Comprehensive Status Report: November 18, 2004

OTRC Project Title: Suction Caissons & Vertically Loaded Anchors
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Project Subtitle: Suction Caissons: Model Tests
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MMS COTR: A. Konczvald

This report provides a comprehensive summary the research completed in all prior Phases of this project (September 1999 – August 2004), and describes research being done in the present Phase (September 2004 – August 2005) to complete this project.

Note that this report addresses one of four related research areas on this project. The other three areas are reported separately under the subtitles – Suction Caissons: Seafloor Characterization for Deepwater Foundations, Suction Caissons: Finite Element Modeling, and Suction Caissons & Vertically Loaded Anchors: Design Analysis Methods.

Suction Caissons: Model Tests

Roy E. Olson and Robert B. Gilbert

BRIEF HISTORY OF OFFSHORE STRUCTURES

The intent of this introductory section is to provide a brief history of evolution that led to use of suction caissons. It is not intended to deal with offshore structures in general, nor with structures not part of the evolution of suction caissons. Further, views here are those of the authors and may not represent the convoluted manner in which developments typically occur.

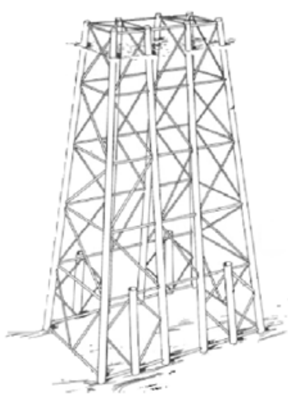

Fig. 1 Jacket Structure (McClelland, 1972)

The early offshore oil production structures were steel frames (Fig. 1), called jackets, which were fixed to the seafloor using open-end steel pipe piles that were driven through the jacket legs and then welded to the jacket.

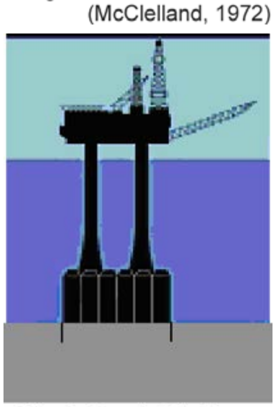

Fig. 2 Gravity Platform

When oil was discovered in the North Sea, the subsoil was generally stiff enough that shallow foundations could be used. This allowed the design to change to *gravity platforms* (Fig. 2) which were concrete structures that were placed directly on the bottom using a mat (also called raft) foundation, a reinforced concrete footing under the entire structure. This change avoided the use piled foundations, whose installation was both time consuming and expensive due to the rough weather condition in the North Sea. To minimize potential problems with scour, the sides of the foundations were provided with vertical walls, called *skirts*, which extended down into the stiff clays. When these platforms were sunk, the skirts would penetrate until equilibrium was established, and then pumps were used to remove water trapped between the sea floor and the bottom of the mat. The pumping caused development of a pressure differential between the exterior water and water trapped inside the skirts, and this unbalanced pressure forced the platforms down against the stiff clay. Any remaining water trapped between the platform and the subsoil was displaced with concrete.

The concept of using skirts was used with smaller steel tubes, with closed tops and open bottoms, which were placed under legs of jacket structures, and installed with the aid of suction, by the Norwegians (Tjelta, 1994; NGI, 1997), who called them *bucket foundations*. The bucket foundations penetrated to depths of the same order as their diameters, a depth that was adequate for the comparatively stiff soils and/or relatively light compressive loads of the structures they supported. The ratio of penetration depth to diameter, termed the *aspect ratio*, generally ranged from 0.5 to 1.5. The early bucket foundations acted only in compression - they were surcharged to resist tensile loading.

As structures were placed in increasingly deep water in the Gulf of Mexico, it became apparent that jacket structures were increasingly uneconomical because of the cost of the large mass of steel required. The difficulties involved with supporting the huge weight of the jacket was a technical and economic problem as well. Design evolved into tension leg platforms (TLP’s), which were floating structures that were anchored with vertical steel pipes connected to foundations which were loaded in tension. Pile foundations were used because of past experience. However, the evolution from skirted gravity structures through bucket foundations led to the concept of using steel tubes with open bottoms and closed tops, which were set on the bottom and allowed to penetrate under self weight, and were then penetrated the rest of the way by pumping water out from inside the pipes, creating a pressure difference on the top. These foundations became known as *suction caissons*. Suction caissons were already in use with mobile offshore drilling units because they could be positioned accurately, maintained with added pumping if necessary, and then pumped back out of the ground for reuse after the conclusion of drilling.

OUR RESEARCH WITH SUCTION CAISSONS

Engineers in the oil industry began discussions of suction caisson foundations as a concept in perhaps the 1980’s. We thought that suction caissons had promise for use in normally consolidated clays in the Gulf of Mexico, provided the aspect ratio could be made large enough.

Initial Tests in Sand

We began with small caissons (4-inch outside diameter, aspect ratio=1.94). The soil medium was sand because the exploratory nature of the testing did not justify the time-consuming preparation of normally consolidated specimens of clay. The initial tests (Pavlicek, 1993; Jones, 1994) demonstrated:

1. When small caissons were pressed into sand, most of the resistance came from the tip, not the sides.
2. Suction caissons could be installed in sand provided that, during installation, there was an upwards hydraulic gradient in the soil plug that was sufficient to liquefy the sand in the plug, and thus reduce the tip bearing capacity greatly and allow penetration. We measured pore water pressures on the inner wall of the caisson to verify this concept and supported it with numerical analyses. It required 60-65 pounds to install the caisson without suction and only 7-12 pounds to install using suction.
3. Under rapid tensile loading with a closed top, substantial negative pore water pressures developed in the plug and caused the plug to come out with the caisson. Under those circumstances, the tensile axial capacity was substantial with a significant part due to the dead weight of caisson and soil plug and most of the rest with tip capacity.
4. If the top was open, or loading was slow enough to allow water to flow around the tip and up into the plug, there was a large reduction in capacity.
5. The failure in tension involved relatively small upwards displacements and there was a substantial reduction in holding capacity after failure (Fig. 3) and this behavior occurred regardless of drainage conditions. Side capacities developed with movements of the order of 0.15 inch, which is the same value observed for driven piles in the field.
6. The caissons withstood cyclic tensile loading well but the resulting suction on the water in the plug resulted in water gradually flowing around the tip and up into the plug, thus causing increased displacements and reduced capacity.
7. Caissons installed using a minimum amount of suction had slow penetration but elevated capacity whereas caissons installed using large suctions had rapid penetration rates but reduced tensile capacity. Presumably the large suctions loosened the sand.

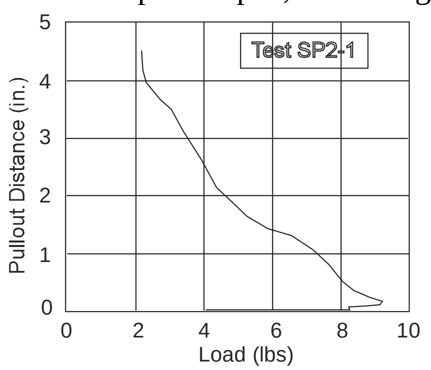

Fig. 3 Pull-out Test on Caisson in Sand

At the time of our tests with sand, the prevailing opinion in the industry was that caissons should not be used with aspect ratios greater than perhaps two because of the danger of sucking soil into the tube and reducing the total penetration, and thus reducing capacity.

Because these early tests were exploratory and there was limited interest in developing design procedures for caissons in sand, we did not attempt to quantify analytical methods. However, side capacities (Q_s) could be estimated using:

$$Q_s = \int f_s dA_s = \int \sigma_v^{'} K \tan(\delta) C dz \qquad (1)$$

where f_s is local side shear, dA_s is a differential element of side area, σ'_v is the free field vertical effective stress, K is the earth pressure coefficient, δ is the sand/caisson friction angle, C is circumference, and dz is a differential depth. Rational estimates of K (slightly greater than K_o) and δ ($<\phi$) led to reasonable predicted capacities.

Initial Tests in Clay

The major interest in the oil industry was in foundations in cohesive soils. Accordingly, we continued in the exploratory mode using clay (El-Gharbawy, 1998). To reduce the sample preparation time, we performed tests in barrels of kaolinite as opposed to preparing large tanks of clay. We accelerated consolidation by applying a partial vacuum at the base of the barrel during consolidation. The vacuum was released prior to testing, causing the top of the sample to be normally consolidated and the bottom overconsolidated. The result was that the rate of increase in undrained shearing strength with depth was larger than for normally consolidated clay and the clay became dilatant over the lower part of the depth of penetration. An added benefit of the overconsolidation was that undrained strengths of the clay were higher than for the normally consolidated clay, thus making it easier to sort out the parts of the tensile load that came from side shear and end bearing from the part due to the dead weight of the caisson and instrumentation.

El-Gharbawy's tests demonstrated that:

1. The concepts developed for sand applied to clays as well, including sudden failure at relatively small movements. Of course, the dissipation of installation pore water pressures was much faster for the sand than in the clay.
2. El-Gharbawy installed caissons with aspect ratios of 2 to 12, the 12 being more than twice the limit suggested by other researchers. He thus proved the feasibility of using deeper penetrating caissons in the Gulf of Mexico.
3. El-Gharbawy found that the axial capacity under conditions in which the soil was fully drained, were much less than when the soil was undrained. When the soil was drained, the failure was in the soil/caisson interface and the caisson pulled out with a clean surface. When the soil was undrained, failure was in the clay perhaps 1/8-1/4 inch out from the outer wall of the caisson and the caisson came out covered with clay. For simplicity, use F_{min} for the capacity with full drainage and F_{max} for the capacity without drainage, and F for any particular applied force.
4. Cyclic axial tensile loads less than F_{min} did not result in measurable accumulated displacements. If the load was equal to F_{max}, then the caisson pulled out at once if the load was maintained or in steps if the peak cyclic load was above F_{max} but the duration was short. For intermediate loads, the number of cycles to failure seemed to be controlled by the amount of time the load was above F_{min} and on the value of $F-F_{min}$, in that loading above the minimum started the consolidation process and the longer the load was applied the more complete was the consolidation (rebound).
5. To provide partial simulation of loading from tension leg platforms (TLP's), axial cyclic loads were also applied with the angle of loading cycling independently of the cycling of the load itself, simulating cyclic loading from a TLP that was drifting. The effect was a slight reduction in the axial capacity of the caissons.

El-Gharbawy's tests were instructive but he lacked time and funding to perform tests on normally consolidated clay, as in the Gulf of Mexico, and he could not measure some needed soil properties. His preliminary results were used by Chicata (2000) who was developing finite element code for suction caissons under axial loading.

Main Tensile Load Tests in Clay

The OTRC Industry Consortia and the MMS began funding a comprehensive and multi-year research program on suction caissons in clay in 1999.

Deficiencies in El-Gharbawy's work were addressed in a series of M.S. theses (Pedersen, 2001; Mecham, 2001, and Luke, 2002).

Pedersen and Mecham set up two tanks, 4 feet by 8 feet in plan, and 6 feet deep (Fig. 4) with suitable frames to handle equipment. A literature survey did not reveal evidence of anyone else using such large tanks but they were clearly needed for our tests. Pederson experimented with a series of clays and finally chose the same kaolinite used by El-Gharbawy. He also experimented with a number of types of apparatus in an effort to measure consolidation properties in the range of effective stresses (1-300 psf) encountered in our tanks. He developed a tilt-table shearing device and measured effective friction angles of the clay and of interfaces between the clay and aluminum and acrylic. He found effective friction angles of the clay in the lowest stress range of about 55 degrees, whereas he found an angle of about 28 deg. in the stress range typically used in laboratory tests. Pedersen measured pore water pressures and water contents at various depths and times, and surface settlements in the first two tanks.

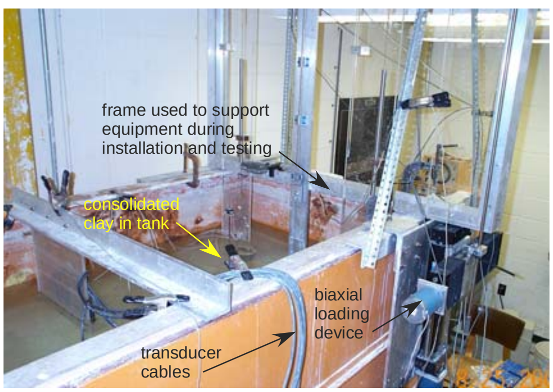

Fig. 4 Partial View of one Tank

Mecham concentrated on facility development. He designed the caisson used in subsequent experiments (Fig. 5). He used a thin aluminum tube so the ratio of outside diameter to wall thickness was comparable to values in field use (about 120). He developed pore pressure sensing devices and mounted them on the inside and outside of the caisson (Fig. 5). He

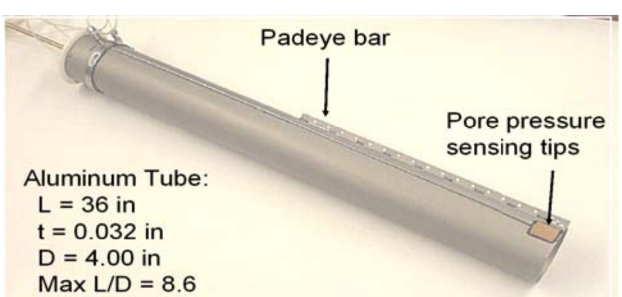

Fig. 5 Caisson Showing Pore Pressure Sensing Tips and the Padeye

also developed an improved data acquisition system to replace the ageing system that had been assembled from abandoned equipment in our laboratory. Mecham also developed the mechanical system used in installing and pulling out the suction caisson.

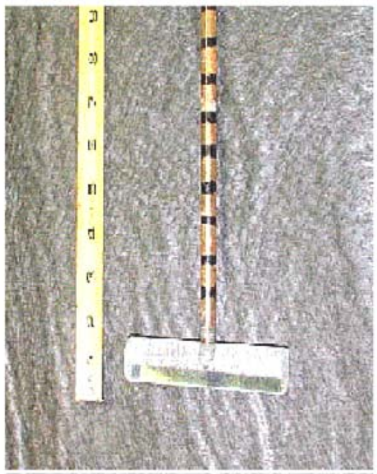

Fig. 6 Tee-Bar Used to Estimate Undrained Shearing Strengths

Four tanks of kaolinite were prepared with mixing in June 2000, June 2001, March 2003, and March 2004. Consolidation times were of the order of nine months. We developed an analytical solution for consolidation of the clay, including effective-stress-dependent properties and large strains, and showed that it worked well with data from our tank (Olson et al., 2003), except that the clay apparently engaged in an ageing phenomenon and thus settled slightly less than predicted. Mecham used the recently developed Tee-bar (Fig. 6) test to measure undrained shearing strengths in the tank.

About 40 suction caisson model tests have been completed to date to investigate axial, lateral, and inclined loading and the impact of installation methods.

Luke then used the equipment developed by Mecham and performed a series of axial loading tests with the new 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 persons doing finite element analyses. The simplified equation usually used for estimation of side shear for both piles and caissons is:

$$f_s = \alpha c_u \tag{2}$$

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:

$$q_p = c_u N_c \tag{3}$$

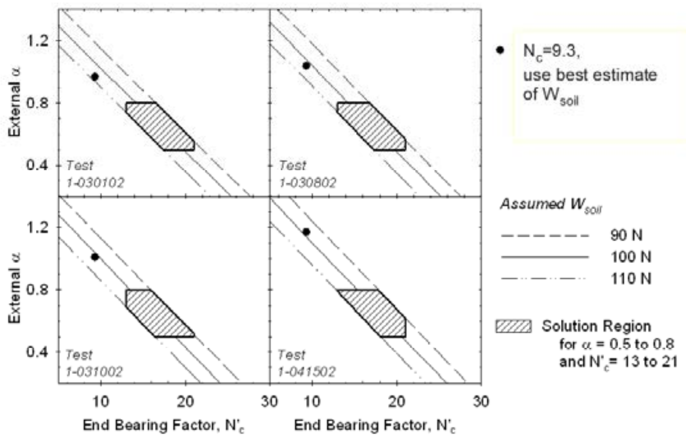

Fig. 7 Possible Combinations of α and N_c to Match Measured Capacities in four Tests

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. The expected value of N_c for normally consolidated clays was 9. 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. When the same values of α were used for failure on the outer surface, then N_c=13-21. It was also possible to assign the more usual values of α=1 and N_c=9 and match calculated and measured capacities (Fig. 7). The question of which of these options is correct remains open, and is important in field design.

Lateral Loading Tests in Clay

In addition to TLP's, the oil industry is now also using floating structures with catenary and taut mooring lines. Accordingly, instead of continuing with work on axial tensile loading, we switched first to purely horizontal loading and then to loading at an angle.

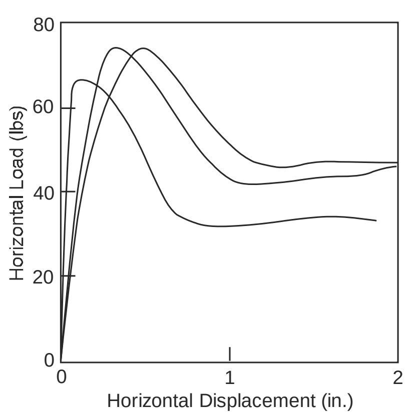

Fig. 8 typical Curves of Lateral Force Versus Lateral Deflection

Horizontal loading also led to definition of a peak capacity and then a significantly reduced capacity after further movement (Fig. 8). 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. 9). At the time we were engaged in the experimental phase of this work, companion projects at UTAustin 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 ("Fall 15") and finite element analyses are included in Fig. 9. The agreement is considered remarkable because:

1. The limiting equilibrium analysis requires input of undrained shearing strengths but the measured

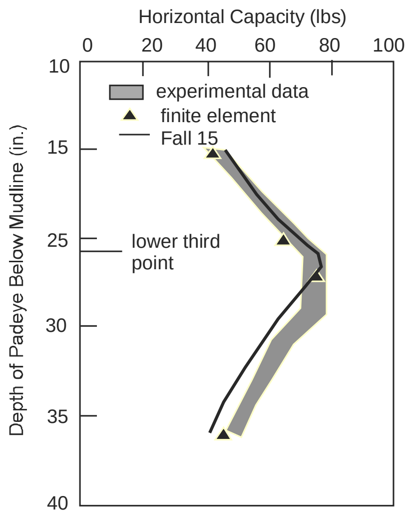

Fig. 9 Comparison of Measured Horizontal Capacity with Theoretical Solutions

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 measured the lateral capacity of the caisson with loading at the lower third point and 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. The test required about 3.5 months so only one test could be performed. The results were surprising in that they indicated the capacity with the soil drained was about 20% less than the capacity undrained. Neither time nor funding allowed further efforts to study the effects of using sustained loading.

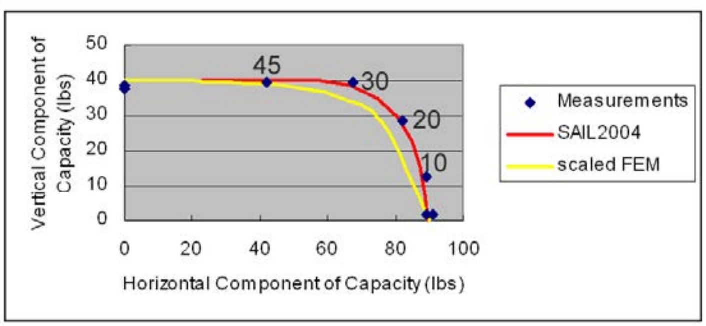

Fig. 10 Comparison Between Measurements and Analyses

With interest developing in taut-line loading, we ran a series of tests with loading at the lower third point with loading at an angle from the horizontal (Fig. 10). The experimental results agree with analyses performed using the limiting equilibrium program developed at TAMU ("SAIL2004"). The finite element analyses were performed with a different soil profile and scaled to fit the data for horizontal and vertical loading. The numbers next to the lines are angles of loading, measured from the horizontal.

CURRENT ACTIVITIES

Measurements of Clay Strengths

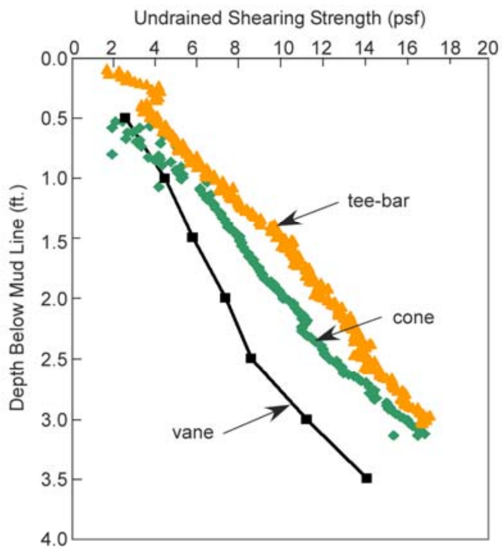

Fig. 11 Measured Undrained Shearing Strengths

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 tee-bar tests. We are in the midst of performing tests (Fig. 11) with a cone, field vane, tee bar, and a new ball test, and are planning tests in triaxial compression although such tests cannot be performed using samples as soft as the soil in the tanks. Measured side and tip capacities are normalized using the measured strengths (α in Eq. 2 and N_c in Eq. 3) so the choice of testing method in the field is important and the effect of testing method on calculated capacity must be understood.

Separation of Components of Resistance

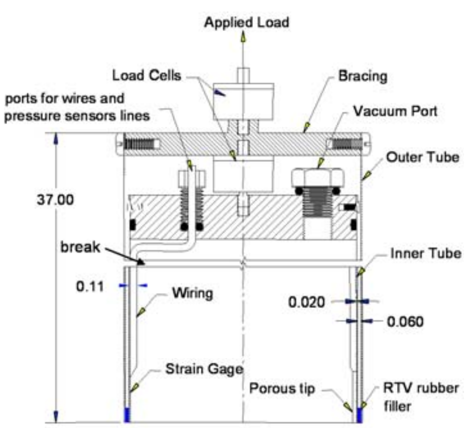

Fig. 12 Cross Section of Double-Wall Caisson

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. We are planning tests with a suction caisson composed of two, concentric, very thin aluminum tubes (Fig. 12) which are sealed at the bottom to prevent intrusion of clay between the walls. The total load in each tube will be measured to allow separation of load transferred on the outside from the load transferred on the inside and at the tip. The inner tube will be installed with strain gauges to allow an estimation of the variation of load with depth and thus, by extrapolation, to allow an estimation of the tip load.

CLOSING COMMENTS

This short report covers only experimental work on behavior of suction caissons as part of the Offshore Technology Research Center at the University of Texas and Texas A & M Universities. Excluded are the extensive activities by other experimental and theoretical researchers, as well as activities by oil companies, the American Petroleum Institute, and consulting firms. The early phases of the work were sponsored by the National Science Foundation and the later phases by the Minerals Management Service, with both phases also supported by various companies in the oil industry.

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 sand and provided some preliminary concepts about development of resistance to tensile loading, including cyclic and inclined cyclic loading. 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. A single long-term test provided surprising data which is not well understood currently.

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. There is a significant uncertainty in the tip bearing capacity factor and that topic is currently under investigation.

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 successful but there is now a need to produce code that others can use so variations in soil properties can be accounted for properly.

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.

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To save space and avoid having this report become a literature survey, we have deliberately held references to a bare minimum. We apologize to our colleagues who may feel slighted.

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