Suction Caissons : Model Tests

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Final Report on Suction Caissons: Model Tests
for the Project
Suction Caissons and Vertically Loaded Anchors

Prepared for the Minerals Management Service
Under the MMS/OTRC Cooperative Research Agreement
1435-01-99-CA-31003
Task Order 16169
1435-01-04-CA-35515
Task Order 35980
MMS Project Number 362

and

OTRC Industry Consortium

December 2005

OTRC Library Number: 12/05B161

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PREFACE

The project Suction Caissons and Vertically Loaded Anchors was conducted as series of inter-related studies. The individual studies are as follows:

- Suction Caissons & Vertically Loaded Anchors: Design Analysis Methods by Charles Aubeny and Don Murff, Principal Investigators
- Suction Caissons: Model Tests by Roy Olson, Alan Rauch and Robert Gilbert, Principal Investigators
- Suction Caissons: Seafloor Characterization for Deepwater Foundation Systems by Robert Gilbert Principal Investigator
- Suction Caissons: Finite Element Modeling by John Tassoulas Principal Investigator

This report summarizes the results of the Suction Caissons: Model Tests study.

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PERFORMANCE OF SUCTION CAISSONS USED TO ANCHOR STRUCTURES IN VERY DEEP WATER

Principal Investigators
Roy E. Olson and Robert B. Gilbert

INTRODUCTION

Suction caissons are closed-top open-bottom tubes that act as anchors in the deep ocean. They are installed by allowing them to penetrate the bottom soils under dead weight and then pumping out entrapped water to develop a sufficient pressure differential on the top plate to cause penetration. They have proved to be an efficient and economic alternative to driven piles. This project was concerned with experimental measurements of the behavior of laboratory-scale suction caissons in normally consolidated clay. The purposes were to develop a better understanding of the capacity of suction caissons, and to provide data to be used in the development of analytical models.

PERSONNEL

The original principal investigator (PI) was Roy Olson, Professor of Civil Engineering at the University of Texas (UT) at Austin. When Alan Rauch joined the UT faculty he became a co-PI. After Prof. Rauch left the faculty, Professor Robert Gilbert became the co-PI through the end of the project. Students who worked on the project, and wrote either M.S. theses or a Ph.D. dissertation, were Robert Pederson (2001), Elliott Mecham (2002), Adam Luke (2002), Rick Coffman (1003), Sandeep Vanka (2004), and Rami El-Sherbiny (2005).

BRIEF HISTORY OF SUCTION CAISSONS

The concept of using suction to install offshore anchors was first introduced by Goodman et al. (1961). The motive for the development of such anchors was the necessity for more rapid mobilization of military field equipment. The "vacuum anchors" proposed by Goodman et al. were installed the same way suction caissons are installed but were intended to have sustained vacuum by continuous water pumping during working conditions. The feasibility of installing vacuum anchors in silts and clays was verified using model tests by Goodman et al. (1961) followed by successful trials in sand by Brown and Nacci (1971). During the 1970s several investigations were conducted on the capacity of vacuum anchors in various soils, including the investigations by Wang et al. (1975), Helfrich et al. (1976), and Wang et al. (1977, 1978).

The first tests on suction caissons with no active suction after insertion were reported by Hogervost (1980). Hogervost (1980) reported successful installations in sand followed by lateral loading and installations in both sand and stiff clay followed by vertical loading.

Hogervost's conclusions were promising and stressed the feasibility of suction anchors in different soil conditions and under different loading conditions. Hogervost also concluded that the applicability of suction caissons was within reach in practical applications with increasing competitiveness in deeper waters. The first commercial application of suction caissons was reported by Senepere and Auvergne (1982) in which 12 installations of suction caissons were reported for a Catenary Anchor Leg Mooring (CALM) in the Gorm field (North Sea). Some problems were encountered during installation of the suction caissons when soil heave inside the caisson prevented reaching the final penetration depth. The problem was resolved by jetting the soil at the top of the plug and final penetration was achieved. However, the experience was not encouraging for future use of suction caissons because the time and cost were in excess of competitive methods (Tjelta, 2001).

Reappearance of suction caissons was mainly due to significant experiences with skirted gravity type platforms (Colliat et al. 1998). In addition, the installation and retrieval of a large suction caisson at the Gullfaks C site in 1985 was a key factor although it was not intended for mooring applications (Tjelta, 2001). The first tension-leg platform (TLP) anchored using suction caissons was the Snorre platform developed in the early 1990's shortly after the Gullfaks C experience. From then on, suction caissons found their way into a number of anchoring applications around the world. Currently, suction caissons in offshore applications are subjected to a wide range of loading conditions. Loads are vertical in tension leg platforms, inclined in taut mooring systems, and nearly horizontal in catenary systems.

OBJECTIVES

Reported problems with suction caissons have involved collapse of the steel top plate or buckling of the side walls, and occasional difficulty in achieving design penetrations. The latter problem may result from excessive conservatism in design and can be addressed, at least in part, using measurements of the behavior of laboratory-scale caissons under carefully controlled conditions, i.e., by the efforts of this project and others like it.

Laboratory studies can be under accelerations of 1 g or multi-g’s, the latter using a geotechnical centrifuge. The centrifuge tests have the advantage of providing data quickly and thus they are especially useful for studies for foundations actually under design. They have the disadvantage of requiring use of expensive equipment. Our studies were in a 1-g environment and were intended to provide a basic understanding of the behavior of suction caissons in normally consolidated clay and to provide data to be used by modelers in determining the applicability of their analytical methods to actual caissons.

More specifically, the main objectives of the tests were:

1. Study the penetration resistance of suction caissons inserted by deadweight and by deadweight followed by suction.
2. Examine the effect of insertion method on the pullout capacity of axially loaded suction caissons.
3. Study the effect of setup time on both the side friction and end bearing components of the capacity under axial loading.
4. Investigate the axial capacity of suction caissons loaded with a vented top cap.
5. Identify the loading point at which horizontal loading of the caisson would provide the maximum holding capacity, the optimum loading point, and examine the effect of load attachment point on the horizontal capacity.
6. Investigate the behavior and capacity of suction caissons under inclined loading with angles varying from horizontal to vertical allowing for full generation of the interaction diagram at the optimum loading point.

APPROACH

The objectives were achieved by conducting tests in large tanks of normally consolidated kaolinite on relatively long prototype suction caissons. Two caissons having an aspect ratio of nine were installed in large tanks of normally consolidated clay. The first caisson was constructed from a single tube and is subsequently referenced as the "single-walled caisson". It was used for both axial and lateral loading tests. The single-walled caisson had a padeye bar attached to its lower half allowing for lateral loading at different depths. The second caisson was composed of two concentric, thin walled, tubes forming a "double-walled" caisson capable of separating components of the capacity under axial loading.

EXPERIMENTAL FACILITY

Pedersen (2001) and Mecham (2002) set up two tanks, 4 feet by 8 feet in plan, and 6 feet deep (Fig. 1) with suitable frames to handle axial tensile loading. A literature survey did not reveal evidence of anyone else using such large tanks but they were clearly needed for our tests. To accelerate the consolidation of the clay slurry that forms the test bed soil, the tanks were equipped with a bottom drainage layer.

Mecham (2001) constructed a gantry frame from slotted lightweight steel angles. The 56-inch wide by 60-inch tall frame contained cross-bracing and stabilizing members to keep the frame from overturning. Mecham designed a mechanincal system to control the caisson through a set of panels for axial insertion and pullout of the caisson and another set for lateral loading. The caisson was controlled through a winch fixed to the side of the tank. Luke (2002) then used the equipment developed by Mecham and performed a series of axial loading tests

Figure 1. The two large tanks with the smaller access tank in between.

Coffman (2003) and El-Sherbiny (2005) built a stiffer frame to handle the larger loads from lateral loading. The frame was constructed from four-inch-wide aluminum channels forming a square structure with a side length of five feet (Fig. 2). The sections were designed to carry the expected loads during axial and lateral loading of the caisson with minimal deflections. In addition, Coffman and El-Sherbiny modified the loading system (Fig. 2) for use in lateral loading tests, expanded the data acquisition system, and developed a computer-controlled loading system. The modified data aquisistion and control system was capable of controlling caisson movement and acquiring data from load cells, displacement transducers, pressure transducers, tilt meter, and strain gages throughout the phases of the test. Coffman and El-Sherbiny used the new equipment to conduct axial and lateral loading tests on both the single–walled and double-walled caissons, and to conduct shear strength tests pn the test bed soil.

TEST BED SOIL

Pederson (2001) experimented with a series of clays and finally chose kaolinite. Kaolinite was chosen because of its high coefficient of consolidation and low compressibility, properties that allow rapid consolidation and use of high mixing water contents while producing specimens of acceptable thicknesses. The kaolinite had a mean particle size of 0.7 μm and a specific gravity of 2.58. The liquid limit of the clay ranged between 54% and 58% and the plasticity index ranged between 20 and 26.

The test beds were prepared from kaolinite consolidated from slurry under self-weight. Four tanks of kaolinite were prepared with mixing in June 2000, June 2001, March 2003, and March 2004 (Luke, 2002, Coffman, 2003, Vanka, 2004, El-Sherbiny, 2005). The test beds were prepared from slurry with target initial water content higher than the liquid limit to facilitate mixing. Consolidation times were of the order of four to nine months.

Measurements of settlement, pore pressures, and water contents during consolidation were reported by Pederson (2001), Coffman (2003), and Vanka (2004).

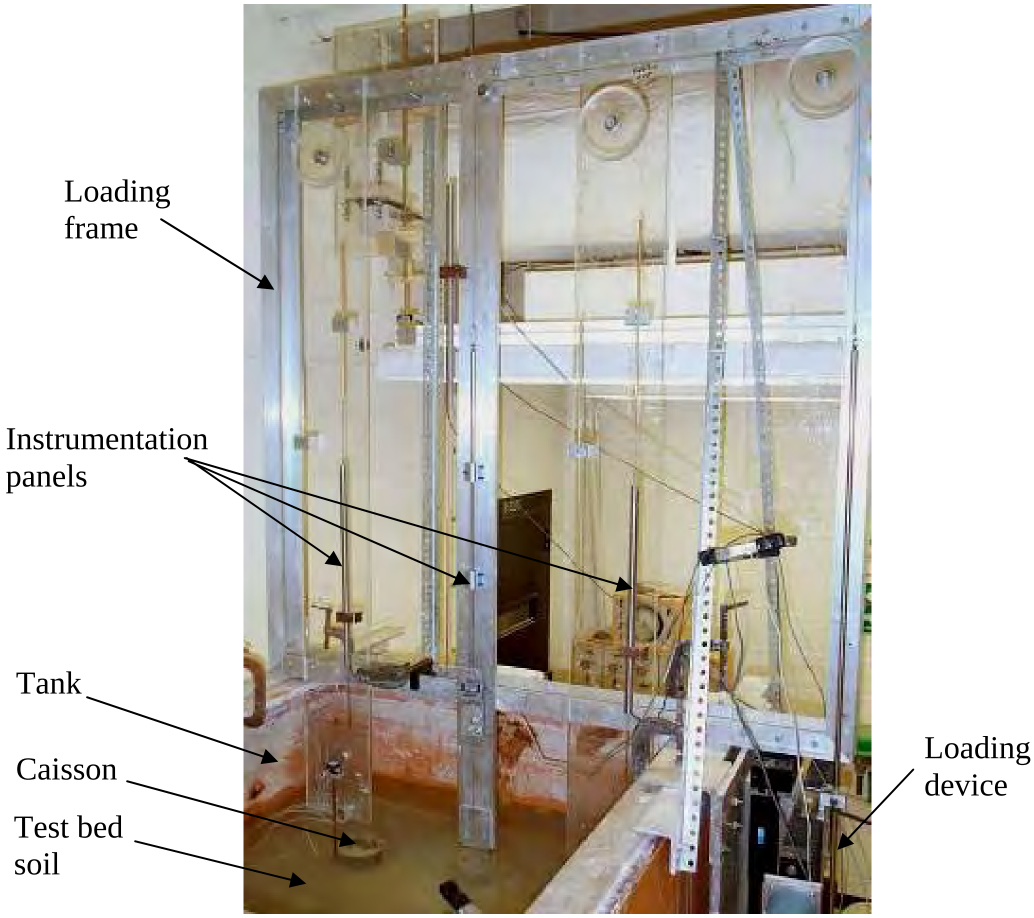

Figure 2. Illustration of the test setup during the setup phase of a lateral loading test.

Due to differences in initial water content of the slurry between the test beds, the consolidation rates and settlements of the soil deposit between test beds were different (Fig. 3). Pederson tried different apparatus in an effort to measure consolidation properties of the kaolinite in the range of effective stresses (1-300 psf) encountered in our tanks. These measurements were used in development of analytical models (Olson et al, 2003). The effect of the initial water content on the settlement was confirmed by modeling the consolidation of test beds two and four using an explicit finite difference approach implemented in the OTRC7 Fortran® code written by R. E. Olson and relationships between effective stresses, void ratio, and permeability measured by Pederson (2001) (Fig. 3). Despite the differences in the initial water-contents of the test beds, the soil had similar water contents at the end of primary consolidation

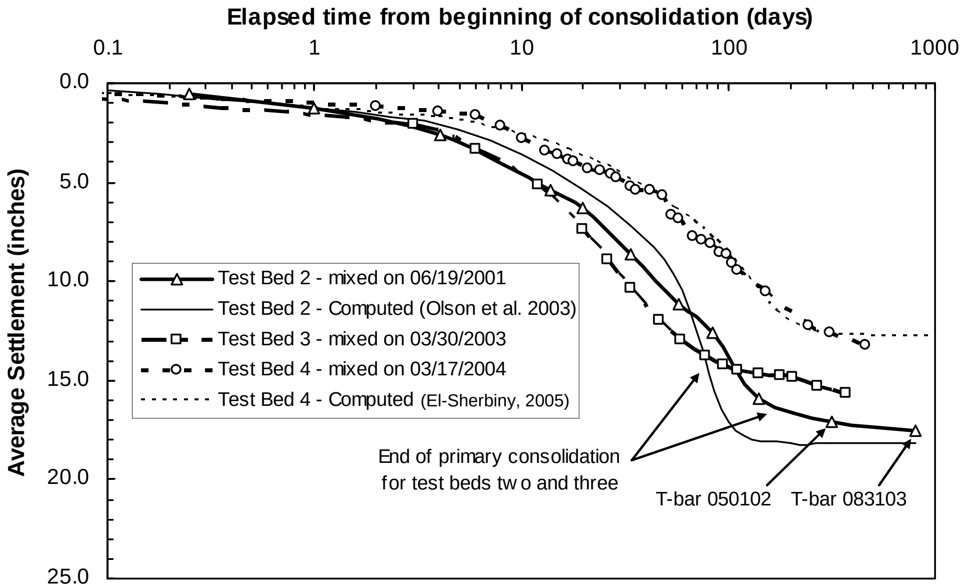

Figure 3. Measured and computed settlements of the slurry during consolidation of the test beds (El-Sherbiny, 2005).

Pedersen (2001) also developed a tilt-table shearing device and measured effective friction angles of the clay and of interfaces between the clay and aluminum (used in current tests) and acrylic (used in earlier tests). He found a distinctly curved effective failure envelope with values of ϕ' up to 55° in the lowest stress range and 28° in the stress range typically used in laboratory tests. The low-pressure shear tests were important because attempts to verify analytical models required soil properties at the appropriate stress levels. Use of values of ϕ' based on typical laboratory tests would have led to erroneous parameters elsewhere in the model and to errors in field application in a different range of stresses.

Analysis of the capacity of a suction caisson under vertical and/or lateral loads requires knowledge of the shearing strength of the surrounding clay. However, experience shows that the measured strength depends on the technique used for the measurement. Devices that have been used for field measurements include triaxial compression, field and laboratory vanes, quasi-static cone, and T-bar tests. Mecham (2001), Luke (2001), and Coffman (2003) used the Tee-bar test to measure undrained shearing strengths in the tank. Therefore, the suction-caisson test results were interpreted using the T-bar test results. However, Vanka (2004) and El-Sherbiny (2005) performed tests with a cone, field vane, tee bar, and a new ball test for comparison (Fig. 4).

Figure 4. Methods of in-situ shear strength measurement

The shear strength tests were mostly performed at locations along the centerlines of the test beds. The undisturbed strength was measured from the initial penetration of the cone, T-bar and ball. The remolded strength was measured after several penetrations /rotations at the same loaction. The results of the shear strength measurements from the different types of tests, all conducted in the same test bed within a short period of time, were reasonably close with the exception of the vane results, which yielded lower undisturbed undrained strengths (Fig. 5). However, the remolded strength profiles measured by the vane were in close agreement with the results of the penetration tests (T-bar, ball, and cone).

To avoid the effects of spatial variability and time dependency of the shear strength, each suction caisson test was assigned the shear strength profile measured in the closest location and at the closest time. Locations of the T-bar tests and slopes and intercepts of the shear strength profiles determined from the best linear fit are presented in Appendix-A.

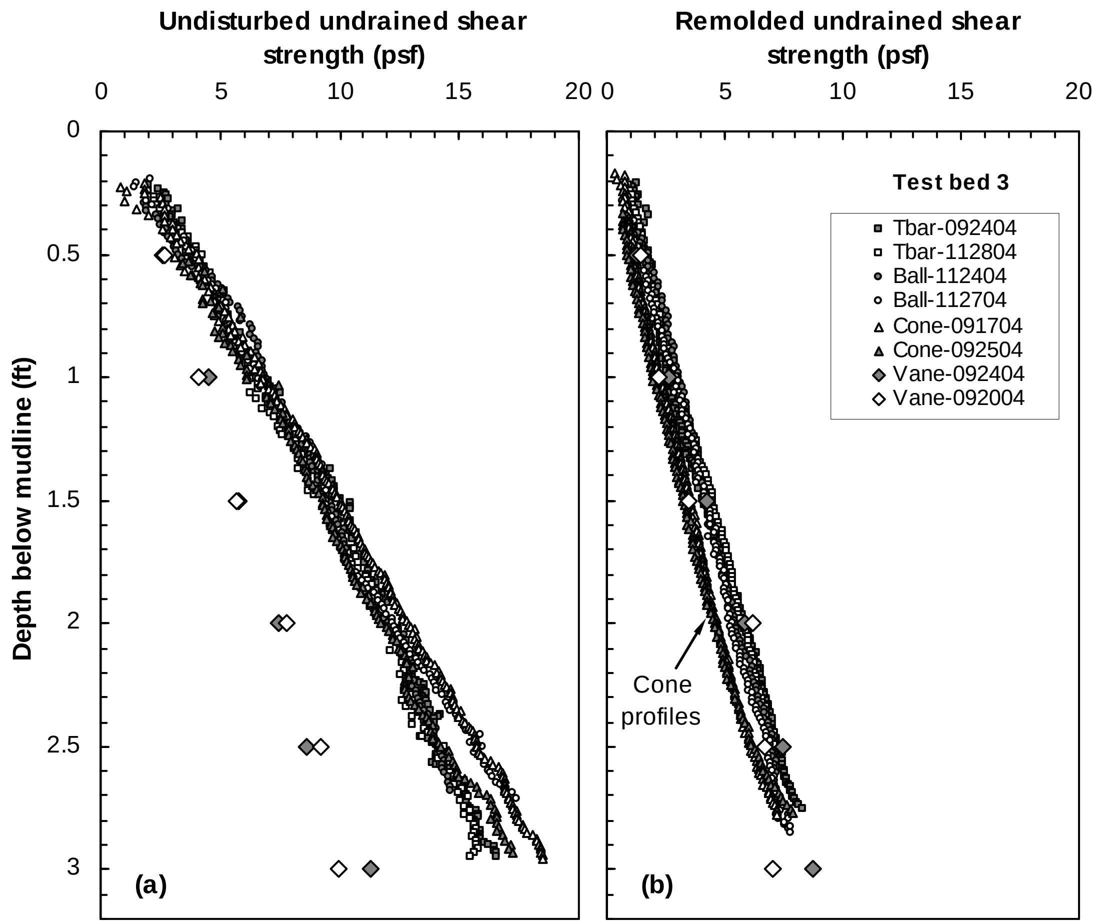

Figure 5. Comparison of the undrained shear strength profiles measured using different methods (El-Sherbiny, 2005).

PROTOTYPE CAISSONS

Suction caisson tests were conducted using two prototype caissons. The prototype caissons represent large-aspect-ratio caissons needed to provide enough capacity in soft soils. The first caisson was constructed from a single tube and was used for axial, horizontal, and inclined loading. The second caisson was constructed from two concentric tubes and was used only for axial loading. The purpose of the "double-walled" caisson was to provide separate measurements of the components of soil resistance, side friction along the walls and reversed end-bearing resistance, during insertion and pullout.

Single-Walled Caisson

Mecham (2001) designed the "single-walled" caisson used in subsequent experiments (Fig. 6). He used a thin aluminum tube so the ratio of outside diameter to wall thickness was comparable to values in field use. The caisson was constructed from a 4.00-inch diameter tube with a wall thickness of 0.032 inches and a length of 35.5 inches. Hence, the caisson had a diameter-to-thickness ratio of 125 and length-to-diameter ratio of about nine. The aluminum tube was anodized to prevent corrosion. A Delrine cap sealed the top end of the caisson. The top cap extended ¾ inch inside the tube and limited clear height inside the tube to about 34.75 inches. A penetration of 32 inches was targeted in caisson penetration tests to provide enough clearance for soil heave during insertion. Therefore, the embedment-to-diameter ratio was about eight.

Excess pore water pressures developing along the caisson were measured using sensing tips made of one-inch-by-one-inch porous bronze patches epoxied to the caisson wall. The sensing tips were connected to the pore pressure transducers using ⅛-inch PVC tubing. Excess pore pressures were measured at seven different locations.

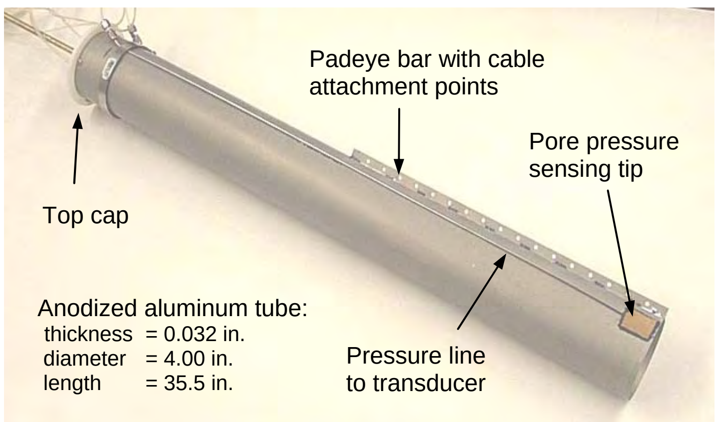

Figure 6. The single-walled caisson (Mecham, 2001).

Double-Walled Caisson

Among the many problems involved in laboratory and field tests, perhaps the one that has resisted measurement the most is the separation of capacities into components from end bearing, shear on the outside, and shear on the inside. The problem is particularly severe for rapid loading with the top sealed, the usual case offshore, because the tip capacity is important and seems generally uncertain. El-Sherbiny (2005) designed a "double-walled" caisson composed of two, concentric, very thin aluminum tubes (Fig. 7). The total load in each tube was measured to allow separation of load transferred on the outside, the inside, and at the tip. The group of tests with the double-walled caisson provided a unique measurement of the components of soil resistance acting on the caisson during insertion, setup, and pullout phases of the tests.

El-Sherbiny (2005) used two custom-made tubes having a wall thickness of 0.02 inch and outer diameters of 4.00 inches and 3.84 inches and a general tolerance of 0.001 inch (Fig. 7). As a result, the overall wall thickness of the caisson was 0.1 inch and the diameter-to-thickness ratio was 40. The caisson outer tube had a length of 36.75 inches, while inner tube had a length of 35.5 inches, leaving 1.25 inches height difference to accommodate the load cell and connections (Fig. 7). A penetration of 32 inches was targeted in caisson penetration tests; therefore, the embedment-to-diameter ratio was about eight.

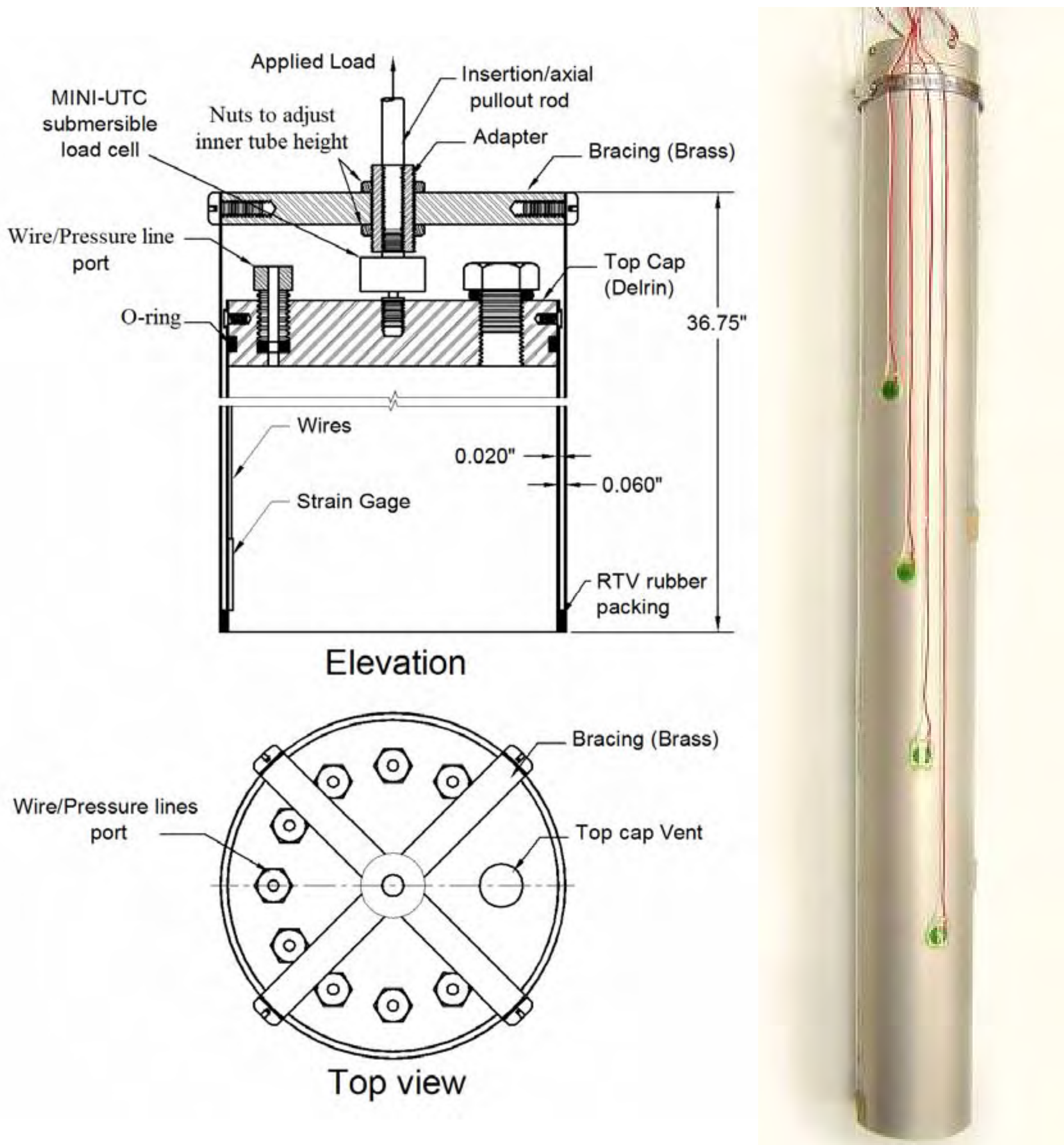

Figure 7. Conceptual diagram of the double-walled caisson.

The tubes were connected at the top end (Fig. 7) and instrumented with load cells, strain gages, and pore pressure sensing tips. The inner tube was sealed with a top cap equipped with a load cell. The outer tube had an open bracing screwed to its top edge. An adapter connected the load cell of the inner tube to the bracing of the outer tube to form a rigid connection. Hence, the two tubes moved together during insertion and pullout with minimum relative motion. The two tubes were separated at the bottom with a silicon packing (Fig. 7). Soft silicon packing was chosen to minimize load transfer between the tubes while maintaining a tight seal. Load transfer between the two tubes was measured to be about 4% (El-Sherbiny, 2005). Loads acting on the inner tube were measured using the load cell attached to the top cap. The total loads acting on the caisson were measured using another load cell connected at the top of the insertion rod. The difference between the two load cell measurements provided the loads acting on the outer tube.

TEST RESULTS

About 43 suction caisson model-tests have been completed to investigate axial, lateral, and inclined loading and the effects of installation methods. Luke (2002) performed a series of axial loading tests with the single-walled caisson in normally consolidated clay. Luke's tests demonstrated that consolidation after caisson installation required about two days as opposed to values around one hour used previously, and provided better data to the researchers developing finite element methods. The simplified equation usually used for estimation of side shear for both piles and caissons is:

$$f_s = \alpha\, c_u \qquad (1)$$

where f_s is the side shearing stress between the soil and the caisson at failure, c_u is the undrained shearing strength of the soil (dependent on the method of measurement), and α is an empirical factor. Based on decades of experience with driven piles in normally consolidated clay, we expected to find values of α in the range of perhaps 0.8 to 1.2.

Further:

$$q_p = c_u\, N_c \qquad (2)$$

where q_p is the tip capacity (stress), N_c is the so-called "tip bearing capacity factor" (dimensionless), and c_u is the undrained shearing strength, which is again dependent on the technique used to make the measurement. Luke (2002) reported the results from 17 tests with the caisson inserted using suction, and with the caisson inserted using deadweight only, at setup times of one hour and 48 hours, and pulled out with vented and sealed top caps (Fig. 8, and Table 1). When the caisson was withdrawn with the top open, the caisson pulled out leaving the plug behind so failure was on both the inside and outside surfaces and there was negligible tip capacity. In that case, and assuming there was the same side shear on the inside and outside, α=0.55-0.67 (Table 1). When the caisson was withdrawn with a sealed top cap, the soil plug was pulled out with the caisson. The results had some uncertainty due to our inability to determine the weight of soil adhering to the sides of the caisson during withdrawal and to the weight of the plug (we could not measure the location of the top of the plug accurately). When the same values of α were used for failure on the outer surface, then N_c =13-21, values higher than the value of 9 usually used. It was also possible to assign the more usual values of α=1 and N_c=9 recommended in API RP2A for piles and match calculated and measured capacities (Table 1).

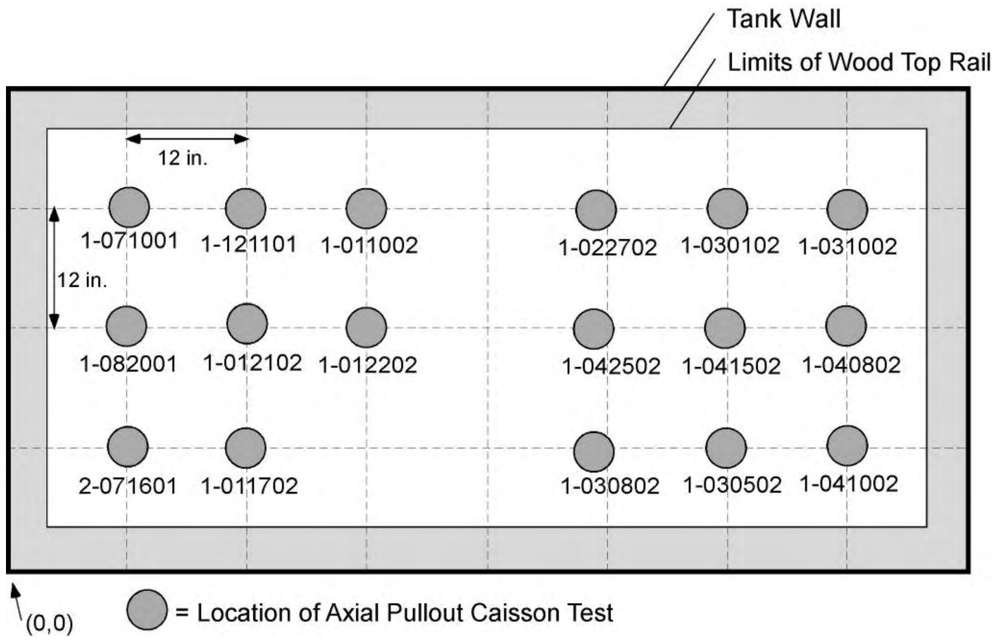

Figure 8. Test Locations in Test Bed 1 (Luke, 2002).

Table 1. Caisson Tests in Test bed 1 (Luke, 2002)

Test	Final Installation Method	Setup time (hr)	Pullout Conditions		Average Insertion α (Undisturbed Soil Profile)	Pullout α_{max} (assuming N_c = 9.3)
			Top Cap	Rate		
1-071001	Deadweight	1	Vented	Rapid	0.19	0.22
2-071601	Deadweight	1	Vented	Rapid	0.16	0.25
1-011002	Suction	1	Vented	Rapid	0.22	0.37
1-011702	Suction	1	Vented	Rapid	0.24	0.33
1-082001	Deadweight	1	Closed	Rapid	0.17	0.86
1-121101	Deadweight	1	Closed	Rapid	0.22	0.69
1-012102	Suction	1	Closed	Rapid	0.21	0.80
1-012202	Suction	1	Closed	Rapid	0.27	0.92
1-022702	Deadweight	48	Vented	Rapid	0.27	-
1-040802*	Deadweight	48	Vented	Rapid	0.26	0.67
1-030502	Suction	48	Vented	Rapid	0.29	0.55
1-041002*	Suction	48	Vented	Rapid	0.27	0.57
1-030102	Deadweight	48	Closed	Rapid	0.29	0.97
1-030802	Suction	48	Closed	Rapid	0.28	1.04
1-031002	Suction	48	Closed	Rapid	0.26	1.01
1-041502*	Suction	48	Closed	Rapid	0.36	1.17
1-042502*	Suction	48	Vented	Slow	0.33	NA

* Indicates tests where pore pressures were measured.

Legend: SW=self-weight, SW+S=self-weight followed by suction

Luke (2002) reported upper and lower bounds enclosing all possible combinations of α and N_c that would result in the measured capacity given the uncertainty in the weight of the plug projected a range of likely solutions (Fig. 9). Luke reported that the results were not conclusive on the effect of deadweight on the capacity due to lack of a sufficient number of tests.

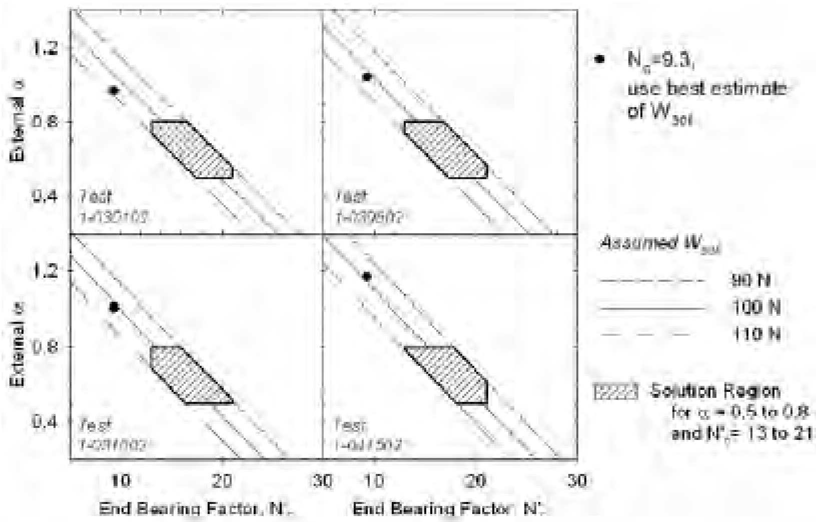

Figure 9. Possible combinations of α and N_c to match measured capacities in four tests

The oil industry is now also using floating structures with catenary and taut mooring lines. Accordingly, we conducted tests with purely horizontal loading and then loading at an angle. The first set of tests aimed at determining the optimum loading point that maximizes the holding capacity of the caisson under pure horizontal loading and the mechanisms associated with the failure (Coffman, 2003; El-Sherbiny, 2005). Ten tests were conducted on the single-walled caisson in test bed two and consisted of 12 tests (Fig. 10, Table 2) loaded horizontally at depths from 15 to 31 inches below the mudline, i.e. the bottom half of the caisson embedment. This range of depths for the load application includes extreme values not normally used in practice, yet helps in identifying the different modes of failure and validating methods of estimating capacity. The caisson was installed half way by deadweight followed by suction. Excess pore pressures developed during insertion were allowed to dissipate for 48 hours.

Horizontal loading resulted in definition of a peak capacity and then a significant loss in capacity with further movement. The capacity under horizontal loading was normalized by the integral of the shear strength over the projected side area of the caisson. The normalized capacity can be considered as an average unit lateral resistance factor over the embedment depth. The normalization minimizes the effect of small differences in shear strength at different test locations on the measured capacity. For the case of rapid loading, in which the soil was essentially undrained, the peak horizontal capacity occurred for loading near the bottom third point (Fig. 11).

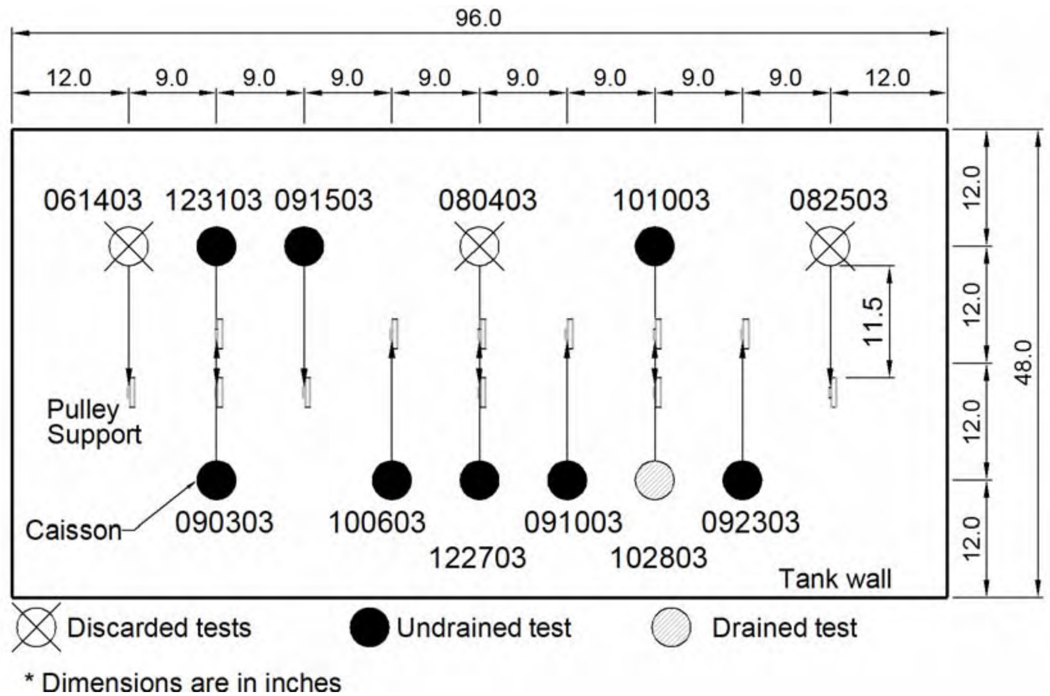

Figure 10. Horizontal loading tests conducted in test bed two.

Table 2. Summary of the horizontal loading tests

Test ID	Depth to attachment point (in.)	Penetration depth (in.)	Insertion method	Setup time (hrs)	Top cap condition during loading	Max. horiz. load (lb)	Padeye displ. at max. Load (in.)	Failure mode**
090303	15.0	31.7	Suction	48	Sealed	42.1	0.11	toward
092303	20.0	32.4	Suction	48	Sealed	68.2	0.12	toward
091503	21.0	32.0	Suction	48	Sealed	74.1	0.23	toward
100603	22.0	31.9	Suction	48	Sealed	73.9	0.13	away
082503*	24.0	31.8	Suction	48	Sealed	66.7	0.12	away
101003	24.0	32.3	Suction	48	Sealed	73.7	0.12	away
123103	24.0	32.1	Suction	48	Sealed	74.4	0.18	away
122703	26.0	32.0	Suction	48	Sealed	63.5	0.16	away
091003	31.0	32.1	Suction	48	Sealed	48.4	0.13	away
102803**	24.0	32.2	Suction	48	Sealed	60.0	0.90	Toward

* Located near end wall of tank, results are not representative
** Rotation of top cap toward or away from the applied load

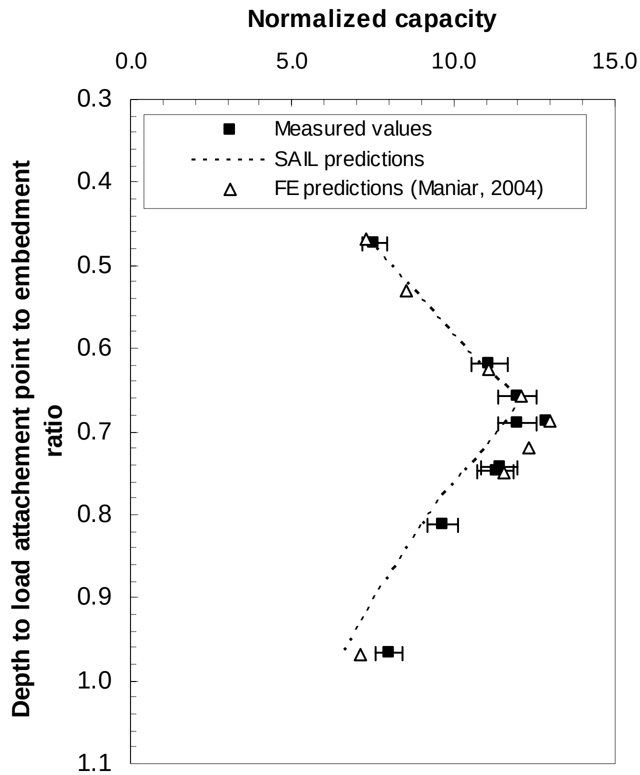

Figure 11. Comparison of measured horizontal capacity with theoretical solutions.

The range of uncertainty in the measured capacities is marked with a bar on Fig. 11. At the time, we were engaged in the experimental phase of this work, companion projects at UT-Austin dealt with finite element analyses and at TAMU dealt with a limiting equilibrium solution. Curves of capacity versus depth of loading from both the limiting equilibrium ("SAIL") and finite element analyses are included in Fig. 7. The agreement is considered remarkable because:

1. The limiting equilibrium analysis requires input of undrained shearing strengths but the measured undrained strengths vary significantly depending on the state of stress used in the test.

2. The finite element analysis requires input of rather sophisticated soil properties, which we have not been able to measure because the clay is so soft that samples slump under their own weight.

El-Sherbiny (2005) attempted to study the lateral capacity of the caisson with the soil drained, by applying fixed lateral loads in small steps and waiting for dissipation of the measured pore water pressures prior to application of the next load. Unfortunately, the first test, with loading at the lower third point, tied up the equipment for 3.5 months and thus further tests could not be performed. The results indicated that the capacity with the soil drained was about 20% less than the capacity with the soil undrained.

With interest developing in taut-line loading, El-Sherbiny (2005) ran a series of seven tests with loading at the lower third point with loading at an angle from the horizontal (Fig. 12, and Table 3), and the soil undrained. The tests covered the entire range of angles from horizontal to vertical, which provides a complete set of data for verification of analytical and numerical models. The experimental results agree with analyses performed using the limiting equilibrium program ("SAIL") developed at TAMU (Fig. 13). The numbers next to the lines on Fig. 13 are angles of loading, measured from the horizontal.

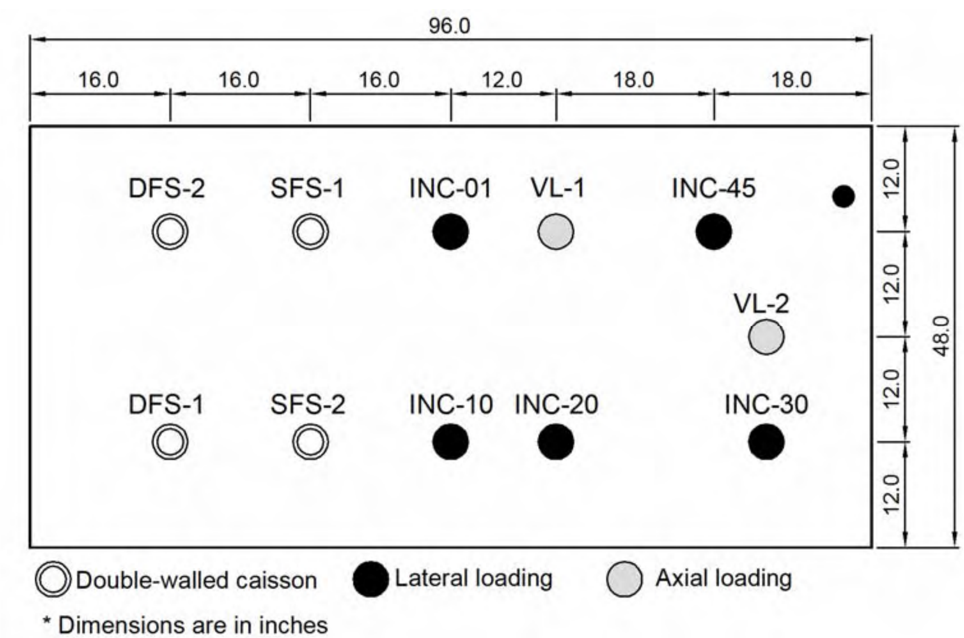

Figure 12. Tests conducted in test bed three.

Table 3. Summary of the inclined loading tests

Test ID	Loading angle from horizontal (deg.)	Penetration depth (in.)	Capacity (lb)	Padeye disp. at failure (in.)	Capacity components (lb)		Failure mode **	Date
					Horizontal	Vertical		
VL-1 *	90	32.2	37.6	0.25	0.0	37.6	none	06-22-04
VL-2	90	32.2	38.5	0.20	0.0	38.5	none	07-30-04
INC-01	1.2	32.0	90.9	0.15	90.9	1.9	towards	09-28-04
INC-10	8.9	32.0	90.1	0.20	89.2	12.5	towards	10-22-04
INC-20	19.2	32.0	87.0	0.15	82.2	28.6	away	10-04-04
INC-30	30.6	32.0	78.1	0.30	67.3	39.6	away	10-11-04
INC-45	43.5	32.0	57.7	0.15	41.8	39.8	away	10-17-04

* The only test with load applied at the top cap; otherwise, the caisson was loaded 24 in. below mudline
** Rotation of top cap towards or away from the applied load

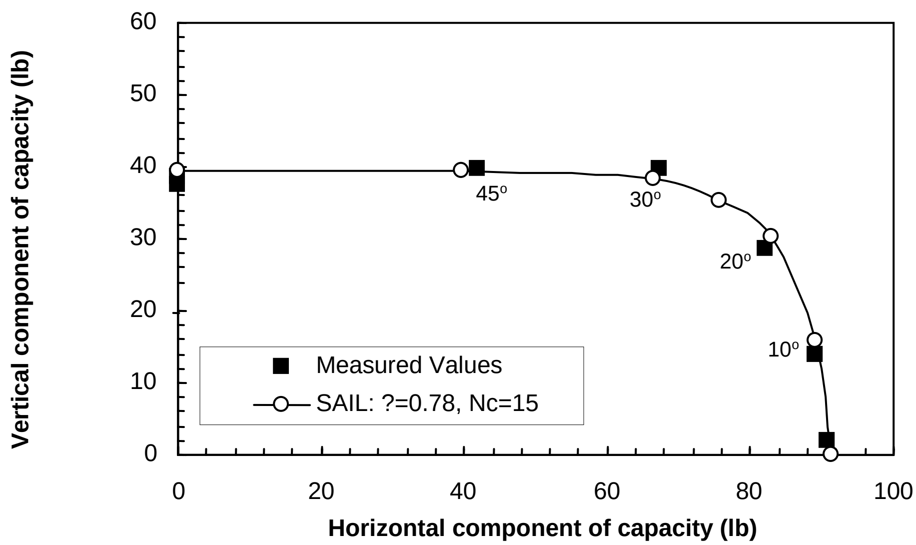

Figure 13. Comparison between measured and predicted components of the suction caisson capacity under inclined loading.

El-Sherbiny (2005) conducted rapid axial loading tests on the double-walled caisson following insertion by deadweight and insertion by suction (Figs. 12 and 14, and Table 4). Tests were conducted with sealed and vented top caps and at varying setup times. The tests were conducted to resolve the components of the axial capacity and reduce some of the uncertainty in previous tests.

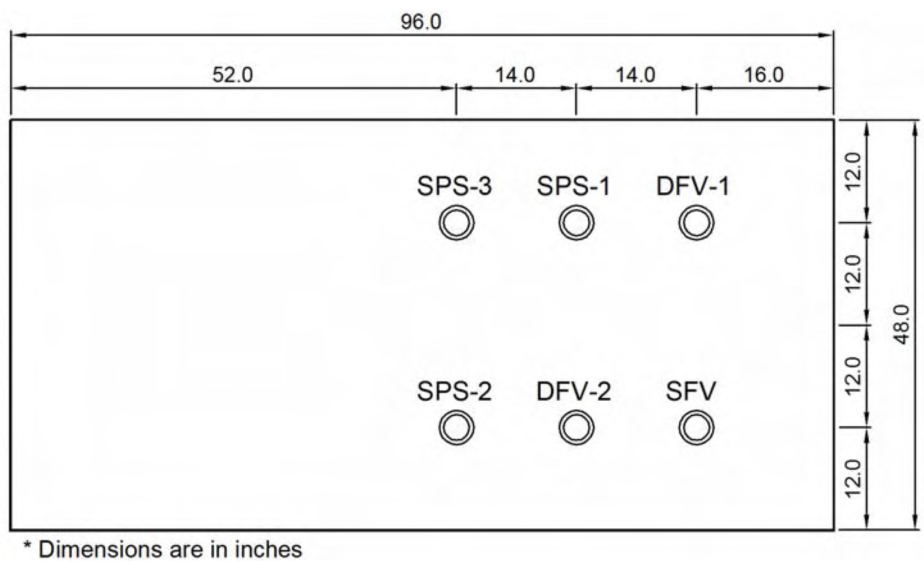

Figure 14. Tests conducted in test bed four.

Table 4. Summary of the axial loading tests.

Test ID *	Test bed	Date	Insertion method	Final pen. depth (in.)	Maximum penetration resistance (lb)			Setup time (hrs)	Top cap cond-ition during loading	Capacity (lb)	Soil resistance to pullout (lb)	Disp. to failure (in.)	Represent-ative T-bar Test
					Inner tube	Outer tube	Total						
SFS-1	3	04-29-05	Suction	31.9	6.5	10.6	17.2	96	Sealed	44.7	42.7	0.64	T-bar 042705
SFS-2	3	05-12-05	Suction	31.8	6.1	9.8	15.9	96	Sealed	43.0	41	0.31	T-bar 042705
DFS-1**	3	05-31-05	Deadweight	31.9	7.3	8.1	15.4	96	Sealed	40.7	38.7	0.37	T-bar 061305
DFS-2	3	06-17-05	Deadweight	31.3	6.6	7.8	14.4	96	Sealed	38.8	36.7	0.41	T-bar 061305
SFV	4	06-25-05	Suction	31.7	3.2	5.2	8.3	96	Vented	21.8	19.8	0.11	T-bar 062305
DFV-1**	4	07-01-05	Deadweight	31.6	3.6	5.6	9.2	96	Vented	20.0	18	0.39	T-bar 062305
DFV-2	4	07-10-05	Deadweight	31.4	4.0	6.0	10.0	96	Vented	24.1	22.2	0.03	T-bar 070905
SPS-1	4	07-17-05	Suction	31.7	3.5	5.6	9.2	1	Sealed	24.2	21.9	0.11	T-bar 070905
SPS-2	4	07-18-05	Suction	31.7	3.3	5.5	8.8	48	Sealed	28.1	25.8	0.07	T-bar 070905

*The first letter of the test ID (S or D) refers to Deadweight versus Suction insertion, the second letter (F or P) refers to Full or Partial setup, and the third letter (S or V) refers to sealed versus Vented top cap during pullout. Repeated test have an extra identifier at the end.
** no inner load cell measurements during pullout

For the insertion stage, El-Sherbiny back-calculated the external α ($\alpha_{external}$), the internal α ($\alpha_{internal}$), and the average of the external and internal α ($\alpha_{average}$) (Table 5, and Fig. 15). Analyses indicated that the side friction acting on the inner wall of the caisson was lower than on the outer wall during both deadweight and suction insertions. Because there was less side friction on the inner wall than on the outside, soil tended to displace into the inside during insertion, resulting in heave of the soil plug during both dead-weight and suction installation. The internal alpha factor ($\alpha_{internal}$) during insertion was reduced from 0.28 in case of deadweight insertion to about 0.21 in case of suction insertion. However, the average alpha ($\alpha_{average}$) only dropped from 0.31 in case of deadweight insertion to 0.28 in case of suction insertion. The external alpha factor ($\alpha_{external}$) was constant at an average value of 0.34.

Table 5. Summary of alpha factors back calculated from double-walled caisson insertions

Test ID	**Test bed**	**Insertion method**	**Final penetration depth (in.)**	**Back-calculated α factor at maximum penetration resistance**		
				$\alpha_{average}$	$\alpha_{internal}$	$\alpha_{external}$
Double-walled caisson tests inserted by						
SFS-1	3	Suction	31.9	0.31	0.24	0.38
SFS-2	3	Suction	31.8	0.28	0.22	0.35
SFV	4	Suction	31.7	0.26	0.20	0.32
SPS-1	4	Suction	31.7	0.27	0.21	0.33
SPS-2	4	Suction	31.7	0.26	0.20	0.33
Average α during insertion by suction				0.28	0.21	0.34
Coefficient of variation				0.07	0.07	0.07
Double-walled caisson tests inserted by deadweight						
DFS-1*	3	Deadweight	31.9	0.32	0.31	0.32
DFS-2	3	Deadweight	31.3	0.31	0.29	0.33
DFV-1*	4	Deadweight	31.6	0.31	0.26	0.35
DFV-2	4	Deadweight	31.4	0.31	0.26	0.35
Average α during insertion by deadweight				0.31	0.28	0.34
Coefficient of variation				0.02	0.10	0.04

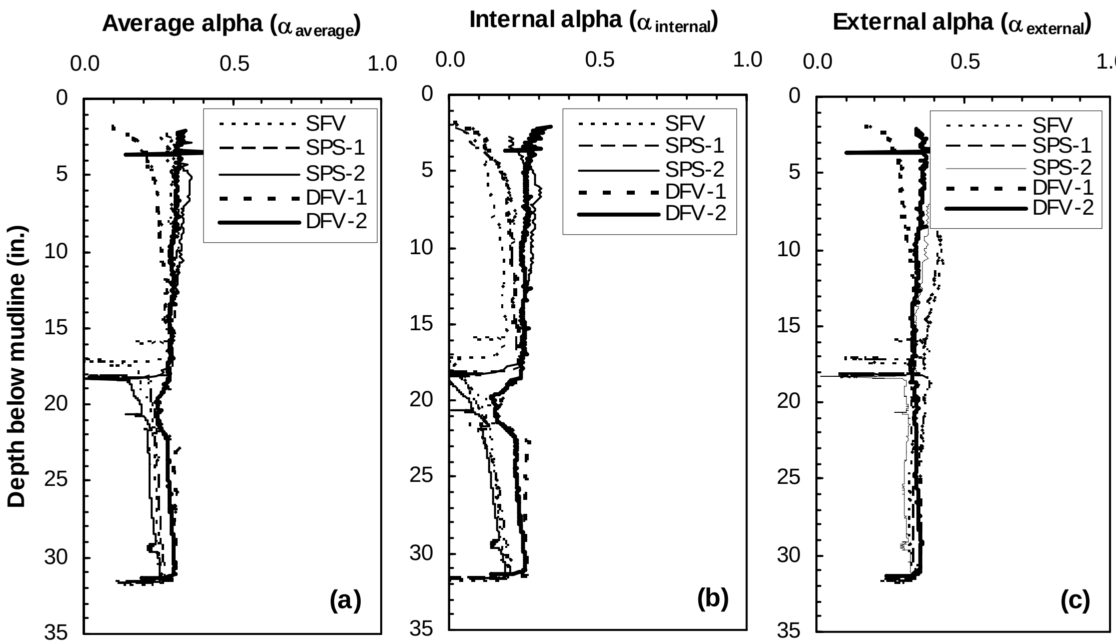

Figure 15. Alpha factors back calculated form insertion of the double-walled caisson in test bed 4: (a) average alpha, (b) internal alpha, and (c) external alpha (Note: test ID starting with S indicates suction insertion, while test ID starting with D indicates deadweight insertion)..

El-Sherbiny (2005) investigated the effect of setup time on the axial capacity of the suction caisson using the double-walled caisson. Tests were conducted with setup times ranging between one hour and 96 hours (Table 4). The pullout capacity of the double-walled caisson loaded with a sealed top cap was found to be almost constant after a 48-hour setup period (Fig. 16). It was observed that the end bearing resistance was not affected by the setup time whereas the side friction resistance was observed to increase with time before reaching a constant value after 48 hours of setup (Fig. 16). The pore pressures dissipated within 48 hours around the caisson's exterior whereas it took 96 hours to dissipate the pore pressures along the internal wall of the caisson. It is believed that the dissipation of the pore pressures around the caisson's exterior indicates soil reconsolidation along the external wall after which no significant increase in strength is expected and the maximum side friction resistance is achieved. On the other hand, the measured pore pressures at the tip of the caisson following insertion seem to be localized around the caisson wall where soil shearing occurred. The bottom of the plug is probably not affected, resulting in the development of full end bearing resistance right after insertion (Fig. 10). The soil plug came out with the caisson when the caisson was pulled out with the top cap sealed. In case of pullout with a vented top cap, the soil plug was left behind during loading. In all tests, the caisson walls were covered with clay following pullout indicating that the failure surface occurred within the clay, not at the interface.

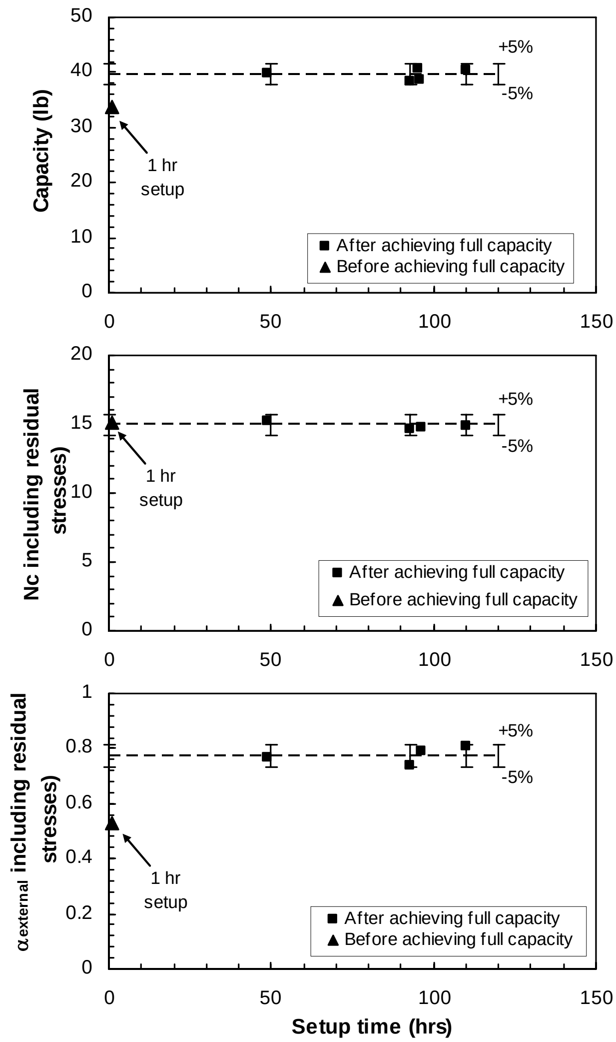

Figure 16. Effect of the setup time on the (a) capacity, (b) end-bearing resistance, and (c) the external side friction of the double-walled caisson during pullout with a sealed top

The results of caisson pullouts after 96 hours of setup were used to investigate the caisson behavior. El-Sherbiny found that the pullout capacity of the caisson under axial loading was the same for caissons inserted by deadweight only and caissons inserted by deadweight followed by suction for the tests with sealed and vented top caps. The mobilized limit equilibrium parameters α and N_c were back calculated from the soil resistance at the maximum total load (Table 6). The average mobilized external alpha factor ($\alpha_{external}$) was about 0.80 while the end-bearing factor (N_c) was about 15 for the sealed top cap case. In case of a vented top cap, the average mobilized external alpha factor ($\alpha_{external}$) was about 0.85 while the average mobilized internal alpha factor ($\alpha_{internal}$) was about 0.5. It was observed that $\alpha_{external}$ was higher in the case of a vented top cap than in the case of a sealed top cap. This is probably due to the effect of the development of end bearing failure at the tip, and associated soil movement, affecting the side shear mobilized close to the tip.

Table 6. Mobilized limit equilibrium parameters calculated at failure during axial pullout of the double-walled caisson

Test ID	Test Date	Test bed	$\alpha_{external}$	$\alpha_{internal}$	N_c
Sealed top cap					
SFS-1	4/29/2005	3	0.81	0.3	14.9
SFS-2	5/12/2005	3	0.74	0.28	14.7
DFS-1	5/31/2005	3	-	-	-
DFS-2	6/17/2005	3	0.79	0.31	15.6
Average			0.78	0.30	15.0
Coefficient of variation			0.05	0.05	0.03
Vented top cap					
SFV	6/25/2005	4	0.82	0.46	7.7
DFV-1	7/1/2005	4	-	-	-
DFV-2	7/10/2005	4	0.86	0.49	7.7
Average			0.84	0.48	7.7
Coefficient of variation			0.03	0.04	0.0
Partial setup - sealed top cap					
SPS-1	7/17/2005	4	0.53	0.06	15.1
SPS-2	7/10/2005	4	0.77	0.16	15.2

El-Sherbiny (2005) compared the results of the tests with the double-walled caisson with results reported by Luke (2002) for axial loading using the single-walled caisson. Luke (2002) reported upper and lower bounds enclosing all possible combinations of $\alpha_{external}$ and N_c that would result in the measured total capacity (Fig. 16). In addition, Luke (2002) projected a range of likely solutions based on $\alpha_{external}$ measured from vented top cap tests.

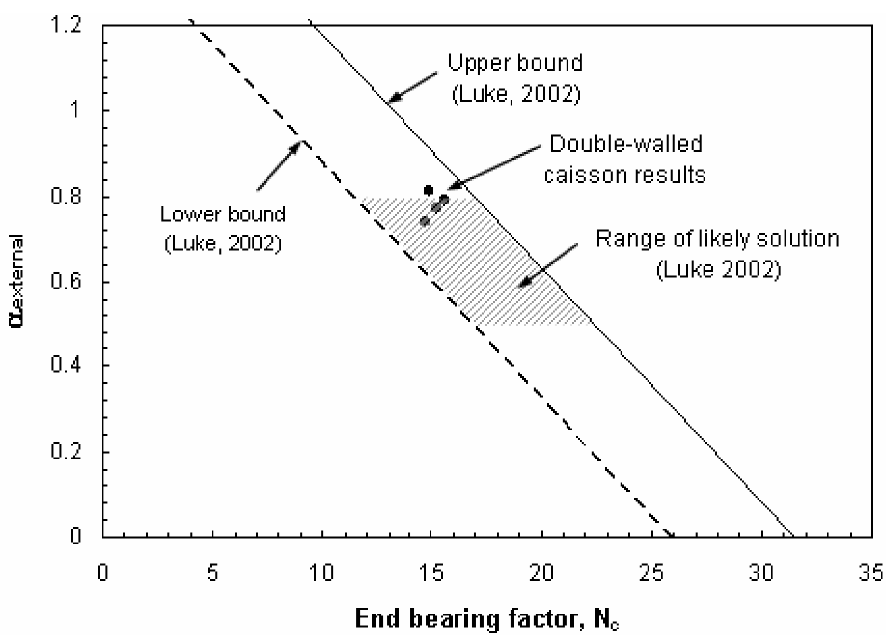

Figure 17. Comparison between limit equilibrium parameters measured from the double-walled caisson versus range of expected values predicated by Luke (2002).

The results from the double-walled caisson tests were within the upper and lower bounds reported by Luke (2002), indicating that the two caissons would yield about the same capacity if tested in the same soil. The results of the double walled caisson were in line with the projected range of likely solutions but suggesting an alpha factor closer to the upper limit of the range. In the case of a vented top cap during loading, the $\alpha_{average}$ reported by Luke (2002) was 0.60, which is slightly lower than the $\alpha_{average}$ of 0.65 back calculated from the double-walled caisson results.

CLOSING COMMENTS

At the beginning of our research, there were persons who considered suction caissons a possible foundation type (particularly in Norway) but there were others who thought suction caissons would never be used. There was little understanding about how suction caissons behave and the assumptions seemed to be that the API recommendations for piles would apply. Data collected on this project gave us a much better understanding of the mechanisms controlling caisson resistance to penetration in clay. The tests in clay have provided data on the effects of axial tensile loading with open tops (simulates long term loading) and sealed top (short-term loading), and of changes in capacity with increasing consolidation times. The tests with lateral loading have provided important information on the effect of the depth of loading and the angle of loading.

The research has supported the long-term understanding that our current design procedures, which use the undrained shearing strength of clays, have a serious problem because the measured strength depends on the state of stress and direction of shear used in the tests. For suction caissons in tension, the tip capacity is an important component of total capacity, unlike the case for driven piles. The back-calculated tip bearing capacity factor reported is based on in-situ measurements of shear strength using penetration tests.

Finally, designers need analytical tools. Efforts to develop a limiting equilibrium solution for lateral and inclined loading, at Texas A & M have been remarkably successful in predicting the behavior that we measured. Efforts to develop finite element code, at the University of Texas, have also been partially successful but suffer from the fact that we have been unable to measure appropriate model parameters because our soil is so soft.

We appreciate the contributions of our sponsors, the commitment of our graduate students to their work, and the collaborative efforts between the experimental and theoretical researchers, and between the two universities involved in this effort.

LIST OF THESES AND DISSERTATIONS

Theses

Pedersen, Robert C. (2001), "Model Offshore Soil Deposit: Design, Preparation, and Characterization", Univ. of Texas, Austin (May).

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Dissertations

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Appendix A: Summary of Shear Strength Tests

Table A.1. Summary of best linear fits to the shear strength profiles

		Undisturbed strength		Remolded strength	
	Test ID	**Slope (psf/ft)**	**Intercept (psf)**	**Slope (psf/ft)**	**Intercept (psf)**
Test bed 1	Average	3.26	1.79	2.09	0.53
Test bed 2	Tbar-052202 (Luke, 2002)	3.24	0.67	NA	NA
	Tbar-060603	4.16	0.38	2.32	0.00
	Tbar-073003	4.65	0.97	2.42	0.00
	Tbar-082203	4.26	0.47	2.18	0.05
	Tbar-083103	4.76	0.34	2.49	0.00
	Tbar-090603	4.81	0.77	2.57	0.00
	Tbar-091203	4.86	0.94	2.59	0.00
	Tbar-091903	4.89	0.62	2.81	0.00
	Tbar-100303	5.25	0.76	3.03	0.00
	Tbar-100603	5.17	0.60	3.07	0.05
Test bed 3	Cone-091704	6.19	0.50	2.40	0.00
	Cone-092504	5.69	0.54	2.38	0.00
	Tbar-092404	5.19	1.52	2.88	0.01
	Tbar-112804	5.11	1.39	2.78	0.25
	Tbar-042705	5.72	1.94	2.57	0.76
	Tbar-061505	5.56	0.93	2.38	0.00
	Ball-112404	5.26	1.37	2.58	0.39
	Ball-112704	5.91	0.71	2.69	0.09
	Vane-092004	3.05	1.22	2.47	0.06
	Vane-092404	3.76	0.28	2.84	0.00
	Vane-113004	3.56	0.00	2.27	0.00
Test bed 4	Tbar-062305	3.13	1.09	1.39	0.61
	Tbar-070905	3.39	1.22	1.66	0.41
	Tbar-071805	3.25	1.53	1.62	0.34

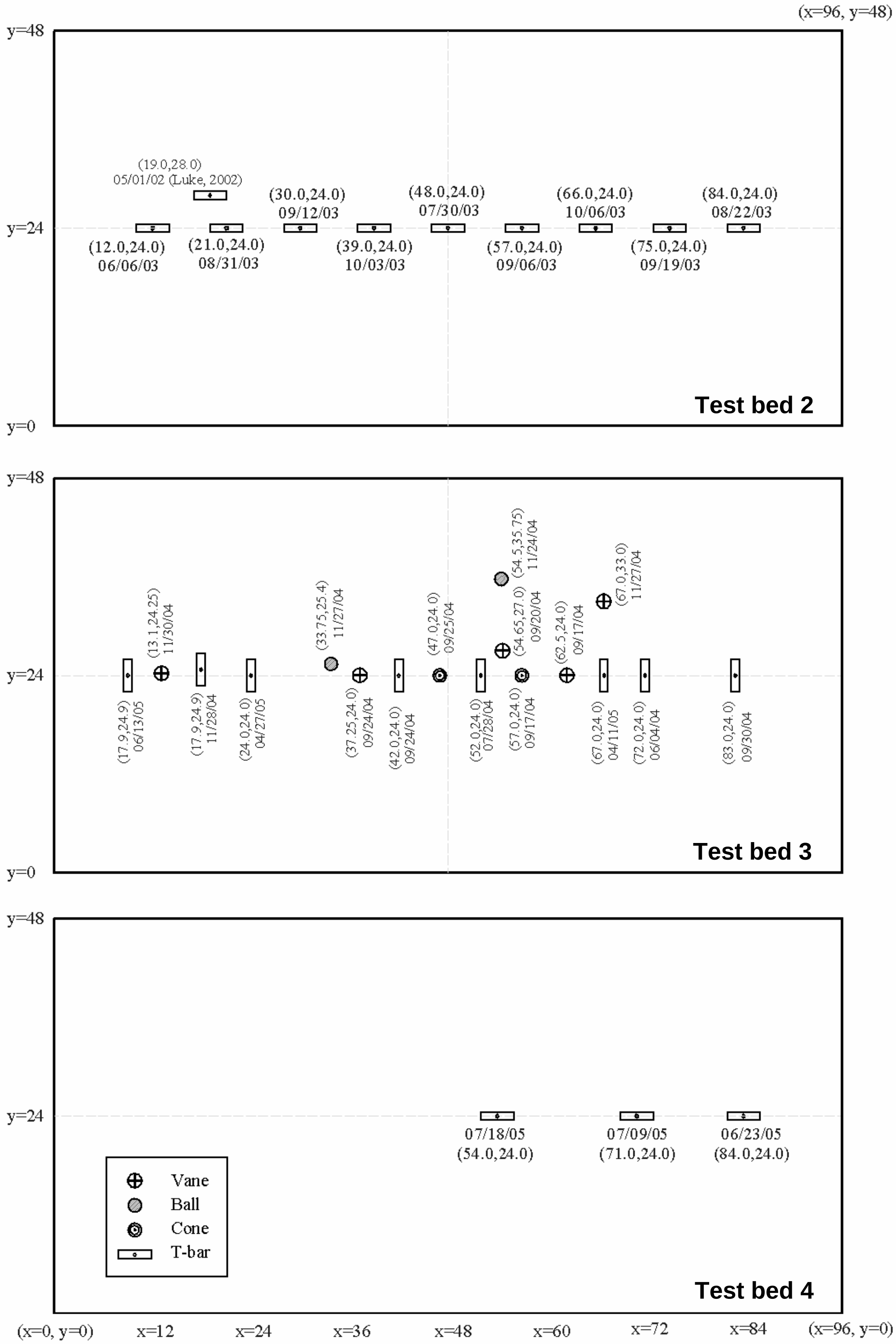

Figure A.1. Locations of the shear strength tests within the test beds.