were selected considering the anticipated soil strengths such that bearing failure or noticeable settlement could be observed using practical loads. Vertical footing loads were applied by pushing down with the excavator bucket. Inclined footing loads were applied by locking the swivel attachment piece at the desired angle and then pushing with the excavator as shown in Figure 10. In order to understand the effects of soil improvement on footing performance, some tests were conducted on geosynthetic fabric topped with gravel. In some tests, the gravel was compacted prior to test, and in others it was not compacted. A typical test using fabric and gravel is shown in Figure 11.

Pile tests

Piles were tested under axial compression, pullout, and lateral loading at different locations. Pile sizes ranged from 7.6 to 16.6 cm (3 to 6.5 inch) diameter and 1.2 to 2.4 m (4 to 8 feet) long, closed-end steel pipes.

Pile compression tests used in this project are best referred to as pile penetration tests, rather than pile load tests. This is to distinguish the test method being used here from a traditional static pile load tests. The test setup with the excavator attachment and load cell provided the ability to continuously record pile resistance as a function of penetration depth during the insertion of the pile into the ground. The pile resistance measured at any given depth is the pile capacity at that depth, since pushing through that depth would require overcoming essentially the same skin friction and tip resistance.

Figure 10. Footing test after applying an inclined load

Figure 11. Footing test with fabric and gravel after a plate test

The pile pullout tests were conducted in a customary manner. Pile pullout tests were carried to complete pullout, with displacement data recorded throughout the test. Lateral pile tests were performed by rotating the swivel to a horizontal position and pushing laterally with the excavator bucket.

Helical anchor tests

Helical anchor tests were done under axial compression, pullout, and lateral loading conditions. A separate excavator attachment was developed to facilitate anchor installation. Installation torque was recorded as the anchors were rotated into position.

TEST RESULTS

A total of 42 footing tests, 19 pile tests, and 9 helical anchor tests were conducted at the site. A complete presentation of the results of the field testing is beyond the scope of this paper, but will be published in future articles. Figure 12 presents results from a typical footing test and represents the quality of data collected. The data from all the field tests was synthesized and compiled into a set of preliminary foundation recommendations. Table 2 presents the recommendations for footing capacities. The recommendations are presented for different foundation types and subgrade treatments, based on experimental results and design factor of safety considerations.

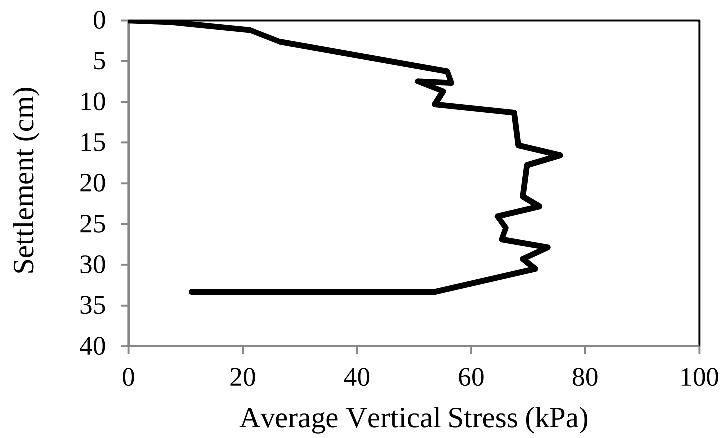

Figure 12. Typical settlement versus pressure graph with bearing failure (Plate test at GS-3, S2- 24”x24”- 20°)

Table 2. Summary of Design Parameters Based on Field Tests

Area	Foundation Type	Vertical Capacity	Uplift Capacity	Lateral Capacity
FS-1	Spread Footing	48 kPa (1,000 lb/ft^2)	Ballast	Ballast
	Spread Footing + Fabric + Gravel	72 (1,500 lb/ft^2)	Ballast	Ballast
	Pile (2.4 m $\times$ 9 cm dia.)	18 kN (4 k)	3 k	1.3 k
	Anchor (2.4 m x 2.7 kN-m torque)	44 kN 10 k	10 k	0.8 k
GS-3	Spread Footing	38 kPa (800 lb/ft^2)	Ballast	Ballast
	Spread Footing + Fabric + Gravel	72 kPa (1,000 lb/ft^2)	Ballast	Ballast
	Pile (2.4 m $\times$ 9 cm dia.)	16 kN (3.5 k)	6.6 kN (1.5 k)	4.4 kN (1.0 k)
	Anchor (2.4 m x 2.7 kN-m torque)	27 kN (6 k)	13 kN (3 k)	2.7 kN (0.6 k)

CONCLUSIONS

The results of the field tests indicate that a number of different foundation systems can be considered for solar development at this site. Results also indicate that the design values obtained from this field load testing program have led to more realistic design parameters compared to traditional sampling, testing, and analysis without load testing.

The project approach has been valuable in opening the door for design optimization amongst different potential foundation technologies. Load test data from combinations of axial, lateral and uplift load applications have helped develop preliminary design recommendations which are based on realistic simulations of likely conditions to be experienced in the field.

In developing sustainable solutions for geotechnically challenging sites, a field testing program development as has been done here can provide realistic data, which can make a difference in a marginal site's feasibility for development.

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