AN EXPERIMENTAL INVESTIGATION
OF THE BEHAVIOR OF A MODEL SUCTION CAISSON
IN A COHESIVE SOIL

by

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ABSTRACT

Suction caissons are large-diameter piles, closed at the top, that can be installed offshore using underbase suction. They have recently been developed as permanent anchorage systems for deep water tension leg platforms. Experimental results from a new automated laboratory device, the caisson element test (CET) cell, provide data regarding fundamental behavioral response of a cohesive soil to successive phases of installation by underbase suction, set-up, and axial tensile loading.

The CET cell comprises a miniature cylindrical caisson in a homogeneous, saturated element of clay with a controlled stress history. The model caisson is an unique two-component design that enables independent automated control of the caisson cap and wall. The model has a diameter, \( D_c = 5.1 \) cm, and a wall thickness, \( t_w = 0.145 \) cm, and penetrates to a depth \( L = 5.1 \) cm. The CET cell is instrumented to measure caisson component force and displacement, clay surface total stress and displacement, and pore pressure within the clay sample using needle-thin probes.

Fourteen CET tests were conducted on \( K_0 \)-normally consolidated re-sedimented Boston Blue Clay (RBBC) samples, at a consolidation stress, \( \sigma_{vc} = 0.75 \) ksc. The tests simulated caisson installation by underbase suction and by 'jacking', post-installation set-up with full pore pressure dissipation, undrained monotonic tensile loading, and sustained tensile loading. The test program investigated the pullout rate and the possible degradation of tensile load capacity following re-equilibration.

The test results reveal a wall resistance pattern consistent with data for long friction piles, loss of soil plug effective stress, and large cap heave during installation by underbase suction and 'jacking'. Post-installation set-up results show rapid redistribution of nearly all equilibrium load to the wall with small caisson settlement. During monotonic pullout, the wall provides approximately 60% of the total capacity. Skin friction calculations are consistent with measured wall resistance. Reverse bearing predictions overestimate cap resistance by \( \sim 50\% \), but more experimentation and analysis is needed to clarify this result. Sustained tensile loads less than undrained wall resistance lead to stable conditions, while greater loads eventually cause caisson failure. Multi-stage sustained load tests reveal a maximum wall resistance similar to that measured during undrained pullout.

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<tr>
<td>A/D</td>
<td>analog to digital</td>
</tr>
<tr>
<td>ABAQUS</td>
<td>commercially-available finite element code</td>
</tr>
<tr>
<td>AXIPLN</td>
<td>finite element model</td>
</tr>
<tr>
<td>BBC</td>
<td>Boston Blue Clay</td>
</tr>
<tr>
<td>CET</td>
<td>caisson element test</td>
</tr>
<tr>
<td>CFT</td>
<td>concrete foundation template</td>
</tr>
<tr>
<td>CL</td>
<td>low plasticity clay designation by Unified Soil Classification System</td>
</tr>
<tr>
<td>CPT</td>
<td>cone penetrometer test</td>
</tr>
<tr>
<td>D/A</td>
<td>digital to analog</td>
</tr>
<tr>
<td>EASYDAT</td>
<td>software that drives the MIT Central Data Acquisition System</td>
</tr>
<tr>
<td>ESP</td>
<td>effective stress path</td>
</tr>
<tr>
<td>GBS</td>
<td>gravity-based structure</td>
</tr>
<tr>
<td>INFIDEL</td>
<td>finite element model</td>
</tr>
<tr>
<td>KCL</td>
<td>potassium chloride</td>
</tr>
<tr>
<td>LVDT</td>
<td>linear voltage displacement transducer</td>
</tr>
<tr>
<td>MADC</td>
<td>multi-channel analog to digital converter</td>
</tr>
<tr>
<td>MIT</td>
<td>Massachusetts Institute of Technology</td>
</tr>
<tr>
<td>MIT-E3</td>
<td>generalized effective stress soil model developed at MIT (Whittle, 1987)</td>
</tr>
<tr>
<td>NCEL</td>
<td>Naval Civil Engineering Laboratory</td>
</tr>
<tr>
<td>RBBC</td>
<td>resedimented Boston Blue Clay</td>
</tr>
<tr>
<td>RTV</td>
<td>rubber adhesive</td>
</tr>
<tr>
<td>SPM</td>
<td>Strain Path Method</td>
</tr>
<tr>
<td>SSPM</td>
<td>Shallow Strain Path Method</td>
</tr>
<tr>
<td>TLP</td>
<td>tension leg platform</td>
</tr>
<tr>
<td>TSP</td>
<td>total stress path</td>
</tr>
<tr>
<td>UU</td>
<td>unconsolidated, undrained triaxial test</td>
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**CET TESTING**

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<td>CONS</td>
<td>control algorithm for CET consolidation</td>
</tr>
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<td>EQ1</td>
<td>first equilibration phase in CET testing</td>
</tr>
<tr>
<td>EQ2</td>
<td>second equilibration phase in CET testing</td>
</tr>
<tr>
<td>errCapv</td>
<td>error between measured and target cap velocity</td>
</tr>
<tr>
<td>errD</td>
<td>error between measured and target displacement</td>
</tr>
<tr>
<td>errFtot</td>
<td>error between measured and target $F_{tot}$</td>
</tr>
<tr>
<td>HOLDSTS</td>
<td>control algorithm for CET hold stress</td>
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<tr>
<td>HSintg</td>
<td>HOLDSTS module integration term</td>
</tr>
<tr>
<td>MONPULL</td>
<td>control algorithm for CET monotonic pullout</td>
</tr>
<tr>
<td>MP1</td>
<td>first monotonic pullout phase in CET testing</td>
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MP2  second monotonic pullout phase in CET testing
RD_{intg}  relative displacement integration term
SD  suction driving phase in CET testing
SL  sustained load phase in CET testing
SUCDRV  control algorithm for CET suction driving

**GENERAL**

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<tbody>
<tr>
<td>A</td>
<td>area</td>
</tr>
<tr>
<td>A_b</td>
<td>base area</td>
</tr>
<tr>
<td>A_c</td>
<td>cross-sectional area of cap</td>
</tr>
<tr>
<td>A_e</td>
<td>exterior wall area</td>
</tr>
<tr>
<td>A_i</td>
<td>internal wall area</td>
</tr>
<tr>
<td>A_o</td>
<td>object cross-sectional area</td>
</tr>
<tr>
<td>A_s</td>
<td>internal and external wall area</td>
</tr>
<tr>
<td>A_{tot}</td>
<td>total caisson cross-sectional area</td>
</tr>
<tr>
<td>A_w</td>
<td>cross-sectional area of wall</td>
</tr>
<tr>
<td>a</td>
<td>non-dimensional parameter</td>
</tr>
<tr>
<td>b</td>
<td>non-dimensional parameter</td>
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<tr>
<td>C</td>
<td>Celsius</td>
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<tr>
<td>C_{C&amp;S}</td>
<td>secondary compression rate</td>
</tr>
<tr>
<td>C_c</td>
<td>compression index</td>
</tr>
<tr>
<td>C_e</td>
<td>outer pile circumference</td>
</tr>
<tr>
<td>CR</td>
<td>compression ratio</td>
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<tr>
<td>C_s</td>
<td>swelling index</td>
</tr>
<tr>
<td>c</td>
<td>cohesion</td>
</tr>
<tr>
<td>c'</td>
<td>drained cohesion</td>
</tr>
<tr>
<td>c_u</td>
<td>undrained shear strength</td>
</tr>
<tr>
<td>c_{ub}</td>
<td>undrained shear strength at base</td>
</tr>
<tr>
<td>c_{um}</td>
<td>mean undrained shear strength along wall</td>
</tr>
<tr>
<td>c_h</td>
<td>horizontal coefficient of consolidation</td>
</tr>
<tr>
<td>c_v</td>
<td>vertical coefficient of consolidation</td>
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<tr>
<td>D</td>
<td>diameter</td>
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<td>inside diameter</td>
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<td>relative density</td>
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<td>e</td>
<td>void ratio</td>
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<td>F</td>
<td>applied force</td>
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<td>F_0</td>
<td>wall force intercept</td>
</tr>
<tr>
<td>F_b</td>
<td>breakout force</td>
</tr>
<tr>
<td>F_{b(L)}</td>
<td>breakout force (lower bound numerical limit analysis)</td>
</tr>
<tr>
<td>F_{b(U)}</td>
<td>breakout force (upper bound numerical limit analysis)</td>
</tr>
<tr>
<td>F_{blat}</td>
<td>lateral breakout force</td>
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</tbody>
</table>
F_{bmax}  maximum breakout capacity
F_c     cap force
F_{esf}  external wall skin friction
F_{ib}  immediate breakout force
F_p     penetration force
F_{reb}  reverse end bearing force
F_{sf}  anchor skin friction
F_{suc}  suction force
F_t     tip bearing capacity
F_{tot}  total force
F_w     wall force
F_{c}   cavity breakthrough factor
F_{q}    cavity breakthrough factor
f       penetrometer friction
f_{s}   average skin friction
f_{w}   wall force gradient
G_{s}   specific gravity of solids
G_{sec}  secant shear modulus
g   gravitational constant
H   height
H_{c}  clay height
H_{c}   average clay height
I_p    plasticity index
K   lateral coefficient of earth pressure
K_0    coefficient of earth pressure at rest
k   hydraulic conductivity
k_f, k_s  skin friction coefficient
k_p    tip resistance coefficient
k_v    vertical hydraulic conductivity
L   penetration length; length
L_{c}  depth factor
LIR   load increment ratio
M   compliance
M_c    cap modulus (=\Delta F_c/\Delta z_w)
M_{tot}  total caisson modulus (=\Delta F_{tot}/\Delta \delta_w)
M_w    wall modulus (=\Delta F_w/\Delta z_w)
m   slope constant; measured
m_v    coefficient of volume change
N_c, N_\gamma, N_q  bearing capacity factors
N_u    uplift coefficient
P_a    air pressure; applied pressure
P_o    output pressure
p'  \((\sigma'_v+\sigma'_h)/2\)
Q   load, collapse load
$Q_b$  downward bearing capacity
$Q_L$  lower bound collapse load
$Q_{\text{max}}$  maximum load
$Q_u$  uplift capacity
$Q_U$  upper bound collapse load
$q$  $(\sigma_v-\sigma_h)/2$
$q_{10}$  shear stress at 10% axial strain
$q_c$  cone resistance
$q_o$  object bearing stress
$q_{\text{ult}}$  ultimate capacity
$R$  breakout factor, radius
$R_o$  breakout factor at reference time, $t_0$
$r$  radius
$r_i$  inside radius
$r_o$  outside radius
$r_w$  radius from the wall
$S_c$, $S_\gamma$, $S_q$  shape factors
$s_{\text{DSS}}$  direct simple shear undrained shear strength
$s_{\text{TC}}$  triaxial compression undrained shear strength
$s_{\text{TE}}$  triaxial extension undrained shear strength
$T$  time factor, period
$T_{\text{cyclic}}$  cyclic tensile load
$T_{\text{static}}$  static tensile load
$t_b$  time to breakout
$tg$  target
$t_m$  reference time
$t_o$  reference time
$t_r$  reference time
$t_w$  wall thickness
$U$  average degree of consolidation
$u$  pore pressure
$u_i$  initial pore pressure
$ar{u}$  average pore pressure
$\Delta u$  excess pore pressure
$\Delta u_{\text{max}}$  maximum excess pore pressure
$V_i$  command voltage
$V_w$  wall volume
$v_w$  wall velocity
$W$  total weight
$W_a$  total anchor weight
$W_s$  total soil weight
$W_w$  water weight
$W_a^*$  buoyant anchor weight
$W_o^*$  buoyant object weight
$W_s^*$  buoyant soil weight
\( w \) width, water content
\( w_l \) liquid limit
\( w_p \) plastic limit
\( x \) lever arm distance
\( z \) depth
\( z_w \) wall displacement

\( \alpha \) adhesion factor
\( \beta \) skin friction factor (Ch. 2-7)
\( \beta \) opening angle of spherical segment (Ch. 2)
\( \gamma \) total unit weight
\( \gamma', \gamma_b \) buoyant unit weight
\( \gamma_d \) dry unit weight
\( \gamma_w \) unit weight of water
\( \Delta \) prefix to denote change (e.g., \( \Delta F_{tot} \))
\( \delta \) interface friction angle
\( \delta_c \) cap displacement
\( \delta_s \) soil surface displacement
\( \delta_w \) wall displacement
\( \varepsilon_a \) axial strain
\( \mu m \) micrometer
\( \sigma_h \) horizontal total stress
\( \sigma_i \) initial stress
\( \sigma_t \) tensile stress
\( \sigma_v \) vertical total stress
\( \sigma_{vc} \) total vertical consolidation stress
\( \sigma'_v \) vertical effective stress
\( \sigma'_{vc} \) vertical effective consolidation stress
\( \sigma'_h \) horizontal effective stress
\( \sigma_b \) average basal stress beneath cap
\( \tau_{max} \) maximum shear stress
\( \phi \) friction angle
\( \phi' \) drained friction angle
\( \phi'_{mo} \) friction angle at maximum obliquity
\( \phi_p \) friction angle at peak shear stress
\( \psi \) angle of shear stress obliquity
CHAPTER 1
INTRODUCTION

The dwindling supply of petroleum located in shallow offshore locations has driven the oil industry to tap oil reserves in deeper waters (>200 m). However, the cost of building conventional oil production support structures and foundations, such as steel jackets with driven piles, rises inexorably with increasing water depth due to the greater structural mass and foundation forces. Tension leg platforms (TLP's), an alternative to steel jackets, are buoyant structures that exert tensile loads via tethers connected to the foundation and, therefore, are well-suited to deep water locations. While TLP's have been constructed with driven pile foundations, the logistical difficulties associated with driving piles in deep water warrant an innovative foundation approach. Suction caissons, large-diameter open-ended piles that are installed with underbase suction, represent a feasible option to driven piles. Furthermore, because the method of installation is easier and quicker, suction caisson foundations allow a potentially significant cost savings compared to conventional driven piles.

Although foundation engineers recently have designed suction caisson foundations for gravity-based and tension leg structures, little experience and research has led to expensive field testing and conservative designs. At MIT, a new automated laboratory device, the Caisson Element Test (CET) cell, was developed to provide data regarding the fundamental behavioral response of a cohesive soil to suction caisson installation, set-up, and axial tensile loading. This thesis describes the CET cell, presents the results of a CET testing program that simulated successive phases of suction caisson events, and analyzes the test results by using principles of soil mechanics and soil behavior.
1.1 DEFINITION OF A SUCTION CAISSON

A suction caisson is a large-diameter hollow cylindrical pile that is open at the base and closed at the top and penetrates into the soil by a combination of its buoyant weight and underbase suction. Underbase suction is achieved by reducing the water pressure in the cavity between the caisson lid and the surface of the soil.

Once installed, the caisson resists compressive or tensile load through wall friction, self-weight of the soil plug, and resistance of the soil below the base of the caisson. For a gravity based structures (GBS), the suction caisson foundation is contiguous with the platform structure and transmits a large compressive stress to the soil. In contrast, under calm sea conditions, a tension leg platform (TLP) applies a tensile load to the foundation, which resists the tensile force through self-weight and ballast. However, during storms the TLP foundation must resist very large tensile load through wall skin friction and suction mobilized beneath the lid.

For both GBS and TLP applications, the suction caissons are arranged in multi-cellular units in order to ensure verticality during installation and provide structural support, and each unit penetrates the soil a short distance relative to the cell diameter. For the Snorre TLP structure, each cell has a large diameter, \( D = 17 \text{ m} \), but a relatively short embedment depth, \( L \approx 12.5 \text{ m} \), to give an embedment depth to diameter ratio, \( L/D \approx 0.7 \). These cells are arranged in 3-cell units, each of which has a maximum equivalent diameter of \( D = 35 \text{ m} \) and a base area of 720 m\(^2\). The entire unit is constructed of concrete, and each cell has a wall thickness, \( t_w = 0.35 \text{ m} \). Additional information regarding suction caisson foundation geometry and characteristics for other applications is contained in Chapter 2.

The experiments conducted for this thesis centered on the tensile loading of model caissons in order to simulate a foundation element for tension leg platforms.
1.2 MIT SUCTION CAISSON PROJECT

The work conducted for this thesis was part of a three year, joint industry-funded research project at MIT that addressed the geotechnical problems associated with the design of suction caissons as permanent anchors for deep water TLP's in order to provide the basis for a more efficient foundation design. This project focused on the caisson behavior in clay during installation by underbase suction, short-term (undrained) pullout, and sustained loading.

The project comprised an integrated program of analytical modeling, laboratory experiments, and evaluations of predictive capabilities using data from the model caisson tests. An analytical framework was developed to describe systematically the changes in soil stresses and pore water pressures during caisson installation and set-up (pore pressure equilibration). The goal of the analysis was to establish the effect of individual caisson geometry, cell configuration, and load duration on the caisson performance (load-deformation response). The study focused on the axial loading conditions encountered in tension leg platform applications.

This thesis encompasses the experimental investigation conducted for the MIT suction caissons project: a series of element level tests on model caissons to assess the specific aspects of caisson performance under well-controlled laboratory conditions. These tests provided data necessary to evaluate the analytic models in terms of their predictive capabilities. Other experimental data also were used to evaluate the analysis, including the results from centrifuge model tests performed by the Laboratoire Central des Ponts et Chausées in Nantes and supplied to the project by Exxon Production Research Company (Clukey & Morrison, 1993).

Results from this MIT project are presented in the Final Report (Whittle et al., 1996), an MIT Research Report (R94-09) on Deformation Analysis of Shallow
Penetration in Clays (Sagaseta et al., 1994), a PhD thesis in progress (Geer, 1997), and this thesis.

1.3 THESIS OBJECTIVES AND SCOPE

The primary objective of this thesis is to determine, through laboratory experiments, the behavioral response of cohesive soils to suction caisson installation, set-up, and axial tensile loading. To accomplish this goal, the research included the following tasks: 1) design and construction of the Caisson Element Test (CET) cell, 2) evaluation of the CET cell, and 3) analysis of the results of a testing program using the CET cell.

The first task comprised the design and construction of the CET cell, a new laboratory device that incorporates existing automation technology. The CET cell was designed to simulate the installation, set-up, and axial tensile loading of a miniature caisson in a homogeneous, saturated element of clay with a controlled stress history. The model is an unique two-component design that enables independent automated control of the caisson cap and wall. The CET cell is instrumented to measure caisson component force and displacement, clay surface total stress and displacement, and pore pressure within the clay sample using needle-thin probes.

The CET cell was evaluated during the first several tests in terms of its ability to: 1) simulate the successive phases of installation, set-up, and axial tensile loading, and 2) measure the behavior of the model caisson and the test soil during the simulation. Several modifications were made to the CET cell to correct limitations revealed by the evaluation.

The final task was performing and analyzing the results of a CET testing program (14 tests) on K_0-normally consolidated resedimented Boston Blue Clay (RBBC). The tests simulated caisson installation by underbase suction and by 'jacking', post-installation set-up with full pore pressure dissipation, undrained monotonic tensile loading, and
sustained tensile loading. In addition, the program investigated the effects of pullout rate and the effects of re-equilibration after partial pullout.

This thesis describes in detail the CET cell, test procedures, test material (RBBC), and the results of the equipment evaluation. In addition, the thesis presents the testing program and the measured behavior during each phase of every test. Most importantly, an analysis of the measured results outlines the fundamental behavior of suction caissons in the CET cell. In order to lay the groundwork for the CET test program analysis, the thesis contains background material, including a general suction caisson description and a review of previous research and applications.

1.4 ORGANIZATION

This thesis is divided into 7 chapters. Chapter 1 contains introductory information including a definition of suction caissons, a description of the joint industry-sponsored suction caisson research program at MIT, and the thesis objectives and scope.

Chapter 2 presents the background. This chapter begins with a discussion of the general characteristics of a suction caisson foundation used for tension leg platforms and the geotechnical problems associated with their installation and loading. It then covers previous research on embedded and surface objects, aspects of which relate directly to suction caisson performance. It continues with a review of prior experimental research on the behavior of suction caissons, including 1g laboratory, centrifuge, and field test programs. The chapter ends with a summary of the status of the geotechnical uncertainties associated with suction caissons.

In Chapter 3, the Caisson Element Test (CET) cell is described in detail. The bulk of the chapter discusses each of the CET cell's five components: consolidation chamber, model caisson, driving system, control system, and instrumentation. In addition, the
chapter summarizes the properties of the resedimented Boston Blue Clay (RBBC) used in the CET experiments. A final section presents a complete description of the CET test procedures.

Chapter 4 evaluates the CET cell in terms of its ability to: 1) simulate the installation, set-up, and axial tensile loading of suction caissons, and 2) measure the behavior of the model caisson and the test soil during the simulation. The chapter starts by chronicling the CET cell development throughout the testing program. It then evaluates the extant limitations of each of the five components that comprise the CET cell.

The results of the CET testing program, which consisted of 14 tests, is presented in Chapter 5. The first section discusses the test program philosophy and summarizes the individual test geometry, instrumentation package, loading schedule, and quality assessment. The heart of the chapter describes the basic characteristics that were measured during installation, equilibration, monotonic pullout, and sustained loading. Discussion focuses on force-displacement relations for the caisson, but also considers pore pressure and surface displacement of the surrounding clay mass.

Chapter 6 interprets the results of the model caisson testing program. The measured behavior during each of the model caisson events is summarized and analyzed using principles of soil mechanics and soil behavior. The chapter begins by discussing the resistance to caisson installation and the effect of underbase suction. It then analyzes the post-installation equilibration phase by focusing on the caisson component (wall and cap) force redistribution and rates of pore pressure dissipation, caisson settlement, and soil surface settlement. Monotonic uplift capacity of suction anchors in the CET cell is evaluated, and the effects of re-equilibration and pullout rate upon the uplift capacity are reviewed. Lastly, this chapter interprets the sustained load tests conducted in the CET cell.

In addition to the 7 chapters, 3 appendices provide supporting information. Appendix A contains the individual test geometry and instrumentation. Appendix B
summarizes prior RBBC properties and provides tabulated consolidation data for each CET test. Finally, appendix C lists the software code that was used to control each phase of the CET tests.
CHAPTER 2
BACKGROUND

A suction caisson is a large diameter hollow cylindrical pile that is open at the base and closed at the top and penetrates into the soil by a combination of its buoyant weight and underbase suction. Underbase suction is accomplished by reducing the water pressure in the space between the lid and the surface of the soil. Once in place, the suction caisson can resist compressive or tensile load through wall friction, self-weight of the soil plug, and resistance of the underlying soil. The size and shape of the caisson depends on its application, which can range from ship anchors to very large offshore oil platform foundations\(^1\). The foundation design for large offshore platforms, such as gravity-based structures (GBS) and tension leg platforms (TLP), incorporates a group of individual caisson cells in order to provide structural stability and ensure verticality during installation. For the Snorre TLP structure\(^2\), each cell has a very large diameter (D=17 m), but a comparatively short embedment depth (L=12-13 m) to give an embedment depth to diameter ratio of L/D≈0.7. These caisson cells are arranged in 3-cell units, each of which has a maximum equivalent diameter of D=35 m and base area of 720 \(m^2\). Additional information regarding typical foundation geometry and characteristics is contained in section 2.1.1. Because the experiments conducted for this thesis centered on the tensile loading of model caissons (see Chapters 5,6), this background chapter focuses on the suction caisson as a foundation element for tension leg platforms.

The chapter begins with a discussion of the general characteristics of a suction caisson foundation used for tension leg platforms and the geotechnical problems associated with their installation and loading. Section 2.2 covers previous research on

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\(^1\)Four suction caisson examples, including a mooring buoy support, a gravity based structure, and two tension leg platform foundations, are shown in Figures 2.28-2.34 (see section 2.4).

\(^2\)The Snorre and Heidrun TLP structures are discussed further in sections 2.1.1 and 2.4.
embedded and surface objects, aspects of which relate directly to suction caisson performance. Section 2.3 reviews prior experimental research on the behavior of suction caissons in particular. In section 2.4, four publicized field applications of suction caissons are reviewed. Finally, section 2.5 summarizes the status of the geotechnical problems associated with suction caissons.

2.1 GENERAL DESCRIPTION OF SUCTION CAISSON ANCHORS FOR TLP'S

2.1.1 Physical Characteristics of a Suction Caisson Anchor

It is not possible to list the attributes of a "typical" TLP suction caisson anchor, as only two prototype TLP's have employed suction anchors (Snorre TLP: Cottrill, 1991; Fines et al., 1991; Christophersen et al., 1992; Støve et al., 1992; Heidrun TLP: Munkejord, 1995; Botros et al., 1996; Miller et al., 1996). Both of these TLP's were installed in the North Sea and were constructed of concrete\(^3\). Details of these two designs are discussed in section 2.4. At this point, however, it is instructive to define briefly the common characteristics including the water and soil environment and caisson geometry.

At the Snorre location, the first 8 meters of soil are relatively soft, compressible clays and/or loose sands. This layer transitions to a stiff clay from 8 to 17 meters and to very stiff clay from 17 to 26 meters. Soil profile information at the Heidrun site has not been documented.

The water depth for the Snorre and Heidrun locations is 310 and 345 meters, respectively. Deep water sites can be a decisive factor in choosing a suction caisson foundation over more conventional designs (e.g., driven piles). In deep water the technical and logistical problems associated with conventional pile driving become very

\(^3\)For suction anchored TLP applications in the Gulf of Mexico, the anchors probably will be fabricated from rolled steel sections (Clukey and Morrison, 1993).
severe and thus the technique becomes more costly. In contrast, for suction caissons, installation is achieved by self weight\(^4\) and differential pressure across the lid of the caisson, which is achieved by pumping water from within the caisson. Hence, suction caissons are particularly well-suited to deep water sites underlain by low permeability cohesive soils.

The Snorre and Heidrun TLP designs have two significant geometric characteristics in common: 1) large diameters (Snorre \(D_0=17\) m, Heidrun \(D_0=9\) m) and 2) short penetration lengths, which lead to low length to diameter ratios (Snorre \(L/D_0=0.7\), Heidrun \(L/D_0=0.5\)). The two designs differ in how the single caissons are arranged to form the entire foundation base. The Snorre tension leg platform foundation is composed of four isolated Concrete Foundation Templates (CFT) (see Figure 2.32). Each CFT comprises 3 caisson cells and is located at one corner of a square plan area. The Heidrun TLP foundation also has four isolated foundation units, one at each corner of a square plan area. However, 19 caisson cells comprise each Heidrun unit (Figure 2.34). Details of the geometry for these designs are presented in section 2.4.

2.1.2 Phases in the Life of a Suction Caisson

The life of a suction caisson anchor can be divided into three separate phases: installation, equilibration, and platform loading. Each of the phases is described in this section with the aid of Figure 2.1. For simplicity, the complex multi-cell geometry of the actual foundation has been replaced by a single cell caisson in this figure.

During the installation phase, the suction caisson foundation structure penetrates the subsea soil by self weight and underbase suction. For the Snorre site, the four CFT structures (each CFT comprising 3 caisson cells) were towed on a barge. At the site, each caisson structure was lowered onto the seafloor by a ship-mounted crane. Trapped air in

\(^{4}\)For steel jacket and gravity platform applications, the self weight includes the buoyant weight of the attached superstructure.
the caisson compartments is allowed to escape through ports at the top of each caisson. Because of its submerged weight, the caisson freely penetrates the seafloor soil to a depth determined by the soil resistance to caisson wall friction and caisson tip bearing. During penetration, the water trapped in the upper part of the caisson escapes through ports at the caisson top. After the caisson reaches an equilibrium depth under self-weight penetration, submersible pumps (which are attached to the caisson tops prior to caisson lowering) reduce the pressure of the sea water beneath the lid of the caisson. By reducing this pressure, a net downward stress ('active suction') is applied to the caisson lid by hydrostatic stress, and this stress drives the caisson to the design depth. Water jetting at the caisson tip can liquefy particularly resistant sand layers and thereby aid the driving process. Verticality of the caisson structure while piercing the mudline can be maintained by the crane lift and pre-installed docking piles. For multi-caisson structures like those at the Snorre and Heidrun sites, verticality during subsequent penetration can be maintained by controlling the separate caisson chamber pressures. Upon reaching the required depth, the pump valves are closed to prevent further movement. During the installation process, the surrounding soil is severely disturbed and excess pore pressures will be induced in low permeability soils. The amount of this disturbance depends upon many factors including the soil properties, caisson geometry, rate of penetration, and the amount of applied suction (and water jetting).

The equilibration phase (see Figure 2.1b) describes the changes in soil stresses and properties that occur following installation and prior to the application of TLP superstructure loads. The caisson structure is allowed to equilibrate with the surrounding soil as the installation excess pore pressures dissipate\(^5\). The time required for complete dissipation depends upon caisson geometry, soil properties, and the gradients imposed by

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\(^5\)Dissipation can be expedited by a drainage system, which was used for the Gullfaks C gravity base structure (Tjelta et al., 1988, 1990).
drainage systems. The resulting changes in effective stresses outside and inside the caisson greatly influence the performance of the structure during platform loading.

The final phase in the life of a suction caisson is platform loading, as shown in Figure 2.1c. The TLP superstructure is designed with enough buoyancy to maintain a continuous tensile load on the structural tendons that connect the platform to the suction caisson foundation. Hence, the platform applies a static upward tensile force ($T_{\text{static}}$ in Figure 2.1c) on the foundation. In addition, sea currents and waves apply a static and cyclic load to the platform ($T_{\text{cyclic}}$), which is free to displace laterally and vertically in response. The displaced platform therefore transmits an eccentric static and cyclic load to the top of the suction caisson foundation, as shown in Figure 2.1c. For the Snorre tension leg platform design, enough ballast (3500 metric tons per CFT) was added to the weight of the suction caisson foundation (3500 metric tons per CFT submerged) to offset the tensile load due to the platform under calm weather conditions. Sea currents and waves impose a cyclic load component to the CFT's, which resist such additional loads through skin friction and suction within the skirts. During an extreme storm event, however, large sustained tensile loads are applied to the soil (e.g., for a "100 year" design storm, the tensile load on each Snorre CFT will vary between 0 and 14,200 metric tons). Since the suction caissons have sealed lids, tensile loads cause suction to be applied to the soil plug. Improvements in future caisson anchor designs require improved understanding of this tensile load response in order to minimize the costly design schemes that incorporate a high factor of safety.

2.1.3 Geotechnical Issues Associated with Suction Caisson Anchors

The research that is reviewed forthwith focuses on many different aspects relevant to suction caisson foundations. Thus it is helpful first to delineate only the important geotechnical problems inherent to a suction caisson foundation used for tension leg platforms. Two universal problems, the effect of caisson geometry and soil properties,
must be addressed when analyzing suction caisson behavior in each of the three phases in
the life of a suction caisson. The remaining problems can be classified according to one of
these three phases: installation, equilibration, and platform loading. The following list
classifies the important geotechnical problems with regard to suction caisson anchors:

Universal Problems

1. Effect of caisson geometry and cell configuration
2. Effect of soil properties

Classified Problems

Installation
1. Prediction of penetration resistance
2. Effect of underbase suction on penetration resistance
3. Effect of temporary delays during penetration

Equilibration
1. Prediction of rate of dissipation of installation excess pore pressures
2. Prediction of changes in soil stress and properties around the suction
caisson
3. Prediction of caisson settlement during equilibration

Loading
1. Prediction of ultimate capacity and frictional resistance of soil for
   normal and extreme storm loading
2. Prediction of foundation movement during normal and extreme storm
   loading
3. Effect of installation disturbance
4. Effect of incomplete equilibration
5. Effect of loading rate

The succeeding sections will focus only on research findings that illuminate the issues
listed above.
2.2 PREVIOUS RESEARCH RELATED TO SUCTION CAISSONS

The suction caisson as a foundation element to support large offshore oil platforms was not seriously researched until the early 1980s. Prior to this time, the influence of suction on objects in contact with the sea bottom was studied for a variety of applications including deep sea submersibles, soil exploration devices, ship and floating object anchorages, and sunken ship salvaging. Certain aspects of this early research apply directly to the loading phase of the suction caisson foundation (see section 2.1.2) and therefore warrant discussion. These studies can be divided into two categories: research on objects embedded below the seafloor and research on objects on the surface of the seafloor.

2.2.1 Embedded Objects

A variety of experimental research has concentrated on the pullout capacity of anchors for terrestrial and marine purposes (Baba et al., 1989; Bemben and Kupferman, 1975; Das and Singh, 1994; Davie and Sutherland, 1977; Meyerhoff and Adams, 1968; Nhiem, 1975; Shin et al., 1994). Results from these studies indicate that much of the pullout capacity is derived from resistance of the soil overlying the buried object, with an additional contribution from underbase suction. These projects produced information that helps interpret some of the geotechnical problems associated with the loading phase of suction caissons such as capacity prediction, pullout rate effects, geometric effects, and the effect of soil properties.

Several theories have been developed to predict the uplift capacity of embedded objects, but most do not account explicitly for the suction contribution. Two major analytically-based models, which do not account for suction, are briefly mentioned here because they often are used to analyze experimental pullout data. Meyerhof and Adams (1968) developed an approximate general theory of uplift resistance in soil based on model
uplift tests in sand and clay. The theory, based on experimental observations, assumes that the resistance of soil is a combination of the soil weight and shear resistance mobilized within a defined failure surface above the model footing. It is applicable to continuous or strip footings with modifications for individual square or circular footings and groups of footings. Figure 2.2 presents a schematic diagram of the geometry of the model tests that were conducted: a circular plate with diameter D (=1" to 4") is located within the soil at depth L and is pulled vertically until reaching capacity \( Q_u \). The model tests show that the uplift capacity is predicted reasonably well in sands\(^6\). The theory, however, underpredicts the uplift capacity of the model footings in clay because it does not account for the extra resistance due to underbase suction. Figure 2.2 shows a comparison of the theory and model tests for circular footings in clay in terms of uplift coefficient, \( N_u \), versus embedment depth ratio, \( L/D \). The \( N_u \) experimental values were backcalculated from the pullout loads based on an uplift capacity equation analogous to the bearing capacity equation (Meyerhof, 1951a):

\[
Q_u = (\pi D^2/4) c_u N_u + W
\]  \hspace{1cm} (2.1)

where, \( Q_u \) = measured uplift capacity corrected for suction  
\( c_u \) = cohesion  
\( W \) = weight of soil above anchor

Note that the measured values in the plot were corrected for suction\(^7\). The semi-theoretical line in Figure 2.2 represents the equivalent \( N_u \) backcalculated from the pullout load based on the theoretical full shear developed in the soil above the footing. The limiting theoretical \( N_u \) value for the maximum uplift capacity is simply taken as the bearing capacity coefficient \( N_c \). The shaded area in the plot represents the range of \( N_u \) values obtained when the data is not corrected for suction. By comparing the average 'uncorrected' data for soft clays with the semi-theoretical lines, it is clear that when suction

\(^6\)For sands, no significant underbase suction occurs due to pore fluid flow beneath the embedded anchor.  
\(^7\)The authors did not indicate the method used to correct for suction.
is included in the backcalculated $N_u$ values, the theory can seriously underpredict actual pullout capacities for undrained loading of soft clays wherein large negative pore pressures can be generated beneath the footing. (For the average measured data for stiff clays, the theory slightly overpredicts the pullout capacity). As an example, consider a model with an embedment ratio of $L/D=2$. The average 'uncorrected' measured data for soft clay yields $N_u=12$, which is 50% higher than the theory predicts for full cohesion $c_u$ ($N_u=8$). For sustained loading of footings in clay, Meyerhof and Adams (1968) estimated the uplift capacity that used the bearing capacity equation with drained strength clay parameters (e.g., $c'$ and $\phi'$). Model sustained load tests yielded pullout capacities that generally agreed with the estimated values.

Vesic (1969, 1971) proposed an embedded object uplift capacity equation based on analytical solutions for the expansion of spherical and cylindrical cavities close to the surface of a semi-infinite rigid-plastic solid. Like the solution suggested by Meyerhof and Adams (1968), the Vesic equation accounts for the resistance provided by the soil weight and shear resistance mobilized within a failure surface above the object, but neglects the effect of suction and soil adhesion to the object bottom. The uplift capacity is defined for embedded plates as:

$$Q_u = (\pi D^2/4) (c_u \bar{F}_c + \gamma L \bar{F}_q)$$

(2.2)

where, $Q_u =$ uplift capacity  
$D =$ width or diameter of object  
$c_u =$ cohesive strength  
$\bar{F}_c, \bar{F}_q =$ cavity breakthrough factors  
$\gamma =$ unit weight of soil above object  
$L =$ embedment depth of object

Vesic compared this theory with the breakout capacity measured in numerous model tests on sand and one model test series on clay and found that the theory is generally applicable for sands at shallow to deep embedment depths ($L/D$ ranging from 3 to 10 depending on the type of soil). Figure 2.3 compares the theoretical and experimental uplift capacity for
circular plates \((D=3\text{"})\) in clay by plotting the breakout factor, \(F_c\), versus embedment depth ratio, \(L/D\). Note that the experimental data does not account for the resistance due to suction, which was observed in the model tests. For low embedment ratios \((L/D < 2)\), the theory slightly underpredicts capacity for soft clay, while the theory overpredicts capacity for plates embedded at any depth in stiff clay. As with the Meyerhoff and Adams (1968) theory, if the suction component is included in the measured data, the Vesic theory cannot accurately predict the total uplift resistance of objects embedded in clay.

Other experimental research on embedded anchor pullout has indicated that suction is an important contribution to uplift. In some cases empirical curves that predict uplift include the suction component. Bemben and Kupferman (1975) conducted pullout tests on model anchor flukes \((D=3\text{"})\) in clay and observed that suction is a major component of the capacity. The model flukes were pulled vertically at a constant rate of 0.001 in/min, and the peak load was recorded. Figure 2.4 shows schematic views of the anchor fluke and a plot of the breakout factor \(N_u\) versus the embedment ratio \(L/D\). In this case, the \(N_u\) value is the ratio of the peak stress on the fluke to the cohesive strength. The peak stress is the peak load divided by the horizontal projected area of the fluke. For the tests without suction, a hole was drilled through the shaft to allow pore pressure equalization above and below the fluke. The results clearly indicate that suction is a significant component of the uplift resistance. The breakout factor due to suction, \(F_{suc}/c_u\), ranged from 5 to 6, where \(c_u\) is the cohesive strength. At deep fluke embedments \((L/D > 4.5)\), this suction accounts for nearly 40% of the capacity, while at shallower depths the suction contribution increases (e.g., at \(L/D = 2\), suction contribution is over 50% of total capacity).

The contribution of suction also was observed in a series of uplift tests on embedded circular anchors conducted by Davie and Sutherland (1977) to study the effect of tensile forces on terrestrial and offshore foundations. They conducted 65 model pullout tests using circular plates \((D=1\text{"} \text{ to } 8\text{"})\) in a glycerine and bentonite mixture called glyben.
For the majority of these tests, full suction was not allowed to develop, but two tests that allowed suction produced an ultimate capacity that was $3c_u$ greater than the non-suction tests. Only empirical curves based on the non-suction tests were developed and compared to the theoretical curves.

The suction contribution to pullout resistance was the focus of experimental research on embedded circular anchors by Baba et al. (1989). They conducted pullout tests on anchors that allowed suction to develop and on anchors that prevented suction. From these two sets of results they deduced the amount of suction contributing to the total uplift force in the tests that allowed suction. To describe the uplift capacity they used the following expression:

$$Q_u = W_a + F_{esf} + F_{suc} + (\pi D^2/4)c_u N_u$$  \hspace{1cm} (2.3)

where, $Q_u$ = uplift capacity  
$W_a$ = buoyant anchor weight  
$F_{esf}$ = friction between anchor side and soil  
$F_{suc}$ = suction force  
$D$ = width or diameter of object  
$N_u$ = uplift capacity factor  
$c_u$ = cohesive strength

For their analysis, Baba et al. (1989) neglected the anchor weight and side friction. The results gave a suction breakout factor, $F_{suc}/c_u$, ranging from 0 to 5.4 depending on the pullout rate and water content. No empirical curves to predict uplift were developed.

This same uplift equation was used to evaluate the results of 12 uplift capacity tests of plate anchors in clay by Das and Singh (1994) and Shin et al. (1994). The uplift tests were conducted on a Plexiglas circular plate anchor of diameter $D=5.1$ cm and thickness $t_\omega=0.13$ cm that was embedded in kaolin or montmorillonite. Figure 2.5a shows a schematic diagram of the model test setup. As in the tests conducted by Baba et al. (1989), the effect of suction is studied by comparing tests conducted with and without suction. As shown in Figure 2.5a, an anchor is pulled out without suction by placing a
pipe beneath the anchor. The effect of suction is shown by Figure 2.5b, which plots the
suction force as a percentage of total capacity vs. the embedment depth. The plot shows
that the suction contribution decreases from 100% at an embedment ratio, L/D=1, to less
than 20% at L/D=5.

From these studies on embedded objects in clay, it is clear that suction plays a
major role in the short-term uplift resistance. It is now appropriate to discuss the relevant
factors that affect the suction component of resistance to uplift of these embedded models.

The most obvious factor affecting suction resistance is soil properties. Of the six
projects discussed, three tested anchors on both cohesive and cohesionless soils (Meyerhof
and Adams, 1968; Vesic, 1969, 1971; Bemberg and Kupferman, 1975) and observed
suction only in the cohesive soil tests. As noted, suction was observed in the other four
projects, which conducted tests on soft clay. Because no stress control was attempted on
the clay specimens, which were constructed from remolded natural clay or from mixing
clay powder and water, the effect of stress history on suction could not be evaluated. Shin
et al. (1994), based on results of their tests in kaolin and montmorillonite, concluded that
varying the clay mineralogy and cohesive strength had little effect on the relationship
between suction contribution and embedment depth, as shown in Figure 2.5b.

Baba et al. (1989) conducted tests on clay with and without suction at four
different rates ranging from 0.2 mm/min to 25 mm/min. The results show that, for a given
clay water content, the percentage of suction contribution to the total uplift capacity
increases with pullout rate. At very slow rates of pullout, pore fluid has time to migrate
and, therefore, negative excess pore pressures can dissipate. The uplift capacity in this
case depends on the drained strength of the soil above the object. Meyerhof and Adams
(1968) analyzed model tests of anchors that were pulled out of cohesive soils in a drained
state (small increments in pullout load were added each day until pullout occurred). They
found that the measured drained capacity compared very well to the estimated drained
capacity according to their theoretical model, which is a modification of the bearing
capacity equation with drained strength parameters. This is reasonable because the theory assumes that the resistance is due purely to mobilized shear above the embedded object, and because a drained condition prevailed, no additional resistance was provided by suction.

2.2.2 Surface Objects

In general, the contribution of suction to the pullout resistance was not the focus of embedded object research. For objects that lie directly on soil or that are only partially embedded, the effect of suction plays a much more important role in the pullout resistance. Several studies have been conducted on the pullout characteristics of surface objects, which range from simple solid objects to surface anchors with skirts. Table 2.1 lists the important experimental research conducted on the pullout resistance of surface objects. Included in this list are the model description, soil tested, and test variables. Due to space limitations, a full description of each test program is not presented, but important findings are summarized. Conclusions from this research that are relevant to suction caisson behavior include the prediction of immediate and long-term pullout capacity, object geometry effects, soil property effects, and load rate effects.

The experimental models developed to predict the uplift resistance of surface objects can be classified into empirical, local shear, and general shear models. The first documented research to predict pullout was reported by Muga (1967), who developed an empirical model based on over 40 pullout tests on objects resting on cohesive soil in the San Francisco Bay as part of a research project for the U.S. Naval Civil Engineering Laboratory (NCEL) to determine the force required to remove objects from the ocean bottom. Some tests were conducted at a constant rate of load until breakout, while others were conducted at a constant rate of load up to a certain load level, which was then held until breakout. Muga correlated applied force to time to arrive at the following equation for immediate and long-term breakout:
\[ F = W_0 + aA_0q_0e^{-R(t-t_m)}, \quad 0 < t < t_m \] 

(2.4)

where, \( F \) = applied force  
\( W_0 \) = buoyant object weight  
\( a \) = non-dimensional parameter (determined at \( t=t_m \))  
\( A_0 \) = horizontal projection of object maximum contact area  
\( q_0 \) = object bearing stress at embedment depth  
\( R \) = non-dimensional parameter  
\( t \) = time elapsed during which \( F \) is applied  
\( t_m \) = reference time interval after which increase in time does not result in decrease in \( F \)

Another empirical model was developed by Lee (1972, 1973), who analyzed the San Francisco Bay tests, additional field tests on similar objects in the Gulf of Mexico, and laboratory pullout tests on smaller objects. By correlating the breakout force to normalized embedment depth (L/D), Lee obtained the following equation for immediate (undrained) breakout:

\[ F_{lb} = W_0 + Q_b(1 - ae^{-b(L/D)}) \]  

(2.5)

where, \( F_{lb} \) = immediate breakout force  
\( Q_b \) = estimated downward bearing capacity  
\( L \) = depth of embedment  
\( D \) = smallest lateral object dimension  
\( a, b \) = non-dimensional parameters (curve-fitted)

For long-term breakout prediction, Lee correlated breakout force to a time factor to produce the following empirical equation:

\[ \log(F_b/F_{lb}) = -a(\log T - b) \]  

(2.6)

where, \( F_b \) = long term breakout force  
\( T = (F_{lb}t_b/A_0L^2)(D/L)^2 \)  
\( t_b \) = time to breakout  
\( A_0 \) = object cross-sectional area  
\( a, b \) = non-dimensional parameters (curve-fitted)

Figures 2.6a-c show the empirical curves produced by Lee (1972, 1973) and the data that was used to produce them. The significant scatter in the data for all three curves may
indicate the effects of uncontrolled soil characteristics, varying object sizes, varying object resting periods prior to pullout, and varying rates of loading. Because of such variability, these curves can only be useful as a guideline for objects and soils similar to those tested.

Das (1991) revisited the issue of long-term breakout time by reviewing previous research and conducting several pullout tests. The model pullout object was a closed-end aluminum cylinder with a diameter D=8.3 cm and height H=8.3 cm. This was penetrated between L=8.3 and 8.8 cm into soft clay and allowed to rest for a specified period of time \( t_r \) to allow some pore pressure dissipation, after which the model was loaded in tension with a predetermined load. The time to breakout then was recorded. Three resedimented silty clays of varying plasticity were tested. Based on the results of these tests and a theoretical study of the bottom breakout force vs. time relationship by Foda (1983), Das formulated the following equation to predict the breakout time for an object resting on soft clay:

\[
\log(R) = \log(R_0) - m \log(t) \tag{2.7}
\]

where, 
- \( R \) = breakout factor = \( \frac{F - W'_a}{W'_a} \)
- \( R_0 \) = \( R \) at \( t = t_o \)
- \( m \) = slope constant
- \( t \) = time required until breakout
- \( t_o \) = reference time for immediate breakout
- \( F \) = applied tensile load
- \( W'_a \) = buoyant anchor weight

Figure 2.7 shows the breakout factor \( R \) plotted vs. breakout time \( t \) for four different resting periods \( t_r \) and three clays. It is clear that the data agree well with the empirical formula. Das made several observations: 1) for a given object resting time \( t_r \), the breakout factor \( R \) has a linear relationship with breakout time \( t \) on a log-log plot, 2) for a given soil and breakout time \( t \), the breakout factor increases with increasing object resting time \( t_r \), and 3) for a given soil, the slope \( m \) increases with increasing rest time \( t_r \), and 4) the slope reaches a limiting value of \( m = 0.08 \) at an in-situ rest time of \( t_r = 24 \) hours. This last
observation may indicate that pore pressure dissipation essentially is completed within 24 hours. However, this observation could not be verified because pore pressure was not measured. If they had, all of these trends could be explained in terms of dissipation of excess pore water pressure that was generated during the model installation. Further observations regarding the effect of soil properties will be made below.

The empirical relations developed by Muga (1967) and Lee (1972, 1973) are useful as a guide in determining the pullout resistance for their respective surface object and soil scenarios. The empirical equation by Das (1991) appears to predict very well the breakout time for a given object and soil. However, further model testing would be required to develop new correlation parameters in order to apply these models to new situations.

The second class of experimental prediction models is based on the assumption that the mode of failure is local shearing of the soil beneath the withdrawn object (Brown and Nacci, 1971; Helfrich et al., 1976; Wang et al., 1975, 1977, 1978). The breakout force in these models is derived by performing a force equilibrium analysis on the observed failure surface. The surface objects in the research programs were all skirted anchors that employed active suction for resistance. Table 2.1 lists the geometric and soil conditions and Figure 2.8 depicts the model anchors and free body diagrams of the anchors at failure. By determining the forces required for equilibrium on both sides of the failure surface, the breakout force can be predicted as follows:

\[ F_b = W'_a + W'_s + F_{sf} + F_{suc} \]  \hspace{1cm} (2.8)

where, \( F_b \) = breakout force
\( W'_a \) = buoyant weight of anchor
\( W'_s \) = buoyant weight of soil plug
\( F_{sf} \) = friction and adhesion force between anchor surface and soil
\( F_{suc} \) = vertical force due to suction acting on the failure surface
All three local shear prediction models employ some form of this equation, but differ in how they interpret the friction force, suction force, and in the exact shape of the local failure surface. Brown and Nacci (1971), who conducted tests in sand, neglected the frictional force on the anchor walls and postulated that the effective stress on the failure surface is equivalent to the negative excess pore pressure developed by the active suction. To obtain the pore pressure on the failure surface for the breakout force prediction, they used electric analog model tests. Helfrich et al. (1976), who used the same prediction model as Brown and Nacci (1971), but with a different failure surface (see Figure 2.8) predicted breakout to within 13% of measured values. Wang et al. (1975, 1977, 1978), who conducted tests on sand, silt, and clay, derived a more rigorous solution to the friction and suction terms based on Mohr-Coulomb failure criterion. For the force acting on the wall, both adhesion and the frictional effect of the overburden and suction were included. For the suction force on the failure surface, the effects of partly drained conditions, overburden, and pressure from steady state pumping were included. The resulting expression includes factors that must be determined using numerical techniques, and the predictions agree very well with the test results in sand and silt (see Figure 2.9). For tests in clay, the model underpredicted the breakout force, possibly because the model does not account for clay consolidation (and the resulting strength increase) due to pumping prior to pullout.

The last class of uplift prediction models is based on an assumption of general soil failure at breakout (Finn and Byrne, 1972; Byrne and Finn, 1977; Rapoport and Young, 1985). They assume that the failure mechanism is similar to that for a bearing capacity failure with the direction of movement reversed. Byrne and Finn (1977) conducted tests on clay to verify this prediction model. Rapoport and Young (1985) analyzed the results of former pullout programs. The general form of the breakout equation, based on bearing capacity (Vesic 1973), is as follows:
\[ F_b = W'_o + A_b (S_c cN_c + S_\gamma'DN_\gamma - S_q\gamma'LN_q) \quad (2.9) \]

where, \( F_b \) = breakout force
- \( W'_o \) = buoyant object weight
- \( A_b \) = object base area
- \( c \) = cohesive strength
- \( D \) = object width
- \( L \) = object embedment depth
- \( \gamma' \) = effective unit weight of soil
- \( S_c, S_\gamma, S_q \) = shape factors
- \( N_c, N_\gamma, N_q \) = bearing capacity factors

The bearing capacity factors \((N_c, N_\gamma, N_q)\) are functions of the drained friction angle if soil consolidation under the object weight was allowed prior to pullout. If no consolidation was allowed, then a \( \phi=0 \) analysis is appropriate (i.e., \( c=c_u, N_\gamma=0, N_q=1 \)) (Rapoport and Young, 1985). Byrne and Finn (1977) modified the reverse bearing equation to model the uplift of a skirted anchor embedded in clay, with \( \phi=0 \), as shown in Figure 2.10. For this situation, there is no resistance contributed by the weight of the soil, \( S_\gamma\gamma'DN_\gamma \), and the resistance contributed by the soil overburden above the bottom of the anchor, \( S_q\gamma'LN_q \), is canceled by the effective weight of the soil plug. They assume that there is shear resistance in the overburden and therefore incorporate a depth factor \( L_c \) applied to the cohesive resistance. In addition, the buoyant weight of the anchor is neglected. The resulting equation is:

\[ F_b = A_b S_c c_u N_c L_c \quad (2.10) \]

where, \( L_c \) = depth factor

Using this reverse bearing equation, the measured maximum force values for six rapid (0.5 mm/min) pullout tests were within 6% of the predicted capacity. Based on these tests, three sustained load tests (partly-drained conditions), and tests conducted at different pullout rates, Byrne and Finn also suggested a method for predicting the time to breakout for a sustained load less than the undrained capacity. They estimate the reduction in
maximum breakout load to be proportional to the average degree of pore pressure dissipation according to the following equation:

$$\frac{F}{F_{bmax}} = 1 - U$$  \hspace{1cm} (2.11)

where, $F =$ sustained load
$F_{bmax} =$ maximum undrained capacity
$U =$ average degree of consolidation

Using the relationship between average degree of consolidation and a time factor for a circular loaded area for their experimental setup, Byrne and Finn found a good comparison between the theoretical and observed breakout reduction.

Rapoport and Young (1985) adopted the reverse bearing theory for uplift capacity after conducting qualitative laboratory tests to observe the failure mode during uplift. They then suggested using equation (2.9) to predict the undrained breakout for any shallow object. In addition, they recognized that if the pullout force is less than the capacity, then, with time, drainage could change effective stresses in the soil and cause long term breakout. They illustrated their procedure by predicting breakout for former experimental program tests that were covered previously by Lee (1972, 1973) and Wang et al. (1977).

The effect of geometry on the resistance to uplift of surface objects was recognized early by Goodman et al. (1961), who conducted laboratory tests on small diameter (D≈3") inverted suction cups penetrated into silt, sand, and clay to study the feasibility of using vacuum to anchor objects in soil. Their results indicated that, given a fixed pump capacity, greater suction could be obtained with deeper penetration of the suction cup. The penetration to diameter ratio for their study was very low, ranging from $L/D = 0.07$ to 0.6. The only other test program that incorporated different object geometries was the active suction anchor resistance test series reported by Wang et al. (1975, 1977, 1978). They tested a variety of skirted anchors with diameters ranging from
D = 4.375" to 13.25", wall thicknesses from \( t_w = 0.125" \) to 0.25", and penetration lengths from 0.375" to 6.375". The resulting penetration length to diameter ratios ranged from \( L/D = 0.073 \) to 0.5 and the diameter to wall thickness ratios ranged from \( D/t_w = 22 \) to 106. For a given sand density and active suction, the anchor capacity was greater for short skirts (\( L/D = 0.07-0.1 \)) than for long skirts (\( L/D = 0.4-0.5 \)). Wang et al. attribute this greater capacity to the fact that the short skirt local shear failure cone is closer to the top of the anchor, where the suction pressure, and hence effective stress, is greater. For the tests in silt and clay, the trend is reversed; anchor capacity was greater for the long skirted anchors than for the short skirted anchors. Since cohesive strength contributes more resistance to the anchors in clay and silt, the effect of greater suction near the top plays a lesser role in the uplift capacity. Hence greater capacity for longer skirts in clay and silt could be attributed to increased strength of the anchor soil plug due to consolidation from suction-induced seepage, increased anchor wall frictional resistance from deeper penetration, and increased soil resistance due to a more general mode of failure.

Only three of the research programs on surface objects conducted tests to evaluate the effect of soil properties on anchor resistance. For the long-term breakout tests on a closed-end aluminum cylinder, Das (1991) studied the effect of three clays with differing plasticity. Figure 2.7, discussed above, showed plots of breakout factor \( R(=[F-W_d]/W_d) \) vs. breakout time for these three soils. The figure also shows a plot of the slope \( m \) relationships vs. in-situ rest time \( t_r \). This plot indicates that for a given \( t_r \), as the soil plasticity increases, the slope \( m \) decreases. Das believes that for a given clay content, increasing plasticity is accompanied by decreasing permeability and therefore a decreasing \( m \), but admits that further lab tests would be needed to validate this assertion. Brown and Nacci (1971) conducted their suction anchor tests on a well-graded sand in both a loose state (\( D_r = 22\% \), \( \gamma_d = 100.6 \) lb/ft\(^3\)) and a dense state (\( D_r = 65\% \), \( \gamma_d = 105.6 \) lb/ft\(^3\)). They observed that for a given active suction, pullout capacity for anchors in the dense sand
was up to twice that of anchors in loose sand. Wang et al. (1975, 1977, 1978) conducted several capacity tests in sand, silt, and clay, the results of which were plotted in Figure 2.9. From their tests on suction anchors in poorly graded sand, they noticed that for a given anchor geometry and suction, anchors in dense sand \( (D_r=90\%, \ \gamma_d=102.5 \text{ lb/ft}^3) \) offered from 20 to 40% greater resistance than those in loose sand \( (D_r=40\%, \ \gamma_d=97.0 \text{ lb/ft}^3) \). They attribute the greater capacity to the higher internal friction angle in the dense sand. The resistance of anchors in silt was lower than resistance in the loose sand, and the silt resistance increased with increasing active suction at a lower rate than for loose sand. The pullout resistance of anchors in clay rose only slightly with increasing suction and therefore was highly dependent on the cohesive strength, as shown in Figure 2.9. For a given clay type, the cohesive strength could vary widely depending on stress history. Wang et al. attributed the lower capacity increases with increasing suction for silt and clay to their lower angles of internal friction.

Of all the research conducted on surface objects, only one program considered the effect of pullout rate on the breakout capacity. Byrne and Finn (1977) examined the rate effect by conducting tests at four different rates, and they discovered that the breakout capacity increased dramatically with increasing rate. The maximum uplift force increases linearly with the log of the strain rate; for one log cycle increase in strain rate, the capacity increases seven fold. Byrne and Finn attributed the capacity differences to partial drainage: the lower the pullout rate, the greater the dissipation of excess pore pressures, and hence the greater the strength decrease. This explanation is plausible because the drainage path from anchor interior to mudline is short \( (L/D = 0.25) \), but it is unclear whether the ultimate "undrained" breakout force at the highest pullout rate \( (7.6 \text{ mm/min}) \) was truly undrained.
2.3 PREVIOUS EXPERIMENTAL STUDIES ON SUCTION CAISSONS

The focus in this section turns to experimental research that was conducted specifically for suction caisson foundations for offshore applications. This research can be divided into three categories: 1g laboratory tests, centrifuge tests, and field scale tests. Within each of these three categories, discussion will address the universal and classified geotechnical problems, which were outlined in section 2.1.3.

2.3.1 1g Lab Model Tests

Five experimental suction caisson test programs have been conducted using 1g laboratory scale models. The scale ranged from 1:20 to 1:100 and no attempt was made to recreate soil profiles identical to those found in the field. The objectives of these studies was to gain insight into the mechanisms underlying installation and loading behavior and to compare the results to analytical predictions. Table 2.2 lists pertinent information regarding the lab programs including the objectives, model geometry, soil properties, and test variables. Prior to discussing the major findings, a brief summary of each program is in order.

Larsen (1989) conducted 15 suction caisson tests using steel models with a diameter of $D_o=10.4-30.5$ cm, a wall thickness of $t_w=0.15$ cm, an embedment ratio of $L/D_o=1.5-4$, and a wall thickness ratio of $D_o/t_w=69-203$. The soils were tested in a container open to the atmosphere and include a dense sand, a remolded-reconsolidated natural clay, and kaolin. The stress history of the clay soils was not specified. In each test, the caisson model was penetrated into the soil using suction and then loaded in tension. A cyclic or static load was applied to the top of the caisson at an angle of between 3 and 6° inclined above horizontal. Because the tensile load was primarily horizontal, the results of the loading phase are of limited value for TLP anchorage applications.
Fuglsang and Steensen-Bach (1991) reported the results of 15 pullout tests on model caissons in kaolin clay of two different uniform strengths, but an unspecified stress history. The piles ($D_o=6.5, 8$ cm) were pushed into the clay to a depth equal to twice the diameter ($L/D_o=2$). They were then vertically pulled out at a constant rate of either 6 or 0.6 cm/min. Additional tests were conducted with the pile top vented to prevent suction and therefore obtain wall resistance values. Note that this test program did not model the suction installation phase, but nonetheless did provide some insight into the passive suction development during extraction.

Pavlicek (1992) and Iskander et al. (1993) conducted 8 suction caisson tests in a stress chamber at the University of Texas/Austin using an acrylic model pile with a diameter, $D_o=10.9$ cm, a wall thickness, $t_w=0.38$ cm, and a penetration between $L=5.3$ and 7.2 cm. This gave a penetration to diameter ratio of $L/D_o=0.5$-0.7 and a diameter to wall thickness ratio of $D_o/t_w=29$. The pile was suction-installed into a container of dense, poorly-graded sand with a 30 cm head of water open to the atmosphere. The caisson then was extracted at a constant rate of 46 cm/min. From test to test, the amount of suction during installation was varied to study the effect of installation disturbance on pullout resistance. In one test, the pile was pulled out with the top vented to isolate the pullout resistance due to wall friction.

Jones et al. (1994) discuss the results of additional UT/Austin suction caisson tests that explored the effects of loading rate on the pullout capacity. For this program, the pile had a diameter of $D_o=12.7$ cm and a wall thickness of 1.6 cm to give a diameter to thickness ratio of $D_o/t_w=8$. Using suction, the pile penetrated a dense sand to a depth of $L=10.2$ to 12.7 cm, which yields a ratio of $L/D_o=0.8$-1.0. To examine the effects of loading, piles were pulled out of dry sand to simulate drained loading and they were pulled out of saturated sand at a rate of 76 N/sec to simulate partially drained loading. The effect of cyclic loading was investigated by one-way stress-controlled cyclic loading at rates between 4 and 15 hertz in several tests.
The final 1g lab study in Table 2.2 comprised 8 pullout tests conducted on a pile that had been pushed into a remolded clay of unspecified stress history (Singh et al., 1994). The pile diameter ranged from $D=4.4$ to 7 cm and the penetration depth was $L=12.5$ cm to give an embedment ratio of $L/D=1.8-2.8$. The wall thickness was $t_w=0.3$ cm, which gave a thickness ratio of $D/t_w=14.5-23$. In three of the tests the pile top was vented to model the pullout of conventional open-ended pipe piles. In the remaining five tests the top was sealed to allow the mobilization of passive suction. Three pullout rates ranging from 0.2 to 25 mm/min were used to examine rate effects.

1g Lab Tests: Installation

Of the studies cited, three provided information regarding suction installation. Larsen (1989) observed that underbase suction caused enough upward seepage through the developing plug of soil to reduce the effective stress to zero in this zone and therefore facilitate pile penetration. He deduced that penetration resistance consisted primarily of friction along the outer wall. Information regarding the effect of the various soils and penetration lengths was not provided.

Pavliceck (1992), who conducted all 8 tests in sand, also observed the dramatic reduction in penetration resistance due to applied suction. Based on pore pressures measured by transducers mounted on the caisson inner walls, he calculated that upward seepage had induced a critical gradient; liquefaction of the sand in the plug had occurred. Penetration resistance prediction was not attempted, but comparisons showed that suction installation resistance was 82 to 88% less than the resistance to installation by pushing. The amount of suction required to advance the caisson to a particular depth was not studied, but a lower suction caused a lower penetration rate, and the final penetration depth was reached when the pile top reached the excess soil plug. The excess soil plug is that soil inside the pile that is above the level of the adjacent soil outside the pile, as shown in Figure 2.11. Based on calculations of wall displacement volume and excess soil plug
volume, Pavlincek concluded that the excess soil plug was created by both material displaced by the wall and material drawn upward by liquefaction. Hence the height of the excess soil plug could be an indicator of the amount of soil disturbance, which lowers the resistance to pullout.

The installation characteristics observed by Pavlincek (1992) were confirmed by Jones et al. (1994), who derived penetration resistance formulae to calculate relative resistance distribution between side shear and tip bearing. They assumed the penetration capacity to be as follows:

\[ F_p = -W_a + F_{sf} + F_t \]  \hspace{1cm} (2.12)

where, \( F_p \) = penetration resistance  
\( W_a \) = submerged weight of pile  
\( F_{sf} \) = inner and outer side friction  
\( F_t \) = tip bearing capacity

Using this equation for a pile pushed into sand, they calculate that the tip bearing contributes 91% and the wall friction adds 9% to the total resistance. In great contrast, for a suction-installed pile, they calculate that the tip bearing contributes 66% and the wall friction 34% to the much-reduced overall capacity. In agreement with Pavlincek's (1992) tests, they measure a 85% reduction in total resistance by using suction instead of pushing.

1g Lab Tests: Load Capacity Prediction

Two of the 1g laboratory scale test programs yielded useful information regarding geotechnical problems associated with the axial tensile load capacity of suction anchors. Fuglsang and Steensens-Bach (1991) observed two different modes of failure from the results of 15 vertical pullout tests of piles in kaolin. Figure 2.12 shows the load vs. pullout diagrams for 1g and 40g (centrifuge) tests with piles at two different diameters (65mm and 80mm). The 40g tests will be discussed below in section 2.3.2. The researchers noticed that in 6 of the 15 tests, the pile failed suddenly in tension at the base of the soil plug after
extensive deformation, as illustrated by the dashed curve. In the remaining tests, the pile failed gradually as the clay underwent large plastic deformations, as shown by the solid curve. Two different capacity equations were developed and applied to each test, regardless of failure mode. The first equation incorporates the resistance due to pile and soil weight, wall friction, and reverse bearing capacity as follows:

\[ F_b = W_a + W_s + W_w + \alpha c_{um} A_c + A_b(c_{ub} N_c - \gamma L) \]  \hspace{1cm} (2.13)

where, \( F_b \) = breakout force
\( W_a, W_s, W_w \) = total weight for anchor, soil, and water above anchor
\( \alpha \) = adhesion factor
\( c_{um}, c_{ub} \) = cohesive strength (mean along pile wall and at base)
\( A_c, A_b \) = exterior wall area and pile base area
\( L \) = object embedment depth
\( \gamma \) = unit weight of soil
\( N_c \) = bearing capacity factor

Note that this equation is a modification of the reverse bearing capacity equation first applied to suction anchors (Byrne and Finn 1977, see section 2.2.2). Fuglsang and Steensø-Bach recognized the effect of pile embedment by adding the wall friction term (\( \alpha c_{um} A_c \)) and subtracting the overburden stress at the tip elevation (\( A_b \gamma D \)). Using this equation for the 65 mm diameter piles in the kaolin with a cohesive strength \( c_u = 10 \text{kPa} \), they found \( N_c = 8.3 \pm 1.4 \text{SD} \). For 80mm diameter piles in the kaolin with strength \( c_u = 20 \text{kPa} \), they calculated an \( N_c = 6.4 \pm 1.4 \text{SD} \).

Fuglsang and Steensø-Bach also backcalculated \( N_c \) values from force equilibrium on the clay plug. The equation is:

\[-uA_b + \alpha c_{um} A_i = W_s + A_b(c_{ub} N_c - \gamma L) \]  \hspace{1cm} (2.14)

where, \( u \) = measured uncorrected pore pressure at plug top
\( W_s \) = weight of soil plug
\( \alpha \) = adhesion factor
\( c_{um}, c_{ub} \) = cohesive strength (mean along pile wall and at base)
\( A_i, A_b \) = interior wall area and pile base area
\( L \) = object embedment depth
\( \gamma \) = unit weight of soil

\( N_c \) = bearing capacity factor

This produced an average \( N_c = 8.8 \pm 1.0 \text{SD} \) in the weaker clay and \( N_c = 7.9 \pm 1.3 \text{SD} \) in the stronger clay. Assuming the plug moved with the pile (i.e., no interior friction), they calculated \( N_c = 7.5 \pm 1.0 \text{SD} \) (weaker clay) and \( N_c = 5.5 \pm 1.0 \text{SD} \) (stronger clay).

The second capacity equation is based on the observed tensile failure in the soil at the base of the soil plug. Essentially, the reverse bearing capacity term, \( A_b(c_{ub}N_c - \gamma D) \) from equation (2.14), is replaced by the total tensile stress, \( \sigma_tA_b \), as in the following equation:

\[
F_b = W_a + W_s + W_w + \alpha c_{uw}A_e + \sigma_tA_b \tag{2.15}
\]

where, \( \sigma_t \) = limiting tensile strength across base of pile

Using the \( N_c \) values calculated from the first equation (2.14), the tensile stress can be calculated for each test. The total tensile stress used in this equation (2.15) is based on the maximum breakout resistance, which was mobilized before tensile fracture. Therefore, the researchers acknowledge that the limiting tensile strength may be different at the moment of fracture. It must be noted that because the stress history of the clay samples was not carefully controlled, the mean and bottom cohesive strengths of the kaolin \( (c_{um}, c_{ub}) \) varied widely from test to test, and this could greatly affect the backcalculated \( N_c \) values.

Figure 2.13 shows the tensile load versus pullout distance for 3 of the 8 pullout tests in sand performed by Pavlicek (1992). Included in these figures are the pore pressure records for three transducers located inside the pile. The curves show that the peak tensile load is reached at a pullout distance of about 20% of the pile diameter, and the pile has little residual capacity following the peak. Pavlicek observed that a plug of sand filling the entire caisson was retained upon pullout. From the load-displacement curves and the observed failure surface, the failure mode appears to be local shear.
Equations to predict the capacity based on failure mode were not developed but the various components of resistance that contributed to the capacity were calculated. A breakout capacity equation identical to equation 2.8, \( F_b = W_a + W_s + F_{esf} + F_{suc} \), can be constructed based on force equilibrium taken on the pile at maximum tensile load. Pavliceck assumed that the buoyant pile and soil weight \((W_b, W_s)\) were constant in each test and calculated the suction force \( F_{suc} \) by linearly extrapolating the pressure gradient to the pile tip based on pore pressure measurements inside the caisson. The friction force \( F_{esf} \) was deduced from the other force values. Using this scheme, Pavliceck found that the submerged pile and plug soil contributed 13 to 18% of the capacity, the wall friction contributed 26 to 44%, and the suction contributed 41 to 61%. The variations can be attributed to the effects on tensile strength of varying degrees of installation disturbance for each test.

The monotonic pullout results of Pavliceck (1992) were corroborated by tests reviewed by Jones et al. (1994), who developed the following equation to estimate side friction during pullout:

\[
F_{esf} = (0.5\gamma'z^2 - \bar{u}z)C_cK\tan(\delta)
\]  

(2.16)

where, \( F_{esf} \) = wall friction  
\( \gamma' \) = submerged unit weight of sand  
\( z \) = penetration depth  
\( \bar{u} \) = change in pore pressure on pile exterior  
\( C_c \) = outer pile circumference  
\( K \) = outside lateral coefficient of earth pressure  
(assumed \( K=0.8; \) API RP2A)  
\( \delta \) = sand/pile friction angle  
(obtained from sand/pile direct shear tests)

Using this equation, Jones et al. calculated the side shear to contribute 29% of the total tensile capacity for pullout under partially drained conditions (pile top closed), which is consistent with Pavliceck's (1992) observations. Because of the small pile size and the high hydraulic conductivity of the sand, imposed stress-controlled cyclic loads greater than the
drained capacity could not sustain suction, and the pile gradually withdrew from the sand without retaining a soil plug.

The pullout load vs. pile displacement results of the tests reviewed by Singh et al. (1994) are shown in Figure 2.14. In comparison to the open-ended piles (i.e., vented top cap), the suction piles have higher capacity, but require larger displacements to mobilize this resistance (i.e., the system is softer at the working load range). Indeed, clay plugs were retained in the suction piles, while in the vented open-ended pipe piles, slip occurred along the inner and outer walls of the pile. Although it is not shown in Figure 2.14, Singh et al. report that the peak load in the suction piles occurred at a displacement equal to approximately 30% of the pile diameter.

**1g Lab Tests: Effects of Geometry and Soil on Loading**

Different caisson geometries were used within each of the lab scale test programs (Table 2.2), but only one program isolated the effect of geometry. Singh et al. (1994) used three different diameter piles at one single embedment depth (L/D=1.8-2.8). As shown in Figure 2.14, the trend of load vs. displacement remains the same for the three piles, and as expected, for a given penetration depth, a greater diameter pile allows more mobilization of the reverse bearing capacity in the clay mass.

The effect of soil properties in the 1g test programs is masked by varying caisson geometries. A global comparison, however, can be made between the Fuglsang and Steensen-Bach tests on kaolin and the Pavlicek tests on sand. Both programs used a similar caisson geometry and pullout rate, but yielded very different load-displacement behavior. Comparing the load-displacement curves for each program, as depicted in Figures 2.12-2.13, it is clear that the piles in kaolin retain much more residual strength following peak load than do the piles in sand.
Ig Lab Tests: Effect of Installation Disturbance

Pavlicek (1992) showed that increasing installation disturbance decreases the tensile capacity. As shown in Figure 2.13, the pullout capacity for the piles installed using minimal suction (test series 4) was higher than those installed using relatively high vacuum (test series 1). The high vacuum installation piles caused more disturbance to sand, as evidenced by an average height of the excess soil plug that was 16% higher than that for the minimal suction tests. Hence, it is clear that for these piles in sand, greater disturbance decreases pullout capacity. In fact, a close look at the components that account for the pullout resistance reveals that the capacity reduction for the pile installed with high suction was due mainly to a reduction in caisson wall shear resistance. The buoyant soil plug weight and underbase suction contributions to resistance appear to be unaffected by variations in installation disturbance.

Ig Lab Tests: Effect of Rate of Loading

In the test program on piles in kaolin (Fuglsang and Steensen-Bach, 1991), the piles were pulled at rates of 6 and 0.6 cm/min and there was no discernible difference was in the pullout behavior. However, for the piles in clay studied by Singh et al. (1994), pullout rates of 0.02, 0.1, and 2.5 cm/min were used and, as Figure 2.14 shows, the capacity increases with increasing rate. Because the clay used in these two test programs was not placed under strict stress control and because the possibility of partial drainage at low pullout rates was not investigated, no general conclusion can be drawn regarding the effect of pullout rate for piles in clay.

2.3.2 Centrifuge Model Tests

The centrifuge has been used in three experimental research programs to model suction caisson behavior using realistic gravity stresses and stress gradients (Fuglsang and Steensen-Bach, 1991; Renzi et al., 1991; Renzi and Maggioni, 1994; Clukey and
Morrison, 1993; Clukey et al., 1995). Table 2.3 lists the important information for each of these experimental studies. Note that all three programs used clay as a test material.

In addition to the 15 1g lab scale model tests (see section 2.3.1), Fuglsang and Steensen-Bach (1991) reported the results of 4 pullout tests conducted in the centrifuge at an acceleration equal to 40 times the acceleration due to gravity (40g). The model pile dimensions, penetration lengths, and pullout rates were identical to those for the 1g tests ($D_o=6.5$, $8$ cm; $L/D_o=2$; $\Delta L/\Delta t=6$, $0.6$ cm/min). The piles were pushed into kaolin clay prior to 40g acceleration and were pulled out at an acceleration of 40g. Because the soil strength and stress history were not measured at 40g, only qualitative comparisons can be made to the tests conducted at 1g.

Renzi and Maggioni (1994) and Renzi et al. (1991) reported the results of 4 suction caisson tests conducted at 100g. The model had an outside diameter of $D_o=15$ cm and a wall thickness of $t_w=0.4$ cm to give a diameter to thickness ratio of $D_o/t_w=37.5$. The soil model was a resedimented clayey silt that was consolidated with a gradient at 1g to give a profile of increasing effective stress with depth. For each of the tests, an acceleration of 100g was maintained throughout testing. First, the soil was reconsolidated under its own weight. Then, a penetration test with a miniature piezocone was conducted to yield a strength profile and to allow correlation between cone penetrometer resistance and caisson skirt wall penetration resistance. The caisson then was penetrated 9 to 10 cm into the clay under self weight. Following this, underbase suction was used to penetrate the caisson at varying rates to a final depth of approximately 17 cm to give a penetration length to diameter ratio of $L/D_o=1.1-1.2$. Dissipation of penetration pore pressures was allowed for 70 minutes. Finally, the vertical pullout test was conducted. The pullout history varied from test to test, but involved sustained loading, cyclic loading, and load-controlled pullout sequences. Figure 2.15 shows the loading program for all four tests. For model tests 1 and 2, the pile underwent sustained loading at a working load (WL) of 2.4 kN for 52 minutes, then one-way cyclic loading for 26 minutes (period $T=8$ sec), and
finally sustained load increments until failure. For model tests 3 and 4, the pile was loaded to failure at a constant rate of stress (0.5 kN/sec) five successive times with a 26 minute reconsolidation period after each failure. After the fifth failure, the pile was reconsolidated under a lower sustained load for 52 minutes, after which the pile was loaded to failure.

Clukey and Morrison (1993) and Clukey et al. (1995) discuss the results of 5 monotonic and 7 cyclic centrifuge tests conducted at 100g. For the soil model, Speswhite kaolin clay was resedimented and consolidated at 1g in several layers at different stresses to produce a soil profile of increasing strength with depth. Three of the monotonic and all of the cyclic tests used a single element caisson with diameter \( D_o = 15.2 \text{cm} \) and wall thickness \( t_w = 0.06 \text{ cm} \) (\( D_o/t_w = 253 \)). For two monotonic tests, a multi-cell caisson was used, but the geometric configuration was not reported. The actual test was divided into two phases. For the installation phase, the soil model was reconsolidated at 100g, a strength test was conducted with a cone penetrometer, and the caisson was penetrated to approximately \( L = 30.5 \text{cm} \) to give a penetration to diameter ratio of \( L/D_o = 2 \). The caisson and soil were then spun down to 1g to replace the installation hardware with pullout hardware. For the pullout phase, the soil was reconsolidated around the caisson at 100g, a cone penetrometer test was conducted\(^8\), and then pullout loads were applied. In the five monotonic tests, a load-controlled pullout test was conducted at a slow or fast displacement rate. The cyclic tests fell into three categories according to load inclination: vertical, inclined (6° from vertical), and variable inclination (load angle varied throughout test). Each cyclic test was performed in multiple steps, with each step incorporating an average (static) load, a specific cyclic load amplitude, and a number of cycles (usually 500) at a certain frequency (1, 2.5, or 5 Hz). If large deformations (\( \delta_w \approx 0.1 \text{ cm} \)) did not occur during cyclic loading, a static post-cyclic pullout test was performed.

\(^8\)Cone penetrometer tests were conducted to yield a strength profile and to compare with caisson wall skin friction distribution, as determined from strain gauge data.
Centrifuge Tests: Installation

In both 100g programs, the caisson was installed using suction, but only Renzi and Maggioni (1994) and Renzi et al. (1991) review the results. To predict the force required to penetrate the caisson with suction to a specified depth, Renzi et al. used the following expression:

\[ F_p = -W_a' + \alpha c_{uw}A_s + A_b(c_{ub}N_c + \gamma' L) \] (2.17)

where, \( F_p \) = penetration force
\( W_a' \) = buoyant anchor weight
\( \alpha \) = adhesion factor
\( c_{uw}, c_{ub} \) = cohesive strength (mean along pile wall and at base)
\( A_s, A_b \) = exterior and interior wall area and pile base area
\( L \) = object embedment depth
\( \gamma' \) = buoyant unit weight of soil
\( N_c \) = bearing capacity factor (\( N_c = 9 \); assumed for deep penetration)

Using this equation, they were able to predict reasonably well the penetration resistance in both the self-weight and suction phases of penetration. Figure 2.16 shows measured and predicted penetration resistance versus depth curves for both self-weight and suction penetration of 4 model tests at 100g. Using an adhesion factor of \( \alpha = 0.25-0.30 \), the force required to penetrate the caisson under self-weight can be predicted well. To predict the force required to penetrate using suction, this factor must be increased to \( \alpha = 0.35-0.40 \). Because the other clay properties in this force equation remained constant in this prediction model, it is assumed that suction only affects the adhesion factor. Renzi et al. also noted that in each of the four tests, a suction of 50 kPa was required to advance the caisson to the desired depth.

Centrifuge Tests: Equilibration

In each of the tests discussed by Clukey and Morrison (1993) and Clukey et al. (1995), the soil and caisson were spun at 100g for 24 hours following penetration and an
average degree of consolidation of at least 90% was achieved prior to pullout. In each of
the four 100g tests reviewed by Renzi and Maggioni (1994) and Renzi et al. (1991), the
excess pore pressures from penetration were allowed to dissipate for only 70 minutes prior
to pullout and the average degree of consolidation for the soil outside the caisson was no
greater than 60%. Furthermore, only 25% of the pore pressures had dissipated inside the
caisson. No predictive models were formulated for the pore pressure and effective stress
changes involved during equilibration.

Centrifuge Tests: Load Capacity Prediction

All three centrifuge test programs used a breakout capacity equation based in part
on the resistance contributed by the buoyant weight of the caisson, skin friction between
the caisson walls and the soil, and reverse bearing capacity of the soil. Fuglsang and
Steensen-Bach (1991) used the same equation to predict both the 1g and centrifuge tests
(equation 2.14). As for the 1g tests, they gauged the effectiveness of their prediction by
backcalculating and comparing the breakout capacity factor $N_c$. They found that the
average breakout factor for the smaller diameter piles ($D_o=6.5$ cm) was $N_c=8.3$, while the
factor for one of the larger diameter piles ($D_o=8$ cm) was $N_c=6.4$. These values compare
very well with the values obtained for the 1g tests. However, it is unclear whether or not
the undrained strength values used in the equation were obtained at 40g. In addition, the
tensile mode of failure, which was observed in some of the 1g tests, was not observed in
the 40g tests. Hence, further testing would be required to validate the prediction model
for both 1g and multi-g tests.

Renzi et al. (1991) used essentially the same equation (2.17) to predict pullout that
they used for installation resistance, which was described above. For installation, the
buoyant weight $W'_b$ is subtracted from the resistance and the overburden $\gamma'L$ is added,
whereas for pullout, the buoyant weight adds to the resistance and the overburden is
subtracted. The pullout equation is:
\[ F_b = W_a' + \alpha c_{um} A_s + A_b (c_{ub} N_c - \gamma' L) \]  \hspace{1cm} (2.18)

where, \( F_b \) = breakout force
\( W_a' \) = buoyant anchor weight
\( \alpha \) = adhesion factor
\( c_{um}, c_{ub} \) = cohesive strength (mean along pile wall and at base)
\( A_s, A_b \) = exterior and interior wall area and pile base area
\( L \) = object embedment depth
\( \gamma' \) = buoyant unit weight of soil
\( N_c \) = bearing capacity factor (\( N_c = 9 \); assumed for deep penetration)

The comparison of the predicted capacities to the measured capacities from the four model tests is inconclusive. Figure 2.17 shows the load displacement curves for the capacity tests on four model tests at 100g. Note that several tests were conducted on models 3 and 4, and the capacity increased with each successive test. This indicates the strengthening effect of reconsolidation between each capacity test. In Figure 2.17, pile capacity is plotted versus the depth of embedment for both measured and predicted values. The results show that as the pile reconsolidates and settles with each successive capacity test, the capacity increases and approaches a predicted capacity assuming near-maximum mobilization of wall friction (adhesion values \( \alpha = 0.90-0.95 \)). This conclusion neglects the strength changes in the clay that may be occurring due to continued consolidation from suction penetration and the numerous loading events. However, Renzi et al. recognize the strength increase from sustained loading, as shown by the final capacities for models 3 and 4. As shown in Figure 2.17, these capacities lie well beyond the predicted lines, and they indicate the significant strength increase due to sustained loading for 52 minutes just prior to the final pullout.

Clukey and Morrison (1993) also incorporate both wall skin friction resistance and reverse end bearing in their prediction model, but assume that the overburden reduction (\( \gamma' L \)) is canceled by the buoyant weight of the soil. In addition they incorporate the contribution of internal skin friction in the reverse end bearing term and include the resistance due to shearing in the overburden in a depth factor, as follows:
\[ F_b = W_a' + F_{esf} + A_b C_{ub} N_c S_c L_c \]  

(2.19)

where, \( F_b \) = breakout force  
\( W_a' \) = buoyant anchor weight  
\( F_{esf} \) = external skin friction  
\( A_b \) = pile base area  
\( N_c \) = bearing capacity factor (\( N_c = 5.14 \))  
\( S_c \) = caisson shape factor (\( S_c = 1.2 \))  
\( L_c \) = depth factor (\( L_c = 1 + 0.18 \tan^{-1}(D/L) \))  
\( D \) = caisson diameter  
\( L \) = penetration length

The external skin friction value is based on an undrained shear strength equal to 75% of the strength determined from cone penetrometer tests (CPT) conducted prior to load testing. This strength reduction is based on prior experience with skin friction calculations\(^9\) and skin friction determinations made during one test.

Internal and external skin friction was examined during one test, which incorporated strain gauges attached to the caisson walls, total stress sensors beneath the caisson lid, and pore pressure instrumentation in the soil plug and below the caisson. The average measured internal and external skin friction distribution was obtained from the strain gauge and total stress data. Using the total and pore pressure data, force equilibrium calculations were made on the soil plug and caisson to obtain separate internal and external skin friction values. The calculated external skin friction was 80% of the average shear strength obtained from the CPT results. This evidence supports Clukey and Morrison's practice of reducing the undrained shear strength data from in-situ testing for external skin friction calculations.

For all five monotonic pullout tests, which include single and multi-cell caissons and two different pullout rates, the measured uplift capacity was 80.5% of the calculated capacity. For the two tests on single cell caissons pulled out at the faster rate, the

\(^9\)Clukey and Morrison (1993) normally use shear strengths obtained from laboratory (UU triaxial) tests to calculate skin friction for pile capacity. In their experience, strengths from UU tests are 70 to 80% of the in-situ (e.g., CPT) strengths. Details regarding the CPT strength calculations were not provided.
measured uplift was 89.1% of the predicted value. To gain a better understanding of the reverse end bearing term, Clukey and Morrison computed the "suction efficiency", which is the ratio of the mobilized reverse end bearing to the calculated reverse end bearing. The mobilized value was obtained by subtracting the calculated external skin friction from the measured capacity, while the calculated value was simply the third term in the above equation. The overall suction efficiency (all five tests) was 73% and the single cell/fast rate suction efficiency was 85.4%. Clukey and Morrison derived their reverse end bearing term from compressive end bearing theory. They hypothesize that the suction efficiency is less than 100% because the reverse end bearing uplift term does not account for all the differences between compression and uplift, such as lower clay strength for extension loading.

**Centrifuge Tests: Effects of Cyclic Loading**

Only the tests discussed by Clukey et al. (1995) incorporated cyclic loading. They found that the caisson response depends directly on the combination of average (static) load, cyclic load, and type of loading (inclination angle). In essence, fewer load cycles are required to fail the caisson as the cyclic load amplitude, static load level, and inclination angle increase. In particular, combined loading, where the load inclination angle is varied throughout the test, causes fewer cycles to failure for a given cyclic load level. Post-cyclic monotonic pullout tests conducted on those caissons that did not fail during cycling yielded a static capacity that is on average 15% higher than predicted capacity based on the monotonic tests. The authors attribute this increase to consolidation around the caisson during cyclic loading.

**Centrifuge Tests: Effects of Geometry and Soil on Loading**

All three centrifuge programs tested clay models and the effect of varying soil properties was not examined. The only centrifuge program to study the effects of varying
geometry was the 100g tests reviewed by Clukey and Morrison (1993), who tested multi-cell and single-cell caissons in the monotonic testing program. The multi-cell caisson had the same overall diameter and penetration length as the single-cell caisson, but the number of cells was not reported. One multi-cell test was conducted at the fast pullout rate and had a suction efficiency of 78.9%, and one was conducted at the slow pullout rate and had an efficiency of 70.8%. These values are lower than the efficiencies calculated for the single-cell/fast-rate (85.4%) and the single-cell/slow-rate (74.8%). Clukey and Morrison suggest that the reduction in efficiency may be due to the additional disturbance to the soil from the multi-cell geometry, but additional tests would be necessary to verify this assertion.

Centrifuge Tests: Effect of Incomplete Equilibration and Rate of Loading

In each of the four 100g tests reported by Renzi et al. (1991), the caisson was allowed to equilibrate for only 70 minutes following suction penetration. According to measurements, only 25% of the installation pore pressures had dissipated just prior to pullout. They observed that the incomplete dissipation reduced the capacity of the piles during the first pullout, but subsequent reconsolidation and pullout events resulted in higher capacities. Because no pullout test was conducted on a caisson that had fully equilibrated following penetration, the effect of incomplete dissipation on pullout capacity could not be quantified.

Clukey and Morrison (1993) reviewed single cell and multi-cell monotonic pullout tests conducted at fast (1.3 kN/sec) and slow (0.3 kN/sec) pullout rates and observed that the fast rate tests had an average suction efficiency of 80.5% and the slow rate tests averaged 60.8%. Note that these average results make no distinction between the single and multi-cell tests. For single cell tests, the average suction efficiency for two rapid tests was 85.4% and the suction efficiency for the one slow test was 66.0%. The decrease in
efficiency for the slower tests is due to the decrease in sustainable suction caused by an increase in pore pressure diffusion time.

2.3.3 Field Scale Model Tests

Four significant field programs on model caissons have been reported, all involving sites in Northern Europe (Andersen et al., 1992, 1993; Andréasson et al., 1988; Dyvik et al., 1993; Hogervorst, 1980; Tjelta et al., 1986). The field test described by Tjelta et al. (1986) was conducted near the proposed foundation site for the Gullfaks C GBS platform and used the same embedment dimensions as the prototype structure\[^{10}\]. The remaining three programs were conducted on model caissons at onshore locations with soil strengths similar to those expected for the prototype foundations. Table 2.4 lists the model geometry, soil conditions, and test variables used for the four field programs.

At three onshore locations in the Netherlands, the first reported field tests of suction caissons were conducted to ascertain installation characteristics and lateral and axial load capacities (Hogervorst, 1980). A single element caisson with a diameter of $D=3.8$ m was penetrated with suction to a depth of $L=7$ m at two primarily sandy sites ($L/D=2.6$) and to a depth of $L=5$ m at a site with overconsolidated clay ($L/D=1.3$). Figure 2.18 shows a schematic drawing of the suction caisson and the three soil profiles. After installation, the pile was allowed to equilibrate for an unreported period of time. Following this, the pile was pulled out either vertically or laterally, but the rate of withdrawal was not reported.

To predict the response of the proposed Gullfaks C fixed gravity platform, a large scale field test was conducted in 1985 in the Gullfaks Field of the North Sea (Tjelta et al., 1986). The test unit was composed of two steel cylinders linked with a rectangular concrete panel, as shown in Figure 2.19. The cylinders each had an outside diameter of $D_o=6.5$ m, a height of $H=23$ m, and a wall thickness of $t_w=35$ mm ($D_o/t_w=185$). The

\[^{10}\] However, the model caisson had a reduced diameter.
concrete panel had a height of $H=23$ m, a width of $w=2.4$ m, and a thickness of $t_w=0.4$ m. This panel simulated the skirt section of the Gullfaks C foundation and extended 3 m ahead of the two cylinders. As shown in Figure 2.19, soil at the Gullfaks Field test site consisted of normally consolidated clay and loose clayey and silty sands. For the two tests conducted, the unit was penetrated approximately 20 m into the soil by a combination of dead weight and applied suction beneath the cylinders. Temporary interruptions were incorporated during penetration to study relaxation and set-up effects and water jetting at the tips was used to enhance penetration. For the first test, the suction beneath the pile cap was released immediately and the unit was allowed to equilibrate for 8 hours. Then, the unit was subjected to a vertical load sequence consisting of three phases: compression, tension, and two-way strain-controlled cycling. Figure 2.20 shows the loading, displacement, and measured tip resistance versus time. In test 2, the suction beneath the cap was not released immediately, but was allowed to dissipate for 18 hours following penetration. Loading consisted of three sequential phases of one-way stress-controlled cycling with a period of $T=1$ min, as shown in Figure 2.20.

Andréasson et al. (1988) report the results of five model tests at a site just north of Gothenburg, Sweden. The test program was designed to model the behavior of a gravity base subject to the overturning moment introduced by sea loads. As shown in Figure 2.21, the model was composed of seven steel cylinders, each with a diameter of $D=0.6$ m, a height of $H=0.6$ m, and a wall thickness of $t_w=4-5$ mm ($D/t_w=133$). One center cylinder was surrounded by the other six, and they were rigidly interconnected. The lateral load was applied to the top of a tower structure rising above the 7 cells. The test site consisted of a homogeneous, highly plastic, sensitive, lightly overconsolidated clay. In each of the five tests, the model was penetrated using self-weight and suction to a depth of approximately $L=0.5$ m ($L/D=0.8$), although in one test the model was penetrated to a depth of $L=0.3$ m ($L/D=0.5$). Following penetration the model was allowed to equilibrate for 8 weeks. The night before lateral loading, a vertical static load was applied. Then the
lateral load was applied either statically or cyclically. Figure 2.21 shows a summary of the
loading program. The first test was simply static and the second test was cyclic. For tests
3, 4, and S85, a cyclic load was applied for several hours and then a post-cyclic static load
was applied until failure. All cycling was one-way stress-controlled, with a period of
T=12 to 25 seconds.

To check the validity of design procedures for the tension leg platform foundation
at the Snorre field in the North Sea, four model tests were conducted in Lysaker clay at a
site on the outskirts of Oslo, Norway (Dyvik et al., 1993; Andersen et al., 1992,1993).
The test model comprised four steel cylindrical cells, which formed a square array and
enclosed an inner core, as shown in Figure 2.22. Each cell had an outside diameter of
D_o=0.91 m, a height of H=0.9 m, and a wall thickness of t_w=22.5 mm to give a diameter
to wall thickness ratio of D_o/t_w=40.6. The soil at the site was a sensitive, soft, uniform
clay. For each test, the model penetrated almost entirely by self weight to a depth, L=0.82
m, to give a length to diameter ratio of L/D_o=0.9. Suction was applied only to maintain
verticality during penetration. After penetration, the cell compartments were vented and a
tensile load equal to the model submerged weight was applied for 8 days. For all four
tests, a tensile load was applied to the top of the model at an angle of 10° from the
vertical. In one test this tensile load was applied to pull the model out at a constant rate of
displacement and thus established a reference static strength. In the remaining three tests,
the tensile load was applied in a one-way, stress-controlled fashion with various
amplitudes and a period of T=10 sec.

Field Tests: Installation

Although Hogervorst (1980) did not measure the penetration resistance of the
single cell suction caisson tests in the Netherlands, he did observe that suction allowed
quick and effective penetration. In sandy soils, he noted that the soil inside the caisson
liquefied, thus eliminating internal skin friction. In clay, internal friction reducers (wider
caisson rim than wall thickness) practically eliminated internal friction. He postulated that external skin friction and tip resistance were reduced by groundwater flow or reduced pore pressure as a result of the underbase suction. From these observations, Hogervorst concluded that penetration resistance to suction installation could be predicted by the following equation:

\[ F_p = 2\pi D k_f L f + \pi D t_w k_p q_c \]  \hspace{1cm} (2.20)

where \( F_p \) = penetration force
\( D \) = caisson diameter
\( L \) = penetration depth
\( t_w \) = caisson wall thickness
\( q_c \) = average cone resistance
\( k_f \) = skin friction coefficient
\( k_p \) = tip resistance coefficient
\( f \) = penetrometer friction

Tjelta et al. (1986) used an empirical method based on cone measurements to predict the tip penetration resistance of the twin cell suction caisson/panel test at the Gullfaks field in the North Sea. Figure 2.23 shows the measured and predicted model tip penetration resistance, as well as the in-situ cone penetrometer results for the Gullfaks C site. Note that the empirical method (also described in Figure 2.23) does not include the effect of suction. Tjelta et al. observed that in the sand layers the tip resistance was 0.2 to 0.6 times less than the cone resistance, a difference that may be attributed to the tip water jetting. Variations in the penetration rate and temporary delays in the penetration had no effect on the tip resistance. No wall friction predictions were reported, but it was observed that inside wall friction in sand was reduced due to suction. The authors concluded that wall friction in clay was unaffected by suction.

For the 7-cell model test in soft clay at Gothenburg, Sweden, the penetration resistance was predicted by assuming the resistance was due to tip resistance and wall friction according to the following simple equation:
\[ F_p = A_s \alpha c_u + A_b N_c c_u \]  \hspace{1cm} (2.21)

where, \( F_p \) = penetration force
\( A_s, A_b \) = exterior and interior wall area and pile base area
\( \alpha \) = adhesion factor
\( c_u \) = undrained shear strength
\( N_c \) = bearing factor (\( N_c = 7 \); based on low embedment)

Figure 2.24 shows a comparison of the predicted and measured penetration resistance for three of the four model tests. For the two tests conducted at a penetration rate of 2 to 3 cm/min, the prediction model compares well to the measured values assuming an adhesion factor of \( \alpha = 0.2 \). For the slower penetration rate test (0.3 cm/min), the measured resistance is higher, and a good comparison is obtained using an adhesion of \( \alpha = 0.4 \).

**Field Tests: Load Capacity Prediction**

Of the four field test programs reviewed, three compared the measured load capacity with predicted values. For the single cell tests in the Netherlands, Hogervorst (1980) assumed the vertical pullout capacity was composed of the submerged pile weight and the outer wall friction, as the following equation illustrates:

\[ F_b = W'_a + \pi D L f \]  \hspace{1cm} (2.22)

where, \( F_b \) = breakout force
\( W'_a \) = submerged pile weight
\( D \) = pile diameter
\( L \) = pile penetration
\( f \) = unit skin friction

Note that this equation assumes no internal wall friction because the piles were fitted with internal friction reducers. Based on the results of nine tests, the predicted capacity was 75 to 92% of the measured value. It is possible that the additional capacity could have been due to passive underbase suction, but no observations regarding suction during pullout were reported.
Andréasson et al. (1988) predicted the static capacity of the 7-cell model tested in Sweden. In each of the four tests conducted, a compressive vertical load was maintained on the model to simulate dead weight load. The lateral load was applied either statically or cyclically. The result of these forces was an overturning moment on the model. From observations of failure surfaces, it was assumed that the failure mode was pure rotation of the model about a point 0.15 to 0.2 m above the skirt tip. To predict the ultimate lateral load, Andréasson et al. assumed a failure surface in the shape of a spherical segment that touched the skirt edges and reached a depth of approximately 0.5 to 0.7 m below the model tip. For such a spherical segment rotating in the soil, the resistance is provided by a moment capacity and moment frictional resistance according to the following equation:

\[ F_{\text{blat}} = \frac{1}{X}(0.5\beta \pi R^3 c_u + 4 L r^2 \alpha c_u) \]  

(2.23)

where, \( F_{\text{blat}} \) = lateral breakout force  
\( X \) = lever arm distance between lateral load application and center of rotation  
\( \beta \) = opening angle of spherical segment  
\( R \) = radius of spherical segment  
\( c_u \) = undrained shear strength  
\( L \) = penetration depth  
\( r \) = radius of foundation (assuming circular idealization)  
\( \alpha \) = adhesion factor (=1 for static loading)

Based on one test, this prediction model overpredicted the measured static capacity by only 4%. A finite element analysis (AXIPLN) that idealized the 7 cell model as a plane strain footing and used a hyperbolic stress strain material model for the soil overpredicted the capacity by 11%.

To predict the static pullout capacity of the multi-cell tension leg foundation model tested in Norway, Andersen et al. (1993) employed a limiting equilibrium analysis. In this method, a computer program is used to search for a failure surface that gives the lowest bearing capacity. The critical failure surface depends on many factors such as load inclination, shear strength profile, and embedment depth to diameter ratio (L/D) of the
model. Triaxial compression, triaxial extension, and direct simple shear tests were conducted on undisturbed samples of the test clay to provide the proper shear strength at a particular point on the proposed failure surface. Strain compatibility was assumed for soil elements along the potential failure surface. For the one static test conducted, this method gave nearly perfect agreement with the measured capacity (a difference of less than 1%). Figure 2.25 depicts the predicted and observed failure surfaces for this test.

Field Tests: Effect of Cyclic Loading

In three of the four field test programs, cyclic loads were applied to the model to simulate wave loading that would be encountered in ocean environments. For the twin cell/panel model tests in the North Sea reported by Tjelta et al. (1986), the cyclic failure load was 50% of the penetration resistance. Predictions were not attempted in this study.

Based on measured results of one static and three one-way, stress-controlled cyclic tests on the 5-cell TLP model in Lysaker clay in Norway, Dyvik et al. (1993) made the following observations regarding cyclic effects:

1) cyclic load capacity was less than static load capacity
2) larger number of cycles at a given load produced failure at a lower cyclic load
3) primary failure mode was small cyclic and large permanent displacements
4) eccentric loading produced lower cyclic failure loads and larger displacements
5) cyclic stiffness decreases with increasing cyclic load level
6) cyclic stiffness decreases with number of cycles at given cyclic load level
7) cyclic stiffness at small cyclic load levels is reduced by previous cycling at higher cyclic load levels
8) difference in geometries tested had no significant effect on cyclic displacements

Using the limiting equilibrium technique that was used for the static test, Andersen et al. (1992) predicted the capacity of the 5-cell tension leg foundation model in Norway that was subjected to one-way, stress-controlled cycling. To obtain the proper strength profile for a model that was subjected to cycling, they conducted cyclic triaxial compression, triaxial extension, and direct simple shear tests on undisturbed samples of the clay from the
test site. They assume the combination of average and cyclic shear strains are the same along the potential failure surface (strain compatibility) and the average shear stresses along the failure surface are in equilibrium with the average loads. This model overpredicted the cyclic capacity by less than 6%. Figure 2.25 shows the predicted and observed failure surface for one of these cyclic tests. Andersen et al. warn that several uncertainties exist in their prediction: 1) cyclic shear strengths contain some uncertainty due to limited number of lab tests, 2) lack of a precise definition of failure in both laboratory strength and model tests may alter the predictions, 3) the assumed constant shear strain along the entire failure surface may not be constant, 4) the load inclination decreases with increasing overturning displacement, which is not accounted for in the cyclic model, 5) tension cracks (which are not included in the model) can reduce capacity (cracks only appeared in the tests after very large displacement).

Andréasson et al. (1988) also did not predict the cyclic capacity of the 7-cell model in Sweden. However, from observations of the three cyclic tests conducted on the model, they concluded that cyclic failure develops at 70 to 80% of the static capacity. Andréasson et al. also noticed the same three patterns of cyclic stiffness degradation that was observed by Dyvik et al. (1993) for the 5-cell TLP model (see observations 5, 6, and 7 above). Post-cyclic static tests indicate that cycling disturbance reduces the static strength by 6 to 12%.

Field Tests: Foundation Movement Prediction

Finite element analyses were used in both the 7-cell model tests in Sweden and the 5-cell model tests in Norway to predict the movements of the model under static or cyclic loads.

To predict foundation movement under cyclic loading, Andréasson et al. (1988) used the same finite element model (AXIPLN) that was used to predict capacity. To analyze the results, the overturning moment was compared to the rotational spring
stiffness, which is defined as the overturning moment relative to the center of rotation divided by the rotation of the model. Figure 2.26 shows the measured and predicted values of spring stiffness versus overturning moment. Each point on the curve represents the stiffness during the first few load cycles at each new overturning moment level. The vertical force for test 2 was higher than for tests 3 and 4, which may explain the higher stiffness for a given overturning moment. The overall trend of decreasing stiffness for increasing overturning moment is clear. The predicted curves also show this trend, but the upper curve represents a test with a lower vertical force, which is contrary to the measured results. The reason for this discrepancy is unclear.

Andersen et al. (1993) predicted the deformation of the tension leg foundation model subject to both static and cyclic loads by using a finite element model called INFIDEL. The predictions were considered class 'A' predictions because they were completed and delivered to the client prior to performing the first model test. INFIDEL models non-linear stress path dependent stress-strain soil behavior and can handle circular and elliptic geometries with axisymmetric or non-axisymmetric loading. Figure 2.27 shows the load-displacement curves for both the measured and predicted values of the static and cyclic tests. In general the predicted rotational, horizontal, and vertical model displacements agree very well with the measured values.

**Field Tests: Effect of Soil and Geometry on Loading**

Single cell caisson pullout tests at the two sandy sites in the Netherlands allow a brief commentary on the effect of soil on loading (Hogervorst, 1980). At the first of these sites (Figure 2.18), the Scharendijke site, stiff sandy clay underlies the upper 5.5 m of sand, while at the second site (Noordland), the profile consists of sand throughout. At both sites, the caisson was penetrated to the same depth (L=7 m). The average pullout capacity of the caisson at the Scharendijke site was 68% higher than that at the Noordland
site. The increase in capacity can be attributed directly to the greater frictional resistance and reverse bearing from the stiff clay layer from 5 to 7m.

The effect of geometry was studied in the Sweden (Andréasson et al., 1986) and Norway (Andersen et al., 1993) programs. For the 7-cell model in Sweden, the embedment depth was varied for two tests subjected to the same cyclic and post-cyclic loading sequence. For the model with the lower embedment depth to diameter ratio ($L/D=0.5$), the measured ultimate horizontal load under cycling loading was only 4% lower than that for the model with the higher embedment depth ($L/D=0.8$). In addition, the ultimate horizontal load measured during the post-cyclic static test was 6% lower for the lower embedment test.

The geometric effect was studied in a different way for the 5-cell model tests in Norway. In terms of the magnitude of load, the three cyclic tests had similar loading schedules. However, as shown in Figure 2.22, the load was applied in different directions. All three tests had a load inclination of $10^\circ$ from the vertical, but in test 3 the load was directed along the diagonal of the model square base plan, while in tests 2 and 4, the load was applied parallel to one side of model base plan. In tests 2 and 3, the point of load application was from the model center, but in test 4 the point of load application was 0.14 m from the center parallel to the side of the square base plan, as shown in Figure 2.22. Of the three tests, test 2 had the highest capacity. Test 3 had a 12% capacity reduction due to the different load direction. In test 4, which had the most eccentric load, the capacity was reduced by 20%.

2.4 FIELD APPLICATIONS

This section describes the following four significant field applications of the suction caisson foundation that have been published to date: Gorm field mooring buoy
support, Gullfaks C gravity based platform foundation, and Snorre and Heidrun tension leg platform structures. All four foundations are used to support offshore oil production facilities in the North Sea. The TLP structures relate directly to the tensile loading of model suction caissons, which is the subject of this thesis. The Gorm buoy support represents the first practical use of the suction caisson concept, while the Gullfaks C GBS is described mainly because it was the first very large offshore structure that employed underbase suction installation technology\textsuperscript{11}. Table 2.5 lists the important physical characteristics of each suction caisson application.

2.4.1 Gorm Field Mooring Buoy Support

In 1981, twelve suction piles were installed in the seabed in the Danish sector of the North Sea known as the Gorm field. They were designed to resist purely horizontal forces at mudline applied by a single buoy mooring, which is used by oil tankers to transfer production oil from the oil field. These piles were developed by Shell Internationale Petroleum Maatschappij B.V. (SIPM), who licensed Single Buoy Mooring, Inc to design the 12 piles. This was the first commercial application of the suction pile concept (Cuckson, 1981; Senpere and Auvergne, 1982). Table 2.5 highlights the characteristics of the Gorm suction piles.

Each pile is made of steel, weighs 25 tons in air (approximately 22 tons submerged), has a diameter of $D=3.5$ m, a wall thickness of $t_w=25$ mm, and a height of approximately $H=9$ m. The diameter to wall thickness ratio is $D/t_w=140$. A schematic diagram of the pile is shown in Figure 2.28. The Gorm field site, located in a water depth of approximately 40 m, consists primarily of sand underlain by soft and stiff clay. Figure 2.28 depicts the soil profile for the 12 suction pile sites. This profile can be divided into four distinct layers. Layer 1 consists of 3 m of loose to medium dense sand ($\phi=35^\circ$, $\gamma_b=10$ kN/m$^3$), which transitions into a 2 to 4 m thick layer of medium to dense

\textsuperscript{11}Unlike TLP structures, the Gullfaks C GBS imposes a compressive load on the caisson foundation.
sand ($\phi=40^\circ$, $\gamma_b=10$ kN/m$^3$). Layer 3 is composed of 0 to 2.8 m of soft clay ($c_u=20$ kPa, $\gamma_b=7$ kN/m$^3$), which becomes a stiff clay ($c_u=70$ kPa, $\gamma_b=9$ kN/m$^3$) that extends well below the bottom of the installed piles. Cone penetrometer tests yielded average sand resistance values of $q_c=30$ MPa with peaks of up to $q_c=36$ MPa.

Successful installation and operation of the suction piles was based on predictions of penetration resistance, ultimate lateral capacity, and behavior under working loads. Penetration resistance was predicted based on empirical methods developed by SIPM (Hogervorst, 1980) that account for reduced skin friction due to suction. The ultimate lateral load was assumed to equal the earth pressure mobilized against the pile plus the basal moment and shear. A factor of safety against collapse was assumed to be the calculated ultimate load divided by the maximum lateral load ($Q_{\text{max}}=200$ tons). Movement under a working load of $Q=130$ tons was calculated using $P-y$ curves.

Installation was quick, as the penetration time averaged 2 hours. Initially the pile was lowered by a barge-mounted crane into the water and allowed to penetrate the seabed from 0.5 to 2.6 m by self-weight. Two retrievable pumps mounted on the top of the pile then reduced the underbase water pressure, which created a pressure difference across the pile top. This pressure difference forced the pile to its final depth of $L=8.5$ to 9 m, which yielded a embedment depth to diameter ratio of $L/D=2.4-2.6$. The only problem encountered during suction penetration was the creation of a soil plug beneath the pile top. The presence of a soil plug can prevent further penetration to the required depth. For the first few piles, the plug was removed by dismantling the pumps and using an airlift system. Because this was time consuming, a jetting system was mounted on the remaining piles to liquefy and pump out any soil in the upper 1.5 m of the pile. A load test performed on three piles indicated a peak load of 230 tons with a maximum pile head deflection of 25 mm$^{12}$.

$^{12}$These load tests have not been documented.
2.4.2 Gullfaks C Gravity Based Platform Foundation

At the time it was installed in 1989, the Gullfaks C structure was the world's largest and heaviest offshore concrete structure, and it was the first fixed platform located at water depth greater than 200 m (actual depth 220 m) in the North Sea. To accommodate the massive load, the base design consisted of 16 large diameter circular suction caissons penetrating 22 m into the seabed. Active underbase suction during set-down was required to overcome the large penetration resistance. During operation the foundation would be subject to vertical compression load from the platform, a static lateral load from sea currents, and a cyclic lateral load from sea tides and waves. The Gullfaks C platform was designed and constructed by Norwegian Contractors for Statoil, which operates the Gullfaks oil field (Tjelta et al., 1988, 1990). See Table 2.5 for a summary of the important physical characteristics of the Gullfaks GBS.

As shown in Figure 2.29, the foundation is composed of 16 cylindrical concrete piles attached together. Submerged, the entire platform and foundation weighs approximately 500,000 tons. In plan view, 14 of these caissons are arranged in a hexagonal ring enclosing an area of nearly 17,000 m². At the center of the hexagon are two additional caissons. Each caisson has an outer diameter of $D_0=28$ m, a wall thickness of $t_w=0.4$ m, with a planned embedment depth, $L=22$ m. The resulting diameter to wall thickness ratio is $D_0/t_w=70$. The foundation soils comprise 28 m of interbedded layers of soft clays and complex clayey sands, which are underlain by contractant clay (see Figure 2.29). Two peculiar issues regarding the soil properties were important considerations in the design: 1) the clayey sand follows triaxial stress paths that initially resemble soft clay, but then show behavior typical of dilatant dense sands, and 2) the contractant clay layer below 28 m exhibits no dramatic strength or stiffness degradation when subject to large undrained strain controlled cyclic loads.

To help predict the resistance to skirt penetration, a field test was conducted at the site using a twin cell caisson with an attached concrete panel section (see section 2.3.3,
Tjelta et al., 1986). This test showed that penetration with suction could be predicted and achieved without the use of remedial actions, such as skirt tip water jetting. Using both limit equilibrium and finite element methods, the critical failure mode and foundation stability was evaluated extensively and the safety level for the Gullfaks C foundation was found to be satisfactory.

For such a massive structure, installation was achieved fairly rapidly. From skirt touchdown to complete penetration, the operation lasted 68 hours. After being lowered to the mudline by a ship-mounted crane, the caisson arrangement penetrated the soil to the design depth by a combination of self weight and underbase suction. A pumping system with intake valves located in each cell served three purposes: 1) evacuation of trapped cell water during penetration, 2) water ballasting for extra penetration force by depositing cell water into ballast chambers, and 3) creation of underbase suction by sluicing pressurized cell water to atmospheric pressure ballast cells. Platform verticality during the last meter of penetration was achieved by adjusting the pressure in various caisson cells. Final penetration was \( L=22 \, \text{m} \), which yielded a embedment depth to cell diameter ratio of \( L/D_0=0.8 \). For six days following penetration, grout was placed in the voids between the platform base and the sea floor. A soil drain system with intakes located along the cell interior walls was designed to accelerate the dissipation of installation pore pressures in order to: 1) strengthen the soil below the penetrated skirt tips, 2) quickly transfer base contact stress to skirt walls and tip, and 3) speed up consolidation settlements to allow earlier well-drilling and pipeline tie-ins.

Measurements of caisson loads and displacements and soil pore pressures during and following installation permitted several observations that are invaluable for future design. The predicted and measured penetration resistance vs. depth are shown in Figure 2.30. The predicted resistance was based on field testing at the site four years prior to installation (see section 2.3.3). From this plot it is clear that the soil resistance in zones between 10 and 21 m generally was overestimated (from 17 to 18 m, the penetration
resistance was slightly underestimated). The effects of natural equilibration and the soil drain system can be seen in Figure 2.31, which depicts excess porewater pressure dissipation, platform settlement, and load transfer between base and skirts vs. time following installation. In 3 months, the soil drain system had reduced excess porewater pressure to values less than zero in the penetrated zone from 0 to 20 m. Just below the skirt tips, from 23 to 25 m, all installation pore pressures had dissipated, and the drainage had a negligible effect on the clay layer located 8 m below the skirt tips. Just after penetration, the base carried nearly all of the platform dead load, but within two months, this load had been transferred to the skirts. Lastly, after three months of active oil drainage, the platform had settled more than 0.5 m.

2.4.3 Snorre Tension Leg Platform Foundation

Using installation technology developed for the Gullfaks C foundation and other concrete gravity structures, the first suction caisson foundation to support a tension leg platform (TLP) was installed in 310 m of water in the Snorre oil field of the North Sea in 1991. The foundation comprises four concrete foundation templates (CFT), each with 3 large interconnected cylindrical caissons and a ballast chamber at the top. The CFTs were penetrated into the seabed by a combination of self-weight and active underbase suction. The floating platform applies a tensile load to the top of each CFT via pretensioned tethers. During normal operating conditions, the foundation is designed to resist the tensile load by the CFT submerged weight and ballast alone. However, additional resistance for storm loading is provided by skin friction along the foundation skirt walls and passive suction beneath the top of each CFT. The foundation was installed by Norwegian Contractors for Saga Petroleum ( Cottrill, 1991; Fines et al., 1991; Christophersen et al., 1992; Støve and Christophersen, 1992). Table 2.5 lists the notable aspects of the Snorre TLP.
The geometry of the foundation system is shown in Figure 2.32. In the plan view of the field layout, one CFT is located at each corner of a square that is 68 m on one side. Each concrete CFT weighs 3500 tons submerged, has a base area of nearly 720 m², a maximum width of almost 35 m, and a maximum height of 19.6 m. Each of the 3 individual "cells" has an outside diameter, \( D_o = 17 \) m, a wall thickness, \( t_w = 0.35 \) m, and a height, \( H = 19.6 \) m. The resulting diameter to wall thickness ratio is \( D_o/t_w = 49 \). A domed watertight roof separates each cell into an upper and lower section. The skirt walls on the lower part penetrate the seabed, while the upper part retains approximately 3500 tons of iron ore ballast. The CFTs were installed to a depth of between \( L = 11.6-12.5 \) m to give an embedment depth to diameter ratio of \( L/D_o = 0.68-0.74 \). Four pretensioned tethers connect each CFT to the floating platform above.

The Snorre TLP foundation is located at a depth of 310 m where the soil is composed of clays for the first 26 m, as shown in Figure 2.32. The upper 8 m of the profile is very soft to soft clay. From 8 to 17 m the soil is medium to stiff clay, which turns into very stiff clay from 17 to 26 m. Below this the profile changes to sand with interbedded clay layers. Figure 2.32 also depicts a characteristic shear strength profile that shows a strength of 10 kPa at mudline linearly increasing to 25 kPa at the bottom of the soft clay (8 m), after which the strength increases at a greater rate to 150 kPa at the bottom of the very stiff clay layer (26 m).

The combined weight of the structure and ballast of each CFT was designed to resist a tensile load of 6950 tons under normal operating conditions. Since ocean currents offset the floating platform, this tensile load is inclined from the vertical, which results in a horizontal load component on the CFT. The tether anchor point on the CFT is located 6 m above mudline so that the horizontal load will introduce an overturning moment. During storms, the skirt wall skin friction and suction mobilized beneath the dome will provide extra resistance. The CFT was designed to resist a "100 year" storm, during which the tensile load will vary cyclically from 0 to 14200 tons. The foundation stability was verified
extensively through field model tests (see section 2.3.3, Andersen et al., 1993) and through design procedures developed originally for the Gullfaks C project.

Installation of the CFTs proceeded smoothly. Each CFT was lowered 2 m into the seabed by a barge-mounted crane. The load encountered during this initial penetration was partially taken by cell interior water pressure and crane lift to avoid fracturing the soil. After 6 hours of this initial penetration, the crane load was released and self-weight penetration continued to about 10 m, while the structure verticality was maintained by controlling the cell interior pressure. The additional driving force needed to penetrate from 10 to 12 m was provided by creating suction under the domes. At this depth, the pump valves were closed to prevent additional movements.

Measurements recorded during penetration show that the record of penetration resistance versus depth was very consistent among the four CFTs, as shown in Figure 2.33. By contrast, the cone penetrometer resistance profiles show a much wider scatter among the different locations. The scatter in the cone data could be due in part to local variations not reflected by the much larger CFTs or to instrument uncertainty. The skin friction during CFT penetration was backcalculated and yielded an adhesion value, $\alpha=0.2$, which conforms with observations at similar clay sites. In the softer, more sensitive clay from 0 to 8 m, the adhesion value was $\alpha=0.15$, and in the less sensitive clay below 8 m, the factor was $\alpha=0.35$.

A permanently installed instrumentation package on the Snorre foundation will continue to yield important data regarding CFT inclination, vertical displacements, and skirt chamber pressure. This information is useful not only to monitor the safety of the Snorre foundation, but also to develop more efficient, less conservative designs in the future.
2.4.4 Heidrun Tension Leg Platform Foundation

The Heidrun TLP suction foundation was installed in 1994 in a water depth of 345m in the Haltenbanken area of the North Sea. At the time of its completion (1995), the Heidrun platform was the largest TLP constructed to date. Like the Snorre design, the Heidrun foundation consists of four concrete units. However, each unit comprises 19 interconnected caisson cells with ballast chambers on each. As for the Snorre TLP, the foundation resists tensile loads applied by the TLP during normal operating conditions through foundation weight (including ballast) alone. Storm loading is resisted by caisson wall skin friction and passive suction beneath the top of each foundation unit. The Heidrun TLP foundation was installed by Norwegian Contractors for Conoco Norway, Inc. (Munkejord, 1995; Botros et al., 1996; Miller et al., 1996). Table 2.5 presents a summary of the important aspects of the Heidrun design. As for the Snorre foundation, the field layout consists of a foundation unit located at each corner of square plan area. Figure 2.34 shows the geometry of one concrete foundation unit. Weighing 12,500 tons submerged, each unit has a base area of approximately 1500 m², a footprint of 43m×48m, and a maximum height (to the top of the "tether porch") of 26.45 m. One foundation unit comprises 19 interconnected cells, each with an outside diameter $D_o=9$ m, height $H=6.9$ m, and capped with lower and upper spherical domes. Through a combination of ballasting and skirt compartment evacuation, the foundation unit penetrated the seafloor to a depth $L=4.8-5.0$ m ($L/D_o=0.53-0.56$). Due to problems encountered as the foundation was lowered to the seafloor, installation of the first unit required 179 hours. The remaining 3 units were installed in an average time of 60 hours.

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13Many important details of the Heidrun TLP foundation (geometry, soil profile, installation) have not been published at this time.
2.5 STATUS OF GEOTECHNICAL ISSUES ASSOCIATED WITH SUCTION CAISSON ANCHORS

This section summarizes the information about geotechnical problems inherent to suction caisson anchors that has been obtained from past laboratory studies and field applications (sections 2.2-2.4). The geotechnical issues can be classified according the phases in the life of a suction caisson anchor: installation, equilibration, and tensile loading (see section 2.1.2).

2.5.1 Installation

During the installation phase, the suction caisson penetrates the seabed by self-weight and underbase suction. Two major geotechnical uncertainties associated with this phase are penetration resistance prediction and the effects of underbase suction. Although there is limited research on the underlying factors involved in suction pile penetration resistance, prior research (Jones et al., 1994; Hogervorst, 1980; Renzi et al., 1991; Andréasson et al., 1988) acknowledges that resistance to penetration is provided by anchor skin friction and tip bearing, which do not explicitly account for the effect of underbase suction. As discussed later, underbase suction dramatically reduces the penetration resistance in sands, but has a small effect on clay resistance.

*Penetration Resistance During Installation*

Penetration resistance is comprised of anchor weight, skin friction, and tip capacity, as the following general equation shows:

\[ F_p = -W_a + F_{sf} + F_t \]  \hspace{1cm} (2.24)

where,  
- \( F_p \) = penetration resistance  
- \( W_a \) = buoyant anchor weight  
- \( F_{sf} \) = skin friction  
- \( F_t \) = tip bearing capacity
The only difference among the various researchers relates to how the skin friction and tip capacity terms \((F_{sf}, F_t)\) are defined, both of which depend on soil properties.

For penetration in sand, the skin friction was calculated as follows:

\[
F_{sf} = A_s \sigma' V K \tan(\delta) \tag{2.25}
\]

(1g tests at UT/Austin; Jones et al., 1994; section 2.3.1)

\[
F_{sf} = A_s k_f f \tag{2.26}
\]

(field tests in Netherlands; Hogervorst, 1980; section 2.3.3)

where, \(A_s\) = interior and exterior wall area
\(\sigma'_V\) = mean vertical effective stress
\(K\) = lateral earth pressure coefficient
\(\delta\) = soil/pile friction angle
\(k_f\) = empirical coefficient relating \(f\) to skin friction
\(f\) = local friction as measured by penetrometer

It was noted in both test programs that variations in the effective stress along the pile wall must be accounted for in the friction calculations (as discussed below, underbase suction greatly reduces effective stress in the soil plug). Tip resistance in sand can be calculated by multiplying the pile tip area by an estimated tip stress, as indicated by the following equation:

\[
F_t = A_b q_b \tag{2.27}
\]

For the 1g model tests at UT/Austin, Jones et al. (1994) estimate the tip stress using standard bearing capacity formulae (e.g., Vesic, 1973), whereas Hogervorst (1980) based tip stress on corrected cone penetrometer data for the field tests in the Netherlands.

Prediction of suction pile penetration in clays also involves skin friction and tip capacity. Skin friction is calculated by multiplying the inner and outer pile sidewall area by an average clay undrained strength and an adhesion factor:

\[
F_{sf} = A_s \alpha \sigma_u \tag{2.28}
\]
For the 100g centrifuge tests reviewed by Renzi et al. (1991) (see section 2.3.2), the average clay strength (obtained by cone penetrometer tests prior to pile penetration) and an adhesion factor of $\alpha=0.35$ to $0.40$ were used. For the 7-cell caisson field test in Sweden, Andréasson et al. (1988) (section 2.3.3) used a shear strength derived from triaxial, direct simple shear, and fall cone data and an adhesion factor $\alpha = 0.2$ to $0.4$. Tip resistance in clay was obtained by the following equations:

$$F_t = A_b(N_c c_u + \gamma L)$$
(100g centrifuge tests; Renzi et al., 1991)

$$F_t = A_b N_c c_u$$
(7-cell caisson field test in Sweden; Andréasson et al., 1988)

For tip resistance in the centrifuge tests, the clay shear strength was obtained from triaxial tests and the bearing capacity factor was assumed $N_c=9$, while for the 7-cell caisson field test, the shear strength was obtained through triaxial, direct simple shear, and fall cone tests and the capacity factor was assumed to be $N_c=7$. Note that, probably due to the shallow embedment of the 7-cell structure, Andréasson et al. (1988) chose a capacity factor that is lower than the standard assumption for pile tip capacity in clay ($N_c=9$), and they neglect the contribution of embedment\(^{14}\). Tip resistance in clay also was predicted by Tjelta et al. (1986) for the twin cell suction caisson/panel test in the Gullfaks field. Their prediction method, based on an average cone penetrometer records (see section 2.3.3; Figure 2.23) predicted well the actual tip resistance of the twin cell model.

Both Renzi et al. (1991) and Andréasson et al. (1988) (see Figure 2.24) used their capacity equations to predict with accuracy the suction pile penetration resistance in clay for centrifuge (100g) and field scale (7-cell caisson) models, respectively.

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\(^{14}\)Andréasson et al. (1988) do not provide an explanation for choosing $N_c=7$ and neglecting the embedment contribution to tip capacity, but it is probably related to the low embedment depth ($L/D=0.25$, where $D$=overall diameter of caisson structure).
The Effect of Underbase Suction During Installation

None of the above equations for predicting penetration resistance explicitly accounts for the effect of underbase suction. But several research projects have noticed that suction dramatically reduces the resistance to pile penetration in sandy soils. In the series of 1g acrylic model tests at UT/Austin (Pavlicek, 1992; Iskander et al., 1994; Jones et al., 1994; see section 2.3.1), the researchers observed that suction caused enough upward seepage in the soil plug to reduce the effective stress to zero in this zone (liquefaction), and thus facilitate penetration\textsuperscript{15}. Indeed, according to the skin friction equation (2.25) for these tests, a vertical effective stress reduction lowers skin friction. This was confirmed by comparing tests wherein the pile was pushed into the sand with tests where the pile was installed with suction; the results show that suction reduces the penetration resistance by 82-88\% (Pavlicek, 1992; Jones et al., 1994).

During field tests at sandy soil sites in the Netherlands, Hogervorst (1980) also noticed soil plug liquefaction which effectively eliminated internal wall friction. This observation was incorporated into installation procedures for the Gorm field bacy support anchors (see section 2.4.1), as the first 5-7 m of the soil profile consisted of loose to dense sand (final penetration was $L=8.5-9$ m). At the Gullfaks site, which was composed of interbedded sand and clay, Tjelta et al. (1986) observed that suction reduced the inside wall friction as the twin cell/panel model penetrated through sand layers.

One problem caused by underbase suction during pile penetration in sand is the formation of an excess soil plug, which is the soil inside the pile that is above the level of the adjacent soil outside the pile (see Figure 2.11). The excess soil plug is formed by wall-displaced material and soil drawn up by liquefaction (Pavlicek, 1992) and has been observed in laboratory tests (Pavlicek, 1992; Jones et al., 1994), field tests (Hogervorst, 1980), and during the Gorm anchor installations (Senpere and Auvergne, 1982). If left

\textsuperscript{15}Pore pressure measurements also indicated a small downward seepage gradient along the wall exterior, which slightly increases effective stress in this zone and thus adds resistance.
unaddressed, the excess soil plug will limit pile penetration depth. However, during the Gorm anchor installation, the problem was solved by using a jetting system to intermittently liquefy and pump out the excess plug until achieving the design penetration depth.

The effect of underbase suction on piles penetrating clay is much less conclusive. For the 100g centrifuge tests in clayey silt, Renzi et al. (1991) used an adhesion factor $\alpha=0.25-0.30$ to predict well the pile penetration resistance under self-weight (up to $L=10$ cm). A slightly higher adhesion factor ($\alpha=0.35-0.40$) was used to predict the resistance during suction penetration (from 10 to 17 cm). The higher adhesion factor could be due in part to the effect of suction. Field test observations did not indicate that underbase suction reduced penetration resistance in clay (Hogervorst, 1980; Tjelta et al., 1986; Andréasson et al., 1988;).

2.5.2 Equilibration

Once the anchor pile has penetrated to the desired depth, it is allowed to equilibrate with the surrounding soil. Important geotechnical issues during this stage include the prediction of the rate of pore pressure dissipation and the concurrent changes in soil stresses and properties around the suction caisson. None of the experimental studies discussed in previous sections (2.2-2.3) investigated this particular phase in the life of a suction caisson anchor. Most of the research allowed complete pore pressure dissipation prior to tensile loading, but some (Das, 1991; Renzi et al., 1991; Renzi and Maggioni, 1994) focused on the effect of incomplete dissipation on the tensile loading behavior. These will be discussed in the following section.

2.5.3 Tensile Loading

Following dissipation of installation pore pressures, tensile load is applied to the suction anchor. For TLP structures, the static tensile load from the platform and the
cyclic tensile load from normal (calm) sea waves and currents is resisted by the anchor buoyant weight and ballast (see section 2.1.2). However, storm waves can apply a cyclic tensile load to the anchor that exceeds the buoyant weight and ballast. The anchor must resist this increased load through passive suction created beneath the anchor cap and anchor wall skin friction. The primary goal of the experimental studies reviewed in sections 2.2-2.4 was to investigate the ultimate capacity and frictional resistance of soil subject to tensile loading from a suction anchor pile. This section summarizes the predictive equations that have been developed for soil resistance to suction pile loading. In addition, this section discusses the experimental work regarding the effect of sustained monotonic and cyclic loads, anchor movements during loading, the effect of installation disturbance on tensile capacity, and the effect of pullout rate.

**Ultimate Capacity During Uplift**

Suction anchor research on sand is limited mainly to the 1g acrylic model tests conducted at UT/Austin (Pavlicek, 1992; Iskander et al., 1993; Jones et al., 1994; see section 2.3.1). They concluded that uplift capacity is composed of buoyant anchor weight, buoyant soil plug weight, external skin friction, and passive suction mobilized at the pile tip. Based on these components, an equation similar to those based on force equilibrium for local shear failure of surface objects (equation 2.8; section 2.2.2) can be constructed:

\[
F_b = W'_a + W'_s + F_{esf} + F_{suc} \tag{2.31}
\]

where, 
- \(W'_a\) = buoyant anchor weight
- \(W'_s\) = buoyant soil plug weight
- \(F_{esf}\) = external skin friction
- \(F_{suc}\) = suction force at pile tip

During the pullout tests, which were partially-drained, pore pressure measurements indicated downward seepage along the external wall due to a differential pressure across
the caisson cap. Seepage induced negative excess pore pressures outside the wall, and this increased the effective stress along the exterior wall. External side shear calculations included the effect of seepage as follows:

\[ F_{esf} = (0.5\gamma'z^2 - \bar{u}z)C_eK\tan(\delta) \]  \hspace{1cm} (2.32)

where, \( C_e \) = pile circumference
\( K \) = lateral earth pressure coefficient
\( \delta \) = soil/pile friction angle
\( z \) = penetration depth
\( \gamma' \) = buoyant soil unit weight
\( \bar{u} \) = change in pore pressure on pile exterior

According to equations (2.31, 2.32), the ultimate capacity in sand cannot be predicted because the suction force at the pile tip (\( F_{suc} \)) and the change in pore pressure on the pile exterior (\( \bar{u} \)) are unknown. Calculating suction force based on pore pressure measurements and deducing the skin friction based on equation (2.31), Pavlicek (1992) found that the buoyant anchor and soil plug contributed 13 to 18\% of the capacity, the external skin friction contributed 26 to 44\%, and the suction 41 to 61\%. For the field tests conducted in the Netherlands, Hogervorst (1980) assumes no internal friction and no buoyant soil plug weight (due to friction reducers; see section 2.3.3) and therefore proposed that the vertical pullout resistance is composed of only external skin friction and buoyant anchor weight:

\[ F_b = W_a + F_{esf} \]  \hspace{1cm} (2.33)

where, \( F_{esf} = \pi D L f \)
\( f \) = local skin friction as measured by penetrometer

Note that this equation neglects the contribution of suction. A comparison of the pullout capacity calculated by this equation and the measured resistance indicates that at sandy soil sites, the calculated capacity is 75 to 92\% of the measured capacity, which suggests that some passive suction resistance may contribute to the overall capacity.
Most of the experimental research on suction anchors was conducted on clay and assumed that the failure mechanism during uplift is similar to that for a bearing capacity failure with the direction of movement reversed. Therefore, taking force equilibrium on the anchor and soil at failure, the ultimate capacity during tensile loading comprises the buoyant anchor and soil plug weight, pile external wall skin friction, and the 'reverse' tip bearing, as shown in the following general equations:

\[
F_b = W'_a + W'_s + F_{esf} + F_{reb} \quad (2.34a)
\]

\[
F_b = W'_a + F_{esf} + F_{reb} \quad (2.34b)
\]

where, \( W'_a = \) buoyant anchor weight
\( W'_s = \) buoyant soil plug weight
\( F_{esf} = \) external wall skin friction
\( F_{reb} = \) reverse end bearing

Note that equation (2.34b) does not include the soil plug weight; as discussed below, two researchers (Byrne and Finn, 1977; Clukey and Morrison, 1993) incorporate the buoyant soil plug weight into the reverse end bearing term. It is also important to note that the soil plug is assumed to travel upward with the anchor, and therefore, the plug weight contributes to the resistance, while the internal skin friction does not. Fuglsang and Steensen-Bach (1991) and Renzi et al. (1991) agree that the skin friction term can be calculated by multiplying an average undrained shear strength by the external wall area and an adhesion factor:

\[
F_{esf} = A_e \alpha c_{um} \quad (2.35)
\]

For their 1g and 100g tests in kaolin, Fuglsang and Steensen-Bach (1991) used an adhesion factor that ranged from \( \alpha = 0.25 \) to 0.30 based on non-suction pile pullout tests and a mean cohesive strength based on lab vane tests (Renzi et al., 1991, did not estimate adhesion factors). Clukey and Morrison (1993), for their centrifuge model tests on kaolin,
based their external skin friction on reduced cone penetrometer test shear strength (see section 2.3.2).

The main difference in the uplift prediction equations for clay is the definition of the reverse bearing term. The most general form of the this term, based on conventional bearing capacity (Vesic, 1973), was reported by Rapoport and Young (1985) as follows:

\[ F_{reb} = A_p(S_c c_u b N_c + S_\gamma D_0 N_\gamma - S_q \gamma L N_q) \]  

(2.36)

where, \( A_p = \) total anchor base area, \((\pi D^2)/4\)
\( S_c, S_\gamma, S_q = \) shape factors
\( N_c, N_\gamma, N_q = \) bearing capacity factors
\( c_u b = \) cohesive strength at pile base
\( \gamma' = \) buoyant unit weight of soil
\( D = \) outer pile diameter
\( L = \) embedment depth

In all model testing on clay, this equation is reduced to one or two terms:

\[ F_{reb} = A_p S_c c_u b N_c L_c \] (2.37)
(1g small skirted anchors; Byrne and Finn, 1977)
(100g tests; Clukey and Morrison, 1993)

\[ F_{reb} = A_p (c_u b N_c - \gamma L) \] (2.38)
(1g and 100g tests; Fuglsang and Steensen-Bach, 1991)

\[ F_{reb} = A_p (c_u b N_c - \gamma L) \] (2.39)
(100g tests; Renzi et al., 1991)

Byrne and Finn (1977) and Clukey and Morrison (1993) both assume that the overburden (\(\gamma L\)) is canceled by the effective weight of the soil plug (see equation 2.34b). They also incorporate shape and depth factors \( (S_c=1.3, L_c=1.09, Byrne and Finn, 1977; S_c=1.2, L_c=1.08, Clukey and Morrison, 1993) \). Fuglsang and Steensen-Bach (1991) and Renzi et al. (1991) include both overburden and soil plug weight terms (see equation 2.34a)\(^{16}\), but do not incorporate shape and depth factors.

\(^{16}\) Although equation 34a contains buoyant anchor, soil plug, and overburden terms, Fuglsang and Steensen-Bach report predictive pullout equations in terms of total weights. The two types of equations (buoyant and total) yield identical results.
Byrne and Finn (1977) assumed all uplift capacity to be due to reverse end bearing (they neglected external skin friction and buoyant anchor weight; see equation 2.34b), but predicted to within 6% the measured maximum force during 1g anchor pullout (see section 2.2.2). Combining equations 2.34b and 2.37, Clukey and Morrison (1993) were able to predict to within 20%, the measured pullout capacity of 100g model anchors (see section 2.3.2). Fuglsang and Steensen-Bach (1991) did not report comparisons between predicted and measured capacities, but they did backcalculate bearing capacity factors that ranged from $N_c=6.4$ (piles with $D_o=8$ cm) to $N_c=8.3$ ($D_o=6.5$ cm). Renzi et al. (1991) did not report predictions.

Of the field test programs, only two involved prediction of vertical uplift capacity. For three vertical pullout tests at a clayey site (see section 2.3.3), Hogervorst (1980) employed the same uplift equation (2.33) that he used for sandy sites. This equation yielded capacities that were 9 to 25% below the measured values (recall that this equation only accounts for external skin friction and buoyant anchor weight). For one static pullout test conducted on the 4-cell TLP model on Lysaker clay in Norway, Andersen et al. (1993) used a limiting equilibrium analysis that incorporated computer program searches for failure surfaces and laboratory strength testing, a method which predicted to within 1% (nearly perfect agreement) the measured capacity (see section 2.3.3, Figure 2.25).

Regarding uplift capacity of suction piles in sand, it is clear that underbase suction can contribute greatly to the overall capacity, but due to the limited research available, three major uncertainties remain: predicting the suction component $F_{suc}$, predicting the distribution of seepage force along the exterior wall (see equations 2.31, 2.32), and estimating the effect of pullout rate. In clay, much research has contributed to the predictive capabilities. However, as shown by equations (2.34-2.39), there still is uncertainty regarding the following issues: how to include soil plug weight and overburden soil weight terms in the reverse bearing capacity, what adhesion and cohesive strength values to use for external skin friction, and the proper shape and depth factors.
The Effect of Sustained Load

Data is scant regarding the effect of a sustained monotonic vertical tensile load that is less than the ultimate capacity on suction anchors. When a tensile load that is less than the undrained capacity is applied to a suction anchor, one of two events may occur: 1) the load is low enough to allow the anchor to stabilize as the cap suction load transfers to the external wall, or 2) the load is too high to allow a complete load transfer from cap suction to external wall, and the anchor fails. None of the experimental programs examined monotonic sustained loading in detail\(^\text{17}\). For an anchor model that does not rely on wall friction for resistance (L/D=0.25), Byrne and Finn (1977) suggested a method for predicting the time required until failure for an anchor subject to a tensile load that is less than the undrained capacity (see section 2.2.2). They estimate the reduction in breakout load to be proportional to the average degree of pore pressure dissipation according to the following equation:

\[
\frac{F}{F_{b\text{max}}} = 1-U 
\]  
(see 2.11)

where, \( F = \) sustained load  
\( F_{b\text{max}} = \) undrained capacity  
\( U = \) average degree of consolidation

Using the relationship between the average degree of consolidation and a time factor, they found a good comparison between the theoretical and observed breakout reduction.

The Effect of Cyclic Loading

Very few cyclic loading tests were conducted on 1g and multi-g suction anchor models, but 3 of the 4 field test models involved cyclic loads. From the 1g acrylic model tests in sand, Jones et al. (1994) observed that when the model anchor was subject to small-amplitude stress-controlled cycling (4 Hz) about an average tensile load below the

\(^{17}\text{Renzi et al. (1991) investigated the effect of sustained load on anchors in silty clay wherein the installation pore pressures were only 50-60\% dissipated.}\)
ultimate drained capacity, the pile was not able to sustain any suction, which led to gradual pile withdrawal. Jones et al. attributed this to the high hydraulic conductivity of the sand and the small size of the model pile ($D_0=12.7$ cm). For one 100g model test in clay, Renzi et al. (1991) observed very little movement (less than 0.1 mm for a pile with $D_0=15$ cm) when the pile was subjected to small-amplitude stress-controlled cycling (0.125 Hz) about an average tensile load below the post-cyclic static capacity. Due to the different load conditions and history for these two cyclic tests, a full comparison cannot be made.

In their review of 100g cyclic model tests in kaolin, Clukey et al. (1995) found that caisson resistance depends on the combination of average (static) load, cyclic amplitude, and load inclination. Fewer number of cycles are required to fail the caisson as the load levels and load inclination increases. Post-cyclic static uplift capacity is greater than calculated static capacity, which can be attributed to consolidation effects around the caisson during cyclic loading.

Each of the three cyclic field tests incorporated a unique cyclic loading program (Tjelta et al., 1986; Andréasson et al., 1988; Dyvik et al., 1993; Andersen et al., 1993; see section 2.3.3), but some general comments can be made. In two of the programs (7-cell model in Sweden, 5-cell model in Norway) the cyclic load capacity was smaller than the static capacity. In addition, cyclic stiffness decreases with increasing cyclic load level and with the number of cycles at a given cyclic load level. Cyclic stiffness at small cyclic load levels is reduced by previous cycling at higher cyclic load levels. Post-cyclic static tests, which were conducted for the 7-cell model, indicated that cycling history reduces the static strength by 6 to 12%.

---

18 In the third field program (twin-cell panel at Gullfaks field; Tjelta et al., 1986) no static tensile capacity test was performed, but the cyclic capacity was 50% of the penetration resistance.
Anchor Movements During Monotonic Pullout

Based on the 1g and centrifuge model tests that reported load-displacement behavior, both anchors in sand (Pavlicek, 1992, Figure 2.13; Jones et al., 1993) and anchors in clay (Fuglsang and Steensen-Bach, 1991, Figure 2.12) reach ultimate capacity at significant displacements of between 20 to 30% of the pile diameter. However, during post-peak pullout, anchors in sand rapidly lose much of their capacity, while those in clay retain a significant residual load.

For the two field programs that included a static pullout test, the load was applied eccentrically, which caused the model to move both vertically and laterally. Results from the test conducted on the 5-cell model in Lysaker clay (Andersen et al., 1993) indicate that at failure\(^{19}\), the model had moved approximately 4 cm in both the vertical and lateral directions, which is approximately 4.4% of the cell diameter and 2.2% of the overall model width (see Figure 2.27).

The Effect of Installation Disturbance

Only the 1g acrylic model tests in sand conducted at UT/Austin looked specifically at the effects of installation disturbance upon the subsequent pullout behavior (Pavlicek, 1992). As described in section 2.3.1, increasing the amount of suction during installation liquefies and draws into the pile a greater amount of sand (as evidenced by the greater excess soil plug). This loosens the sand outside the pile, thus decreasing the vertical stress, which reduces the pile wall skin friction resistance to pullout (see equations 2.31-2.32).

\(^{19}\)The definition of failure for this test program was 0.04 to 0.05 radians of rotation (Andersen et al., 1993).
The Effect of Pullout Rate

In three of four of the experimental programs that investigated rate effects, the ultimate capacity increased with increasing pullout rate\textsuperscript{20}. For tests on small anchors in silty clay, Byrne and Finn (1977) noticed a seven-fold increase in maximum uplift force for one log cycle increase in strain rate. Singh et al. (1994) noticed a smaller rate effect (15 to 36\% capacity increase per log cycle increase in strain rate) for pullout tests in remolded clay. Slightly different embedment depths prevented direct comparisons between different pullout rates for 100\textsuperscript{g} tests on kaolin reviewed by Clukey and Morrison (1993), but they did notice an increase in suction efficiency with increasing pullout rate [suction efficiency = (measured $F_b$ - $F_{est}$)/(calculated $F_{reb}$)]. Both Singh et al. (1994) and Clukey and Morrison (1993) claim that the strength increase is due to an increase in the suction component of resistance, which can be attributed to lower pore pressure diffusion. However, no generalizations can be made regarding the magnitude of increase due to the different anchor geometries and soils used in these test programs.

\textsuperscript{20}For 1\textsuperscript{g} and 40\textsuperscript{g} tests on kaolin, Fuglsang and Steensen-Bach (1991) did not observe a rate effect.
<table>
<thead>
<tr>
<th>Researcher</th>
<th>Model</th>
<th>Geometry</th>
<th>Soil</th>
<th>Test Variables</th>
</tr>
</thead>
<tbody>
<tr>
<td>Goodman et al., 1961</td>
<td>inverted cups (active suction)</td>
<td>$D_o=3.6''$ $t_w=0.25''$</td>
<td>sand</td>
<td>suction level L/D fluid viscosity</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$L=0.25-2.0''$ $L/D_o=0.07-0.6$</td>
<td>silt clay</td>
<td></td>
</tr>
<tr>
<td>Muga, 1967</td>
<td>solid objects in field/lab</td>
<td>cubes, prisms, spheres, cylinders, cones, ellipsoids (least dimension ranged: 2&quot;-100&quot;)</td>
<td>clay</td>
<td>pullout rate L</td>
</tr>
<tr>
<td>Lee, 1972, 1973</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Brown and Nacci, 1971</td>
<td>skirted anchor (active suction)</td>
<td>$D_o=10''$ $t_w=0.75''$</td>
<td>sand</td>
<td>suction level D_t</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$L=1.82''$ $L/D_o=0.18$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Finn and Byrne, 1972</td>
<td>skirted anchor</td>
<td>$D_o=25.4$mm $t_w=?$</td>
<td>silty clay (undisturbed block)</td>
<td>pullout rate</td>
</tr>
<tr>
<td>Byrne and Finn, 1977</td>
<td></td>
<td>$L=6.4$mm $L/D_o=0.25$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wang et al., 1975, 1977, 1978</td>
<td>skirted anchor (active suction)</td>
<td>$D_o=4.4-13.3''$ $t_w=0.125-0.25''$</td>
<td>sand</td>
<td>$D_t$ clay strength suction level L/D</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$L=0.073-0.5''$</td>
<td>silt clay</td>
<td></td>
</tr>
<tr>
<td>Helfrich et al., 1976</td>
<td>skirted anchor (active suction)</td>
<td>$D_o=40$cm $t_w=?$</td>
<td>sand</td>
<td>suction level</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$L=26$cm $L/D_o=0.65$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Das, 1991</td>
<td>closed-ended cylinder</td>
<td>$D=8.3$cm $L=8.3-8.8$cm</td>
<td>3 clays</td>
<td>clay type, sustained load, consolidation level</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$L/D=1-1.1$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 2.1  Experimental Research on Pullout of Objects Resting on Soil
<table>
<thead>
<tr>
<th>Researcher</th>
<th>Test Program</th>
<th>Model</th>
<th>Soil</th>
<th>Test Variables</th>
</tr>
</thead>
</table>
| Larsen, 1989        | 15 tests suction penetration horizontal loading (static/cyclic) | steel  
D=10.4, 20.4, 30.5cm  
t_w=1.5mm  
L=45cm  
L/D=1.5, 2.2, 4.3  
D/t_w=69, 136, 203 | sand  
natural clay*  
kaolin* | loading  
L/D |
| Fuglsang and Steensen-Bach, 1991 | 15 tests push penetration vertical loading | D=6.5, 8cm  
L/D=2 | kaolin* | pullout rate clay strength |
| Pavlicek, 1992      | 8 tests suction penetration vertical static load | acrylic  
D_o=10.9cm  
t_w=0.38cm  
L/D_o=0.5-0.6  
D_o/t_w=29 | sand (SP) | installation disturbance |
| Jones et al., 1994  | suction penetration vertical loading (static/cyclic) | acrylic  
D_o=12.7cm  
t_w=1.6cm  
L/D_o=0.8-1  
D_o/t_w=8 | sand (SP) | loading |
| Singh et al., 1994  | 8 tests suction penetration vertical static load | D=4.4, 6, 7cm  
t_w=0.3cm  
L/D=1.8-2.8  
D/t_w=14.5-23 | natural clay*  
sand | soil pullout rate L/D |

* unspecified stress history

Table 2.2  Experimental Research on Suction Caissons: 1g Laboratory Scale Tests
<table>
<thead>
<tr>
<th>Researcher</th>
<th>Test Program</th>
<th>Model*</th>
<th>Soil</th>
<th>Test Variables</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fuglsang and Steensen-Bach, 1991</td>
<td>4 tests 40g push penetration vertical static load</td>
<td>D=6.5, 8cm (2.6,3.2m) L/D=2</td>
<td>kaolin (unspecified stress history)</td>
<td>pullout rate clay strength D</td>
</tr>
<tr>
<td>Renzi et al., 1991</td>
<td>4 tests 100g suction penetration vertical loading (static/cyclic)</td>
<td>aluminum D_o=15cm (15m) t_w=0.4cm L/D_o=1-1.2 D_o/t_w=37.5</td>
<td>silty clay (resedimented and consolidated with gradient)</td>
<td>loading L/D_o</td>
</tr>
<tr>
<td>Renzi and Maggioni, 1994</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clukey and Morrison, 1993</td>
<td>5 tests 100g suction penetration vert. static load vert. &amp; eccentric cyclic load</td>
<td>single cell caisson D_o=15.2cm (15.2m) t_w=0.06cm (6cm) L=31.4-33.2cm L/D_o=2.1-2.2 D_o/t_w=253</td>
<td>Speswhite kaolin (layered to obtain strength gradient)</td>
<td>pullout rate #cells loading</td>
</tr>
<tr>
<td>Clukey et al., 1995</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Prototype dimensions in parentheses

Table 2.3 Experimental Research on Suction Caissons: Centrifuge Tests
<table>
<thead>
<tr>
<th>Researcher (Field Location)</th>
<th>Test Program</th>
<th>Model</th>
<th>Soil</th>
<th>Test Variables</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hogervorst, 1980 (Netherlands)</td>
<td>9 tests suction penetration vertical loading</td>
<td>single cell D=3.8m $t_w=?$ L/D=1.3-2.6</td>
<td>3 sites -sandy -layered sand/clay -OC clay</td>
<td>soil L/D</td>
</tr>
<tr>
<td>Tjelta et al., 1986 (Gullfaks field, North Sea)</td>
<td>2 tests suction penetration vertical loading (static/cyclic)</td>
<td>*steel twin cell with concrete rect. panel $D_o$(one cell)=6.5m $t_w$(panel)=0.4cm L(panel)=22m L(cells)=19m</td>
<td>layered sand/clay</td>
<td>loading</td>
</tr>
<tr>
<td>Andréasson et al., 1988 (Gothenburg, Sweden)</td>
<td>4 tests suction penetration overturning moment (static/cyclic)</td>
<td>7 cell caisson $D_o$(cell)=0.6m $t_w=4.5mm$ L/$D_o=0.5-0.8$ $D_o/t_w=120-150$</td>
<td>NC clay</td>
<td>L/$D_o$ loading</td>
</tr>
<tr>
<td>Andersen et al., 1992</td>
<td>4 tests suction penetration eccentric vert. load (static/cyclic)</td>
<td>4 cell caisson $D_o$(cell)=0.9m $t_w=22.5mm$ L/$D_o=0.9$ $D_o/t_w=40.6$</td>
<td>NC soft clay</td>
<td>loading</td>
</tr>
</tbody>
</table>

*see Figure 2.19

Table 2.4 Experimental Research on Suction Caissons: Field Tests
<table>
<thead>
<tr>
<th>Field Application</th>
<th>Gorm Field Mooring Buoy Support</th>
<th>Gullfaks C Gravity Platform Foundation</th>
<th>Snorre Tension Leg Platform Foundation</th>
<th>Heidrun Tension Leg Platform Foundation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>North Sea, Danish Sector</td>
<td>North Sea, Norwegian Sector</td>
<td>North Sea, Norwegian Sector</td>
<td>North Sea, Norwegian Sector</td>
</tr>
<tr>
<td>Water Depth</td>
<td>40m</td>
<td>220m</td>
<td>310m</td>
<td>345m</td>
</tr>
<tr>
<td>Soil Profile</td>
<td>0-7m sands 7-10m soft clay &gt;10m stiff clay</td>
<td>0-1m loose sand 1-9.5m soft clay 9.5-28m clayey sand 28-43m stiff clay</td>
<td>0-8m soft clay 8-17m med.-stiff clay 17-26m v. stiff clay 26-30m sand w/clay layers</td>
<td>Not Available</td>
</tr>
<tr>
<td>Caisson Arrangement</td>
<td>single isolated cells $A_b=9.6 \text{ m}^2$ (per cell)</td>
<td>14 caissons form hexagonal skirt wall with 2 caissons at center $A_b=17,000 \text{ m}^2$ (total)</td>
<td>4 isolated 3-cell units $A_b=720 \text{ m}^2$ (per unit)</td>
<td>4 isolated 19-cell units $A_b=1500 \text{ m}^2$ (per unit)</td>
</tr>
<tr>
<td>Caisson Material</td>
<td>steel</td>
<td>reinforced concrete</td>
<td>reinforced concrete</td>
<td>reinforced concrete</td>
</tr>
<tr>
<td>Submerged Weight</td>
<td>22 metric tons</td>
<td>500,000 metric tons</td>
<td>3500 metric tons (per unit)</td>
<td>12,500 metric tons (per unit)</td>
</tr>
<tr>
<td>Single Caisson Geometry</td>
<td>$D=3.5m$ $t_w=25mm$ $L=8.5-9m$ $L/D=2.4-2.6$ $D/t_w=140$</td>
<td>$D_o=28m$ $t_w=0.4m$ $L=22m$ $L/D_o=0.8$ $D_o/t_w=70$</td>
<td>$D_o=17m$ $t_w=0.35m$ $L=11.6-12.5m$ $L/D_o=0.68-0.74$ $D_o/t_w=49$</td>
<td>$D_o=9m$ $t_w=?$ $L=4.8-5.0m$ $L/D_o=0.53-0.56$ $D_o/t_w=?$</td>
</tr>
</tbody>
</table>

Table 2.5  Field Applications of Suction Caisson Foundations
Figure 2.1 Phases in the Life of a Suction Caisson

a) Phase 1: Installation
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Figure 2.6  Empirical Breakout Curves Based on Naval Civil Engineering Laboratory Tests (Lee, 1972, 1973)

a) Immediate Breakout vs. Embedment Depth
\[ \log_{10}(F_b/F_{lb}) = -a(\log_{10} T - b) \]

- Indicates test discontinued before occurrence of breakout

Dashed-line band represents Equation 4 plus or minus one-half log cycle of the normalized time

- San Francisco Bay field tests
- Gulf of Mexico field tests
- FY-68 laboratory tests

Dashed-line band represents Equation 5 plus or minus one-half log cycle of normalized time

- \( a = -0.193 \)
- \( b = 4.24 \)

- \( a = -0.193 \)
- \( b = 3.84 \)

Figure 2.6  Empirical Breakout Curves Based on Naval Civil Engineering Laboratory Tests (Lee, 1972, 1973)

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Soil Profiles of 3 Test Sites

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7 Cell Suction Caisson

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Type of loading</th>
<th>Vertical load (static), kN</th>
<th>Depth of embedment, m</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>S</td>
<td>70</td>
<td>0.52</td>
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<tr>
<td>2</td>
<td>C</td>
<td>70</td>
<td>0.52</td>
</tr>
<tr>
<td>3</td>
<td>C&amp;S</td>
<td>35</td>
<td>0.50</td>
</tr>
<tr>
<td>4</td>
<td>C&amp;S</td>
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<tr>
<td>5</td>
<td>C&amp;S</td>
<td>4</td>
<td>0.52</td>
</tr>
</tbody>
</table>

*S = static test, C = cyclic test*

**Loading Schedule**

Figure 2.21  Schematic Drawing of 7 Cell Caisson and Load Schedule for Field Test Program Near Gothenburg, Sweden (Andréasson et al., 1988)
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Prediction of Caisson Tip Penetration Resistance

### Key

- **D**: Diameter of the pile
- **I**: Average cone resistance below the tip of the pile over a depth which may vary between 0.7D and 4D.
- **II**: Minimum cone resistance recorded below the pile tip over the same depth of 0.7D to 4D.
- **III**: Average of the envelope of minimum cone resistances recorded above the pile tip over a height which vary between 6D and 8D. In determining this envelope, values above the minimum value selected under II are to be disregarded.
- **q_p**: Ultimate unit point resistance of the pile.

### Note

To account for geometry D = 0.5 used in predictions.

### Tip Resistance vs. Penetration Depth

![Diagram showing tip resistance vs. penetration depth with labeled axes and data points]

**Figure 2.23** Measured and Predicted Tip Resistance During Suction Installation for Twin Cell Caisson in Gullfaks Field, North Sea (Tjelta et al., 1986)
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Test 4 - Cyclic

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Figure 2.32  Suction Caisson Foundation Installed in 1991 for Snorre TLP in Norwegian Sector, North Sea (Støve and Christophersen, 1992)

a) Field Layout
b) Soil Profile
c) CFT Geometry
Figure 2.33  Measured Penetration Resistance vs. Depth for Cone Penetrometer Tests and the 4 Concrete Foundation Templates of the Snorre Tension Leg Platform Suction Caisson Foundation (Støve and Christophersen, 1992)
Heidrun TLP Foundation

Heidrun Foundation Unit

Base Area, $A_b = 1500 \text{ m}^2$
Submerged Weight = 12,500 metric tons
Height (to tether porch), $H_{tot} = 26.45 \text{ m}$
No. Cells = 19
Cell Diameter, $D_o = 9 \text{ m}$
Cell Height, $H = 6.9 \text{ m}$
Final Penetration Depth, $L = 4.8-5.0 \text{ m}$

Figure 2.34  Suction Caisson Foundation Installed in 1994 for Heidrun TLP in Norwegian Sector, North Sea (Botros et al., 1996)
CHAPTER 3
DESIGN OF CET CELL

This chapter describes the design of the Caisson Element Test (CET) apparatus for simulating the installation, set-up, and axial tensile loading of a miniature caisson in a uniform, saturated 'element' of clay. The CET cell comprises five components, as shown in Figure 3.1: 1) the sealed test chamber, which contains a clay sample with consolidation stresses maintained by air pressure, 2) the model caisson, an unique two-piece design that enables independent control of the caisson wall and cap, 3) the driving system, which controls displacement of the caisson wall and cap and applies total stress on the clay surface, 4) a continuous automated feedback control system that can simulate various stages in the life of a suction caisson, and 5) the instrumentation package, which provides input signals for feedback control and data for test interpretation. Section 3.1 describes each of these components in detail. The samples of resedimented Boston Blue Clay (RBBC) are consolidated one-dimensionally in the test chamber (independent of the CET apparatus) using methods developed previously at MIT. This resedimentation procedure produces a highly uniform CET test sample with known stress history and engineering properties. Section 3.2 summarizes the properties of the RBBC used in the CET experiments. Finally, section 3.3 presents a complete description of the CET test procedures.
3.1 CAISSON ELEMENT TEST CELL.

3.1.1 Consolidation Chamber

The basic building block of the CET cell is the consolidation chamber, which is a rigid-walled, stainless steel cylinder attached to a 1.75 cm thick stainless steel baseplate (see Figure 3.2). The cylinder has an inside diameter of $D_1=30$ cm, a height of $H=25$ cm, and a wall thickness of $t=1$ cm. This chamber has been used at MIT since 1980 for the preparation of resedimented Boston Blue Clay (RBBC) samples (Germaine, 1982). For the CET experiments, the original consolidation chamber has been modified to enable penetration of the model caisson into a consolidated clay sample and to include additional instrumentation.

Figure 3.2 shows a schematic cross-section of the original chamber. During consolidation, the top of the clay sample is loaded incrementally through a rigid cap (and attached piston), which is sealed against the chamber wall by a lubricated O-ring. Movements of the piston assembly are guided by a stacked linear ball bearing assembly that minimizes friction. Pore water is free to drain through a 0.5 cm thick porous plastic disc attached to the top cap and through a 0.85 cm thick porous stone that fits on the baseplate. The plastic disc is secured to the top cap with screws in order to facilitate cleaning procedures. The porous stone, which is made of vitrified aluminum oxide ($\text{Al}_2\text{O}_3$) with an average grain size diameter of 120 µm and an induced porosity of 45%, simply lies in a recess within the baseplate. To prevent the migration of soil particles and porous plate penetration into the clay surface during consolidation, filter paper (0.014 cm thick) separates the clay from the porous disc at the top and the porous stone at the bottom.

Two major modifications were made to the consolidation chamber in order to conform to the requirements of the CET experiments. Figure 3.3 shows additional access ports that were machined into the chamber sidewalls and baseplate to allow for installation of pressure transducers. Eight ports were bored into the sidewalls at four different vertical
locations for total stress or pore pressure transducers. Similarly, nine access ports were machined through the baseplate and porous disk to allow installation of miniature pore pressure probes through the base and into the clay sample. These sidewall transducers and baseplate probes are described further in section 3.1.5.

The second major modification was a new sealed top cover and air-pressurized cavity for the model caisson experiments (see Figure 3.4). The main purpose of this design is to allow controlled caisson installation while maintaining the total vertical stress on the consolidated clay sample. The model caisson and soil sample are isolated from the pressurized air cavity by a cylindrical slip tube assembly, which can be divided into four parts: the drainage filter, rubber membrane, outer membrane slip ring, and inner membrane slip tube. The surface of the clay is covered by two layers of filter material: 1) 0.014 cm thick paper filter, and 2) 0.022 cm thick cloth filter. The paper filter allows vertical drainage of pore water while the cloth filter allows both vertical and lateral drainage. A 0.03 cm thick rubber membrane covers the filter and separates the saturated soil sample from the pressurized air cavity. Hence, the air pressure provides a total stress on the surface of the soil sample. The outside edge of the rubber membrane is sealed with RTV (rubber adhesive) to an aluminum slip ring, which is sealed to the chamber sidewall by two "X-rings" that allow the ring to slide up and down in response to soil deformations. The beveled outside bottom edge of the ring (see Figure 3.4) allows pore water to collect and drain freely through hollow tubing, which is connected to the atmosphere. The inside diameter of the rubber membrane is sealed with RTV to an inner slip tube, which is 18 cm tall, has a wall thickness of 0.164 cm, and has an inside diameter of 5.35 cm (see Figure 3.4). The inside diameter narrows to 5.1 cm at the bottom to minimize the clearance between the slip tube and the wall of the model caisson.

A 1.3 cm thick aluminum cover plate encloses the pressure cavity and has one central entry port for the inner membrane slip tube and caisson. The inner slip tube is sealed to the cover plate at the entry port location by one O-ring that allows the tube to
move freely in response to clay surface deformations. The cover plate also has access ports for applying and monitoring the chamber air pressure, draining the pore fluid, and making LVDT electrical connections.

This elaborate new test chamber cover assembly enables the application of consolidation stresses to the soil, free drainage and non-uniform deformations of the soil surface, and caisson access to the soil. The surface deformations are measured from within the pressure cavity as described in section 3.1.5.

3.1.2 Model Caisson

The centerpiece of the CET apparatus is the model caisson, which comprises two components, an outer caisson wall and an inner caisson cap, as shown in Figure 3.5. Although the prototype caisson cells are monolithic units, the two-piece design for the CET model is used for two reasons. First, it enables the model to simulate installation by underbase suction (as opposed to self-weight penetration). The clay sample is consolidated initially with the wall tip retracted to a position flush with the cap at the surface of the clay, as shown in Figure 3.13. With the caisson in this position, a uniform stress can be applied across the entire surface of the clay sample (i.e., by applying loads to the caisson that balance exactly the applied air pressure). Installation by underbase suction can then be simulated by driving the caisson wall at a constant rate, measuring the force increment picked up by the wall, and applying an equal but opposite force increment on the cap, as described in section 3.3.4.

The second reason for the two-piece caisson design is that the two components can be controlled independently, a feature that allows independent measurement of the cap and wall force contributions\(^1\). Separate wall and cap data provide important insights into the caisson behavior during successive phases of the CET experiments.

\(^1\)Alternative methods of instrumenting the caisson walls are impractical for such a small model.
Figure 3.5 illustrates the construction of the model caisson. The caisson wall is a brass Shelby tube with the tapered cutting edge removed, such that the tip is blunt. The tube has an outside diameter of $D_o=5.08$ cm and a wall thickness of $t_w=0.145$ cm. The wall is 33 cm tall (although only the bottom 5 cm penetrate the clay sample) and is attached to the drive assembly by four screw bolts, as shown in Figure 3.5. Figure 3.6 shows the caisson cap, a 5.7 cm tall brass cylinder with an outside diameter of 4.65 cm. Pore pressures can be measured at the center of the base of the cap through a ceramic porous stone and pressure transducer assembly (section 3.1.5 gives full details of the instrumentation). The transducer is secured in place by a threaded brass locking ring. The cap is connected to the cap driver by a hollow aluminum rod which screws into the threaded center hole at the top of the cap.

In order to prevent metal to metal contact between the cap and wall, there is an O-ring around the outer perimeter of the cap. Relative displacements between the cap and wall inevitably generate frictional forces at the O-ring seal and hence, create some uncertainty in the relative force contributions of the cap and wall. However, this does not influence the total caisson force measurement (see section 4.3 for discussion). Efforts to reduce this source of friction include minimizing the O-ring squeeze and using a liberal amount of lubrication. In addition to the frictional problem, separating the wall and cap introduces an air gap of 0.07 cm between the tip of the wall and the cap at the soil surface. During all phases of the suction caisson test, there is no applied stress on the soil at the surface of this gap. Hence, soil must form an arch across this gap. Section 4.3 evaluates the uncertainty arising from O-ring friction and the effect of soil arching between the wall and cap.

\footnote{In the first test CET1, the tapered edge was left in place.}
3.1.3 Driving System

The third component of the CET cell is a three part mechanical driving system, which applies the total stress on the clay surface and drives the movements of the caisson wall and cap. Figure 3.7 highlights each of the three driving subsystems within the CET apparatus. Each of these three driving operations acts independently and is controlled automatically by the computer control system, described in section 3.1.4. This section describes the function and design of each of the driving subsystems in detail.

The first driving subsystem applies a vertical total stress on the surface of the clay outside the caisson by using air pressure in the chamber cavity (see Figure 3.8). The air pressure originates from the central geotechnical laboratory air compressor, which maintains a continuous source of pressure between 11 and 13 ksc. This pressure is reduced and regulated by a Fairchild precision pressure regulator (Kendall Model 10). For this testing program, the air pressure is reduced to a level no greater than 1 ksc. This regulator is driven by a reduction gear system connected to a velocity-controlled direct current motor made by Electro-Craft Co. (Model E352). The motor has a power supply and feedback controller that operates the motor at a speed proportional to an analog command signal provided by the computer control system (see section 3.1.4).

Figures 3.9 and 3.10 show the subsystems that drive the caisson wall and cap and constitute the entire superstructure rising 1.5 m above the consolidation chamber (see Figure 3.7). To support the caisson wall and caisson cap driving subsystems, four stainless steel threaded rods (1.8 m long by 1.27 cm diameter) are secured with hex nuts to the bottom of the consolidation chamber and rise 1.5 m above the top of the chamber. Even though caisson movements during testing are restricted to less than the clay sample height (which ranges between 12 and 14 cm) the rather tall superstructure is required to allow coaxial placement of both the wall and cap drivers.

The caisson wall driving subsystem is shown separately in Figure 3.9. Analog command signals from the computer control system are sent to an electric motor, which
drives a linear ball screw actuator via a reduction gear box. The actuator, in turn, is attached to a multi-part drivetrain that is rigidly connected to the caisson wall. The electric motor is identical to the one used for the total stress driving subsystem discussed above (Electro-Craft Model E352). The 100:1 reduction gear box reduces the motor speed from a maximum of 7500 rpm to 50 rpm, while increasing the maximum torque from 0.5 to 16.1 kg-cm. To transfer rotational motion from the motor to linear movement necessary to drive the caisson vertically, the gear box is connected to a linear ball screw actuator made by Duff-Norton (Model M28630-8). The actuator rod is connected to the wall drivetrain comprising 1) the wall force transducer, which is fastened to 2) a square plate that is connected by 3) four 1.27 cm diameter stainless steel rods\(^3\) to 4) a circular drive plate, which is secured rigidly to the caisson wall by 5) a cylindrical connector that fits snugly inside the caisson wall. Four threaded hex head bolts, with washers that conform to the caisson wall exterior, fasten the connector to the wall.

Ancillary components to the caisson wall driving subsystem include the support plate and the actuator compression springs. The linear ball screw actuator housing is bolted to the upper support plate, which is secured by hex nuts to the four support rods extending from the consolidation chamber. The dual purpose of this support plate is to maintain the vertical position of the wall drivetrain and actuator and to react against the soil resistance encountered during testing.

The actuator compression spring setup is used solely to keep the actuator worm screw in tension throughout testing, and hence avoid problems associated with system compliance or lashback. Compliance is definitely a problem during the initial phase of pullout, when the travel direction of the worm screw reverses. Without a compression spring, there is a time lag in the load applied by the actuator such that unacceptable relative displacements can develop between the caisson cap and wall (see section 4.5.4).

\(^3\)This drivetrain rod extension is necessary to accommodate the caisson cap actuator that is located concentric within the rods (see Figure 3.7).
These problems are mitigated by including two springs, which are compressed between the support plate and a rectangular plate attached to the top of the worm screw actuator (see Figure 3.9). In this design, spring compression applies an upward force on the rectangular plate, which, in turn, applies a tension on the worm screw.

The driving system for the caisson cap is shown in Figure 3.10 and functions in the same way as the wall driver. Analog signals are sent from the computer control system to an electric motor, which drives a (Duff-Norton) linear ball screw actuator via a reduction gear box. The cap drivetrain includes a force transducer and hollow extension rod. The force transducer is located between the worm screw and the extension rod and measures the cap force for computer control feedback and data acquisition. The extension rod is an aluminum tube that screws into the top of the caisson cap, thereby connecting the cap with the transducer. In order to make the connection, the rod must pass through the circular drivetrain plate and inside the caisson wall itself (see Figure 3.9). The rod is hollow to allow access of electrical lines to the cap pore pressure transducer (see sections 3.1.2 and 3.1.5). The ball screw actuator housing is mounted on a support plate that is secured to the four threaded rods extending vertically from the chamber. This support plate maintains the cap drivetrain position, provides the necessary reaction force to the cap, and acts as a fixed reference surface for the displacement transducer that measures wall movements (see section 3.1.5). Finally, the cap actuator operates in tension throughout the test using a pair of pre-compressed springs similar to those used for the wall actuator.

3.1.4 Control System

Once the driving system superstructure and model caisson are assembled on top of the consolidation chamber, testing proceeds under automated control. The concept for control of the CET apparatus is based on previous technology that was used first to automate triaxial testing at MIT (Sheahan and Germaine, 1992). This section first explains briefly how the CET cell conforms with existing automation and outlines the
feedback control loop, which is the essential component of the automation. This leads to a detailed discussion of the hardware and software components and how they work together to operate the control loop.

There are two philosophical goals behind laboratory automation: 1) flexibility, to allow the widest range of testing capability for a particular device; and 2) simplicity, to enable future modifications to the control system. To attain these goals, the control system (hardware and software) is necessarily modular in nature. Each of the driving systems in the CET apparatus (cell air pressure, caisson cap and wall actuators) is controlled separately, such that the device requires three independent axes of control. And each phase of the caisson element test is controlled by a separate algorithm, such that complex event sequences can be simulated for individual tests without overly sophisticated software. Another modular component consistent with laboratory automation is the instrumentation and data acquisition setup, which is discussed in section 3.1.5.

The essential ingredient for automation is the feedback control loop for the driving systems of the CET apparatus. The basis for the control loop is a simple proportional gain adjustment algorithm, a procedure wherein the command signal is generated by an analytical expression that compares a transducer signal to a target value. The command signal is specified by the difference between the target and actual values, divided by a gain factor. The gain factor is equal to a rate of change of the control variable to a unit input voltage over a unit of time. This gain factor is commonly referred to as the virtual stiffness of the variable. The vertical stress driving system is controlled by this feedback control loop in all phases of testing and is a good example to illustrate the algorithm. The air pressure transducer measures the chamber air pressure (e.g., \( P_a(m) = 0.74 \ [\text{ksc}] \)) and sends the voltage signal to the computer, which then calculates the proper command signal. This signal is equal to the difference between the actual value and the target value (e.g., \( P_a(tg) = 0.75 \ [\text{ksc}] \)) divided by the air pressure gain factor (e.g., \( \text{Gain} = 0.05 \ [\text{ksc/volt-sec}] \)). The resulting command signal for one second of time is:
\[
\text{Command Signal} = \frac{P_s(m) - P_s(tg)}{\text{Gain}} = \frac{(0.74 \text{ ksc} - 0.75 \text{ ksc})}{0.05 \text{ ksc/volt - sec}} \times \frac{1}{1 \text{ sec}} = -0.20 \text{ volts}
\]

Thus, a command signal of -0.20 volts is sent for one second to the motor that drives the air pressure in order to maintain a target air pressure of 0.75 ksc. After the motor stops, the loop repeats. During certain test phases, the other two driving subsystems (caisson wall and cap) are controlled by a feedback loop that uses proportional adjustment and an additional "integration" term to effect better control. These loops will be discussed below in the control system software section. Note that during all phases of testing, command signals are sent to the air pressure system at the rate of one second per control loop, while signals to the caisson wall and cap driving systems are continuous. That is, the caisson wall and cap motors operate continuously, but their command signals are updated during each control loop.

Both the hardware and software components of the control system cooperate in order to conduct the feedback control loop. The hardware includes the computer equipment, electronic signal converters, driver interfaces, and instrumentation. The software consists of the computer programs that generate the signals necessary to operate the driving systems.

*Control System Hardware*

The main function of the control system hardware is to convey electronic information along the digital feedback control loop that operates the three independent driving systems. Figure 3.11 shows a schematic drawing of the control system hardware components. Note that the computer and the electronic signal converters (analog to digital converter and digital to analog converter) are the only hardware components common to all three control axes.
The first hardware component is the primary transducer. For illustrative purposes, let us say the feedback control loop starts at the primary transducer for a particular driving system. This transducer measures a physical quantity that the computer software is attempting to control. For the CET air pressure system, the primary transducer is always the air pressure transducer, while the primary transducer for the wall and cap systems can be either the displacement or force transducer, depending on the phase of the test. Detailed information on the characteristics of these transducers is provided in section 3.1.5.

The output from the primary transducer is sent to an analog-to-digital (A/D) converter, which converts this continuous, variable analog signal into a digital form (number of bits) that the computer can understand. The A/D converter is the heart of a circuit board that is placed in an expansion slot in the computer. The CET apparatus uses the multi-channel analog-to-digital converter device (MADC) developed by Sheahan (1991) at MIT for computer-automated triaxial testing. A low cost alternative to commercial A/D circuit boards, the MADC was designed to allow a minimum 18 bit resolution (b.r.) during signal conversion from analog to digital. This means that voltage signals from the primary transducers can be converted with a precision of $\pm3.8\times10^{-5}$ volts over a range of 10 volts (voltage resolution $= 10 \text{ V}/2^{\text{b.r.}}$). The key element of the MADC is the Analog Devices AD1170 analog-to-digital converter, which performs the basic function of translating analog signals in volts to digital signals in bit counts.

An IBM-compatible personal computer (PC-AT with an Intel 80286 processor, an expansion slot for the MADC board, and a parallel printer port for the digital-to-analog board) houses the control system software that determines the new command signal to be sent to the driving system to control the test phase (control system software is discussed in this section below). A monochrome monitor is used with the computer to display values of the measured variables during testing and to allow the user to interface with the control software.
The command signal generated by the software then is converted back into an analog signal through a digital-to-analog (D/A) converter board that is located within the computer and is connected to the parallel printer port. The CET apparatus uses a commercial board (12 bit resolution with a 10 volt range) sold by Strawberry Tree Incorporated.

From the D/A card, the analog command signals are sent via an amplifier to the electric motor that drives one of the three independent systems. The driving subsystem then moves the CET cell components according to the command signal received, thus completing the feedback control loop. A new cycle begins once the MADC converts a new analog signal from the primary transducer.

**Control System Software**

The control software consists of three programs that are written in the BASIC programming language: 1) MASTER.BAS allows the user to operate the driving system motors manually and to evaluate the status of each of the primary transducers; 2) SETUP.BAS lets the user input test-specific variables that are passed on to the control program, which is 3) CETEST.BAS. This last program actually controls the test and consists of separate modules that perform the various phases that simulate the event history of a suction caisson including consolidation, suction driving, holding stress, and monotonic pullout. Each of these programs were derived from programs used to control the phases in triaxial testing. Figure 3.12 depicts the interaction among the three programs.

MASTER.BAS is a menu-driven program that allows the user to operate individually the air pressure and caisson wall and cap control motors. Any of the three motors can be operated in a step-wise or continuous mode. Both the step increment and rotational speed can be varied. In addition, the transducer readings and input voltage can be displayed on the monitor.
SETUP.BAS is a program that generates the data input file of test parameters to be used by the control program CETEST.BAS. In an on-screen format, the user inputs caisson wall and cap dimensions and weights, transducer zero values, and calibration factors.

CETEST.BAS is the test control program and is composed of four separate control modules: CONS, HOLDSTS, SUCDRV, and MONPULL. CETEST.BAS begins by setting up the computer keyboard, analog-to-digital conversion card (MADC), and the three control motors and obtains an initial set of readings for all primary transducers. The user then starts a particular phase of the suction caisson test by choosing one of the four control modules from an on-screen menu. Each of the software modules incorporates a feedback control loop with at least proportional gain adjustment to control one or more of the three system variables: air pressure, cap force or displacement, and wall force or displacement.

CONS is the software module that starts each increment of consolidation on the CET clay specimen. The algorithm applies a user-specified schedule of incremental vertical loads on the clay surface while maintaining zero relative displacement between the caisson cap and wall. Figure 3.13 illustrates the control methodology for the CONS module. The user first decides the amount and time duration for each increment of load. The principal transducers are the chamber air pressure, caisson cap force, and wall displacement transducers (AP, L2, and D1, respectively in Table 3.1). At the start of a load increment, the air pressure and cap load are increased until reaching the target pressure $P_a (=\sigma'_{vc})$ and cap force $F_c (=\sigma'_{vc}A_c)$, respectively. Then, the vertical stress across the entire clay surface is maintained for the specified time duration. To maintain constant stress and zero relative displacement, a feedback control loop is placed on the air pressure transducer, the cap force transducer, and the wall displacement transducer. At the end of the specified consolidation time, the loading continues with the next increment. At any time during the CONS module, the user can escape to the menu screen or switch
directly to the HOLDSTS option (see Figure 3.12). If the user does not intervene, the computer automatically enters the HOLDSTS module following the end of the last increment.

HOLDSTS is a module that allows simulation of caisson set-up after installation by underbase suction or caisson set-up during sustained tensile axial loading (See Figure 3.14). This algorithm maintains a constant vertical stress over the clay surface (outside the caisson) and a constant total force on the caisson \((F_{\text{tot}}=F_c+F_w)\) with zero relative displacement between the cap and wall. Since the total force is kept constant and the wall and cap are "locked" together, the caisson is allowed to deform as a unit while the stresses redistribute following suction driving or application of tension forces. The constant vertical stress \((\sigma_v)\) outside the caisson is achieved through feedback control on the air pressure transducer using a simple proportional algorithm. Control signals are sent in 1 second bursts for each control loop. The target value \(\sigma_v\) is obtained through user-input either from the last increment in CONS or the original menu of test phases.

The total force on the caisson is kept constant by feedback control on either the caisson cap or caisson wall. The target caisson force, \(F_{\text{tot}}\), is specified either through user input (if HOLDSTS was accessed directly from the test menu) or from the last transducer readings from the preceding control module. During each round of the feedback control loop, the actual cap force and wall force values are measured and summed to give the actual total force, \(F_{\text{tot}}\). Then the difference between the actual \(F_{\text{tot}}\) and the target \(F_{\text{tot}}\) is computed to give an error value, \(errF_{\text{tot}}\). This value is then multiplied by a gain factor \((a)\) and added to the command voltage \((V_i)\) that was sent on the previous loop to give the HOLDSTS integration term \((HS_{\text{intg}})\) as follows:

\[
HS_{\text{intg}} = V_i + a(errF_{\text{tot}})
\]

Adding this integration term to a proportional term yields the command signal as follows:
\[ V_{i+1} = HS_{\text{intg}} + b(\text{err}F_{\text{tot}}) \]

where \( b \) is a gain factor. Thus the feedback control loop for the total caisson force, \( F_{\text{tot}} \), is proportional with an integration term. This term is necessary because the virtual stiffness of both the wall and cap changes throughout the test depending on how deep the wall has penetrated the soil, and such changes must be incorporated into the control loop. The gain factors \( a \) and \( b \) are obtained through trial and error.

The last control loop in HOLDSTS maintains zero relative displacement between the cap and wall by simple proportional feedback control on the displacement transducer (either \( D_1 \) or \( D_2 \)) of the caisson component not currently under force control. From the HOLDSTS module, the user can proceed directly to SUCDRV, MONPULL, or the test menu.

SUCDRV maintains \( \sigma_v \) on the clay surface outside the caisson and drives the caisson wall into the clay sample at a constant rate of displacement. The algorithm models installation by underbase suction by applying increments of load to the cap that are equal and opposite to the wall force caused by driving. Figure 3.15 illustrates the control features of SUCDRV. After obtaining the user-specified drive rate and final penetration depth, the computer sends a continuous signal to the caisson wall motor, which drives the wall at a constant rate until reaching the final depth. The total force on the caisson \( F_{\text{tot}} \) is maintained constant from the start of driving. As the wall penetrates the soil, the computer uses a simple proportional feedback loop on \( F_{\text{tot}} \) with continuous adjustment of cap force, \( F_c \). The difference between the actual and target values of \( F_{\text{tot}} \) is calculated to give an error value, \( \text{err}F_{\text{tot}} \). Then, \( -\text{err}F_{\text{tot}} \) is added to the cap force. Due to the large increase in wall force that occurs at the start of driving, the algorithm simultaneously sends an initial control signal (to reduce load on the cap) and a signal to drive the wall. At any point during the driving phase, the user can switch to HOLDSTS or return to the test menu. If the user does not intervene, the computer will drive the wall to the prescribed
depth and then automatically switch to HOLDSTS, which maintains the prescribed $\sigma_y$ (from CONS or the menu) and $F_{tot}$ (from the last reading in SUCDRV).

In MONPULL, the computer keeps $\sigma_y$ constant and pulls the caisson wall and cap together at a constant rate (see Figure 3.16). The computer sends a continuous command signal to withdraw the wall at a constant rate until the caisson reaches a specified displacement or total force. As in all the modules, simple proportional feedback control is performed on the air pressure regulator to maintain a constant $\sigma_y$ on the clay outside the caisson. For zero relative displacement between the wall and cap, feedback control is performed by incorporating a proportional term for the cap displacement rate and both proportional and integral terms for the relative displacement. The relative displacement integral term ($RD_{intg}$) is first calculated by multiplying the relative displacement error ($errD$) by a gain factor (a) and adding the resultant to the prior control loop command signal ($V_i$) as follows:

$$RD_{intg} = V_i + a(errD)$$

The cap command signal then is generated by adding the integration term to two proportional terms. The first proportional term is the relative displacement error ($errD$) multiplied by a gain factor (b); the second proportional term is the cap velocity error ($errCapv$) multiplied by a gain factor (c). $ErrCapv$ is the difference between the prescribed wall velocity and the measured cap velocity, which, in turn, is computed by dividing the measured cap displacement by the elapsed time during one control loop. The cap command signal equation is:

$$V_{i+1} = RD_{intg} + b(errD) + c(errCapv)$$
At the start of pullout, the algorithm simultaneously sends a control signal to pull the wall and a separate signal to pull the cap. Thereafter, the feedback control loop maintains zero relative displacement. As in SUCDRV, the user can switch to HOLDSTS or the test menu. Barring user interaction, the computer automatically proceeds to HOLDSTS once the target displacement or total force is reached.

3.1.5 Instrumentation

The fifth and final component of the CET apparatus is the instrumentation package, which is consistent with the automated laboratory testing concept. The measurements are made by a variety of transducers all connected to the Central Data Acquisition System in the MIT Geotechnical Laboratory. The transducers serve a dual purpose. Five of the instruments (chamber air pressure, caisson wall force and displacement, and caisson cap force and displacement - AP, L1, D1, L2, D2, respectively) serve as primary transducers because they provide voltage signals for the feedback control loops (see section 3.1.4), while the remaining instrumentation is used to monitor parameters of interest that affect the performance of the caisson and clay during a caisson element test. This section describes the physical characteristics, function, and location of the instrumentation used in a typical CET experiment. Chapter 4 evaluates the quality of the measurements obtained in the CET test program.

Each of the transducers is connected to the Central Data Acquisition System, which consists of an IBM compatible PC interfaced with an expanded channel Hewlett-Packard HP3497A data acquisition unit. The system is driven by software called EASYDAT. It is capable of monitoring 120 channels throughout the geotechnical laboratory. One advantage of the Central System is that it simplifies the programming requirements of the CET control software. Since the system is task driven and each task can monitor up to 20 channels at user-specified times greater than one second, test measurements can be tailored to each particular task without altering the program or
interfering with the CET control. Another advantage is the high degree of measurement precision. The Central System uses a very high integrating analog-to-digital converter with autoranging capability to produce a precision of one microvolt with a range of 1000 volts.

Typical CET tests require less than 20 channels to monitor instrumentation and power input voltage. A 14 transducer instrumentation package used for test CET10 is illustrated in Figures 3.17 and 3.18. Table 3.1 lists the CET10 devices, their location, and their capacity and precision when connected to the Central Data Acquisition System.

Figure 3.17 shows the locations of the five primary transducers, which consist of a pressure transducer to measure chamber air pressure (AP), two force transducers to measure caisson wall and cap force (L1, L2), and two LVDTs for wall and cap displacements (D1, D2). As discussed in section 3.1.4, one purpose of these five transducers is to supply input voltages for the feedback control loops. The other purpose, of course, is to provide data for test interpretation. The wall displacement (D1) is measured relative to the fixed support for the cap driving system (see Figure 3.17) and this provides an absolute displacement of the wall relative to the test chamber. The displacement of the cap is measured relative to the wall, as differential displacements are of primary concern, and subtracting two absolute measurements would introduce precision error. As discussed in section 3.1.4, the control software maintains a specific relative displacement of zero in CONS, HOLDSTS, and MONPULL modules.

Displacements of the clay surface are measured by direct current Linear Voltage Displacement Transducers (LVDT) located within the pressure chamber (S1-S5 as shown in Table 3.1, Figure 3.18). Each of these transducers is mounted on a cross arm radiating from a circular frame that is secured to the inside of the chamber wall by four screws, as shown in Figure 3.19. Two of the cross arms have mounting holes located at a radius of 5.2 cm from the chamber center. The remaining cross arms have mounting holes at radii r=7.7, 9.8, 12.1 cm. In order to obtain displacement data close to the caisson wall, one of
the LVDTs mounted at a radius of 5.2 cm has a core with an elbow extension to measure displacement at a radius of 4.2 cm (labeled S1 in Figure 3.19). The moving core of each transducer rests directly on the rubber membrane covering the clay surface.

Pore pressures beneath the caisson cap are measured by a 1.75 ksc capacity Data Instruments pressure transducer (CP, Table 3.1) mounted directly behind a 0.93 cm thick ceramic porous stone with a 1 ksc air entry pressure. As shown in Figure 3.6, the stone is set flush with the cap bottom surface and has a diameter of 2.2 cm. The stone surface area represents 22% of the total cap area. The pressure transducer is secured to the inside of the cap by a threaded brass fitting and O-ring that seals the transducer face approximately 0.01 cm from the back of the stone. Note that the threaded cap hole also provides a rigid connection for the cap drivetrain extension rod. This rod is hollow to provide a conduit for the pressure transducer wiring.

Pore pressures within the soil mass are measured using stainless steel hypodermic needle probes, which were developed iteratively during the course of this research in order to achieve rapid response and reliable measurements. A full description of the probe analysis is provided in Chapter 4. Figure 3.20 shows a cross sectional drawing of the latest generation CET pore pressure probe, which consists of a Kulite pressure transducer, a transducer block, and a stainless steel tube. The tube has an inside diameter of 0.023 cm and has a 0.2 cm long, 20 micron porous stone press-fitted to the tip. The tube is soldered to a stainless steel transducer block to form one monolithic unit. A threaded coupling connects the pressure transducer to the transducer block, and an O-ring on the pressure transducer seals the void space between the porous stone and the transducer. The transducer block is designed to minimize the clearance (0.013 cm) between the transducer face and the block. The probes are inserted into the clay through holes in the chamber baseplate and porous stone. The baseplate has nine ports for pore pressure probes, as shown in Figure 3.21. The probes are held in place by a threaded brass connector that screws into the baseplate and laterally compresses an O-ring surrounding the probe tube.
For a typical CET experiment, pore pressures are typically measured at \( r=0.0, 1.78 \) cm inside the caisson wall and at \( r=3.18 \) cm outside the wall\(^4\).

Total lateral stress and pore pressure on the side of the clay sample can be measured by pressure transducers that are secured to the chamber sidewalls. Figure 3.22 shows the consolidation chamber sidewall with entry ports for total stress and pore pressure measurement. The chamber has 8 entry ports at different heights along the sidewall. Stresses are measured by a Kulite transducer that threads into a brass connector. This connector screws into the sidewall and seals against the wall with an O-ring. The connector is designed so that, for total stress measurement, the center of the transducer face is flush with the inside of the chamber wall. Due to the curvature of the chamber wall, part of the transducer face protrudes into the chamber space, which may introduce some errors in the measurements. For the pore pressure measurement, the connector is designed with a 0.93 cm thick ceramic porous stone affixed to the end. The total stresses have been measured in 3 tests (CET1,2,6) and no sidewall pore pressure data have been obtained in the current program.

During the course of testing over two years, different model transducers were used to measure force, displacement, and pore pressure. A complete breakdown of the instruments used for each test are given in Table A1 of Appendix A. The locations of the transducers for each test are illustrated in Figures A1 through A14 of Appendix A.

3.2 RESEDIMENTED BOSTON BLUE CLAY

Resedimented Boston Blue Clay (RBBC) is the standard test material used for the experimental program for the following reasons: 1) procedures for manufacturing uniform samples of resedimented BBC are well-established (Germaine, 1982); 2) the engineering

\(^4\)Note that the caisson wall has an outside radius of 2.54 cm.
properties of RBBC are well-established from previous laboratory tests; 3) there has been extensive analytical research to model the properties of RBBC (e.g., Whittle, 1987); and 4) the engineering behavior of RBBC is typical of natural, uncedmented clay deposits with similar index properties.

3.2.1 Origin and Processing

Natural Boston Blue Clay (BBC) was deposited in the Boston basin about 12,000 to 14,000 years ago following the Wisconsin glacial period (Kenney, 1964). The source material for the current CET test program was obtained in 1992 from the base of an excavation for MIT's Biology Building (Building #68). Approximately 2500 kg of soil was obtained at a depth of about 12 meters, with an overconsolidation ratio ranging from 1.3 to 4.3 (Berman, 1993). The material was softened with tap water and mixed into a thick slurry. Then, the slurry was passed through a #10 US standard sieve to remove all non-natural material, gravel, coarse sand, and large shell fragments. The slurry was oven-dried at 60°C in preparation for grinding. The dried material, which consisted of pieces ranging in size from 1 to 15 cm across, was ground to 95% passing a #100 US sieve by the Sturtevant Company using a roller mill. Finally, the material was manually randomized by two blending operations. The dry powder, now known as Series IV Boston Blue Clay, is stored in sealed 40 gallon containers5.

3.2.2 Index Properties

The properties of natural BBC vary widely over the Metropolitan Boston Area even though the basic mineralogy of the clay is the same. Therefore, each time new material is obtained for resementation, it is necessary to perform several index and engineering tests to verify that the soil is sufficiently similar to the prior material.

5Previous tests from 1988-1994 used Series III BBC, which was obtained by augering from a depth of 23 meters during construction of a parking garage near Kendall Square in Cambridge, Massachusetts.
Resedimentation of BBC at MIT has produced close to 70 recorded batches of testing material. This has produced an extensive database of material index and engineering properties. Table B.1 in Appendix B lists specific gravity, Atterberg limits, clay fraction, and salt concentration values for RBBC used in research since 1961. To make use of this database and to ensure that the clay was uniform from sample to sample, numerous index and engineering tests were conducted on all samples used for the CET testing program. Table 3.2 lists the index data for the Series IV BBC powder, 2 RBBC batches used for the research of Sinfield (1994), and 14 batches used in this research. This section discusses the uniformity and repeatability of index properties for the RBBC used in the CET test program and compares these properties with results from previous testing programs.

Figure 3.23 shows the grain size distribution for Series IV BBC powder and resedimented BBC (RBBC). The tests on RBBC batches 406-409, 411 were performed on material after the addition of salts and phenol and additional batching operations (see section 3.3.1 for batching procedure). The distributions show that the soil has a fine fraction (% passing the #200 sieve) greater than 98%. The average clay fraction (% less than 2μm) is 58±1.2%. The fine fraction and clay fraction is slightly higher than the respective data from Series III BBC6.

Data from Atterberg limit tests on Series IV BBC reveal batch to batch consistency. Figure 3.24a depicts the plastic and liquid limits for BBC powder and batches RBBC 401-411, 413-417. The average plastic limit is \( w_p = 23.5 \pm 1.1\% \), the average liquid limit is \( w_l = 46.1 \pm 0.9\% \), and the average plasticity index is \( I_p = 22.7 \pm 1.2\% \) (see Table 3.2). These data are plotted in a plasticity chart in Figure 3.24b, which confirms that the material is a low plasticity (CL) clay.

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6Data from Series III BBC (see Figure B.1, Appendix B) indicate a fine fraction of 90-95% and a clay fraction of 46-56%.
Measurements of specific gravity for Series IV RBBC yielded an average value of $G_s=2.81$, which is higher than previous research\(^7\), but is within the expected range for illitic clays ($G_s=2.60$ to $2.84$ for illite; Lambe and Whitman, 1968).

Salt content was measured by conductivity and calibrated against a KCL standard. For previous research, the salt content varied, 2-35 g/l (see Table B.1, Appendix B). For Series IV RBBC, the average salt content was $11.6\pm1.5$ g/l. As described in the batching procedure in section 3.3.1, salt is added to the powder to make the clay samples.

Organic content by combustion yielded a value of 4.4% for Series IV RBBC. These data are not available for Series I through III material.

3.2.3 Engineering Properties

This section presents a compact summary of the most important engineering properties of normally consolidated re-sedimented Boston Blue Clay. Compression, consolidation, and flow characteristics are derived from consolidometer tests on BBC Series IV (RBBC 401,404-411,413-417). Undrained stress-strain-strength behavior is culled from tests on BBC Series III and include triaxial compression and extension tests (Sheahan, 1991) and direct simple shear tests (Ortega, 1992).

*Compression, Consolidation, and Flow Properties*

Figure 3.25 summarizes the compression, consolidation, and flow properties resulting from the consolidometer tests for the CET test program\(^8\). As described in the following section 3.3.2, the consolidometer test in the CET program is the stage wherein the RBBC is consolidated using a rigid top cap. In all tests the slurry was loaded incrementally with a load increment ratio of LIR=1 to a maximum stress of $\sigma_v'=0.5$ ksc.

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\(^7\)Specific gravity for Series I-III BBC ranges from $G_s=2.75$ to $2.785$ (see Table B.1, Appendix B)

\(^8\)Appendix B contains tabulated data for each individual consolidometer test and the calculation method for the consolidation parameters.
In four tests (RBBC 401, 404-406), the clay was unloaded in two increments to a stress of \( \sigma' \approx 0.125 \) ksc.

Figure 3.25 shows compression curves (void ratio vs. consolidation stress) for all presentable consolidometer data and tabulates the compression index \( C_c \), coefficient of volume change \( m_v \), vertical hydraulic conductivity \( k_v \), and vertical coefficient of consolidation \( c_v \). Aside from some variation in the void ratio at a particular consolidation stress\(^9\), the consistency of the slope of the compression lines from batch to batch is unmistakable. There is very little scatter in the compression indices; in the stress range from \( 0.125 < \sigma' < 0.25 \) ksc the compression index was \( C_c = 0.588 \pm 0.046 \), while for \( 0.25 < \sigma' < 0.5 \) ksc the index was lower at \( C_c = 0.525 \pm 0.033 \). These values are slightly higher than data (\( C_c = 0.557 \pm 0.068, 0.479 \pm 0.064 \)) for the same two respective stress intervals from 6 consolidometer tests on Series III BBC (Seah, 1990). The swelling index, calculated for two stress intervals, \( 0.125 < \sigma' < 0.25 \) ksc and \( 0.25 < \sigma' < 0.5 \) ksc, averaged \( C_s = 0.022 \) and \( 0.010 \), respectively.\(^{10}\)

The vertical coefficient of consolidation \( c_v \) for 5 stress levels from \( \sigma' = 0.0313 \) to \( 0.5 \) ksc is tabulated in Figure 3.25. At the lowest stress level of \( \sigma' = 0.0313 \) ksc, the average value is \( c_v = 3.27 \times 10^{-4} \) cm\(^2\)/sec. This rises to \( c_v = 10.59 \times 10^{-4} \) cm\(^2\)/sec at the maximum stress of \( \sigma' = 0.5 \) ksc. Both the magnitude of \( c_v \) and the trend with stress level is consistent with previous consolidometer tests on Series III BBC (Seah, 1990).

The vertical hydraulic conductivity decreases with increasing consolidation stress, as the data in Figure 3.25 suggest. From \( \sigma' = 0.0625 \) to \( 0.125 \) ksc, the computed hydraulic conductivity averaged \( k_v = 47.4 \times 10^{-8} \) cm/sec. This decreased to \( k_v = 41.6 \times 10^{-8} \) cm/sec for the stress interval from \( \sigma' = 0.125 \) to \( 0.25 \) ksc and dropped further to \( k_v = 23.7 \times 10^{-8} \) cm/sec for the range from \( \sigma' = 0.25 \) to \( 0.5 \) ksc. Once again, this behavior is consistent with prior consolidometer testing (Seah, 1990).

\(^9\)Variation in the void ratio is due to uncertainty in the phase relations calculations. See Appendix B.
\(^{10}\)Although four tests (RBBC401-402, 405-406) incorporated swelling increments, only two (401 and 405) yielded presentable swell data. See Appendix B.
Undrained Triaxial Compression

Typical behavior for normally consolidated reseedimented BBC during undrained triaxial compression is derived from two tests (CTX-11,13) conducted by Sheahan (1991). The test specimens were trimmed from Series III RBBC, \( K_o \)-consolidated to \( \sigma'_{vc} = 2.8 \) ksc, and sheared in triaxial compression at the standard rate of \( \dot{e}_a = 0.5\% / \text{hr} \). The average lateral earth pressure coefficient (\( K_o = \sigma'_{h}/\sigma'_{v} \)) during consolidation beyond a vertical stress of \( \sigma'_{v} = 0.6 \) ksc was \( K_o = 0.47 \). The undrained shear strength ratio of the normally consolidated RBBC in compression averaged \( s_u/\sigma'_{vc} = 0.32 \), which was mobilized at an average axial strain of \( e_a = 0.15\% \). The friction angle at peak shear stress and maximum obliquity averaged \( \phi'_p = 25.0^\circ \) and \( \phi'_{mo} = 33.4^\circ \), respectively. Gradual post-peak strain softening was evident, as the mobilized shear resistance at \( e_a = 10\% \) strain was \( q_{10\%}/\sigma'_{vc} = 0.25 \), which is nearly 80\% of the peak strength (\( q_{10\%}/s_u = 0.78 \)). Measurement of the shear stiffness at very small strains (\( e_a < 0.01\% \)) was hampered by the lack of on-the-specimen strain measurement equipment. The average normalized secant shear modulus measured at \( e_a = 0.01\% \) strain was \( G_{sec}/\sigma'_{vc} = 457 \).

Undrained Triaxial Extension

Sheahan (1991) also conducted two standard undrained triaxial extension tests (CTX-9,50), wherein the specimen was \( K_o \)-consolidated to an average stress of \( \sigma'_{vc} = 2.8 \) ksc and sheared at the standard rate of \( \dot{e}_a = 0.5\% / \text{hr} \). As for the compression test, the lateral earth pressure coefficient during consolidation beyond \( \sigma'_{v} = 0.6 \) ksc was \( K_o = 0.47 \). An average peak normalized strength of \( s_u/\sigma'_{vc} = 0.13 \) (60\% lower than in compression) was reached at an average strain of \( e_a = 12.2\% \) (much higher than in compression) and at a friction angle of \( \phi'_p = 35.0^\circ \). The friction angle at maximum obliquity was \( \phi'_{mo} = 35.3^\circ \). At an axial strain of \( e_a = 0.01\% \), the normalized secant shear modulus was \( G_{sec}/\sigma'_{vc} = 551 \), which is about 20\% higher than in compression.
Direct Simple Shear

Three $K_0$-normally consolidated undrained direct simple shear tests (DSS-222, 228, 233) were performed on Series III RBBC by Ortega (1992). The DSS specimens were consolidated to a stress of $\sigma'_{vc}=8.0$ ksc prior to shearing at an average rate of $\dot{\gamma}_a=3.8\%$/hr. The measured undrained shear strength, $s_{UDSS}/\sigma'_{vc}=0.20\pm0.01$ ($s_{UDSS}=\tau_{max}$), was mobilized at an average strain of $\gamma=5.53\pm0.65 \%$. Note that the peak normalized strength in DSS is only 62.5% of the strength measured in triaxial compression ($s_{UDSS}/s_{UTC}=0.625$), but is 54% higher than the strength found in triaxial extension ($s_{UDSS}/s_{UTE}=1.54$). At peak shear stress in the DSS tests, the angle of shear stress obliquity averaged $\psi=20.1\pm1.2^\circ$ [$\psi=\tan^{-1}(\tau/\sigma'_{vc})$]. At large strains ($\gamma \approx 25\%$), the normalized shear stress had dropped to an average of $\tau/\sigma'_{vc}=0.12\pm0.02$, which is 60% of the peak. Measurement of initial shear stiffness is precluded by large system compliance.

3.3 CAISSON TESTING PROCEDURE

The caisson element test procedure comprises the following four separate stages: 1) BBC resedimentation, 2) RBBC consolidation using the rigid top cap, 3) RBBC consolidation in the CET apparatus, and 4) model caisson test event sequences. The test sequence for a model caisson typically includes installation by underbase suction, equilibration (set-up), axial pullout and/or sustained loading. The resedimentation stage takes approximately 16 hours, while RBBC consolidation (stages 2 and 3) requires a minimum of about 6 days. The model caisson testing stage (4) lasts at least one day. Therefore, the total time for one caisson element test from resedimentation through model testing requires at least 8 days. The following describes each of the caisson element test stages in detail.
3.3.1 BBC Resedimentation

The BBC powder is resedimented using the equipment shown in Figure 3.26 (Germaine, 1982). Fifteen kilograms of oven-dried powder is added under vacuum to 15 kg of deaired, distilled, and deionized water to create a soil slurry. Then, about 100 gm of sodium chloride (a flocculant) and 2 ml of phenol (a bacterial growth inhibitor) are added to the slurry. At an initial water content of nearly 100%, these components are combined with mixing blades rotating at approximately 60 rpm in the upper chamber, which is isolated from the lower chamber. After all the components are added, the slurry is mixed at approximately 120 rpm for 30 minutes. Then, the valve between the two chambers is opened, and the slurry is sprayed through the lower free-fall chamber and into the consolidometer. The entire resedimentation process, from equipment set-up to application of the first consolidation load, takes about 16 hours.

3.3.2 RBBC Consolidation Using Rigid Cap

The slurry is consolidated to a maximum vertical stress of $\sigma_v=0.5$ ksc using a rigid piston top cap and incremental loads$^{11}$. After removal of the free fall chamber, the slurry is loaded incrementally in the 30 cm diameter consolidation chamber from 0.0313 ksc to 0.5 ksc at a load increment ratio (LIR) of one. Thus, the loading schedule requires five increments. To allow full primary consolidation and some secondary consolidation, each increment must be applied for about 48 hours. Therefore, the time required to consolidate the clay with the rigid top cap is approximately 10 days. This time can be reduced by allowing little or no secondary consolidation during each increment. In this case the increment duration is between 9 and 20 hours, depending upon the load level, and the rigid top consolidation time is reduced to two days. Throughout this consolidation phase, vertical deformation is measured by a single LVDT located on top of the rigid piston top.

$^{11}$The consolidation of the RBBC slurry using a rigid cap is more commonly referred to as a consolidometer test in previous research (Seah 1959, O'Neill 1985).
cap. After the final increment under the rigid top cap, the clay element and chamber are ready to be positioned beneath the CET superstructure.

3.3.3 RBBC Consolidation in CET Apparatus

After consolidating the sample to 0.5 ksc, the rigid top cap is removed and the CET apparatus is connected to the consolidation chamber. The rigid top cap and one layer of filter paper are removed from the clay surface to reveal the second filter paper, which has a center hole for caisson access. A cloth filter with a center access hole then is placed over the paper filter (see Figure 3.4). The rubber membrane and attached (inner and outer) diameter slip rings, are lowered to the surface of the filter paper. The membrane was previously sealed to the rings with RTV, a silicone rubber adhesive sealant. At this point, the soil is completely covered with the exception of the caisson center access hole. Next, the clay surface displacement LVDT bracket is screwed into the chamber inside wall above the soil surface and the LVDTs are fixed in place. The LVDT wiring and top surface drainage tubing are attached to ports in the cover plate, which is lowered over the inside diameter slip ring and bolted to the chamber top. The superstructure, which is composed of the caisson driving system supported by four 1.8 m long by 1.3 cm diameter threaded rods, is lowered through non-threaded holes in the cover plate, chamber top, chamber bottom, and baseplate. The superstructure and chamber are secured together by hex nuts at the top of the cover plate and bottom of the baseplate. The entire CET cell is leveled using the extension feet in the baseplate. Prior to lowering the caisson to the soil, wall force, cap force, and chamber air pressure zeroes are recorded on the Central Data Acquisition System. By manually operating the caisson wall and cap actuator motor controllers, the cap and wall are brought to the soil surface. At this time, pore pressure probes are inserted through the baseplate and bottom porous stone to a desired depth in the clay and secured to the baseplate with brass connectors.
The probes and the caisson cap porous stone are saturated prior to test set-up using a four step process. After oven drying the cap and probes at 35°C to remove residual moisture, the instruments are evacuated under high vacuum (~10mTorr) for at least one day. They then are flooded with distilled, deaired, deionized water and placed in an ultrasonic vibrating bath for one hour to remove any residual air bubbles. This process has proven to be an effective saturation technique.

After specifying the calibration factors, zero readings, caisson weights, and dimensions as inputs for the program SETUP.BAS, reconsolidation can proceed via computer control of the chamber air pressure and caisson cap and wall actuators. The program CETEST.BAS automatically controls consolidation in the CET apparatus (section 3.1.4) according to a prescribed load increment schedule chosen by the test operator. The current test procedure applies an initial load of 0.0625 ksc, and then reconsolidates the sample in five increments, each with a load increment ratio of LIR=1, into virgin compression range (i.e., \( \sigma'_v \geq 0.50 \) ksc). At this point, the clay can be consolidated further into the virgin range and tested as a normally consolidated sample, or unloaded and tested as an overconsolidated sample. The CET test program described in this thesis uses samples that are normally consolidated to 0.75 ksc using an LIR=0.1 (from \( \sigma'_v = 0.5 - 0.75 \) ksc) to minimize soil extrusion. The maximum load increment is maintained for at least 24 hours prior to the CET event sequence to simulate aging of natural Boston Blue Clay.

3.3.4 Model Caisson Test Events

Once the clay element has been reconsolidated according to the specified stress history, the caisson test event sequence can proceed with caisson penetration, set-up, and axial loading. The test events are initiated by sending control commands to the CETEST.BAS program. Each phase of the event sequence is fully automated, as described in section 3.1.4. At any time during a particular phase, the test operator can
interact with the computer, stop the phase, and either proceed to a different phase or end the test. This flexibility allows the operator to custom-design any test sequence.

The simplest event sequence that involves all phases of a suction caisson element test is a suction driving/set-up/pullout test (standard procedure used in tests CET1 through CET8). In the first phase, the operator selects the suction driving module SUCDRV. Wall penetration rate and target penetration depth comprise the input control for this routine. This program simulates installation by underbase suction by removing load from the caisson cap to balance the force required to penetrate the wall at a constant rate of displacement (to the prescribed depth). Once the caisson walls reach the required depth, the computer automatically switches to the HOLDSTS module, which maintains a constant total load on the caisson while keeping zero relative displacement between the cap and wall. Zero relative cap/wall displacement allows the caisson to displace freely as a monolithic unit. After monitoring the pore pressures within the clay to ensure complete dissipation, the test operator exits HOLDSTS and selects the monotonic pullout module, MONPULL, from the test menu. Input parameters for this module include the wall displacement (withdrawal) rate and target displacement. Once activated, MONPULL performs the axial pullout test. When the caisson has met the displacement target, the computer transfers control to HOLDSTS to maintain constant total force and zero relative cap/wall displacement. The operator then ends the test by exiting HOLDSTS. Once all test phases are finished, the CET cell superstructure, chamber cover plate, and rubber membrane are removed. At this point the clay surface is examined to note the general surface topography and the existence of any unusual features (such as cracks). Then, the soil plug from within the caisson is removed immediately for water content determination. The clay cake is divided into four layers of equal thickness along the height, and soil (~200 g) from each layer is removed for water content determination. Following this, the base of the chamber is dismantled, and the remaining clay is stored in sealed plastic containers for index testing (Atterberg limits, grain size, specific gravity).
The caisson testing sequence of the caisson element test just described requires less than one day. More elaborate sequences including sustained tensile loading have been performed in the current test program (CET9-14).
<table>
<thead>
<tr>
<th>ID #</th>
<th>Device</th>
<th>Measurement</th>
<th>Radial Location from Centerline (cm)</th>
<th>Capacity</th>
<th>Precision</th>
<th>Comments</th>
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</thead>
<tbody>
<tr>
<td>L1</td>
<td>DI Load Cell</td>
<td>Caisson Wall Force</td>
<td>0.0</td>
<td>90.7 kg</td>
<td>0.06 kg</td>
<td>Primary Transducer</td>
</tr>
<tr>
<td>L2</td>
<td>DI Load Cell</td>
<td>Caisson Cap Force</td>
<td>0.0</td>
<td>90.7 kg</td>
<td>0.06 kg</td>
<td></td>
</tr>
<tr>
<td>D1</td>
<td>TT LVDT</td>
<td>Caisson Wall Displ.</td>
<td>0.0</td>
<td>15.2 cm</td>
<td>0.002 cm</td>
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</tr>
<tr>
<td>D2</td>
<td>TT LVDT</td>
<td>Caisson Cap Displ.</td>
<td>0.0</td>
<td>15.2 cm</td>
<td>0.002 cm</td>
<td></td>
</tr>
<tr>
<td>AP</td>
<td>Tyco Pressure Trans.</td>
<td>Chamber Air Press.</td>
<td>(cover plate)</td>
<td>1.76 ksc</td>
<td>0.0002 ksc</td>
<td></td>
</tr>
<tr>
<td>CP</td>
<td>DI Pressure Trans.</td>
<td>Cap Pore Press.</td>
<td>0.0</td>
<td>1.76 ksc</td>
<td>0.0002 ksc</td>
<td>Monitor Transducer</td>
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<tr>
<td>P1</td>
<td>Cooper Pressure Trans.</td>
<td>Clay Pore Pressure</td>
<td>0.0</td>
<td>14.1 ksc</td>
<td>0.01 ksc</td>
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</tr>
<tr>
<td>P2</td>
<td>Kulite Pressure Trans.</td>
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<td>1.8</td>
<td>1.76 ksc</td>
<td>0.0002 ksc</td>
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<tr>
<td>P3</td>
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<td>14.1 ksc</td>
<td>0.02 ksc</td>
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<td>S1</td>
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<td>Clay Surface Displ.</td>
<td>4.2</td>
<td>1.9 cm</td>
<td>0.0004 cm</td>
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<td>S2</td>
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</table>

Notes: Precision based on central data acquisition system with allowance for 3 bits of signal noise.  
DI = Data Instruments  
TT = Trans-Tek  
LVDT = Linear Voltage Displacement Transducer  
HP = Hewlett-Packard

Table 3.1 Characteristics of Instrumentation used for a Typical Caisson Element Test (CET10)
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<th>Year</th>
<th>Researcher</th>
<th>Source Batch</th>
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<th>$w_1$</th>
<th>$w_p$</th>
<th>$I_p$</th>
<th>Clay Frac. &lt;2μm (%)</th>
<th>Organic Content</th>
<th>Salt (g/l)</th>
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<td>Sinfield</td>
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<td>22.4</td>
<td>57.6</td>
<td>11.6</td>
<td>±1.5</td>
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<td>±0.9</td>
<td></td>
<td></td>
<td>±1.1</td>
<td>±1.4</td>
<td>±0.8</td>
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<td>powder,</td>
<td>2.81</td>
<td>46.1</td>
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<td>58.0</td>
<td>11.6</td>
<td>±1.5</td>
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<td>±1.4</td>
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<tr>
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<td>413-417</td>
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Table 3.2  Index Properties of Resedimented Boston Blue Clay Batches and Powder from Series IV Boston Blue Clay
CET COMPONENTS

3. Driving System

2. Model Caisson

1. Consolidation Chamber

 Analog-to-Digital Converter

 Personal Computer

 Digital-to-Analog Converter

4. Control System

5. Instrumentation
- wall force, displacement
- cap force, displacement
- chamber air pressure
- cap pore pressure
- clay cake pore pressure
- soil surface displacement

Figure 3.1 Schematic Diagram of the Caisson Element Test Cell Illustrating the Five Main Components
Figure 3.2  Original Chamber for 1-D Consolidation of Clay Samples
PLAN VIEW

CROSS-SECTIONAL VIEW

Figure 3.3  Modified Consolidation Chamber: Cross Sectional Side and Plan Views
outer membrane slip ring/
surface drainage connection

Figure 3.4  New Consolidation Chamber Top Assembly for Caisson Element Experiments
Figure 3.5  Model Caisson: Cross Sectional Side Views of Wall and Cap Components
Figure 3.6  Caisson Cap with Pressure Transducer: Cross Sectional Side Views
Figure 3.7  Schematic Diagram of the Caisson Element Test Cell with Highlights of the Three Driving Subsystems
Consolidation Chamber Cross Section

Figure 3.8 Total Stress Driving Subsystem
driving system support rods (4)
sliding compression spring plate
compression spring
wall support plate
tension spring guide tube

actuator worm screw
actuator housing, gearbox, and motor
actuator worm screw
force transducer
drivetrain square plate
drive rods (4)
drivetrain circular plate
cylindrical wall connector
wall connector bolts (4)
caisson wall

Figure 3.9  Caisson Wall Driving Subsystem
driving system support rods (4)

compression spring guide tube

sliding compression spring plate

compression spring cap support plate

actuator worm screw

actuator housing, gearbox, and motor

actuator worm screw

force transducer

hollow extension rod

caisson cap (cross section)

Figure 3.10 Caisson Cap Driving Subsystem

NOT TO SCALE
Figure 3.11  Schematic Drawing of the Control System Hardware Components
Figure 3.12  Schematic Drawing of the Control System Software Components
CONS
consolidation

\[ F_w \]

\[ F_c \]

\[ \sigma_v \]

\[ \text{SOIL} \]

\[ \text{AIR} \]

<table>
<thead>
<tr>
<th>Control Axis</th>
<th>Control Feature</th>
</tr>
</thead>
<tbody>
<tr>
<td>Air Pressure</td>
<td>$\sigma_v = \text{constant}$</td>
</tr>
<tr>
<td>Caisson Wall</td>
<td>$D_w = D_c$</td>
</tr>
<tr>
<td>Caisson Cap</td>
<td>$F_c = F_{\text{tot}} - F_w$ ($F_{\text{tot}} = \text{constant}$)</td>
</tr>
</tbody>
</table>

**Legend**

- $F_w$ = wall force
- $F_c$ = cap force
- $D_w$ = wall displacement
- $D_c$ = cap displacement
- $\sigma_v$ = vertical stress

**Figure 3.13** Schematic Drawing of Consolidation Chamber Air/Soil Interface Illustrating the Control Methodology for the CONS Module
HOLDSTS

holding stress:
- equilibration
- sustained load

![Diagram of consolidation chamber air-soil interface](image)

<table>
<thead>
<tr>
<th>Control Axis</th>
<th>Control Feature</th>
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<tbody>
<tr>
<td></td>
<td>wall pen. &lt; 1 cm</td>
</tr>
<tr>
<td>Air Pressure</td>
<td>$\sigma_v = \text{constant}$</td>
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<tr>
<td>Caisson Wall</td>
<td>$D_w = D_c$</td>
</tr>
<tr>
<td>Caisson Cap</td>
<td>$F_c = F_{\text{tot}} - F_w$</td>
</tr>
<tr>
<td></td>
<td>($F_{\text{tot}} = \text{constant}$)</td>
</tr>
</tbody>
</table>

**Legend**
- $F_w$ = wall force
- $F_c$ = cap force
- $D_w$ = wall displacement
- $D_c$ = cap displacement
- $\sigma_v$ = vertical stress

Figure 3.14  Schematic Drawing of Consolidation Chamber Air/Soil Interface Illustrating the Control Methodology for the HOLDSTS Module
**SUCDRV**

suction driving

\[ F_w \]

\[ F_c \]

\[ \sigma_v \]

\[ D_w \]

\[ \Delta t \]

<table>
<thead>
<tr>
<th>Control Axis</th>
<th>Control Feature</th>
</tr>
</thead>
<tbody>
<tr>
<td>Air Pressure</td>
<td>( \sigma_v = \text{constant} )</td>
</tr>
<tr>
<td>Caisson Wall</td>
<td>( D_w/\Delta t = \text{constant} )</td>
</tr>
<tr>
<td>Caisson Cap</td>
<td>( F_c = F_{\text{tot}} - F_w ) ( F_{\text{tot}} = \text{constant} )</td>
</tr>
</tbody>
</table>

**Legend**

- \( F_w \) = wall force
- \( F_c \) = cap force
- \( D_w \) = wall displacement
- \( D_c \) = cap displacement
- \( \sigma_v \) = vertical stress
- \( \Delta t \) = elapsed time

Figure 3.15  Schematic Drawing of Consolidation Chamber Air/Soil Interface Illustrating the Control Methodology for the SUCDRV Module
MONPULL
monotonic pullout

\[
\begin{align*}
F_w & \\
& \uparrow \\
F_c & \\
& \uparrow \\
\sigma_y & \\
& \downarrow \\
\text{AIR} & \\
\sigma_y & \\
& \downarrow \\
\text{SOIL} & \\
D_w & \\
& \downarrow \\
\end{align*}
\]

<table>
<thead>
<tr>
<th>Control Axis</th>
<th>Control Feature</th>
</tr>
</thead>
<tbody>
<tr>
<td>Air Pressure</td>
<td>(\sigma_y = \text{constant})</td>
</tr>
<tr>
<td>Caisson Wall</td>
<td>(D_w/\Delta t = \text{constant})</td>
</tr>
<tr>
<td>Caisson Cap</td>
<td>(D_c/\Delta t = D_w/\Delta t)</td>
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</tbody>
</table>

**Legend**

- \(F_w\) = wall force
- \(F_c\) = cap force
- \(D_w\) = wall displacement
- \(D_c\) = cap displacement
- \(\sigma_y\) = vertical stress
- \(\Delta t\) = elapsed time

Figure 3.16  Schematic Drawing of Consolidation Chamber Air/Soil Interface Illustrating the Control Methodology for the MONPULL Module
Figure 3.17  Schematic Drawing of CET Cell Showing Typical Instrumentation
Package Used for CET Test: Primary Transducers
Figure 3.18  Consolidation Chamber Cross Sectional Side View Showing Typical Instrumentation Package Used for CET Test: Chamber Transducers for CET 10
Top View
(LVDT bracket only)

Cross Sectional Side View
A - A'

Note: LVDT at S' measures at radius of 4.2 cm using elbow extension

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<thead>
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<th>LVDT</th>
<th>Radius (cm) from centerline</th>
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<tr>
<td>S1</td>
<td>5.2 (4.2)</td>
</tr>
<tr>
<td>S2</td>
<td>5.2</td>
</tr>
<tr>
<td>S3</td>
<td>7.7</td>
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<td>S4</td>
<td>9.8</td>
</tr>
<tr>
<td>S5</td>
<td>12.1</td>
</tr>
</tbody>
</table>

Figure 3.19  Consolidation Chamber Cross Section Showing Side and Top Views of Clay Surface LVDTs with Mounting Bracket
poreous stone
- 20 micron
- stainless steel

hollow tube
- $D_l=0.023 \text{ cm}$
- stainless steel

transducer block
- stainless steel

O-ring

threaded coupling
- brass

Kulite pressure transducer
- face diameter
  0.39 cm

NOT TO SCALE

**Intact View**   **Exploded View**

Figure 3.20  Pore Pressure Probe with Kulite Transducer: Cross Sectional Views
Figure 3.21 Consolidation Chamber Baseplate Showing Radial Location of Pore Pressure Probe Ports and a Probe in Place: Cross Sectional Side and Top Views
Figure 3.22 Consolidation Chamber Sidewalls Showing Location of Total Stress and Pore Pressure Ports: Cross Sectional Side and Top Views
Figure 3.23  Grain Size Distribution of Series IV Boston Blue Clay
Figure 3.24  Atterberg Limits for Series IV Boston Blue Clay
a) Plastic and Liquid Limits for BBC IV Powder and RBBC 401-411, 413-417
b) Plasticity Chart with Data from Series IV BBC
Consolidation with Rigid Cap
Compression, Flow, and Consolidation
Average Values

<table>
<thead>
<tr>
<th>Stress Interval (ksc)</th>
<th>n</th>
<th>C_c</th>
<th>m_v (cm^2/kg)</th>
<th>k_v (x10^-8 cm/s)</th>
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<tbody>
<tr>
<td>0.0625 - 0.125</td>
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<td>0.571 ± 0.086</td>
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<tr>
<td>0.125 - 0.25</td>
<td>9</td>
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<td>0.502 ± 0.065</td>
<td>41.6 ± 14.5</td>
</tr>
<tr>
<td>0.25 - 0.5</td>
<td>11</td>
<td>0.525 ± 0.033</td>
<td>0.230 ± 0.031</td>
<td>23.7 ± 3.5</td>
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<tr>
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Figure 3.25 RBBC Consolidation with Rigid Cap: Compression, Consolidation, and Flow Characteristics
Figure 3.26  Schematic Diagram of the Resedimentation Equipment for Boston Blue Clay (from Germaine 1982)
CHAPTER 4
EVALUATION OF CET CELL

This chapter evaluates the Caisson Element Test (CET) apparatus in terms of its ability to: 1) simulate the installation, set-up, and axial tensile loading of suction caissons, and 2) measure the behavior of the model caisson and the test soil during the simulation. Results from the initial test (CET1) revealed numerous limitations that were corrected for subsequent tests. Section 4.1 chronicles the CET cell development throughout the testing program, from CET1 through CET14. (Chapter 3 fully describes the CET apparatus in its final form, prior to tests CET13 and 14.) Sections 4.2 through 4.6 evaluate the five components of the CET cell: consolidation chamber, model caisson, driving system, control system, and instrumentation.

4.1 EQUIPMENT DEVELOPMENT CHRONOLOGY

During the first year of suction caisson research at MIT, from September 1992 to September 1993, the original CET apparatus was designed, fabricated, and proof tested (Whittle and Germaine, 1993). Data from the first test, CET1, conducted in early September 1993, illuminated many of the limitations of the apparatus in simulating suction caisson installation and loading. In order to eliminate, or at least lessen, these deficiencies, significant modifications were made to the CET cell. Subsequent tests revealed other persistent problems, which were solved by further equipment changes. Table 4.1 lists the test date\(^1\) for all 14 CET tests in the testing program and the equipment modifications that

\(^1\)The test date is the first day of model caisson test events, as described in section 3.3.4.
were made to the CET apparatus in between tests. Note that the improvements are categorized according to the five components of the CET cell.

4.1.1 Cylindrical Slip Ring Assembly Modifications

Modifications to the slip ring assembly that covers the clay cake in the consolidation chamber fall into one of three categories: air leak prevention, surface drainage improvement, and caisson/slip ring separation. Air leakage from the pressurized cavity through the slip ring/rubber membrane cover and into the test soil was a chronic problem for tests CET1 through CET6, as evidenced by visual and audible observation. Several attempts were made to stop the leaks including adding more rubber adhesive (RTV) to bond the membrane and slip rings, drilling air leak relief ports in the outer slip ring, and re-machining the outer slip ring to allow better contact with the rubber membrane and more O-ring friction between the outer slip ring and the inner chamber wall. As listed in Table 4.1, these modifications followed tests CET2, 3 and lessened the air leak problem. However, air leakage was eliminated following CET6 by replacing the outer slip ring O-ring seals with "X-rings". As shown in Figure 4.1, the X-ring seals the outer ring to the inner chamber with two points of contact thus doubling the protection against pressurized air penetrating the seal, while still allowing the slip ring/membrane assembly to move vertically. Compared to the O-ring, the X-ring is much more flexible, which enables air pressure to increase the contact pressure.

The first surface drainage improvement, which followed CET1, was placement of a cloth filter over the original paper filter on top of the clay cake in order to encourage radial fluid flow toward the outer ring collection point (see Figure 4.2). Attempts were made to increase surface drainage flow after CET3 by drilling two more drain ports in the outer slip ring, and after CET6 by widening the drainage tubing leading from the outer slip ring to the atmosphere via the cover plate. Unfortunately, these modifications did little to
improve top drainage; this topic is discussed further in section 4.2.2, which evaluates the slip ring assembly as a whole unit.

The final consolidation chamber modifications involved the inner slip ring, which is sealed to the rubber membrane. This ring moves vertically through a port in the cover plate and separates the model caisson from the pressurized air cavity (see Figure 4.3). During model installation in CET2, the model caisson caught the inside lip of the inner slip ring and dragged the ring into the clay cake. This destroyed the slip ring/rubber membrane connection and severely disturbed the clay cake, and the test was abandoned. To prevent a future occurrence of this, the inside lip of the inner slip ring was tapered, as shown in Figure 4.3. During installation in CET7, the caisson wall was in contact with the inside wall of the inner slip ring, a condition which added approximately 5 kg of force to the wall. For tests CET8 through CET14, this problem was solved by placing thin (~0.5 mm thick) shims between the caisson wall and the inside wall of the inner slip ring prior to installation in order to maintain proper caisson alignment. The shims were removed before the start of caisson wall penetration.

4.1.2 Model Caisson Improvements

Figure 4.4 shows the two improvements made to the model caisson: removal of the taper at the wall tip and reduction of the O-ring friction between the cap and wall. The original caisson wall was a standard 1" diameter Shelby tube with a tapered tip. Following CET1, the taper was removed to create a blunt end, which better simulates prototype caisson wall tips. After CET2, the O-ring groove on the caisson cap was deepened in order to reduce the O-ring "squeeze" between the cap and wall, which in turn dropped the cap/wall friction from ±5 kg to ±1 kg. The subject of cap/wall friction is discussed further in section 4.3.2.
4.1.3 Driving System Modifications

Section 3.1.3 described the driving system, which is composed of the total stress, cap, and wall driving subsystems. The most serious mechanical problem during CET1 was the limitation on the stroke of the cap actuator during the pullout phase. The original design used a pressure-volume controller to supply oil to an hydraulic actuator. This system was completely replaced by a ball screw actuator, identical to the control mechanism for the caisson wall.

Following CET5, tighter control over caisson forces and displacements were achieved by two modifications to the driving systems. First, to allow the actuator to respond to very small control signals from the computer, the gear reduction between the electric dc motor and the linear ball screw actuator was increased by a factor of ten. In addition, to remove the actuator compliance (or lashback) arising from changes in direction of the worm screw, compression springs were added to both the wall and cap drive systems to maintain a constant upward force on the worm screw throughout all phases of testing (refer to Figures 3.9 and 3.10).

4.1.4 Control Software Changes

Numerous modifications were made to the control software modules (SUCDRV, HOLDSTS, MONPULL) during the course of the testing program in order to maintain complete control over cap and wall force and displacement. The final software system was described fully in section 3.1.4. SUCDRV, which is used to control the driving system during caisson installation with underbase suction, underwent serious modification early in the testing program. In CET1, the algorithm was unable to match the reduction in the cap force to the increase in wall force. The original control algorithm used feedback control on the total force $F_{tot}$ to compute the required amount of cap force reduction. The control signals for the cap actuator were sent for a fixed time interval of 1 second in each loop, which was found to be ineffective. For CET3, the system used feedback
control on the total force $F_{\text{tot}}$ and the rate of change in wall force ($F_w/t$), with continuous adjustment of the cap force $F_c$. These modifications reduced fluctuations in total force $F_{\text{tot}}$ during installation from $\pm 39$ kg in CET1 to $\pm 5$ kg in CET3. Further proof testing showed feedback control on the rate of wall force ($F_w/t$) was unnecessary. Hence, for all tests beyond CET3, the algorithm used proportional feedback control on the total force $F_{\text{tot}}$ with continuous adjustment of the cap force $F_c$, as described in section 3.1.4.

HOLDSTS controls the equilibrium (setup) and sustained tensile load phases, both of which are simulated by maintaining a constant total force on the caisson and zero relative displacement between the cap and wall. Three major modifications were made to HOLDSTS. The original algorithm was not able to hold a constant total force on the caisson. The algorithm in CET1 used proportional feedback control on the cap force and wall displacement to set a constant total caisson force $F_{\text{tot}}$, and zero relative displacement between cap and wall. The control signals for the cap and wall actuators were sent for fixed time intervals of 1 second in each loop. However, friction between the cap and the wall caused unacceptably large total force fluctuations, $\Delta F_{\text{tot}} = \pm 20$ kg. This problem was solved in CET3 by reducing the cap/wall friction (as described in section 4.1.2), using feedback control on the cap based on the total caisson force, and providing a keyboard "toggle" to allow manual adjustment of the cap force gain factor during testing. These modifications reduced fluctuations of total force to within $\Delta F_{\text{tot}} = \pm 1.0$ kg.

Following CET5, the HOLDSTS module was streamlined by providing continuous control of the wall and cap actuators, keyboard toggles for gain factors, and a "stiffness trigger" to allocate force control over the stiffer component of the cap/wall system (see section 3.1.4). The stiffness trigger initially helped overcome the large actuator compliance associated with low load levels. However, actuator compression springs (see section 4.1.3) significantly lessened compliance, and therefore eliminated the need for the stiffness trigger.
The final improvement to HOLDSTS came following CET9. In order to maintain zero relative displacement control during sustained tensile loading, an integration term was added to the original proportional term for the command signal (see section 3.1.4).

Major modifications were made to the pullout algorithm, MONPULL. During CET3, constant control signals were sent to the cap and wall driving systems to withdraw the caisson at a constant predetermined rate. These signals were calculated based on the stiffness of each actuator, which was determined during proof testing, and were not changed throughout pullout. Due to the difference in actuator stiffness between proof testing and actual test conditions, the caisson wall and cap were pulled at different rates, which caused a relative displacement. This problem was reduced significantly in tests CET4,5 by performing proportional feedback control for the cap displacement rate and both proportional and integral control on the wall to maintain zero relative displacement. While the target tolerance for the relative displacement is 0.001 cm, this modification improved the relative displacement during the first 0.3 cm of pullout from 0.1 cm in CET3 to 0.01 cm in tests CET4 and 5. With the addition of compression springs to reduce actuator compliance (see section 4.1.3), the relative displacement during the first 0.01 cm of pullout was reduced to less than 0.003 cm in CET6.

4.1.5 Instrumentation Improvement

Inadequate time response and total stress sensitivity of the stainless steel probes used to measure pore pressures within the clay necessitated several probe modifications following CET1. Figure 4.5 shows the probes used for CET1 through CET5. The probes did not respond as expected during the pullout phase of CET1. This behavior may have been associated with soil forming a plug, rather than a filter, at the tip of the needle (and, hence, causing the probe to act as a total stress sensor). Better response during CET3 was achieved by crimping the tip of the probe, such that the aperture diameter decreased from 0.023 cm to 0.011 cm. During CET4, it was discovered that the Motorola
transducers attached to the bottom of the needle probes were incompatible with water; long term fluid contact caused unstable voltage output due to diffusion of water through the gelatin seal used to protect the electronics. For CET5 the probes were redesigned to fit much larger, water-compatible transducers (Data Instruments). These showed excellent response when tested in water, but when placed in clay, the probes responded very sluggishly. As described in section 4.6.2, the poor response of the probes in CET5 prompted a lengthy theoretical and experimental study of the design of pore pressure probes. For CET6, 20 micron porous stones were press-fitted into the 0.023 cm diameter probe tip to prevent soil plugging and one Data Instruments transducer was replaced with a less compliant Cooper transducer. Research involving tests of the probes in triaxial clay specimens revealed that the Cooper probe responded 70 times faster than the Data Instruments probe. Following CET6, the remaining two Data Instruments transducers were replaced with Kulite transducers. The Kulite transducers responded 140 times faster than the Data Instruments transducers.

Sluggish response of the cap pore pressure sensor in CET1 was attributed to inadequate saturation of the pore pressure stone, while the pressure transducer failed during equipment setup in CET3. For CET4 through CET14, good pore pressure response was achieved by fitting the cap with a new Data Instruments transducer and using a reliable pore pressure stone saturation technique (see section 3.3.3).

The final instrumentation improvement was the addition of a fifth clay surface displacement transducer prior to CET4. As discussed in section 3.1.5, an elbow extension on the LVDT core at the fifth radial location enabled measurements close to the caisson wall (r=4.2 cm).
4.2 CONSOLIDATION CHAMBER

Evaluation of the CET cell begins with the consolidation chamber, which has two main areas of concern: 1) the effect of the rigid wall boundary during installation, set-up, and axial tensile loading, and 2) the ability of the cylindrical slip ring assembly to prevent air leaks, provide surface drainage, and allow caisson passage.

4.2.1 Boundary Total Stress

Ideally, the rigid-walled cylindrical chamber that contains the clay cake should be large enough so that penetration and pullout behavior of the model caisson is not affected by the proximity of the rigid boundary. Measurements of total stress on the chamber sidewall during CET6 indicate that the model caisson events in the clay cake did cause total stress changes at the boundary. Figure 4.6 shows a cross section of the consolidation chamber for CET6 with the total stress transducer on the rigid boundary. Note that the total stress\(^2\) was measured on the sidewall approximately 11.7 cm above the rigid porous bottom plate and 2 cm below the clay cake surface prior to caisson installation.

Figure 4.7 shows the horizontal total stress record for all five phases of CET6: suction installation, post-installation equilibration, monotonic pullout 1, re-equilibration, and monotonic pullout 2. Prior to installation, the horizontal total stress is \(\sigma_h = 0.27\) ksc. Assuming there is no excess pore pressure and the clay cake has a uniform vertical effective stress prior to installation, then the horizontal effective stress is \(\sigma'_h = 0.27\) ksc and the vertical effective stress is \(\sigma'_v = 0.75\) ksc, which yields a lateral stress ratio of \(K = 0.36\). This ratio is low compared to previous data (typically in triaxial testing, \(K_0 = 0.4-0.5\) during virgin compression; Sheahan, 1991). Soil arching prevents the full transmission of horizontal stress to the total stress sensor in the chamber sidewall. Hence, the sensor

\(^2\)Total stress was also measured in CET1 and 2, but severe problems during these tests preclude a presentation of total stress measurements (see section 5.1).
measures a lower stress. This soil arching phenomenon has been observed in other devices in which sidewall total stress is measured, such as the K Cell (Ladd, 1965).

After the caisson wall has penetrated $z_w=5.1$ cm, the total force increases slightly by $\Delta \sigma_h=0.025$ ksc to reach $\sigma_h=0.295$ ksc. During the subsequent equilibration phase, the horizontal total force falls back to approximately $\sigma_h=0.26$ ksc. This suggests that the slight total force buildup during penetration is due to a positive change in excess pore pressure, which dissipates during equilibration at this boundary location. It is important to note that the 'cycling' pattern measured by the total force sensor (approximately 3.5 cycles per 1cm during penetration) is caused by the cycling air pressure, which is applied to the surface of the clay cake\(^3\). In Figure 4.8, which presents the air pressure record for all five phases of CET6, it is apparent that although cycling exists throughout the test, cyclic magnitude is small ($P_a=0.75\pm0.02$ ksc).

During both phases of monotonic pullout, the horizontal total force decreases by approximately $\Delta \sigma_h=-0.04$ (from $\sigma_h=0.26$ to 0.22 ksc during pullout 1 and from $\sigma_h=0.25$ to 0.21 ksc during pullout 2). During the intervening re-equilibration phase, the total force rises to nearly $\sigma_h=0.34$ ksc before falling to $\sigma_h=0.25$ ksc. As with installation, this behavior suggests that pullout and the initial portion of re-equilibration induce at the chamber sidewall a slight excess pore pressure, which dissipates by the end of re-equilibration.

A comparison of the total stress decrease ($\Delta \sigma_h=-0.04$ ksc) with the induced excess pore pressure within the soil plug ($\Delta u<-0.4$ ksc, see Figures 5.41, 5.74) shows that the total stress decrease is less than 10% of induced negative excess pore pressure within the caisson\(^4\). Given the magnitude of air pressure cycling ($\pm0.02$ ksc), the boundary effect is acceptably small.

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\(^3\)The pore pressure probe measurements also reflect the air pressure cycling.

\(^4\)A comparison of boundary total stress versus near-caisson excess pore pressure during installation is precluded by the countering effects of cap suction and wall penetration.
4.2.2 Cylindrical Slip Ring Assembly

The slip ring assembly is comprised of a drainage filter, rubber membrane, inner membrane slip tube, and outer membrane slip ring (see section 3.1.1). Evaluation of this assembly centers on its ability to maintain total stress without air leaks, allow clay surface drainage, and permit caisson passage.

Air leaking from the pressurized chamber cavity through the slip ring assembly prevents the complete dissipation of positive excess pore pressure within the clay cake, de-saturates the clay cake, and potentially dries out the clay cake surface. Hence, constant vigilance was necessary throughout the testing program. Air leaks were detected in tests CET1 through CET6 by a combination of one or more of the following signs: slight rise in clay cake pore pressure\(^5\), audible bubbling noise emitting from the annulus between the inner slip tube and caisson wall, and visible air bubbles emanating from the surface drainage collected in the slip tube/caisson wall annulus and from the drainage port connected to the clay cake bottom. Figure 4.9 shows the detected air leak path, which starts from the pressurized chamber, and penetrates through the slip ring assembly at the following locations: a) between the side of the outer slip ring and the chamber sidewall, b) between the rubber membrane and the bottom of the outer slip ring, and c) between the rubber membrane and the bottom of the inner slip ring. Once the air has penetrated the slip ring assembly, the air travels radially beneath the rubber membrane to the annulus between the inner slip ring and the caisson wall. The air then rises vertically in the annulus and into the atmosphere. As described in section 4.1.1, modifications to the slip ring assembly blocked these air leak routes; tests CET7 through CET14 showed no overt signs of air leakage.

The slip ring assembly was designed to permit free drainage through the surface of the clay cake. Analysis of the consolidation results from the first test CET1 revealed that the drainage rate through the slip ring assembly was slower than the drainage rate through

\(^5\)The pore pressure rise due to air leaks never exceeded \(\Delta u=0.05\) ksc.
the rigid top cap used in standard RBBC consolidation (Whittle and Germaine, 1993). Attempts to improve the drainage through modification of the slip ring assembly following tests CET3 and 6 (see section 4.1.1) did not yield consistent results.

In order to show that the drainage conditions were not consistent, comparisons were made between the measured consolidation data and one-dimensional consolidation theory. Figure 4.10a shows the time curves (vertical strain vs. log time) for consolidation in the CET apparatus during the load increment in the virgin range from $\sigma'_v=0.67$ to $0.73$ ksc ($LIR=\Delta\sigma_v/\sigma_i=0.1$) for tests CET3-136. Figure 4.10a also shows the theoretical time curves based on one-dimensional consolidation theory (Terzaghi, 1923) for single and double drainage conditions. The shape of the CET time curves approximates that of the theoretical curves, and the measured rate of compression during CET consolidation exceeds the typical secondary compression rate ($C_{\alpha \varepsilon}=0.0012$; RBBC217) for normally consolidated RBBC. Hence, the load increment ratio appears to be large enough to produce a "Type I" curve (Leonards and Altschaeffl, 1964). However, additional data, including longer load application duration, would be required to verify this assertion.

Based on Figure 4.10a, it is clear that the drainage conditions during consolidation in the CET cell is not consistent from test to test, but varies roughly between single and double drainage behavior. Figure 4.10b shows that the measured time curves for two load increments ($LIR=1$; RBBC217) during incremental batch consolidation under the original rigid top cap closely approximate the 1D theoretical curve for double drainage conditions7. The lack of consistent double drainage during consolidation in the CET apparatus indicates that the slip ring modifications to improve drainage did not work, and there are other obstacles preventing free surface drainage. Note that the only serious consequence of slower drainage rates is increased consolidation times.

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6During consolidation in the CET apparatus, a very low load increment ratio ($LIR=0.1$) was used to prevent soil extrusion, and little or no secondary compression was allowed in order to expedite testing (see section 3.3.3).

7The measured RBBC217 curves differ slightly from each other and from the 1D double drainage curve due to differences in the coefficient of consolidation.
The last slip ring assembly function subject to evaluation is the ability of the assembly to allow vertical passage of the caisson wall into the clay cake. As mentioned in section 3.1.1, the inner slip ring allowed adequate caisson passage in all tests except two. During penetration in CET2, the wall caught the inner lip at the bottom of the ring and destroyed the slip ring/membrane connection; the inside bottom lip of the inner lip ring was tapered for all subsequent tests (see Figure 4.3). In CET7, the caisson wall was not concentric within the inner ring, but was in contact with the inside bottom edge of the ring; in the tests following CET7, shims were used to ensure concentric wall placement prior to installation.

4.3 MODEL CAISSON

The unique design of the two-component model caisson and its position relative to the inner slip ring leads to two major areas of concern: 1) the stability of the soil surface through soil arching between the cap and wall and between the wall and the inner slip ring, and 2) friction arising between the wall and inner slip ring and between the cap and wall.

4.3.1 Clay Surface Stability Through Soil Arching

Figure 4.11 shows the clay cake surface with the slip ring assembly and model caisson in position just prior to the penetration phase. There is an annular gap of 0.07 cm between the cap and wall and an annular gap of 0.01 cm between the wall and inner slip ring. Since the clay surface in these two gaps remains uncovered throughout model caisson testing, the surface stability must be maintained through soil arching. During the consolidation phase of the test, the slip ring assembly and model caisson apply a compressive total stress of $\sigma_v=0.75$ ksc to the clay cake. Because of consolidation, the soil beneath the slip ring assembly, wall tip, and cap compresses in response to this load,
but the soil directly beneath the annular gap tends to yield upward. Hence, there is relative movement between the two soil masses. Excessive upward movement of the soil into the annular gap is opposed by shearing resistance in the zone of contact between the two moving soil masses. This leads to a transfer of pressure from the limited zone of upward yielding soil mass to the much larger zone of compressing soil. This transfer of pressure from the upward-yielding soil to the downward-yielding soil on either side is called the arching effect because the soil arches over the annular gap soil. Stability of the annular gaps during other phases of the test (penetration, sustained load, monotonic pullout) also can be explained by the soil arching concept.

From visual observations made during and after testing, annular gap stability was only a problem for test CET1. During consolidation in the CET apparatus, load was applied using a load increment ratio of LIR=1. Excessive soil extrusion between the wall and inner slip ring (~0.6 cm) and between the wall and cap (2 cm) proved that the load increment ratio was excessively large as the clay cake was consolidated into the virgin compression zone (\(\sigma'_{\text{v}}\) = 0.5 to 1.00 ksc). Thereafter, for tests CET2-14, the load increment ratio was lowered to LIR=0.1. In these tests there was no soil extrusion between the wall and inner slip ring and only minimal extrusion (1-3 mm) between the wall and cap.

4.3.2 External and Internal Model Caisson Friction

During the model caisson test events, the proximity of the inner slip ring to the caisson wall and the connection between the cap and wall lead to frictional force contributions to the wall and cap force records. Figure 4.12 shows the clay cake surface prior to penetration in order to illustrate the four locations where friction arises: 1) metal-to-metal contact between the inner slip ring and the caisson wall, 2) extruded soil between the inner slip ring and the caisson wall, 3) extruded soil between the cap and wall,

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8This is the "soil arching" phenomenon defined by Terzaghi (1943).
and 4) O-ring connection between the cap and wall. The first two sources of friction are 'external' and contribute force to both the wall and total force records. For CET 3-14, visual inspection of the inner slip ring following the conclusion of each test verified that no soil had extruded between the inner slip ring and caisson walls; this source of external friction was considered insignificant. However, as mentioned in section 5.2.1, wall force records during wall penetration indicate the likelihood of metal-to-metal contact between the caisson wall and inner slip ring in tests CET7 and 10. In these two tests, this external source of friction contributed 4-5 kg of friction to the wall force (see Figure 5.6). Total force variations in the remaining tests indicate external friction on the order of ±1.5 kg.

The last two sources of friction are 'internal' and do not affect the total force. Internal friction contributes force to the wall force signature, which is balanced by an equal but opposite contribution to the cap force. The only caisson testing phase that is seriously affected by internal friction is caisson installation, during which the wall moves at a constant rate past the cap. Visual inspection of the model caisson following tests CET3-14 revealed approximately 1 to 3 mm of extruded clay between the cap and wall. This amount of extruded clay contributes between 0.2 and 0.7 kg of friction to the wall and cap when the two components are in relative motion\(^9\). Following CET2, proof testing revealed that the O-ring connecting the wall and cap contributed ±1 kg of internal friction\(^\text{10}\). The two sources of internal friction combine to contribute as much as ±1.7 kg to the wall and cap force records. As discussed in section 5.2.1, internal friction is the most likely cause of the 'transition zone' behavior during caisson installation. As indicated by the measured wall force for tests CET3-14 (Table 5.5a), the internal friction at the transition zone 'peak' ranges from 0 to 6.3 kg, with an average 1.6±2.1 kg. The higher

\(^9\)The extruded soil resistance was computed by assuming an undrained strength ratio of \(K_o\)-normally consolidated RBBC in direct simple shear, \(\frac{\sigma_{ud}}{\sigma_{vc}}=0.2\) (Ladd 1991).

\(^\text{10}\)Prior to CET3, the O-ring connection contributed as much as ±5 kg of internal friction (see section 4.1.2).
values may be due to higher initial O-ring friction caused by non-uniform lubrication between the O-ring and the caisson wall.

4.4 DRIVING SYSTEM

Evaluation of the driving system centers on the ability of the mechanical driving subsystems (chamber air pressure, wall drive, cap drive) to respond to control signals sent by the computer and maintain the desired target values. In these three driving subsystems, there is a certain amount of compliance, which delays the mechanical system response. As mentioned in section 4.1.3, compression springs maintained tension in the wall and cap ball screw actuators, and this eliminated much of the compliance in these two driving systems. The total stress driving system had the relatively simple task of maintaining a constant air pressure in the chamber throughout testing; compliance in the electro-mechanical system was insignificant and ultimately tempered by the total stress control algorithm, which is evaluated in the next section.

4.5 CONTROL SYSTEM

The control system uses the four software algorithms11 described in section 3.1.4, to manage the caisson test event sequence. This section presents quantitative evidence of the effectiveness of the algorithms in maintaining: 1) constant total stress on the clay surface throughout testing, 2) constant total force on the caisson during suction installation, 3) constant total force and zero relative displacement during equilibration and sustained loading, and 4) constant rate caisson withdrawal during monotonic pullout.

11CONS, HOLDSTS, SUCDRV, MONPULL
4.5.1 Constant Total Stress

For CET2 through 14, the control system was able to maintain a constant total stress (air pressure) of $\sigma_{vc}'=0.75\pm0.04$ ksc on the surface of the clay cake. As an example of a typical total stress record, Figure 4.13 shows the air pressure versus log of time for the installation, equilibration, and pullout phases of CET8. Note that, with the exception of the initial portion of installation and pullout\(^{12}\), the air pressure was maintained at the target stress of $\sigma_{vc}'=0.75$ ksc with a cyclic variation of $\pm0.04$ ksc; this variation, which has a period of 50 seconds, was acceptable for this testing program.

4.5.2 Constant Total Force During Installation by Underbase Suction

During suction driving, the wall penetrates at a constant rate, while increments of load are removed from the cap to balance load increments picked up by the wall. Hence, the total force ($F_{tot}=F_w+F_c$) is maintained constant. Figures 5.4 and 5.5, which are discussed in section 5.2.1, depict the total force versus wall tip penetration for tests CET3-12. With the exception of CET3\(^{13}\), the total force was held constant at or within 2 kg of the target value of $F_{tot}=15.2$ kg. This small variation, which can be traced to slight target adjustments in the control algorithm, indicates very good control.

4.5.3 Constant Total Force During Equilibration and Sustained Load

During the equilibration and sustained load phases of caisson testing, the control features are identical: a constant total force is maintained with zero relative displacement between the cap and wall. Measurements during equilibration for tests CET4 through 14 indicate that within 3 minutes of the start of set-up, the total force reached the target of $F_{tot}=15.2$ kg and maintained this level to within $\pm0.3$ kg for the remainder of equilibration (see Figure 5.27). Likewise for the sustained load tests (CET9-14), the target tensile load

\(^{12}\)Air pressure drifts slightly during the transition from one module to the next.

\(^{13}\)Significant SUCDRV algorithm improvements were made following CET3 (see section 4.1.4)
levels, which ranged from $F_{tot} = -2.2$ to -12.9 kg, were held to within ±0.3 kg (see Figures 5.46-5.50).

Zero relative displacement control was excellent for both equilibration and sustained load phases. Cap and wall displacement measurements indicated that the relative displacement was no more than ±0.0015 cm. However, because these phases usually required at least 24 hours to complete, electrical power surges occasionally disrupted the otherwise tight displacement control.\(^{14}\)

4.5.4 Constant Rate Withdrawal During Monotonic Pullout

The key measurement to evaluate the effectiveness of constant rate withdrawal during monotonic pullout is relative displacement between the cap and wall. As described in section 3.1.4, during pullout the control algorithm sends a constant signal to move the wall at a constant rate and continually adjusts the cap rate to match the wall rate; ideally, there should be no relative displacement between the cap and wall, as the caisson moves upward as one monolithic unit. Following extensive driving and control system improvements (see section 4.1.3 and 4.1.4), measurements revealed that the relative displacement was less than ±0.003 cm for tests CET6-14. Considering the relatively high stiffness of the caisson/soil system at the start of pullout\(^{15}\), this low relative displacement represents excellent control.

4.6 INSTRUMENTATION

For each caisson element test, an array of up to 14 measuring devices collected data. Five 'primary transducers' (chamber air pressure, caisson wall force and

---

\(^{14}\)Electrical surges during some tests caused a relative displacement of no more than ±0.01 cm, after which excellent relative displacement control resumed.

\(^{15}\)Figure 5.37 plots the total force vs. wall tip displacement during early pullout.
displacement, and caisson cap force and displacement) served a dual purpose by providing data for the control system and for test interpretation, while the remaining sensors (cap pore pressure, clay cake pore pressure, and clay cake surface displacement) monitored parameters of interest that affected the caisson and clay performance during a caisson element test. All instrumentation was checked to ensure proper resolution and precision for the testing program\textsuperscript{16}. In addition, each sensor was calibrated for accuracy. Section 3.1.5 described the physical characteristics, function, and location of this instrumentation. This section briefly evaluates the accuracy of the instrumentation. Section 4.6.1 covers the primary sensors and the clay surface LVDTs, section 4.6.2 evaluates the pore pressure probes, and section 4.6.3 discusses the cap pore pressure sensor.

4.6.1 Primary Transducers and Clay Surface LVDTs

The accuracy of the caisson force and displacement transducer measurements is largely dependent upon the compliance of the cap and wall drivetrains. As described in section 3.1.3, the wall drivetrain comprises the ball screw actuator, the wall force transducer, an aluminum square plate, four steel rods, an aluminum circular drive plate, an aluminum cylindrical connector, and then the caisson wall itself (see Figure 3.9). Under a compressive load, all of these components will compress. The compliance is the amount of drivetrain compression for a given increment of compressive load (or the amount of extension for a given tensile load increment). This compliance introduces a bias in the displacement measurements during the various phases of model caisson testing. But because the test material (normally consolidated RBBC) was soft relative to the drivetrain compliance, this measurement bias remained insignificant. Compliance also is inherent to the cap drivetrain, but like the wall drivetrain, the measurement bias is very small. The fifth primary transducer, the chamber air pressure sensor, is located in the plate covering

\textsuperscript{16}Table 3.1 lists the instrumentation location, capacity, and precision for a typical caisson element test (CET10).
the consolidation chamber, directly measures chamber pressure, and, therefore, does not suffer any loss of accuracy.

The clay surface LVDTs have moving cores that rest directly on the rubber membrane, which covers two thin filters that overlie the soil surface (see section 3.1.1). Because the rubber membrane and filter system have a very thin combined thickness (<0.07 cm), and the chamber air pressure remains constant throughout testing, compliance is not a problem. Hence, the surface displacement measurement accuracy is not seriously compromised.

4.6.2 Pore Pressure Measuring Devices

As described in section 4.1.5, following CET5, the pore pressure probes were redesigned to fit water-compatible transducers (Data Instruments) and showed excellent time response when tested in water. Experiments were then performed with the probe penetrated into a triaxial specimen. Measurements in a free-draining fine sand showed that the probe was insensitive to changes in total stress. However, when placed in clay, the probe responded sluggishly. This prompted a major theoretical and experimental study of the design of pore pressure probes to understand the factors that contribute to probe response and to determine exactly what type of probe would be required to accurately measure pore pressure changes during a caisson element test. The physical characteristics of the latest generation CET pore pressure probe, which consists of a Kulite pressure transducer, a transducer block, and a stainless steel tube, were presented in section 3.1.5. The time response study evaluated three different transducers with different transducer face diameters and deflections (Data Instruments, Cooper, and Kulite transducers). Table 4.2 lists the physical characteristics for each of these sensors.
Probe Response in Water

To gauge the effectiveness of the saturation procedure and to obtain values for comparison with the probe response in soil, the probe response in water was investigated first. A theoretical time response equation was developed for a saturated pore pressure probe subjected to a unit step pressure increase by Henderson (1994):

$$\frac{P_0}{P_a} = (1 - e^{-bt}) ; \quad b = \frac{kA}{\gamma_w LM} ; \quad M = \frac{\Delta V(t)}{\Delta P(t)}$$

where: $P_0 = \text{probe output pressure}$
$P_a = \text{applied pressure}$
$t = \text{time}$
$k = \text{probe tip hydraulic conductivity}$
$A = \text{probe tip cross-sectional area}$
$\gamma_w = \text{unit weight of water}$
$L = \text{probe tip length}$
$M = \text{probe system compliance}$
$\Delta V(t) = \text{change in volume of probe system}$
$\Delta P(t) = \text{change in probe system pressure}$

This equation was used to calculate the time response of the three different probes used in this study. All the variables in the response equation are equal for the three transducers except $M$, the probe compliance. The greatest contribution to probe compliance is the transducer face compliance, which is due to the face deflection in response to a pressure change. The predictions show that the probe with the slowest response (Data Instruments) also has the greatest face deflection (see Table 4.2). Given equal probe geometries, the only factor other than compliance that could cause variation among the different probes is the hydraulic conductivity of the porous stone in the tip. Figure 4.14 shows the theoretical time response in terms of pressure normalized by the step increase versus time. The Kulite has the fastest predicted response, registering 95% of the applied pressure within 0.0008 sec. The Cooper registers 95% within 0.004 sec., while the Data Instruments registers 95% within 0.17 sec.
The response time for each probe in water was measured by placing the probe into a sealed chamber of water, applying a step increase of pressure to the chamber water, and recording the pressure increase in the probe. Table 4.3 shows that the measured response times are slower than the calculated response times. Note that the Kulite and Cooper probes had response times that were quicker than the data acquisition system was able to record for a monotonic increase in chamber pressure. Therefore, a cyclic pressure was applied to these probes and the response time was based on the phase lag between the applied and measured cyclic pressures. Comparisons are only possible for the Data Instruments probe, and they show that the measured response is about three times as slow as the computed value.

Probe Response in Soil

In order to validate the accuracy of pore pressure probe measurements during a CET test, it was necessary to test the probe response in soil under controlled conditions. A theoretical model was developed to calculate the response time of a probe inserted into the middle of a triaxial specimen of clay, which is then subjected to a hydrostatic stress increase. Based on closed form solutions for pore pressure probes with rigid spherical porous inclusion stones in an elastic soil (DeJosselin De Jong, 1953; Gibson, 1963; Kutter, 1990), an equation was developed to model the three probes used in the water response tests:

\[
\frac{P_0}{P_a} = (1 - e^{-dt}) ; \quad d = \frac{4 \pi r k_s}{\gamma_w M} ; \quad M = \frac{\Delta V(t)}{\Delta P(t)}
\]

where: \( r = \) radius of the porous tip
\( k_s = \) soil hydraulic conductivity
The calculated response for each of the probes is presented in Figure 4.15 for a given soil hydraulic conductivity, $k_s = 1.4 \times 10^{-7}$ cm/s. As for the response in water, the results show that for the given soil parameters, as the probe compliance increases, response time increases. The calculated response times at 95% of the applied pressure for the Kulite, Cooper, and Data Instruments probes are 1 sec, 5.1 sec, and 212 sec, respectively. Note that the Kulite, which is the least compliant, has the quickest response. Most importantly, though, the calculated probe response time in soil is more than three orders of magnitude larger than the response time in water. The effect of varying hydraulic conductivity on the Kulite response is shown in Figure 4.16. It is clear that the response time is proportional to clay hydraulic conductivity.

The measured response in clay was conducted as follows. The probe was inserted into a triaxial clay specimen, and the system was backpressure saturated. Then, the specimen was hydrostatically consolidated to an effective stress of 0.75 ksc. An increment of hydrostatic pressure was applied and the pore pressure response was measured in the probe and in the top of the specimen. Figure 4.17 shows the response time of all three probes. Note that the measured pressure increment never equals the applied increment (i.e., the normalized pressure never reaches 1), which is most likely due to system compliance. The results confirm the theoretical predictions that the more compliant the probe, the slower the response. Note that the Kulite, Cooper, and Data Instruments probes measured 95% of the applied pressure within 4, 10, and 710 seconds, respectively. These response times are higher than the theoretical values (1, 5.1, and 212 seconds, respectively) by approximately the same ratio (see Table 4.3).

**Probe Response Conclusions**

The probe response study yielded several conclusions. Overall, the most important probe characteristic is the transducer compliance; the higher the compliance, the slower the probe response. The measured response in soil and water is slower than the
theoretical response by a factor of between 2 to 4 due to compliance in the measurement system and, possibly, uncertainties in the soil and porous stone hydraulic conductivity values. However, the theoretical equations can be validated by comparing the ratio of the response times for soil and water for the Data Instruments probe (Table 4.3). For both the theoretical and the measured response times, the ratio of probe response in soil to probe response in water is approximately the same, as shown in the following:

$$\text{Theory: } \frac{t_{95(\text{soil})}}{t_{95(\text{water})}} = \frac{212}{0.17} = 1247$$

$$\text{Measured: } \frac{t_{95(\text{soil})}}{t_{95(\text{water})}} = \frac{710}{0.6} = 1183$$

This indicates that the theoretical expressions used to describe response time for the probes are sound. For the Kulite and Cooper probes, the lack of rapid response data acquisition prevented the collection of measured response times in water. Based on applied cyclic pressure and recorded phase lags, Table 4.3 lists the measured response times in water as $t_{95} < 0.05$ and $t_{95} < 0.1$ seconds for the Kulite and Cooper probes, respectively. However, assuming the theoretical soil to water response time ratio is approximately equivalent to the measured soil to water ratio, the measured response time in water for the two probes would be $t_{95} = 0.0032$ and $t_{95} = 0.0078$ seconds, respectively.

For the CET tests, the probes were positioned in the clay sample, which was normally consolidated at an effective stress of $\sigma'_{vc} = 0.75$ ksc. Based on previously published data, RBBC at this stress level has a hydraulic conductivity of $k_s = 1.2 \times 10^{-7}$ to $2.0 \times 10^{-7}$ cm/s, which is consistent with the value used in the theoretical response calculation (Sheahan, 1991; Seah 1990). Hence the Kulite, Cooper, and Data Instruments probes should have a response time of approximately 1.0, 5.1, and 212 seconds, respectively, during CET testing. For the rates of caisson movement during the
installation \((z_w/t=0.005 \text{ cm/s})\) and pullout \((z_w=-0.0005 \text{ cm/s})\) phases of the CET program, the Kulite and Cooper probes are sufficiently accurate, while the Data Instruments probe does not respond quickly enough to register accurate changes in pore pressure\(^{17}\).

4.6.3 Cap Pore Pressure Sensor

There are two factors affecting the accuracy of pore pressure measurements directly beneath the caisson cap: 1) susceptibility to cavitation, and 2) time response. As described in section 3.1.5, pore pressure beneath the cap is measured by a Data Instruments pressure transducer mounted directly behind a 0.93 cm thick ceramic porous stone with a porosity of 45% and a 1 ksc air entry pressure. The stone is set flush with the cap bottom surface, has a diameter of 2.2 cm, and represents 22% of the total cap area (see Figure 3.6). The stone does not cover the entire cap surface, and therefore, the sensor measures only the zone directly beneath the porous stone.

Data indicated that cavitation occurred directly beneath the cap during monotonic pullout in CET1 and CET8. In CET1, a large consolidation stress \((\sigma_{vc}=1.0 \text{ ksc})\) was applied to the clay cake. As a result, very large negative excess pore pressure was induced in the soil plug during pullout\(^{18}\). In CET8, it is possible that the soil separated along the outside edge of the cap during later stages of pullout. This would reduce the soil/cap contact area and increase the tensile stress on the cap. A large enough increase in tensile stress on the center of the cap would induce very large negative excess pore pressure, which would lead to cavitation. A small amount of air trapped between the wall and cap could have encouraged soil separation at the outer edges of the cap. Hence for tests CET 9-14, the cap/wall annular space was packed with lubricant to eliminate the air space. No cavitation was detected in these tests.

\(^{17}\)Only pore pressure data from Kulite and Cooper probes are presented in Chapter 5 (see section 5.1).
\(^{18}\)A total stress of \(\sigma_v=0.75 \text{ ksc}\) was used in all subsequent tests.
The final area of concern for the cap pore pressure sensor is its time response. Using the equations\textsuperscript{19} developed for the pore pressure probes described in section 4.6.2, the theoretical time response was calculated for the cap sensor in both water and soil (see Figure 4.18 and Table 4.3). When subjected to a step increase in stress in water, the cap sensor registers 95% of the applied stress within 0.32 seconds. Note that this is nearly twice as slow as the Data Instruments (D.I.) probe because the cap sensor has a porous stone with a much lower hydraulic conductivity. However, in soil with a hydraulic conductivity of $k=1.4 \times 10^{-7}$ cm/s, the theory predicts that the cap will measure 95% of the applied stress within 5.3 seconds, which is 36 times faster than the D.I. probe. Even though the cap sensor and the D.I. probe both use a Data Instruments transducer, the cap sensor has a much quicker predicted response in soil because the porous stone has a much larger diameter ($D_{\text{cap, stone}} = 2.2$ cm, $D_{\text{probe, tip}} = 0.058$ cm), which allows a much larger pore fluid flow rate in response to a pressure change. As with the Kulite and Cooper pore pressure probes, the cap sensor time response is deemed rapid enough for the caisson movement rates encountered in the CET testing program.

\textsuperscript{19}For time response in soil, the predictive equation was developed for a probe surrounded by soil. The application of this equation for the cap sensor is approximate because the cap maintains planar contact with, but is not surrounded by, the soil.
<table>
<thead>
<tr>
<th>Date</th>
<th>Test</th>
<th>CET Apparatus Component</th>
<th>Modification</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>9-10-93</td>
<td>CET 1</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
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<td>X</td>
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</tr>
<tr>
<td>2-22-94</td>
<td>CET 2</td>
<td>X</td>
<td></td>
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<td></td>
<td>X</td>
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<tr>
<td>3-16-94</td>
<td>CET 3</td>
<td>X</td>
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<td>X</td>
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</table>

**CET Apparatus Component Key**

1. Consolidation Chamber
2. Model Caisson
3. Driving System
4. Control System
5. Instrumentation

Table 4.1 CET Equipment Modification Chronology
<table>
<thead>
<tr>
<th>Date</th>
<th>Test</th>
<th>CET Apparatus Component</th>
<th>Modification</th>
</tr>
</thead>
<tbody>
<tr>
<td>7-26-94</td>
<td>CET 4</td>
<td>X</td>
<td>Fitted pore pressure probes with liquid-compatible Data Instruments transducers</td>
</tr>
<tr>
<td>8-18-94</td>
<td>CET 5</td>
<td>X</td>
<td>Machined new gears for cap and wall reduction gear box</td>
</tr>
<tr>
<td></td>
<td></td>
<td>X</td>
<td>Incorporated tension springs for wall and cap actuators</td>
</tr>
<tr>
<td></td>
<td></td>
<td>X</td>
<td>Improved HOLDSTS: continuous caisson control, keyboard toggles for all gain factors, relative cap/wall stiffness trigger</td>
</tr>
<tr>
<td></td>
<td></td>
<td>X</td>
<td>Improved pore pressure probes: porous filter tips, replaced one Data Instruments with Cooper transducer</td>
</tr>
<tr>
<td>3-5-95</td>
<td>CET 6</td>
<td>X</td>
<td>Widened drain tubing diameter</td>
</tr>
<tr>
<td></td>
<td></td>
<td>X</td>
<td>Replaced outer slip ring O-rings with X-rings</td>
</tr>
<tr>
<td></td>
<td></td>
<td>X</td>
<td>Improved pore pressure probes: replaced two Data Instruments with Kulite transducers, researched probe response</td>
</tr>
<tr>
<td>6-7-95</td>
<td>CET 7</td>
<td>X</td>
<td>Incorporated shims to separate inside wall of slip ring from caisson wall</td>
</tr>
<tr>
<td>6-26-95</td>
<td>CET 8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7-31-95</td>
<td>CET 9</td>
<td>X</td>
<td>Improved cap and wall control in HOLDSTS</td>
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<td>8-28-95</td>
<td>CET 10</td>
<td></td>
<td></td>
</tr>
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<td>9-19-95</td>
<td>CET 11</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10-23-95</td>
<td>CET 12</td>
<td>X</td>
<td>Added pore pressure probe with Kulite transducer</td>
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<td>11-30-95</td>
<td>CET 13</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12-16-95</td>
<td>CET 14</td>
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**CET Apparatus Component Key**

1. Consolidation Chamber  
2. Model Caisson  
3. Driving System  
4. Control System  
5. Instrumentation

Table 4.1 CET Equipment Modification Chronology (cont.)
<table>
<thead>
<tr>
<th>Transducer Diaphragm</th>
<th>Kulite</th>
<th>Cooper</th>
<th>Data Instruments</th>
</tr>
</thead>
<tbody>
<tr>
<td>diameter, cm</td>
<td>0.39</td>
<td>0.55</td>
<td>1.59</td>
</tr>
<tr>
<td>area, cm²</td>
<td>0.12</td>
<td>0.24</td>
<td>1.98</td>
</tr>
<tr>
<td>thickness, cm</td>
<td>0.016</td>
<td>0.013</td>
<td>0.042</td>
</tr>
<tr>
<td>*deflection cm²pa(t)/N</td>
<td>1.68x10⁻⁶</td>
<td>8.92x10⁻⁶</td>
<td>3.72x10⁻⁴</td>
</tr>
</tbody>
</table>

*pa(t) = applied pressure

Table 4.2 Important Physical Characteristics of the Three Probe Transducers

<table>
<thead>
<tr>
<th>*t95, seconds</th>
<th>Pore Pressure Probes</th>
<th>Cap Sensor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Kulite</td>
<td>Cooper</td>
</tr>
<tr>
<td>Theoretical in water ((k_{stone} = **))</td>
<td>0.0008</td>
<td>0.004</td>
</tr>
<tr>
<td>Measured in water</td>
<td>&lt;0.05†</td>
<td>&lt;0.1†</td>
</tr>
<tr>
<td>Theoretical in soil ((k_{soil} = 1.4x10^{-7} \text{ cm/s}))</td>
<td>1.0</td>
<td>5.1</td>
</tr>
<tr>
<td>Measured in soil</td>
<td>4</td>
<td>10</td>
</tr>
</tbody>
</table>

*\(t_{95}\) = time required for probe to reach 95% of applied pressure

**\(k_{stone} = 6.08x10^{-3} \text{ cm/s (probe tip)}\); \(k_{stone} = 8.6x10^{-6} \text{ cm/s (cap stone)}\)

†applied cyclic pressure and recorded phase lag

‡cap sensor time response not measured

Table 4.3 Time Response to Applied Pressure of Pore Pressure Probes and Cap Sensor in Water and Soil: Theoretical vs. Measured Values
Figure 4.1 Slip Ring Assembly Modification: Replacement of O-rings with X-rings in Outer Slip Ring to Prevent Air Leaks
Surface Drainage Improvements

1. Added cloth filter (after CET1)
2. Increased drain ports from 2 to 4 (after CET3)
3. Widened drainage tubing (after CET6)

Figure 4.2 Slip Ring Assembly Modification: Three Surface Drainage Improvements
Figure 4.3  Slip Ring Assembly Modification: Two Modifications to Improve Caisson Wall Passage
Figure 4.4  Two Model Caisson Modifications
CET 1-2  
CET 3-4  
CET 5  
CET 6-14

Figure 4.5  Pore Pressure Probe Development: Five Different Probes
Figure 4.6  Cross-section of the Consolidation Chamber for CET6 Showing Instrumentation, Including Horizontal Total Stress Transducer
Figure 4.7  Horizontal Total Stress Record for Five Test Phases of CET6
Figure 4.8 Chamber Air Pressure Record for Five Test Phases of CET6
Slip Ring Assembly
Air Leak Path

1) From air chamber
   a) outer ring/sidewall
   b) outer ring/membrane
   c) inner ring/membrane
2) Beneath rubber membrane
3) Between inner ring and caisson wall
to atmosphere

Figure 4.9 Air Leak Path Through Cylindrical Slip Ring Assembly
Figure 4.10  Measured and Calculated Time Curves During Consolidation
a) Consolidation in CET Apparatus ($\sigma'_v=0.67$ to 0.73 ksc) for tests CET 3-13.
b) Consolidation in Original Chamber for RBBC 217
Figure 4.11  Slip Ring Assembly and Model Caisson in Position Prior to Wall Installation to Illustrate Annular Gaps Between Slip Ring and Wall and Between Wall and Cap.
Model Caisson Friction

1. Slip Ring/Caisson Wall Metal-to-Metal Contact
2. Slip Ring/Caisson Wall Extruded Soil
3. Caisson Wall/Cap Extruded Soil
4. Caisson Wall/Cap O-ring Connection

Figure 4.12 Slip Ring Assembly and Model Caisson in Position Prior to Wall Installation to Illustrate Four Locations of Model Caisson Friction
Figure 4.13  Chamber Air Pressure versus Log Time for Suction Installation, Equilibration, and Monotonic Pullout for CET8
Figure 4.15: Theoretical Time Response of Three Pore Pressure Probes Inserted into a Triaxial Specimen of Clay.
Figure 4.16  Theoretical Time Response of Kulite Probe Inserted into a Triaxial Specimen of Clay at Three Different Soil Hydraulic Conductivities
Figure 4.17  Measured Time Response of Three Probes Inserted into a Triaxial Specimen of Clay
Figure 4.18 Theoretical Time Response of Cap Sensor in Water and Soil
CHAPTER 5
CHARACTERISTICS OF SUCTION CAISSONS IN THE CET CELL

This chapter describes the results of the testing program using the Caisson Element Test (CET) apparatus. The main objectives of the testing were to gain insight into the installation, set-up, and axial tensile loading of a suction caisson and to provide experimental data for comparison with analytical investigations. The tests were conducted in the CET cell, which simulates suction caisson behavior by using a miniature caisson installed in a uniform, saturated 'element' of clay (for a full description of the CET cell, see Chapter 3). The test program consisted of 14 tests and has yielded a tremendous amount of data regarding caisson performance. Most of the data from the test program is presented in this chapter in the form of compilation plots to exhibit characteristics of caisson behavior. Section 5.1 presents the testing program philosophy and a summary of the individual test geometry, instrumentation package, loading schedule, and quality assessment. Then, sections 5.2 through 5.8 describe the basic characteristics that were measured during installation, equilibration, monotonic pullout, and sustained loading. In each of these sections, the discussion focuses on force-displacement relations for the caisson, and pore pressure and surface displacement of the surrounding clay mass.
5.1 THE TESTING PROGRAM

5.1.1 Program Objectives

The two main objectives of this testing program were to gain insight into fundamental suction caisson behavior and provide data for comparisons with analytical predictions. In order to achieve these goals, the tests were designed to illuminate the following principal parameters:

1) penetration resistance for a caisson installed by underbase suction
2) time frame for the equilibration of pore pressures after installation
3) caisson displacement during the equilibration phase
4) ultimate pullout capacity and wall friction contribution for monotonic axial tensile loading
5) time frame for release of underbase suction and caisson displacement during sustained tensile loading
6) effect of installation disturbance on tensile load capacity of the caisson

Other parameters considered in the tests include the penetration rate, the rate of tensile load application, and the effect of reconsolidation on pullout capacity.

5.1.2 Test Geometry and Instrumentation

The model caisson and clay sample geometry were similar for all tests. Figure 5.1 shows a schematic cross section of the model caisson fully installed in a clay element. Table 5.1 lists the dimensions of the model caisson and clay cake for each of the 14 tests. The caisson is a two-component cylindrical model, comprising an outer caisson wall and an inner caisson cap. The wall is blunt-tipped with an outside diameter of $D_o=5.08$ cm and a wall thickness of $t_w=0.145$ cm to give a diameter to thickness ratio of $D_o/t_w=35$.1

The caisson for each test penetrated $L=5.08$ cm into the clay element to give a embedment

1The model for CET1 had a tapered tip geometry, but because of test control problems, the results are not reported.
to diameter ratio of \( \frac{L}{D_0} = 1 \). The clay element has a diameter of 30.5 cm and a pre-installation height that ranges from \( H_c = 12.1 \) to 14.3 cm with an average \( \bar{H}_c = 13.2 \) cm.

The instrumentation package for each test includes between 12 and 15 transducers (Table 5.1). All tests include the five primary transducers, which measure the caisson wall and cap force (\( F_w \) and \( F_c \)) and displacement (\( \delta_w \) and \( \delta_c \), respectively) and consolidation chamber air pressure (\( P_a \)). Section 3.1.4 explains the role of the primary transducers in the automated feedback control loop for the CET tests. The remaining transducers include a pressure transducer in the cap to measure pore pressure in the soil beneath the cap, pressure transducers to measure pore pressure in the clay cake and total stress along the chamber wall, and displacement transducers to measure clay surface displacement (Figure 5.1, Table 5.1). Most of the pore pressure probes were located in the clay at a depth of approximately 2.5 cm below the clay surface with radial locations shown in Figure 5.1. This geometry provides pore pressure data inside and outside the caisson walls during and after caisson penetration. In two tests (CET 13,14) pore pressures were measured near the base of the clay sample, and in three other tests (CET 1,2,6) total stress was measured on the sidewall of the chamber (Figure 5.1). A series of displacement transducers measures the vertical displacements of the clay surface (Figure 5.1). Section 5.1.4 reviews the quality of these measured data.

5.1.3 Test Loading Schedule

The load history for each test can be divided into a series of driving, equilibration, and tensile loading stages. Table 5.2 lists the phases of each test and includes the elapsed time, imposed loads, and imposed displacements, where appropriate. The chronology of each test is shown schematically by caisson force and wall tip displacement timelines in Figure 5.2. Each timeline depicts the record of the total caisson force \( F_{tot} \) (positive in compression) and caisson displacement throughout the test. Note that the displacement during driving refers only to the wall displacement, while displacement thereafter refers to
entire caisson movements. In each test the clay element was consolidated into the virgin compression range to a consolidation stress of $\sigma'_{vc}=0.75 \text{ksc}$ with the caisson wall tip flush with the caisson cap at the surface of the clay\(^2\). In all tests except CET2, this consolidation phase was held for at least 24 hours prior to penetration.

In the majority of tests (CET1-12), installation by underbase suction (SD, Figure 5.2) was simulated by penetrating the wall into the clay at a constant displacement rate while maintaining a constant total force on the caisson (see timelines in Figure 5.2). In order to simulate underbase suction, net increases in the wall force are balanced by equal and opposite load increments applied to the cap. The wall was penetrated to a final depth, $L=5.08 \text{ cm}$, at a rate of $0.3 \text{cm/min}$ (CET1-5, 7-12). A much slower penetration rate was used initially in test CET6; the wall was penetrated to a depth of $L=1.05 \text{ cm}$ at a rate of $0.01 \text{ cm/min}$ (the remaining penetration was performed at the standard rate, as shown in Table 5.2).

In order to study the effects of installation disturbance by underbase suction, tests CET13 and 14 used different penetration control schemes. In CET13, the cap force was held constant as the wall penetrated the clay (comparable to an open-ended pile). In CET14, zero cap displacement was attempted during wall penetration. Due to control problems (see section 5.1.4), the cap displaced slightly. Nevertheless, an unique installation disturbance was established for comparison with suction installation. Figure 5.2 shows variations in the total force during installation for both CET13 and 14.

Following the penetration phase in all tests, the caisson was allowed to equilibrate (EQ1, Figure 5.2) for at least 18 hours prior to tensile loading. During the equilibration phase, a constant total force of $F_{tot}=15.2 \text{ kg}$ was maintained on the caisson, allowing no relative displacement between the cap and wall. Figure 5.2 indicates that the caisson tended to settle slightly during this period.

\(^2\)A total force of $F_{tot}=15.2 \text{ kg}(=A_{sw}\sigma'_{wc})$ was applied to the caisson to ensure constant contact pressure across the surface of the clay sample.
After the first set-up phase, a variety of tensile loading schemes were applied. These can be classified into two categories: monotonic pullout (MP1) to failure and sustained loading (SL), as shown in Figure 5.2. Six tests were pulled monotonically to failure, while six were subject to a sustained load following the first equilibration period.

In three tests (CET3,4,8) the caisson was withdrawn at a rate of 0.03cm/min beyond peak tensile load and then was pulled at a rate of 0.3cm/min until complete extraction. In tests CET5, 6, and 7, the caisson was pulled at 0.03cm/min until just after peak load, whereupon the caisson was re-equilibrated in the clay with a compressive force of $F_{tot}=15.2$ kg for more than a day. After reconsolidation in these tests, the caisson was then pulled again (MP2) at a rate of 0.03cm/min. Figure 5.2 shows the timelines for these two types of monotonic pullout tests.

For the sustained loading test, CET9, the caisson was withdrawn at a rate of 0.03cm/min (MP1) until reaching a tensile load of $F_{tot}=-2.2$ kg, which was held for 13.4 hours. Then, the caisson then was loaded monotonically to failure (MP2) at 0.03cm/min. For tests CET10-14, the caisson was pulled at 0.03cm/min to a predetermined tensile load level and held at this load until either the caisson began to fail or more than 24 hours had passed, whichever came first. If no failure occurred, then an increment of tensile load was applied and maintained for up to 24 hours. The process was repeated until failure, whereupon the test was either ended (CET10,11) or the caisson was re-equilibrated (EQ2; CET12-14). After the post-loading set-up in CET12-14, the model was pulled a second time (MP2).

5.1.4 Test Quality Assessment

Before presenting test results, it is important to discuss the quality of the data in order to clarify certain perturbations in the results that are due to control or instrumentation errors. This quality assessment ensures the integrity of comparisons made in the presentation and analysis of the test results. In order to assess the test quality, a
system was developed to rate the quality of control in each of the test phases and the accuracy of the instrumentation. Table 5.3 lists the rating for the control and instrumentation for each phase in all of the 14 tests using a scale from 1 (good) to 4 (unusable). Table 5.4 lists the guidelines used to grade the control and instrumentation aspects of the tests. In terms of test control, the grade represents how well the automated feedback control was able to maintain the target force or displacement for that particular test phase. (Note that the accuracy benchmark for test control and instrumentation was outlined in the evaluation of controls and instrumentation in Chapter 4.) Most of the data for test phases and instrumentation that received ratings of 1, 2, or 3 are included in the results and analysis that are presented. In order to present clearly the results of several tests in one plot, small data perturbations that do not reflect true soil behavior have been "smoothed" out using a data averaging process. Data that received a rating of "4" are considered unusable and are not presented in this thesis.

As listed in Table 5.3, the control for tests CET1 and 2 received a "4" rating and therefore these test data are omitted from this thesis. In CET1 proper testing was hampered by severe control problems in each phase of testing, while CET2 was aborted after the caisson caught and dragged the inner membrane slip tube into the clay cake (see section 4.1.1). The control for the remaining tests were rated above "4" and are included in the presentation. In general, the quality improved with each successive test, so that most of the data in the later tests were considered either fair or good.

The quality of the instrumentation data also progressed with successive tests. In the early tests, the cap and particularly the probe pore pressure instruments were undergoing intense research development, and hence much of the data were not worth reporting. Later in the testing, the probes became much more accurate and responsive. Note that a few surface displacement transducers became stuck or were beyond the linear range during testing and therefore became unreliable.
5.2 SUCTION INSTALLATION

This section presents the results of installation by underbase suction in 9 tests (CET 3-5, 7-12). Following clay cake consolidation at $\sigma'_v=0.75$ ksc for a period of at least 24 hours, the caisson wall penetrated the soil at a rate of 0.3 cm/min to a depth of L=5.1 cm to give an embedment depth to diameter ratio of L/D_o=1. During driving, the cap force was reduced to offset the increase in force needed to drive the wall. Only the basic measured characteristics including caisson forces, soil pore pressures, and clay surface displacement, will be shown.

5.2.1 Caisson Force Distribution

In order to illustrate typical caisson force behavior during suction installation, Figure 5.3 shows the caisson wall, cap, and total ($F_{tot}=F_w+F_c$) forces as a function of penetration depth for CET9, which was chosen for the quality of its test control and instrumentation data. The total force remains constant at $F_{tot}=15.5$ kg, which is just 0.3 kg above the design value of $F_{tot}=15.2$ kg ($=A_c\sigma'_v$, where $A_c$ is the total cross-sectional area of the caisson). The wall force rises sharply from $F_w\approx-0.7$ kg to $F_w\approx13$ kg during the first 0.3 cm of penetration and increases linearly with depth at a rate of 1.3 kg/cm from approximately $z_w=0.8$ cm to the end of penetration at $z_w=5.1$ cm. There is a transition zone from $z_w=0.3$ to 0.8 cm where the wall force drops 1.5 kg and then remains essentially constant. The caisson cap shows the inverse response with a sharp drop from $F_c\approx16$ to 2.5 kg at $z_w=0.3$ cm, a transition phase from $z_w=0.3$ to 0.8 cm, and a linear decrease to $F_c\approx-2.5$ kg.

Total Force

The caisson force behavior exhibited in CET9 is similar to that in the suction installation phase in the remaining eight tests (CET3-5, 7-8, 10-12). It is important now
to establish the individual caisson force trends during suction penetration for all nine tests. Figures 5.4 and 5.5 show the total force versus wall tip penetration for these nine tests at two different scales. It is clear that the total force is constant at, or within 2 kg above, the design value of $F_{\text{tot}}=15.2$ kg. Note that the total force for CET3 exhibits greater fluctuations about a constant value than the rest because the software control module was still in the early stages of development (see section 4.1.4).

**Wall Force**

There appears to be no correlation between the amount of total force deviation above the design value of 15.2 kg and the wall force behavior. Figure 5.6 depicts the wall force versus wall tip penetration for eight tests. Note that while the total force for CET8 is approximately 2 kg larger than in CET9, the wall force records are nearly identical. It is not surprising that variations in the total force do not affect the wall force because the wall follows a prescribed penetration rate independent of the total force, as explained in section 3.1.4.

Further inspection of Figure 5.6 reveals three zones of wall force penetration behavior: initial, transition, and deep penetration. Initial, or early, penetration is characterized by a sharp increase in wall force of at least 10 kg during the first few tenths of a centimeter (wall thickness is 0.145 cm), with the force increasing at a decreasing rate. During deep penetration, the wall force increases linearly with depth at a rate of approximately 1.5 kg/cm starting from a depth of between $z_w=0.8$ and 1.8 cm and lasting until the end of penetration at $z_w=5.1$ cm. In between the initial and deep penetration is the transition zone. The wall force behavior during this zone and the extent of this zone varies from test to test and is largely a result of friction between the caisson wall and cap components.

Before discussing each of the three penetration zones in detail, it is helpful to review caisson component friction, which accounts for the variation in initial and deep
penetration wall force behavior and may account for much of the wall force behavior during the transition zone. As stated in section 4.3.2, caisson intercomponent friction can arise from four different sources: 1) metal-to-metal contact between the caisson wall and chamber inner slip ring, 2) extruded soil between the wall and inner slip ring, 3) extruded soil between the caisson wall and cap, and 4) the O-ring separating the wall and cap. The locations of these four sources of friction are illustrated in Figure 4.12. During penetration all sources of friction potentially can contribute a compressive component to the wall force.

The initial and transition penetration zones for the nine suction installation tests (CET3-5, 7-12) are represented clearly in Figure 5.7, which shows the wall force for the first two centimeters of penetration. Ideally, at the start of penetration ($z_w=0$ cm), the caisson wall should apply a force of $F_w=1.7$ kg ($=\sigma'_{vc}A_w$) to the surface of the clay. However, it is likely that soil extruded between the caisson components during the consolidation process prior to penetration, and this soil introduced tensile or compressive components of force to the caisson wall. As a result, the initial wall force ranges from $F_w=-1.2$ to 5 kg and is within $\pm 3.3$ kg of the ideal initial force. Initial penetration is characterized by a stiff wall response. During the first $z_w=0.2$ cm of penetration in 7 of 9 tests, the wall force increases by more than 10 kg. In order to compare the initial stiffness among the tests, a "penetration modulus" can be defined by dividing the wall force increment by the initial penetration depth ($M_w=\Delta F_w/\Delta z_w$). The moduli for penetration depths of $z_w=0.05$, 0.1, and 0.2 are listed in Table 5.5. In most tests the modulus decreases with increasing penetration, as the wall approaches the transition zone. Note that the initial stiffness for tests CET5, 10-12 was lower than the remaining tests, particularly at depths of $z_w=0.05$ and 0.1 cm. In these four tests, it is possible that the soil surface was disturbed by a premature wall penetration during the final consolidation stage prior to penetration. In CET10 a power shutdown led to a wall penetration of 0.4 cm, and in CET12 a displacement transducer malfunction caused a wall penetration of 0.3 cm. In
both tests, the wall was repositioned to be flush with the cap before consolidation continued. By a penetration depth of \( z_w = 0.2 \text{ cm} \), the moduli for the tests with potentially disturbed surfaces have approached the moduli for the remaining tests, as listed in Table 5.5. The end of the initial penetration zone is not clear, as various intercomponent frictional forces and perhaps soil surface effects combine to create a transition zone between initial and deep penetration. However, it will be shown in the subsequent discussion regarding the most representative and "best estimate" of wall force behavior, that a depth, \( z_w = 0.2 \text{ cm} \) (i.e., approximately equal to the wall thickness), is a reasonable endpoint for the initial penetration zone.

The transition zone of wall penetration is defined by a "hump-shaped" rise and fall in wall force, with a local maximum wall force at a penetration depth of between \( z_w = 0.3 \) and \( 1.0 \text{ cm} \), as shown in Figure 5.7. The end of the transition zone, at a depth ranging from \( z_w = 0.8 \) to \( 1.8 \text{ cm} \), occurs when the incremental penetration modulus becomes constant (i.e., linear variation of \( F_w \) with \( z_w \)). The transition zone peak and final wall force values and depths are listed in Table 5.5. To illustrate the size and extent of the transition "hump", consider CET9 in Figure 5.7. The wall force peaks at \( F_w = 12.9 \text{ kg} \) at a depth of \( z_w = 0.4 \text{ cm} \) and falls to a value of \( F_w = 11.5 \text{ kg} \) at a depth of \( z_w = 0.8 \text{ cm} \) to mark the end of the transition zone and the start of deep penetration. In general, the transition is probably caused by the build-up and subsequent release of intercomponent friction contributed by one or more of the sources mentioned above (see Figure 4.12). As the wall passes by the cap during early penetration, friction builds up between the O-ring and the wall and between the extruded soil and the wall until reaching a maximum contributing value, which is depicted as the peak of the transition "hump". Following this, the friction contribution drops due to a combination of one or more of the following events: 1) O-ring friction drops to a residual value, 2) the cap, which rises during wall penetration, releases the friction caused by the soil extruded between the cap and wall, 3) the wall drags down the extruded soil between the wall and inner slip ring, thus reducing this
frictional source. Once a residual friction value is reached, deep penetration can be characterized by a linear relation between wall force and penetration depth. Variations in the frictional contributions from test to test are reflected in variations in the size and extent of the "hump". Note that in CET12, no "hump" exists, which suggests the lack of a transition zone. In test CET4, the transition zone is denoted by two "humps", the first peaking at \( z_w = 0.3 \) cm and the second peaking at \( z_w = 0.6 \) cm. The remaining tests display fairly recognizable transition zones.

Following the initial and transition zones, the wall enters a deep penetration zone, where the wall force increases at a constant rate with depth. Figure 5.8 shows the wall force versus depth for the deep penetration zone in tests CET3-5, 7-12. Due to variations in the extent of the transition behavior, the onset of deep penetration ranges from \( z_w = 0.8 \) to \( 1.8 \) cm. In order to compare the linear increase in force among tests, a first-order regression was performed on the wall force vs. penetration data in the deep penetration zone for each test. The starting depth and regression coefficients for each test are listed in Table 5.5. For all tests, the slope of the secant deep penetration modulus ranges from \( M_w = \Delta F_w / \Delta z_w = 1.11 \) to \( 2.04 \) kg/cm with an average value of \( M_w = 1.47 \) kg/cm.

Of the nine tests depicted in Figure 5.8, four of them (CET3,5,7,10) reveal deep penetration behavior that is significantly different from the remaining five tests. The deep penetration modulus for CET3, \( M_w = 2.04 \), is 39% higher than the average and 27% higher than the next highest slope (CET7). This is probably due to a build-up of friction from the O-ring separating the wall and cap. As noted in section 4.1.2, the O-ring "squeeze", and hence friction, was quite high for tests CET1-3. For the remaining tests, this O-ring "squeeze" was lessened significantly. In CET5, the slope is \( M_w = 1.11 \) kg/cm, which is 26% lower than the average due to an unusually low slope during the initial 1.2 cm of deep penetration. The most likely cause is continued drop-off in intercomponent friction from the transition zone. Both CET7 and CET10 show a deep penetration zone with much higher starting and final wall force values relative to the remaining tests. This
indicates that the residual intercomponent friction contribution at the end of the transition zone was much higher than the other tests. It is possible that in these two tests, the wall had continuous metal-to-metal contact with the inner chamber slip ring. Neglecting these outliers, the slope of the regression line ranges from $M_w=1.29$ to 1.60 kg/cm with an average value of $M_w=1.45$ kg/cm.

Combining the most reliable wall force data for the initial and deep penetration zones facilitates the interpretation of the overall behavior of the caisson wall force during suction driving. Figure 5.9 shows the most representative wall force data for both the initial and deep penetration zones. For initial penetration, only two tests (CET8-9) are depicted. These are the only tests that did not show possible evidence of surface disturbance, which led to decreased initial penetration "moduli", and also did not suffer from excessive intercomponent friction, as revealed in the transition and deep penetration zones. For both tests, the endpoint was chosen to coincide with the extension of the deep penetration regression line for that particular test. For deep penetration, five tests (CET4, 8-9, 11-12) showed consistently linear behavior without excessive intercomponent friction. The transition zone is not shown for any plot in Figure 5.9 because it is assumed that the wall force behavior in this zone is the result of caisson intercomponent friction and not strictly the caisson-soil interaction. In order to show continuity between the initial and deep zones, dotted lines extend from a depth of $z_w=0.2$ cm to the depth at which deep penetration begins for each of the five tests. These dotted lines are simply an extension of the regression lines that were calculated and listed in Table 5.5 for each test. Hence, deep penetration is assumed to begin immediately following the initial penetration. The penetration data presented in Figure 5.9 clearly shows the distinct nature of the wall force behavior during shallow and deep penetration. Without the masking effect of caisson intercomponent friction, the wall force shows a very stiff response ($M_w>55$ kg/cm) during the first $z_w=0.2$ cm of penetration. This is followed immediately by a linear increase in force with depth at an average rate of $M_w=1.45$ kg/cm for the remainder of penetration.
The variation in magnitude of the slope of deep penetration reflects the variable contributions of normal residual intercomponent friction and the minor differences in soil resistance from test to test.

The measured behavior for suction installation presented in Figure 5.9 can be simplified by assuming that the wall force is bi-linear. Figure 5.10 shows the initial and deep zones represented by two linear ranges. Starting at $F_w = 1.7$ kg ($\sigma'_{vc}A_w$), the wall force increases linearly to a depth of $z_w = 0.2$ cm. This depth was chosen because it is the endpoint for initial penetration in CET8, which had the least amount of intercomponent friction (see Figures 5.7 and 5.9). At this depth the wall force coordinate, which ranges from $F_w = 10.4$ to $11.05$ kg, represents the intersection of the initial zone line and the deep zone regression lines that were presented in Figure 5.9. Deep penetration, represented by the dashed lines, begins at $z_w = 0.2$ cm and ends at $z_w = 5.1$ cm. The lower boundary starts at $F_w = 10.4$ kg and rises at a rate of $M_w = \Delta F_w/\Delta z_w = 1.29$ kg/cm, which is the lowest slope of the regression lines calculated for the deep penetration records. Starting at $F_w = 11.05$ kg, the upper boundary rises at the highest slope, $\Delta F_w/\Delta z_w = 1.60$ kg/cm. The wall force at the end of penetration ranges from $F_w = 16.7$ to $18.9$ kg. Admittedly, this bi-linear model of wall force during penetration is only a "best estimate" of the true behavior. While the model neglects much of the intercomponent friction that clouds the true wall force signature, particularly at penetration depths within the transition zone from $z_w = 0.8$ to $1.8$ cm, it is based on measured data in the initial and deep zones (see Figure 5.9) that does incorporate some friction. Nevertheless, this model is an important representation of wall force installation behavior, upon which interpretations regarding caisson wall bearing capacity and adhesion are based in Chapter 6.

Cap Force

During suction driving, the cap force is controlled as the difference between the measured wall force and the preset total force ($F_c = F_{tot} - F_w$). The cap force behavior,
therefore, should be equal but opposite to the wall force behavior. Figure 5.11 shows the cap force versus wall tip penetration for nine tests (CET3-5, 7-12). The cap force behavior is defined by the same three wall penetration zones that were described above in the discussion of wall force behavior. During the initial penetration zone from $z_w=0.0$ cm to approximately $z_w=0.2$ cm, the cap force drops sharply by more than $\Delta F_c=10$ kg at a decreasing rate with depth. During the "hump-shaped" transition zone, which starts from approximately $z_w=0.2$ cm, the cap force continues dropping until reaching a local minimum before rising toward the end of the transition, which occurs at depths ranging from $z_w=0.8$ to $1.8$ cm. In Figure 5.11, the individual test "hump" minima are identified by symbols except for CET10, which did not exhibit a transition zone. Following the transition zone, the cap force drops at a constant rate with depth until the end of penetration at $z_w=5.1$ cm. At this depth the cap force has decreased to tensile values between $F_c=0.0$ and $-7.0$ kg. Table 5.5 lists the cap force and depth coordinates for the three penetration zones for each test.

Using the criterion outlined for the wall force, the most representative cap force behavior for both the initial and deep penetration zones is depicted in Figure 5.12. For the initial zone, only those tests not suffering from excessive intercomponent friction or soil surface disturbance are shown (CET8-9). For deep penetration, the five tests that do not exhibit excessive friction are shown (CET4, 8-9, 11-12). Continuity between the initial and deep penetration zones is not illustrated by extending deep penetration regression lines because of the effect of variations in the total force. Unlike the wall force, the cap force trend is affected by variations in the total force from the design value of $F_{tot}=15.2$ kg. This effect can be seen by comparing tests CET8 and CET9. In CET8, the total force was approximately 2 kg too high at $F_{tot}=17.2$ kg. In CET9, the total force was much closer to design at $F_{tot}=15.4$ kg. The wall force in these two tests is nearly identical during deep penetration as depicted in Figure 5.9, but the cap force for CET8 is approximately 1.8 kg higher than that for CET9 during deep penetration. Despite the effect of total force, the
general cap force behavior during initial and deep penetration is shown clearly by Figure 5.12. The cap responds stiffly ($\Delta F_c/\Delta z_w=-55$ kg/cm) during the first $z_w=0.2$ cm of penetration. First order regression analyses on the deep penetration cap force behavior indicates an average cap modulus $M_c=\Delta F_c/\Delta z_w=-1.5$ kg/cm.

5.2.2 Pore Pressure Generation Beneath Cap

At the start of suction penetration, there is no excess pore pressure beneath the caisson cap. Measurements of excess pore pressure generated beneath the cap during driving for 8 tests (CET4-7, 8-12) are shown in Figure 5.13. The first $z_w=2$ cm of penetration are shown in Figure 5.14 for clarification. Note that no record is shown for CET3 due to poor transducer response (see section 5.1.4). Aside from variations in magnitude, the figure shows remarkable uniformity in the general pore pressure behavior throughout penetration. After an initial drop ranging from $\Delta u=-0.05$ to -0.26 ksc during the first $z_w=0.2$ cm of driving, positive excess pore pressures are generated during the transition zone at a decreasing rate until reaching a peak at a depth that ranges from $z_w=1.0$ to 2.4 cm. During deep penetration, the excess pore pressure drops at a constant rate, reaching values between $\Delta u=+0.04$ to -0.27 ksc at the end of penetration ($z_w=5.1$ cm). The variations in magnitude of pore pressure can be traced to differences in cap force magnitude, which, as noted above, were due to variations in total force and intercomponent friction from test to test. Note that in CET10, the excess pore pressure does not rise significantly after the initial drop, but instead descends from $\Delta u=-0.04$ ksc at $z_w=0.4$ cm to $\Delta u=-0.27$ ksc by the end of penetration. This lack of intermittent cap pore pressure rise in CET10 is due directly to the lack of a transition penetration zone for the wall and cap force, as shown in Figures 5.6 and 5.11, respectively. A notable cap pore pressure feature visible in all tests but CET10 in Figure 5.13 is that the end of the transition zone is at a greater depth than the transition end in wall force or cap force. For example, consider test CET11. Excess pore pressure reaches a peak of $\Delta u=0.16$ ksc at
$z_w=1.4$ cm, which is the end of transition and the beginning of deep penetration. However, the end of transition for both wall and cap force is at the shallower depth of $z_w=0.6$ cm (see Figures 5.6 and 5.11). It is likely that the linear decrease in excess pore pressure beneath the cap starts later than the linear decrease in cap force because of positive pore pressures generated by the penetrating wall.

5.2.3 Cap Displacement

Figure 5.15 shows the caisson cap displacement versus wall tip penetration for 9 tests (CET3-5, 7-12) during suction driving. The cap rises at a nonlinear rate throughout penetration and reaches $\delta_c=0.55$ to 0.98 cm by the end of wall penetration. Although there is considerable scatter among the 9 tests, scrutiny of the shape of the cap rise curves reveals two general patterns. In the first pattern, which is followed by tests CET3, 5, 7, and 12, the cap rises at a gradually increasing rate until a wall depth ranging from $z_w=3.5$ to 4.6 cm, whereupon the cap rise rate decreases with continued wall penetration until the end. These tests show the largest upward movements of the cap, $\delta_c=0.78$ to 0.98 cm. In the second pattern, followed by CET 8-10, the cap initially rises at a gradually increasing rate that is greater than in the first pattern. However, the cap reaches a peak rate at a shallower depth range ($z_w=1.8$ to 2.2 cm) and then rises at a continually decreasing rate until reaching a total rise of between $\delta_c=0.54$ to 0.64 cm by the end of wall penetration. Tests CET4 and 11 do not fit either pattern. In CET4 the cap rises very swiftly, but then actually begins dropping until the wall has penetrated $z_w=3$ cm, whereupon the cap rises rapidly to a final height of $\delta_c=0.56$ cm. In test CET11, the cap begins to rise gradually according to pattern 1, but continues to rise at a linear rate beginning at a wall depth of $z_w=2$ cm to reach a final rise of $\delta_c=0.56$ cm by the end.

Comparisons between the cap displacement curves and cap force, wall force, and cap excess pore pressure behavior have not yielded an explanation for either the two general patterns of cap rise or the two aberrational patterns (CET4,11). Complicating the
interpretation of the cap rise patterns is the combination of many interacting factors including intercomponent friction and variation in total force.

5.2.4 Pore Pressure Generation in Soil Mass

The generation of excess pore pressure within the clay cake during the suction driving phase was measured by needle probes. As shown in Figure 5.1, the probe tips were located at a depth of approximately 2 to 2.5 cm below the clay surface and at three radial locations from the caisson centerline: r=0.0, 1.8, and 3.2 cm (P1, P2, and P3, respectively). Probes P1 and P2 measure pore pressures within the soil plug inside the caisson wall, while the third probe (P3), was located outside the wall during penetration. The overall behavior of the clay mass pore pressures is influenced by both the actions of the advancing caisson wall and the retreating cap.

The centerline pore pressure behavior, as measured by probe P1, is shown in Figure 5.16, which plots excess pore pressure versus wall tip penetration for 5 tests (CET7,9-12). Note that the probe tip depth is marked with a horizontal dashed line for each test. For the first $z_w=3.5$ cm of penetration, the pore pressure record for four of the tests (CET7, 9, 11-12) is similar in shape but muted in magnitude when compared to the pore pressure record beneath the cap (see Figure 5.13). During the first $z_w=0.3$ cm of penetration, the pressure drops slightly ($\Delta u=-0.005$ to -0.07 ksc) in response to the immediate tension applied to the cap. In CET9, 11, and 12, the pressure then gradually rises to maximum excess values ranging from $\Delta u=0.04$ to 0.06 ksc as the caisson wall approaches the depth of the probe tip. The maximum pressure during this range occurs when the wall is approximately 0.4 cm above the probe tip. Test CET7 does not show this gentle rise due to a slightly sluggish probe response (see Table 5.3a) and to a greater than average tensile force applied to the cap (see Figure 5.11). After reaching the maximum excess pore pressure in CET9, 11, and 12, and just after the wall passes the probe in CET7, the pore pressure begins a steady decline as the retreating cap exerts
greater tension on the soil plug. This decline continues to the end for CET7 and 12. In CET9 and 11, the pore pressure drops until the wall reaches $z_w=3.4$ to 3.6 cm, after which the pressure rises for the remainder of penetration. Note that at the end of penetration ($z_w=5.1$ cm), the magnitude of centerline pore pressure for each test is within $\Delta u=0.1$ ksc of the value measured beneath the cap (see Figure 5.13).

Centerline excess pore pressure behavior for CET10 deviates from the remaining four tests because of different cap behavior. After dropping initially to $\Delta u=-0.04$ ksc, the pore pressure declines sharply as the wall advances from $z_w=1$ to 2.6 cm. This drop is consistent with the drop in cap force and cap pore pressure (see Figures 5.11, 5.13). However, after reaching a minimum of $\Delta u=-0.26$ ksc, the pore pressure rises to a local peak of $\Delta u=-0.12$ ksc at $z_w=4.2$ cm and then falls to $\Delta u=-0.18$ ksc by the end of penetration. This pore pressure rise and fall during the second half of penetration could be directly related to the slight rise and fall in cap force, as shown in Figure 5.11.

Probe P2 measured the excess pore pressure of the soil plug at a radial distance of 0.6 cm from the inside of the wall. The pore pressure records for four tests (CET7-10) are shown in Figure 5.16. Two patterns of behavior are plainly visible. In CET7 and 10, after a slight initial drop, the excess pore pressure declines linearly from near-zero values at $z_w=1$ cm to a value ranging from $\Delta u=-0.19$ to -0.26 ksc by the end of penetration. The wall has no significant effect on the pore pressure in these tests because of the dominating effect of the tensile stress applied by the cap (see Figure 5.11), which is a direct result of the higher than average wall force magnitude (see Figure 5.6). Recall from section 5.2.1 that in these two tests, there possibly was metal-to-metal contact between the wall and inner slip ring, which would contribute friction to the wall force signature. In contrast, both the wall and cap contribute to the shape of the curves for CET8 and 9. In these tests, the excess pore pressure increases to a peak of between $\Delta u=0.2$ to 0.25 ksc by the time the wall has passed the probe tips and has reached a depth of $z_w=3.2$ to 3.5 cm. This
decrease in cap force causes the pore pressure to drop to values of $\Delta u = 0.06$ to 0.08 ksc by the end of penetration.

Excess pore pressure behavior at a radial distance of 0.6 cm outside the wall indicates some influence from the passing wall and little, if any, influence from the retreating cap. Figure 5.16 shows the excess pore pressure record as measured by probe P3 for four tests (CET7-8, 12). In the most representative tests, CET7-8, the pore pressure rises at a decreasing rate until reaching a peak of between $\Delta u = 0.14$ to 0.16 ksc at a wall depth of $z_w = 2.0$ to 2.1 cm. At this point, the wall is just 0.4 cm above the probe tip depth. With continued penetration, the pressure dissipates toward zero. During the final $z_w = 1.5$ cm of penetration for CET7, the pressure behavior is clouded by probe response instability (see Table 5.3a). Slow response masks the pore pressure behavior in CET12, but it is clear that suction driving generates only slightly positive pore pressures. In this test, excess pore pressure rises very slowly to a peak of $\Delta u = 0.05$ ksc at a wall depth of $z_w = 4$ cm and declines slightly toward the end of driving.

The magnitude of excess pore pressure beneath the cap and at the three locations within the soil mass (P1, P2, and P3) generated during installation by underbase suction never exceeded $\Delta u = \pm 0.3$ ksc. Much larger pore pressures ($\Delta u > 0.5$ ksc) are generated during the monotonic pullout phases, as described in sections 5.5 and 5.8.

5.2.5 Soil Surface Displacement

The vertical displacement of the clay surface exterior to the caisson was measured by up to five transducers located within the CET chamber at various radial distances from the caisson wall (see Figure 5.1). There are three general characteristics of clay surface settlement during suction penetration: 1) the clay surface generally depresses at a rate that increases with increasing wall penetration, 2) the magnitude of settlement decreases with increasing radial distance from the caisson wall, and 3) the overall magnitude of settlement is small relative to the wall penetration depth. These three settlement characteristics are
illustrated by Figure 5.17, which plots the clay surface displacement versus wall tip penetration for CET8. At a radial distance from the centerline of \( r = 4.2 \) cm, the surface has compressed \( \delta_s = -0.004 \) cm by the end of penetration. Note that this value is less than 0.1% of the final wall depth, \( z_w = 5.1 \) cm. A short distance further from the wall, the settlement drops dramatically to \(-0.0005 \) cm at \( r = 5.2 \) cm. By a radial distance of \( r = 9.8 \) cm, the depression is nearly immeasurable.

Clay surface settlement records for all tests are displayed in Figure 5.18, which shows the displacement at a specific radial distance for several tests. In general this figure shows that, allowing for some scatter in settlement magnitude from test to test, the clay surface settlement behavior is consistent with that described above for CET8. However, at the three closest radial locations, \( r = 4.2, 5.2, 7.8 \) cm, many of the tests indicate that the surface heaves slightly \( (<0.001 \) cm) prior to settlement. Moreover, the wall penetration depth at which peak heave occurs increases with increasing radial distance from the caisson wall. To illustrate this last point, consider the peak heave points for the first three radial distances (S1, S2, S3) in Figure 5.18. At a radial distance of \( r = 4.2 \) cm, peak heave occurs at a wall depth that ranges from \( z_w = 0.4 \) to 1.3 cm. At \( r = 5.2 \) cm, peak heave happens at a wall depth ranging from \( z_w = 0.5 \) to 2.8 cm. Finally, at a distance of \( r = 7.8 \) cm, the peak heave occurs at a corresponding wall depth that ranges from \( z_w = 2.3 \) to 3.2 cm. Note that at the furthest radial locations, \( r = 9.8 \) and 12.1 cm, there is no evidence of soil heave.

The soil settlement records displayed in Figure 5.18 represent the most reliable soil surface behavior for the suction penetration phase of testing. Important links between the movement of the outer soil surface, the soil displaced by the penetrating wall, and the soil plug rising beneath the cap are explored in Chapter 6.
5.3 OTHER INSTALLATION METHODS

In three caisson element tests, the caisson was installed using a method that differed from the standard underbase suction described in section 5.2. Recall that during suction driving, the wall penetrates the soil at a constant rate, while force is removed from the cap to offset increases in the wall force. Throughout driving, therefore, the total force \(F_{\text{tot}} = F_w + F_c\) is held constant. In test CET6, the caisson was installed using the same algorithm, but using a much slower penetration rate \(v_w = 0.03 \text{ cm/min}\) over the first \(z_w = 1\) cm. The wall penetrated the final \(z_w = 4.1\) cm at the standard rate \(v_w = 0.3 \text{ cm/min}\). For CET13, the cap force was held constant \(F_c = 14 \text{ kg}\) and the wall was driven at the standard rate. This simulation corresponds to installation of an open-ended pile. Finally, test CET14 targeted zero cap displacement during installation. Poor control of the cap displacement in CET14 led to cap movements up to \(\delta_c = 0.22\) cm (see Figure 5.24). In this section the results of these three tests are compared to the standard results from suction driving. Later sections illustrate that the different installation methods have little effect on subsequent equilibration and caisson pullout behavior.

5.3.1 CET6 - Partial Slow Rate Suction Installation

The slow initial penetration rate in CET6 \(v_w = 0.03 \text{ cm/min} \) over first \(z_w = 1\) cm was the result of an incorrect rate input parameter. The results from this two-phase installation test indicate that: 1) the overall caisson force results do not differ significantly from the standard tests, but 2) the minor change in installation rate does affect the cap movements and pore water pressure.

The caisson force behavior was relatively unaffected by the slow rate installation phase. The total force versus wall tip penetration for CET6 is shown in Figure 5.19. Throughout suction driving the total force for CET 6 remains constant at a value just above the design force of \(F_{\text{tot}} = 15.2 \text{ kg}\). During the slow drive rate in the first \(z_w = 1.05\) cm
of penetration, the total force is $F_{tot}=15.5$ kg. This rises to an average of $F_{tot}=16$ kg for the remainder of penetration, which occurs at the standard rate. These values of total force are within the range observed for the 9 standard suction driving tests presented in Figures 5.4 and 5.5. The wall force behaves like a standard suction installation test (see Figure 5.20). The initial penetration modulus, defined as the increase in wall force divided by the penetration increment ($M_w = \Delta F_w/\Delta z_w = 180$ kg/cm) is similar to those suction driving tests that did not suffer from a potentially disturbed soil surface (see Table 5.5). The definition of a transition zone is unclear in CET6 because the slow rate driving phase ends early at a depth of $z_w=1.05$ cm. However, a transition is apparent during the subsequent standard penetration as the wall force falls from $F_w=16$ kg to approximately $F_w=14$ kg from $z_w=1.05$ to 2.1 cm. Deep penetration behavior is consistent with the tests reported in section 5.2.1, as the wall force increases at a constant rate of $M_w=1.67$ kg/cm to reach $F_w=19$ kg by the end of driving. Figure 5.21 depicts the cap force versus wall tip penetration and reveals that the slow rate of driving during the first $z_w=1.05$ cm did not have any significant effect on the cap force behavior. As for the standard rate suction driving tests displayed in Figure 5.11 the cap force drops swiftly by more than $\Delta F_C=-10$ kg during the first $z_w=0.2$ cm of wall penetration. During deep penetration from $z_w=2.4$ to 5.1 cm, the cap force declines at a constant rate until reaching a final tensile value of $F_C=-2.5$ kg.

Figure 5.22 shows the excess pore pressure measured beneath the cap versus wall tip penetration. Very little positive excess pore pressure was generated during the two phases of suction driving in CET6. Unlike the standard rate suction installation tests portrayed in Figure 5.13, the slow rate of driving during the first $z_w=1.05$ cm allows significant partial drainage of pore fluid in the vicinity of the caisson. As a result, only a small excess pore pressure of $\Delta u=0.05$ ksc is registered after the wall has penetrated $z_w=0.05$ cm. For the remainder of slow penetration, this excess starts to decline due to both dissipation and a slight decrease in cap stress. During the standard rate penetration
from \( z_w = 1.05 \) to 5.1 cm, pressure barely builds to \( \Delta u = 0.1 \) ksc before falling to a slightly negative value of \( \Delta u = -0.02 \) ksc by the end as the cap slowly acquires a tensile stress.

Figure 5.23 shows the pore pressure generated by the centerline probe P1 in this two-phase suction driving test. Only very slight positive pore pressures (\( \Delta u = 0.04 \) ksc) are generated during the slow phase, and somewhat higher positive pore pressures (\( \Delta u = 0.12 \) ksc) are generated during the standard rate phase. Note that the slow rate of penetration prevented the development of negative excess pore pressures, an occurrence common to all standard rate tests (see Figure 5.16).

There is a large difference in cap movements between the two-phase test CET6 and the standard rate tests, as described in section 5.2.3. As shown in Figure 5.24, the cap in CET6 nudges upward by only \( \delta_c = 0.02 \) cm for the first \( z_w = 0.5 \) cm of penetration in order to develop the tension necessary to offset the wall force increase. This initial behavior is also apparent in standard rate suction driving tests, as shown in Figure 5.15. However, the cap in the standard tests continues rising, whereas the cap in CET6 drops to its original position by the end of slow penetration at \( z_w = 1.05 \) cm. This behavior may be related to the combination of dissipating pore pressure beneath the cap and the potential for tensile forces due to friction in the O-ring between the cap and wall. During the standard rate driving of CET6, the cap rises at a rate that generally increases with increasing wall penetration. Due to the slow initial wall penetration, the final cap rise is low at \( \delta_c = 0.37 \) cm. This value is lower than the cap rise in all standard rate tests, which displayed a cap rise ranging from \( \delta_c = 0.55 \) to 0.98 cm.

Due to the slow initial rate of penetration during the first \( z_w = 1.05 \) cm in CET6, the soil surface compresses a small amount (0.001 to -0.005 cm depending on the radial distance from the wall), as shown in Figure 5.25. This is in contrast to the initial near-wall soil heave exhibited in standard rate suction installation tests (see Figure 5.18). However, during the standard rate portion of CET6, from \( z_w = 1.05 \) to 5.1 cm, the soil surface displaces in a fashion similar to that for standard rate tests discussed in section 5.2.5.
Near the caisson wall at \( r = 5.2 \) cm the soil heaves approximately \( \delta_s = 0.001 \) cm until \( z_w = 3.7 \) cm, whereupon it compresses to the end of penetration. Similar behavior is apparent at greater distances from the wall until arriving at a radius of \( r = 12.1 \) cm, where the soil surface remains steady throughout standard rate penetration.

5.3.2 CET13,14 - Fixed Cap Force, Cap Displacement Installation

In both CET13 and CET14, the wall was driven at the standard rate of \( v_w = 0.3 \) cm/min. In test CET13, the cap force was held constant at \( F_c = 14 \) kg, while in test CET14, the algorithm attempted to maintain zero cap displacement. These two tests are discussed together because installation is achieved by supplying additional force to the system.

Figure 5.19 shows that the total force recorded in CET13 and 14 is significantly higher than in the standard rate suction installation tests. The total force signature for CET13 reflects that for the wall force and reaches \( F_{tot} = 37.6 \) kg by the end of driving. This value is more than double the total force measured for suction driving tests. The total force in CET14 varies widely during the first 0.3 cm of penetration. However, once the cap displacement is under control (\( \delta_c = 0 \)), the total force rises rapidly and is similar in magnitude (\( F_{tot} = 34.5 \) kg) to that measured in CET13.

As shown in Figure 5.20, which depicts wall force versus wall tip penetration, the wall force for CET13 behaved very much like a caisson wall with intercomponent friction in a standard suction driving test, but without a "hump-shaped" transition zone. During the initial zone, the wall responds stiffly, acquiring approximately \( \Delta F_w = 10.5 \) kg during the first \( z_w = 0.2 \) cm of penetration. Thereafter, the wall force continues increasing at a decreasing rate until a depth of \( z_w = 0.5 \) cm, whereupon the wall force increases at an average constant rate of \( M_w = \Delta F_w / \Delta z_w = 2.2 \) kg/cm until reaching a final value of \( F_w = 23.5 \) kg. The high rate of force pickup during deep penetration suggests a high friction contribution arising from sources described in section 5.2.1. Similar wall force behavior
was shown in the standard suction driving test CET10, which is illustrated in Figure 5.6. The wall force behavior in CET14 is unclear during the initial and transition stages due to poor cap control, but, like CET13, it is similar in overall shape to standard suction driving wall behavior. Examination of Figure 5.20 reveals a stiff initial wall response, as the force increases from $F_w=2$ to 11.2 kg during the initial $z_w=0.02$ cm of penetration. Uncontrolled movements in the cap from $z_w=0.02$ to 1.4 cm created sudden friction contributions arising from the O-ring connecting the cap and wall. Thereafter, from $z_w=1.4$ to 5.1 cm, the wall force rises at a constant rate of approximately $M_w=1.3$ kg/cm, which is just below the average and within the range of rates recorded for suction driving tests (see Table 5.5).

The cap force was controlled to be constant at $F_c=14$ kg throughout wall penetration in CET13, as Figure 5.21 shows. In contrast, for CET14 zero cap displacement was attempted. The cap force record for this test shows the significant effect of small uncontrolled cap movements, which are plotted versus wall tip penetration in Figure 5.24. In response to a small cap penetration of $\delta_c=0.22$ cm, the cap force shoots to nearly $F_c=32$ kg. As the cap is brought back to zero displacement, the cap force drops to tensile values before rising for the remainder of wall penetration to a final value of $F_c=18$ kg.

In general, large positive excess pore pressures are generated in the soil plug created by the penetrating caisson and in the soil mass external to the caisson. Figure 5.22 shows the development of excess pore pressure measured beneath the cap. Although the cap force remains constant throughout penetration, large excess pore pressures ($\Delta u=0.6$ ksc) developed during the initial 1 cm of penetration. Thereafter during deep penetration, the pressure slowly rises to $\Delta u=0.68$ ksc by the end. This pore pressure behavior is a result of pore pressure generated by the wall penetration and by displacements of the caisson cap (see below). The pore pressure generation beneath a fixed cap is obscured by small cap movements that occurred during CET14. As shown in Figure 5.24, the cap
dropped $\delta_c=-0.22$ cm during the first $z_w=0.5$ cm. This caused the excess pore pressure to rise to $\Delta u=0.9$ ksc, but stabilization of the cap dropped the pressure to $\Delta u=-0.25$ ksc. With the cap position held approximately constant for the remainder of penetration, displaced soil from the continued wall penetration caused the excess pore pressure to climb to $\Delta u=0.98$ ksc by the end of driving.

Only one pore pressure probe in test CET13 was considered reliable (adequately responsive), while all four probes performed well in CET14 (see Table 5.3). Figure 5.23, which shows the excess pore pressure in the soil mass vs. wall tip penetration, clearly indicates that the large total force applied to the system in these two tests raised the soil mass pore pressure to values larger than any recorded outside the wall for any test with underbase suction installation. In CET13, wherein the cap force was held constant, the probe was located at a distance from the outside of the caisson wall of $r_w=1.9$ cm (from centerline, $r=4.45$ cm). As the wall approaches the probe tip depth, a large excess positive pore pressure is generated, leveling out at $\Delta u=0.3$ ksc when the wall is 0.5 cm above the tip depth. This excess pressure is maintained as the wall passes the tip depth, and begins to slowly decline once the wall has penetrated approximately 0.6 cm beyond the probe depth. At the end of penetration, there is still an excess pressure of $\Delta u=0.25$ ksc at this location.

Probes P2 through P4 in CET14 measured pressure at a depth of 2.9 cm and were spaced radially from centerline at $r=1.8$, 3.2, and 4.45 cm, respectively. Probes P2 and P3 both recorded pore pressure behavior similar to that beneath the cap. Figure 5.23 shows the pore pressure generated at a distance of 0.5 cm (P2) from the inside wall for CET14. Except for the magnitude of initial peaks and valleys, this curve mimics that for the pore pressure beneath the cap. In fact, the excess pore pressure from a wall penetration of $z_w=1.5$ to the end is nearly identical, indicating that the entire soil plug created within the caisson is experiencing the same pore pressure generation pattern. Just outside the caisson at a distance from the wall of $r_w=0.6$ cm, P3 also measures a similar pattern.
However, as the wall approaches to within 0.3 cm of P3's tip depth, the increasing excess pressure peaks and begins slowly falling. As the wall passes the tip, the excess pore pressure slowly continues falling and reaches $\Delta u=0.5 \text{ ksc}$, which is half the value measured within the soil plug. Probe P4, which was located $r_w=1.91 \text{ cm}$ from the caisson wall, only measured a gradual increase in excess pore pressure, reaching $\Delta u=0.35 \text{ ksc}$ by the end of penetration.

At 2 cm above the clay bottom, probe P1 in CET14 measures a gradually rising excess pore pressure that reaches $\Delta u=0.25 \text{ ksc}$ by the end of penetration when the wall is approximately 7 cm above the probe tip. Note that the positive and negative swings in pore pressure so apparent at the surface location just beneath the cap (see Figure 5.22) are not reflected at all at the clay bottom. However, at the end of driving, the combined effect of the cap movements and the wall penetration was enough to raise the pressure to a value of $\Delta u=0.25 \text{ ksc}$, which is larger than any soil mass pore pressure recorded for any of the test suction installation tests.

Cap displacements during wall penetration are shown in Figure 5.24. The cap in CET13, which has a controlled constant cap force of $F_c=14 \text{ kg}$, initially moves downward by $\delta_c=-0.06 \text{ cm}$ during the initial $z_w=1.2 \text{ cm}$ of penetration. This movement implies partially plugged penetration, which is probably a result of the constant stress distribution on the rigid cap. As the wall initially penetrates and picks up load, the inside surface of the caisson becomes highly stressed, which could possibly cause more soil to displace toward the outside. For the remainder of penetration, the cap movement is similar to pattern 2 exhibited by CET8-10, as described in section 5.2.2. From $z_w=1.2$ to $3.2 \text{ cm}$, the cap rises at an increasing rate, but thereafter it rises at a decreasing rate until reaching a final rise of $\delta_c=0.52 \text{ cm}$, which is just below the range for standard rate tests. The goal of caisson installation in CET14 was to maintain zero cap displacement, but as Figure 5.24 shows, problems with control led to some cap movements. The cap drop of $\delta_c=-0.22 \text{ cm}$ at a wall penetration of $z_w=0.5 \text{ cm}$ and the slight cap rise of $\delta_c=0.06 \text{ cm}$ at $z_w=3.2 \text{ cm}$
severely affected the cap force, total force, and pore pressure behavior throughout penetration, as noted before. However, the change in cap directions only lightly affected the wall force behavior, which was similar in overall shape and magnitude to the most representative standard suction driving tests.

Responding to the fixed cap force in CET13, the soil surface heaves $\delta_s=0.003$ to 0.0075 cm, depending on the radial location. Figure 5.25 shows that at locations closer to caisson wall, $r=4.2$ and 5.2 cm, the surface heaves initially and then compresses toward the end of wall penetration. The heaving is caused by the movement of soil displaced by the dropping cap, while the compression is in response to the soil volume moving up with the cap (cap movements were shown in Figure 5.24). At the farther radial locations, $r=7.8$, 9.8, and 12.1 cm, the surface heaves less, but the heaving continues right to the end of wall penetration without any compression. Note that the peak heave at both $r=4.2$ and 5.2 cm is approximately $\delta_s=0.0075$ cm, which is 5 times greater than the peak heave of 0.0015 cm recorded for standard rate suction tests (see S2 for CET5 in Figure 5.18), but this magnitude is still quite small (~5% of wall thickness). As for the caisson force and soil pore pressure records, the soil displacement record for CET14 is a product of the unplanned cap deviations from zero-displacement control. Responding to compression and uplift cycle of the cap during the first $z_w=1.5$ cm of wall penetration, the soil surface first heaves then compresses, as shown in Figure 5.25. After the cap is stabilized, the soil surface undergoes tremendous heave, as the wall-displaced soil has nowhere to go but outside the walls. At a distance of $r=5.2$ cm, the soil initially heaves $\delta_s=0.013$ cm, compresses to $\delta_s=-0.002$, and then reaches a final heave value of $\delta_s=-0.021$ cm (14.4% of wall thickness). At each radial location beyond $r=5.2$ cm this pattern is exhibited, with the magnitude diminishing with increasing distance from the wall. Note that at the closest radial location, $r=4.2$ cm, the initial heave and compression is larger than that at $r=5.2$ cm, but the final heave is only $\delta_s=0.012$ cm. This suggests that by the end of penetration, the
soil surface topography incorporates a "bulge" that peaks at a radial distance located between \( r = 4.2 \) and \( 7.8 \) cm.

5.4 EQUILIBRATION

Immediately following installation for all tests, the caisson was allowed to equilibrate with the surrounding soil for at least 18 hours (Table 5.2). The basic behavior of the caisson forces, caisson settlement, soil pore pressure, and soil compression are presented in this section\(^3\). During the set-up phase, the total force on the caisson \( F_{\text{tot}} = F_w + F_c \) was held constant at \( F_{\text{tot}} = 15.2 \) kg \((= \sigma'_{vc} A_{\text{tot}})\), while allowing zero relative displacement between the cap and wall.

5.4.1 Caisson Force Distribution

Typical caisson force behavior during equilibration is illustrated in Figure 5.26, which plots the caisson wall, cap, and total forces as a function of time (log scale) for CET9, which was chosen for the quality of its test control and instrumentation data (Table 5.3b). The total force remains constant at \( F_{\text{tot}} = 15.1 \) kg, which is just 0.1 kg below the design value of \( F_{\text{tot}} = 15.2 \) kg \((= A_c \sigma'_{vc})\). Redistribution of the wall and cap forces is rapid. After reaching a peak wall force of \( F_w = 17 \) kg by the end of penetration, the wall sheds 3 kg within 2 minutes and maintains a range of \( F_w = 14 \) to 15 kg for the remainder of set-up. Similarly, the cap, which holds a tensile load of \( F_c = -1.5 \) kg at the end of installation, acquires 2.5 kg of compressive load in 2 minutes and holds a slightly positive load of between \( F_c = 0 - 1 \) kg for over 33 hours. As discussed in section 4.3.2, the minor force fluctuations observed beyond 2 minutes probably are due to inter-component friction. The

\(^3\)Electrical power fluctuations marred otherwise excellent equilibration behavior in CET4, 5, and 10, etc. most results from these tests are not presented.
caisson force distribution pattern exhibited by CET9 is similar to the pattern in every test. The remainder of this section examines this similarity by focusing on each caisson force component, starting with total force.

**Total Force**

Figure 5.27 shows the total force versus log of time for 6 standard suction driving tests (CET3, 7-9, 11-12) and two non-standard tests (CET13, 14) during the equilibration phase. Note that for both the suction driving and equilibration phases, the design total force was $F_{\text{tot}}=15.2$ kg. As explained in section 4.5.1, small differences in the control program led to some variation in total force during driving. This variation is visible at the start of set-up, at which point the total force varies from $F_{\text{tot}}=15.2$ to 17.2 kg. Within 3 minutes of the start of set-up, however, the total force for all tests reaches the design value of $F_{\text{tot}}=15.2 \pm 0.3$ kg. For CET13, in which the caisson was driven with a constant cap force, the total force at the end of penetration was $F_{\text{tot}}=33.6$ kg. Within 2 minutes of the start of set-up, the design value of total force had been established. In CET14 the cap position was fixed during penetration and this led to a final total force of $F_{\text{tot}}=37.6$ kg. Because no data was recorded during the first 5 minutes of set-up, the dashed line in Figure 5.40 shows the total force dropping to the design value within 5 minutes of the start of set-up. It is likely that the design total force of $F_{\text{tot}}=15.2$ kg would have been established within 3 minutes, as it was for the other tests.

**Wall Force**

The wall force measurements during set-up exhibit a quick redistribution of force from the end of penetration, but with a bit more scatter in equilibrium force from test to test compared to the total force records. Figure 5.27 shows the wall force versus log of time for CET3, 7-9, and 11-14. The wall force for the six standard rate suction driving tests (CET3, 7-9, 11-12) starts at a value ranging from $F_w=17$ to 22.5 kg and falls in 3
minutes to a range of $F_w = 13.5$ to $16.5$ kg, which is then held with a few minor deviations until the end of equilibration. The variation in force among the tests can be attributed to the different values of contributing intercomponent friction, which was described in section 5.2.1. In CET13 the wall force starts high at $F_w = 23.5$ kg, drops to $F_w = 3$ kg in 30 seconds, and then slowly builds to a value of approximately $F_w = 14$ kg by the end of equilibration. The wall force drop to $F_w = 3$ kg and subsequent build-up to equilibrium value was caused by control program overcompensation: the wall was lifted a bit too much as the control program attempted to reduce the high final penetration total force ($F_{tot} = 33.6$ kg) to the set-up design value ($F_{tot} = 15.2$ kg). Similar overcompensation caused the wall force in CET14 to drop first from $F_w = 15.9$ kg to 3.5 kg in 5 minutes before slowly building up to approximately $F_w = 10.5$ kg by the end of set-up. Because of the lack of recorded data before 5 minutes, it is possible that the wall force reached a value lower than $F_w = 3.5$ kg within 5 minutes of the start of set-up. Note that the shape of the wall force curve in CET14 indicates that the wall may still be accumulating force at the end of 24.9 hours. Given more set-up time, it is conceivable that the wall force may reach the equilibration range of $F_w = 13.5$ to $16.5$ kg.

Cap Force

Because the total force is held constant during set-up, the cap force behavior is a nearly perfect reflection of the wall force record. As Figure 5.27 shows, the initial tensile cap force ranges from $F_c = -6$ kg to 0.2 kg for the suction driving tests (CET3, 7-9, 11-12). Within 3 minutes the cap has shed most of its force. For the remainder of set-up, the cap force ranges from $F_c = -0.5$ to 1.5 kg with a few small deviations. In CET13 and 14 the cap force at the end of penetration is high at $F_c = 14$ and 17.8 kg, respectively. Due to the time required to dissipate the large installation pore pressures, the compressive cap force in these tests slowly declines. By the end of equilibration, it appears that the cap force in
CET13 has reached an equilibrium value of $F_c=2$ kg. However, in CET14 the cap force would probably continue dropping if allotted more set-up time.

From the evaluation of force components during post-installation equilibration in suction driving tests, it is clear that the cap quickly sheds any residual tensile force from installation and carries very little load for the remainder of set-up. The wall force balances the cap behavior and carries nearly all of the total caisson force for the rest of equilibration. For tests wherein the caisson was installed using non-suction methods, the longer dissipation of much greater installation pore pressures delays the redistribution of cap and wall forces; by the end of equilibration the wall carries most of the load and the cap retains a small (~2-5 kg) compressive load.

5.4.2 Pore Pressure Dissipation

The pore pressure pattern measured beneath the cap and in the soil mass generally reflects the redistribution of the caisson forces during the set-up stage. Figure 5.28 plots the excess pore pressure beneath the cap that was generated during installation for 6 suction driving tests (CET4, 7-9, 11) and 2 non-suction tests (CET13-14). Pressure for the suction driving tests starts at low positive or negative values ($\Delta u=0.04$ to -0.21 ksc) due to the low positive or tensile stress on the cap at the end of suction driving (see Figures 5.11, 5.13). As the cap force rises during the first 3 minutes of equilibration, the pore pressure in these tests rises to a range $\Delta u=0.02-0.08$ ksc. Beyond 3 minutes the cap force is constant, and complete dissipation of the small excess pore pressures occurs in approximately 16 hours. The excess pore pressure in tests CET13 and 14 responds immediately to the pre-set change in total force $F_{tot}$. For CET13 the pre-set value, $F_{tot}=15.2$ kg, was achieved within approximately 3 minutes and is accompanied by a pore pressure drop beneath the cap from $\Delta u=0.68$ ksc to 0.3 ksc. Thereafter, as the loads on the cap and wall redistribute, these remaining excess pressures dissipate in approximately
16 hours. The excess pore pressure record in CET14 also reflects the unloading of the caisson (t<5 minutes) and then the dissipation accompanied by stress redistribution within 16 hours.

The same dissipation trends are evident in excess pore pressure measurements within the soil mass. Figure 5.28 shows the excess pore pressure versus time (log scale) for probes P1, P2, P3, and P4 located at radial distances from centerline of 0.0, 1.8, 3.2, and 4.45 cm, respectively. The behavior of the pore pressure within the soil plug inside the caisson, as measured by P1 and P2, is very similar to the dissipation of pore pressure beneath the cap, which was discussed above. As shown in Figures 5.28, the soil plug excess pore pressure rises during the first 3 minutes of set-up to a slightly positive range between Δu=0.07 to 0.14 ksc in response to cap force equilibration. Thereafter, the excess pore pressure dissipates completely by approximately 16 hours (1000 minutes). The excess pressure at the end of set-up is not zero, but is approximately Δu=0.00±0.05 ksc due to temperature and barometric pressure variations occurring throughout the approximately 24 hours of set-up. Note in Figure 5.28 that the pore pressure measured at the clay bottom in CET14 remains steady at approximately Δu=0.22 ksc until 30 minutes into set-up, whereupon it drops rapidly for 10 minutes and then slowly dissipates for the remainder of set-up. This sudden drop is due to the evacuation of soil that had been blocking the chamber bottom drainage line. At the probe P3 location just outside the caisson at a radius of r=3.2 cm, the excess pore pressure dissipation behavior is identical to that for soil plug inside the caisson except for the initial 3 minutes for the suction installation tests. The redistribution of the tensile cap force during initial set-up does not affect the fluid pressure in the soil mass exterior to the caisson, as the excess pore pressure at the probe P3 location begins dissipating within only a few seconds of the start of set-up. Measurement of pore pressure at the furthest location by P4 (r=4.45 cm) was only

4All probes were located at a depth of between 2.0 and 2.9 cm below the surface of the clay prior to installation except for probe P1 in CET14. This probe measured pressure at 2 cm above clay bottom.
available for the non-suction installation tests, CET13 and 14. Figure 5.28 shows that the excess pressure dissipates completely within 10 hours for CET13 and to a negligible value by the end of set-up in CET14\(^5\).

5.4.3 Caisson Displacement

During equilibration the model caisson displaces as a monolithic unit. This is achieved by controlling to cap to follow the wall displacement (section 3.1.4). Figure 5.29 shows the typical displacement pattern for CET8 including the displacements of the wall and cap, and the relative displacement of the wall and cap. The caisson wall and cap settle at an increasing log rate until reaching approximately -0.015 cm after 70 minutes. For the remainder of set-up, the caisson settles at a constant log-linear rate of approximately -0.035 cm per log time cycle until reaching a final settlement of just over -0.07 cm. Note that in this test, the wall displaces downward nearly -0.002 cm relative to the cap during the first 10 minutes. Figure 5.30 shows the wall and cap displacement, respectively, for the other reliable tests. The overall trend is clear. The caisson settlement log rate increases until reaching a certain point in time, after which the settlement rate is approximately log-linear. The rate of log linear settlement varies from test to test. This variation is probably a result of drainage condition differences from test to test because the effectiveness of the clay surface drainage varied. The final settlement varied from -0.021 to -0.092 cm (i.e., approximately 10-65% of the wall thickness). Note that differences between the cap and wall displacement for a particular test indicates a relative displacement at some point during set-up that usually was due to electrical control disturbances. The caisson in test CET13 heaves by approximately 0.012 cm due to unloading of the caisson (t<3 minutes) and then undergoes a net settlement of -0.04 cm as the pore pressures equilibrate.

\(^5\)The excess pore pressure at the end of set-up in CET14 is \(\Delta u<0\) due to a slightly inaccurate zero reading at the start of installation.
The caisson settlement behavior for CET14 is depicted separately in Figure 5.31. Settlement appears to proceed much like the other tests until 30 minutes into set-up, whereupon the settlement rate abruptly increases and leads to a higher than normal final settlement of approximately -0.25 cm. This is due directly to the evacuation of a soil plug in the chamber bottom drainage line, an action that caused the abrupt increased dissipation of excess pore pressure near the clay bottom (see section 5.4.2).

In Chapter 6, comparisons are made between these caisson settlements and one-dimensional compression data for Resedimented Boston Blue Clay (RBBC).

5.4.4 Soil Surface Settlement

The settlement of the soil surface outside of the caisson follows the same pattern as the caisson. However the magnitude of settlement generally decreases with increasing distance from the caisson. Figure 5.32 shows the soil surface settlement versus log of time at five radial locations in CET8 during post-installation equilibration. The soil surface settles at an increasing log rate until between 5 to 8 hours have elapsed, after which settlement continues at a linear rate with log time. Note that up to the constant log linear settlement zone, the settlement rate decreases with increasing radial distance from the caisson, ranging from $\delta_s=-0.035$ cm at S1 to $\delta_s=-0.013$ cm at S5. The surface settlement for the remaining reliable tests is shown in Figure 5.33. As for the caisson settlement, the variation in settlement rates among the tests can be attributed to variations in the effectiveness of pore fluid drainage. Although there are some exceptions, in general the settlement decreases with increasing distance from the caisson wall. At a radius of $r=4.2$ cm, the settlement varies from $\delta_s=-0.027$ to -0.083 cm, decreasing to a range $\delta_s=-0.002$ to -0.057 cm at $r=12.1$ cm.

As for the caisson displacement, the soil surface settlement in CET14 was affected greatly by the increased drainage approximately 30 minutes after the start of set-up. As
shown in Figure 5.34, the final surface settlement ranges from $\delta_s = -0.253$ to -0.3, which is 3 to 10 times (depending on radial location) greater than the settlement in other tests.

As for the caisson settlement, comparisons are made in Chapter 6 between the soil surface settlement record and the previous compression patterns for RBBC.

5.5 MONOTONIC PULLOUT 1

The results of the initial monotonic pullout phase in 11 tests (CET3-6,8-14)$^6$ are presented in this section. After the caisson had settled and the installation pore pressures had dissipated during the equilibration phase, the caisson was withdrawn from the soil at a rate $v_w = 0.03$ cm/min while maintaining zero relative displacement between the cap and wall. In five tests (CET3-6,8) the caisson was pulled for at least $\delta_w = 0.3$ cm$^2$ in order to measure tensile capacity. Thereafter the caisson was either pulled at a rate of $v_w = 0.3$ cm/min. until complete extraction (CET3,4, and 8) or was re-equilibrated within the soil (CET5-6). For the last six tests (CET9-14), the caisson was pulled to a specific tensile load level in order to investigate the effects of sustained tension. The total displacements in these tests are less than 0.01 cm.

5.5.1 Caisson Force Distribution

Figure 5.35 shows relative contributions of the caisson force components ($F_w$, $F_c$, and $F_{tot}$) as a function of wall displacement (at two scales) during monotonic tensile loading in CET8. The caisson exhibits a very stiff initial load-displacement response; an applied tensile load of $\Delta F_{tot} = 30$ kg was required to displace the wall by 0.02 cm (i.e., $F_{tot} = -15$ kg in tension at $\delta_w = 0.02$ cm). The caisson response is dominated initially by the

$^6$Results for CET7 are not shown due to excessive inner slip ring/caisson wall friction (see section 5.2.1).
$^7$0.3 cm = 2$t_w$, where $t_w$ is the caisson wall thickness.
behavior of the wall, which carried an applied load of $\Delta F_w=24.5$ kg in the same
displacement range ($\Delta F_w/\Delta F_{tot}=82\%$), and reached a tensile force $F_w=-11$ kg at $\delta_w=0.02$
cm. In contrast, the measurements show a very abrupt change in cap force ($\Delta F_c=5$ kg),
which then remains constant in the range 0.001 to 0.02 cm, and accounts for only 20\% of
the total tensile load on the caisson at $\delta_w=0.02$ cm. The enlarged displacement scale
shows that $\delta_w=0.02$ cm corresponds to a yield point in the system response. Further
displacement causes tensile hardening response of both the cap and wall forces, such that
the maximum resistance of both components and the total caisson occurs at a
displacement $\delta_w=0.2$ cm. Figure 5.35 shows that at a displacement $\delta_w=0.23$ cm in CET8,
the caisson has a tensile load capacity $F_{tot}=-22.4$ kg, to which the cap force contributes
40\% ($F_c=-8.9$ kg) and the wall force 60\% ($F_w=-13.5$ kg). With a couple of exceptions, the
monotonic pullout force behavior exhibited by CET8 is typical of all tests.

**Total Force**

Figure 5.36 shows the total force versus wall displacement for six tests (CET3-6,8). The overall response is very similar to that outlined above for CET8. In all cases,
the caisson exhibits a very stiff initial response, reaching a tensile force $F_{tot}=-7$ to -12.5 kg
within a displacement $\delta_w=0.01$-0.02 cm. There is a well-defined yield point at $\delta_w=0.01$-
0.02 cm, after which the caisson continues to pick up additional tensile load, but at a
rapidly decreasing rate. The maximum caisson capacity ranges from $F_{tot}=-17.6$ to -23.95
kg and occurs at $\delta_w=0.17$ to 0.3 cm. There is only slight post-peak softening of the
resistance for displacements up to $\delta_w=0.4$ cm. The total capacity in CET3 is slightly
lower than the other tests by about 3 kg. Control problems caused the cap to move more
slowly than the wall after a displacement of approximately $\delta_w=0.01$ cm, and thus the cap
was not able to mobilize as much tension as the other tests (see Figure 5.36). If the data
from test CET3 is neglected, the total force behavior appears very consistent. The
average caisson capacity for the remaining 4 tests is $F_{\text{tot}} = -22.8 \pm 0.7$ kg and occurs at a displacement $\delta_w = 0.25 \pm 0.03$ cm.

The total force record during early withdrawal is particularly consistent. Figure 5.37 plots the caisson force components in 8 tests (CET6,8-14) for $\delta_w = 0-0.02$ cm. Note that CET3, 4, and 5 are not included because they were hampered by relative displacement control problems during early withdrawal (see section 4.1.4). Starting from an equilibrium compressive force that ranges from $F_{\text{tot}} = 14.1$ to 15.6 kg, the caisson in all tests responds very stiffly to axial withdrawal. The initial stiffness ranges from $M_{\text{tot}} = \Delta F_{\text{tot}} / \Delta \delta_w = 5000-9000$ kg/cm.

**Wall Force**

Individual wall force versus wall displacement records for the 5 capacity tests (CET3-6,8) are shown in Figure 5.36. The wall initially reacts stiffly and carries a tensile load $F_w = -3$ to -12 kg at the yield displacement $\delta_w = 0.01-0.02$ cm. The maximum wall resistance ranges $F_w = -12$ to -14 kg (neglecting one outlier test CET4) and is mobilized at a displacement range $\delta_w = 0.15-0.25$ cm, which is similar to the range of mobilization for the maximum caisson capacity ($\delta_w = 0.17-0.3$ cm). In CET4 the wall force exhibits an uncharacteristic cycling pattern that is caused by changing frictional conditions between the caisson wall and cap. Note that the cap force for this test shows a complementary cycle. The total force in CET4 is unaffected by the inter-component friction. If the percentage wall force contribution is calculated for each of the 3 tests that had representative cap and wall force behavior (CET5, 6, and 8), the wall contributes an average of 58% of the total capacity.

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8In the sustained load tests (CET9-14), the total force record ends at $\delta_w \leq 0.003$ cm of displacement, but it is clear that total force for these tests was rising according to a trend similar to that for CET 6 and 8, which show a total force increasing at a decreasing rate.
Figure 5.37 shows the initial wall response ($\delta_w=0.0$ to $0.02$ cm) for 8 tests (CET6,8-14). The initial compressive forces range (at equilibrium) from $F_w=9.7$ to $16.35$ kg, and the data reveal an initial wall stiffness in the range, $M_w=\Delta F_w/\Delta \delta_w=2600-3000$ kg/cm, which is significantly lower than that of the overall caisson. Even though tests CET9-14 were interrupted at small displacements ($\delta_w<0.007$ cm), their trend is clearly within the limits set by the capacity tests, CET6 and 8.

**Cap Force**

Figure 5.36 shows the cap force records for CET3-6,8, which contains a subset of reliable tests CET5-6,8. In these more reliable tests, the cap carries almost no load at initial equilibrium, but mobilizes $F_c=-4$ to $-7$ kg of tension within $\delta_w=0.001$ cm (i.e., almost instantaneously), and then slowly accumulates $\Delta F_c=-3.5$ to $-5$ kg as the displacement mobilizes peak capacity at $\delta_w=0.2$ cm. The maximum cap force almost coincides with the displacement necessary to mobilized total caisson capacity. Control problems beset the cap force data in tests CET3-5, causing a characteristic delay in the initial force mobilization (at $\delta_w<0.02$ cm). In test CET3, the cap withdrew at a slower rate than the wall, and this rendered the cap force record non-representative. In CET4, the cycling behavior evident in the cap force was due to changing friction between the caisson cap and wall, as discussed for the wall force record above. Considering just the representative tests CET5-6,8 the average cap force at the moment of total capacity was $F_c=-9.7\pm1.4$ kg. Based on those tests that had both representative cap and wall behavior (CET5, 6, and 8), the cap force contributes an average of 42% to the total capacity.

Figure 5.37 focuses on the initial cap behavior ($\delta_w=0.0$ to $0.02$ cm) behavior for 8 tests (CET6,8-14). These data suggest that the cap exhibits a very stiff initial response, $M_w=\Delta F_c/\Delta \delta_w=2500-6400$ kg/cm with a well-defined yield point at $\delta_w=0.003$ cm.

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950% of the peak cap level is mobilized almost instantaneously at $\delta_w<0.001$ cm.
**Best Estimate of Component Force Behavior**

In order to interpret the behavior of the caisson force components during monotonic pullout, a "best estimate" of the total, wall, and cap force behavior was developed. Figure 5.38 depicts the most representative total, wall, and cap force records at both small displacements, $\delta_w=0.0-0.02$ cm, and for displacements up to peak resistance ($\delta_w=0.0-0.4$ cm). The lower and upper boundaries are derived from the most reliable tests depicted in Figures 5.36-5.37. The initial small displacements show clearly the yield points for the cap ($\delta_w=0.003$ cm) and wall ($\delta_w=0.02$ cm) components. The best estimate of caisson capacity ranges from a low, $F_{tot}=-22.2$ kg at $\delta_w=0.257$ cm to a high, $F_{tot}=-23.8$ kg at $\delta_w=0.237$ cm, with an average $F_{tot}=-23$ kg at $\delta_w=0.25$ cm. At this displacement, the best estimate of wall force ranges from $F_w=-11.4$ to $-14.9$ kg (50-65% of total), while the cap ranges from $F_c=-9$ to $-11.8$ kg (40-51% of total).

**Fast Rate Monotonic Pullout**

In tests CET3, 4, and 8, the caisson was loaded monotonically to beyond peak resistance at a rate of $v_w=-0.03$ cm/min. Thereafter, the rate of loading was increased to $v_w=-0.3$ cm/min until complete caisson extraction (i.e., $\delta_w=5.1$ cm). Figure 5.39 shows the total, wall, and cap force behavior, respectively, for both the slow and fast pullout phases for these three tests. In each case, the applied forces relax as the load rate is adjusted (because the control program was manually stopped and restarted). Note also that after the end of slow pullout in test CET4, the caisson continued to displace an additional 0.15 cm due to a small sustained tensile load. At $\delta_w=0.3$ cm in CET3, the total force is $F_{tot}=-18$ kg, which relaxes to $F_{tot}=-17$ kg. Loading at the higher displacement rate causes a very soft response such that after $\Delta\delta_w=0.1$ cm, the total force reaches nearly $F_{tot}=-25$ kg, an increase of $\Delta F_{tot}=-7$ kg over the peak resistance for slower loading. The total force in CET4 and 8 increased by approximately $\Delta F_{tot}=-3$ kg over their total force values at the end of slow pullout. Due to control problems during the slow pullout in
CET3 and 4 mentioned above, the magnitude of the increased capacity resulting from the faster pullout cannot be compared from test to test. However, the shape of the fast rate total force curve appears similar in all three tests. In all caissons, peak resistance occurs within 0.1 cm displacement at the higher imposed velocity, after which the total force decreases at a constant rate until reaching approximately $\delta_w=4$ cm, at which time the remaining tensile force is shed rapidly.

As shown in Figure 5.39, the wall and cap force records during the fast rate pullout do not show any consistent trends. In CET3, the wall force only rises $\Delta F_w=-1$ kg above the final slow rate value before dropping at a constant rate for the remainder of pullout. Most of the rate sensitivity is due to the behavior of the cap force, which increases by approximately $\Delta F_c=-8$ kg within $\Delta \delta_w=0.1$ cm at the higher displacement rate. In both CET4 and 8, the wall and cap records are affected by intercomponent friction, but appear to contribute equally to the total caisson force throughout fast rate pullout. Further insight into the rate effects on caisson forces are discussed in section 5.8, which covers the second round of monotonic pullout tests (Figures 5.70-5.78).

5.5.2 Pore Pressure Generation

Monotonic pullout of the caisson generates large negative excess pore pressure in the soil plug within the caisson. Figure 5.40 plots the excess pore pressure measured beneath the cap versus the wall displacement for two tests in which the caisson was displaced to its capacity (CET5-6) and six additional tests in which only small displacements were imposed (CET9-14). The underbase excess pore pressure matches very closely the average basal stress due to the cap load, $\bar{\sigma}$. In 7 tests (CET5,9-14) between $\Delta u=-0.2$ and -0.4 ksc is generated at the cap yield displacement $\delta_w=0.003$ cm. With continued displacements, more (negative) excess pressure is generated, reaching a peak at $\delta_w=0.2$ cm, where the cap force is also mobilized (see Figure 5.36). The maximum excess pore pressures for CET5 and 6 are $\Delta u=-0.5$ and -0.46, respectively. The
variation in magnitude can be attributed to variations in the mobilized cap load (see Figure 5.36). Friction between the cap and wall caused the cycling pore pressure measurement in CET7, but the overall trend of large excess negative pressure generation is consistent with the trends for CET5 and 6.

Figure 5.41 shows that the pore pressures measured by probe P1, located approximately 2 to 3 cm above the wall tip on the centerline of the caisson (see Table 5.1), are very similar to those measured below the cap. In CET5, P1 was located at the centerline, but at a depth above the wall tip of approximately 1 cm. The records from 7 tests (CET5-6,9-12, 14) show that large negative excess pressures ($\Delta u = -0.42$ to $-0.5$ ksc) are mobilized within the soil plug at full caisson resistance ($\delta_w = 0.2-0.3$ cm). During the initial phase of the tests ($\delta_w = 0-0.02$ cm), much greater negative pore pressures are generated for tests CET9-12 than for CET6. There is no apparent explanation for this discrepancy. Note that practically no excess pore pressure is generated at the base of the clay cake, as evidenced by the probe response in CET14.

The probe P2 measures pore pressure at the same depth as P1, but at a radius $r = 1.8$ cm (i.e., 0.6 cm from the inside caisson wall). Figure 5.41 presents the P2 excess pore pressure record for six tests (CET4, 8-10, 13-14). The results show large negative excess pore pressures: $\Delta u = -0.25$ and $-0.4$ ksc generated at $\delta_w = 0.003$ cm. However, as the caisson displacements continue, the P2 excess pore pressure become significantly higher than those measured at other locations within the plug (see Figures 5.40, 5.41), reaching a peak value, $\Delta u = -0.68$ ksc, at $\delta_w = 0.21$ cm for CET8. With continued loading of the caisson, the P2 pore pressure decreases slightly.

Completely different pore pressure behavior is measured in the soil mass outside the caisson. Figure 5.41 show the pore pressure records for probes P3 and P4, respectively, both of which were located at the same depth as P1 and P2, but at radii $r = 3.2$ cm.

\footnote{Note that P1 in CET14 was located approximately 2 cm above the clay cake bottom and 6 cm below the caisson wall tip.}
cm (i.e., 0.66 cm from the outside wall) and \( r = 4.45 \) cm, respectively. The P3 probe measurements at small displacements \( \delta_w < 0.003 \) cm show a wide variation in response. In tests CET9-11, the P3 measurements follow closely the pore pressure measurement inside the caisson (\( \Delta u = -0.2 \) to -0.4 ksc, see Figures 5.40, 5.41), while much smaller negative pressures occur in tests CET8,14. In test CET8, the P3 pore pressure stabilizes at \( \Delta u = 0.1 \) ksc by \( \delta_w = 0.02 \) cm and decreases with continued caisson extraction. By the time the caisson capacity is fully mobilized at \( \delta_w = 0.2 \) cm, P3 pore pressure has reached \( \Delta u = -0.1 \) ksc. There is minimal data available from probe P4 (tests CET13,14 only).

**Fast Rate Monotonic Pullout**

There is very limited pore pressure data for the three pullout tests at the higher displacement rate \( v_w = -0.3 \) cm/min. However, a significant rate effect can be seen in these data. Figure 5.42 shows the excess pore pressure measured by probe P2 inside the caisson for CET8. The relaxation of total stress that occurs as the displacement rate is adjusted causes a significant change in the P2 pore pressure (from \( \Delta u = -0.44 \) to -0.21 ksc). Loading at the higher displacement rate causes the P2 pore pressure to rise rapidly to a peak value which is -0.15 lower than that observed at the end of the previous phase. For the remainder of pullout, the negative excess pressure declines at a constant rate.

The only record of fast pullout rate pore pressure behavior in the soil mass outside the caisson was measured by probe P3 (\( r = 3.2 \) cm) in CET8 (see Figure 5.42). Fast rate pullout generates no significant pore pressures at this location until reaching a displacement of \( \delta_w = 2.05 \) cm, whereupon negative excess pore pressure is generated as the wall tip rises past the probe tip. After reaching a maximum negative pressure of \( \Delta u = -0.18 \) ksc by a pullout distance of \( \delta_w = 3.3 \) cm, the negative excess pore pressure declines.
5.5.3 Soil Surface Displacement

Data from the five soil surface displacement transducers (S1-S5, Figure 5.2) indicate a consistent pattern of surface movement during monotonic pullout from test to test. Figure 5.43 shows the surface displacement at five radial locations during wall displacement, $\delta_w=0-0.4$ cm, for CET8. This figure illustrates the following characteristics of soil surface displacement during monotonic pullout that are common to all tests: 1) soil surface movements are small, less than $\delta_s=0.015$ cm ($\delta_s/\delta_w<4\%$) even close to the wall (S1, S2), 2) the soil surface close to the wall (S1, S2) heaves during the initial phase of loading, $\delta_w<0.1-0.15$ cm, 3) at radial distances farther from the wall (S3, S4, S5) settlements increase monotonically with pullout displacement of the caisson, and 4) the magnitude of surface settlements decreases with radial distance.

Figure 5.44 shows the soil surface displacement at five radial locations versus wall displacement during monotonic pullout for 4 tests (CET4-6,8). The data from S1, S2, and S3 show some significant scatter, but very consistent trends as discussed above for CET8. The surface heave ($\delta_s=0.001-0.002$ cm) measured by S1 is slightly larger than that at S2 (0.001 cm), but both transducers measure similar settlements as the caisson reaches full capacity at $\delta_w=0.2-0.3$ cm ($\delta_s=-0.008$ to -0.002 cm for S1 and S2). The settlement rates for S1 and S2 are also very similar: $\Delta\delta_s/\Delta\delta_w=-0.006$ to -0.007 cm/cm for both S1 and S2. The records for S3, S4, and S5 show settlements increasing approximately linearly with wall displacements at rates of $\Delta\delta_s/\Delta\delta_w=0.003$, 0.0018, 0.001 cm/cm, respectively.

**Fast Rate Monotonic Pullout**

The soil surface displacement during both slow and fast rate withdrawal in tests CET4,8, as measured by S1-S5, is shown in Figure 5.45. The surface settlement patterns do not reveal any particular rate effect. Instead, the dramatic settlement near the caisson wall ($\approx 0.2$ cm at $\delta_w=2$ cm for S1 and S2) indicates that the soil mass at these near-wall locations is clearly participating in the general failure as the caisson moves at the faster
rate, $v_w=0.3$ cm/min. In contrast, the soil surface at distant locations (S4,S5) ceases to settle significantly as the caisson is withdrawn at the fast rate beyond $\delta_w=1$ cm, thereby indicating that the soil mass at these locations is not part of the general failure mechanism.

5.6 SUSTAINED LOAD

This section presents the results of the sustained load phase for six tests that can be divided into two main groups: 1) single sustained load level (CET9,10) and 2) step sequence of sustained loads (CET11-14). In the single sustained load tests, the caisson was pulled at a constant displacement rate to a specified tensile force ($F_{tot}$), which was then held constant until a either the pore pressures equilibrated or the soil system developed a failure mechanism. In CET9 the caisson was allowed to equilibrate at $F_{tot}=-2.2$ kg over a period of approximately 10 hours, after which the caisson was pulled immediately to failure at $v_w=0.03$ cm/min. In CET10 a failure mechanism developed at $F_{tot}=-11.4$ kg.

In the step sequence of sustained load tests, the tensile loads were maintained for 24 hours before applying an additional increment of tensile load (typically $\Delta F_{tot}=-2$ kg). This process was repeated until a failure mechanism developed during sustained loading. In three of these tests (CET12-14), further caisson testing events were carried out after failure under sustained loading (see sections 5.7,5.8). The specific loading schedule for individual tests are listed in Table 5.2. The total force and displacement timelines are illustrated in Figure 5.2.

The caisson and soil response during sustained loading phases are reported as a function of time after load application (on a log scale)\textsuperscript{11}. The time frame for load increment application is very short.

\textsuperscript{11}In each case the log time axes start at $t=0.1$ minutes (6 seconds).
5.6.1 Caisson Force Distribution

Figure 5.46 shows the total, wall, and cap forces versus (the log of) time for tests CET9 and 10, with single sustained tensile load stages \( F_{\text{tot}} = -2.2 \text{ kg} \) and \(-11.4 \text{ kg}\), respectively. In the first minute, the wall and cap adjust to the applied load, after which there is a progressive transfer of load to the wall of the caisson in both tests. In test CET9 the wall initially carries a small compressive force of \( F_w = 1 \text{ kg} \) and acquires a tensile load of \( F_w = -3.6 \text{ kg} \) within 8 hours. The cap, meanwhile, sheds its initial tensile load of \( F_c = -3 \text{ kg} \) and carries a small compressive load, \( F_c = 1.4 \text{ kg} \), after 8 hours. During the final 5 hours of sustained load, the caisson load redistributes slightly, as the cap loses its compressive load and the wall sheds 1 kg of tension. In test CET10, 73% of the applied total tensile load (\( F_{\text{tot}} = -11.4 \text{ kg} \)) is initially (at 1 minute) carried by the cap (\( F_c = -8.3 \text{ kg} \) while \( F_w = -3.1 \text{ kg} \)). The cap force decreases by 5 kg within 40 minutes, but the cap is unable to shed the remaining tensile load \( F_c = -3.5 \pm 0.5 \text{ kg} \) (30% of the total tension). This behavior indicates a failure mechanism, which also is indicated by the displacement and pore pressure responses (Figures 5.51 and 5.56). The wall increases its share of tensile load from \( F_w = -3.1 \text{ kg} \) to -7.9 kg during the first 40 minutes of sustained loading and maintains nearly -8 kg of tension for the rest of the test.

In tests CET11-14, at least 3 increments of sustained load were required before the caisson began to fail. Figures 5.47-5.50 show the total, wall, and cap force versus log of time for the sustained load increments in these tests. Each individual figure shows all increments of sustained load for one particular test. During the first tensile load increment (SL1, Figures 5.47-5.50) in each of these multiple step tests, the cap and wall force redistribution pattern is similar to that described above for CET9. The cap initially carries all of the tensile load, but then sheds most of this tension within 1 hour. Meanwhile, the wall initially carries little or no tension, but gradually absorbs all the tensile load that the cap relinquishes. For subsequent load increments (SL2-6), the wall immediately carries most of the additional tensile load with almost no change in the cap force. For load
increments that cause failure, small tensile cap forces can develop (e.g., SL4 in CET11, Figure 5.47) as pore pressures develop within the soil plug due to displacement of the caisson. Consider test CET13 as depicted in Figure 5.49. During the first increment of sustained total load (SL1), $F_{\text{tot}}=6.9 \text{ kg}$, the cap initially carries all of the tension, but then drops all tensile load within 20 minutes and accepts a slightly compressive load of $F_{c}=1 \text{ kg}$ for the remainder of the increment. For each of the five subsequent total tensile load increments, the cap load remains compressive between $F_{c}=1$ and $2 \text{ kg}$ as the wall accepts increasingly higher tensile load. After approximately 3.5 hours (210 minutes) of the sixth and final sustained load increment (SL6; $F_{\text{tot}}=12.9 \text{ kg}$), tensile cap forces begin to develop as the caisson pulls out of the soil (compare Figures 5.49 and 5.54). Similar caisson component load behavior is found in the remaining sustained load tests (CET11-12, 14). The only significant difference among these tests is the total tensile load level at which the caisson begins to fail.

5.6.2 Caisson Displacement

Figures 5.51-5.55 plot the wall and cap displacement versus time (log scale) for each level of sustained tensile load$^{12}$. Figure 5.51 compares the displacement response for tests CET9 and 10. In CET9, the caisson reaches a stable displacement, $\Delta \delta_w=0.012 \text{ cm}$ in approximately 100 minutes. In contrast, the displacement rate in test CET10 increases continuously with time, reaching $\delta_w=0.2 \text{ cm}$ in 80 minutes, and thus indicating a well-defined failure in sustained loading. During the first load step in tests CET11-14, the displacement pattern is similar to that in CET9, reaching a stable displacement $\delta_w<0.02 \text{ cm}$ for CET11 ($F_{\text{tot}}=2.9 \text{ kg}$) and $\delta_w=0.02 \text{ to } 0.03 \text{ cm}$ for CET12-14 ($F_{\text{tot}}=6.9 \text{ kg}$) within approximately 200 minutes. Thereafter, the intermediate (pre-failure) load increments cause smaller caisson displacements ($\delta_w<0.02 \text{ cm}$). In all of these steps, the displacement rate decreases with log time for the period up to $t=1000 \text{ minutes}$.

$^{12}$The cap and wall displacement are nearly identical as the caisson is controlled to move as a single unit.
Caisson failure is identified by a caisson displacement rate that increases with increasing time. Failure does not occur at the same level of tensile load nor does it occur at the same time following the initiation of a particular load increment. The lowest level of tensile load at which failure occurs is $F_{tot} = 8.9$ kg in CET11 (Figure 5.52), wherein the caisson withdrew 0.1 cm within 3.3 hours (200 minutes). In CET12 the caisson failed at $F_{tot} = 9.9$ kg, reaching a displacement $\delta_w = 0.1$ cm within 25 hours (see Figure 5.53). Higher tensile loads were required to fail the caisson in CET13 and 14. At a load of $F_{tot} = 12.9$ kg in CET13, the caisson pulled out at an increasing rate with the log of time and reached 0.1 cm by 6.7 hours (400 minutes). In CET14 a load of $F_{tot} = 10.9$ kg was required to fail the caisson, which lifted 0.1 cm within 11.7 hours (700 minutes).

5.6.3 Excess Pore Pressure

Measurement of pore pressure in the soil both within and outside of the caisson indicates four pore pressure characteristics during sustained loading: 1) negative pore pressure generated during the brief monotonic pullout stage dissipates within 100 minutes of the initial sustained loading phase unless the caisson fails during the first stage, 2) subsequent sustained load stages do not generate significant excess pore pressure in the soil plug within the caisson unless the caisson cap carries a tensile load, 3) intermediate sustained load stages do not generate significant excess pore pressures in the soil exterior to the caisson, and 4) for sustained loading to failure, increasing tensile force on the cap generates increasing negative excess pore pressure in the soil plug. The excess pore pressure record for tests CET9 through CET14 are shown in Figures 5.56 through 5.60. Each figure plots the excess pore pressure beneath the cap or within the soil mass at a radius from centerline of $r = 0, 1.8, 3.2, \text{ or } 4.45$ cm. The depth of the probes that measure the pressure at the radial locations ranges from 2 to 3 cm above the caisson wall tip (see Table 5.1).
The dissipation of negative excess pore pressure generated during initial pullout is illustrated best by Figure 5.56, which shows the pore pressures beneath the cap and those measured by probes P1, P2, and P3 for CET9 and 10. In CET9, the initial negative excess pore pressure \( \Delta u = -0.2 \text{ ksc} \) fully dissipated within 40 minutes, with small positive pore pressures (\( \Delta u = 0.02 \) to \( 0.04 \text{ ksc} \)) remaining up to \( t = 800 \text{ minutes} \). Within the soil plug, P1 and P2 measured similar initial values, \( \Delta u = -0.15 \) to \(-0.2 \text{ ksc} \), which also dissipates to zero values over 40 minutes. In CET10, the higher tensile load causes much larger initial pore pressure inside the caisson (\( \Delta u = -0.52 \) to \(-0.57 \text{ ksc} \)), these dissipate to \( \Delta u = -0.2 \) to \(-0.24 \text{ ksc} \) in 100 minutes. Thereafter, the plug pore pressures remain constant, as the mechanisms of dissipation and generation (due to caisson displacement) become balanced and the caisson fails.

Dissipation of negative excess pore pressures during the initial step of the staged loading tests (CET11-14) are very similar to results described for CET9 (see Figures 5.57-5.60). During the intermediate load stages, no significant pore pressure is generated within the caisson soil plug unless the cap acquires tensile load. The pore pressure record for CET13, as shown in Figure 5.59, illustrates this behavior. Pore pressure data recorded beneath the cap and by probe P2 (\( r = 1.8 \text{ cm} \)) indicate that sustained load stages SL2 (\( F_{\text{tot}} = -8.9 \text{ kg} \)) through SL6 (\( F_{\text{tot}} = -12.9 \text{ kg} \)) generally do not generate substantial pore pressure within the soil plug. During stage SL2 a small amount of negative excess pressure, \( \Delta u = -0.13 \text{ ksc} \), is initially generated as the tensile load increases from \( F_{\text{tot}} = -6.9 \) to \(-8.9 \text{ kg} \). However, this pressure dissipates within several minutes as the cap tension is redistributed to the wall. During the final stage (SL6, \( F_{\text{tot}} = -12.9 \text{ kg} \)), negative excess pore pressure begins building as the caisson fails and tensile loads are carried by the cap. The general lack of pore pressure generation during intermediate stages of sustained loading is also apparent in CET11,12, and 14 (see Figures 5.57-5.58,5.60).

Probes P3 and P4 measured pore pressure outside the caisson. Presentable data at these locations were available in tests CET9, 12-14, as shown in Figures 5.56, 5.58-5.60.
It is clear upon examination of the data that monotonic pullout and all subsequent stages of sustained load do not generate significant excess pore pressure within the soil mass exterior to the caisson.

The fourth and final characteristic common to all tests that failed during sustained loading is the generation of negative excess pore pressure in the soil plug as the caisson fails. Figures 5.57-5.59 show this telltale sign of failure for tests CET11-13, respectively. Note that the amount of generated negative excess pore pressure is small ($\Delta u = -0.04$ to $-0.06$ ksc), and the generation occurs during the last several hours of the failure stage as the cap acquires tensile load due to the increasingly rapid caisson rise. As shown in Figure 5.60, this pore pressure generation in CET14 is almost undetectable perhaps because the caisson was stopped before the caisson accelerated as rapidly as the caisson in the other tests.

5.6.4 Soil Surface Displacement

With the exception of the sustained load stage during which caisson failure occurs, the characteristic movement of the soil surface during sustained load appears to be very subtle. However, four surface movement trends can be identified. Figures 5.61 through 5.65 illustrate the clay surface displacement during all sustained load stages for tests CET9 through 14. Each figure plots the surface displacement versus the log of time for one or more radial locations for all the sustained load phases of a particular test. Although measurements were made at five radial locations from the wall in all tests, poor instrument performance prevented the inclusion of some data in two tests, CET11 and 12 (see Table 5.3).

The first trend common to most tests occurs during the initial tensile load increment. Due to the caisson displacement, the soil surface near the caisson wall tends to heave slightly upon load application, and later settles as the tensile load transfers to the wall of the caisson. This small heave was measured only at the three radial locations
nearest the wall (r=4.2, 5.2, and 7.8 cm), and the amount of heave varied from test to test. Figure 5.61 shows that in CET9, the soil at r=5.2 cm (S2) heaves nearly $\delta_s=0.002$ cm within 1 hour of the start of sustained load before compressing to almost $\delta_s=-0.003$ cm by the end of the 18 hour period. In addition, an almost negligible amount of heave (< 0.001 cm) is measured at r=4.2 and 7.8 cm. At r=4.2 cm in CET10, the soil heaves $\delta_s=0.002$ cm, but then rapidly compresses as the caisson fails under the large initial tensile load of $F_{\text{tot}}=-11.4$ kg. Similar amounts of soil heave were detected in tests CET12, 13, and 14, as shown in Figures 5.63-5.65, respectively.

For intermediate load stages, there is a net surface settlement at all points, with slightly larger settlements occurring further away from the caisson wall. Figure 5.65 clearly shows this trend for CET14. At the end of the first load stage, (SL1, $F_{\text{tot}}=-6.9$ kg), the soil surface at S1 (r=4.2 cm) settles $\delta_s=-0.015$ cm, which rises to $\delta_w=-0.048$ cm for S5 (r=12.1 cm). This trend is also apparent in tests CET9, 11-13.

The third surface displacement trend appears in CET12 through 14. For intermediate sustained load stages, the surface settlement at one particular radial distance decreases with each successive sustained load increment. Consider the settlement at S3 (r=7.8 cm) in CET14, as shown in Figure 5.65. The compression after 16.7 hours (1000 minutes) of the second load increment (SL2, $F_{\text{tot}}=-8.9$ kg) is -0.01 cm. At the same elapsed time for the third increment (SL3, $F_{\text{tot}}=-9.9$ kg), the settlement is only $\delta_s=-0.005$ cm, and for the fourth increment (SL4, $F_{\text{tot}}=-10.9$ kg), $\delta_s<-0.004$ cm. This trend appears at all radial locations in CET12 through 14. However, Figure 5.62 shows the opposite trend in CET11, where after 16.7 hours, SL2 ($F_{\text{tot}}=-4.9$ kg) causes a surface settlement of $\delta_s=-0.002$ cm at S3 (r=7.8 cm). At the same time during the third increment (SL3, $F_{\text{tot}}=-6.9$ kg), the settlement is $\delta_s>-0.004$ cm. Although there is no clear explanation for this trend reversal, the lower level of sustained tensile loading in CET11 could be a factor. The last trend concerns the soil surface behavior for sustained loads that cause failure. If the caisson is allowed to reach large uplift displacement values (e.g.,
CET10 and 11), then the soil surface compression increases dramatically, especially at points near the caisson. The data also show that the surface compression decreases with increasing radial distance from the wall as soil nearer to the wall is more readily drawn downward and into the base of the caisson during uplift. This trend is shown clearly in Figure 5.62, which depicts surface movements at five radial locations during sustained loading for CET11. The sustained load stage ended after 30.8 hours, at which time the soil surface had settled $\delta_s = -0.14$ cm at S1, decreasing to $\delta_s = -0.007$ cm at S5.

5.7 RE-EQUILIBRATION

This section describes measurements of re-equilibration behavior for five tests (CET 5-6, 12-14), which were previously loaded to failure (i.e., to the point of maximum tensile resistance). The caisson was re-equilibrated in the clay for at least 24 hours. In 2 tests, CET5-6, the caisson capacity was mobilized by monotonic tensile loading (approximately undrained) with $\delta_w = 0.3$ cm (see Figure 5.36). In tests CET12-14, the caisson was failed under sustained tensile loading prior to re-equilibration.

During the re-equilibration phase, a constant total force $F_{tol} = 15.2$ kg (i.e., no net load compared to initial conditions) is applied to the caisson with zero relative displacement between the cap and wall. In all cases the re-equilibration phase lasts for at least 24 hours. The data are presented in a format similar to section 5.4, as a function of time after the end of tensile loading (log scale). Active control of the system typically occurs within 5 seconds after the end of tensile loading\textsuperscript{13}.

\textsuperscript{13}Data were not recorded between 2.2 and 200 minutes for CET6; the missing portion is represented by a dashed line in the figures.
5.7.1 Caisson Force Distribution

Figure 5.66 shows the total, wall, and cap force components during re-equilibration. By the end of this phase, the wall carries more than 90% ($F_w = 12.5-14$ kg) of the total caisson load, while the cap force accounts for only 1-3 kg. This result is similar to observations of the original (post-installation) caisson equilibration. However, because both the cap and wall begin this phase with large tensile loads, the time frame required for cap and wall forces to redistribute is longer than that during post-installation equilibrium.

Figure 5.66 shows that up to 10 minutes is required to unload the caisson and restore the equilibrium condition, $F_{tot} = 15.2\pm0.3$ kg. This is 7 minutes longer than the time required for the total force to reach equilibrium in post-installation set-up (see section 5.3.1, Figure 5.27). More time is required for re-equilibration because the target total force is more than 15 kg greater than that at the end of tensile loading, whereas the target $F_{tot}$ in the first equilibrium is identical to that throughout installation for most tests (i.e., $F_{tot} = 15.2$ kg for CET5-6, 12).

The wall force initially rises to between $F_w = 12-15$ kg before dropping to a range, $F_w = 7-10$ kg (see Figure 5.66). Thereafter, wall force increases throughout re-equilibration, reaching approximately $F_w = 14$ kg (or 92% of the total load) at an elapsed time of 8.3 hours (500 minutes). After this point, intercomponent friction causes the wall force to vary $\pm2$ kg$^{14}$. The time frame for re-distribution of the wall force (400-500 minutes) is much longer than that measured for equilibration following installation by underbase suction (3 minutes), but is comparable to tests CET13 and 14, where there was a large change in the net force at the end of penetration (see Figure 5.19).

Within the first minute of re-equilibration for CET6, 12-14, the cap force rises to a compressive value that ranges from $F_c = 4$ to 7.5 kg, as shown in Figure 5.66. For the next

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$^{14}$The wall force behavior in CET6 between $t=2.2$ and 2200 minutes is interpolated, but generally indicates a pattern similar to the other four tests.
500 minutes, the cap sheds most of this load to reach an equilibrium value, \( F_c = 1 \pm 1 \) kg. Intercomponent friction causes the cap force to fluctuate \( \pm 2 \) kg for the remainder of re-equilibration. In CET5 the cap force inexplicably creeps up slowly to \( F_c = 1.5 \) kg within the first 10 minutes and drops gradually to \( F_c = 0.5 \) kg by the end of the set-up phase. The time frame for cap force re-equilibration is similar to that measured during the post-installation of tests CET13 and 14.

5.7.2 Pore Pressure Dissipation

The pore pressure measured beneath the cap and within the soil plug (i.e., probes P1 and P2) during re-equilibration reflect changes in the cap force. Much smaller changes in pore pressure occur in the soil mass outside the caisson. Figure 5.67 shows the excess pore pressure beneath the cap versus log of time for the five tests that incorporated a re-equilibration phase (CET 5-6, 12-14). The excess pore pressure at the start of re-equilibration for tests CET5-6 are in the range \( \Delta u = -0.25 \) to \(-0.45 \) ksc, which reflects the large negative pore pressure generation that occurred in the preceding monotonic tensile load test. In contrast, tests CET12-14 start out with nearly zero (\( \pm 0.1 \) ksc) excess pore pressure, as the preceding sustained tensile load stage was a fully drained process\(^{15}\). Complete dissipation of excess pore pressure occurs within 500 minutes. By the end of equilibration in all tests, \( \Delta u = \pm 0.1 \) ksc.

Figure 5.67 shows the excess pore pressure versus log of time for probes P1 and P2 within the soil plug. Most of these probes were located at a depth of approximately 2.5 cm above the wall tip, except in CET5 where P1 and P2 are at the wall tip elevation, and in CET14 where P1 is located 6 cm below the caisson. In general, the pore pressure measured by P1 and P2 follow very closely the magnitude and dissipation behavior measured beneath the cap; full dissipation occurs within 500 minutes.

\(^{15}\)In these tests, the preceding sustained load event only generated significant pore water pressure at failure.
Probes P3 and P4 were located outside the caisson at radial distances \( r = 3.2 \) and 4.45 cm, respectively. As shown in Figure 5.67, these probes measured a muted pore pressure response with initial excess pore pressure, \( \Delta u = 0.1 - 0.26 \) ksc at P3 and \( \Delta u = 0.08 \) ksc at P4. After several hours, the excess pressure has dissipated toward zero. Note that the slight positive pressure at the end of set-up in CET12 may be due to the effect of temperature and barometric variation.

5.7.3 Caisson Displacement

The preceding loading history has a major effect on the settlement measured during re-equilibration. Figure 5.68 plots the wall and cap settlement, respectively, versus the log of time in minutes. As mentioned in section 5.7.1, the large difference in total force between the end of the tensile load stage (\( F_{\text{tot}} = -2 \) to \(-22 \) kg) and the target total force for re-equilibration (\( F_{\text{tot}} = 15.2 \) kg) affected the zero relative displacement condition during the first ten minutes. The control program drove the wall ahead of the cap in order to attain the total force target, \( F_{\text{tot}} = 15.2 \) kg (see Figure 5.68). By the end of the re-equilibration phase, the caisson has settled an amount that ranges from -0.14 to -0.37 cm, which is much larger than displacements observed in post-installation equilibration (\(-0.1 \) cm, see section 5.4.4).

There is a distinct difference between the settlement curves for the caissons that re-equilibrated after monotonic tensile loading (CET5-6) and those that followed sustained loading (CET12-14). After approximately 10 minutes have elapsed in CET5-6, the caisson settles at a nearly log-linear rate that ranges from 0.06 to 0.07 cm per log cycle of time. Note that this rate is approximately double the log-linear caisson settlement rate during the post-installation phase. In CET12-14, the caisson rapidly settles an amount that is approximately equal to the upward displacements of the caisson achieved during the preceding sustained load stage. The pullout displacements measured during the previous stages of sustained loading were 0.2, 0.41, and 0.16 cm for CET12, 13, and 14,
respectively. As shown in Figure 5.68, the caisson settlement during the rapid settlement stage for these tests was 0.19, 0.35, and 0.14 cm, respectively. Beyond the rapid drop stage, the caisson settles at a log-linear rate of approximately 0.02 cm per log time cycle.

5.7.4 Soil Surface Displacement

Figure 5.69 plots the surface compression for the five re-equilibration tests (CET5-6, 12-14). The figure shows the settlement for several tests at a specific radial distance, r. Soil compression was much larger in the tests where re-equilibration followed monotonic pullout (CET5-6) than in those following sustained loading (CET12-14). For example, S1 measures settlement \( \delta_s = -0.11 \) to -0.12 cm for CET5 after 1000 minutes, but only \( \delta_s = -0.02 \) cm in tests CET13,14. This difference in soil surface compression between the monotonic pullout tests and the sustained load tests is apparent at each of the five radial locations.

As expected, the settlement magnitude decreases with increasing radial distance from the caisson. For test CET5, the compression after 1000 minutes decreases from a range of \( \delta_s = -0.11 \) to -0.12 cm at S1 (r=4.2 cm) to \( \delta_s = -0.015 \) to -0.03 cm at S5 (r=12.1 cm). For CET13 and 14, the corresponding ranges are \( \delta_s = -0.02 \) cm at S1 to \( \delta_s < -0.005 \) cm at S5.

5.8 MONOTONIC PULLOUT 2

This section presents the results of a second series of tensile monotonic pullout tests performed after re-equilibration in CET5-6, 12-14 and immediately following the final increment of sustained load in CET9. In each test the caisson loading was performed at a constant displacement rate \( v_w = -0.03 \) cm/min, allowing no relative displacement between the cap and wall. In all six tests the caisson was pulled to a displacement of at least \( \delta_w = 0.3 \) cm in order to determine the total capacity. After establishing the caisson
capacity, three tests (CET12-14) were loaded to large displacement at a faster rate of \( v_w = 0.3 \) cm/min until complete extraction. The following discussion focuses on the similarities and differences in the caisson response measured in the first (section 5.5) and second phases of monotonic loading.

5.8.1 Caisson Force Distribution

Measurements of the caisson forces during the second pullout phase indicate that re-equilibration of the caisson increased the wall resistance, but did not affect the cap resistance. Hence, the caisson capacity is greater during the second pullout phase than during the initial pullout. For test CET9, where the caisson did not re-equilibrate, the wall resistance and caisson capacity during the second pullout was slightly lower than the average values during initial pullout. For all five re-equilibrated tests, the initial stiffness of the caisson and wall force are higher in the second monotonic test than in the first. The following discussion compares the total, wall, and cap forces during the second monotonic pullout series (MP2) with the 'best estimate' of caisson response from the first test series (MP1, section 5.5.1).

**Total Force**

Figure 5.70 plot the total, wall, and cap forces versus the wall tip displacement for 6 tests (CET5-6,9,12-14) at a displacement scale from \( \delta_w = 0.0 \) to 0.4 cm, while the small displacement response (\( \delta_w = 0.00 \) to 0.02 cm) is shown in Figure 5.71. The results of the MP2 tests can be subdivided into 3 groups: 1) CET5 and 6, which were re-equilibrated after being loaded to failure in an (undrained) monotonic mode, 2) CET12-14, which were re-equilibrated after failure under long-term sustained tensile loads, and 3) CET9, which corresponds to reloading with no re-equilibration.

In the MP2 tests, CET5,6 mobilize a maximum capacity \( F_{tot} = -25 \) to -27 kg at a displacement \( \delta_w = 0.25 \) cm, and show no well-defined yield point in the load-deformation
response. In the second group of tests, CET12-14, the series MP2 data show maximum caisson resistance, \( F_{\text{tot}} = -25 \) to \(-29 \) kg at \( \delta_w = 0.1 - 0.15 \text{ cm} \), and a well-defined yield point at \( \delta_w = 0.02 \text{ cm} \). They have a much higher pre-yield stiffness than CET5,6. Comprising the last test group, CET9 reaches a maximum caisson resistance \( F_{\text{tot}} = -21 \) kg at \( \delta_w = 0.1 \text{ cm} \), with a very stiff initial response and yield at \( \delta_w = 0.02 \text{ cm} \).

Comparison of these results with previous monotonic test data (Figure 5.36-5.37) reveals that all of the re-equilibrated caissons have tensile load capacities 20-25% higher than in MP1 tests (\( F_{\text{tot}} = -22 \) to \(-24 \) kg). The initial stiffness and yield displacement in CET12-14 are comparable to MP1 behavior, but the CET5 and 6 data have lower stiffness compared to previous performance. Test CET9 has a lower stiffness and capacity than that measured in MP1 tests.

**Wall Force**

Figure 5.70 shows the wall force behavior for tests CET 5-6,9,12-14. It is more difficult to discern major trends among these data than were seen in the total force plots. However, the wall force in CET9 is notably lower than those from all 5 tests where re-equilibrium occurred. The initial wall response for the re-equilibrated tests is very similar to the response observed in the MP1 pullout. At the point of peak total load during MP2, the wall force from the five re-equilibrated tests averages \( F_w = -17.3 \pm 1.6 \text{ kg} \), which corresponds to 63% of the average peak tensile load, \( F_{\text{tot}} = -27.3 \text{ kg} \). This result confirms a small but pervasive increase in wall capacity achieved in the MP2 tests.

The wall resistance in test CET9, \( F_w = -11.3 \text{ kg} \) at \( \delta_w = 0.18 \text{ cm} \) (maximum caisson resistance), corresponds to 65% of the average wall force for the re-equilibrated tests.

**Cap Force**

In contrast to the wall in the re-equilibrated tests, the cap does *not* mobilize more capacity during the second pullout. The cap force versus wall tip displacement is plotted
in Figures 5.70 and 5.71. The initial cap stiffness is very similar to behavior measured in the MP1 tests. At maximum caisson resistance, the cap force for all five re-equilibrated MP2 tests averages $F_c=-10.0\pm2.1$ kg, which compares very closely with the average value $F_c=10.4$ kg, quoted from the MP1 tests. The resulting cap force contribution corresponds to 37% of the MP2 pullout capacity.

The cap response in CET9 is very similar to the other MP2 tests. At maximum capacity, the cap force is $F_c=-9.60$ kg, which is within the range exhibited by the re-equilibration tests. However, due to the low wall force of $F_w=-11.3$ kg, the cap force contribution to the total capacity is 46%, which is similar to the MP1 behavior.

*Fast Rate Monotonic Pullout 2*

After the caisson was pulled to capacity at $v_w=-0.03$ cm/min, tests CET12-14 continued loading up to complete extraction at a faster rate, $v_w=-0.3$ cm/min. The general caisson force behavior is very similar to MP1 tests with a similar load sequence (section 5.5.1). Figure 5.72 shows the total, wall, and cap forces versus wall tip displacement for both rates of pullout. The caisson relaxes during the interval between the two loading phases as the control program is stopped and restarted manually. As shown in Figure 5.72, the total force relaxation ranges from $\Delta F_{tot}=3$ to 7 kg. After incremental displacements of $\Delta \delta_w=0.18$ to 0.22 cm at the fast displacement rate, the caisson has mobilized a tensile load that is $\Delta F_{tot}=-2$ to -3 kg higher the total tensile load at the end of slow pullout. This tensile load increase between slow and fast pullout is similar in magnitude to that during MP1 tests. After reaching peak load, the total tensile load declines at an approximate rate of $F_{tot}/\delta_w=5$ kg/cm, which also is similar to the rate of decline in MP1.

The wall and cap force records (Figure 5.72) indicate that the increased capacity during fast rate pullout can be attributed to an increased cap contribution. The wall force in both CET12 and 14 does not increase significantly at the start of the faster pullout, but
instead continues following the trend of declining tensile load established during the slow pullout. In test CET13, the wall force initially does jump $\Delta F_w=-3$ kg above the tensile wall load at the end of slow pullout, but sheds load at a slightly quicker rate than the other two tests for the remainder of pullout. In Figure 5.72 all three tests show an increase in cap tensile load due to increased displacement rate. The cap tensile load increase at the start of fast pullout ranges from $\Delta F_c=-2$ to -6 kg above the tensile load at the end of slow pullout. However, relatively large incremental displacements ($\Delta \delta_w=0.4$ to 1.1 cm) are necessary to mobilize this increased cap tensile load.

5.8.2 Pore Pressure Generation

Figure 5.73 plots the excess pore pressure beneath the cap versus wall tip displacement for the MP2 monotonic pullout tests. Due to the stiff cap response, large negative excess pore pressures are generated at very small displacements. At $\delta_w=0.003$ cm, the excess pressure ranges from $\Delta u=-0.14$ to -0.5 ksc, a range that is similar to behavior measured in MP1 tests. For the three sustained load tests CET12-14, the peak negative excess pore pressure ($\Delta u=-0.5$ to -0.78 ksc) is mobilized rapidly at $\delta_w<0.04$ cm. In CET5-6 and CET9, much larger caisson displacements are necessary to mobilize the peak negative excess pore pressure ($\delta_w<0.3$-0.4 cm in CET5,6).

Figure 5.74 shows the excess pore pressure versus wall displacement for probe P1 in 5 tests (CET5,6,9,12,14). In CET 5, 6, 9, and 12, the record shows that the magnitude of peak negative pore pressure is similar to that for the pressure beneath the cap. Note that pullout does generate a measurable amount ($\Delta u=0.1$ ksc) of negative pore pressure near the bottom of the clay, as measured by P1 in CET14.

Although only two records of pore pressure are available for probe P2 (located within the caisson walls at $r=1.8$ cm), the pattern generated is similar to that revealed in pullout MP1. The data for CET9 and 14 (Figure 5.74) show larger negative excess pore pressures than those measured below the base or at P1. In CET14, at $\delta_w=0.022$ cm, the
excess pressure reaches a peak, $\Delta u=-0.9$ ksc, which is -0.12 ksc greater than the peak reached beneath the cap ($\Delta u=-0.78$ ksc). For CET9, the caisson must displace $\delta_w=0.15$ cm before reaching a peak pressure of $\Delta u=-0.68$ ksc, which is -0.17 ksc greater than the peak attained below the cap ($\Delta u=-0.51$ ksc).

Very small excess pore pressures are generated during the MP2 tests in the soil outside the caisson walls. Figure 5.74 shows the limited data available for probes P3 and P4. By the end ($\delta_w=0.4$ cm), both probes measure approximately $\Delta u=-0.1$ ksc of excess pore pressure.

*Fast Rate Monotonic Pullout 2*

Enough data was collected to show that there is a significant load rate effect on pore pressure development within the soil plug. Figure 5.75 shows the excess pore pressure measured beneath the cap for both pullout rates in tests CET12-14. During the time interval between the end of slow pullout and the start of fast pullout, the excess pressure reduces due to relaxation of the applied force. Within $\Delta \delta_w=0.5$ cm of restarting pullout, the negative excess pore pressure jumps -0.2 to -0.25 ksc below the value at the end of slow pullout. Thereafter, the excess pressure reduces at a constant rate with wall displacement. Similar pore pressure behavior is measured by the P1 probes (Figure 5.76); by the time the caisson has withdrawn $\delta_w=1$ cm, the large negative excess pore pressure generated during the initial 0.1 cm of pullout has dissipated to approximately $\Delta u=-0.37$ ksc. Fast pullout induces the excess pore pressure to jump back to nearly $\Delta u=-0.5$ ksc before it begins dissipating at a constant rate. Note in this figure that fast pullout has little effect on the pore pressure at the bottom of the clay, as measured by P1 in CET14. The excess pore pressure at this location never rises more than $\Delta u=-0.14$ ksc during the first 1 cm of fast pullout before dropping back to negligible values. At the soil plug location $r_w=0.6$ cm from the inside caisson wall, probe P2 measures a pore pressure pattern consistent with the other soil plug patterns measured beneath the cap and by P1. Figure
5.76 shows the excess pore pressure measured by P2 in CET14 for both pullout rates. After only 0.1 cm of fast withdrawal, the negative excess pressure rises to $\Delta u = -86$ ksc, which is 0.26 ksc higher than the value at the end of slow pullout.

Figure 5.76 plots the excess pore pressure measured outside the caisson in CET14 by probes P3 and P4. At the farther location $r_w = 1.91$ cm from the exterior caisson wall, probe P4 measures no rate effect on pore pressure, as the slight amount of negative excess pressure generated during slow pullout continues dissipating to negligible values. Closer to the wall, probe P3 ($r = 3.2$ cm) reveals very interesting pore pressure behavior, as shown in Figure 5.76. At the onset of fast pullout, positive excess pore pressure is generated, so that by a caisson displacement of $\delta_w = 1.77$ cm, the excess pore pressure is $\Delta u = 0.16$ ksc. Then, as the caisson wall passes the depth at which probe P3 is located (approximately 2 cm above the wall tip prior to slow pullout), large negative excess pore pressure is generated and reaches a peak of nearly $\Delta u = -0.6$ cm at a caisson displacement of $\delta_w = 3$ cm. With further displacement the negative excess pore pressure dissipates until reaching zero by the end of extraction.

5.8.3 Soil Surface Displacement

The soil surface displacement patterns measured during the MP2 pullout tests are nearly identical to those for MP1 (compare Figures 5.77 and 5.44). Figure 5.77 illustrates these four trends for MP2 tests: 1) surface movements are small at $\delta_s < 0.015$ cm in most cases, 2) soil surface close to the wall (S1, S2) heaves during the initial loading phase ($\delta_w < 0.2$ cm), 3) at radial distances farther from the wall (S3, S4, S5), settlement increases monotonically with pullout displacement of the caisson, and 4) the magnitude of surface settlement decreases with radial distance.
Fast Rate Monotonic Pullout #2

As for the slow rate surface displacement, the fast pullout rate surface displacement characteristics during MP2 are very similar to those for pullout MP1, which were discussed in section 5.5.3. Namely, the surface settlement pattern does not reveal a particular rate effect, but instead, large settlement near the wall ($\delta_s \approx 0.2$ cm for S1, S2) indicates that the soil mass at these locations is participating in the general failure as the caisson moves at a faster rate. As for MP1, the soil surface at more distant locations (S3, S4, and S5) does not settle significantly at the faster rate of pullout, and, therefore, is not involved in the general failure of the soil mass.
Common Geometry
Clay Cake Diameter = 30.5 cm
Clay Cake Height = 12.1-14.3 cm
Caisson Outside Diameter = 5.08 cm
Caisson Wall Thickness = 0.145 cm
Caisson Penetration = 5.08 cm

Common Instrumentation
Caisson Wall Force, L1
Caisson Cap Force, L2
Caisson Wall Displacement, D1
Caisson Cap Displacement, D2
Chamber Air Pressure, AP
Caisson Cap Pore Pressure, CP

Clay Height and Other Instrumentation

<table>
<thead>
<tr>
<th>Test</th>
<th>Clay Height (H_c) (cm)</th>
<th>Pore Pressure Probes</th>
<th>Surface Displacement LVDTs</th>
<th>Total Stress (\sigma_H)</th>
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<tbody>
<tr>
<td>CET1</td>
<td>12.4</td>
<td>d1 cm</td>
<td>d2 cm</td>
<td>P1</td>
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| CET2  | 12.8                    | 5.3   | 7.1   | X  | X  | X  | -  | -  | X  | X  | X  | -
| CET3  | 12.1                    | 5.4   | 7.4   | X  | X  | X  | -  | -  | X  | X  | X  | -
| CET4  | 14.3                    | 4.9   | 7.2   | X  | X  | X  | -  | -  | X  | X  | X  | -
| CET5  | 13.5                    | 5.8   | 8.5   | X  | X  | -  | X  | X  | X  | -
| CET6  | 13.9                    | 4.0   | 9.5   | X  | X  | X  | -  | X  | X  | X  | X  |
| CET7  | 13.9                    | 2.2   | 11.7  | X  | -  | -  | -  | X  | X  | X  | X  |
| CET8  | 12.9                    | 2.5   | 10.4  | X  | X  | X  | -  | X  | X  | X  | X  |
| CET9  | 13.3                    | 2.4   | 10.9  | X  | X  | X  | -  | X  | X  | X  | X  |
| CET10 | 13.4                    | 2.5   | 10.8  | X  | X  | X  | -  | X  | X  | X  | X  |
| CET11 | 12.9                    | 2.3   | 11.2  | X  | X  | X  | -  | X  | X  | X  | X  |
| CET12 | 13.7                    | 2.0   | 11.2  | X  | X  | X  | -  | X  | X  | X  | X  |
| CET13 | 12.7                    | 2.5   | 10.2  | X  | X  | X  | -  | X  | X  | X  | X  |
| CET14 | 14.0                    | 2.9   | 11.1  | X  | X  | X  | X  | X  | X  | X  | X  |

Notes: \(H_c\) = clay height prior to driving
\(\sigma_H\) = total stress transducer along chamber sidewall in CET1,2,6
\(d1\) = depth from clay surface prior to driving
\(d2\) = height from clay bottom prior to driving
\(X\) = transducer in use for test
\(X^*\) = these probes located 2 cm above clay bottom

Table 5.1 Individual Test Geometry and Instrumentation
<table>
<thead>
<tr>
<th>Test</th>
<th>Consol. at 0.75ksc</th>
<th>Suction Driving</th>
<th>Equil. 1 15.2 kg</th>
<th>Monotonic Pullout 1</th>
<th>Sustained Loading</th>
<th>Equil. 2 15.2 kg</th>
<th>Monotonic Pullout 2</th>
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*σ, = 1.0 ksc
†CET2 aborted during driving (see section 5.1.4)

Table 5.2 Individual Test Loading Phases
<table>
<thead>
<tr>
<th>Test</th>
<th>Consol. at 0.75 ksec</th>
<th>Suction Driving</th>
<th>Equil. 1 15.2 kg</th>
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<td>-6.9 kg, 25 2 hr -8.9 kg, 120.5 hr -9.9 kg, 27 hr</td>
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<td>-6.9 kg, 26.3 hr -8.9 kg, 24.5 hr -9.9 kg, 24.1 hr -10.9 kg, 25 hr -11.9 kg, 24.1 hr -12.9 kg, 14.1 hr</td>
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<td>153.8 hr</td>
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* CET13 drove with constant cap force
** CET14 drove with zero cap displacement

Table 5.2 Individual Test Loading Phases
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**KEY**
1 = good
2 = fair
3 = poor
4 = unusable

**Table 5.3** Quality Assessment for Individual Test Control and Instrumentation
a) Test Control, Installation Phase Instrumentation
### Table 5.3 Quality Assessment for Individual Test Control and Instrumentation

#### b) Equilibration and Monotonic Pullout 1 Phase Instrumentation

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| **Instrumentation for Monotonic Pullout 1** |   |   |   |   |   |   |   |   |   |    |    |    |    |    |
| Cap Pore Pressure | 4 | - | 4 | 3 | 1 | 1 | 1 | 1 | 4 | 1 | 1 | 1 | 1 | 1 |
| Pore Pressure Probe 1 | 4 | - | 4 | 4 | 2 | 1 | 2 | 4 | 1 | 1 | 1 | 1 | 4 | 1 |
| Pore Pressure Probe 2 | 4 | - | 4 | 3 | 4 | - | 1 | 1 | 1 | 2 | 4 | 4 | 3 | 1 |
| Pore Pressure Probe 3 | 4 | - | 4 | - | 4 | - | 1 | 1 | 1 | 3 | 3 | 2 | 4 | 1 |
| Pore Pressure Probe 4 | - | - | - | 4 | - | - | - | - | - | - | - | - | - | 1 |
| Horizontal Total Stress | 2 | - | - | - | - | 1 | - | - | - | - | - | - | - | - |
| Surface Displacement 1 | - | - | 1 | 1 | 4 | 1 | 1 | 1 | 1 | 4 | 1 | 1 | - | - |
| Surface Displacement 2 | 1 | - | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 |
| Surface Displacement 3 | 4 | - | 1 | 1 | 2 | 1 | 2 | 1 | 1 | 1 | 1 | 1 | 1 | 1 |
| Surface Displacement 4 | 4 | - | 1 | 4 | 4 | 1 | 4 | 1 | 1 | 4 | 1 | 1 | 1 | 1 |
| Surface Displacement 5 | - | - | 1 | - | 1 | - | 2 | 1 | 3 | 1 | 1 | 4 | 1 | 2 |

**KEY**
- 1 = good
- 2 = fair
- 3 = poor
- 4 = unusable

*Table adapted from the original document.*
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| **Instrumentation for Re-Equilibration** |   |   |   |   |   |   |   |   |   |   |   |   |   |   |
| Cap Pore Pressure | - | - | - | 1 | 1 | 1 | - | - | - | - | 1 | 1 | 1 | 1 |
| Pore Pressure Probe 1 | - | - | - | 2 | 1 | 2 | - | - | - | - | 1 | 4 | 1 | 1 |
| Pore Pressure Probe 2 | - | - | - | 4 | 1 | - | - | - | - | 4 | 3 | 1 | 1 | 3 |
| Pore Pressure Probe 3 | - | - | - | 4 | - | 1 | - | - | - | - | 4 | 4 | 1 | 1 |
| Pore Pressure Probe 4 | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| Horizontal Total Stress | - | - | - | 1 | - | - | - | - | - | - | - | - | - | - |
| Surface Displacement 1 | - | - | - | 1 | 4 | 1 | - | - | - | - | 1 | 1 | 1 | 1 |
| Surface Displacement 2 | - | - | - | 1 | 1 | 1 | - | - | - | - | 1 | 1 | 1 | 1 |
| Surface Displacement 3 | - | - | - | 1 | 1 | 1 | - | - | - | - | 1 | 1 | 1 | 1 |
| Surface Displacement 4 | - | - | - | 4 | 1 | 1 | - | - | - | - | 1 | 1 | 1 | 1 |
| Surface Displacement 5 | - | - | - | 1 | 1 | 1 | - | - | - | - | 1 | 1 | 1 | 1 |

*unusable in sustained load stage 1

**KEY**
1 = good
2 = fair
3 = poor
4 = unusable

**Table 5.3** Quality Assessment for Individual Test Control and Instrumentation

c) Sustained Load and Re-Equilibration Phase Instrumentation
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KEY
1 = good
2 = fair
3 = poor
4 = unusable

Table 5.3 Quality Assessment for Individual Test Control and Instrumentation
d) Monotonic Pullout 2 Phase Instrumentation
Rating Key
1 = good
2 = fair
3 = poor
4 = unusable

Test Control Guideline

Installation:  
1 - $F_{tot} = \pm 1$kg
2 - $F_{tot} = \pm 3$kg
3 - $F_{tot} > 3$kg
4 - $F_{tot}$ = no control

Equilibration/Sustained Loading:  
1 - $\text{Relcapd} = \pm 0.001$cm
2 - $\text{Relcapd} = \pm 0.01$cm
3 - $\text{Relcapd} > 0.01$cm
4 - $\text{Relcapd}$ = no control

Monotonic Pullout:  
1 - $\text{Relcapd} = \pm 0.003$cm
2 - $\text{Relcapd} = \pm 0.01$cm
3 - $\text{Relcapd} > 0.01$cm
4 - $\text{Relcapd}$ = no control

Instrumentation Data Guideline

Pore Pressure Probe:  
1 - good response
2 - fair response
3 - poor response
4 - no response

Surface Displacement LVDT:  
1 - in linear calibrated range
2,3 - in range, but unstable
4 - stuck or out of range

$F_{tot}$ = total force on caisson
**Relcapd = relative displacement between cap and wall

Table 5.4  Guidelines for Quality Assessment Code
### Initial Penetration Zone

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<td>Cap Force (kg)</td>
<td>Total Force (kg)</td>
<td>Wall Pen. (cm)</td>
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<td>Cap Force (kg)</td>
<td>Total Force (kg)</td>
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### Transition Penetration Zone

| CET Test | Start | | | | | Peak Wall Force (Local Peak) | | | End | | | | | |
|----------|-------|------|----------|------|----------|------|------|----------|------|------|----------|------|------|----------|------|
|          | Wall Pen. (cm) | Wall Force (kg) | Cap Force (kg) | Total Force (kg) | Wall Pen. (cm) | Wall Force (kg) | Cap Force (kg) | Total Force (kg) | Wall Pen. (cm) | Wall Force (kg) | Cap Force (kg) | Total Force (kg) |
| 3        | 0.2    | 14.75 | 3.01 | 17.76 | 0.41 | 16.58 | 2.05 | 18.63 | 0.83 | 11.33 | 2.47 | 13.80 |
| 4        | 0.2    | 12.49 | 3.40 | 15.89 | 0.27 | 13.77 | 2.52 | 16.29 | 1.62 | 13.17 | 2.84 | 16.01 |
| 5        | 0.2    | 13.30 | 2.92 | 16.22 | 0.32 | 14.13 | 2.57 | 16.70 | 0.48 | 13.58 | 2.54 | 16.12 |
| 6        | 0.2    | 12.55 | 2.72 | 15.27 | -    | -    | -    | -    | 2.40 | 14.36 | 2.71 | 16.07 |
| 7        | 0.2    | 18.06 | -1.11 | 16.95 | 0.35 | 19.36 | -2.44 | 16.93 | 0.98 | 16.31 | -0.12 | 16.19 |
| 8        | 0.2    | 10.87 | 6.22 | 17.08 | 1.02 | 12.28 | 4.93 | 17.16 | 1.19 | 12.05 | 5.07 | 17.11 |
| 9        | 0.2    | 12.21 | 3.52 | 15.72 | 0.38 | 12.93 | 2.78 | 15.71 | 0.80 | 11.41 | 3.86 | 15.27 |
| 10       | 0.2    | 11.22 | 3.20 | 14.41 | -    | -    | -    | -    | 0.90 | 16.86 | -1.59 | 15.27 |
| 11       | 0.2    | 10.14 | 5.49 | 15.63 | 0.44 | 11.98 | 3.48 | 15.45 | 0.59 | 11.30 | 3.70 | 15.00 |
| 12       | 0.2    | 6.61 | 10.51 | 17.12 | 0.63 | 17.60 | -2.26 | 15.33 | 1.83 | 14.06 | 1.01 | 15.07 |
| 13       | 0.2    | 11.80 | 13.73 | 25.52 | -    | -    | -    | -    | 0.44 | 13.90 | 13.77 | 26.76 |
| 14       | 0.2    | 10.66 | 24.73 | 35.39 | -    | -    | -    | -    | 1.40 | 11.38 | -4.12 | 7.26 |

Note: No obvious local peak in CET 6, 10, 13, and 14

Table 5.5  Caisson Force Characteristics During Installation:
- Initial and Transition Zones
Deep Penetration Zone

<table>
<thead>
<tr>
<th>CET Test</th>
<th>Start Wall Pen. (cm)</th>
<th>Wall Force (kg)</th>
<th>Cap Force (kg)</th>
<th>Total Force (kg)</th>
<th>End Wall Pen. (cm)</th>
<th>Wall Force (kg)</th>
<th>Cap Force (kg)</th>
<th>Total Force (kg)</th>
<th>Slope (kg/cm)</th>
<th>Force intercept (kg)</th>
<th>R²</th>
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<tr>
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<td>18.27</td>
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<td>10.74</td>
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Table 5.5  Caisson Force Characteristics During Installation:
- b) Deep Zone
Monotonic Pullout 1: 0.03 cm/min

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<td>Cap Force (kg)</td>
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Monotonic Pullout 1: 0.3 cm/min

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Monotonic Pullout 2: 0.03 cm/min

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Monotonic Pullout 2: 0.3 cm/min

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Table 5.6  Caisson Force Characteristics During Monotonic Pullout 1 and 2
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\[ \sigma_N \] Total Stress  
(chamber sidewall)

---

![Diagram](image)

**Figure 5.1**  Schematic Cross Section of CET Chamber with Caisson Fully Penetrated to Illustrate Test Geometry and Instrumentation
Figure 5.2  Individual Test Force and Displacement Timelines
a) CET3-10
Figure 5.2 Individual Test Force and Displacement Timelines
b) CET11-14
Figure 5.3 Measured Forces on Caisson Components During Suction Driving in CET9
Figure 5.4  Measured Total Force on Caisson During Suction Driving for CET3-5, 7-12 at Large Scale (-5 to 25 kg)
Figure 5.5  Measured Total Force on Caisson During Suction Driving for CET3-5, 7-12 at Small Scale (14 to 19 kg)
Figure 5.6  Measured Wall Force Component During Suction Driving for CET3-5, 7-12
Figure 5.7  Measured Wall Force Component During Suction Driving for CET3-5, 7-12: Initial and Transition Zones
Figure 5.8  Measured Wall Force Component During Suction Driving for CET3-5, 7-12: Deep Zone
Figure 5.9  Most Representative Measured Wall Force During Initial and Deep Penetration of Suction Driving
Figure 5.10  Best Estimate of Wall Force versus Wall Tip Penetration During Suction Driving
Figure 5.11  Measured Cap Force Component During Suction Driving for CET3-5, 7-12
Figure 5.12  Measured Cap Force Component During Suction Driving for CET3-5, 7-12: Initial and Deep Zones
Figure 5.13  Measured Excess Pore Pressure Beneath Cap During Suction Driving for CET4-5,7-12
Figure 5.14  Measured Excess Pore Pressure Beneath Cap During Suction Driving for CET4-5, 7-12: Initial and Transition Zones
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Figure 5.16 Measured Excess Pore Pressure by Probes P1, P2, and P3 During Suction Driving for CET7-12
Figure 5.17  Measured Soil Surface Displacement at Three Radial Locations During Suction Driving for CET8
Figure 5.18  Measured Soil Surface Displacement by S1-S5 During Suction Driving for CET3-5,8-10
Figure 5.19  Measured Total Force on Caisson During Installation for CET6, 13, 14
Figure 5.20  Measured Wall Force Component During Installation for CET6, 13, 14
Figure 5.21  Measured Cap Force Component During Installation for CET6,13,14
Figure 5.22  Measured Excess Pore Pressure Beneath Cap During Installation for CET6,13,14
Figure 5.23  Measured Excess Pore Pressure Beneath Cap and by P1-P4 During Installation for CET6, 13, 14
Figure 5.24 Measured Cap Displacement During Installation for CET6, 13, 14

Cap Displacement (cm)

Wall Tip Penetration (cm)
Figure 5.25  Measured Soil Surface Displacement by S1-S5 During Installation for CET6, 13, 14
Figure 5.26  Measured Forces on Caisson Components During Post-Installation Equilibration in CET9
Figure 5.27  Measured Total, Wall, and Cap Force on Caisson During Post-Installation Equilibration for CET3,8-9,11-14
Figure 5.28  Measured Excess Pore Pressure Beneath Cap and by P1-P4 During Post-Installation Equilibration for CET4,8-9,11-14
Figure 5.29  Measured Caisson Settlement During Post-Installation Equilibration for CET8
Figure 5.30  Measured Wall and Cap Settlement During Post-Installation Equilibration for CET3-6,8-9,12-13
Figure 5.31  Measured Caisson Settlement During Post-Installation Equilibration for CET14
Figure 5.32  Measured Soil Surface Displacement at Five Radial Locations During Post-Installation Equilibration for CET8
Figure 5.35 Measured Soil Surface Displacement by S1-S5 During Post-Installation Equilibration for CET3-6,8-9,11-13
Figure 5.34  Measured Soil Surface Displacement at Five Radial Locations During Post-Installation Equilibration for CET14
Figure 5.36  Measured Total, Wall, and Cap Force on Caisson During Monotonic Pullout 1 for CET3-6,8
Figure 5.37  Measured Force Components During Early (0.0 to 0.02 cm) Monotonic Pullout 1 for CET6,8-14
Figure 5.38  Best Estimate of Total, Wall, and Cap Force versus Wall Displacement During Monotonic Pullout 1
Figure 5.39  Measured Total, Wall, and Cap Force on Caisson During Monotonic Pullout 1 at Two Pullout Rates for CET3-4,8
Figure 5.40  Measured Excess Pore Pressure Beneath Cap During Monotonic Pullout 1 for CET5-6,9-14
Figure 5.41  Measured Excess Pore Pressure Beneath Cap and by P1-P4 During Monotonic Pullout 1 for CET5-6,9-14
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Figure 5.44  Measured Soil Surface Displacement by S1-S5 During Monotonic Pullout 1 for CET8-11,13-14.
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Monotonic Pullout 2

\( v_w = 0.03 \) cm/min

+ CET 6

\( \cdot \) CET 8

\( \times \) CET 12

\( \triangle \) CET 13

Wall Force, \( F_w \)

Wall Displacement (cm)

Cap Force, \( F_c \)

Force Components (kN)
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- Monotonic Pullout 2
- Excess Pore Pressure Beneath Cap
- \(-0.03\) cm/min
- \(-0.3\) cm/min

Legend:
- ▲ CET12
- ♦ CET13
- X CET14
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CHAPTER 6
INTERPRETATION OF SUCTION CAISSON BEHAVIOR IN THE CET CELL

This chapter interprets the results of the model caisson testing program conducted using the Caisson Element Test (CET) cell. The measured behavior during each of the model caisson events is summarized and then analyzed using principles of soil mechanics and soil behavior. Section 6.1 discusses the resistance to caisson installation and the effect of underbase suction. Section 6.2 evaluates the post-installation equilibration phase by focusing on the caisson component (wall and cap) force redistribution and rates of pore pressure dissipation, caisson settlement, and soil surface settlement. This section also compares the post-installation equilibration with the re-equilibration phase that follows axial tensile loading. Section 6.3 analyzes the monotonic uplift capacity of suction anchors in the CET cell. In addition, this section discusses the effects of re-equilibration and pullout rate upon the uplift capacity. Lastly, section 6.4 interprets the sustained tensile load tests conducted in the CET cell.

6.1 INSTALLATION

Prototype suction caissons that are used as anchorages for tension leg platforms (TLP's) have very large diameters and penetrate a short distance into the seabed through a combination of self-weight and underbase suction. For example, the Snorre TLP foundation consists of four Concrete Foundation Templates (CFT's), and each CFT comprises three cylindrical concrete cells of diameter $D_0 = 17$ m and wall thickness
$t_w=0.35 \text{ m} \ (D_o/t_w=49)$ (section 2.4.3). During installation, each CFT penetrated the seabed approximately $L=10 \text{ m}$ by self-weight and an additional 1.6 to 2.5 m through the application of suction beneath the caisson lid ($L/D_o=0.7$; Christophersen et al., 1992).

The standard installation procedure for the CET testing program simulates installation by underbase suction with no self-weight component of penetration. This is achieved using the two-component model caisson (section 3.1.2). Prior to penetration, the clay specimen (resedimented Boston Blue Clay) is normally consolidated to a vertical effective stress of $\sigma'_v=0.75 \text{ ksc}$ with the cap and wall components set flush at the clay surface (see Figure 3.13). This corresponds to a compressive total force on the caisson, $F_{\text{tot}}=15.2 \text{ kg}$. In order to simulate pile installation with underbase suction, the wall penetrates the clay at a constant rate ($v_w=0.3 \text{ cm/min}$), while increments of wall force are counterbalanced by equal but opposite increments of cap force, such that the total force on the caisson does not change ($\Delta F_w = -\Delta F_c; \Delta F_{\text{tot}}=0$). The wall penetrates to a depth of $L=5.1 \text{ cm} \ (L/D_o=1)$, whereupon the model caisson is allowed to equilibrate under a compressive load of $F_{\text{tot}}=15.2 \text{ kg}$. This section begins with a summary of the measured suction caisson behavior during installation and then focuses on the wall penetration resistance and the effect of underbase suction.

6.1.1 Summary of Installation Behavior

Fundamental insights into the installation behavior are found by comparing the most representative behavior of the CET tests that incorporated installation with underbase suction (CET4,8-9,11-12) with the one test (CET13) that installed the wall ($v_w=0.3 \text{ cm/min}$) with a constant cap stress, $\sigma'_v=0.75 \text{ ksc}$, as shown in Figure 6.1. In the standard suction installation tests, the total force on the caisson is held constant in the range $F_{\text{tot}}=15$ to 17 kg. In contrast, the total force in CET13 increased to $F_{\text{tot}}=24 \text{ kg}$ within the first $z_w=0.05 \text{ cm}$ of wall penetration and then rose steadily at an average rate of 2.33 kg/cm (from $z_w=0.4$ to 5.1 cm) to reach $F_{\text{tot}}=37.6 \text{ kg}$ by the end of penetration.
Hence, the total additional force required to 'jack' the caisson into the clay is $\Delta F_{\text{tot}} = 22.4$ kg, which is more than twice the total force maintained on the suction-installed tests ($F_{\text{tot}} = 15.2$ kg).

The measured wall force data reveal three distinct zones of behavior: initial, transition, and deep. The initial penetration zone, $z_w = 0$ to 0.2 cm, is characterized by a very stiff wall response, as the wall acquires more than 10 kg of compressive load for the two most representative standard tests (CET8, 9). The penetration moduli for these two tests were $M_w = \Delta F_w / \Delta z_w = 60.5, 63.5$ kg/cm. Note in Figure 6.1 that the softer initial response measured in tests CET4, 11, and 12 is probably due to clay surface disturbance during consolidation (see section 5.2.1, Figure 5.7).

The transition zone extends from $z_w = 0.2$ to 0.6 cm (and in some cases, up to 1.8 cm; see Table 5.5a). The wall force signature during this zone is defined by a "hump-shaped" rise and fall, with a local maximum force at a depth from $z_w = 0.3$ to 1.0 cm. The transitional behavior is probably caused by the build up and subsequent release of intercomponent friction between the cap and wall components and between the wall and inner slip ring (see sections 4.3.2, 5.2.1, Figure 4.12).

Deep penetration occurs for the remainder of the penetration, and is characterized by a wall force that increases at a constant rate with depth. Considering the five most representative suction installation tests, as shown in Figure 6.1, the slope of the regression line in the deep zone ranges from $M_w = 1.29$ to 1.60 kg/cm with an average value of $M_w = 1.45$ kg/cm. By the end of penetration, the wall carries a compressive load, $F_w = 17$–18.7 kg.

The behavior for the wall that was 'jacked' into position (CET13) follows the same general characteristics as the other tests (Figure 6.1). The wall responds very stiffly at first, acquiring $\Delta F_w = 10.5$ kg during the first $z_w = 0.2$ cm of penetration ($M_w = 52$ kg/cm).

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1 The initial penetration zone endpoint ($z_w = 0.2$ cm) was chosen based on results from CET8, which had the least amount of intercomponent friction (see section 5.2.1).
In the deep zone ($z_w=0.4$ to $5.1$ cm), the wall force increases steadily at a rate of $M_w=2.1$ kg/cm to reach $F_w=23.6$ kg. Both the average gradient of the wall force during deep penetration and the final wall load for CET13 are higher than values measured in the suction installation tests (see Table 5.5b). This result may be due to the higher pressure inside the caisson during installation.

In the standard suction installation tests, the cap force behavior is equal but opposite to the wall force behavior ($\Delta F_c=-\Delta F_w$). Figure 6.1 shows that, allowing for some scatter (due to inter-component friction and slight differences in the magnitude of $F_{tot}$), the cap responds stiffly during initial penetration and decreases linearly with penetration depth during deep wall penetration. At the end of wall penetration ($L=5.1$ cm), the cap carries $F_c=+0.5$ to $-3.5$ kg. Note that the cap force for CET13 is held constant at $F_c=15$ kg, as the wall is jacked into position.

Pore pressures were measured directly beneath the cap and by needle probes embedded at a depth of approximately 2 and $2.5$ cm below the caisson prior to penetration. Figure 6.2 shows the excess pore pressure record measured beneath the cap and at four radial locations in the soil mass (P1 and P2 at radii within the caisson wall; P3 and P4 at radii outside the caisson wall). In most cases, small excess pore pressures ($\Delta u<\pm0.2$ ksc) were generated in the soil mass during installation with underbase suction. At the caisson cap/clay interface, the pressure drops ($\Delta u=-0.05$ to $-0.2$ ksc) within the first $z_w=0.2$ cm of penetration as the cap loses load (see Figure 6.1). With continued wall penetration, positive pore pressure generation ($\Delta u<0.3$ ksc) is followed by a general decrease in excess pore pressure so that by the end of penetration, nearly zero excess pore pressure ($-0.2$ ksc $< \Delta u < +0.05$ ksc) remains².

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²Lower excess pore pressure values measured in CET12 reflect the large intercomponent external friction in this test (see Figure 6.1). External friction contributes as much as 1.5 kg to the wall and total force records in most tests and arises from metal-to-metal contact between the wall and/or inner slip ring or soil extruded between the wall and inner slip ring (see section 4.3.2).
At the locations within the soil mass, P1-P3 measure only a slight effect from the passing wall and retreating cap. At the central location within the soil plug (P1, r=0.0 cm), with the exception of CET12, a negligible amount of pore pressure is generated (Δu=±0.1 ksc). At the soil plug location 0.6 cm from the inside caisson wall (P2, r=1.8 cm), low positive pore pressures (Δu_{max}=0.2-0.25 ksc) are generated as the wall approaches and passes the probe tip depth. This pore pressure then drops to negligible values (Δu<0.1 ksc) by the end of penetration. Outside the caisson wall, P3 (r=3.2 cm) measures a similar generation and decline of small excess pore pressure.

Much higher pore pressures were generated by the caisson that was jacked into place (CET13; F_c=15 kg). Figure 6.2 shows that the advancing wall generates Δu=0.6 ksc beneath the cap during the first z_w=1 cm. Thereafter, the excess pressure slowly increases to Δu=0.68 ksc by the end of penetration. Even at 1.9 cm from the wall exterior (P4), positive excess pore pressures were measured (Δu=0.3 ksc as the wall passed the probe; Δu=0.25 ksc by the end of penetration).

Figure 6.3 shows that the vertical displacements of the caisson cap during installation are non-linear functions of the wall tip penetration. This figure also shows the theoretical cap displacement corresponding to 100% of the wall volume displaced (δ_c=V_w/A_c). There are large variations in the pattern of cap movements among the five depicted suction installation tests, but by the end of wall penetration, the net upward cap displacement in four of the tests is similar (δ_c=0.54-0.6 cm). This amount of displacement represents 86-94% of the volume displaced by the wall. The average cap heave (δ_c=0.62 cm) for all five suction tests by the end of installation represents 97% of the displaced wall volume. In test CET13, 'jacking' the wall into place causes the cap to move down δ_c=−0.06 cm during the initial z_w=1.2 cm of penetration. However, by the

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3 In test CET12, the cap displacement is much greater (δ_c=0.84 cm), which could be due the effects of intercomponent friction.
end of penetration, the cap has lifted $\delta_c=0.52$ cm (83\% of the displaced wall volume), which is similar to the cap heave in the suction installation tests.

Figure 6.4 shows the soil surface displacement at five radial locations for five suction installed tests (CET4,8-9,11-12) and the one 'jacked' installation test CET13. Generally, the soil displacements are very small relative to the wall penetration depth for both types of caisson installation. By the end of penetration in the suction installed tests, the surface compresses an amount that decreases with increasing distance from the caisson: $\delta_z=-0.001$ to $-0.004$ cm at S1 ($r=4.2$ cm), $\delta_z>-0.001$ cm at S5 ($r=12.1$ cm). Note that the surface heaves slightly ($\delta_z<0.001$ cm) at the three closest radial locations (S1-S3) for early penetration ($z_w<3$ cm). 'Jacking' the wall to depth (CET13) causes the soil surface to heave at all radial locations. The surface at the two radial locations closest to the wall initially heaves (nearly $\delta_z=0.008$ cm at S1, $r=4.2$ cm), but then compresses with continued wall penetration as soil is drawn inside the caisson.

6.1.2 Wall Penetration Resistance

The resistance to wall penetration is provided by the wall tip bearing capacity during initial penetration and the wall interface friction during deep penetration. Figure 6.5a shows the wall force as a function of wall tip penetration for the five suction installation tests that best represent initial and deep penetration (CET4,8-9,11-12)\textsuperscript{4} and for the 'jacked wall' test (CET13). During deep penetration, the wall resistance increases linearly due to friction ($f_s=$average skin friction) between the caisson wall and soil. Therefore, the tip capacity from the shallow penetration response can be estimated by extrapolating the linear 'deep' response back to $z_w=0$ cm.

\textsuperscript{4}The measured transition zone is a result of intercomponent friction and is not included in this plot (see sections 6.1.1 and 5.2.1). To show continuity between the initial and deep zones, the deep penetration regression lines (dotted in figure) are extended between the end of the initial zone ($z_w=0.2$ cm) and the start of deep penetration.
Assuming that the penetration is rapid enough (v_w=0.3 cm/min; full penetration to L=5.1 cm in 17 minutes), both the tip capacity and wall interface friction can be related to the undrained shear strength of the clay. Tip capacity is calculated using standard bearing capacity formulae (e.g., Terzaghi, 1943):

\[ q_{ult} = N_c s_u + \sigma_{vc} \]  \hspace{1cm} (6.1)

where, \( q_{ult} = \) wall tip capacity  
\( N_c = \) bearing capacity factor  
\( s_u = \) undrained shear strength  
\( \sigma_{vc} = \) consolidation vertical total stress

For convenience, this equation can be rearranged by solving for the 'net' tip resistance (\( q_{ult} - \sigma_{vc} \)) and normalizing both sides by the vertical consolidation effective stress (\( \sigma'_{vc} \)):

\[ \frac{q_{ult} - \sigma_{vc}}{\sigma'_{vc}} = N_c \frac{s_u}{\sigma'_{vc}} \]  \hspace{1cm} (6.2)

where, \( \sigma'_{vc} = \) consolidation vertical effective stress

In the CET experiments, the consolidation vertical total stress was \( \sigma_{vc}=0.75 \) ksc (=\( \sigma'_{vc} \); no excess pore pressure prior to penetration). Tip capacity can be obtained by dividing the wall force intercept \( F_0 \) by the wall tip area \( A_w (=2.248 \) cm²):

\[ q_{ult} = F_0/A_w \]  \hspace{1cm} (6.3)

The intercepted \( F_0 \) is found by extending the wall force gradient during deep penetration back to the ordinate (\( z_w=0 \) cm) in the plot of wall force versus wall tip penetration.
Figure 6.5b shows the extended regression lines for the five most representative suction installation tests (CET4,8-9,11-12) and for CET13, where the wall was jacked into place. The resulting force intercept for the suction installed tests is consistent at $F_0 = 10.2 \pm 0.2$ kg, while $F_0 = 12.4$ kg for CET13 (see Table 6.1).

These data yield a net normalized tip resistance factor, $(q_{ult}-\sigma_{vc})/\sigma'_{vc} = 5.13 \pm 0.16$ for installation by underbase suction and $(q_{ult}-\sigma_{vc})/\sigma'_{vc} = 6.30$ for CET13. In comparison, field measurements of net tip resistance factors from piezocone tests in $K_0$-normally and lightly overconsolidated BBC (OCR=1.0-1.2) range from 2.5-4.0 (Morrison, 1984; Ladd, 1991). Theoretical predictions for closed-ended piles, based on the Strain Path Method and MIT-E3 model (at OCR=1.0) range from 2.5-2.6 (Aubenay, 1992). Thus, the tip resistance for the model caisson (a thin-walled open-ended pile) used in the CET testing program is higher ($\times 1.3$ to 2.5) than for closed-ended penetrometer geometries considered in previous investigations.

Using equation 6.2 and assuming a reference undrained strength ratio for $K_0$-normally consolidated RBBC in triaxial compression, $s_{uTC}/\sigma'_{vc} = 0.32$ (Sheahan, 1991), the resulting tip bearing factor, $N_c = 16.0 \pm 0.5$ for suction installed tests and $N_c = 19.7$ for CET13.

The second component of wall resistance is the wall skin friction, which is related to the wall force gradient during deep penetration ($f_w = M_w = \Delta F_w/\Delta z_w$; see section 6.1.1). Assuming maximum mobilization of the undrained shear strength of the clay at all points along the inside and outside surfaces of the wall, the average skin friction can be computed:

$$f_s = \frac{f'_w}{2\pi(n + r_0)} \quad (6.4)$$

where, $f_s =$ average skin friction
$f_w = \text{wall force gradient}$  
$r_i = \text{inside wall radius (2.395 cm)}$  
$r_o = \text{outside wall radius (2.54 cm)}$

Regression analyses on the deep penetration behavior for those tests depicted in Figure 6.5b reveal a wall force gradient, $f_w = 1.45 \pm 0.13 \text{ kg/cm}$ for installation by underbase suction and $f_w = 2.13 \text{ kg/cm}$ for CET13 (see Table 6.1). Using equation 6.4, the average skin friction for the suction installed tests is $f_s = 0.047 \pm 0.004 \text{ ksc}$. The conventional skin friction $\beta$ and $\alpha$ factors are obtained by normalizing the skin friction with the consolidation vertical effective stress and the direct simple shear undrained shear strength, respectively:

$$\beta = \frac{f_s}{\sigma'_{vc}} = 0.063 \pm 0.005 \quad (6.5)$$

$$\alpha = \frac{f_s}{s_{uDSS}} = 0.31 \pm 0.03 \quad (6.6)$$

For installation with underbase suction, $\beta = 0.063 \pm 0.005$, which is much lower than the undrained shear strength ratio of $K_0$-normally consolidated RBBC in direct simple shear, $s_{uDSS}/\sigma'_{vc} = 0.205$ (Ladd, 1991), and hence $\alpha = 0.31 \pm 0.03$. The skin friction is significantly higher for CET13 ($f_s = 0.069$, $\beta = 0.092$, $\alpha = 0.45$). The lower skin friction measured in the suction-installed tests suggests perhaps that the suction imposed on the developing soil plug may lower the inside wall adhesion.

Overall, the measured values of skin friction are in very good agreement with previous predictions for undrained deep penetration of open and closed-ended piles in BBC using the Strain Path Method and MIT-E3 model ($\beta = 0.073-0.096$; Whittle and Baligh, 1988; Whittle, 1992).
6.1.3 Effective Stress Changes and Soil Movements During Installation

An analysis of the caisson force redistribution, pore pressure generation, and soil displacements during installation reveals that both installation by underbase suction and installation by 'jacking' cause the same two very interesting conditions in the soil: 1) the loss of most, if not all, of the effective stress in the soil plug created by the advancing wall, and 2) the movement of most of the wall-displaced soil volume toward the caisson interior. Figure 6.6 shows the cap total stress ($\sigma_v = F_c / A_c$), measured excess pore pressure beneath the cap ($\Delta u$), and the resulting estimated effective stress at the cap/soil interface ($\sigma'_v = \sigma_v - \Delta u$) for the suction-installed tests (CET4, 8-9, 11-12) and the 'jacked-in' test CET13. As shown by the average value of the underbase suction tests, the effective stress falls rapidly from the consolidation value of $\sigma'_v \approx 0.85$ ksc to $\sigma'_v \approx 0.1$ ksc after the wall has penetrated only $z_w = 0.5$ cm. With continued wall penetration, the effective stress recedes essentially to zero. Despite a completely different imposed stress condition (constant $F_c$) for the jacked installation test CET13, the effective stress at the interface drops rapidly toward zero (from $\sigma'_v \approx 0.8$ ksc to $\sigma'_v \approx 0.25$ ksc) within the first 0.5 cm of penetration, but maintains a slightly positive effective stress ($\sigma'_v \approx 0.1$ ksc) for the remainder of installation. The effective stress behavior for CET13 is probably more reliable because: 1) there is guaranteed cap/soil contact, which leads to more uniform conditions across the cap and 2) the positive excess pore pressures are less suspect than the negative excess pore pressures generated during suction installation. It is clear from the figure that the loss of effective stress during suction installation is caused by the reduction of cap stress, while the

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5 The effective stress calculation is an estimate because the pore pressure is measured beneath only 22% of the cap area, while the total stress is calculated using the entire cap area.
6 While the total stress on the caisson during consolidation is held constant at $\sigma'_vc = 0.75$ ksc ($= Fc / A_{tc}$), internal inter-component friction between the wall and cap may reduce the wall load and increase the cap load. Hence, the consolidation stress on the cap prior to installation is slightly high at $\sigma'_vc = 0.85$ ksc.
7 The calculated 'slightly negative' effective stress values are the result of uncertainty in the cap force measurement due to cap/wall friction (see section 4.3.2) and uncertainty in the cap pore pressure measurement (see footnote 5).
effective stress drop during jacked installation is a result of a large increase in excess pore pressures.

While no total stress measurements were available at soil plug locations below the cap interface, the vertical effective stress could be estimated at the pore pressure probe tip depth by considering total force equilibrium on the soil plug as the wall tip reaches that depth. The skin friction is assumed constant during penetration and equal along the inner and outer surfaces of the wall. From total force measurements at the cap interface ($F_c$), the average frictional force along the inside caisson wall ($f_s \pi D_i z_w$), and excess pore pressure measurements at the centerline probe (P1), the average effective stress of the soil plug can be calculated at the wall tip level. At a wall penetration of approximately $z_w=2.0$ to 2.5 cm, data from tests CET4,11, and 12 yield an average vertical effective stress across the soil plug bottom of $\sigma'_v<0.2$ ksc. At the final penetration depth ($L=5.1$ cm), the effective stress at the probe tip depth (center of the soil plug) has dropped to $\sigma'_v<0.15$ ksc. While no pore pressures were measured at the wall tip depth after full penetration, one can estimate the vertical effective stress at this depth by assuming a pore pressure pattern similar to that measured at $z_w=2.0$ to 2.5 cm by P1 as the wall passed that depth. Taking force equilibrium on the entire soil plug after full penetration, calculations yield a near-zero vertical effective stress ($\sigma'_v<0.06$ ksc) at the soil plug bottom ($z_w=5.1$ cm). Even with the uncertainty inherent in these assumptions and calculations, it is clear that suction installation causes a dramatic, if not complete, loss of vertical effective stress within the entire soil plug. No interior caisson pore pressure data was available to confirm this behavior for the jacked-in test CET13.

The second striking condition caused by both underbase suction and jacked installation is the large upward movement of the cap. As shown in Figure 6.3 (section 6.1.1), the average cap heave by the end of wall penetration for the suction-installed tests corresponds to 97% of the wall-displaced volume, while the cap heaved nearly as much (81% of the wall-displaced volume) in the jacked-in test CET13. This phenomenon has
been observed in other experimental studies. For example, Dyvik et al. (1993) report heave inside a 4-cell field scale model in clay during penetration with underbase suction.

A simple analysis of the stress conditions imposed within the soil plug can be made by assuming that the soil elements are subject to an undrained extension mode of shearing (i.e., upward movement of the cap must balance inward radial displacements of soil contained within the plug). Considering just the soil plug created during wall installation, the vertical stress on the top of the plug decreases, as measured ($\Delta \sigma_v<0$). The change in horizontal stress on the developing plug is not measured and therefore unknown. Hence, the total and effective stress paths cannot be drawn in $p'$-$q$ space [$p'=(\sigma'_v+\sigma'_h)/2$; $q=(\sigma_v-\sigma_h)/2$].

However, a clearer picture of the extension mode of shear of the soil plug can be observed by considering the vertical effective stress behavior as a function of axial strain. The vertical effective stress on the soil plug is the effective stress at the cap/soil interface, which has been depicted in Figure 6.6. Axial strain can be approximated by dividing the cap heave ($\delta_c$) by the soil plug height ($z_w+\delta_c$). Figure 6.7 shows the axial strain plotted versus wall tip penetration for the five suction installation tests (CET4,8-9,11-12), the average of the suction tests, and the jacked-in test CET13. The average suction plot shows that the axial strain increases to $\varepsilon_a=5\%$ within the first $z_w=0.2$ cm of penetration; thereafter, the axial strain rises at an approximately linear rate to $\varepsilon_a=12\%$ by the end of installation. Figure 6.8 shows the vertical effective stress ($\sigma'_v$) normalized by the consolidation stress ($\sigma'_{vc}$) as a function of the axial strain for the soil plug during CET installation and for two triaxial extension (unloading) tests on normally-consolidated RBBC (CKoUTXE(U); Sheahan, 1991). For clarity, the complete strain history is shown only for the average value of the suction-installed tests. Prior to installation, the normalized effective stress is slightly higher than one ($\sigma'_v/\sigma'_{vc}=1.05$ to 1.3) because intercomponent friction between the cap and wall apportions more force to the cap (see section 4.3.2). During installation, the effective stress drops rapidly as the soil plug
extends, reaching a value of $\sigma'_v/\sigma'_{vc}=0$ at an axial strain of $\varepsilon_a=10\%$. This behavior is similar to triaxial extension, where the effective stress drops rapidly to a residual value of $\sigma'_v/\sigma'_{vc}=0.1$ at the same axial strain.

Despite a much higher total stress in the jacked-in test CET13, the soil plug axial strain and effective stress behavior are essentially the same as for the suction-installed tests. As shown in Figures 6.3 and 6.7, the higher stress in CET13 causes initial downward cap motion, which causes initial soil plug axial compression. However, deeper penetration causes large soil plug heave similar to suction-installation behavior. Figure 6.8 shows the effective stress versus soil plug axial strain behavior for CET13 without the initial compression period. The effective stress falls from an initial value of $\sigma'_v/\sigma'_{vc}=1.07$ to a residual value of $\sigma'_v/\sigma'_{vc}=0.1$ to 0.2 as the soil plug extends to $\varepsilon_a=2\%$. While the total stress paths for the suction-installed tests and the jacked-in test are different, the effective stress and strain behavior are similar. Thus, the effective stress behavior of the soil plug during caisson installation is a strain-controlled process.

Research on open pile installation wherein the pile is jacked into the clay shows that the measured heave inside the caisson is much less than that measured in CET13. At Delft Geotechnics, researchers conducted two centrifuge tests (150g and 300g) in kaolin using a model with prototype length $L=36$ m, outside diameter $D_o=13.05$ m, and wall thickness $t_w=1.275$ m ($L/D_o=2.8$, $D_o/t_w=28$) (Hjortnæs-Pedersen and Bezuijen, 1992a,b). The models were jacked into the clay at a rate of 0.78 cm/min. The results show that the measured heave of the soil plug ($\delta_c=3.8-5.1$ m) by the end of penetration corresponds to only 20-26% of the volume displaced by the caisson wall. However, the soil surface was stress-free ($\sigma_v=\sigma'_v=0$) outside the model, which does not simulate CET boundary conditions, where $\sigma_v=\sigma'_v=0.75$ ksc at the surface.

Using a new method of analysis, referred to as the Shallow Strain Path Method (SSPM; Sagaseta et al., 1995), for predicting the ground deformations caused by undrained penetration of piles and caissons, researchers at MIT (Whittle et al., 1996)
simulated the Delft test boundary conditions. The SSPM formulation combines the Strain Path Method developed by Baligh (1985) for steady, deep penetration in clays, with the work of Sagaseta (1985) for estimating deformations near the stress-free ground surface. Figures 6.9 and 6.10 compare SSPM predictions of the surface heave profile at the end of penetration ($L=5.1$ cm, $L/D_0=1$) and vertical displacements of the cap during penetration with measured data from CET13 and from the suction-installed tests (CET4,8-9,11-12). The total predicted heave within the caisson ranges from $\delta_z=0.09$-0.12 cm. This range increases to $\delta_z=0.17$-0.22 cm when an inherent clearance correction is included in the analysis. The measured cap displacements ($\delta_c=0.52$-0.84 cm) are much larger than the predictions for installation by both underbase suction and wall jacking. Furthermore, the measurements show much larger rates of cap displacement for wall tip penetration, $z_w>1$ cm (Figure 6.10). Outside the caisson (Figure 6.9) the analyses predict surface heave movements across the entire CET cell ($\delta_s=0.1$ cm on the outside wall of the caisson), while the experimental data show very small net displacements of the original surface.

Discrepancies between the SSPM predictions and the data from the CET tests can be attributed to the different boundary conditions upon the soil. A large reduction in effective stress below the cap combined with a constant soil surface stress of $\sigma_v^*=\sigma_v=0.75$ ksc outside the caisson leads to stress gradient between the inside and outside of the caisson. This gradient may explain why much of the wall-displaced volume of soil (81-97%) moves inside the caisson.

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8The inherent clearance correction accounts for the fact that the shaft radius of a simple tube is not exactly constant, but expands with distance from the tip, especially at points close to the free surface.
6.2 EQUILIBRATION

In all CET tests, installation is followed immediately by an equilibration phase, during which the total force is maintained constant at $F_{\text{tot}}=15.2$ kg, and no relative displacement is allowed between the cap and well ($\Delta \delta_w = \Delta \delta_c$). This phase lasted from 18 to 33 hours in order to ensure complete dissipation of installation pore pressures. For most of the tests in which the caisson was installed with underbase suction, the total force on the caisson remains constant through both the installation and equilibration phases ($F_{\text{tot}}=15.2$ kg). In contrast, for the jacked-in test CET13, the total force was reduced from $F_{\text{tot}}=33.6$ kg at the end of penetration to $F_{\text{tot}}=15.2$ kg during the equilibration phase. Differences between the suction-installed tests and the jacked-in test are highlighted throughout this section, which starts by summarizing the measured behavior of the caisson during post-installation equilibration. Following this, the settlement and dissipation rates are analyzed. Lastly, the post-installation equilibration behavior is compared to the post-tensile loading equilibration period (re-equilibration).

6.2.1 Summary of Equilibration Behavior

Figure 6.11 shows the total, wall, and cap force records on a log time scale for 5 suction-installed tests (CET3,8-9,11-12) and the jacked-in test CET13. For the suction driving tests, while the total force remains steady at $F_{\text{tot}}=15.2$ kg, the wall and cap forces redistribute rapidly to equilibrium values within 3 minutes. For the remainder of the equilibration period, the wall carries nearly all of the total load. The wall force starts in the range $F_w=17-20$ kg and falls slightly to $F_w=13.5-16.5$ kg. The cap force begins at $F_c=0$ to -4 kg and rises only to $F_c=-0.5$ to 1.5 kg. The total force in CET13 starts out much higher at $F_{\text{tot}}=33.6$ kg, but is lowered quickly (within 2 minutes) to the target value $F_{\text{tot}}=15.2$ kg (with corresponding reductions in the wall and cap forces: $F_w=23.5$ to 1.7 kg, $F_c=14$ to 3.5 kg). Due to the large installation excess pore pressures in CET13, a
much greater amount of time was required (t=100 to 200 minutes) for redistribution of the wall and cap forces, but the equilibrium values (F_w≈13 kg, F_c≈2 kg) are roughly equivalent to those achieved in the standard suction tests.

The excess pore pressure dissipation record as measured directly beneath the cap and at four probe locations is shown in Figure 6.12. Installation by underbase suction generated very minor amounts of excess pore pressure (∆u=-0.2 to +0.05 ksc) at the cap/soil interface and at the centerline location within the soil plug (P1). As the cap force equilibrated within the first 3 minutes of equilibration, slight positive pore pressures (∆u=0 to 0.1 ksc; very little variation from test to test) were generated at these two locations. This excess pore pressure dissipated after roughly 16 hours (1000 minutes) of equilibration. Because there is zero excess pore pressure at the clay surface outside the caisson, the flow proceeds from the soil plug surface, down through the soil plug and around the wall tip, and then up to the exterior clay surface. Small installation pore pressures (∆u=0 to 0.1 ksc) measured at the other probe locations (P2,P3) also dissipated within 16 hours. Jacking the wall into position (CET13) generated much larger excess pore pressures within the soil plug (∆u>0.6 ksc at the cap/soil interface, while ∆u>0.2 ksc at P4), but this pressure also dissipates after 16 hours of equilibration.

Cap and wall settlement behavior, as depicted in Figure 6.13, is clear. The suction-installed caisson barely settles (δ_w=δ_c<0.005 cm) during the first ten minutes, but settles at an increasing rate until reaching a certain point in time (t=70 to 100 minutes), after which the settlement rate is approximately log-linear. The rate of log-linear settlement varies among the tests; this variation reflects the differences in the drainage conditions from test to test (see section 4.2.2). The final settlement was small and varied from δ_w=δ_c=-0.02 to -0.09 cm (i.e., approximately 10-65% of the wall thickness). The caisson in test CET13 heaves nearly 0.012 due to unloading of the caisson (t<3 minutes) and then undergoes a net settlement of -0.04 cm as the pore pressures dissipate.
Figure 6.14 portrays the soil surface settlement at five radial locations (S1-S5) and shows the same pattern as the caisson settlement. In general, the final settlement is small. At a radius of \( r=4.2 \) cm (S1), the final settlement varies \( \delta_s=-0.027 \) to \(-0.083 \) cm (18-57% of wall thickness). At \( r=12.1 \) cm (S5), the settlement varies \( \delta_s=-0.002 \) to \(-0.057 \) cm (1-40% wall thickness).

The typical measured equilibration behavior is summarized in Figure 6.15, which presents the caisson forces, excess pore pressure beneath the cap, cap displacement, and soil surface displacement at \( r=5.2 \) cm (S2) for the average of the suction installation tests (CET8-9, 11-12) and for test CET13.

6.2.2 Post-Installation Equilibration Analysis

Three significant geotechnical uncertainties are associated with the equilibration period that follows the installation of a suction caisson: 1) the rate of excess pore pressure dissipation, 2) the rate of caisson settlement, and 3) the change in properties of the soil surrounding the caisson. The rate of pore pressure dissipation is a function of the coefficient of consolidation \( (c_v,c_h) \) and drainage path length (related to caisson radius \( r \) and penetrated length \( L \)). As discussed above (section 6.2.1, Figures 6.12,6.15), the suction installation process generated negligible excess pore pressure, which precludes the construction of time curves that relate change in pore pressure to a time factor. However, the coefficient of consolidation can be approximated by assuming the pore pressures dissipate according to one-dimensional consolidation theory. Assuming a drainage path from the top of the soil plug to the exterior soil surface (2L=10.2 cm) and 90% consolidation by \( t=16 \) hr, the coefficient of consolidation is approximately \( c_v=1.5 \times 10^{-3} \) cm\(^2\)/sec, which is consistent with previous measurements for normally consolidated RBBC \( (c_v=1.31\pm0.22 \times 10^{-3} \) cm\(^2\)/sec for \( \sigma'_v=0.5 \) to 1.0 ksc; Seah, 1990).

During the initial \( t<100 \) minutes of equilibration, the caisson in the CET cell settles a very small amount \( \delta_c<-0.02 \) cm as the excess pore pressures begin dissipating (Figures
6.13, 6.15). However, for \( t > 100 \) minutes, the rate of settlement is proportional to \( \log t \), generating final settlements of up to \( \delta_c = 0.09 \) cm (CET9). These rates of settlement are approximately constant across the surface of the clay (but vary significantly from test to test; see section 6.2.1) and are consistent with typical secondary compression rates for RBBC. Estimated rates of secondary compression for those tests depicted in Figure 6.13 (CET3–6, 8-13) range from \( C_{ae} = \frac{d\delta_c}{d(\log t)} \approx 0.001 \) to \( 0.005 \), with an average value, \( C_{ae} = 0.0034 \pm 0.0016 \). Based on an approximate compression ratio, \( CR = 0.12 \), for resedimented BBC in the stress range \( \sigma'_v = 0.5-1.0 \) ksc (Seah, 1990), the secondary compression ratio is \( C_{ae}/CR = 0.028 \pm 0.013 \), which is lower than value estimated for inorganic soft clays \((C_{ae}/CR = 0.04 \pm 0.01\); Mesri & Castro, 1987). However, the CET secondary compression values are well within the range reported previously for RBBC \((C_{ae}/CR = 0.015-0.04\); Sheahan, 1991). Hence, in terms of settlement, suction installation in the CET cell simply causes a brief \((t=2 \) hr\) interruption of the ongoing secondary compression of the clay sample.

The change in soil properties surrounding the caisson is probably the most difficult to determine, given the lack of appropriate instrumentation in the CET cell. However, some information regarding the state of stress within the soil plug can be deduced from load and pore pressure data. Within 3 minutes following installation by underbase suction, the wall carries at least 89% \((F_w > 13.5 \) kg\) of the total force \((F_{tot} = 15.2 \) kg\), while the cap load is very low \((F_c < 1.7 \) kg\). Since these conditions remain constant throughout equilibration, and the small installation pore pressures dissipate slowly, the vertical effective stress within the soil plug remains near zero (average effective stress beneath the cap, \( \sigma'_v \approx 0.06 \) ksc). Outside the caisson, constant vertical stress \((\sigma_v = 0.75 \) ksc\) conditions prevail on the clay surface, which allows the vertical effective stress to return to \( \sigma'_v = 0.75 \) ksc after excess pore pressures have dissipated. The extent of the disturbed zone surrounding the penetrated wall is undetermined, but reconsolidation allows the soil adjacent to the wall exterior to regain strength.
6.2.3 Re-Equilibration

Following the point of maximum tensile resistance in five tests (CET5-6, 12-14), the caisson was re-loaded with the original compressive force, $F_{tot}=15.2$ kg, for a period of at least 24 hours. This post-tensile loading period is referred to as re-equilibration and can be compared to the post-installation equilibration period discussed in sections 6.2.1-6.2.2. In 2 tests, CET5 and 6, the caisson capacity was mobilized by monotonic tensile loading with $\delta_w=0.3$ cm (see Figure 5.36), while in tests CET12-14, the caisson failed under sustained tensile loading prior to re-equilibration. Figures 6.16-6.19 show the caisson forces, excess pore pressures, and caisson and soil displacements as a function of $\log t$ during re-equilibration.

Figure 6.16 shows the total, wall, and cap forces during re-equilibration for CET5-6, 12-14. For comparison, the average forces during post-installation set-up for 4 suction-installed tests (CET8-9, 11-12) are also shown. Up to 4 minutes are required to unload the tensile forces upon the caisson and restore the original compressive load, $F_{tot}=15.2$ kg. The time frame for re-distribution of the wall force (400-500 minutes) is much longer than that measured for equilibration following installation by underbase suction (3 minutes). However, at the end of this long redistribution period, the wall force carries most of the total load ($F_w>90\%F_{tot}$), as it does during post-installation set-up ($F_w(avg.)=95\%F_{tot}$). Note that as the equilibrium total load is placed on the caisson, the cap acquires compressive values of up to $F_c=12$ kg (CET14) within the first minute before shedding these loads for the remainder of re-equilibration (this 'overshoot' behavior is also evident in the wall force behavior). The cap force rise is responsible for generating positive excess pore pressures in the soil plug.

Figure 6.17 shows the excess pore pressure dissipation directly beneath the cap and at four soil mass locations (P1-P4) for five tests during the re-equilibration period and for the average of four suction-installed tests during the original set-up. It is clear that re-loading the caisson in the CET cell causes a preliminary rise in cap force, which in turn
creates positive excess pore pressures ($\Delta u=0.2$-0.7 ksc) within the soil plug, as measured beneath the cap and at P1-P2 ($r=0$-1.8 cm). Smaller positive excess pore pressures ($\Delta u<0.35$ ksc) are measured outside the caisson at P3-P4 ($r=3.2$-4.45 cm). As with the post-installation set-up, complete dissipation occurs within 1000 minutes.

The tensile loading history had a significant effect on the caisson settlement during re-equilibration, as shown in Figure 6.18. Final re-equilibration settlements ($\delta_s=-0.18$ to -0.37 cm) were 2 to 4 times greater than the average final equilibration settlement ($\delta_s=-0.08$ cm) of four suction-installed tests. In two tests (CET5-6), the caisson was re-equilibrated following monotonic tensile loading to capacity. In these tests, the caisson settled at an increasing rate vs. $\log t$ until reaching approximately $\delta_c=0.07$ cm. Thereafter, the settlement rate was consistent with secondary compression of the clay (for CET5, $C_{\alpha e}=0.0044$). In three tests (CET12-14), the caisson was re-equilibrated following sustained tensile loading. The initial settlement ($\delta_c=0.2$, 0.41, 0.16 cm, respectively) was nearly equal to the amount of upward displacement ($\delta_c=-0.19$, -0.35, -0.14 cm) achieved during the preceding sustained load stage. The caisson then settled at the secondary compression rate ($C_{\alpha e}=0.0011$, 0.0008, 0.0014). The secondary compression rates observed during re-equilibration are consistent with those measured during post-installation equilibration. Note that, while larger than the post-installation set-up displacements, the final re-equilibration settlements are still small ($\delta_c=1$ to 2.5 times the wall thickness $t_w=0.145$ cm).

Soil surface displacements during re-equilibration reflect only the secondary compression of the clay. Figure 6.19 shows the displacement during re-equilibration (CET5-6,12-14) and during post-installation set-up for the average of four suction-installed tests (CET8-9,11-12). All settlements are small ($\delta_s>-0.15$ cm) and show a trend of decreasing displacement with increasing radial distance from the caisson. The larger compression for CET5 (see S1-S3 in Figure 6.19) reflects a larger secondary compression rate.
6.3 MONOTONIC UPLIFT CAPACITY

Under normal operational conditions (i.e., calm seas), tension leg platforms apply an eccentric cyclic tensile load to a suction anchor foundation, which resists such loads through self-weight and ballast. During storms, TLP's apply a much greater tensile load to the suction anchor, which must rely on sidewall skin friction and underbase suction for additional resistance. While the CET design currently is not capable of duplicating cyclic eccentric tensile loading, it can simulate vertical monotonic and sustained axial tensile loading, which provide fundamental insights into the effects of tensile loads on suction caissons. This section interprets the behavior of monotonic uplift capacity tests on the two-component model caisson in the CET cell.

These capacity tests established a reference strength and behavior for later comparison with sustained load tests (section 6.4). Following the equilibration period, the caisson was pulled vertically at a rate of \( v_w = -0.03 \) cm/min while maintaining zero relative displacement between the cap and wall (i.e., the caisson was pulled as one monolithic unit). The 'virgin' capacity tests covered in this section include three that were pulled to complete extraction (CET3,4,8) and two that were pulled to capacity and then re-equilibrated (CET5,6). The initial 'monotonic' portion (\( \delta_w < 0.01 \) cm) of six sustained load tests (CET9-14) are also included. Section 6.3.1 summarizes the force, excess pore pressure, and soil displacement behavior during 'virgin' monotonic loading. Section 6.3.2 discusses the estimated mode of failure and components of capacity. In sections 6.3.3 and 6.3.4, the wall interface friction and reverse end bearing components of capacity are evaluated, respectively. An alternative calculation of the caisson capacity is presented in section 6.3.5. The monotonic pullout behavior of previously tensile-loaded and re-equilibrated caissons are compared to virgin pullout behavior in section 6.3.6. Lastly,

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\(^9\)Section 2.1 provides a complete description of the life of a suction caisson for TLP applications.
section 6.3.7 looks at the effect of increasing the pullout rate during several virgin and re-equilibrated capacity tests.

6.3.1 Summary of Monotonic Tensile Loading Behavior

Figure 6.20 shows the total, wall, cap force components for the four most reliable tensile capacity tests (CET5,6,8; only $F_{tot}$ for CET4), and Figure 6.21 depicts the early portion ($\delta_w=0.0$ to 0.02 cm) of CET5,6 combined with the initial monotonic portion of five sustained load tests (CET9-14). In all tests, the caisson exhibits a very stiff initial response ($M_{tot}=\Delta F_{tot}/\Delta \delta_w=5000-9000$ kg/cm) within the first $\delta_w=0.002$ cm of pullout. After reaching a well-defined yield point between $\delta_w=0.01$ and 0.02 cm, the caisson continues to pick up load, but at a rapidly decreasing rate. Maximum capacity ranges from $F_{tot}=-22.4$ to -24.0 kg and occurs at $\delta_w=0.23$ to 0.3 cm. The average capacity of CET5,6, and 8 is $F_{tot}=-22.8\pm0.7$ kg, which is reached at $\delta_w=0.25\pm0.03$ cm. For displacements beyond peak up to $\delta_w=0.4$ cm, there is only slight post-peak softening (e.g., in CET8, $\Delta F_{tot}=0.4$ kg from peak at $\delta_w=0.23$ cm to $\delta_w=0.4$ cm.

At equilibrium (prior to the start of pullout), the wall maintains an initial compressive force that ranges from $F_w=9.7$ to 16.4 kg. The initial wall stiffness ranges from $M_w=\Delta F_w/\Delta \delta_w=2600-3300$ kg/cm, which is significantly lower than that of the overall caisson. By the time the wall reaches yield ($\delta_w=0.01$ to 0.02 cm), it carries a tensile load ranging from $F_w=-6$ to -1.7 kg. Maximum wall resistance ranges from $F_w=-12$ to -14 kg, and is mobilized at a displacement range $\delta_w=0.15$ to 0.25 cm, which is close to the mobilization range for total capacity ($\delta_w=0.23-0.3$ cm). Based on just the three reliable tests CET5,6, and 8, the wall contributes an average of 58% of the total capacity.

The cap carries almost no load at equilibrium prior to pullout ($-4.4kg < F_c < 1$ kg), but mobilizes $F_c=-4$ to -7 kg almost instantaneously (i.e., within $\delta_w=0.001$ cm), and then slowly accumulates $\Delta F_c=-3.5$ to -5 kg up to the displacement necessary ($\delta_w=0.2$ cm) for peak capacity. The cap shows a very stiff initial response, as $M_c=\Delta F_c/\Delta \delta_w=2500-6400$
kg/cm, with a well-defined yield point at $\delta_w=0.003$ cm. At the average displacement ($\delta_w=0.25$ cm) required to mobilize maximum total capacity ($F_{tot}=-22.8$ kg), the average cap force is $F_c=-9.7\pm1.4$ kg, or 42% of the capacity.

Figure 6.22 shows the 'best estimate' of the total, wall, and cap force behavior during 'virgin' pullout (monotonic pullout 1). For each caisson force component, a range of values is bounded by low and high estimates, which were obtained from the most representative data at both early displacement ($\delta_w=0.0$ to 0.02 cm) and for displacements up to peak resistance ($\delta_w=0.0$-0.4 cm). The best estimate of caisson capacity averages $F_{tot}=-23$ at $\delta_w=0.25$ cm. At this displacement, the wall force ranges from $F_w=-11.4$ to -14.9 kg, while the cap ranges from $F_c=-9$ to -11.8 kg.

Excess pore pressures, as measured beneath the cap and at four locations in the soil (P1-P4), are depicted in Figure 6.23. In general, monotonic pullout of the caisson generates large negative excess pore pressure in the soil plug within the caisson. Based on 7 tests (CET5,9-14) at cap yield ($\delta_w=0.003$ cm), the excess pore pressure beneath the cap and at the two soil plug probe locations (P1-P2)$^{10}$ ranges from $\Delta u=-0.2$ and -0.4 ksc. Peak excess pore pressure is generated at approximately the same displacement ($\delta_w=0.2$ cm) as maximum cap force ($\Delta u_{max}=-0.4$ to -0.5 ksc beneath the cap and at P1 for CET5,6). Much greater peak pore pressure ($\Delta u_{max}=-0.68$ ksc) is generated in the soil plug near the wall (P2, CET8). Completely different pore pressure behavior is measured outside the caisson. At small displacements ($\delta_w\leq0.003$ cm), the pore pressure pattern near the outside wall of the caisson (P3) follows closely the measurements inside ($\Delta u=-0.2$ to -0.4 ksc) for three tests (CET9-11), while much smaller negative pressures are generated in CET8,14. With continued extraction ($\delta_w=0.02$ cm), the exterior pore pressure stabilizes at $\Delta u=0$ ksc, but then decreases toward $\Delta u=-0.1$ ksc once the capacity is fully mobilized ($\delta_w=0.2$ cm).

$^{10}$P1(r=0 cm) and P2(r=1.8 cm) are located approximately 2 to 3 cm above the wall tip prior to pullout.
Figure 6.24 shows the soil surface movements at five radial locations (S1-S5) for four tests (CET4-6,8) wherein the caisson was pulled to capacity. At radial distances close to the wall (S1, S2), the soil surface heaves slightly ($\delta_s=0.001-0.003$ cm) during the initial phase of loading, $\delta_w<0.1-0.15$ cm. At locations farther from the wall (S3, S4, S5), settlements increase monotonically with pullout displacement of the caisson. However, by the end of pullout ($\delta_w=0.3$ cm) the soil surface has moved very little, less than $\delta_s=0.015$ cm ($\delta_s/\delta_w<4\%$).

6.3.2 Failure Mode and Components of Capacity

In order to predict the ultimate tensile capacity of the caisson in the CET cell, an understanding of the failure mode is required. Prior experimental research on the pullout of caissons installed in clay have based capacity predictions on observations of a failure mechanism that is similar to that for a bearing capacity failure with the direction of movement reversed, which is also known as 'reverse end bearing' failure (Finn and Byrne, 1972; Byrne and Finn, 1977; Rapoport and Young, 1985; Fuglsang and Steensen-Bach, 1991; Renzi et al., 1991; Clukey and Morrison, 1993; and Clukey et al., 1995)\(^{11}\). In this failure mode, axial tensile load on the caisson mobilizes: 1) the limiting skin friction at the soil/caisson interface on the exterior surface of the caisson wall, and 2) the shear resistance of the soil mass (reverse end bearing) below the full basal area (footprint) of the caisson wall.

Figure 6.25 shows a schematic drawing of the reverse end bearing failure mechanism and a derivation of the components of resistance to breakout (adapted from Clukey & Morrison, 1993). The failure pattern is analogous to that for compressive bearing capacity, which is based on solutions using plasticity theory. In zone I, the conical volume of soil directly below the caisson basal area, the soil undergoes vertical extension as a result of the vertical tensile load, while in zone III, the soil extends horizontally due to

\(^{11}\)These research studies were reviewed in Chapter 2.
the release of horizontal load. Zone II is a radial shear zone where the extension direction changes from vertical to horizontal. The overburden soil located above the tip of the caisson wall (Zone A) does have shear resistance, but this is neglected in the plasticity theory solutions (a conservative assumption).

Considering force equilibrium at failure individually on both the soil plug and the caisson, the breakout force can be determined:

\[ F_b = W_a' + W_s' + F_{esf} - F_{reb} \]  

(6.7)

where, 
- \( F_b \) = breakout force (ultimate capacity) 
- \( W_a' \) = buoyant anchor weight 
- \( W_s' \) = buoyant soil plug weight 
- \( F_{esf} \) = external wall skin friction 
- \( F_{reb} \) = reverse end bearing

An important assumption in this analysis is that the soil plug travels upward with the caisson, and therefore the internal skin friction does not contribute to the breakout force. In the CET cell design, the model caisson was not instrumented to measure internal skin friction to evaluate this assumption. However, a soil plug was found adhering to the cap following complete caisson extraction from the clay cake in each CET test, which indicates that the soil plug traveled with the caisson during pullout.

Some researchers (Byrne and Finn, 1977; Clukey and Morrison, 1993) incorporate the soil plug weight term (\( W_s' \)) into the reverse end bearing term (\( F_{reb} \)). In the following analysis of the ultimate capacity in the CET cell, the end bearing incorporates the plug weight term, while the buoyant anchor weight (\( W_a' < 0.1 \) kg) is neglected to yield the following equation:

\[ F_b = F_{esf} + F_{reb} \]  

(6.8)

12This is identical to equation 2.34a in Chapter 2.
6.3.3 Wall Interface Friction

The calculation of the wall resistance term $F_{esf}$ from equation 6.8 is based on three assumptions: 1) the brass wall is relatively inextensible, such that the total resistance is provided by $f_s$, the limiting skin friction, 2) the skin friction $f_s$ is equivalent to the undrained shear strength of the soil adjacent to the soil/wall interface, and 3) the clay fails in a mode of undrained direct simple shear, $s_{udSS}$. All of the CET tests were conducted using $K_o$-normally consolidated (to $\sigma'_{vc}=0.75$ ksc) resedimented Boston Blue Clay, which has an undrained strength ratio, $s_{udSS}/\sigma'_{vc}=0.205$ (Ladd, 1991). The resulting equation for external skin friction during pullout in the CET cell is:

$$F_{esf} = f_s A_e$$

(6.9)

where, $f_s$ = average limiting skin friction ($=s_{udSS}$)

$A_e$ = penetrated external wall area ($=\pi D_o L$)

$D_o$ = outside diameter of caisson wall ($=5.1$ cm)

$L$ = wall penetration depth ($=5.1-\delta_w$)

$\delta_w$ = wall displacement at peak caisson resistance

Table 6.2 lists the calculated external skin friction $F_{esf}$ and the measured mobilized wall force $F_w$ at caisson capacity ($F_{tot}=$maximum) for the three most reliable 'virgin' pullout tests (CET5,6,8). The average computed value $F_{esf}=11.95\pm0.10$ kg compares very well with the measured average $F_w=13.2\pm2.1$ kg ($F_{esf}\geq90\%F_w$). The lower estimate may be due to actual inside wall shear tractions that are not accounted for in the external friction equation 6.9.
6.3.4 Reverse End Bearing

The most general form of the reverse bearing term $F_{reb}$ is based on conventional bearing capacity (Vesic, 1973) and was reported by Rapoport and Young (1977) as follows:

$$F_{reb} = A_b (S_c c_{ub} N_c + S_q \gamma D_0 N_q - S_q \gamma L N_q)$$  \hspace{1cm} (6.10)^{13}

where, $A_b$ = total anchor base area, $(\pi D_o^2)/4$

$S_c, S_q, S_q$ = empirical shape factors to convert the solutions from plane strain to axisymmetric geometry

$N_c, N_q, N_q$ = bearing capacity factors

$c_{ub}$ = cohesive strength at pile base

$\gamma$ = buoyant unit weight of soil

$D_o$ = outer pile diameter

$L$ = embedment depth

Because undrained conditions prevail in the monotonic pullout tests in the CET cell, a $\phi=0$ analysis is appropriate (i.e., $N_{\gamma}=0$, $N_{q}=1$), which reduces the general uplift capacity equation to the following form:

$$F_{reb} = A_b (S_c c_{ub} N_c - S_q \gamma L)$$  \hspace{1cm} (6.11)

This can be modified further by assuming the following: 1) the resistance reduction provided by the soil overburden above the anchor bottom ($-A_b S_q \gamma L$) is canceled by the effective weight of the soil plug, and 2) the shearing resistance of the soil above the caisson rim (Zone A in Figure 6.25) contributes to the end bearing resistance through a depth factor $L_c$ applied to the cohesive resistance. These modifications yield:

---

^13This is identical to equation 2.36 in Chapter 2.
\[ F_{reb} = A_b S_c c_{ub} N_c L_c \] (6.12)

This equation is identical to that used by Byrne and Finn (1977) for predicting uplift capacity of 1g skirted anchors embedded in clay and by Clukey and Morrison (1993) for predicting capacity of 100g suction anchors in clay.

Considering the model caisson geometry in the CET cell and conventional assumptions regarding the strength of the clay model, the reverse end bearing can be computed for the CET tests. The cylindrical geometry of the model caisson in the CET cell yields a base cross-sectional area \( A_b = 20.27 \text{ cm}^2 \) and a shape factor \( S_c = 1.2 \). Since the caisson applies an immediate tensile load to the soil directly below the caisson, the uplift capacity is controlled by the undrained shear strength in a triaxial extension mode of failure. For \( K_0 \)-normally consolidated RBBC, the undrained strength ratio is \( s_{u_{TE}}/\sigma'_{vc} = 0.16 \) (Sheahan, 1991). All RBBC cakes in the CET tests were normally consolidated to \( \sigma'_{vc} = 0.75 \text{ ksc} \), which yields \( s_{u_{TE}} = 0.12 \text{ ksc} \). The RBBC cake is homogeneous with a uniform shear strength, which yields a bearing capacity factor \( N_c = 5.14 (\pi + 2) \). Based on an expression originally proposed by Kulhawy et al. (1983) for the uplift of transmission tower footings, the depth factor is:

\[ L_c = 1 + 0.33 \tan^{-1}(D_o/L) \] (6.13)

Using the model caisson diameter \( D_o = 5.08 \text{ cm} \) and a penetrated length \( L = 5.1 \text{ cm} \), the depth factor is \( L_c = 1.26 \). Given these five terms, the best estimate of the reverse end bearing capacity for the model caisson in the CET cell is:

\[ F_{reb} = (20.27 \text{ cm}^2)(1.2)(0.12 \text{ ksc})(5.14)(1.26) = 18.9 \text{ kg} \]
As listed in Table 6.2, the measured cap force at the displacement corresponding to total capacity ranged from $F_c = -8.75$ to $-11.3$ and averaged $F_c = -9.65 \pm 1.43$ for the three most reliable virgin pullout tests CET5, 6, and 8. This average measured cap force $F_c$ is approximately 50% of the estimated reverse end bearing ($F_{reb} = 18.9$ kg). Such a large discrepancy most likely is due to uncertainty in the assumptions (i.e., the empirical factors $S_c, L_c$) used to develop the reverse end bearing equation. In addition, the undrained strength used in the end bearing calculation ($s_{uTE}/\sigma'_{vc} = 0.16$) is based on triaxial extension loading of a $K_o$-normally consolidated undisturbed clay specimen. As discussed in section 6.1, the suction installation process severely disturbs the soil plug and causes a large decrease in the effective stress (see Figure 6.8). It is likely that installation also disturbed a significant volume of soil below the caisson wall tip, such that the undrained strength in this region would be reduced.

The combined best estimate of the uplift capacity in the CET tests, $F_b = -30.8$ kg ($= F_{est} + F_{reb}$), overpredicts the average measured total capacity $F_{tot} = -22.9 \pm 0.9$ kg ($= F_w + F_c$) by 35%. While the skin friction calculation ($F_{est}$) contains some uncertainty, the reverse end bearing term ($F_{reb}$) accounts for most of the difference between the measured and computed capacities.

6.3.5 Numerical Limit Analysis

The CET monotonic uplift capacity also can be calculated using numerical limit analysis (Ukritchon, 1995; Ukritchon, 1996). These computations are based on finite element discretization and linear programming optimization methods, and yield rigorous lower and upper bounds on the true collapse load (after Sloan, 1988a; Sloan and Kleeman, 1994). The current programs are restricted to plane strain problems with an isotropic Tresca failure criterion for the clay ($s_u = (\sigma_1 - \sigma_2)/2$), but include beam and joint elements which are capable of simulating failure conditions in a caisson structure. This analysis assumes no transfer of shear force or bending moment at the junction of the caisson cap
and wall. The boundary conditions represent CET cell conditions prior to pullout: the cell base and sidewalls are rigid, while a surcharge of $\sigma_v = 0.75$ ksc is placed on the flexible soil surface outside the caisson. To approximate the effect of installation disturbance, the soil plug located within the caisson is assigned zero strength. The remaining soil exterior to the caisson is assigned an average undrained shear strength equivalent, $s_u = s_uDSS = 0.205(0.75) = 0.15$ ksc, with full adhesion permitted along the soil-structure surface.

By maximizing the total vertical force acting on the cap and wall, the lower bound caisson capacity computation yields $Q_L = -0.26$ kg/mm width. After scaling to account for the axisymmetric geometry, the lower bound force is $F_b(L) = -15.6$ kg. The upper bound collapse load is computed from the principle of virtual work, assuming that the caisson cap and wall are rigid and displace together vertically. The program minimizes the total vertical force on the caisson and generates an upper bound collapse load, $Q_U = -0.31$ kg/mm width. After scaling, the upper bound force becomes $F_b(U) = -18.9$ kg. Thus, the true collapse load is bounded by $-15.6 \leq F_b \leq -18.9$ kg (i.e., a 20% range of uncertainty). Hence the numerical limit analysis predicts collapse loads that are 17 to 32% lower than the measured capacity ($F_{tot} = -22.9$ kg) of the CET model caissons.

6.3.6 Effect of Re-Equilibration

Prototype suction anchors mobilize wall skin friction and underbase suction to resist large TLP tensile loads during extreme storm conditions. After the storm passes, however, the anchor is allowed to re-equilibrate with the surrounding soil under calm sea conditions (i.e., tensile loads are resisted by anchor weight and ballast alone). Additional storms then, once again, apply large tensile loads to the suction caisson. This loading-equilibration-loading sequence was simulated in the CET testing program to study the effect of the 're-equilibration phase' upon ultimate capacity.

This section compares the results of the second monotonic pullout stage (MP2) to those from 'virgin' pullout (MP1). In five tests (CET5-6, 12-14), the model caisson was
monotonically pulled to capacity following a re-equilibration stage\textsuperscript{14} of at least 24 hours, during which a compressive load of $F_{\text{tot}}=15.2$ kg was maintained on the caisson. In CET9, the MP2 proceeded immediately following the sustained loading stage (i.e., without re-equilibration). The control conditions during monotonic pullout 2 were identical to those during virgin pullout: a constant caisson displacement rate of $v_w=-0.03$ cm/min with no relative displacement between the cap and wall. The results discussed below illustrate the strengthening effect of re-equilibrating the caisson.

Figure 6.26 shows the caisson force components for early displacement ($\delta_w=0.0-0.02$ cm) and for displacement to capacity ($\delta_w=0.0-0.4$ cm) for the second monotonic pullout (MP2; CET6,9,12-14) and for the best estimate of the 'virgin' pullout (MP1). Total capacity for the second pullout for those tests that were re-equilibrated (CET6,12-14) ranges from $F_{\text{tot}}=-25.2$ to -30.5 kg and averages $F_{\text{tot}}=-27.2\pm2.4$ kg, which is 19\% higher than the virgin pullout average $F_{\text{tot}}=-22.9$ kg. The one test CET9 that was not re-equilibrated reached a capacity of only $F_{\text{tot}}=-20.9$ kg, which is 7\% lower than the lower bound value ($F_{\text{tot}}=-22.3$ kg) from the virgin 'best estimate' range. Initial stiffness behavior ($\Delta F_{\text{tot}}/\delta_w$ for $\delta_w=0.0$ to 0.002 cm) for MP2 is similar to that for MP1.

An increase in wall resistance is responsible for the total capacity increase for the re-equilibrated tests. At the point of peak total load during pullout 2, the wall force from the re-equilibrated tests ranges from $F_w=-15.2$ to -19.4 kg and averages $F_w=-17.0\pm1.7$ kg. This represents a 29\% increase over the average virgin wall resistance $F_w=-13.2\pm2.1$ kg. The wall resistance during MP2 for CET9, which was not re-equilibrated, reaches $F_w=-11.3$ kg, which conforms to the lower bound for virgin pullout ($F_w=-11.0$ kg).

The cap mobilizes approximately the same amount of resistance during monotonic pullout 2 as that measured during monotonic pullout 1 (virgin pullout). As shown in Figure 6.26, the MP2 cap resistance at the displacement corresponding to total capacity

\textsuperscript{14}The re-equilibration stage followed the initial tensile loading, which consisted of monotonic pullout to capacity (CET5-6) or partial monotonic pullout followed by sustained tensile load (CET112-14) (see Table 5.2).
ranges from \( F_c = -7.8 \) to \( -13.7 \) kg with an average \( F_c = -10.1 \pm 2.2 \) kg, which is comparable to the virgin cap resistance range \( F_c = -8.9 \) to \( -11.3 \) kg and average \( F_c = -9.7 \pm 1.4 \) kg.

Measurements of excess pore pressure and soil surface displacement during pullout 2 are very consistent with the behavior described for the initial 'virgin' pullout stage (see section 5.8.2-5.8.3). Generally, large negative excess pore pressures (\( \Delta u = -0.15 \) to \(-0.9 \) ksc) are generated within the soil plug, while very small excess pressures (\( \Delta u < -0.2 \) ksc) develop outside the caisson (see Figure 5.74). The soil surface exterior to the caisson initially heaves slightly at location near the caisson (\( \delta_s < 0.003 \) cm at \( r = 1.7, 2.7 \) cm), but by the end of pullout (\( \delta_w \approx 0.4 \) cm) the soil has compressed very little at all radial locations (\( \delta_s < 0.02 \) cm).

As listed in Table 6.2, reverse end bearing (\( F_{reb} \)) and wall skin friction (\( F_{esf} \)) values were computed for the tests subject to monotonic pullout 2. Note that these estimates are nearly the same as those computed for virgin pullout because the underlying assumptions in the equations are the same. For those tests that included a re-equilibration stage (CET6,12-14), the average computed value for wall friction is \( F_{esf} = 12.0 \pm 0.2 \) kg, which is only 70% of the average measured value \( F_w = 17.0 \pm 1.7 \) kg (for virgin pullout, \( F_{esf} > 90\% F_w \)). Clearly, the skin friction equation does not account for the increased strength (due to radial consolidation during re-equilibration) of the zone of clay around the exterior caisson wall. As for the virgin pullout, the reverse end bearing \( F_{reb} = 18.9 \) kg overpredicts the maximum cap force \( F_c = 10.2 \pm 2.5 \) kg due to uncertainty in the end bearing equation assumptions.

6.3.7 Effect of Pullout Rate

Prototype suction caissons are subject to tensile cyclic loading sequences that vary in frequency, magnitude, and duration according to the particular storm characteristics. Although the CET testing program did not simulate cyclic loading histories, some tests did
include monotonic tensile loading at two different rates of withdrawal. These tests provide some insight into the effect of pullout rate on monotonic uplift capacity.

A total of seven CET tests incorporated a fast rate \( v_w = -0.3 \text{ cm/min} \) pullout phase following the standard rate \( v_w = -0.03 \text{ cm/min} \) monotonic pullout. After reaching capacity during 'virgin' monotonic pullout (MP1) at the standard rate in CET3,4, and 8, the caisson was pulled at the faster rate to complete extraction. This same procedure was employed during the second monotonic pullout phase (MP2) for CET7,12-14\(^{15} \). It is not possible to make a one-to-one comparison of tensile capacity at the two loading rates because 'virgin' fast rate pullout capacity tests were not attempted. However, the larger mobilization of total caisson force and soil plug excess pore pressure during fast pullout does warrant some comment\(^{16} \).

In most of the tests, the higher imposed withdrawal rate mobilizes \( \Delta F_{tot} = -2 \text{ to } -3 \text{ kg} \) over the total force measured at the end of standard rate pullout (see Figures 5.39 and 5.72). This represents a total force increase of between 11 and 20\%, depending on the test. The strain softening behavior that follows the fast rate peak load is similar to that for the standard rate pullout; the total tensile load declines at a rate of \( \Delta F_{tot}/\Delta \delta_w \approx -5 \text{ kg/cm} \). Examination of the component force records reveals that the larger caisson capacity can be attributed to an increased cap resistance.

Pore pressure measurements indicate that increasing the pullout rate induces larger negative excess pore pressure within the soil plug. At all locations within the soil plug (beneath cap, mid-plug depth at \( r = 0.0 \text{ cm} \), mid-plug depth at \( r = 1.8 \text{ cm} \)), the excess pore pressure increases by \( \Delta u = -0.2 \text{ to } -0.25 \text{ ksc} \) over values recorded at the end of standard rate pullout (see Figures 5.75-5.76).

\(^{15}\)Table 5.2 lists the loading phases for each CET test.

\(^{16}\)Sections 5.5 and 5.8 reviewed in detail the suction caisson behavior during fast rate pullout portion of monotonic pullout 1 and 2, respectively.
There is no particular rate effect regarding soil surface displacement, but large settlement near the wall ($\delta_s=0.2$ cm) indicate that the soil mass at this location is participating in the general failure as the caisson moves at a faster rate (see Figure 5.77).

6.4 SUSTAINED TENSILE LOADING

Critical to the design of a TLP suction anchor is an understanding of the anchor response to sustained tensile loads. Heavy storms lasting hours, even days, apply large cyclic tensile loads to the anchor, which initially must rely on underbase suction and caisson wall skin friction to resist such loads. With time, the negative excess pore pressure generated by the suction will dissipate as a result of pore fluid drainage in the surrounding soil. Hence, the underbase suction component is relieved as load is transferred to the wall. The CET testing program simulated sustained tensile loading of suction anchors with a series of 6 single stage and multi-stage sustained load tests on the two-component model caisson. Section 6.4.1 summarizes the measured response to sustained load in terms of caisson force and displacement, pore pressure, and soil surface displacement. Section 6.4.2 compares the long-term capacity of the suction caisson with the ultimate capacity.

6.4.1 Summary of Sustained Tensile Loading Behavior

The sustained tensile load CET tests can be divided into three groups: 1) low-level single-stage load leading to stable conditions (CET9), 2) high-level single-stage load leading to caisson failure (CET10), and 3) multi-stage load ultimately leading to caisson failure (CET11-14). Prior to the application of tensile loading in all tests, the caisson carried a compressive load of $F_{tot}=15.2$ kg, statically equivalent to the in-situ consolidation stress, $\sigma_{vc}=0.75$ ksc. In the single-stage load tests, the caisson was pulled
at a constant displacement rate (MP1; \( v_w = -0.3 \, \text{cm/min} \)) to a specified tensile force \( F_{tot} \), which was held constant until either the pore pressures equilibrated (\( F_{tot} = -2.2 \, \text{kg, CET9} \)) or the caisson failed (\( F_{tot} = -11.4 \, \text{kg, CET10} \)). The caisson acted as a monolithic unit, as no relative displacement was allowed between the cap and wall (\( \Delta \delta_w = \Delta \delta_c \)). In the multi-stage tests (CET11-14), the tensile loads were maintained for 24 hours before applying an additional increment of tensile load (typically \( \Delta F_{tot} = -2 \, \text{kg} \)). This process was repeated until the caisson failed.

In order to illustrate typical tensile loading behavior, Figures 6.27 and 6.28 show the caisson forces and displacement, excess pore pressures, and soil surface displacements for the two single stage tests (CET9-10). Figures 6.29-6.32 show the behavior for each of the four stages in the multi-stage test CET11.

The small tensile load (\( F_{tot} = -2.2 \, \text{kg} \)) applied to the caisson in the single-stage test CET9 causes a stable response (Figure 6.27). Initially, the cap carries slightly more than the applied load (\( F_c = -3 \, \text{kg} \)), but rapidly sheds this tensile load (\( F_c = 0 \, \text{kg at t=10 minutes} \)), and then acquires a small compressive load \( F_c = 1.4 \, \text{kg} \) by the end of the stage. Meanwhile the wall starts out with a small compressive load of \( F_w = 1.8 \, \text{kg} \), but carries slightly more than the applied tensile load (\( F_w = -3.6 \, \text{kg} \)) within 8 hours. Both the wall and cap displace upward a very small amount, reaching an equilibrium displacement, \( \delta_w = 0.012 \, \text{cm} \) in approximately 100 minutes. Application of the tensile load initially generates excess pore pressure ranging from \( \Delta u = -0.15 \) to \(-0.28 \, \text{ksc within the soil plug, but this dissipates within 100 minutes as the caisson equilibrates under the tensile load. Soil surface displacements outside the caisson are subtle. Near the caisson (S1,S2; \( r = 4.2, 5.2 \, \text{cm} \)) tensile loading initially causes the soil to heave slightly (\( \delta_s < 0.002 \, \text{cm} \)). By the end of the sustained load stage, the entire soil surface is settling at a rate consistent with secondary compression. (\( \delta_s > 0.005 \, \text{cm} \)).

A much higher applied tensile load, \( F_{tot} = -11.4 \, \text{kg} \), leads to failure in the other single-stage test CET10. The cap initially carries 73% of this tensile load (at t=1 minute;
$F_c = -8.3 \text{ kg, } F_w = -3.1 \text{ kg}$. Within 40 minutes the cap loses 5 kg of tension, but cannot shed the remaining tensile load, $F_c = -3.5 \text{ kg}$ (30% of the total tension). The wall increases its share of the tensile load from $F_w = -3.1 \text{ kg}$ to $-7.9 \text{ kg}$ during the first 40 minutes of loading and maintains nearly -8 kg of tension for the rest of the test. A rapidly increasing upward caisson displacement is the earliest sign of failure in CET10. After $t = 0.4 \text{ minutes}$ (24 seconds), the displacement rate increases with the log of time; caisson displacement reaches $\delta_w = 0.11 \text{ cm}$ (76% of wall thickness) after 40 minutes. Another sign of failure is the maintenance of negative excess pore pressure as the caisson withdraws. The large applied tensile load initially generates $\Delta u = -0.5$ to -0.6 ksc within the soil plug. Approximately half of this dissipates within 40 minutes, but $\Delta u = -0.2$ to -0.3 ksc remains for the duration of loading as the mechanisms of dissipation and generation (due to caisson displacement) become balanced. The soil surface at locations close to the caisson (S1, S2; $r = 4.2, 5.2 \text{ cm}$) initially heaves $\delta_s < 0.001 \text{ cm}$, as it does in low-level tensile loading (CET9). However, the high-level tensile load in CET10 causes rapid upward caisson displacement, which in turn draws in soil exterior to the caisson, as measured by the high surface compression rate after approximately 40 minutes.

Results from the 4 multi-stage sustained load tests (CET11-14) show consistent behavior. The initial stage resembles behavior depicted for the low-level single-stage test CET9. As shown in Figure 6.29 for CET11, the cap carries a tensile load ($F_c = -7.5 \text{ kg}$) after the application of $F_{\text{tot}} = -2.9 \text{ kg}$, which generates a sizable amount of negative excess pore pressure ($\Delta u = -0.32 \text{ ksc}$) within the soil plug. However, as the cap sheds all of the tension within 15 minutes, the excess pore pressure dissipates completely. Results from the first stable stage of the other sustained load tests (CET9,12-14; Figures 5.58-5.60, 6.27) show that the negative excess pore pressure dissipates within approximately 100 minutes. This dissipation rate is one order of magnitude greater than that during post-installation equilibration ($t \approx 1000 \text{ minutes};$ see section 6.2.1)
As for CET9, caisson movement during the initial stage of the multi-stage test CET11 is minimal (δ_w<0.02 cm). Additional increments of tension (ΔF_{tot}≈-2 kg) are carried by the wall, which does not significantly affect pore pressures, caisson displacement, or soil surface displacement. Figures 6.30-6.31 show this intermediate stage load behavior for F_{tot}=-4.9 and -6.9 kg in CET11. The final stage (4, F_{tot}=-8.9 kg) leads to failure and causes behavior similar to that recorded for the high-level single-stage test CET10. As shown in Figure 6.32, the cap acquires the -2 kg of applied tensile load and does not shed this tension as the caisson fails. After t=10 minutes, it is clear that the caisson is displacing at an increasing rate with log of time and the soil plug maintains a negative excess pore pressure just below Δu≈-0.1 ksc. The soil surface shows no sign of failure until much later (t>500 minutes), when it rapidly compresses as exterior soil is drawn toward the caisson interior. Similar multi-stage behavior to CET11 was recorded for CET12-14 and was described in section 5.6. The one difference among the multi-stage tests is that the level of load at which failure occurs ranges from F_{tot}=-9 to -13 kg, which indicates some uncertainty in the long-term capacity of the CET model caissons.

6.4.2 Drained Versus Undrained Capacity

By plotting the successive stages of the sustained tensile load tests in terms of force versus displacement, a comparison can be made between drained and undrained pullout behavior. Figure 6.33 shows the cap, wall, and total force versus displacement (δ_w=0.0 to 0.08 cm) for the six sustained load tests and the best estimate of the undrained pullout tests. The maximum drained capacity ranges from F_{tot}=-8.9 to -12.9 kg, which is 39 to 56% of the undrained capacity average (F_{tot}=-23 kg). Drained capacity occurs at a displacement ranging from δ_w=0.015 to 0.055 cm (10 to 38% of the wall thickness), which is much earlier than in the undrained tests (δ_w=0.25 cm at capacity). However, the displacement at peak drained total capacity is consistent with the displacement at peak drained wall capacity.
Nearly all of the drained capacity is supplied by the wall resistance. In three multi-stage tests (CET12-14), the limiting wall resistance ranges $F_w = -10$ to $-14$ kg, which is within 10% of the total capacity. This wall resistance range also is consistent with the best estimate of wall force ($F_w = -11.4$ to $-14.9$ kg) during undrained pullout.

The plot of cap force versus caisson displacement (Figure 6.33) shows that the cap does not contribute to the overall caisson resistance during drained pullout. During the failure stage of three multi-stage tests, the cap resistance is $F_c = 0.0 \pm 1$ kg. Only the high-level single stage test (CET10; $F_{tot} = -11.4$ kg) generates and maintains significant cap resistance ($F_c = -4$ kg at $\delta_w = 0.06$ cm).
<table>
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<tr>
<th>Test</th>
<th>$F_0$ (kg)</th>
<th>$q_{ult}$ (ksc)</th>
<th>$(q_{ult} - \sigma_{vc})/\sigma'_{vc}$</th>
<th>$N_c$</th>
<th>$f_w$ (kg/cm)</th>
<th>$f_s$ (ksc)</th>
<th>$\beta$</th>
<th>$\alpha$</th>
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**KEY**

- $F_0$ = wall force intercept
- $q_{ult}$ = wall tip capacity
- $(q_{ult} - \sigma_{vc})/\sigma'_{vc}$ = norm. net tip resist.
- $N_c$ = bearing capacity factor
- $f_w$ = wall force gradient
- $f_s$ = average skin friction
- $\alpha, \beta$ = skin friction factors
- $A_w$ = wall tip area (=2.248cm²)
- $\sigma_{vc}, \sigma'_{vc}$ = vert. consolidation total & effective stress (=0.75 ksc)
- $s_u/T/\sigma'_{vc}$ = 0.32 (Sheahan, 1991)
- $s_u/DDSS/\sigma'_{vc}$ = 0.205 (Ladd, 1990)
- $r_i, r_o$ = inside, outside wall radius (=2.395, 2.54 cm)

**EQUATIONS**

\[
\frac{q_{ult} - \sigma_{vc}}{\sigma'_{vc}} = N_c \frac{s_u}{\sigma'_{vc}} \tag{6.2}
\]

\[
q_{ult} = \frac{F_0}{A_w} \tag{6.3}
\]

\[
f_s = \frac{f'_w}{2\pi(r_i + r_o)} \tag{6.4}
\]

\[
\beta = \frac{f_s}{\sigma'_{vc}} \tag{6.5}
\]

\[
\alpha = \frac{f_s}{s_u/DDSS} \tag{6.6}
\]

**Table 6.1** Wall Tip Capacity and Skin Friction Values for Suction Installed Tests (CET4, 8, 9, 11-12) and Jacked-in Test CET13
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<td></td>
<td>$\delta_w$ (cm)</td>
<td>$F_w$ (kg)</td>
</tr>
<tr>
<td>CET6</td>
<td>0.325</td>
<td>-15.2</td>
</tr>
<tr>
<td>CET9</td>
<td>0.181</td>
<td>-11.3</td>
</tr>
<tr>
<td>CET12</td>
<td>0.108</td>
<td>-16.6</td>
</tr>
<tr>
<td>CET13</td>
<td>0.260</td>
<td>-19.4</td>
</tr>
<tr>
<td>CET14</td>
<td>0.224</td>
<td>-16.9</td>
</tr>
<tr>
<td>Mean:</td>
<td>0.229±</td>
<td>-17.0±2.1</td>
</tr>
<tr>
<td>SD</td>
<td>0.09</td>
<td></td>
</tr>
</tbody>
</table>

**KEY**

- $\delta_w = \text{wall displacement at total capacity}$
- $F_w = \text{measured wall force}$
- $\tau_s/\sigma'_vc = \text{measured norm. shear stress}$
- $F_c = \text{measured cap force}$
- $F_{tot} = \text{measured total capacity}$
- $F_{esf} = \text{calculated skin friction}$
- $F_{reb} = \text{calculated reverse end bearing}$
- $F_b = \text{breakout resistance}$
- $f'_s = \text{limiting skin friction}$
- $L_c = 1+0.33\tan^{-1}(D_o/L) = 1.26$

**EQUATIONS**

- $\tau_s/\sigma'_vc = F_w/(A_e\sigma'_vc)$
- $F_{esf} = f'_sA_e$
- $f'_s/\sigma'_vc = s_{uDSS}/\sigma'_vc$
- $F_{reb} = A_bS_cC_{ub}N_cL_c$

*KEY*

- $A_e = \text{external wall area} = \tau D_o (L-\delta_w)$
- $L = \text{wall penetration} (= 5.1 \text{ cm})$
- $D_o = \text{outside wall diameter} = 5.08 \text{ cm}$
- $A_b = \text{base cross-sect. area} (=20.27\text{cm}^2)$
- $\sigma'_vc = \text{vert. cons. effective stress} (=0.75 \text{ ksc})$
- $C_{ub} = \text{base undrained shear strength} (=s_{uTE})$
- $s_{uTE}/\sigma'_vc = 0.16 \text{ (Sheahan, 1991)}$
- $s_{uDSS}/\sigma'_vc = 0.205 \text{ (Ladd, 1990)}$
- $N_c = \text{bearing capacity factor} = 5.14$
- $S_c = \text{shape factor} = 1.2$

Table 6.2 Calculated External Skin Friction and Reverse End Bearing versus Measured Wall and Cap Resistance During Monotonic Pullout 1 and 2
Figure 6.1  Caisson Forces During Suction Installation (CET4,8-9,11-12) and Constant Cap Force Installation (CET13)
Figure 6.2  Excess Pore Pressure During Suction Installation (CET4,8-9,11-12) and Constant Cap Force Installation (CET13)
Figure 6.3  Cap Displacement During Suction Installation (CET4,8-9,11-12) and Constant Cap Force Installation (CET13)
Figure 6.4  Clay Surface Displacement During Suction Installation (CET4, 8-9, 11-12) and Constant Cap Force Installation (CET13)
Figure 6.5  Interpretation of Wall Penetration Resistance

a) Most Representative Wall Force Behavior During Penetration

b) Wall Force Regression Lines During Deep Penetration to Determine Wall Tip Capacity and Wall Skin Friction
Figure 6.6  Total Stress, Excess Pore Pressure, and Estimated Effective Stress Beneath Cap During Installation (CET4, 8-9, 11-13)
Figure 6.7 Equivalent Plug Axial Strain versus Wall Tip Penetration During Installation (CET4, 8-9, 11-13)
Figure 6.8 Normalized Vertical Effective Stress versus Axial Strain: Comparison of CET Installation to Triaxial Extension Unloading (CK₀UTXE-Unloading)
Figure 6.9 Comparison of Predicted and Measured Surface Heave in CET at End of Installation (after Whittle et al., 1996)
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Figure 6.11  Caisson Forces During Post-Installation Equilibration (CET3,8-9,11-13)
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Figure 6.14  Clay Surface Displacement During Post-Installation Equilibration  
(CET3-6,8-13)
Figure 6.15  Caisson Forces, Excess Pore Pressure Beneath Cap, Cap Displacement, and Soil Surface Displacement (S2, r=5.2 cm) for Suction Installation (Average) and Constant Cap Force Installation (CET13)
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Figure 6.17 Excess Pore Pressure During Re-Equilibration and Post-Installation Equilibration
Figure 6.18  Wall and Cap Settlement During Re-Equilibration and Post-Installation Equilibration
Figure 6.19  Clay Surface Displacement During Re-Equilibration and Post-Installation Equilibration
Figure 6.20  Caisson Forces During Monotonic Pullout 1 (CET4-6,8)
Figure 6.21  Caisson Forces During Early Displacement (0.0 to 0.02 cm) of Monotonic Pullout 1 (CET6,8-14)
Figure 6.22 Best Estimate of Caisson Forces During Monotonic Pullout 1
Figure 6.23  Excess Pore Pressure During Monotonic Pullout 1 (CET5-6,9-14)
Figure 6.24  Clay Surface Displacement During Monotonic Pullout 1 (CET4-6,8)
General Failure Mechanism

\[ \Delta F_{PS} + F_{ISF} = W'_{B} + F_{REB} \]

\[ F_{ISF} = (W'_{B} + F_{REB}) - \Delta F_{PS} \]

\[ F_{ESF} = F'_{REB} - \Delta F_{PS} \]

Where:

Figure 6.25 Reverse End Bearing Failure Mode and Derivation of Components of Capacity for Vertically Loaded Suction Caisson (after Clukey and Morrison, 1993)
Figure 6.26 Caisson Forces During Monotonic Pullout 2 versus Best Estimate During Monotonic Pullout 1
Figure 6.27  Caisson Forces, Cap and Soil Surface Displacement, and Excess Pore Pressure During Low-Level Single Stage Sustained Loading (CET9)
Figure 6.28  Caisson Forces, Cap and Soil Surface Displacement, and Excess Pore Pressure During High-Level Single Stage Sustained Loading (CET10)
Figure 6.29  Caisson Forces, Cap and Soil Surface Displacement, and Excess Pore Pressure During First Stage (SL1) of Sustained Loading (CET11)
Figure 6.30  Caisson Forces, Cap and Soil Surface Displacement, and Excess Pore Pressure During Second Stage (SL2) of Sustained Loading (CET11)
Figure 6.31  Caisson Forces, Cap and Soil Surface Displacement, and Excess Pore Pressure During Third Stage (SL3) of Sustained Loading (CET11)
Sustained Load
Stage 4
CET 11
$F_{tot} = -8.9 \text{ kg}$

Figure 6.32  Caisson Forces, Cap and Soil Surface Displacement, and Excess Pore Pressure During Fourth Stage (SL4) of Sustained Loading (CET11)
Figure 6.33  Caisson Forces versus Wall Displacement for Sustained Loading, Best Estimate of Monotonic Pullout 1, and Monotonic Pullout 2 (CET9)
CHAPTER 7
SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

This chapter summarizes the suction caisson research presented in this thesis, draws conclusions from this study, and presents recommendations for future work. The material presented in this document represents the culmination of a four-year experimental effort to investigate the fundamental behavior of suction caissons, which are large-diameter piles, closed at the top, and installed using underbase suction. Suction caissons can support gravity-based platforms and tension leg platforms (TLP's), but this research focused on the TLP loading conditions. A new automated 1g laboratory device, the Caisson Element Test (CET) cell, was developed to simulate successive phases of installation by underbase suction, set-up, and axial tensile loading of a miniature pile in cohesive soil.

Section 7.1 presents a summary of each of the integral chapters (2-6) in this thesis. Section 7.2 lists conclusions based on the development and implementation of the CET apparatus and the results of the suction caisson testing program. Finally, section 7.3 provides recommendations for further modification of the CET apparatus and additional CET testing programs to expand the current state of fundamental knowledge regarding the behavior of suction caissons.
7.1 SUMMARY

Each chapter in this thesis presents an important segment of this experimental suction caisson research project. This section summarizes the most important information contained within each chapter.

7.1.1 Background

The background chapter (2) first presented a general description of suction caisson foundations that are used for tension leg platforms and discussed the relevant geotechnical issues. The bulk of the chapter reviewed previous experimental research related to suction caissons, prior experimental suction caisson studies, and field applications. A final section culled together the significant conclusions regarding suction caisson installation, set-up, and tensile loading from previous research and applications.

At the time of this publication, only two prototype TLP’s had employed suction anchors: the Snorre and Heidrun TLP’s located in the North Sea. Both structures are located in deep water (310 and 345 m, respectively), which is particularly suitable for suction caisson installation. After penetrating under self-weight, the caisson penetrates to depth by the differential pressure across the lid caused from pumping water from within the caisson. The Snorre and Heidrun TLP designs share two significant geometric traits: 1) large diameters (Snorre D₀=17 m, Heidrun D₀=9 m) and 2) small embedment ratios L/D₀=0.7, 0.5 (Snorre and Heidrun, respectively). The single caissons are arranged in four multicellular concrete units (3 cells per Snorre unit, 19 cells per Heidrun unit), and each unit is located at the corner of a square plan area (see section 2.4 and Figures 2.29,2.34 for details).

The life of a suction caisson anchor can be divided into three separate phases: installation, equilibration (set-up), and tensile loading. The installation phase consists of caisson penetration by self-weight and underbase suction. This is followed by an
equilibration phase, during which the caisson is allowed to equilibrate with the surrounding soil as the installation excess pore pressures dissipate. The final phase is platform loading. Calm sea conditions apply a lateral and vertical load to the buoyant platform, which in turn applies an eccentric cyclic and static tensile load to the suction anchor. The caisson resists this load through self-weight and ballast (e.g., each Snorre concrete foundation unit with ballast weighs ~3500 metric tons). Extreme storm conditions apply large sustained tensile loads to the caisson (e.g., for a "100 year" design storm, the tensile load on each Snorre CFT varies between 0 and 14,200 metric tons). The caisson resists such additional load through wall skin friction and the suction that is developed beneath the lid.

The important geotechnical uncertainties inherent to suction anchor design can be classified into two groups: 1) universal problems and 2) problems classified according to the three phases in the life of a suction caisson. The following lists these two groups:

**Universal Problems**
1. Effect of caisson geometry and cell configuration
2. Effect of soil properties

**Classified Problems**

*Installation*
1. Prediction of penetration resistance
2. Effect of underbase suction on penetration resistance
3. Effect of temporary delays during penetration

*Equilibration*
1. Prediction of rate of dissipation of installation excess pore pressures
2. Prediction of changes in soil stress and properties around suction caisson
3. Prediction of caisson settlement during equilibration

*Loading*
1. Prediction of ultimate capacity and frictional resistance of soil for normal and extreme storm loading
2. Prediction of foundation movement during normal and extreme storm loading
3. Effect of installation disturbance
4. Effect of incomplete equilibration
5. Effect of loading rate
The review of prior experimental research contained in Chapter 2 focused on programs that clarified these geotechnical uncertainties.

Early research on objects embedded below the seafloor and objects on the surface of the seafloor was conducted for a variety of applications including deep sea submersibles, soil exploration devices, ship and floating object anchorages, and sunken ship salvaging. Certain aspects of this research apply directly to the tensile loading phase of the suction caisson foundation, and laid the groundwork for later suction caisson research. Results from embedded object research indicate that much of the pullout capacity is derived from the resistance of the soil overlying the buried object, with an additional, sometimes significant, contribution from underbase suction (see section 2.2.1; Meyerhoff and Adams, 1968; Vesic, 1969, 1971; Bemben and Kupferman, 1975; Davie and Sutherland, 1977; Baba et al., 1989; Das and Singh, 1994; Shin et al., 1994). No predictive equations for the suction resistance were developed.

Several studies have been conducted on the pullout characteristics for objects that lie directly on soil or that are only partially embedded (Table 2.1 lists the important surface object research and section 2.2.2 describes the research). The experimental models developed to predict the uplift resistance of surface objects can be classified into empirical, local shear, and general shear models. In general, the empirical models correlate breakout force to time and the physical characteristics of the surface object (Muga, 1967; Lee, 1972, 1973; Das, 1991). The resulting empirical curves contain much scatter and are useful as a guideline for objects and soils similar to those studied.

In local shear models, the breakout force is derived by performing a force equilibrium analysis on the observed failure surface beneath the withdrawn object (Brown and Nacci, 1971; Helfrich et al., 1976; Wang et al., 1975, 1977, 1978). The breakout force comprises the sum of the buoyant anchor and soil plug weight, frictional force between the anchor surface and soil, and the vertical suction force acting on the failure
surface. All three programs employed active suction during testing, and the test results generally agreed well with the predictive equations.

The last class of surface object uplift prediction models is based on the assumption of a general soil failure mechanism similar to that for a bearing capacity failure, but with the direction of movement reversed (Finn and Byrne, 1972; Byrne and Finn, 1977; and Rapoport and Young, 1985). This research led to the concept of 'reverse bearing capacity'. The predictive breakout equation resembles the conventional bearing capacity equation with two exceptions: in reverse bearing, the object buoyant weight contributes additional resistance and the soil unit weight term reduces resistance (see equation 2.9). For undrained conditions (φ=0), the equation is reduced to one term; the breakout force is equivalent to the undrained shear strength of the soil below the caisson multiplied by empirical correction factors for caisson shape and depth, and by the bearing capacity $N_c$ (see equation 2.10).

Eleven 1g, multi-g, and field scale experimental programs on suction caissons were reviewed in section 2.3 (see Tables 2.2-2.4). Most of the 1g and multi-g programs studied a single cell anchor, while 3 of the 4 field tests studied multi-cell caissons. This research illuminated many of the geotechnical uncertainties associated with the phases in the life of a suction anchor. A detailed summary of the status of these geotechnical issues was provided in section 2.5.

Two major geotechnical problems connected with the installation phase are penetration resistance prediction and the effect of underbase suction. Prior research on both clays and sands acknowledges that resistance is provided by anchor skin friction and tip bearing, which do not explicitly account for the effect of underbase suction (see section 2.3.1, equation 2.24; Jones et al., 1994; Hogervorst, 1980; Renzi et al., 1991; Andréasson et al., 1988). Several projects have noticed that suction dramatically reduces the resistance to pile penetration in sandy soils. Suction causes enough upward seepage in the soil plug to reduce the effective stress to zero in this zone (liquefaction), and thus facilitate
penetration (Larsen, 1989; Pavlicek, 1992; Iskander et al., 1994; Jones et al., 1994; Hogervorst, 1980). One problem caused by underbase suction in sand is the formation of an excess soil plug, which is the soil inside the pile that is above the level of the adjacent soil outside the pile (see Figure 2.11). During the Gorm anchor installation, the problem was solved by using a jetting system to maintain the excess plug in a liquefied state until achieving the design depth (Senpere and Auvergne, 1982). The effect of underbase suction on piles penetrating clay is much less conclusive.

Important geotechnical issues during the equilibration stage include the prediction of the rate of pore pressure dissipation and the concurrent changes in soil stresses and properties around the suction caisson. None of the reviewed experimental studies investigated this particular phase.

The experimental programs primarily investigated the ultimate capacity and frictional resistance of soil subject to tensile loading from a suction anchor pile. Laboratory (1g) research on suction piles in sand at UT/Austin led to the conclusion that uplift capacity is composed of buoyant anchor and soil plug weight, external skin friction, and passive suction mobilized beneath the cap (see section 2.3.1, equation 2.8; Pavlicek, 1992; Iskander et al., 1993; Jones et al., 1994). The skin friction calculation in this capacity equation includes the effect of seepage. Most of the experimental research on suction anchors was conducted on clay and assumed a reverse end bearing failure mechanism during uplift. Taking force equilibrium on the anchor and soil at failure, the ultimate capacity comprises the anchor and soil plug weight, pile wall skin friction, and the reverse tip bearing (see equation 2.34; Fuglsang and Steensen-Bach, 1991; Renzi et al., 1992; Clukey and Morrison, 1993). There still is uncertainty regarding the accuracy of this mechanism. The following issues remain unresolved: the amount of internal skin friction that develops, the adhesion and cohesive strength values to use for external skin friction, and the proper shape and depth factors to include.
None of the experimental programs examined monotonic sustained loading in detail. However, for anchors that do not rely on wall friction resistance (very short skirts), Byrne and Finn (1977) suggested a method for predicting the time required until failure for an anchor subject to a tensile load that is less than the drained capacity (see section 2.2.2). They estimate the reduction in breakout force to be proportional to the average degree of pore pressure dissipation. They found a good comparison between the theoretical and observed breakout reduction.

Very few cyclic loading tests were conducted on 1g and multi-g suction anchor models, but 3 of the 4 field test models reviewed involved cyclic loads. For piles in sand, Jones et al. (1994) observed that when the model anchor was subject to small-amplitude stress-controlled cycling about an average tensile load below the ultimate drained capacity, the pile was not able to sustain any suction, which led to gradual pile withdrawal. For piles in clay, Renzi et al. (1991) observed very little movement when the pile was subjected to small-amplitude stress-controlled cycling about an average tensile load below the post-cyclic static capacity. Tests with different cyclic load histories were conducted in one centrifuge program (Clukey et al., 1995) and three field studies (see section 2.3.3; Tjelta et al., 1986; Andréasson et al., 1988; Dyvik et al., 1993; Andersen et al., 1993). In general the caisson resistance depends on the combination of static load, cyclic load history, and load inclination.

Load-displacement behavior during pullout was not the focus of experimental research, but some comments are warranted. Centrifuge and 1g model tests reveal that anchors in clay and sand reach ultimate capacity at significant displacements of between 20 to 30% of the pile diameter (Pavlicek, 1992; Jones et al., 1993; Fuglsang and Steensen-Bach, 1991). However, during post-peak pullout, anchors in sand rapidly lose much of their capacity, while those in clay retain a significant residual load. For the two field programs that included a static pullout test, the load was applied eccentrically, which caused the model to move both vertically and laterally. Results from the test conducted
on the 5-cell model in Lysaker clay (Andersen et al., 1993) indicate that at failure (defined as 0.004-0.006 radians rotation), the model had moved approximately 4 cm in both the vertical and lateral directions, which is approximately 4.4% of the cell diameter and 2.2% of the overall model width. The load-displacement behavior indicates that ultimate capacity would be reached at greater displacements (see section 2.3.3).

Only the 1g acrylic model tests in sand conducted at UT/Austin looked specifically at the effects of installation disturbance upon subsequent pullout behavior (Pavlicek, 1992). Increasing the amount of suction during installation liquefies and draws into the pile a greater amount of sand. This loosens the sand outside the pile, thus decreasing the vertical stress, which reduces the pile wall skin friction resistance to pullout (see section 2.3.1).

In three of the four programs that investigated rate effects, the ultimate capacity increased with increasing pullout rate (Byrne and Finn, 1977; Singh et al., 1994; Clukey and Morrison, 1993). All of the test programs were conducted on clay, and both Singh et al. (1994) and Clukey and Morrison (1993) claim that the strength increase is due to an increase in the suction component of resistance, which can be attributed to lower pore pressure dissipation.

7.1.2 Design of CET Cell

The Caisson Element Test (CET) apparatus was designed to simulate successive phases of installation, set-up, and axial tensile loading of a miniature caisson in a uniform, saturated 'element' of clay. The CET cell comprises five components: 1) the sealed test chamber, 2) the model caisson, 3) the driving system, 4) the control system, and 5) the instrumentation package. This section summarizes each of these components, the properties of the test material (RBBC), and the CET test procedures. A full detailed description was presented in Chapter 3.
The consolidation chamber is a rigid-walled, stainless steel cylinder (with inside diameter $D_i=30$ cm and height $H=25$ cm) that is attached to a 1.75 cm thick stainless steel baseplate (Figure 3.2). This chamber has been used at MIT since 1980 for the preparation of resedimented Boston Blue Clay (RBBC) samples, but for the CET experiments, the original chamber was modified to enable penetration of the model caisson into a consolidated clay sample and to include additional instrumentation. The original rigid cap (and attached piston) still is used for incremental consolidation with top and bottom drainage prior to CET testing. However, a new elaborate cylindrical slip tube assembly allows application of consolidation stresses to the soil, free drainage and non-uniform deformations of the soil surface, and caisson access to the soil (see Figure 3.4). Access ports were machined into the chamber sidewalls for total stress or pore pressure transducers, and ports were bored in the baseplate to allow installation of miniature pore pressure probes through the base and into the clay sample (Figure 3.3).

The centerpiece of the CET apparatus is the model caisson, which comprises two components, an outer caisson wall and an inner caisson cap (Figure 3.5). The wall is a brass Shelby tube with a blunt tip and has an outside diameter, $D_o=5.08$ cm, and a wall thickness, $t_w=0.145$ cm. The cap is a 5.7 cm tall brass cylinder with an outside diameter, $D_o=4.65$ cm, and a ceramic porous stone epoxied at the center of the cap base for pore pressure measurements. An O-ring around the perimeter of the cap separates the wall and cap. The two-piece design allows the simulation of installation by underbase suction by driving the wall at a constant rate, measuring the force increment picked up by the wall, and applying an equal but opposite force increment on the cap (see section 3.3.2). The unique design also enables independent control over both components, which allows measurement of cap and wall force contributions.

The third component of the CET cell is a three part mechanical driving system, which applies the total stress on the clay surface and moves the caisson wall and cap (Figure 3.7). Each part acts independently and is controlled automatically by the
computer control system. The total stress driving subsystem comprises a velocity controlled dc motor and reduction gear system operating an air pressure regulator, which maintains the air pressure (total stress) on the clay surface within the chamber cavity (Figure 3.8). The wall and cap driving systems are similar. They both use a dc electric motor to drive linear ball screw actuators via a reduction gear box. The actuator is connected to the drivetrain, which is comprised of a force transducer and various stainless steel components that are rigidly secured to the caisson component (Figures 3.9-3.10). One important ancillary component to the caisson driving systems is the reaction frame that maintains the vertical position of the caisson and reacts against the soil resistance encountered during testing. Actuator compression springs maintain tension on the actuator worm screw in order to avoid problems associated with system compliance or lashback.

The control system is comprised of hardware and software. The hardware consists of a 286 personal computer equipped with an analog to digital converter, and three dc motor driver interfaces (Figure 3.11). The computer receives input signals from the primary transducers (air pressure, cap force or displacement, and wall force or displacement) and sends signals to the driver interfaces to command motor operations. The system uses digital closed loop feedback control with at least a simple proportional gain adjustment algorithm (some software modules incorporate derivative or integral gain). The computer determines the status of each sensor under control and compares these values with the desired status (or target value) of each device. The difference between the two values is the error, which is divided by a gain factor (chosen to represent the actual stiffness of the device) to determine the magnitude of the signal to be sent to the motor driver. The motor then rotates in proportion to this signal. The system adjusts the wall and cap motors continuously throughout each phase of the test, while the system incrementally adjusts the air pressure motor in one second bursts.
The control software consists of three programs that are written in the BASIC programming language (Figure 3.12). One program, MASTER.BAS allows the user to operate the driving system motors manually and to evaluate the status of each of the primary transducers. SETUP.BAS lets the user input test-specific variables that are passed on to the control program, which is CETEST.BAS. This last program actually controls the test and consists of separate modules that perform the various phases that simulate the event history of a suction caisson including consolidation, suction driving, holding stress, and monotonic pullout.

The fifth and final component of the CET apparatus is the instrumentation package, which is consistent with the automated laboratory testing concept. The measurements are made by a variety of transducers all connected to the Central Data Acquisition System in the MIT Geotechnical Laboratory. A typical CET test requires less than 20 data acquisition channels to monitor instrumentation and power input voltage. (A 14 transducer instrumentation package used for test CET10 is illustrated in Figures 3.17 and 3.18). Five primary transducers (chamber air pressure, caisson wall force and displacement, and caisson cap force and displacement) provide voltage signals for the feedback control loops, while the remaining instrumentation monitors parameters of interest. Displacements of the clay surface are measured by direct current LVDT's located within the pressure chamber. Pore pressures beneath the cap are measured by a pressure transducer mounted directly behind a ceramic porous stone epoxied into the cap base. Pore pressures within the soil mass are measured using stainless steel needle probes, which were developed during the course of this research in order to achieve rapid response and reliable measurements (Figure 3.20).

Resedimented Boston Blue Clay (RBBC) was chosen as the standard test material for the CET testing program for four reasons: 1) procedures for manufacturing RBBC are well-established, 2) the engineering properties of RBBC are well-established from previous laboratory tests, 3) there has been extensive analytical research to model RBBC
properties, and 4) the engineering behavior of RBBC is typical of natural, uncedmented clay deposits with similar index properties. The RBBC (Series IV) used in the CET test program originated from the base of an excavation for MIT's Biology Building (#68), was processed into a dry powder, and stored in sealed 40 gallon containers. This soil has a fine fraction greater than 98%, an average clay fraction of 58±1.2%, an organic content of 4.4%, and an average salt content of 11.6±1.5 g/l. The average plastic limit is \( w_p = 23.5\pm1.1\% \), the average liquid limit is \( w_l = 46.1\pm0.9\% \), and the average plasticity index is \( I_p = 22.7\pm1.2\% \) (a low plasticity clay). Measurements of specific gravity yielded an average value of \( G_s = 2.81 \). All of these index properties and additional compression, consolidation, and flow properties are consistent with data from previous series (I-III) of RBBC (Tables 3.2, B.1).

The caisson element test procedure comprises the following four separate stages: 1) BBC resedimentation, 2) RBBC consolidation using the rigid top cap, 3) RBBC consolidation in the CET apparatus, and 4) model caisson test event sequences. The test sequence for a model caisson typically includes installation by underbase suction, equilibration (set-up), axial pullout and/or sustained loading. For resedimentation, which lasts approximately 16 hours, equal parts (~15 kg) soil and water are mixed into a slurry and sprayed under vacuum into the consolidation chamber of the CET cell. The slurry is consolidated to a maximum vertical stress of \( \sigma_v = 0.5 \) ksc using the rigid top cap and incremental loads (LIR=1). Depending on the amount of allowed secondary consolidation, this phase lasts from 2 to 10 days. After consolidating to 0.5 ksc, the rigid cap is removed and the CET apparatus is connected to the consolidation chamber. Reconsolidation into the virgin range (LIR=1 to \( \sigma_{vc} = 0.5 \) ksc; LIR=0.1 to \( \sigma_{vc} = 0.75 \) ksc) proceeds via computer control of the chamber air pressure and caisson cap and wall actuators. The maximum load is maintained for at least 24 hours prior to the CET event sequence to simulate aging of natural Boston Blue Clay. This phase takes at least 6 days.
Once the clay element has been consolidated, the caisson test event sequence can proceed with caisson penetration, set-up, and axial loading. The test events are initiated by sending control commands to the CETEST.BAS program. Each phase of the event sequence is fully automated. Inherent flexibility in the software allows the test operator to interact with the computer, stop a phase, and either proceed to a different phase or end the test. This allows the user to custom-design any test sequence. The simplest event sequence is a suction driving/set-up/pullout test (standard procedure for tests CET1-8), which requires less than one day.

In the first phase, the operator selects the suction driving module SUCDRV. This program simulates installation by underbase suction by removing load from the caisson cap to balance the force required to penetrate the wall at a constant rate of displacement (to the prescribed depth). Once the caisson wall reaches the required depth, the computer automatically switches to the HOLDSTS module, which maintains a constant total force on the caisson while keeping zero relative displacement between the cap and wall. After monitoring complete pore pressure dissipation, the operator exits HOLDSTS and selects the monotonic pullout module, MONPULL, from the test menu. In this module, the caisson is withdrawn from the soil at a constant rate (i.e., no relative displacement between cap and wall). When the caisson has met the displacement target, the computer transfers control to HOLDSTS to maintain constant total force and zero relative cap/wall displacement. More elaborate sequences including sustained tensile loading have been performed in the CET test program (CET9-14).

7.1.3 Evaluation of CET Cell

Chapter 4 evaluated the CET cell in terms of its ability to: 1) simulate the installation, set-up, and axial tensile loading of suction caissons, and 2) measure the behavior of the model caisson and the test soil during the simulation. The chapter initially
chronicled the development of the CET cell throughout the testing program, and then evaluated the limitations of each of the five CET components.

Data from the first test CET1 illuminated many device limitations, which resulted in significant modifications to the CET cell. Subsequent tests revealed other persistent problems, which were solved by further equipment changes (see Table 4.1). The cylindrical slip ring assembly was improved several times to eliminate air leaks, improve surface drainage, and eliminate caisson/slip ring contact. The major modification to the model caisson was reducing the O-ring 'squeeze' between the cap and wall, which in turn dropped the cap/wall friction from ±5 kg to ±1 kg. The original hydraulic actuator for the cap driving system was severely inadequate during CET1 and was replaced by a ball screw actuator. In order to maintain tighter control over caisson forces and displacements, the gear reduction between the motor and actuators was increased by 10. Compression springs were added to both the wall and cap actuators to eliminate lashback. Numerous modifications were made to the control software modules during the course of the testing program in order to maintain complete control over cap and wall force and displacement. Inadequate time response and total stress sensitivity of the stainless steel probes used to measure pore pressures within the clay necessitated several probe modifications throughout the testing program. The cap pore pressure sensor response was improved by using more reliable transducers and a better porous stone saturation technique.

The consolidation chamber had two main areas of concern: 1) the effect of the rigid wall boundary during pile testing, and 2) the ability of the cylindrical slip ring assembly to prevent air leaks, encourage surface drainage, and allow caisson passage. Measurements of total stress on the chamber sidewall during CET6 indicate that the model caisson events in the clay cake did cause small total stress changes at the boundary. During pullout, the total stress decrease at the sidewall is less than 10% of the induced negative excess pore pressure within the caisson. Air leaks were detected at three locations in the slip ring assembly (Figure 4.9) in tests CET1 through CET6, but several
modifications to the slip ring finally eliminated the leaks; tests CET7 through CET14 showed no signs of leakage. Analysis of the consolidation results from test CET1 revealed that the drainage rate through the slip ring assembly was slower than the rate through the rigid top cap. Attempts to improve drainage through slip ring modification proved futile, but the only serious consequence of slower drainage is increased consolidation times. In all tests but two, the slip ring allowed adequate passage of the model caisson into the soil sample. During penetration in CET2, the wall caught the inner lip of the slip ring and destroyed the slip ring/membrane connection; the inside bottom lip of the inner slip ring was tapered for all subsequent tests (Figure 4.3). Metal-to-metal contact between the caisson wall and inner slip ring caused excessive friction in CET7. Subsequent tests used shims to ensure concentric wall placement prior to installation.

The unique design of the two-component model caisson and its position relative to the inner slip ring leads to two major areas of concern: 1) the stability of the soil surface through soil arching between the cap and wall and between the wall and the inner slip ring, and 2) friction arising between the wall and inner slip ring and between the cap and wall. From visual observations made during and after testing, annular gap stability was only a problem for test CET1, which employed a load increment ratio of LIR=1 during consolidation. Excessive soil extrusion between the wall and inner slip ring (~0.6 cm) and between the wall and cap (2 cm) proved that the LIR was excessively large as the clay was consolidated into the virgin compression zone (σ'v=0.5 to 1.00 ksc). Thereafter, LIR was lowered to 0.1, and no excessive extrusion was observed.

Model caisson friction can arise at four different locations and can be classified as either external or internal friction (Figure 4.12). External friction arises between the wall and the inner slip ring and contributes force to both the wall and total force records. A review of the tests revealed that external metal-to-metal friction between the wall and slip ring was only a problem in CET7 and 10; external friction contributed 4-5 kg to the wall force in these tests. In the remaining tests, the external friction was within ±1.5 kg.
Internal friction contributes force to the wall force signature, which is balanced by an equal but opposite contribution to the cap force. Internal friction does not affect the total force. Visual inspection of the model caisson following tests CET3-14 revealed approximately 1 to 3 mm of extruded clay between the cap and wall, which contributes a small amount of friction (0.2 to 0.7 kg) when the two components are in relative motion. The O-ring connection between the cap and wall contributes ±1 kg to the wall and cap force records. The combined internal friction (1.2 to 1.7 kg) most likely is the cause of the 'transition zone' behavior during caisson installation.

The only serious concern with the driving system was compliance, which delays the mechanical system response. Compression springs maintained tension in the wall and cap ball screw actuators, and this eliminated much of the compliance in these two driving systems.

The control system was evaluated in terms of the algorithms' effectiveness in maintaining: 1) constant total stress on the clay surface throughout testing, 2) constant total force on the caisson during suction installation, 3) constant total force and zero relative displacement during equilibration and sustained loading, and 4) constant rate caisson withdrawal during monotonic pullout. The control system was able to maintain a constant total stress (air pressure) of $\sigma'_{vc}=0.75\pm0.04$ ksc on the surface of the clay cake. During suction installation, the total force was held constant at or within 2 kg of the target value of $F_{tot}=15.2$ kg. Within 3 minutes after the start of set-up, the total force reached the target of $F_{tot}=15.2$ kg and maintained this level to within ±0.3 kg for the remainder of equilibration (Figure 5.27). Likewise for the sustained load tests (CET9-14), the target tensile load levels were held to within ±0.3 kg (Figures 5.46-5.50). Relative displacement between the cap and wall during set-up was no more than ±0.0015 cm. Extensive driving and control system improvements reduced the relative displacement during pullout to less than ±0.003 cm for tests CET6-14. These quoted levels of control are considered to be adequate to excellent.
All instrumentation was checked to ensure proper resolution and precision for the testing program (Table 3.1). In addition, each sensor was calibrated for accuracy. Driving system compliance introduces a bias in the caisson force measurements during the various phases of model caisson testing. But because the test material (normally consolidated RBBC) was soft relative to the drivetrain compliance, this measurement bias remained insignificant. A major theoretical and experimental study of the design of pore pressure probes was necessary to understand the factors that contribute to probe response and to determine exactly what type of probe would be required to accurately measure pore pressure changes during a caisson element test (section 3.1.5). The results of this probe response study indicate that for the rates of caisson movement during installation and pullout phases of the CET program, the Kulite and Cooper probes are sufficiently accurate, while the Data Instruments probe does not respond quickly enough to register accurate changes in pore pressure. Calculations also indicate that the cap pore pressure sensor responds rapidly enough to provide accurate data.

7.1.4 Characteristics of Suction Caissons in the CET Cell

The CET test program consisted of 14 tests and yielded a tremendous amount of data regarding caisson performance. Chapter 5 described the basic suction caisson characteristics (force-displacement relations for the caisson, and pore pressure and surface displacement of the surrounding clay mass) that were measured during the installation, equilibration, monotonic pullout, and sustained loading phases of the test program.

The two main objectives of the testing program were to gain insight into fundamental caisson behavior and provide data for comparisons with analytical prediction. To achieve these goals, tests were designed to illuminate the following principal parameters:

1) penetration resistance for a caisson installed by underbase suction
2) time frame for equilibration of pore pressures after installation
3) caisson displacement during the equilibration phase
4) ultimate pullout capacity for monotonic axial tensile loading
5) time frame for release of underbase suction and caisson displacement during sustained tensile loading
6) effect of installation disturbance on tensile load capacity of the caisson

Other parameters include the penetration rate, the rate of tensile load application, and the effect of reconsolidation on pullout capacity.

The model caisson and clay sample geometry were similar for all tests. The wall is blunt-tipped, with an outside diameter, $D_o=5.08$ cm, and a wall thickness, $t_w=0.145$ cm to give an aspect ratio of $D_o/t_w=35$. The caisson for each test penetrated $L=5.08$ cm into the clay element to give an embedment to diameter ratio of $L/D_o=1$. The clay element had a diameter of 30 cm and a pre-installation height that ranged from $H_c=12.1$ to 14.3 cm (Figure 5.1, Table 5.1).

The instrumentation package for each test included between 12 and 15 transducers including five primary transducers (caisson wall force and displacement, cap force and displacement, and chamber air pressure), a cap pore pressure sensor, one to three pore pressure probes in the clay cake, and clay surface displacement transducers. The probes were located approximately 2.5 cm below the clay surface at radial locations to provide data inside and outside the caisson walls during and after caisson penetration.

The load history for each test can be divided into a series of driving, equilibration, and tensile loading stages (Table 5.2). In each test, the clay element was consolidated into the virgin compression range to a consolidation stress of $\sigma'_{vc}=0.75$ ksc with the caisson wall tip flush with the caisson cap at the clay surface. In all tests except CET2, this consolidation phase was held for at least 24 hours prior to penetration.

In most tests, installation by underbase suction was simulated by penetrating the wall into the clay ($L=5.1$ cm) at a constant displacement rate ($v_w=0.3$ cm/min) while maintaining a constant total force on the caisson, as net increases in the wall force were
balanced by equal and opposite load increments applied to the cap. One test CET6 employed a slower penetration rate \((v_w=0.01 \text{ cm/min})\) for the first L-1.05 cm of penetration. Two tests studied the effect of installation disturbance by penetrating the wall with a constant cap force (CET13; comparable to an open-ended pile jacking) and penetrating the wall with zero cap displacement (CET14).

Following penetration in all tests, the caisson was allowed to equilibrate with a constant total force \(F_{tot}=15.2 \text{ kg (}=F_w+F_c)\) and zero cap/wall relative displacement for at least 18 hours. After set-up, a variety of tensile loading schemes were applied. These can be classified into two categories: 1) monotonic pullout to failure (6 tests), and 2) sustained loading (6 tests).

In three tests, the caisson was withdrawn \((v_w=-0.03 \text{ cm/min})\) beyond peak tensile load and then pulled faster \((v_w=0.3 \text{ cm/min})\) until complete extraction. In three other tests, after initial pullout to capacity, the caisson was re-equilibrated and pulled a second time.

For one sustained load test, the caisson was pulled at the standard rate until reaching a low tensile load \((F_{tot}=-2.2 \text{ kg})\), and then held for 24 hours, whereupon it was pulled to failure. For the remaining 5 sustained load tests, the caisson was pulled to a predetermined tensile load and held at this level until either the caisson began to fail or more than 24 hours had passed, whichever came first. If no failure occurred, then an increment of load was applied and maintained for up to 24 hours. The process was repeated until failure, where upon the test was either ended (2 tests), or the caisson was re-equilibrated and pulled a second time (3 tests). See Table 5.2 and Figure 5.2 for individual test loading stages.

In order to ensure the integrity of comparisons made in the presentation and analysis of the test results, a quality assessment system was developed to rate the quality of control and instrumentation for each test (Tables 5.3-5.4). In general, the quality of
both control and instrumentation data improved with successive tests, as the CET cell was continually refined.

**Installation Behavior**

In the standard suction installation tests, the total force on the caisson is held constant in the range $F_{\text{tot}}=15$ to 17 kg. In great contrast, the total force in CET13 (jacked-in pile) climbed to $F_{\text{tot}}=24$ kg within the first $z_w=0.05$ cm of wall penetration and then rose steadily at an average rate of $2.33$ kg/cm (from $z_w=0.4$ to 5.1 cm) to reach $F_{\text{tot}}=37.6$ kg by the end of penetration. Hence, the total additional force required to 'jack' the caisson into the clay is $\Delta F_{\text{tot}}=22.4$ kg, which is more than twice the total force maintained on the suction-installed tests ($F_{\text{tot}}=15.2$ kg).

The measured wall force data reveal three distinct zones of behavior: initial, transition, and deep. The initial penetration zone, which ends at $z_w=0.2$ cm, is characterized by a very stiff wall response, as the wall acquires more than 10 kg of compressive load for the two most representative standard tests (CET8,9). The transition zone starts at $z_w=0.2$ cm and ends at a wall tip displacement ranging from $z_w=0.6$ to 1.8 cm, depending on the test (see Table 5.5a). The wall force signature during this zone is defined by a "hump-shaped" rise and fall, with a local maximum force at a depth from $z_w=0.3$ and 1.0 cm. The transitional behavior is probably caused by the buildup and subsequent release of intercomponent friction between the cap and wall components and between the wall and inner slip ring (see sections 4.3.2, 5.2.1, Figure 4.12). Following the transition zone, the wall enters the deep penetration zone, which extends from the end of transition ($z_w=0.6$ to 1.8 cm) to the final penetration depth, $L=5.1$ cm. In this zone the wall force increases at a constant rate with depth. By the end of penetration, the wall carries a compressive load, $F_w=17-18.7$ kg.

The pattern of behavior for the wall that was 'jacked' into position (CET13) is similar in shape, but slightly greater in magnitude, compared to that for the suction-
installed models. As shown in Figure 6.1, the wall responds very stiffly at first, acquiring $\Delta F_w = 10.5$ kg during the first $z_w = 0.2$ cm of penetration. In the deep zone ($z_w = 0.4$ to $5.1$ cm), the wall force increases steadily at a rate of $M_w = 2.1$ kg/cm to reach $F_w = 23.6$ kg. Both the average gradient of the wall force during deep penetration and the final wall load for CET13 are higher than the corresponding values in any suction installation test (see Table 5.5b).

Because the total force was held constant in the standard suction installation tests, the cap force behavior is equal but opposite to the wall force behavior ($\Delta F_c = -\Delta F_w$). Figure 6.1 shows that, allowing for some scatter due to a disturbed clay surface, intercomponent friction, and slight differences in the magnitude of the constant total force, the cap responds stiffly during initial penetration. At the end of wall penetration ($L = 5.1$ cm), the cap carries a nearly zero to slightly tensile load, $F_c = +0.5$ to $-3.5$ kg. Note that the cap force for CET13 is held constant at $F_c = 15$ kg, as the wall is jacked into position.

Pore pressures were measured directly beneath the cap and by needle probes embedded at a depth of approximately 2 and 2.5 cm below the caisson prior to penetration (Figure 6.2). Overall, small excess pore pressures ($\Delta u = \pm 0.2$ ksc) were generated in the soil mass during installation with underbase suction. At the caisson cap/clay interface, the pressure drops ($\Delta u = -0.05$ to $-0.2$ ksc) within the first $z_w = 0.2$ cm of penetration as the cap loses load. With continued wall penetration, low positive pore pressure generation ($\Delta u < 0.3$ ksc) is followed by a general decrease in excess pore pressure so that by the end of penetration, nearly zero excess pore pressure ($-0.2$ ksc $< \Delta u < +0.05$ ksc) remains. At the locations within the soil mass, probes measure only a slight effect from the passing wall and retreating cap.

Much higher pore pressures were generated by the caisson that was jacked into place (CET13; $F_c = 15$ kg). Figure 6.2 shows that the advancing wall generates $\Delta u = 0.6$ ksc beneath the cap during the first $z_w = 1$ cm. Thereafter, the excess pressure slowly increases to $\Delta u = 0.68$ ksc by the end of penetration. Even at 1.9 cm from the wall exterior (P4),
positive excess pore pressures were measured ($\Delta u=0.3$ ksc as the wall passed the probe; $\Delta u=0.25$ ksc by the end of penetration).

The rates of caisson cap movement vary throughout installation, and there are large variations in the pattern of cap movements among the tests. However, by the end of wall penetration, the net upward cap displacement in four of the tests is similar ($\delta_c=0.54-0.6$ cm) and corresponds to 86-94% of the volume displaced by the wall. In test CET13, 'jacking' the wall into place causes the cap to move down $\delta_c=-0.06$ cm during the initial $z_w=1.2$ cm of penetration. However, by the end of penetration, the cap has lifted $\delta_c=0.52$ cm, which is similar to the cap heave in the suction installation tests.

Generally, the soil displacements are small relative to the wall penetration depth for both types of caisson installation. By the end of penetration in the suction installed tests, the surface compresses $\delta_s=-0.001$ to -0.004 cm across the width of the chamber.

Post-Installation Equilibration and Post-loading Re-Equilibration

In all CET tests, installation is followed immediately by an equilibration phase, during which the total force is maintained constant at $F_{\text{tot}}=15.2$ kg, and no relative displacement is allowed between the cap and wall ($\Delta \delta_w = \Delta \delta_c$). For the suction installation tests, the wall and cap forces redistribute rapidly to equilibrium values within 3 minutes, and the wall carries nearly all of the total load. The wall force starts in the range $F_w=17-20$ kg and falls slightly to $F_w=13.5-16.5$ kg. The cap force begins at $F_c=0$ to -4 kg and rises only to $F_c=-0.5$ to 1.5 kg. The total force in CET13 starts out much higher at $F_{\text{tot}}=33.6$ kg, but is lowered quickly (within 2 minutes) to the target value $F_{\text{tot}}=15.2$ kg. The wall and cap forces in CET13 also were much higher at the start: $F_w=23.5$ kg, $F_c=14$ kg. Due to the large installation excess pore pressures in CET13, a much greater amount of time was required ($t=100$ to 200 minutes) for redistribution of the wall and cap forces, but the equilibrium values ($F_w \approx 13$ kg, $F_c \approx 2$ kg) are roughly equivalent to those achieved in the standard suction tests.
Installation by underbase suction generated minor excess pore pressures ($\Delta u = -0.2$ to $+0.05$ ksc) at the cap/soil interface and at the centerline location within the soil plug (P1). As the cap force equilibrated within the first 3 minutes of equilibration, slight positive pore pressures ($\Delta u = 0$ to $0.1$ ksc) were generated at these two locations. These excess pore pressures dissipated after roughly 16 hours (1000 minutes) of equilibration. Small installation pore pressures ($\Delta u = 0$ to $0.1$ ksc) measured at the other probe locations (P2, P3) also dissipated within 16 hours. Jacking the wall into position (CET13) generated relatively large excess pore pressure ($\Delta u > 0.6$ ksc at cap/soil interface, $\Delta u > 0.2$ ksc at P4), which also dissipated after 16 hours of equilibration.

The suction-installed caisson settles as a monolithic unit at an increasing log rate until reaching a certain point in time ($t = 70$ to 100 minutes), after which the settlement rate is approximately log-linear. The rate of log-linear settlement varies among the tests; this variation reflects the differences in the drainage conditions from test to test (see section 4.2.2). The final settlement was small and varied from approximately 10 to 65% of the wall thickness ($t_w = 0.145$ cm). The caisson in test CET13 heaves nearly $8\%t_w$ due to unloading of the caisson ($t < 3$ minutes) and then undergoes a net settlement of $28\%t_w$ as the pore pressures dissipate.

Following the point of maximum tensile resistance in five tests (CET5-6, 12-14), the caisson was re-loaded with the original compressive force, $F_{tot} = 15.2$ kg, for a period of at least 24 hours, which was called the re-equilibration phase. Up to 10 minutes are required for the total force to reach the target, and thereafter, 400-500 minutes are required for caisson forces to redistribute and reach equilibrium. However, at the end of this long redistribution period, the wall force carries most of the total load ($F_w > 90\%F_{tot}$), as it does during post-installation set-up ($F_w(\text{avg.}) = 95\%F_{tot}$).

It is clear that re-loading the caisson in the CET cell raises the cap force, which in turn creates positive excess pore pressures ($\Delta u = 0.2$ to $0.7$ ksc) within the soil plug, as measured beneath the cap and at P1-P2 ($r = 0$ to 1.8 cm). Smaller positive excess pore
pressures (Δu<0.35 ksc) are measured outside the caisson at P3-P4 (r=3.2-4.45 cm). As with the post-installation set-up, complete dissipation occurs within 1000 minutes.

The tensile loading history had a significant effect on the caisson settlement during re-equilibration (Figure 6.18). Final re-equilibration settlements (δₙ=-0.18 to -0.37 cm) were 2 to 4 times greater than the average final equilibration settlement (δₙ=-0.08 cm) of four suction-installed tests. For the two tests (CET5-6) in which the caisson was re-equilibrated following monotonic tensile loading, the caisson settled at an increasing rate vs. log t until reaching approximately δₑ=0.07 cm. Thereafter, the settlement rate was consistent with secondary compression of the clay. In the three tests that included sustained tensile loading (CET12-14), the initial settlement (δₑ=0.2, 0.41, 0.16 cm, respectively) was nearly equal to the amount of upward displacement (δₑ=-0.19, -0.35, -0.14 cm) achieved during the preceding sustained load stage. The caisson then settled at the secondary compression rate. The rates observed during re-equilibration are consistent with those measured during post-installation equilibration. Note that, while larger than the post-installation set-up displacements, the final re-equilibration settlements are still small (δₑ=1 to 2.5 times the wall thickness).

*Monotonic Pullout 1*

In all tests, the caisson exhibits a very stiff initial response for δₑ<0.002 cm. After reaching a well-defined yield point between δₑ=0.01 and 0.02 cm, the caisson continues to pick up load, but at a rapidly decreasing rate. Maximum capacity ranges from $F_{tot}=-22.4$ to $-23.95$ kg and occurs at $δₑ=0.23$ to 0.3 cm. The average capacity of CET5,6, and 8 is $F_{tot}=-22.8±0.7$ kg, which is reached at $δₑ=0.25±0.03$ cm. For displacements beyond peak and up to $δₑ=0.4$ cm, there is only slight post-peak reduction in the uplift resistance.

The initial wall stiffness is significantly lower than that of the overall caisson. By the time the wall reaches yield ($δₑ=0.01$ to 0.02 cm), it carries a tensile load ranging from $F_w=-6$ to -12 kg. Maximum wall resistance ranges from $F_w=-12$ to -14 kg, and is
mobilized at a displacement range \( \delta_w = 0.15 \) to 0.25 cm, which is close to the mobilization range for total capacity \( (\delta_w = 0.23 - 0.3 \text{ cm}) \). The wall contributes an average of 58% of the total capacity.

The cap carries almost no load at equilibrium prior to pullout \((-4.4 \text{ kg} < F_c < 1 \text{ kg})\), but mobilizes \( F_c = -4 \) to -7 kg almost instantaneously (i.e., within \( \delta_w = 0.001 \) cm), and then slowly accumulates \( \Delta F_c = -3.5 \) to -5 kg up to the displacement necessary \( (\delta_w = 0.2 \) cm) for peak capacity. The cap shows a very stiff initial response with a well-defined yield point at \( \delta_w = 0.003 \) cm. At the average displacement \( (\delta_w = 0.25 \) cm) required to mobilize maximum total capacity \( (F_{\text{tot}} = 22.8 \text{ kg}) \), the average cap force is \( F_c = -9.7 \pm 1.4 \) kg, or 42% of the capacity.

The best estimate of caisson capacity averages \( F_{\text{tot}} = -23 \) at \( \delta_w = 0.25 \) cm. At this displacement, the wall force ranges from \( F_w = -11.4 \) to -14.9 kg, while the cap ranges from \( F_c = -9 \) to -11.8 kg (Figure 6.22).

In general, monotonic pullout of the caisson generates large negative excess pore pressure in the soil plug within the caisson (Figure 6.23). Peak excess pore pressure is generated at approximately the same displacement \( (\delta_w \approx 0.2 \) cm) as maximum cap force \( (\Delta u_{\text{max}} = -0.4 \) to -0.5 ksc beneath the cap and at P1 for CET5,6). Much greater peak pore pressure \( (\Delta u_{\text{max}} = -0.68 \) ksc) is generated in the soil plug near the wall (P2, CET8). Completely different pore pressure behavior is measured outside the caisson. At small displacements \( (\delta_w \leq 0.003 \) cm), the pore pressure pattern near the outside wall of the caisson (P3) follows closely the measurements inside \( (\Delta u = -0.2 \) to -0.4 ksc) for three tests (CET9-11), while much smaller negative pressures are generated in CET8,14. With continued extraction \( (\delta_w = 0.02 \) cm), the exterior pore pressure stabilizes at \( \Delta u = 0.1 \) ksc, but then decreases toward \( \Delta u = -0.1 \) ksc once the capacity is fully mobilized \( (\delta_w = 0.2 \) cm).

At radial distances close to the wall (S1, S2), the soil surface heaves slightly \( (\delta_s = 0.001 - 0.003 \) cm) during the initial phase of loading, \( \delta_w < 0.1 - 0.15 \) cm (Figure 6.24). At locations farther from the wall (S3, S4, S5), settlements increase monotonically with
pullout displacement of the caisson. However, by the end of pullout ($\delta_w = 0.3$ cm) the soil surface has moved very little, less than $\delta_g = 0.015$ cm ($\delta_g / \delta_w < 4\%$).

**Monotonic Pullout 2: the effect of re-equilibration**

Total capacity for the second pullout for those tests that were re-equilibrated (CET6,12-14) ranges from $F_{tot} = -25.2$ to -30.5 kg and averages $F_{tot} = -27.2 \pm 2.4$ kg, which is 19\% higher than the virgin pullout average $F_{tot} = -22.8$ kg (Figure 6.26). In one test (CET9), the caisson was not re-equilibrated following 13.4 hours of sustained load ($F_{tot} = -2.2$ kg). Instead, the caisson immediately was pulled monotonically to reach a capacity of only $F_{tot} = -20.9$ kg, which is 7\% lower than the lower bound value ($F_{tot} = -22.3$ kg) from the virgin 'best estimate' range. Initial stiffness behavior ($\Delta F_{tot} / \delta_w$ for $\delta_w = 0.0$ to 0.002 cm) for MP2 is similar to that for MP1.

An increase in wall resistance is responsible for the total capacity increase for the re-equilibrated tests. At the point of peak total load during pullout 2, the wall force from the re-equilibrated tests ranges from $F_w = -15.2$ to -19.4 kg and averages $F_w = -17.0 \pm 1.7$ kg. This represents a 29\% increase over the average virgin wall resistance $F_w = -13.2 \pm 2.1$ kg. The wall resistance during MP2 for CET9, which was not re-equilibrated, reaches $F_w = -11.3$ kg, which conforms to the lower bound for virgin pullout ($F_w = -11.0$ kg).

The cap mobilizes approximately the same amount of resistance during monotonic pullout 2 as that measured during virgin pullout. The MP2 cap resistance at the displacement corresponding to total capacity ranges from $F_c = -7.8$ to -13.7 kg with an average $F_c = -10.1 \pm 2.2$ kg, which is comparable to the virgin cap resistance range $F_c = -8.9$ to -11.3 kg and average $F_c = -9.7 \pm 1.4$ kg.

Measurements of excess pore pressure and soil surface displacement during pullout 2 are very similar to the behavior described for the initial 'virgin' pullout stage (see section 5.8.2-5.8.3).
**Sustained Load**

The sustained tensile load CET tests can be divided into three groups: 1) low-level single-stage load leading to stable conditions (CET9), 2) high-level single-stage load leading to caisson failure (CET10), and 3) multi-stage load ultimately leading to caisson failure (CET11-14). The small tensile load \( F_{\text{tot}} = -2.2 \, \text{kg} \) applied to the caisson in the single-stage test CET9 causes a stable response (Figure 6.27). Initially, the cap carries slightly more than the applied load \( F_c = -3 \, \text{kg} \), but rapidly sheds this tensile load \( F_c \approx 0 \, \text{kg} \) at \( t = 10 \) minutes), and then acquires a small compressive load \( F_c = 1.4 \, \text{kg} \) by the end of the stage. Meanwhile the wall starts out with a small compressive load of \( F_w = 1.8 \, \text{kg} \), but carries slightly more than the applied tensile load \( F_w = -3.6 \, \text{kg} \) within 8 hours. Both the wall and cap displace upward a very small amount, reaching an equilibrium displacement, \( \delta_w = 0.012 \, \text{cm} \) in approximately 100 minutes. Application of the tensile load initially generates excess pore pressures ranging from \( \Delta u = -0.15 \) to \(-0.28 \, \text{ksc} \) within the soil plug, which dissipate within 100 minutes as the caisson equilibrates under the tensile load. Soil surface displacements outside the caisson are subtle. Near the caisson \( (S1, S2; r=4.2, 5.2 \, \text{cm}) \) tensile loading initially causes the soil to heave slightly \( \delta_s < 0.002 \, \text{cm} \). By the end of the sustained load stage, the entire soil surface is settling at a rate consistent with secondary compression. \( \delta_s < 0.005 \, \text{cm} \).

A much higher applied tensile load, \( F_{\text{tot}} = -11.4 \, \text{kg} \), leads to failure in the other single-stage test CET10. The cap initially carries 73% of this tensile load (at \( t = 1 \) minute; \( F_c = -8.3 \, \text{kg}, F_w = -3.1 \, \text{kg} \)). Within 40 minutes the cap loses 5 kg of tension, but cannot shed the remaining tensile load, \( F_c \approx -3.5 \, \text{kg} \) (30% of the total tension). The wall increases its share of the tensile load from \( F_w = -3.1 \, \text{kg} \) to -7.9 kg during the first 40 minutes of loading and maintains nearly -8 kg of tension for the rest of the test. A rapidly increasing upward caisson displacement is the earliest sign of failure in CET10. After \( t = 0.4 \) minutes, the displacement rate increases with the log of time; caisson displacement reaches \( \delta_w = 0.11 \, \text{cm} \) (76% of wall thickness) after 40 minutes. Another failure indicator is the pore
pressure behavior. The large applied tensile load initially generates \( \Delta u = -0.5 \) to \(-0.6 \) ksc within the soil plug. Approximately half of this dissipates within 40 minutes, but \( \Delta u = -0.2 \) to \(-0.3 \) ksc remains for the duration of loading as the mechanisms of dissipation and generation (due to caisson displacement) become balanced. The soil surface at locations close to the caisson \((S1, S2; r=4.2, 5.2 \text{ cm})\) initially heaves \( \delta_s < 0.001 \) cm, as it does in low-level tensile loading \((\text{CET9})\). However, the high-level tensile load in CET10 causes rapid upward caisson displacement, which in turn draws in soil exterior to the caisson, as measured by the high surface compression rate after approximately 40 minutes.

Results from the 4 multi-stage sustained load tests \((\text{CET11-14})\) show consistent behavior. The initial stage resembles behavior depicted for the low-level single-stage test CET9. For CET11, the cap carries a tensile load \( (F_c = -7.5 \text{ kg})\) after the application of \( F_{\text{tot}} = -2.9 \text{ kg} \), which generates a moderate amount of negative excess pore pressure \( (\Delta u = -0.32 \text{ ksc})\) within the soil plug \((\text{Figure 6.29})\). However, as the cap sheds all of the tension within 15 minutes, the excess pore pressure dissipates completely. As for CET9, caisson movement is minimal \( (\delta_w < 0.02 \text{ cm})\). Additional increments of tension \( (\Delta F_{\text{tot}} \approx -2 \text{ kg})\) are carried by the wall, which does not significantly affect pore pressures, caisson displacement, or soil surface displacement. The final stage \((4, F_{\text{tot}} = -8.9 \text{ kg})\) leads to failure and causes behavior similar to that recorded for the high-level single-stage test CET10. The cap acquires the \(-2 \text{ kg}\) of applied tensile load and does not shed this tension as the caisson fails \((\text{Figure 6.32})\). After \( t=10 \) minutes, it is clear that the caisson is displacing at an increasing rate with log of time, and the soil plug maintains a negative excess pore pressure just below \( \Delta u = -0.1 \text{ ksc} \). The soil surface shows no sign of failure until much later \((t > 500 \text{ minutes})\), when it rapidly compresses as soil is drawn into the caisson. Similar multi-stage behavior to CET11 was recorded for CET12-14 and was described in section 5.6. The one difference among the multi-stage tests is that the level of load at which failure occurs ranges from \( F_{\text{tot}} = -9 \) to \(-13 \text{ kg}\), which indicates some uncertainty in the long-term capacity of the CET model caissons.
7.1.5 Interpretation of Suction Caisson Behavior in the CET Cell

This section summarizes the interpretation of the model caisson events, which was presented in detail in Chapter 6.

**Wall Penetration Resistance During Installation**

Prototype suction caissons that are used as anchorages for tension leg platforms (TLP's) have very large diameters and penetrate a short distance into the seabed through a combination of self-weight and underbase suction. The resistance to penetration is provided by a combination of the caisson structure tip capacity and the skirt wall skin friction. Similarly for the installation phase in the CET testing program, the resistance to model caisson penetration is provided by the wall tip bearing capacity during initial penetration and the wall interface friction during deep penetration.

Considering shallow penetration, the data yield a net normalized tip resistance factor, \( \frac{q_{ult} - \sigma_{vc}}{\sigma'_{vc}} = 5.13 \pm 0.16 \) for installation by underbase suction and \( \frac{q_{ult} - \sigma_{vc}}{\sigma'_{vc}} = 6.30 \) for 'jacked-in' installation (CET13). These values are 1.3 to 2.5 times greater than for closed-ended penetrometer geometries considered in previous experimental and theoretical investigations on lightly overconsolidated Boston Blue Clay (Morrison, 1984; Ladd, 1991; Aubeny, 1992). Using standard bearing capacity formulae \( q_{ult} = N_c s_u + \sigma_{vc} \) and assuming a reference undrained strength ratio for \( K_0 \)-normally consolidated RBBC in triaxial compression, \( s_{u TC}/\sigma'_{vc} = 0.32 \) (Sheahan, 1991), the resulting tip bearing factor, \( N_c = 16.0 \pm 0.5 \) for suction installed tests and \( N_c = 19.7 \) for CET13.

The second component of wall resistance is the wall skin friction. Regression analyses on the deep penetration behavior for representative tests (Figure 6.5b) reveal a wall force gradient, \( f_w = 1.45 \pm 0.13 \) kg/cm for installation by underbase suction and \( f_w = 2.13 \) kg/cm for CET13. For suction-installed tests, this leads to an average inside and outside skin friction, \( f_s = 0.047 \pm 0.004 \) ksc. The conventional skin friction \( \beta \) and \( \alpha \) factors are obtained by normalizing the skin friction with the consolidation vertical effective stress
and the direct simple shear undrained shear strength, respectively. For installation with underbase suction, $\beta=0.063 \pm 0.005$, which is much lower than the undrained shear strength ratio of $K_0$-normally consolidated RBBC in direct simple shear, $s_{udSS}/\sigma'_{vc}=0.205$ (Ladd, 1991), and hence $\alpha=0.31 \pm 0.03$. The skin friction is significantly higher for CET13 ($f_s=0.069$, $\beta=0.092$, $\alpha=0.45$). The measured values of skin friction for CET13 are in very good agreement with previous predictions for undrained deep penetration of open and closed-ended piles in BBC using the Strain Path Method and MIT-E3 model ($\beta=0.073-0.096$; Whittle and Baligh, 1988; Whittle, 1992). The skin friction values for the suction-installed tests ($f_s=0.047$, $\beta=0.063$, $\alpha=0.31$) are slightly lower than previous predictions, perhaps due to the effect of underbase suction.

Effective Stress Changes and Soil Movements During Installation

An analysis of the caisson force redistribution, pore pressure generation, and soil displacements during installation reveals that both installation by underbase suction and installation by 'jacking' cause the same two very interesting conditions in the soil: 1) the loss of most, if not all, of the effective stress in the soil plug created by the advancing wall, and 2) the movement of most of the wall-displaced soil volume toward the caisson interior. As indicated by the average value of the underbase suction tests, the effective stress directly beneath the cap falls rapidly from the consolidation value of $\sigma'_v=0.85$ ksc to $\sigma'_v=0.1$ ksc after the wall has penetrated only $z_w=0.5$ cm. With continued wall penetration, the effective stress recedes essentially to zero. Despite a completely different imposed stress condition (constant $F_D$) for the jacked installation test CET13, the effective stress at the interface still dives toward zero (from $\sigma'_v=0.8$ ksc to $\sigma'_v=0.25$ ksc) within the first 0.5 cm of penetration and maintains a slightly positive value ($\sigma'_v=0.1$ ksc) for the remainder of installation. It is clear that the loss of effective stress during suction installation is caused by the loss of cap stress, while the severe effective stress drop during jacked installation is a result of a large increase in excess pore pressures.
Force equilibrium calculations on the developing soil plug during suction penetration reveal a dramatic, if not complete, loss of vertical effective stress within the entire soil plug. No interior caisson pore pressure data was available to confirm this behavior for the jacked-in test CET13.

The second striking condition caused by both underbase suction and jacked installation was the large upward movement of the cap. The average cap heave by the end of wall penetration for the suction-installed tests corresponds to 97% of the wall-displaced volume, while the cap heaved nearly as much (81% of the wall-displaced volume) in the jacked-in test CET13. For the suction-installed tests, an analysis of the stress conditions imposed on the soil and the resulting soil plug displacements in terms of axial strain indicate that the plug effective stress and strain behavior is similar to that for undrained shear in a clay specimen subject to triaxial extension unloading. For the jacked-in test CET13, the vertical total stress on the plug surface is constant, but the vertical effective stress and approximate strain behavior by the end of penetration resemble the suction-installed tests. Hence, the shearing mode approximates undrained triaxial extension. In general, the results from both types of tests indicate that the problem is strain-controlled.

Post-Installation Equilibration

Three significant geotechnical uncertainties are associated with the equilibration period that follows the installation of a suction caisson: 1) the rate of excess pore pressure dissipation, 2) the rate of caisson settlement, and 3) the change in properties of the soil surrounding the caisson. The suction installation process generated negligible excess pore pressure inside the caisson, which precludes a full analysis of the rate of pore pressure dissipation.

During the initial \( t < 100 \) minutes of equilibration, the caisson in the CET cell settles a very small amount \( \delta_c < 0.02 \) cm, but for \( t > 100 \) minutes, the rate of settlement is proportional to \( \log t \), generating final settlements of up to \( \delta_c = 0.09 \) cm (CET9). These
rates of settlement are approximately constant across the surface of the clay and therefore are caused by secondary compression of the clay. Estimated rates of secondary compression range from $C_{\alpha \varepsilon} = \frac{de}{d(\log t)} = 0.001$ to 0.005, with an average value, $C_{\alpha \varepsilon} = 0.0034 \pm 0.0016$. The secondary compression ratio is $C_{\alpha \varepsilon}/\text{CR} = 0.028 \pm 0.013$, which is lower than value estimated for inorganic soft clays ($C_{\alpha \varepsilon}/\text{CR} = 0.04 \pm 0.01$; Mesri & Castro, 1987), but is well within the range reported previously for RBBC ($C_{\alpha \varepsilon}/\text{CR} = 0.008 - 0.040$; Seah, 1990; Sheahan, 1991).

The change in soil properties surrounding the caisson is probably the most difficult to determine, given the lack of appropriate instrumentation in the CET cell (i.e., no total stress or pore pressure sensors attached to the wall). However, because the cap load remains very low ($F_c < 1.7$ kg) throughout equilibration, and the small installation pore pressures slowly dissipate, the vertical effective stress within the soil plug remains near zero (average effective stress beneath the cap, $\sigma'_v = 0.06$ ksc). Outside the caisson, constant vertical stress ($\sigma'_v = 0.75$ ksc) conditions prevail on the clay surface, which allows the vertical effective stress to return to $\sigma'_v = 0.75$ ksc after excess pore pressures have dissipated.

_Uplift Failure Mode and Components of Capacity_

Under calm sea conditions, TLP's apply tensile loads that the suction anchor can resist through self-weight and ballast alone. However, during storms TLP's apply large tensile loads to the suction anchor, which must rely on sidewall skin friction and underbase suction for additional resistance. Prior experimental research on the pullout of caissons installed in clay have based capacity predictions on reverse end bearing failure theory. In this failure mode, axial tensile load on the caisson mobilizes: 1) the limiting skin friction at the soil/caisson interface on the exterior surface of the caisson wall, and 2) the shear resistance of the soil mass (reverse end bearing) below the full basal area (footprint) of the caisson wall.
The reverse end bearing failure pattern is analogous to that for compressive bearing capacity, which is based on solutions using plasticity theory (Figure 6.25). In zone I, the conical volume of soil directly below the caisson basal area, the soil undergoes vertical extension as a result of the vertical tensile load, while in zone III, the soil extends horizontally due to the release of horizontal load. Zone II is a radial shear zone where the extension direction changes from vertical to horizontal. The overburden soil located above the tip of the caisson wall (Zone A) does have shear resistance, but this is neglected in the plasticity theory solutions (a conservative assumption).

Considering force equilibrium at failure individually on both the soil plug and the caisson, the breakout force comprises the buoyant anchor and soil plug weight, the external wall skin friction, and the reverse end bearing. An important assumption in this analysis is that the soil plug travels upward with the caisson, and therefore the internal skin friction does not contribute to the breakout force. In the CET cell design, the model caisson was not instrumented to measure internal skin friction to evaluate this assumption. However, a soil plug was found adhering to the cap following complete caisson extraction from the clay cake in each CET test, which indicates that the soil plug traveled with the caisson during pullout. In the analysis of the ultimate capacity in the CET cell, the end bearing incorporates the plug weight term, while the buoyant anchor weight ($W_a < 0.1$ kg) is neglected. Hence the breakout resistance $F_b$ is composed of just external wall friction $F_{esf}$ and reverse end bearing $F_{reb}$.

**Wall Interface Friction**

The calculation of the wall resistance term is based on three assumptions: 1) the brass wall is relatively inextensible, such that the total resistance is provided by $f_s$, the limiting skin friction, 2) the skin friction $f_s$ is equivalent to the undrained shear strength of the soil adjacent to the soil/wall interface, and 3) the clay fails in a mode of undrained direct simple shear, $s_{uDS}$.$^1$ The resulting equation for external skin friction during pullout
in the CET cell is: \( F_{esf} = f_s A_c \), where \( A_c \) is the penetrated external wall area. The average computed value \( F_{esf} = 11.95 \pm 0.10 \) kg compares very well with the measured average \( F_w = 13.2 \pm 2.1 \) kg \((F_{esf} > 90\% F_w)\) for 'virgin' pullout (Table 6.2). The lower estimate may be due to actual inside wall shear tractions that are not accounted for in the external friction equation.

**Reverse End Bearing**

Because undrained conditions prevail in the monotonic pullout tests in the CET cell, a \( \phi=0 \) analysis for the reverse end bearing is appropriate. In addition, the following conditions are assumed: 1) the resistance reduction provided by the soil overburden above the anchor bottom is canceled by the effective weight of the soil plug, and 2) the shearing resistance of the soil above the caisson rim contributes to the end bearing resistance through a depth factor \( L_c \) applied to the cohesive resistance. These modifications yield a reverse end bearing equation: \( F_{reb} = A_b S_c c_{ub} N_c L_c \), where \( A_b \) is the base cross-sectional area, \( S_c \) is the shape factor, \( c_{ub} \) is the undrained shear strength (triaxial extension mode) of the soil below the caisson, and \( N_c \) is the bearing capacity factor. This equation is identical to that used by Byrne and Finn (1977) for predicting uplift capacity of 1g skirted anchors embedded in clay and by Clukey and Morrison (1993) for predicting capacity of 100g suction anchors in clay. The best estimate of the reverse end bearing capacity for the model caisson in the CET cell is \( F_{reb} = 18.9 \) kg. The measured cap force at the displacement corresponding to total capacity ranged from \( F_c = -8.75 \) to \(-11.3 \) and averaged \( F_c = -9.65 \pm 1.43 \) for the virgin pullout tests (Table 6.2). This average measured cap force \( F_c \) is approximately 50\% of the estimated reverse end bearing. Such a large discrepancy most likely is due to uncertainty in the assumptions (i.e., the empirical shape and depth factors) regarding the reverse end bearing equation.

The combined best estimate of the uplift capacity in the CET tests, \( F_b = -30.8 \) kg \((= F_{esf} + F_{reb})\), overpredicts the average measured total capacity \( F_{tot} = -22.9 \pm 0.9 \) kg.
\( (=F_w+F_c) \) by 35\%. While the skin friction calculation \( (F_{esf}) \) contains some uncertainty, the reverse end bearing term \( (F_{reb}) \) accounts for most of the difference between the measured and computed capacities.

The CET caisson uplift capacity also was calculated using numerical limit analyses based on finite element discretization and linear programming optimization methods (Ukritchon, 1996; after Sloan, 1988a; and Sloan and Kleeman, 1994). These analyses predict upper and lower bound collapse loads that are 17-32\% lower than the average capacity measured in the CET tests.

**Effect of Re-Equilibration**

Prototype suction anchors mobilize wall skin friction and underbase suction to resist large TLP tensile loads during extreme storm conditions. After the storm passes, however, the anchor is allowed to re-equilibrate with the surrounding soil under calm sea conditions (i.e., tensile loads are resisted by anchor weight and ballast alone). Subsequent storms, once again, apply large tensile loads to the suction caisson. This loading-equilibration-loading sequence was simulated in the CET testing program to study the effect of the 're-equilibration phase' upon ultimate capacity.

Reverse end bearing \( (F_{reb}) \) and wall skin friction \( (F_{esf}) \) values were computed for the tests subject to monotonic pullout 2 (Table 6.2). Note that these estimates are nearly the same as those computed for virgin pullout because the underlying assumptions in the equations are the same. For those tests that included a re-equilibration stage, the average computed value for wall friction is \( F_{esf}=12.0\pm0.2 \) kg, which is only 70\% of the average measured value \( F_w=17.0\pm1.7 \) kg (for virgin pullout, \( F_{esf}>90\%F_w \)). Clearly, the skin friction equation does not account for the increased strength (due to radial consolidation during re-equilibration) of the zone of clay around the exterior caisson wall. As for the virgin pullout, the reverse end bearing \( F_{reb}=18.9 \) kg overpredicts the maximum cap force \( F_c=10.2\pm2.5 \) kg due to uncertainty in the end bearing equation assumptions.
**Effect of Pullout Rate**

Prototype suction caissons are subject to tensile cyclic loading sequences that vary in frequency, magnitude, and duration according to the particular storm characteristics. Although the CET testing program did not simulate cyclic loading histories, some tests did include monotonic tensile loading at two different rates of withdrawal. These tests provide some insight into the effect of pullout rate on monotonic uplift capacity.

After reaching capacity during a few 'virgin' monotonic pullout tests at the standard rate \( v_w = -0.3\) cm/min, and without reconsolidation, the caisson was pulled at the faster rate \( v_w = -0.03\) cm/min to complete extraction. This same procedure was employed during the second monotonic pullout phase for some tests. In most of the tests, the faster rate mobilizes \( \Delta F_{\text{tot}} = -2\) to \(-3\) kg over the total force measured at the end of standard rate pullout (Figures 5.39 and 5.72). This represents a total force increase of between 11 and 20\%, depending on the test. The strain softening behavior that follows the fast rate peak load is similar to that for the standard rate pullout. The increase in caisson capacity is attributed to an increased cap resistance. Pore pressure measurements indicate that increasing the pullout rate induces larger negative excess pore pressure within the soil plug by \( \Delta u = -0.2\) to \(-0.25\) ksc over the standard rate pullout (Figures 5.75-5.76). Large soil surface settlement near the wall \( (\delta_z = 0.2\) cm) indicates that the soil mass at this location is participating in the general failure as the caisson moves at a faster rate (Figure 5.77).

**Drained versus Undrained Capacity**

Critical to the design of a TLP suction anchor is an understanding of the anchor response to sustained tensile loads. Heavy storms lasting hours, even days, apply large cyclic tensile loads to the anchor, which must rely on underbase suction and caisson wall skin friction to resist such loads. With time, the negative excess pore pressure generated by the suction may dissipate as a result of pore fluid drainage in the surrounding soil. Hence, the underbase suction component is relieved as load is transferred to the wall. The
CET testing program simulated sustained tensile loading of suction anchors with a series of 6 single stage and multi-stage sustained load tests on the two-component model caisson.

Because the suction caisson cannot rely on underbase suction as a component of resistance to sustained tensile load, the drained capacity is lower than the undrained capacity. In most multi-stage sustained load tests, when failure occurred, the applied total load was carried almost entirely by the wall (Figure 6.33). The limiting wall force in these tests ($F_w = -10$ to $-13$ kg) matched closely the maximum wall force measured during virgin undrained pullout ($F_w = -11$ to $-15.2$ kg). In these tests, the drained total capacity was 44 to 57% of the average undrained total capacity ($F_{tot} = -22.9$ kg). In one multi-stage sustained load test (CET11), the cap maintained some resistance to uplift ($F_c = -2$ kg), while the limiting wall resistance ($F_w = -7$ kg) was much lower than that measured during undrained pullout. In this test the drained capacity was nearly 40% of the average undrained capacity. Based on the variation in measured limiting wall resistance, $F_w = -10.8 \pm 1.7$ kg, there remains some uncertainty in the long-term capacity of suction caissons in the CET cell. However, all of the sustained load tests showed that the drained capacity was never more than 57% of the undrained capacity.

7.2 CONCLUSIONS

Suction caissons only recently have been developed as permanent anchorage systems for deep water tension leg platforms (TLP). Therefore, there are many significant geotechnical uncertainties germane to suction caisson foundation design. The overall goal of the research described herein was to whittle down and remove some of these uncertainties by revealing the fundamental response of suction caisson installation, set-up, and tensile loading. Major contributions toward this goal were achieved by conducting a
test program using a new automated laboratory device, the Caisson Element Test (CET) cell, which simulates a cylindrical model caisson installed in a uniform clay sample with a controlled stress history. The following list summarizes the main findings of this research pertaining to the CET cell and the major aspects of suction caisson performance revealed by the CET testing program.

**CET Cell Design**

1) The caisson element test apparatus allows reliable, well-controlled experiments on a miniature pile installed in a cohesive sample with controlled stress history. The two-component caisson accurately models installation by underbase suction and allows independent measurement and control of the cap and wall throughout all phases of testing. Instrumentation provides accurate measurements of caisson force and displacement, chamber air pressure, soil surface displacements, and soil sample pore pressures.

**Installation**

2) Caissons installed with underbase suction and by 'jacking' exhibit three distinct zones of wall force behavior: a) shallow penetration ($\delta_w \leq 0.2$ cm - approximately the wall thickness) characterized by a very stiff wall response ($\Delta F_w \approx 10$ kg), b) transitional penetration ($0.2 < \delta_w < 1.8$ cm) where the wall force is variable due to caisson intercomponent friction, and c) deep penetration ($1.8 < \delta_w < 5.1$ cm) where the wall force increases at a constant rate with depth.

3) Tip bearing analysis of early penetration yields an average net tip resistance, $(q_{ult} - \sigma_{vc})/\sigma'_{vc}=5.13 \pm 0.16$ for installation by underbase suction and $(q_{ult} - \sigma_{vc})/\sigma'_{vc}=6.30$ for 'jacked' installation. These values are 1.3 to 2.5 times greater than for closed-ended penetrometer geometries considered in previous experimental and theoretical investigations on lightly overconsolidated Boston Blue Clay (Morrison, 1984; Ladd, 1991;
Aubeny, 1992). The backcalculated tip bearing capacity factor averages \( N_c = 16.0 \pm 0.5 \) for suction installation tests and \( N_c = 19.7 \) for jacked installation.

4) Constant wall force gradient \( (f_w) \) during deep penetration can be equated to the limiting wall skin friction \( (f_s = f_w / A_w) \) mobilized along the inner and outer wall surfaces. This leads to an average skin friction ratio, \( \beta = f_s / \sigma'_{vc} = 0.063 \pm 0.005 \) for suction-installed tests and \( \beta = 0.092 \) for jacked installation. These values are much lower than the undrained shear strength ratio of \( K_0 \)-normally consolidated RBBC (\( s_d / \sigma'_{vc} = 0.205 \); Ladd, 1991), but are in good agreement with previous predictions for undrained deep penetration of open and closed-ended piles in BBC (Whittle and Baligh, 1988).

5) Pore pressure measurements indicate that most, if not all, of the original effective stress \( (\sigma'_{vc} = 0.75 \text{ ksc}) \) within the soil plug is lost by the time the caisson reaches deep penetration zone for both installation by underbase suction and jacking.

6) Large upward vertical displacements of the cap \( (\delta_c = 0.5 \text{ to } 0.6 \text{ cm}) \) during suction and jacked installation balance very closely the volume of soil displaced by the wall, while the surrounding soil surface does not displace significantly \( (\delta_s \leq 0.004 \text{ cm}) \). An analysis of the stress conditions imposed on the soil and the resulting soil plug displacements reveal that the plug effective stress and strain behavior is similar to that for undrained shear for a clay specimen subject to triaxial extension.

**Equilibration**

7) Suction installation \( (F_{tot} = 15.2 \text{ kg during installation and set-up}) \) generates negligible installation pore pressures, which dissipate within 3 minutes of the start of the equilibration phase. The release of total load surcharge acquired during jacked installation \( (F_{tot} = 33.6 \text{ kg at installation end}; F_{tot} = 15.2 \text{ kg during set-up}) \) decreases the large
installation pore pressures, but residual pore pressures require $t=200$ to 300 minutes to dissipate. This dissipation time is consistent with a coefficient of consolidation, $c_v=0.002$ to 0.003 cm$^2$/sec, which is similar to previous data on RBBC (Seah, 1990; Sheahan, 1991). Regardless of the installation method, by the time installation pore pressures have dissipated, the caisson has redistributed most of the total load to the wall ($F_{w-avg}=95\%F_{tot}$ for post-suction installation set-up; $F_{w}=85\%F_{tot}$ for post-jacked installation set-up).

8) Caisson settlement after $t=24$ hours of post-installation set-up is minimal (10-65\% of wall thickness). Beyond a certain point in time ($t=70$ to 100 minutes), the caisson and clay surface settlement rate are equivalent, proportional to $\log t$, and therefore caused by secondary compression. Estimated rates of secondary compression average $C_{ces}=d\varepsilon/d(\log t)=0.0034\pm0.0016$, which yields a ratio, $C_{ces}/CR=0.028\pm0.013$. This is lower than the value estimated for inorganic soft clays (Mesri & Castro, 1987), but is well within the range reported previously for RBBC (Sheahan, 1991).

9) During post-tensile loading re-equilibration, the caisson forces redistribute in the same fashion as during post-installation set-up; by the time excess pore pressures have dissipated, the wall carries most of the total load ($F_{w}>90\%F_{tot}$). However, final re-equilibration caisson settlements ($\delta_s=-0.18$ to -0.37 cm) were 2 to 4 times greater than the average post-installation set-up caisson settlement. The increased settlement is due to the disturbed soil zone below the wall tip created by caisson withdrawal during tensile loading.

*Monotonic Pullout*

10) Maximum capacity for 'virgin' pullout averages $F_{tot}=-22.9\pm0.9$ kg at a caisson displacement of $\delta_w=0.25\pm0.03$ cm. At this displacement, the wall resistance contributes
an average of 58% of the total capacity, while the cap contributes 42%. The maximum wall resistance is mobilized at a displacement, $\delta_w=0.15$ to 0.25 cm, slightly before total capacity mobilization. The average peak cap force occurs at a displacement, $\delta_c=0.23$ to 0.28 cm, nearly coinciding with total capacity mobilization.

11) The caisson exhibits a very stiff initial response, starting from a compressive load, $F_{tot}=15.2$ kg, and acquiring $\Delta F_{tot}=-10$ to -20 kg within $\delta_w=0.002$ cm of pullout. After reaching a well-defined yield point between $\delta_w=0.01$ and 0.02 cm ($F_{tot}=-9$ to -14 kg), the caisson continues to pick up load to capacity, but at a rapidly decreasing rate. For displacements beyond peak up to $\delta_w=0.4$ cm, there is only slight post-peak softening. Initial wall stiffness ($\Delta F_w=-7$ to -13 kg within $\delta_w=0.002$ cm) is lower than that of the overall caisson, as the wall starts from a compressive load near $F_w=15$ kg and reaches yield ($\delta_w=0.01$-0.02 cm) at $F_w=-6$ to -12 kg. The cap shows high initial stiffness ($\Delta F_c=-4$ to -7 kg within $\delta_c=0.001$ cm) and then yields at $\delta_c=0.003$ cm.

12) Large negative excess pore pressures are generated within the soil plug, with peak excess pore pressure ($\Delta u_{max}=-0.4$ to -0.7 ksc) mobilized at nearly the same displacement ($\delta_c=0.2$ cm) as peak cap force. Outside the caisson, probes initially measure negative excess pore pressures (up to $\Delta u=-0.4$ ksc), but with continued extraction ($\delta_w=0.02$ cm), the exterior pore pressure stabilizes to levels of $\Delta u=\pm 0.1$ ksc (i.e., near zero).

13) The soil surface outside the caisson is relatively unaffected by monotonic pullout. While the soil near the caisson ($r=4.2$ to 5.2 cm) heaves slightly ($\delta_s=0.001-0.003$ cm) during the initial phase of loading, by the end of pullout ($\delta_w=0.3$ cm), the entire soil surface has compressed a small amount ($\delta_s<0.015$ cm; $\delta_s/\delta_w<4\%$).
14) Assuming the resistance comprises external wall skin friction and reverse end bearing, the best estimate of uplift capacity, \( F_b = F_{esf} + F_{reb} = -30.8 \) kg, overpredicts by 35% the average measured total capacity, \( F_{tot} = F_w + F_c = -22.9 \pm 0.9 \) kg. The average computed wall friction, \( F_{esf} = -11.95 \pm 0.10 \) kg compares very well with the average measured wall resistance, \( F_w = -13.2 \pm 2.1 \) kg (\( F_{esf} > 90\% F_w \)). However, the estimated reverse end bearing, \( F_{reb} = -18.9 \) kg, overpredicts by 50% the average measured cap resistance, \( F_c = -9.65 \pm 1.43 \) kg.

15) For caissons that had been axially loaded in tension and then re-equilibrated with the compressive load, \( F_{tot} = 15.2 \) kg, the total capacity during the second pullout averaged \( F_{tot} = -27.2 \pm 2.4 \) kg, which is 19% higher than virgin capacity. The higher capacity is caused by increased wall resistance, which is due to radial consolidation of the zone of clay surrounding the wall during re-equilibration.

16) Increasing the pullout rate by an order of magnitude \((v_w = -0.03 \text{ to } -0.3 \text{ cm/min})\) during post-peak monotonic pullout mobilizes an additional \( \Delta F_{tot} = -2 \) to \(-3 \) kg over the total force measured during the slower rate. This represents an increase of 11 to 20%, which can be attributed to an increased cap resistance. Increasing the pullout rate induces larger negative excess pore pressures within the soil plug by \( \Delta u = -0.2 \) to \(-0.25 \) ksc over values measured at the end of the slower pullout.

**Sustained Load**

17) Small total tensile load application \((F_{tot} = -2 \text{ to } -7 \) kg; much less than the limiting wall resistance measured during undrained pullout\) on a caisson that originally is under a compressive equilibrium load \((F_{tot} = 15.2 \) kg\) causes a stable response. The cap initially carries most of the applied load, but rapidly sheds this within 10 to 100 minutes, and carries small load \((F_c < \pm 2 \) kg\). Negative excess pore pressures \((\Delta u > -0.4 \) ksc\) generated by
the tensile load application generally dissipate within 100 minutes. Additional increments of total tension that do not lead to failure generally are carried by the wall and do not generate excess pore pressure.

18) Large initial total tensile loads that are approximately equal to or greater than the limiting wall resistance \( F_w = -11 \) to \(-15.2 \) kg during virgin pullout) cause caisson failure. The cap initially carries most of the applied tension, sheds some, but retains a significant percentage (30% in CET10), as the caisson rapidly displaces upward. Likewise, large soil plug excess pore pressures \( \Delta u = -0.5 \) to \(-0.6 \) ksc) dissipate to a certain level \( \Delta u = -0.2 \) to \(-0.3 \) ksc) that is held throughout failure as the mechanisms of dissipation and generation (due to caisson displacement) become balanced.

19) The drained total caisson capacity, as measured by the maximum applied tensile load in multi-stage sustained load tests, ranged from 40 to 57% of the average undrained total capacity \( F_{tot} = -22.9 \) kg). In most of the tests, the suction caisson cannot rely on underbase suction as a component of resistance, as any generated excess pore pressures are allowed to dissipate. Therefore, the wall provides most of the resistance. The limiting wall force in most tests \( F_w = -10 \) to \(-13 \) kg) matches closely the wall contribution during virgin undrained pullout \( F_w = -11 \) to \(-15.2 \) kg). However, some uncertainty remains regarding the components of resistance in drained capacity. In one test (CET11), the cap maintained some resistance \( F_c = -2 \) kg), and the wall resistance \( F_w = -7 \) kg) was lower than the undrained limiting wall resistance.
7.3 RECOMMENDATIONS FOR FUTURE WORK

The successive phases of installation by underbase suction, set-up, and axial tensile loading for suction caisson poses a complex soil-structure interaction problem. Within the framework of experimental research, this thesis has addressed many of the geotechnical uncertainties pertaining to suction anchor design for tension leg platform applications (see section 2.1.3). However, some answers are still incomplete, and many new questions have been raised. Based on the experience gained during this research, the following paragraphs provide suggestions for additional test interpretation, improving the Caisson Element Test (CET) cell, additional CET testing, and new tests to remove uncertainties not addressed in this research.

Additional CET Test Interpretation

1) There is still some uncertainty regarding the interpretation of the measured data arising from the CET test program conducted for this research. Further investigation, including additional finite element and other numerical analyses, would help to clarify the components of penetration resistance and soil plug behavior during caisson installation and the components of capacity and load-deformation response during axial tensile loading.

CET Cell Improvements

2) The current surface drainage system does not provide efficient drainage from the top of the clay sample surface to the atmosphere. A reliable surface drainage system would allow true double drainage (top and bottom) conditions in the CET consolidation chamber, which would reduce consolidation times and allow accurate interpretation of pore pressure dissipation during sustained load and post-tensile loading set-up phases.
3) Model caisson friction, while greatly reduced since early testing, should be lessened even more in order to enhance control and improve test interpretation, particularly the transition zone from early to deep penetration. Providing better control over caisson verticality would lessen the external source of friction (between the wall and inner slip ring), while improving the O-ring connection between the cap and wall would reduce the internal model friction.

4) Exterior and interior wall strain gauges, would provide information regarding the soil shear stress changes throughout testing. Such data would clarify the soil plug state of stress during penetration and allow evaluation of plug movements during pullout.

5) Development of a method to measure displacement of the top (and perhaps bottom) of the soil plug would verify cap/soil contact and yield additional plug strain development.

6) Improvement of the pullout software algorithm is required before conducting reliable fast rate pullout tests. The current algorithm is not sophisticated enough to maintain zero relative displacement between the cap and wall during pullout at rapid rates.

7) The O-ring that separates the wall and cap should be moved to a position closer to the cap base in order to reduce the volume of trapped air between annulus between the wall and cap. The existence of trapped air possibly leads to soil/cap separation as the pressure drops during pullout.

8) The drive system and support structure should be altered to allow easy take-down and excavation of the soil mass without disturbing the caisson or soil. This would allow thorough examination of the final caisson and soil geometry.
Clarification CET Tests

9) A 'jacked' penetration test similar to CET13, but with no total stress on the clay surface (free stress field conditions) would help identify the cause of large cap heave observed during penetration in CET13 and all suction installation tests.

10) Performing more tests similar to CET14 (which suffered from control problems) to simulate the process of fixed-piston sampling. These data would help to clarify the behavior of the soil plug during caisson installation and would also have direct impact on the understanding of sampling disturbance problems.

11) Conducting a standard rate pullout capacity test with pore pressure probes located below the caisson soil plug would provide soil stress information to clarify uncertainty in the reverse end bearing analysis.

12) Performing pullout tests with a monolithic (one-piece) model caisson identical in geometry to the existing two-component model may verify the maximum available cap resistance (many previous investigations indicate a higher cap contribution percentage during pullout). Because of the difficult logistics involved in consolidating the clay sample with a monolithic caisson in the CET cell, these tests could be performed with free stress field conditions, or consolidation could proceed with the caisson already in place.

13) Additional sustained load tests would help verify the limiting wall resistance during drained pullout.

New CET Tests

14) The original CET testing program scheme called for parametric studies involving different model geometries. The effect of geometry on caisson performance can be
addressed by penetrating the current model deeper to investigate L/D ratio, using models with different wall thicknesses to look at D/tw ratio, and testing multi-cellular models. The CET cell can be easily adapted to handle larger single cell diameters, but large multi-cellular models may require a larger consolidation chamber.

15) The effect of soil properties (overconsolidated RBBC and other resedimented clays) can be investigated very easily in the CET cell. The effect of a soil profile with an effective stress gradient from top to bottom would be a particularly useful model for prototype soil conditions. However, modifications to the cell drainage system would be required to create a soil sample with a stress gradient.

16) Delays during caisson penetration are common conditions for prototype suction anchor installations. The effect of delays on penetration resistance can be checked very simply in the CET cell.

17) CET tests that install the caisson with additional surcharge (F_{tot}), which then is maintained throughout set-up prior to pullout, would more closely simulate prototype conditions, wherein the caisson is allowed to set-up under its own self-weight. The effect of installation disturbance on tensile load behavior could be checked by varying the amount of surcharge load.

18) All of the CET tests in this research allowed complete pore pressure dissipation during caisson set-up. Additional tests that allow only partial dissipation would enable evaluation of this effect on pullout resistance.

19) New CET tests should be conducted to evaluate the effect of pullout rate.
20) The current CET apparatus cannot effectively apply cyclic loading, although cycling with large periods (T>10 seconds) are possible with minimal adjustment. CET tests with cyclic tensile load histories of varying frequency, stress amplitude, and number of cycles would more closely approximate prototype loading conditions. However, the CET wall and cap driving systems would require replacement (with sophisticated hydraulic actuators). The existing CET cell manages only vertical axisymmetric caisson movement. Eccentric loading would be particularly useful, but this would require major CET cell modification.
REFERENCES


Ukritchon, B. (1996) PhD in progress, Department of Civil & Environmental Engineering, MIT, Cambridge, MA.


APPENDIX A
INDIVIDUAL CET TEST
GEOMETRY AND INSTRUMENTATION
**Common Geometry**
Clay Cake Diameter = 30.5cm  
Clay Cake Height = 12.1-14.3cm  
Caisson Outside Diameter = 5.08cm  
Caisson Wall Thickness = 0.145cm  
Caisson Penetration = 5.08cm

**Common Instrumentation**
Caisson Wall Force, L1  
Caisson Cap Force, L2  
Caisson Wall Displacement, D1  
Caisson Cap Displacement, D2  
Chamber Air Pressure, AP  
Caisson Cap Pore Pressure, CP

**Clay Height and Other Instrumentation**

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<th>Pore Pressure Probes</th>
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Notes:

- $H_c$ = clay height prior to driving
- $\sigma_H$ = total stress transducer along chamber sidewall in CET1,2,6
- $d_1$ = depth from clay surface prior to driving
- $d_2$ = height from clay bottom prior to driving
- X = transducer in use for test
- X* = these probes located 2cm above clay bottom

Table A.1 Individual Test Geometry and Instrumentation
## CET 1

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<tr>
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$\sigma_n$ Total Stress (chamber sidewall)

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**Figure A.1** CET1 Chamber Geometry and Instrumentation
CET 2

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Figure A.2  CET2 Chamber Geometry and Instrumentation
CET 3

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Figure A.3 CET3 Chamber Geometry and Instrumentation
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Figure A.4  CET4 Chamber Geometry and Instrumentation
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Figure A.5  CET5 Chamber Geometry and Instrumentation
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**Figure A.6** CET6 Chamber Geometry and Instrumentation
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Figure A.7  CET7 Chamber Geometry and Instrumentation
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**Figure A.8**  
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CET9 Chamber Geometry and Instrumentation
**CET 10**

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### CET 11

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CET 12

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CET 14

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Figure 14 CET!4 Chamber Geometry and Instrumentation
APPENDIX B

ADDITIONAL RESEDIMENTED BOSTON BLUE CLAY PROPERTIES

This appendix presents two sets of data on resedimented Boston Blue Clay: 1) a database of index properties from previous MIT research on RBBC, and 2) tabulated data for each consolidometer test on Series IV RBBC that was used for this research program. The data herein serves as a supplement to the discussion in section 3.2 of index and engineering properties of resedimented Boston Blue Clay.

Table B.1 lists index properties of Series I through III RBBC batches that were used in 15 MIT research programs dating back to 1961. Included in this list are specific gravity $G_s$, Atterberg limits ($w_l$, $w_p$, $I_p$), clay fraction, and salt concentration. Specific gravity has varied little over the years, as the range is only $G_s=2.75$ to 2.78. More scatter can be found in the Atterberg limits. The liquid limit ranges from $w_l=30.0$ to 47.6, the plastic limit from $w_p=17.5$ to 24.9, and the plasticity index from $I_p=12.5$ to 24.3. Although these ranges may seem large, the resulting soil classification for all of the RBBC used is CL$^1$, which is a low to medium plasticity clay. Figure B.1a shows a plasticity chart with data from the 15 researchers cited in Table B.1. The clay fraction, or percentage (by weight) of RBBC particles less than 2 microns in diameter, ranged from 35 to 58%. More information on RBBC particle size is shown in Figure B.1b, which shows the grain size distribution for 9 tests on Series III BBC. Note that only one of the tests was conducted on resedimented BBC (RBBC201 by Seah, 1990), while the remaining 8 were conducted on BBC powder. Figure B.1 indicates that the fine fraction (% by weight less than #200 sieve, or 74$\mu$m) for Series III BBC lies between 90 and 95%. Excepting the research

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$^1$CL=low to medium plasticity clay according to the Unified Soil Classification (Wagner, 1957).
conducted by Bailey (1961), the salt concentration varied from 8 to 24 g/l for Series I-III RBBC.

Tables B.3 through B.9 list the data arising from the consolidometer tests (RBBC 401, 404–411, 413–417) performed on Series IV RBBC for this research. As described in section 3.3, the consolidometer test was the second stage of the four stages comprising each CET test. Note that in the first four tests (RBBC 401, 404–406), the clay was incrementally consolidated using a load increment ratio of LIR=1 to a maximum stress of $\sigma'_{vm}=0.5$ ksc and rebounded to $\sigma'_{vc}=0.125$ ksc. In the remaining 10 tests (RBBC 407–411, 413–417), the clay was loaded to a maximum stress of $\sigma'_{vm}=0.5$ ksc before continued consolidation in the CET apparatus (stage 3). The individual tables include clay cake height $H$, water content $w$, void ratio $e$, coefficient of consolidation $C_v$, compression index $C_c$, coefficient of volume change $m_v$, and hydraulic conductivity $k_v$. The methods of calculation for these values are listed in Table B.2. Note that missing parameter entries in these tables indicate lack of sufficient data required to compute the parameter.

---

2Bailey (1961) studied the effect of salt concentration on engineering properties of RBBC.
3Salt is added to BBC powder during the batching procedure in order to replace the salt that is leached during BBC processing.
4Stage 1 is BBC resedimentation. Stage 2 is the consolidometer test (or consolidation using a rigid cap). Stage 3 is consolidation using the CET apparatus. Stage 4 is the model caisson testing event sequence.
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Table B.1  Index Properties of Resedimented Boston Blue Clay from Series I-III Boston Blue Clay (after Seah, 1990)
Consolidometer Test Data
Computation Table

1. Height of Solids, \( H_s = \frac{V_T}{(1 + e_T)} \), \( e_T = \frac{G_S w_f}{S} \)

2. Height of Sample, \( H = H_s (1 - e) \)

3. Void Ratio, \( e = \frac{(H - H_s)}{H_s} \)

4. Water Content, \( w = \frac{(S) e}{G_S} \)

5. Coefficient of Consolidation,
\[
c_v = \frac{0.848 \left( \frac{H_{so}}{2} \right)^2}{60(t_{so})} \quad \text{Root Time Method}
\]
\[
c_v = \frac{0.197 \left( \frac{H_{so}}{2} \right)^2}{60(t_{so})} \quad \text{Log Time Method}
\]

6. Compression Index,
\[
C_c = \frac{\Delta e_{100}}{\Delta \log \sigma'_{w}}
\]

7. Coefficient of Volume Change, \( m_v = \frac{0.435 (C_e)}{(1 + e_0) \sigma'_{w}} \), where \( e_0 = e_T \) from previous inc.

8. Hydraulic Conductivity, \( k_v = [c_v(\text{avg})] m_v \gamma_w \)

Table B.2 Equations Used to Compute Consolidometer Test Parameters
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**DONE BY:** DFC/JVS

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<td>11.957</td>
<td>11.904</td>
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<tr>
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<td>11.588</td>
<td>11.248</td>
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</tr>
<tr>
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<td>11.216</td>
<td>11.235</td>
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<tr>
<td>7</td>
<td>0.125</td>
<td>11.257</td>
<td>11.270</td>
<td>11.303</td>
</tr>
</tbody>
</table>

Height of Solids, $H_s$=4.717 cm

### CONSOLIDATION DATA SHEET: RBBC 404, CET2

**DATE:** 12/8/93  
**DONE BY:** DFC/JVS

<table>
<thead>
<tr>
<th>Inc. Number</th>
<th>$\sigma_{ve}$ (ksc)</th>
<th>Height of Sample, cm</th>
<th>Water Content, %</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
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<td>$H_{50}$</td>
<td>$H_{100}$</td>
<td>$H_{f}$</td>
<td>$w_{100}$</td>
</tr>
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<td>16.039</td>
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<td>15.283</td>
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<td>0.500</td>
<td>14.849</td>
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<td>14.416</td>
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Height of Solids, $H_s$=5.511 cm

### Table B.3 Consolidometer Test Results for RBBC 401,404
# Consolidation Data Sheet: RBBC 405, CET3

<table>
<thead>
<tr>
<th>Inc. Number</th>
<th>( \sigma'_{ve} ) (ksc)</th>
<th>Height of Sample, cm</th>
<th>Water Content, %</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
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<td>( H_0 )</td>
<td>( H_{100} )</td>
<td>( H_f )</td>
<td>( w_{100} )</td>
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<td>15.705</td>
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<td>0.125</td>
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<td>-</td>
<td>14.358</td>
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<td>4</td>
<td>0.265</td>
<td>13.836</td>
<td>13.311</td>
<td>13.270</td>
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<tr>
<td>5</td>
<td>0.500</td>
<td>12.881</td>
<td>12.493</td>
<td>12.456</td>
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<td>6</td>
<td>0.250</td>
<td>12.466</td>
<td>12.476</td>
<td>12.484</td>
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<tr>
<td>7</td>
<td>0.125</td>
<td>12.493</td>
<td>12.503</td>
<td>12.519</td>
</tr>
</tbody>
</table>

Height of Solids, \( H_a = 5.253 \) cm

# Consolidation Data Sheet: RBBC 406, CET4

<table>
<thead>
<tr>
<th>Inc. Number</th>
<th>( \sigma'_{ve} ) (ksc)</th>
<th>Time, min</th>
<th>Coefficient of Consolidation, cm²/s</th>
<th>( C_c )</th>
<th>( m_v ) (cm²/kg)</th>
<th>( k_v ) x10⁻⁴ (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Duration (hr)</td>
<td>( t_{90} )</td>
<td>( t_{50} )</td>
<td>( c_v )</td>
<td>( c_v ) logt</td>
<td>( c_v ) average</td>
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<td>-</td>
<td>0.000366</td>
<td>-</td>
</tr>
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<td>2</td>
<td>0.0625</td>
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<td>2379</td>
<td>565</td>
<td>0.000358</td>
<td>0.000358</td>
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<td>0.125</td>
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<td>0.000731</td>
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<td>239</td>
<td>26.7</td>
<td>4.9</td>
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<td>0.125</td>
<td>259</td>
<td>41.7</td>
<td>18.1</td>
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Height of Solids, \( H_a = 6.204 \) cm

Table B.4   Consolidometer Test Results for RBBC 405,406
### CONSOLIDATION DATA SHEET: RBBC 401, CET1

**DATE:** 6/11/93  
**DONE BY:** DFC/JVS

<table>
<thead>
<tr>
<th>Inc. Number</th>
<th>$\sigma'_{ve}$ (ksc)</th>
<th>Height of Sample, cm</th>
<th>Water Content, %</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$H_{50}$</td>
<td>$H_{100}$</td>
<td>$H_{r}$</td>
<td>$w_{100}$</td>
</tr>
<tr>
<td>1</td>
<td>15.698</td>
<td>14.506</td>
<td>14.408</td>
<td>74.97</td>
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<tr>
<td>3</td>
<td>13.165</td>
<td>12.813</td>
<td>12.798</td>
<td>61.76</td>
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<tr>
<td>4</td>
<td>12.400</td>
<td>11.957</td>
<td>11.904</td>
<td>55.10</td>
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<tr>
<td>5</td>
<td>11.588</td>
<td>11.248</td>
<td>11.192</td>
<td>49.55</td>
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<tr>
<td>6</td>
<td>11.216</td>
<td>11.235</td>
<td>11.243</td>
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</tr>
<tr>
<td>7</td>
<td>11.257</td>
<td>11.270</td>
<td>11.303</td>
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**Height of Solids, $H_r=4.717$ cm**

### CONSOLIDATION DATA SHEET: RBBC 404, CET2

**DATE:** 12/8/93  
**DONE BY:** DFC/JVS

<table>
<thead>
<tr>
<th>Inc. Number</th>
<th>$\sigma'_{ve}$ (ksc)</th>
<th>Time, min</th>
<th>Coefficient of Consolidation, cm²/s</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Duration (hr)</td>
<td>$t_{90}$</td>
<td>$t_{50}$</td>
<td>$c_v$</td>
</tr>
<tr>
<td>1</td>
<td>61</td>
<td>3283</td>
<td>655</td>
<td>0.000342</td>
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<tr>
<td>2</td>
<td>54</td>
<td>1664</td>
<td>324</td>
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<td>3</td>
<td>48</td>
<td>759</td>
<td>153</td>
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<td>261</td>
<td>775</td>
<td>167</td>
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<td>810</td>
<td>0.001154</td>
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<td>7</td>
<td>0.125</td>
<td>70</td>
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**Height of Solids, $H_r=5.511$ cm**

### Table B.3

Consolidometer Test Results for RBBC 401, 404
### CONSOLIDATION DATA SHEET: RBBC 407, CET5

<table>
<thead>
<tr>
<th>Inc. Number</th>
<th>$\sigma'_{ve}$ (ksc)</th>
<th>Height of Sample, cm</th>
<th>Water Content, %</th>
<th>Void Ratio</th>
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<td>$H_{100}$</td>
<td>$H_f$</td>
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<td>0.03125</td>
<td>0.0500</td>
<td>0.1400</td>
<td>0.0550</td>
</tr>
<tr>
<td>2</td>
<td>0.0625</td>
<td>0.0510</td>
<td>0.1410</td>
<td>0.0560</td>
</tr>
<tr>
<td>3</td>
<td>0.126</td>
<td>0.0520</td>
<td>0.1420</td>
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<tr>
<td>4</td>
<td>0.242</td>
<td>0.0530</td>
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<tr>
<td>5</td>
<td>0.491</td>
<td>0.0540</td>
<td>0.1440</td>
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Height of Solids, $H_s$ = 5.921 cm

### CONSOLIDATION DATA SHEET: RBBC 408, CET6

<table>
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<th>Inc. Number</th>
<th>$\sigma'_{ve}$ (ksc)</th>
<th>Time, min</th>
<th>Coefficient of Consolidation, cm²/s</th>
<th>$C_c$</th>
<th>$m_v$ (cm²/kg)</th>
<th>$k_v \times 10^4$ (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
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<td>$t_{100}$</td>
<td>$t_f$ logt</td>
<td>$c_v$ logt</td>
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<td>3280</td>
<td>937</td>
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<td>0.000379</td>
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<td>144</td>
<td>4107</td>
<td>1028</td>
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<td>0.000272</td>
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<tr>
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<td>63</td>
<td>2258</td>
<td>527</td>
<td>0.000442</td>
<td>0.000440</td>
</tr>
<tr>
<td>4</td>
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<td>1413</td>
<td>333</td>
<td>0.000615</td>
<td>0.000607</td>
</tr>
<tr>
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<td>57</td>
<td>923</td>
<td>220</td>
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<td>0.000803</td>
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Height of Solids, $H_s$ = 6.280 cm

**Table B.5** Consolidometer Test Results for RBBC 407,408
### CONSOLIDATION DATA SHEET: RBBC 409, CET7

**DATE:** 2/16/95  
**DONE BY:** DFC/SS

<table>
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<tr>
<th>Inc. Number</th>
<th>$\sigma_{ve}'$ (ksc)</th>
<th>Height of Sample, cm</th>
<th>Water Content, %</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$H_{50}$ $H_{100}$ $H_{r}$</td>
<td>$w_{100}$ $w_{r}$ $e_{50}$ $e_{100}$ $e_{r}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.03125</td>
<td>19.092 17.240 17.493</td>
<td>75.22 $76.85$ 2.448</td>
<td>2.114 2.159</td>
</tr>
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<td>0.0625</td>
<td>16.914 16.329 16.334</td>
<td>69.36 69.36    2.055</td>
<td>1.949 1.950</td>
</tr>
<tr>
<td>3</td>
<td>0.129</td>
<td>15.808 15.265 15.306</td>
<td>62.52 62.79    1.855</td>
<td>1.757 1.764</td>
</tr>
<tr>
<td>4</td>
<td>0.250</td>
<td>14.779 14.256 14.265</td>
<td>56.04 56.09    1.669</td>
<td>1.575 1.576</td>
</tr>
<tr>
<td>5</td>
<td>0.498</td>
<td>13.803 13.359 13.325</td>
<td>50.27 50.06    1.493</td>
<td>1.413 1.407</td>
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</table>

<table>
<thead>
<tr>
<th>Inc. No.</th>
<th>$\sigma_{ve}'$ (ksc)</th>
<th>Time, min</th>
<th>Coefficient of Consolidation, cm$^2$/s</th>
<th>$C_c$</th>
<th>$m_v$ (cm$^2$/kg)</th>
<th>$k_v$ x10$^{-8}$ (cm/s)</th>
</tr>
</thead>
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<td>-</td>
<td>-</td>
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<td>0.0625</td>
<td>70</td>
<td>2065 481 0.0000489 0.0000488 0.0000489</td>
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<td>1.603 78.4</td>
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<td>0.946 59.7</td>
<td>-</td>
</tr>
<tr>
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<td>0.624 79.6</td>
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<td>793</td>
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<td>0.542</td>
<td>0.245 27.3</td>
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Height of Solids, $H_s$=5.537 cm

### CONSOLIDATION DATA SHEET: RBBC 410, CET9

**DATE:** 3/26/95  
**DONE BY:** DFC/SS

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<th>Height of Sample, cm</th>
<th>Water Content, %</th>
<th>Void Ratio</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>$H_{50}$ $H_{100}$ $H_{r}$</td>
<td>$w_{100}$ $w_{r}$ $e_{50}$ $e_{100}$ $e_{r}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.03125</td>
<td>19.671 17.608 17.677</td>
<td>70.15 70.57    2.319</td>
<td>1.971 1.983</td>
</tr>
<tr>
<td>2</td>
<td>0.0625</td>
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<td>- - - - - - - - - - -</td>
<td>- - - - - - - - - - -</td>
</tr>
<tr>
<td>3</td>
<td>0.125</td>
<td>15.346 15.911 15.952</td>
<td>59.96 60.21    1.758</td>
<td>1.685 1.685</td>
</tr>
<tr>
<td>4</td>
<td>0.248</td>
<td>15.410 14.892 14.895</td>
<td>53.84 53.86    1.600</td>
<td>1.513 1.513</td>
</tr>
<tr>
<td>5</td>
<td>0.494</td>
<td>14.357 13.828 13.984</td>
<td>47.45 48.39    1.423</td>
<td>1.333 1.333</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Inc. No.</th>
<th>$\sigma_{ve}'$ (ksc)</th>
<th>Time, min</th>
<th>Coefficient of Consolidation, cm$^2$/s</th>
<th>$C_c$</th>
<th>$m_v$ (cm$^2$/kg)</th>
<th>$k_v$ x10$^{-8}$ (cm/s)</th>
</tr>
</thead>
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<td>-</td>
<td>-</td>
</tr>
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<td>2</td>
<td>0.0625</td>
<td>43</td>
<td>- - - - - - - - - - -</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
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<td>0.125</td>
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<td>1584 409 0.000596 0.000536 0.000566</td>
<td>-</td>
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<tr>
<td>4</td>
<td>0.248</td>
<td>44</td>
<td>1174 277 0.000715 0.000705 0.000710</td>
<td>0.575</td>
<td>0.499 35.4</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>0.494</td>
<td>2694</td>
<td>780 241 0.000933 0.000702 0.000818</td>
<td>0.599</td>
<td>0.280 22.9</td>
<td>-</td>
</tr>
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</table>

Height of Solids, $H_s$=5.926 cm

**Table B.6** Consolidometer Test Results for RBBC 409,410
### CONSOLIDATION DATA SHEET: RBBC 411, CET8

**DATE:** 6/18/95  
**DONE BY:** DFC

<table>
<thead>
<tr>
<th>Inc. Number</th>
<th>$\sigma'_{ve}$ (ksc)</th>
<th>Height of Sample, cm</th>
<th>Water Content, %</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$H_{50}$</td>
<td>$H_{100}$</td>
<td>$H_{f}$</td>
</tr>
<tr>
<td>1</td>
<td>0.03125</td>
<td>-</td>
<td>-</td>
<td>18.406</td>
</tr>
<tr>
<td>2</td>
<td>0.0625</td>
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</tr>
<tr>
<td>3</td>
<td>0.125</td>
<td>-</td>
<td>-</td>
<td>16.699</td>
</tr>
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<td>0.248</td>
<td>-</td>
<td>-</td>
<td>15.319</td>
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<tr>
<td>5</td>
<td>0.502</td>
<td>14.722</td>
<td>14.128</td>
<td>13.975</td>
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</table>

Height of Solids, $H_s = 5.773$ cm

### CONSOLIDATION DATA SHEET: RBBC413, CET10

**DATE:** 8/10/95  
**DONE BY:** DFC/AJV

<table>
<thead>
<tr>
<th>Inc. Number</th>
<th>$\sigma'_{ve}$ (ksc)</th>
<th>Height of Sample, cm</th>
<th>Water Content, %</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$H_{50}$</td>
<td>$H_{100}$</td>
<td>$H_{f}$</td>
</tr>
<tr>
<td>1</td>
<td>0.03125</td>
<td>21.042</td>
<td>19.052</td>
<td>19.056</td>
</tr>
<tr>
<td>2</td>
<td>0.0625</td>
<td>-</td>
<td>-</td>
<td>17.376</td>
</tr>
<tr>
<td>3</td>
<td>0.125</td>
<td>-</td>
<td>-</td>
<td>16.416</td>
</tr>
<tr>
<td>4</td>
<td>0.243</td>
<td>15.925</td>
<td>15.443</td>
<td>15.431</td>
</tr>
<tr>
<td>5</td>
<td>0.537</td>
<td>14.889</td>
<td>14.352</td>
<td>14.349</td>
</tr>
</tbody>
</table>

Height of Solids, $H_s = 5.9315$ cm

### CONSOLIDATION DATA SHEET: RBBC413, CET10

**DATE:** 8/10/95  
**DONE BY:** DFC/AJV

<table>
<thead>
<tr>
<th>Inc. Number</th>
<th>$\sigma'_{ve}$ (ksc)</th>
<th>Time, min</th>
<th>Coefficient of Consolidation, cm²/s</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td>$t_{50}$</td>
<td>$t_{100}$</td>
<td>$t_{f}$</td>
</tr>
<tr>
<td>1</td>
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<td>114</td>
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<td>25</td>
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<td>-</td>
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<tr>
<td>3</td>
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<td>20</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
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<td>27</td>
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</tr>
<tr>
<td>5</td>
<td>0.537</td>
<td>71</td>
<td>516</td>
<td>142</td>
</tr>
</tbody>
</table>

Height of Solids, $H_s = 5.773$ cm

Table B.7 Consolidometer Test Results for RBBC 411,413
### CONSOLIDATION DATA SHEET: RBBC414, CET11

**DATE:** 8/17/95  
**DONE BY:** DFC/AJV

<table>
<thead>
<tr>
<th>Inc. No.</th>
<th>$\sigma_{tv}$ (ksc)</th>
<th>Height of Sample, cm</th>
<th>Water Content, %</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$H_{50}$</td>
<td>$H_{100}$</td>
<td>$H_f$</td>
</tr>
<tr>
<td>1</td>
<td>0.03125</td>
<td>-</td>
<td>-</td>
<td>19.815</td>
</tr>
<tr>
<td>2</td>
<td>0.0625</td>
<td>-</td>
<td>-</td>
<td>16.543</td>
</tr>
<tr>
<td>3</td>
<td>0.125</td>
<td>15.854</td>
<td>15.156</td>
<td>15.165</td>
</tr>
<tr>
<td>4</td>
<td>0.249</td>
<td>14.690</td>
<td>14.225</td>
<td>14.223</td>
</tr>
<tr>
<td>5</td>
<td>0.498</td>
<td>13.757</td>
<td>13.294</td>
<td>13.259</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Inc. No.</th>
<th>$\sigma_{tv}$ (ksc)</th>
<th>Time, min</th>
<th>Coefficient of Consolidation, cm²/s</th>
<th>$C_c$ (cm²/kg)</th>
<th>$m_v$ x10⁴ (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Duration (hr)</td>
<td>$t_{50}$ Vt</td>
<td>$t_{50}$ log t</td>
<td>$c_v$ Vt</td>
</tr>
<tr>
<td>1</td>
<td>0.03125</td>
<td>44</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>0.0625</td>
<td>47</td>
<td>3649</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>0.125</td>
<td>53</td>
<td>1312</td>
<td>333</td>
<td>0.000677</td>
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<tr>
<td>4</td>
<td>0.249</td>
<td>47</td>
<td>884</td>
<td>220</td>
<td>0.000863</td>
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<td>0.498</td>
<td>413</td>
<td>547</td>
<td>183</td>
<td>0.001222</td>
</tr>
</tbody>
</table>

Height of Solids, $H_s$ = 5.668 cm

### CONSOLIDATION DATA SHEET: RBBC415, CET12

**DATE:** 9/7/95  
**DONE BY:** DFC/AJV

<table>
<thead>
<tr>
<th>Inc. No.</th>
<th>$\sigma_{tv}$ (ksc)</th>
<th>Height of Sample, cm</th>
<th>Water Content, %</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$H_{50}$</td>
<td>$H_{100}$</td>
<td>$H_f$</td>
</tr>
<tr>
<td>1</td>
<td>0.03125</td>
<td>-</td>
<td>-</td>
<td>18.966</td>
</tr>
<tr>
<td>2</td>
<td>0.0625</td>
<td>18.173</td>
<td>17.387</td>
<td>17.379</td>
</tr>
<tr>
<td>3</td>
<td>0.127</td>
<td>16.849</td>
<td>16.276</td>
<td>16.298</td>
</tr>
<tr>
<td>4</td>
<td>0.250</td>
<td>15.758</td>
<td>15.220</td>
<td>15.234</td>
</tr>
<tr>
<td>5</td>
<td>0.498</td>
<td>14.781</td>
<td>14.327</td>
<td>14.306</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Inc. No.</th>
<th>$\sigma_{tv}$ (ksc)</th>
<th>Time, min</th>
<th>Coefficient of Consolidation, cm²/s</th>
<th>$C_c$ (cm²/kg)</th>
<th>$m_v$ x10⁴ (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Duration (hr)</td>
<td>$t_{50}$ Vt</td>
<td>$t_{50}$ log t</td>
<td>$c_v$ Vt</td>
</tr>
<tr>
<td>1</td>
<td>0.03125</td>
<td>93</td>
<td>5239</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>0.0625</td>
<td>156</td>
<td>2954</td>
<td>728</td>
<td>0.000395</td>
</tr>
<tr>
<td>3</td>
<td>0.127</td>
<td>47</td>
<td>2135</td>
<td>503</td>
<td>0.000470</td>
</tr>
<tr>
<td>4</td>
<td>0.250</td>
<td>49</td>
<td>1124</td>
<td>290</td>
<td>0.000781</td>
</tr>
<tr>
<td>5</td>
<td>0.498</td>
<td>197</td>
<td>713</td>
<td>191</td>
<td>0.001083</td>
</tr>
</tbody>
</table>

Height of Solids, $H_s$ = 5.904 cm

---

Table B.8  Consolidometer Test Results for RBBC 414,415
<table>
<thead>
<tr>
<th>Inc. No.</th>
<th>( \sigma'_{ve} ) (ksc)</th>
<th>Height of Sample, cm</th>
<th>Water Content, %</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( H_{50} )</td>
<td>( H_{100} )</td>
<td>( H_f )</td>
</tr>
<tr>
<td>1</td>
<td>0.03125</td>
<td>-</td>
<td>-</td>
<td>14.663</td>
</tr>
<tr>
<td>2</td>
<td>0.0625</td>
<td>19.165</td>
<td>23.646</td>
<td>25.491</td>
</tr>
<tr>
<td>3</td>
<td>0.127</td>
<td>27.242</td>
<td>29.968</td>
<td>29.050</td>
</tr>
<tr>
<td>4</td>
<td>0.249</td>
<td>31.456</td>
<td>33.809</td>
<td>33.949</td>
</tr>
<tr>
<td>5</td>
<td>0.498</td>
<td>35.937</td>
<td>37.911</td>
<td>37.060</td>
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</table>

**Height of Solids, \( H_s = 5.589 \) cm**

<table>
<thead>
<tr>
<th>Inc. No.</th>
<th>( \sigma'_{ve} ) (ksc)</th>
<th>Time, min</th>
<th>Coefficient of Consolidation, cm²/s</th>
<th>Cc</th>
<th>( m_v ) (cm²/kg)</th>
<th>( k_v ) x10⁻⁸ (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Duration (hr)</td>
<td>( t_{90} ) VT</td>
<td>( t_{50} ) logt</td>
<td>( c_v ) vt</td>
<td>( c_v ) logt</td>
</tr>
<tr>
<td>1</td>
<td>0.03125</td>
<td>43</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>0.0625</td>
<td>722</td>
<td>2540</td>
<td>657</td>
<td>0.000409</td>
<td>0.000368</td>
</tr>
<tr>
<td>3</td>
<td>0.127</td>
<td>43</td>
<td>3645</td>
<td>510</td>
<td>0.000231</td>
<td>0.000384</td>
</tr>
<tr>
<td>4</td>
<td>0.249</td>
<td>71</td>
<td>964</td>
<td>247</td>
<td>0.000775</td>
<td>0.000703</td>
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<tr>
<td>5</td>
<td>0.498</td>
<td>267</td>
<td>707</td>
<td>177</td>
<td>0.000923</td>
<td>0.000857</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Inc. No.</th>
<th>( \sigma'_{ve} ) (ksc)</th>
<th>Height of Sample, cm</th>
<th>Water Content, %</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( H_{50} )</td>
<td>( H_{100} )</td>
<td>( H_f )</td>
</tr>
<tr>
<td>1</td>
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<td>-</td>
<td>-</td>
<td>8.300</td>
</tr>
<tr>
<td>2</td>
<td>0.0625</td>
<td>15.756</td>
<td>-</td>
<td>23.778</td>
</tr>
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<td>0.127</td>
<td>-</td>
<td>28.300</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>0.251</td>
<td>-</td>
<td>33.338</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>0.497</td>
<td>35.459</td>
<td>37.579</td>
<td>37.728</td>
</tr>
</tbody>
</table>

**Height of Solids, \( H_s = 5.983 \) cm**

*Coefficient of Consolidation (c_v) computed assuming double drainage and single drainage (in parentheses).*
Figure B.1  Index Properties of Resedimented Boston Blue Clay
a) Plasticity Chart with Data from Series I to III RBBC
b) Grain Size Distribution of Series III Boston Blue Clay
APPENDIX C
CONTROL SYSTEM SOFTWARE
10 ' edited on 8/02/91 by jtg
11 ' edited on 6/19/93 by dfc
13 PROGRAM$="MASTER rev 3.0"
15 '**********************************************************************
20 '
30 ' This is the master program for the computer controlled caisson element test
40 ' system at MIT. It contains the code for constants that are used throughout
50 ' the test.
70 '
80 ' These subroutines include:
90 '    moving of motors          lines 360-800, 1700-2150
100 '    reading of transducers   lines 2160-3000
110 '    menu functions           lines 800-1700
120 '
122 CLS :PRINT
123 PRINT " You are using the caisson element test control program ";PROGRAM$
124 PRINT
125 PRINT
130 '
131 PRINT "The following hardware is REQUIRED;"
132 PRINT " -Strawberrytree digital to analog converter"
133 PRINT " -the Sheahan data acquisition card"
134 PRINT " -the MIT three axis controller using Electro Craft motors and;"
135 PRINT " -channel 1 for caisson wall"
136 PRINT " -channel 2 for caisson cap"
137 PRINT " -channel 3 for chamber air pressure"
138 PRINT :PRINT
139 '
140 ' Written by Amy Chu and Tom Sheahan 1989.
141 ' Revised for DC servo motors by Jack Germaine 3/1990 (rev 2.0)
142 ' Revised for caisson element test by Doug Cauble 6/1993 (rev 3.0)
143 '
144 ' Use of this code without the expressed oral or written consent of
145 ' Tom Sheahan, Jack Germaine, or Doug Cauble is prohibited.
150 '
160 '
170 '**********************************************************************
180 '
210 KEY OFF: FOR I=1 TO 10: KEY I,"": NEXT I 'Disable F-keys
220 '
230 ' Define constants
240 INTTIME=22
242 AD1170=768 'the decimal I/O address of the AD1170
244 INTBIT=13 'to set the bit precision of the AD1170
246 AD1170KEY=0
247 VINCHANNEL=6 'input voltage pin connection
248 GRDCHANNEL=15 'multiplexer connection to ground
249 REFCHANNEL=14 '5.0 volt loaction on multiplexer
250 MUXI=776 'decimal I/O address of multiplexer
260 MOTOR$=6928 'decimal I/O address of analog out card
270 LOBIT0!=240 : HIBIT0!=7 'decimal value equal to zero volts (+/-5 range)
271 LOBITSF!=255 : HIBITSF!=255 'decimal value to step forward
272 LOBITSB! = 0 : HIBITSB! = 0 'decimal value to step backward
273 STEPTIME = 1 : GOSUB 345 'seconds for each step
275 LOBIT! = LOBIT0! : HIBIT! = HIBIT0!
280 DEVICE! = 0 : GOSUB 470 'set outputs to zero -stop motors
282 MDIR = 1 'SET MOTOR DIRECTIONS TO FORWARD
283 GOSUB 3500 'lock out keyboard
284 REM ** return for subroutine
285 OUT MOTORS! + 4, 7 'open relays 1, 2, 3 to unlock motors
300 '
310 MAXITEMS = 5 'not including "Exit"
315 DIM VOLTS(7), COUNTS(7) 'for use in the readings subroutine
320 DIM ITEMS$(MAXITEMS), DESCRIP$(MAXITEMS)
340 GOTO 1500 'Skip the subroutines; execute main program
344 '
345 '************ SET COUNTER TO DELAY STEP VOLTAGE ************
346 '
347 I = 1
348 ON TIMER(2) GOSUB 351 'Take two second sample
349 TIMER ON
350 I = I + 1 : GOTO 350
351 STEPINC = (I/2) * STEPTIME
352 TIMER OFF
353 RETURN 354
354 RETURN
359 '
360 '*************** SPEED CONTROL OF MOTORS **********************
362 '
364 'Slow the motors down
366 SPEED = SPEED/1.3
368 GOTO 376
370 '
372 'Speed the motors up
374 SPEED = SPEED*1.3
376 IF SPEED < .01 THEN SPEED = .01
377 IF SPEED > 5 THEN SPEED = 5
378 GOTO 530
380 '
384 'Subroutines to step and stop the motor(s) once *
390 '**********
400 '
422 'create one backward step
425 'IF DEVICE! = 14 THEN OUT MOTORS! + 4, 15 : GOTO 446 'close relay #4
430 OUT MOTORS!, LOBITSB!
440 OUT MOTORS! + 1, HIBITSB!
442 GOTO 450
444 'create one forward step
445 'IF DEVICE! = 14 THEN OUT MOTORS! + 4, 7 'open relay #4
446 OUT MOTORS!, LOBIFS!
447 OUT MOTORS! + 1, HIBIFS!
450 OUT MOTORS! + 2, DEVICE!
460 FOR I = 1 TO STEPINC : NEXT I 'delay
460 FOR I = 1 TO STEPINC : NEXT I
595

MASTER.BAS

465 'stop all motors
470 OUT MOTORS!,LOBIT0!
480 OUT MOTORS!+1,HIBIT0!
490 OUT MOTORS!+2,0   'stop all motors
492 OUT MOTORS!+2,255  'lock all output voltages
495 MOVES$="N"         'prevent movement in cont. mode
500 RETURN
505 '
510 ************** Subroutine to move one motor continuously **************
514 '
515 'move backward
516 MDIR=-1
517 ' IF DEVICE!=14 THEN MDIR=1 : OUT MOTORS!+4,15 'close relay #4
518 GOTO 525
519 'move forward
520 MDIR=1
521 ' IF DEVICE!=14 THEN OUT MOTORS!+4,7      'open relay #4
525 MOVES$="Y"
526 SPEED=1             'set basic speed
530 'convert volts to bits
531 IF MOVES$="Y" THEN RETURN
532 VOLTS=SPEED*MDIR
540 BITS$=INT((VOLTS+5)*409.5)   '12 bit =/- 5volt range
550 HIBIT!=INT(BITS$/256)
560 LOBIT!=BITS!-HIBIT!*256
570 OUT MOTORS!,LOBIT!
580 OUT MOTORS!+1,HIBIT!
590 OUT MOTORS!+2,DEVICE!
595 OUT MOTORS!+2,255
600 RETURN
605 '
610 ************** Subroutine to setup function keys **************
615 '
620 SPEED=1
625 KEY(1) ON:ON KEY(1) GOSUB 444   'forward
630 KEY(2) ON:ON KEY(2) GOSUB 422   'backward
635 KEY(3) ON:ON KEY(3) GOSUB 520   'rapid lowering
640 KEY(4) ON:ON KEY(4) GOSUB 515   'rapid raising
645 KEY(11) ON:ON KEY(11) GOSUB 374 'speed up with up arrow
650 KEY(14) ON:ON KEY(14) GOSUB 366 'slow down with down arrow
652 KEY 19,CHR$(0)+CHR$(&H39)
653 KEY(19) ON:ON KEY(19) GOSUB 465 'stop motors on space bar
656 KEY 20,CHR$(0)+CHR$(&H1C)
657 KEY(20) ON:ON KEY(20) GOSUB 710 'stop motors/return to main on enter
660 CLS
665 RETURN
670 '
675 ************** Subroutine to control movement ***********************
680 LOCATE 17,10: PRINT "Press ";CHR$(24);" to increase motor speed"
685 LOCATE 18,10: PRINT "Press ";CHR$(25);" to decrease motor speed
690 LOCATE 20,10: PRINT "Press space bar to stop movement"
LOCATE 21,10: PRINT "Hit <Enter> to return to main menu"
GOTO 700
return to main on enter
KEY (1) OFF: KEY(2) OFF : KEY(3) OFF: KEY(4) OFF : KEY(19) OFF : KEY(20) OFF
GOSUB 470
RETURN 730
RETURN

Generic subroutines related to menu functions
Activate keys for menu functions
ON KEY(12) GOSUB 930
ON KEY(13) GOSUB 1030
KEY 19,CHR$(0)+CHR$(&H1C)
ON KEY (19) GOSUB 875
KEY(12) ON: KEY(13) ON : KEY(19) ON
RETURN
RETURN 1640

Deactivate the menu keys
KEY(12) OFF: KEY(13) OFF : KEY(19) OFF
RETURN

Move the cursor left
LOCATE 5, COL+CHOICE*10: COLOR 3, 8: PRINT ITEMS$(CHOICE)
LOCATE 7,1: PRINT SPACES$(80)
CHOICE=CHOICE-1: IF CHOICE=-1 THEN CHOICE=NUMITEMS
LOCATE 5, COL+CHOICE*10: COLOR 0, 7: PRINT ITEMS$(CHOICE)
LOCATE 7,(80-LEN(DESCRIPT$(CHOICE)))/2-5
COLOR 2,0: PRINT"===> ": DESCRIPT$(CHOICE)
LOCATE LLINE,Z
RETURN

Move the cursor right
LOCATE 5, COL+CHOICE*10: COLOR 3, 8: PRINT ITEMS$(CHOICE)
LOCATE 7,1: PRINT SPACES$(80)
CHOICE=CHOICE+1: IF CHOICE=NUMITEMS+1 THEN CHOICE=0
LOCATE 5, COL+CHOICE*10: COLOR 0, 7: PRINT ITEMS$(CHOICE)
LOCATE 7,(80-LEN(DESCRIPT$(CHOICE)))/2-5
COLOR 2,0: PRINT"===> ": DESCRIPT$(CHOICE)
LOCATE LLINE,Z
RETURN

Print the menu
CLS:COLOR 3,8
LOCATE 2,(80-LEN(TITLES))/2-2:COLOR 5,0:PRINT TITLES
MASTEB.BAS

1180 FOR I=0 TO NUMITEMS
1190 READ ITEMS$(I),DESCRIPT$(I) 'Read item names and descriptions
1200 NEXT I
1210 COL=(80-(NUMITEMS+1)*10)/2 'Set starting column of menu items
1220 CHOICE =0
'Cursor is on the 1st item
1230 LOCATE 5,COL:COLOR 0,7:PRINT ITEMS$(CHOICE)
1240 LOCATE 7,(80-LEN(DESCRIPT$(CHOICE)))/2-5
1250 COLOR 2,0:PRINT"":DESCRIPT$(CHOICE)
1260 FOR I=1 TO N ITEMS
1270 LOCATE 5,COL+I*10:COLOR 3,8:PRINT ITEMS$(I) 'Print other items
1280 NEXT I
1290 RETURN
1500 '
1510 '*************************************************************************** Opening Menu***************************************************************************
1520 '
1530 RESTORE
1540 NUMITEMS = 5 'not including "Exit"
1550 TITLES="MIT Computer Controlled Caisson Element Testing System"
1560 GOSUB 1140 'Make and display the menu
1570 LOCATE 20,22:PRINT "Use ",CHR$(26)," or ",CHR$(27)," to choose an item."
1580 LOCATE 21,22:PRINT " Then press <Enter>."
1590 GOSUB 830 'Turn ON the menu keys
1600 Z=0;BOUNCE=2 'Constants for the visual diversion
1620 GOTO 3100 'Visual diversion
1630 GOTO 1620
1640 GOSUB 890 'Something's been selected; Turn OFF menu keys
1650 COLOR 3,8
1660 ON (CHOICE+1) GOSUB 1730,1880,2030,2180,3000,3060
1670 GOTO 1500
1680 '
1690 '*************************************************************************** Subroutines related to the opening menu***************************************************************************
1700 '
1710 '*************************************************************************** MOVE THE CAISSON WALL MOTOR***************************************************************************
1720 '
1730 DATA "Motor 1","Raise/lower caisson wall"
1735 GOSUB 610 'setup function keys
1736 DEVICE1=14
1770 LOCATE 11,10: PRINT "Raise/lower caisson wall"
1780 LOCATE 13,10: PRINT "Press <F1> to lower the wall step-wise"
1790 LOCATE 14,10: PRINT "Press <F2> to raise the wall step-wise"
1792 LOCATE 15,10: PRINT "Press <F3> to lower the wall continuously"
1794 LOCATE 16,10: PRINT "Press <F4> to raise the wall continuously"
1795 GOSUB 675 'transfer control
1850 RETURN
1860 '*************************************************************************** MOVE THE CAISSON CAP MOTOR***************************************************************************
1870 '
1880 DATA "Motor 2","Raise/lower caisson cap"
1885 GOSUB 610 'setup function keys
1886 DEVICE1=13
1920 LOCATE 11,10: PRINT "Raise/lower caisson cap"
1930 LOCATE 13,10: PRINT "Press <F1> to lower the cap step-wise"
1940 LOCATE 14,10: PRINT "Press <F2> to raise the cap step-wise"
1942 LOCATE 15,10: PRINT "Press <F3> to lower the cap continuously"
1944 LOCATE 16,10: PRINT "Press <F4> to raise the cap continuously"
1945 GOSUB 675 "transfer control
1990 RETURN
2000'
2100 *************MOVE CLAY CHAMBER AIR PRESSURE MOTOR ***************
2200'
2300 DATA "Motor 3","Adjust the air pressure"
2305 GOSUB 610 'setup function keys
2306 DEVICE! =11
2370 LOCATE 11,10: PRINT "Adjust the air pressure"
2380 LOCATE 13,10: PRINT "Press <F1> to decrease pressure step-wise"
2390 LOCATE 14,10: PRINT "Press <F2> to increase pressure step-wise"
2392 LOCATE 15,10: PRINT "Press <F3> to decrease pressure continuously"
2393 LOCATE 16,10: PRINT "Press <F4> to increase pressure continuously"
2394 GOSUB 675 "transfer control
2440 RETURN
2510'
2610 *************TAKE DATA READINGS ************
2710'
2790 'This subroutine will scan the channels on the multiplexer and take and
2810 'and display the readings.  The null feature of the AD1170 is used to
2820 'to take the input voltage reading (which exceeds the 5 V limit of the
2821 'AD1170.  The bit precision of the AD1170 was set at the top of the
2822 'program.
2823 '
2835 DATA "Readings","Scan the measurement channels"
2840 CLS: PRINT : PRINT : PRINT
2850 PRINT "Monitoring channels 1-6 on the multiplexer to take all of your"
2860 PRINT "transducer readings (in millivolts)"
2870 PRINT :INPUT "Continuous or single scan of all the channels (c or s)? ",YS
2872 PRINT : PRINT "Press <F1> to take another set of readings"
2874 PRINT "Press spacebar to stop continuous readings"
2876 PRINT "Press 'enter' to return to main menu"
2878 ON KEY(1) GOSUB 2900 : KEY(1) ON
2880 PRINT : PRINT " 1 2 3 4 5 6 "
2890 PRINT : PRINT
2930 OUT AD1170,69 :WAIT AD1170,1,1 'set default calibration time to 167 ms
2932 OUT AD1170+1,INTBIT 'load the data format into 2nd bit slot
2934 OUT AD1170,48:WAIT AD1170,1,1 'lock in the data format loaded
2934 AD1170KEY=0 'reset the key to take individual readings
2935 FOR N=1 TO 6
2936 CHANNEL=(N-1)
2938 OUT MUX!,CHANNEL 'select mux channel
2939 IF N=VINCHANNEL THEN GOSUB 2610 'converts input voltage using null
2940 OUT AD1170,INTTIME:WAIT AD1170,1,1 'conversion using preset time
2942 OUT MUX!,GRDCHANNEL 'set to ground after conversion
2944 LOWBYTE=INP(AD1170+1) : MIDBYTE=INP(AD1170+2) : HIBYTE=INP(AD1170+3)
2946 ' this read in the data byte
2948 CTS=LOWBYTE+256*MIDBYTE+65536!*HIBYTE


MASTER.BAS

2440  VTS=(CTS*10/2^((INTBIT+7)-5))*1000
2460  VOLTS(N)=VTS:GOTO 2480
2480  NEXT N
2520  PRINT USING "###.###",VOLTS(1),VOLTS(2),VOLTS(3),VOLTS(4),VOLTS(5),VOLTS(6)
2540  OUT AD1170,192:WAIT AD1170,1,1 'do one background calibration
2562  AS$=INKEY$: IF AS$=CHR$(32) THEN GOTO 2572
2570  IF Y$="c" OR Y$="C" THEN GOTO 2340
2572  AS$=INKEY$: IF AS$=CHR$(13) THEN GOTO 2590
2580  IF AD1170KEY=1 THEN GOTO 2340 ELSE GOTO 2572
2590  KEY(1) OFF : RETURN
2600  ' nested subroutine to get input voltage
2620  'set mux to ref input voltage
2640  OUT AD1170,112 : WAIT AD1170,1,1 'measure the null signal
2650  OUT AD1170,120 : WAIT AD1170,1,1 'enable the null
2660  OUT MUX!,CHANNEL 'return to the input voltage channel
2670  OUT AD1170,INTTIME : WAIT AD1170,1,1 'convert using 167 ms
2680  OUT AD1170,128:WAIT AD1170,1,1 'disable null
2682  OUT MOTORS,FULLSTOP+240 'set input to ground
2690  RETURN 2410
2900  'set the key to take another reading
2910  AD1170KEY=1
2920  RETURN
3000  *********************** START TESTING ***********************
3010  ' DATA "Testing","Begin test set-up program"
3035  CHAIN "setup.bas" 'Enter caisson test info
3040  'select a test
3050  ' DATA "Exit","Leave this program"
3085  OUT MOTORS!+4,0 'OPEN RELAYS TO LOCK MOTORS
3090  OUT MOTORS,FULLSTOP:CLS:END
3100  ' THE VISUAL DIVERSION ***********************
3120  Z=Z+1
3130  LLINE=10
3140  LOCATE LLINE,Z:PRINT "-FATCAT-"
3150  LLINE=LLINE+2
3160  LOCATE LLINE,Z:PRINT "Geotechnical Laboratories"
3170  LLINE=LLINE+2
3180  LOCATE LLINE,Z:PRINT "At MIT"
3190  FOR I=1 TO 1000:NEXT I
3200  IF Z=50 THEN GOTO 3130
3210  IF Z=52 THEN BOUNCE=-2
3220  IF Z=2 THEN BOUNCE=2
3230  GOTO 3130
3400  GOSUB 3500
3500 '  
3510 ****** TAKE CONTROL OF KEYBOARD *********  
3520 '  
3530 GOSUB 3660  
3540 PRINT "Press ENTER to continue"  
3550 KEY 19,CHR$(0)+CHR$(&H1C)  
3560 ON KEY (19) GOSUB 3640  
3570 KEY (19) ON  
3580 A$=INKEY$  
3590 FOR I=1 TO 100 : NEXT I  
3600 IF A$<>CHR$(13) THEN GOTO 3590  
3610 PRINT "Turn off both the NUMBER LOCK and CAPS LOCK keys"  
3620 PRINT " and"  
3630 GOTO 3540  
3640 KEY (19) OFF  
3650 RETURN 284  
3660 'setup keyboard locks  
3670 KEY 15,CHR$(0)+CHR$(&H45)  
3680 KEY 16,CHR$(0)+CHR$(&H3A)  
3690 KEY 17,CHR$(4)+CHR$(70)  
3700 KEY 18,CHR$(12)+CHR$(83)  
3710 ON KEY (15) GOSUB 3790  
3720 ON KEY (16) GOSUB 3790  
3730 ON KEY (17) GOSUB 3790  
3740 ON KEY (18) GOSUB 3790  
3750 KEY (15) ON  
3760 KEY (16) ON  
3770 KEY (17) ON  
3780 KEY (18) ON  
3790 RETURN  
3800 END
SETUP.BAS

10 ' edited 8/02/91 by jtg
11 ' edited 5/26/93 by dfc
12 PROGRAM$="SETUP"
15 '**********************************************************************
20 ' This subprogram is an editing utility for the data files that contain
40 ' pertinent parameters that may be selected for various soil tests. It is
50 ' executed immediately after the "Testing" option from the main program,
60 ' "MASTER.BAS", has been chosen. After execution, this section chains to
70 ' all other phases of testing. "CETEST.BAS" must be accessed through this
80 ' program.
100 ' The editor utility in this program was written to handle 30
120 ' different entry fields, each 10 characters wide and in a specific
130 ' position that is found in the DATA statements at the end of this file.
140 ' Each field has a distinct number referenced through the variable CHOICE.
150 ' To widen the maximum width of all the fields, alter the constant
160 ' FIELDWIDTH. You may have to change the positions of the fields if you
170 ' change FIELDWIDTH. To add more fields, increase the constant
180 ' NUMOFCHOICES and add an appropriate line to each of the sections at the
190 ' end of this file, and change the lines with PRINT #1 and INPUT #1.
195 ' Since the fields are stored in arrays that are referenced through CHOICE,
196 ' make sure that the corresponding parts of each section match.
200 ' Depending on the position that you choose for the
210 ' new fields, you may need to change the cursor movement routines.
220 ' The variables DUMMY1 and DUMMY2 were necessary for the input voltage
240 ' to make up for the lack of conversion factor and zero for that item.
250 ' 30 Jan 1989  Amy G Chu **********
260 '**********************************************************************
270 ' 410 NUMOFCHOICES=29: FIELDWIDTH=10
420 DIM TAG$(NUMOFCHOICES),DESCRIPT$(NUMOFCHOICES)
430 DIM ROW(NUMOFCHOICES),COL(NUMOFCHOICES)
440 DONE$="no"
445 CLS: PRINT: PRINT
450 PRINT "This program is part of the MIT FATCAT System"
460 PRINT " The program is - ;:PROGRAM$
465 PRINT : PRINT
470 PRINT "Enter the name of your apparatus data file"
480 PRINT " or enter 'new' for a new data file"
490 INPUT " or enter 'quit' to return to the manual controller: ",FILENAME$
500 IF FILENAME$="new" OR FILENAME$="NEW" THEN GOSUB 3120: GOTO 600
510 IF FILENAME$="quit" OR FILENAME$="QUIT" THEN GOTO 955
520 ON ERROR GOTO 580
530 OPEN FILENAME$ FOR INPUT AS #1
540 INPUT
541 #1,FILENAME$,DAT$,INITIAL$,WALLWT,WALLAR,WALLDIA,CAPWT,CAPAR,ZWALLD,CFW
542 ALLD,ZWALLF,CFWALLF,ZCAPD,CFCAPD,ZCAPF,CFCAPF,ZAIRP,CFAIRP,WALLDCHANNEL,
543 WALLFCHANNEL,CAPFCHANNEL,CAPFCHANNEL,AIRPCHANNEL,DUMMY1
550 INPUT #1,DUMMY2,VINCHANNEL
SETUP.BAS

560 CLOSE #1
570 GOTO 600
580 PRINT "File not found"
590 RESUME 460
600 ' 610 ' Get the tags and their positions
620 FOR I=0 TO NUMOFCHOICES
630 READ ROW(I),COL(I),TAG$(I),DESCRP$(I)
640 NEXT I
650 GOSUB 3000 'Print the current set of data
660 KEY(11) ON: ON KEY(11) GOSUB 1090 'Activate Up arrow
670 KEY(12) ON: ON KEY(12) GOSUB 1190 'Left arrow
680 KEY(13) ON: ON KEY(13) GOSUB 1260 'Right arrow
690 KEY(14) ON: ON KEY(14) GOSUB 1320 'Down arrow
700 CHOICE=0: OFFSET=0: BLANKS$=SPACES$(FIELDWIDTH)
710 LOCATE 22,(80-LEN(DESCRP$(CHOICE))/2-5):PRINT "===> ":DESCRP$(CHOICE)
720 '
730 WHILE (INSTR(DONE$,"yes")=0 AND INSTR(DONE$,"YES")=0)
740 LOCATE ROW(CHOICE),COL(CHOICE)+15+OFFSET: PRINT CHR$(178) 'Print cursor
750 AS$=INKEY$ 'Wait for key to be pressed
760 IF AS$=CHR$(13) THEN GOSUB 2500:GOSUB 1260:GOTO 820 'Carriage return
770 IF AS$=CHR$(8) THEN GOSUB 1030:GOTO 820 'Backspace
780 IF NOT(("a"<=AS$ AND AS$="z") OR ("A"<=AS$ AND AS$="Z") OR ("0"<=AS$ AND AS$="9")
OR AS$="." OR AS$="."
OR AS$="") THEN 820 'Ignore char if invalid
790 IF OFFSET=FIELDWIDTH-1 THEN OFFSET=FIELDWIDTH-1
800 LOCATE ROW(CHOICE),COL(CHOICE)+15+OFFSET: PRINT AS$: 'Print the char
810 MIDS$(BLANKS$,OFFSET+1)=AS$: OFFSET=OFFSET+1
820 WEND
830 ' 840 GOSUB 3000 'Show the data entered
850 LOCATE 23,10: INPUT"Do you really want to quit (yes/no)":AS$
860 IF AS$="yes" AND AS$="YES" THEN DONES$="no" :GOTO 650
870 PRINT: PRINT "Writing data to file ":FILENAME$; "...": PRINT
880 OPEN FILENAMES FOR OUTPUT AS #1
890 PRINT
#1,FNAME$,,DATS$,,INITIAL$,,WALLWT,WALLAR,WALLDIA,CAPWT,CAPAR,ZWA
LLD,CFWALLD,ZWALLF,CFWALLF,ZNAPF,CFNAPF,ZCAPF,CFCAPF,ZAIRP,CFAIRP,WALLDC
HANNEL,WALLFCHANEL,CAPDCHANEL
900 PRINT #1,CAPFCHANEL,AIRPCHANNEL,DUMMY1,DUMMY2,VINCHANNEL
910 CLOSE #1
920 PRINT "Indicate the next phase of testing:
921 PRINT " a. Caisson Element Test."
926 PRINT " b. Return to manual controller."
928 PRINT: INPUT " Select a or b == ", AS$
931 IF AS$="a" OR AS$="A" THEN CHAIN "CETEST.BAS",10,ALL
936 IF AS$="b" OR AS$="B" THEN GOTO 955
950 GOTO 920
955 CHAIN "master3.bas" 'return to manual controller
960 END
1000 ' 1010 ****************************** Move the Cursor Around ******************************
1020 '  
1030 ' Backspace  
1040 LOCATE ROW(CHOICE),COL(CHOICE)+15+OFFSET: PRINT " "  
1050 OFFSET=OFFSET-1: IF OFFSET=-1 THEN OFFSET=0  
1060 MID$(BLANK$,OFFSET+1)=" "  
1070 RETURN  
1080 '  
1090 ' Up arrow  
1100 GOSUB 1410 ' Print the current field according to old CHOICE  
1110 ' The next few IF/THEN statements pick a new CHOICE according to the positioning of the field blocks on the screen  
1120 IF CHOICE<3 THEN CHOICE=NUMOFCHOICES: GOSUB 1460: RETURN  
1130 IF CHOICE>3 AND CHOICE<=12 THEN CHOICE=CHOICE-2: GOSUB 1460:RETURN  
1140 IF CHOICE=NUMOFCHOICES THEN CHOICE=NUMOFCHOICES-2: GOSUB 1460: RETURN  
1150 CHOICE=CHOICE-3 'We are moving in the large block for transducers  
1160 GOSUB 1460 'Print the next field according to the new CHOICE  
1170 RETURN  
1180 '  
1190 ' Left arrow  
1200 GOSUB 1410  
1210 LOCATE ROW(CHOICE),COL(CHOICE)+15: GOSUB 2000  
1220 CHOICE=CHOICE-1: IF CHOICE <=-1 THEN CHOICE=NUMOFCHOICES  
1230 GOSUB 1460  
1240 RETURN  
1250 '  
1260 ' Right arrow  
1270 GOSUB 1410  
1280 CHOICE=CHOICE+1: IF CHOICE= NUMOFCHOICES+1 THEN CHOICE =0  
1290 GOSUB 1460  
1300 RETURN  
1310 '  
1320 ' Down arrow  
1330 GOSUB 1410  
1340 IF CHOICE=0 THEN CHOICE=3: GOSUB 1460: RETURN  
1350 IF CHOICE>1 AND CHOICE<=10 THEN CHOICE=CHOICE+2: GOSUB 1460: RETURN  
1360 IF CHOICE>=26 AND CHOICE<=28 THEN CHOICE=NUMOFCHOICES: GOSUB 1460: RETURN  
1370 IF CHOICE=NUMOFCHOICES THEN CHOICE=0: GOSUB 1460: RETURN  
1380 CHOICE=CHOICE+3  
1390 GOSUB 1460  
1400 RETURN  
1410 '  
1420 ' Clear the field's screen position and print its value  
1430 LOCATE ROW(CHOICE),COL(CHOICE)+15: PRINT SPACE$(FIELDWIDTH+1)  
1440 LOCATE ROW(CHOICE),COL(CHOICE)+15: GOSUB 2000  
1450 RETURN  
1460 '  
1470 ' Print the next field for editing  
1480 LOCATE ROW(CHOICE),COL(CHOICE)+15: GOSUB 2000  
1490 LOCATE 22,1:PRINT SPACE$(80) 'Erase then print description of item  
1500 LOCATE 22,(80-LEN(DESCRIP$(CHOICE)))/2-5:PRINT " ==> ",DESCRIP$(CHOICE)
SETUP.BAS

1510 BLANKS$=SPACES$(FIELDWIDTH): OFFSET=0
1520 RETURN
2000 ' 2010 ' ************ Print the data field determined by "CHOICE" ************
2020 ' 2030 ON CHOICE+1 GOTO
2100,2110,2120,2130,2140,2150,2160,2170,2180,2190,2200,2210,2220,2230,2240,2250,2260,2270,2280
2290,2300,2310,2320,2330,2340,2350,2360,2370,2380,2390
2100 PRINT FILENAMES$: RETURN
2110 PRINT DAT$: RETURN
2120 PRINT INITIALL$: RETURN
2130 PRINT WALLWT: RETURN
2140 PRINT TYPES$: RETURN
2150 PRINT WALLAR: RETURN
2160 PRINT WALLDIA: RETURN
2170 PRINT CAPWT: RETURN
2180 PRINT MEMBRANES$: RETURN
2190 PRINT CAPAR: RETURN
2200 PRINT AREACORR$: RETURN
2210 PRINT ZWALLD: RETURN
2220 PRINT CFWALLD: RETURN
2230 PRINT WALLDCHANG: RETURN
2240 PRINT ZWALLF: RETURN
2250 PRINT CFWALLF: RETURN
2260 PRINT WALLFCHAN: RETURN
2270 PRINT ZCAPD: RETURN
2280 PRINT CFCAPD: RETURN
2290 PRINT CAPDCHAN: RETURN
2300 PRINT ZCAPF: RETURN
2310 PRINT CFCAPF: RETURN
2320 PRINT CAPFCCHAN: RETURN
2330 PRINT ZAIRP: RETURN
2340 PRINT CFAIRP: RETURN
2350 PRINT AIRPCHAN: RETURN
2360 PRINT DUMMY1: RETURN
2370 PRINT DUMMY2: RETURN
2380 PRINT VINCHAN: RETURN
2390 PRINT DONE$: RETURN
2400 END
2400 ' 2500 ' 2510 ' *********************** Stuff the new value ***********************
2520 ' 2530 IF BLANKS$=SPACES$(FIELDWIDTH) THEN RETURN
2540 A$=BLANKS$: BLANKS$=SPACES$(FIELDWIDTH)
2550 IF (CHOICE>=0 AND CHOICE<=2) OR CHOICE=4 OR CHOICE=8 OR CHOICE=10 OR
CHOICE=NUMOFCHOICES THEN 2560 ELSE A$=VAL(A$)
2560 ON CHOICE+1 GOTO
2600,2610,2620,2630,2640,2650,2660,2670,2680,2690,2700,2710,2720,2730,2740,2750,2760,2770,2780
2790,2800,2810,2820,2830,2840,2850,2860,2870,2880,2890
2600 FILENAMES$=A$: RETURN
2610 DAT$:=A$: RETURN
SETUP.BAS

2620 INITIAL$ = A$: RETURN
2630 WALLWT = A$: RETURN
2640 TYPE$ = A$: RETURN
2650 WALLAR = A$: RETURN
2660 WALLDIA = A$: RETURN
2670 CAPWT = A$: RETURN
2680 MEMBRANES$ = A$: RETURN
2690 CAPAR = A$: RETURN
2700 AREACORRS$ = A$: RETURN
2710 ZWALLD = A$: RETURN
2720 CFWALLD = A$: RETURN
2730 WALLOWITALACHANNEL = A$: RETURN
2740 ZWALLF = A$: RETURN
2750 CFWALLF = A$: RETURN
2760 WALLFCHANNEL = A$: RETURN
2770 ZCAPD = A$: RETURN
2780 CFCAPE$ = A$: RETURN
2790 CAPDCHANNEL = A$: RETURN
2800 ZCAPF = A$: RETURN
2810 CFCAPEF = A$: RETURN
2820 CAPFCHANNEL = A$: RETURN
2830 ZAIRP = A$: RETURN
2840 CFIRA$ = A$: RETURN
2850 AIRPCHANNEL = A$: RETURN
2860 DUMMY1 = A$: RETURN
2870 DUMMY2 = A$: RETURN
2880 VINCHANEL = A$: RETURN
2890 DONE$ = A$: RETURN
3000 ' 
3010 ' ******** Print all of the data fields at their current values ********
3020 ' 
3030 CLS
3040 LOCATE 2, 30: PRINT "Test Initialization"
3050 LOCATE 11, 15: PRINT "Transducer Zero Conversion Factor Channel No."
3060 COLOR 3, 8
3070 FOR I = 0 TO NUMOFCHOICES
3080 LOCATE ROW(I), COL(I): PRINT TAG$(I)
3090 LOCATE ROW(I), COL(I) + 15: CHOICE = I: GOSUB 2000 'Print the field
3100 NEXT I
3110 RETURN
3120 ' 
3130 ' ************** Set up new set of test parameters **************
3140 ' 
3150 FILENAME$ = "**": DAT$ = DATES: INITIAL$ = "**"
3160 WALLWT = 0: WALLAR = 0: CAPWT = 0
3170 CAPAR = 0:
3180 ZWALLD = 0: CFWALLD = 0: ZWALLF = 0: CFWALLF = 0
3190 ZCAPD = 0: CFCAPE$ = 0: ZCAPF = 0: CFCAPEF = 0
3200 ZAIRP = 0: CFIRA$ = 0
3210 WALLOWITALACHannel = 0: WALLFCHANNEL = 0: CAPDCHANNEL = 0: CAPFCHANNEL = 0
3220 AIRPCHANNEL = 0: VINCHANEL = 0
SETUP.BAS

3230 DONES="no"
3240 RETURN
3500 ' Positons and Tags for the Data
3510 '***************
3520 'DATA 4,2,"Filename:","Enter the name of the file."
3600 DATA 4,2," Date:","Enter today's date."
3620 DATA 4,55,"Initials:","Enter your initials."
3630 DATA 6,10,"Wall Weight:","Weight of pile wall and accessories (kg)"
3640 DATA 6,50,"xxxx:","This field not in use"
3650 DATA 7,10,"Wall Area:","Cross-sectional area of pile wall tip (cm^2)"
3660 DATA 7,50,"Wall diameter:","Diameter of pile wall (cm)"
3670 DATA 8,10,"Cap Weight:","Weight of pile cap and accessories (kg)"
3680 DATA 8,50,"xxxx:","This field not in use"
3690 DATA 9,10,"Cap Area:","Area of pile cap (cm^2)"
3700 DATA 9,50,"xxxx:","This field not in use"
3710 DATA 12,2,"WALLD:","What is the zero for the PILE WALL DCDT (v/v)?"
3720 DATA 12,27","Enter conversion factor for the PILE WALL DCDT (cm/v/v)"
3730 DATA 12,48","Multiplexer channel for the PILE WALL DCDT"
3740 DATA 13,2,"WALLF:","What is the zero for the PILE WALL force transducer (v/v)?"
3750 DATA 13,27","Enter conversion factor for PILE WALL force transducer (kg/v/v)"
3760 DATA 13,48","Multiplexer channel for the PILE WALL force transducer"
3770 DATA 14,2,"CAPD:","What is the zero for the PILE CAP DCDT (v/v)?"
3780 DATA 14,27","Enter conversion factor for the PILE CAP DCDT (cm/v/v)"
3790 DATA 14,48","Multiplexer channel for the PILE CAP DCDT"
3800 DATA 15,2,"CAPF:","What is the zero for the PILE CAP force transducer (v/v)?"
3810 DATA 15,27","Enter conversion factor for the PILE CAP force transducer (kg/v/v)"
3820 DATA 15,48","Multiplexer channel for the PILE CAP force transducer"
3830 DATA 16,2,"AIRP:","What is zero for the CELL CHAMBER AIR pressure transducer (v/v)?"
3840 DATA 16,27","Enter conv. factor for CELL CHAMBER AIR pressure transducer (ksc/v/v)"
3850 DATA 16,48","Multiplexer channel for the CELL CHAMBER AIR pressure transducer"
3860 DATA 17,2,"VIN:","Leave this alone"
3870 DATA 17,27","Leave this alone"
3880 DATA 17,48","Multiplexer channel for the INPUT voltage"
3890 DATA 19,30,"Done:","Have you finished completing this form? (yes/no)"

□
CETEST.BAS

10 'last edited 12/95 by dfc
11 PROGRAM$="CETEST"
20 ' 30 ' This program performs a Caisson Element Test, which includes chamber
40 ' clay consolidation, suction caisson driving, caisson hold stress,
50 ' and caisson pullout.
60 ' 70 ' This program is based on TXTEST3, a triaxial test program written
80 ' by T Sheahan and J Germaine for the MIT Geotechnical Laboratory.
90 ' 100 ' 110 ' **********************************************************************
120 ' 121 MINC=1
122 CLS :PRINT :PRINT
124 PRINT "This program performs the caisson element test"
126 PRINT " This program is - ",PROGRAM$
128 PRINT : PRINT
130 PRINT " The following hardware is required:"
140 PRINT " -Strawberrymie d to a converter ",
150 PRINT " -The Sheahan a to d converter"
160 PRINT " -The MIT three axis controller with;"
170 PRINT " -channel 1 for wall force/displacement
180 PRINT " -channel 2 for cap force/displacement
190 PRINT " -channel 3 for air pressure"
200 ' 210 ' **********************************************************************
220 KEY OFF : FOR I=1 TO 10 : KEY I,"": NEXT I ' disable F-keys
225 BLKS=$SPACE$(79) ' line eraser
226 H1$="TRANSDUCER READINGS in volts"
227 H2$=" Walld Wallf Capd Capf Airp Input"
228 H3$=" Walld Wallf Capd Capf Airp FTOT DELCAPD ",
229 H4$=" cm kg cm kg ksc kg cm"
230 P1$="####.##":P2$="####.####":P3$="####.####"
236 VINREAD=10 ' period to read vin & update screen
237 VINFLAG=VINREAD ' input voltage counter
240 ENTERFLAG=0 ' trap for enter key
250 GOSUB 3890 ' lock out keyboard
260 GOSUB 4160 ' set enter key
270 A$=INKEY$
280 IF ENTERFLAG=1 THEN GOTO 330
290 IF A$<>CHR$(13) THEN GOTO 270
295 CLS :LOCATE 20,1
300 PRINT " turn off both the NUMBER LOCK and CAPS LOCK keys"
310 PRINT " and"
320 GOTO 260
330 STEPTIME=1 ' time for each motor step in sec.
340 GOSUB 3750 ' calibrate steptime
360 '
370 ************ SET UP GAINS, ARRAYS, VARIABLES AND CURRENT READINGS ********
380 '
CETEST.BAS

390 ' set the A/D converter up and define performance variables
400 INTTIME=22 'to specify the integration time of the A/D converter
410 'inttime=16+N where N=0 1 msec N=4 100 msec
420 ' N=1 10 msec N=5 166.7 msec
430 ' N=2 16.7 msec N=6 300 msec
440 ' N=3 20 msec
450 ' can set a variable integration time using the EIS command
460 INTBIT=13 'specify the bit precision INTBIT=(bit precision-7)
470 AD1170=768 'the decimal I/O address of the A/D converter
474 MUXI=776 'decimal I/O of channel selector
475 ' CORRESPONDS TO SWITCH SETTING 00001
480 OUT AD1170,60 :WAIT AD1170,1,1 'set the default calibration time
490 OUT AD1170+1,INTBIT 'load the data format into the 2nd byte
500 OUT AD1170,48 :WAIT AD1170,1,1 'lock in the data format loaded
510 OUT AD1170,176 :WAIT AD1170,1,1 'begin background calibration
530 '
540 ' set default values and flags
550 '
560 ROW=2
570 TADJUST=0 'to adjust time for a change in date during test
580 ENTERFLAG=0 'for breaking a loop on the enter key
590 NUMCHANNELS=6 :MAXINC=25 :STARTDATE=DATES
600 DIM AIRP(MAXINC),CAPF(MAXINC),TIME(MAXINC),VOLTS(10)
610 DIM MFLAGS(3), CONTROL(3), GAIN(5), MVOLTS(3)
620 GNDCHANNEL=15
625 REFCHANNEL=14
630 OUT MUXI,GNDCHANNEL 'set input to AD1170 to ground
635 '
640 ******************* Setup the DC servo motors **************
650 '
670 MOTOR1=6928 'decimal I/O address of analog out card
672 LOBIT=240 :HIBIT=7 'decimal value for zero volts (+/-5 range)
673 LOBIT=LOBIT : HIBIT=HIBIT 'variable to specify motors to stop; 0=all
680 STOPDEVICE=0 'stop all motors
685 GOSUB 3110 'close relays and unlock motors
700 '**** set gain values *****
710 GAIN(1)= .000833 cm/volt-sec walld motor 1
720 GAIN(2)= -5 'kg/volt-sec wallf motor 1
730 GAIN(3)= .00125 'sec capd motor 2
740 GAIN(4)= -2 'kg/volt-sec capf motor 2
750 GAIN(5)= .05 'ksc/volt-sec airp motor 3
760 'DEVICE(1)=14 :CONTROL(1)=1 'set motor 1 to wall and disp control
770 DEVICE(2)=13 :CONTROL(2)=1 'set motor 2 to cap and disp control
780 DEVICE(3)=11 :CONTROL(3)=1 'set motor 3 to airp and stress control
780 '
785 ' Reminders and gain values; ' 0' at the end to signify initial value
790 CLS :PRINT :PRINT
795 PRINT "Ensure that the control box is on."
800 PRINT :PRINT
810 INPUT " press ENTER to continue ",A$
880 '  
890 CLS: GOSUB 4463 'data set and basic screen 
900 PRINT**  
910 PRINT"Please select the next phase of testing:"*PRINT  
911 PRINT* 1. Consolidation 4. Monotonic Pullout*  
912 PRINT* 2. Suction Driving 5. Re-initialize FTOT, SIGV*  
913 PRINT* 3. Hold Stress 6. End Program*  
920 LOCATE 20,1*PRINT BLKS$  
921 LOCATE 20,1*INPUT Enter OPTION number *,CHOICES$  
922 CH=VAL(CHOICES$)  
923 IF CH<1 OR CH>6 THEN GOTO 920  
924 ON CH GOTO 1000,7210,2000,9000,5000,2100  
925 '  
1000 ****************************************** CONSOLIDATION ******************************************  
1001'  
1002 KEYFLAG=0  
1006 MODULE=1  
1010 CONTROL(1)=1  
1020 CONTROL(2)=2  
1025 MVMOTOR=1  
1030 CLS: GOSUB 4465 'read & basic screen  
1040 ROW=CSRLIN  
1050 LOCATE 23,1 : PRINT BLKS$  
1060 COLOR 0,7  
1070 LOCATE ROW,11  
1080 PRINT "CONSOLIDATION INCREMENTS"  
1090 COLOR 3,8  
1100 PRINT "For each LOAD INCREMENT specify the AIRP pressure and TIME duration"  
1110 PRINT "Enter a '99' for AIRP when finished"  
1120 PRINT "Enter a '999' for AIRP to return to MAIN MENU"  
1130 NUMINCS=0  
1140 FOR I=1 TO MAXINCS: AIRP(I)= -1: CAPF(I)= -1: TIME(I)= -1: NEXT I  
1150 PRINT "Increment #:TAB(20); "AIRp";TAB(40);"CAPF";TAB(60);"TIME (minutes)"  
1160 WHILE (AIRP(NUMINCS)<999 AND AIRP(NUMINCS)<99 AND NUMINCS<MAXINCS)  
1170 NUMINCS=NUMINCS+1  
1175 ROW=CSRLIN  
1180 IF ROW<24 THEN GOTO 1250  
1182 I=24-NUMINCS  
1190 LOCATE 18,1  
1200 FOR ROW=19 TO 23  
1210 PRINT ROW-I;TAB(20);AIRP(ROW-I);TAB(40);AIRP(ROW-I)*CAPAR;TAB(60);TIME(ROW-I)  
1220 NEXT ROW  
1230 PRINT SPACES$(70)  
1240 ROW=ROW-1  
1250 LOCATE ROW,1 : PRINT NUMINCS;TAB(20);:INPUT AIRP(NUMINCS)  
1260 IF AIRP(NUMINCS)<0 THEN 1250  
1270 IF AIRP(NUMINCS)=99 THEN 1160 'finished entering data  
1280 IF AIRP(NUMINCS)=999 THEN 1160 'return to main menu  
1290 LOCATE ROW,40 : PRINT**`: PRINT AIRP(NUMINCS)*CAPAR  
1300 LOCATE ROW,60 : PRINT**`: :INPUT TIME(NUMINCS)
1310 IF TIME(NUMINCS)<1 THEN 1300
1320 IF NUMINCS=MAXINCS THEN PRINT "Max # increments =";MAXINCS:GOTO 1160
1330 WEND
1340 IF AIRP(NUMINCS)=99 THEN NUMINCS=NUMINCS-1
1350 IF AIRP(NUMINCS)=999 THEN GOTO 890
1360 '
1370 CLS:PRINT
1380 PRINT "Increment ";TAB(20);"Airp";TAB(40);"Capf";TAB(60);"Time (minutes)"
1390 FOR I=1 TO NUMINCS
1400 PRINT I;TAB(20);AIRP(I);TAB(40);AIRP(I)*CAPAR;TAB(60);TIME(I)
1410 TIME(I)=TIME(I)*60
1420 NEXT I
1430 LOCATE 22,1
1440 INPUT "Is this schedule okay (yes or no) ?";A$ 
1450 IF A$="yes" OR A$="YES" THEN 1470
1460 IF A$="no" OR A$="NO" THEN 1000 ELSE 1440
1470 '
1475 INPUT "press ENTER to begin CONSOLIDATION ",A$
1477 '
1480 ' *************** APPLY THE CONSOLIDATION INCREMENTS ***************
1490 '
1500 ' save the pre-increment stresses
1510 MFLAG$(3)="go"
1520 INCR=1  'loop to apply the large increments
1530 REM  'return point of loop
1540 CLS : GOSUB 4450  'readings and basic screen
1550 GOSUB 4290  'esc flag to abort increment
1560 GOSUB 4160  'enter flag for next increment
1570 COLOR 0,7 : LOCATE 25,65 : PRINT "CONSOLIDATION"; : COLOR 3,8
1580 OLDAIRP=AIRP : OLDCAPF=CAPF : OLDCAPD=CAPD
1590 LOCATE 18,57 : PRINT "Increasing Pressure ",CAPF
1600 LOCATE 20,57 : PRINT "of Increment ";INCR
1610 DAIRP=(AIRP(INCR)-AIRP)
1620 CAPF(INCR)=AIRP(INCR)*CAPAR
1630 DCAPF=(CAPF(INCR)-CAPF)
1640 INCTIME=TIMER:TADJUST=0  'set the start time for the increment
1650 NEWAIRP=OLDAIRP : NEWCAPF=OLDCAPF
1660 '
1670 'This section is the minor increment loop
1680 '
1690 CTR=0
1700 WHILE(ENTERFLAG=0 AND CTR<>1) 'loop to apply the split increment
1705 STOPDEVICE!=2
1710 CTR=CTR+1
1720 LOCATE 19,57 : PRINT "for Step ";CTR
1730 NEWAIRP=NEWAIRP+DAIRP : NEWCAPF=NEWCAPF+DCAPF
1735 SIGV=NEWAIRP
1740 IF CTR=10 THEN NEWAIRP=AIRP(INCR):NEWCAPF=CAPF(INCR)
1741 GOSUB 3260  'take set of readings
1742 DELCAPD=CAPD-OLDCAPD
1744 NEWWALLD=WALLD+DELCAPD
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1750  GOSUB 2720 'move motors
1751  ROW=13
1752  IF TARGETFLAG=0 THEN GOTO 1760 ELSE
1754  GOSUB 4500 'print target/control
1756  IF ROW=20 THEN ROW=13
1757  LOCATE ROW+1,1 : PRINT SPACES(75)
1758  LOCATE ROW,1:PRINT USING "####.####"
1759  %WALLD,WALLF,CAPD+WALLD,CAPF,AIRP,TOT,DEL,CAPD
1759  TARGETFLAG=0
1760  IF ENTERFLAG<>0 THEN GOTO 1790
1770  IF ABS(AIRP-NEWAIRP)>0.005 THEN GOTO 1741 'tolerance check
1780  IF ABS(CAPF-NEWCAPF)>1 THEN GOTO 1741
1785  VINFLAG=VINREAD
1790  WEND
1794  STOPDEVICE!=0
1795  GOSUB 3110
1796  OLDFTOT=(NEWAIRP)*(3.14159/4*WALLDIA^2)
1797  GOTO 2000 'HOLD STRESS
1800 ' 1948 '*************** manage action on abort increment ***************
1950 ' 1952 KEY (19) OFF : KEY (20) OFF
1954  GOSUB 3260 'readings
1956  NUMINCS=NUMINCS-INCR+1
1958  FOR I=1 TO NUMINCS
1960  CAPF(I)=CAPF(INCR+I-1)
1962  TIME(I)=TIME(INCR+I-1)/60
1964 NEXT I
1966  GOTO 1370
1990 ' 2000 '*************** HOLD STRESS ***********************
2003 ' 2004 STOPDEVICE!=3
2006 IF CAPS<-1 THEN GOTO 2008 ELSE GOTO 2016
2008 WKP=,.15
2009 WKPP=.01
2010 CKP=0
2011 CKPP=0
2012 CONTROL1(1)=2
2013 CONTROL1(2)=1
2014 GOSUB 4431 'F5 to change wallf GAIN activation
2015 GOSUB 4440 'F6 to change capd GAIN activation
2016 IF CAPS<-1 THEN GOTO 2025
2017 WKP=0
2018 WKPP=0
2019 CKP=.5
2020 CKPP=.01
2021 CONTROL1(1)=1
2022 CONTROL1(2)=2
2023 GOSUB 4085 'F5 to change walld GAIN activation
2024 GOSUB 4408 'F6 to change capf GAIN activation
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2025 '  
2030 CLS : GOSUB 4450 'get initial readings and print voltages  
2032 OLD CAPD = CAPD  
2034 OLD WALLD = WALLD  
2036 NEW AIRP = SIGV  
2038 IF MODULE <= 1 THEN OLD FTOT = FTOT  
2040 MODULE = 3  
2041 GAIN(1) = -.01  
2042 GAIN(3) = -.025  
2044 GOSUB 4290 'ESC activation  
2046 IF DRIVEFLAG = 1 THEN GOSUB 4380 'T2 to SUCTION DRIVE Activation  
2048 IF PULL FLAG = 1 THEN GOSUB 4400 'T3 to MONOTONIC PULLOUT Activation  
2050 IF OLD FTOT <= 1 THEN GAIN(2) = OLD FTOT  
2051 MVTS1 = 0  
2052 MVTS2 = 0  
2053 VIN FLAG = VIN READ  
2054 ROW = 12  
2055 WHILE (ENTER FLAG = 0)  
2056   GOSUB 3260 'take set of readings  
2057   DEL CAPD = CAPD-OLDCAPD  
2058   IF OLD FTOT < 1 THEN OLD FTOT = GAIN(2)  
2059   IF CAPD > 1 THEN GOTO 2067  
2060   ERRFTOT = FTOT-OLD FTOT  
2062   MVTS1 = MVTS1+WKP*ERRFTOT  
2064   MVOLTS(1) = MVTS1+WKP*ERRFTOT  
2065   NEW CAPD = CAPD-DL..CAPD  
2066   GOTO 2071  
2067   ERRFTOT = FTOT-OLD FTOT  
2068   MVTS2 = MVTS2+CKP*ERRFTOT  
2069   MVOLTS(2) = MVTS2+CKP*ERRFTOT  
2070   NEW WALLD = WALLD+DEL CAPD  
2071 '  
2072   GOSUB 2720 'control motors to holdstress  
2074   IF TARGET FLAG = 0 THEN GOTO 2090 ELSE  
2076   GOSUB 4500 'print target/control  
2078   IF ROW = 20 THEN ROW = 13  
2080   LOCATE ROW+1,1:PRINT SPACES$(75)  
2082   LOCATE ROW,1:PRINT USING "###.###", WALLD, WALLF, CAPD+WALLD, CAPF, AIRP, FTOT, DEL CAPD  
2084   TARGET FLAG = 0  
2090 WENL'  
2092 GAIN(2) = -.< 'temp toggle reset  
2094 GAIN(4) = -.2 'temp toggle reset  
2097 STOP DEVICE! = 0  
2098 GOSUB 3110  
2099 GOTO 890  
2100 '  
2101 '**********END PROGRAM *************  
2102 '  
2110 OUT MOTORS!+4,0 'LOCK MOTORS  
2120 INPUT "Hit <Enter> to chain to next section =>", Z$
2130 PRINT "Chaining to next section of the program..."  
2135 'STOP :END  
2140 CHAIN "master.bas"  
2150 '  
2610 '****************** SET THE FLAG ******************  
2620 '  
2630 ' Needed to maintain the syntax of the ON KEY() statements  
2640 '  
2650 KEYFLAG=1  
2660 RETURN  
2720 '  
2730 '****************** CONTROL THE MOTORS ******************  
2740 '  
2750 ' The big control loop  
2760 '  
2770 ' GOSUB 3260 ' take a set of readings  
2780 '  
2790 ' Calculate the difference between readings and target values  
2800 '  
2805 IF MODULE=1 THEN IF CONTROL1(1)=1 THEN MVOLTS(1)=(NEWWALLD-WALLD)/GAIN(1)  
2810 IF MODULE=3 THEN IF CONTROL1(1)=1 THEN MVOLTS(1)=(NEWWALLD-WALLD)/GAIN(1)  
2820 IF MODULE=4 THEN IF CONTROL1(1)=1 THEN MVOLTS(1)=(NEWWALLD-WALLD)/GAIN(1)  
2830 IF CONTROL1(1)=2 THEN MVOLTS(1)=(NEWWALLF-WALLF)/GAIN(2)  
2840 IF MODULE=3 THEN IF CONTROL1(2)=1 THEN MVOLTS(2)=(NEWCAPD-CAPD)/GAIN(3)  
2850 IF CONTROL1(2)=2 THEN MVOLTS(2)=(NEWCAPF-CAPF)/GAIN(4)  
2860 IF MODULE=2 THEN MVOLTS(2)=(NEWCAPD)/GAIN(3)  
2870 IF SUCFLAG=1 THEN MVOLTS(2)=0  
2880 IF CONTROL1(3)=1 THEN MVOLTS(3)=(NEWAIRP-AIRP)/GAIN(5)  
2860 FOR I=1 TO 3  
2870 IF MFLAG$(I)="stop" THEN MVOLTS(I)=0  
2872 IF MVOLTS(I)<-5 THEN MVOLTS(I)=-5  
2873 IF MVOLTS(I)>+5 THEN MVOLTS(I)=+5  
2880 NEXT I  
2890 IF MVOLTS(1)>0 THEN GOTO 2917  
2900 'MVOLTS(1)=MVOLTS(1)*(-1) 'REVERSE TYPE 356 MOTOR  
2910 'OUT MOTORSI+4,15 'close relay #4  
2915 'GOTO 2930  
2917 'OUT MOTORSI+4,7 'open relay #4  
2920 '  
2930 'Calculate the bit output required for each motor  
2940 '  
2950 FOR I=1 TO 3  
2960 BITSI=INT((MVOLTS(I)+5)*409.5)  
2970 HIBIT!(I)=INT(BITSI/256)  
2980 LOBIT!(I)=BITSI-HIBIT!(I)*256  
2990 NEXT I  
3000 '  
3010 'Move the motors
3020'
3025  IF MVMOTOR=0 THEN GOTO 3130
3030  FOR I=1 TO 3
3035  NEXT I
3040  OUT MOTORS!,LOBIT(I) 'set voltage register
3045  OUT MOTORS!+1,HIBIT(I)
3050  OUT MOTORS!+2,DEVICE(I) 'activate motor
3055  OUT MOTORS!+2,255 'close register
3060  NEXT I
3065  FOR I=1 TO STEPINC! : NEXT I 'run time
3100'
3110 ******* stop motors **************
3120'
3130  OUT MOTORS!,LOBIT0! 'zero register
3135  OUT MOTORS!+1,HIBIT0!
3140  OUT MOTORS!+2,STOPDEVICE! 'stop required motors
3145  OUT MOTORS!+2,255 'close register
3150  RETURN
3160
3170 '*************** ADJUST FOR CHANGE IN DATE DURING TEST ***************
3180'
3190  TADJUST=86400!*INCTIME+TADJUST
3200  INCTIME=0 : 'WONT NEED THIS ANYMORE AFTER THE FIRST ADJUSTMENT
3210  STARTDATES$=DATE$
3220  RETURN
3230
3240 '*************** TAKE SET OF READINGS AND CONVERT TO ENGINEERING UNITS ********
3250'
3260 ' This routine takes the transducer readings from NUMCHANNELS number
3270  of channels and converts volts to engineering units.
3280 ' The input voltage should only be checked periodically.
3290 ' Automatic background calibration is enabled whenever this
3300 ' routine is not active.
3310 '3320'
3330  OUT AD1170,184:WAIT AD1170,1,1 'disable the background calibration
3340 FOR L=1 TO NUMCHANNELS 'all channels plus ground
3350  CHANNEL=(L-1)
3360  OUT MUX!,CHANNEL 'select the mux channel
3370  IF MODULE=4 THEN INTTIME=18
3380  OUT AD1170,184:WAIT AD1170,1,1 'conversion using preset time
3390  OUT MUX!,GNDCHANNEL 'ground the input to the AD1170
3400 ' read the three data bytes
3410  LOWBYTE=INP(AD1170+1) : MIDBYTE=INP(AD1170+2) : HIBYTE=INP(AD1170+3)
3420  CTS=LOWBYTE+256*MIDBYTE+65536*HIBYTE ' total number of bits
3430  VTS=(CTS*10^2^(INTBIT+7)-5) 'convert to volts
3440 VOLTS(L)=VTS
3450 IF VINFLAG=-1 AND L=VINCHANNEL THEN VOLTS(L)=VOLTS(L)+5
3460 NEXT L
3470 OUT AD1170,176:WAIT AD1170,1,1 'reenable background calibration
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3495 VINFLAG=VINFLAG+1
3496 LOCATE 20,5 :PRINT USING "###.###", MVOLTS(1),MVTS1,ERRFTOT
3498 LOCATE 20,45:PRINT USING "###.###"; MVOLTS(2),ERRCAPD,ERRCAPV
3500 LOCATE 23,1:PRINT SPACES(80); 
3510 LOCATE 23,7 :PRINT USING "###.###"; VOLTS(WALLDCHANNEL),VOLTS(WALLFCHANNEL),VOLTS(CAPDCHANNEL),VOLTS(CAPFC
3512 'LOCATE 23,1 : PRINT USING "###.###
3530 ' ;mvolts(1),errwalld,errwavl,mvols(2),errcapd,erwwalv
3530 '
3540 ' convert to engineering units
3550 '
3555 WALLD=(VOLTS(WALLDCHANNEL)/VOLTS(VINCHANNEL)-ZWALLD)*CFWALLD 
3560 WALLF=(VOLTS(WALLFCHANNEL)/VOLTS(VINCHANNEL)-
ZWALLF)*CFWALLF+WALLWT 
3565 CAPD=(VOLTS(CAPDCHANNEL)/VOLTS(VINCHANNEL)-ZCAPD)*CFCAPD 
3570 CAPF=(VOLTS(CAPFCHANNEL)/VOLTS(VINCHANNEL)-ZCAPF)*CFCAPF+CAPWT 
3575 AIRP=(VOLTS(AIRPCHANNEL)/VOLTS(VINCHANNEL)-ZAIRP)*CFAIRP 
3584 FTOT=WALLF+CAPF 
3585 CAP=CAPF/CAPAR
3586 IF COUNT=1 THEN ZTIME=TIMER : N0=0
3587 MINC=TIMER-N0 "use for rate calculation
3588 N0=TIMER 
3590 RETURN 
3600 ' 
3610 'nested subroutine to check the input voltage of the transducers 
3620 OUT MUX!,REFCH N E 
3630 OUT AD1170,112: WAIT AD1170,1,1 'measure the null signal 
3640 OUT AD1170,120 :WAIT AD1170,1,1 'enable the null 
3650 OUT MUX!,CHANNEL 
3660 OUT AD1170,INTTIME=WAIT AD1170,1,1 'convert using preset time 
3670 OUT AD1170,128 :WAIT AD1170,1,1 'disable the null 
3680 VINFLAG=1 'reset the flag 
3682 TARGETFLAG=1 'print new target/control values 
3685 TINC=TIMER-T0 
3687 T0=TIMER 
3730 GOTO 3430 
3740 ' 
3750 '********** Set counter for delay loop **********
3770 I=1 
3780 ON TIMER (2) GOSUB 3810 '2 second sample 
3790 TIMER ON 
3800 I=I+1 :GOTO 3800 
3810 STEPINC1=(I/2)*STEPTIME 
3820 TIMER OFF 
3830 RETURN 3850 
3840 ' 
3850 'generic return center
3860 ' 
3870 RETURN 
3880 '
3890 '******** Subroutine to set soft function keys ******
3900 '  
3901 KEY 1,CHR$(0)+CHR$(&H3B)    'F1 to HOLD STRESS
3902 KEY 2,CHR$(0)+CHR$(&H3C)    'F2 to SUCTION DRIVE
3903 KEY 3,CHR$(0)+CHR$(&H3D)    'F3 to MONOTONIC PULLOUT
3904 KEY 5,CHR$(0)+CHR$(&H3F)    'Wallf gain toggle
3905 KEY 6,CHR$(0)+CHR$(&H40)    'Capd gain toggle
3910 KEY 15,CHR$(0)+CHR$(&H45)
3920 KEY 16,CHR$(0)+CHR$(&H3A)
3930 KEY 17,CHR$(0)+CHR$(70)    'control break  changwe 0 to 4
3940 KEY 18,CHR$(12)+CHR$(83)    'reset sequence
3950 KEY 19,CHR$(0)+CHR$(&H1C)    'ENTER KEY
3960 KEY 20,CHR$(0)+CHR$(&H12)    'ESC KEY
3970 ON KEY (1) GOSUB 4340    'F1 to HOLD STRESS
3980 ON KEY (2) GOSUB 4370    'F2 to SUCTION DRIVE
3990 ON KEY (3) GOSUB 4390    'F3 to MONOTONIC PULLOUT
3995 ON KEY (5) GOSUB 4428    'F5 to change wallf gain
3996 ON KEY (6) GOSUB 4440    'F6 to change capd gain
4000 'ON KEY (10) GOSUB 4410
4010 ON KEY (15) GOSUB 3870
4020 ON KEY (16) GOSUB 3870
4030 ON KEY (17) GOSUB 3870
4040 ON KEY (18) GOSUB 3870
4050 ON KEY (19) GOSUB 4100
4060 ON KEY (20) GOSUB 4230
4070 FOR I=15 TO 19 : KEY (I) ON : NEXT I
4080 RETURN
4085 KEY(11) ON:ON KEY(11) GOSUB 4088    'toggle for walld gain
4086 KEY(14) ON:ON KEY(14) GOSUB 4091
4087 RETURN
4088 GAIN(1)=1.1*GAIN(1)
4089 LOCATE 4,43:PRINT USING P3$;GAIN(1)
4090 RETURN
4091 GAIN(1)=.9*GAIN(1)
4092 LOCATE 4,43:PRINT USING P3$;GAIN(1)
4093 RETURN
4095 ' 4100 ' ******** GENERIC enter deactivation ****
4110 '  
4120 ENTERFLAG=1
4130 KEY(19) OFF
4135 KEY(20) OFF
4140 RETURN
4150 ' 4160 ' ******** generic enter activation *****
4170 '  
4175 LOCATE 25,15
4180 PRINT "ENTER to continue";
4190 ENTERFLAG=0
4200 KEY(19) ON
4210 RETURN
4220 '  
4230 ' ******* generic ESC deactivation ******
4240 '  
4245 STOPDEVICE!=0
4250 ENTERFLAG=2
4260 KEY(20) OFF
4265 KEY(19) OFF
4270 RETURN
4280 '  
4290 ' ******* generic ESC activation ******
4300 '  
4305 LOCATE 25,1
4310 PRINT "ESC to abort";
4320 ENTERFLAG=0
4330 KEY (20) ON
4338 RETURN
4339 '  
4340 ' ******* F1 TO HOLD STRESS De-Activation ***************
4341 '  
4342 ENTERFLAG=1
4343 KEY (1) OFF
4344 RETURN
4349 '  
4350 ' ******* F1 TO HOLD STRESS Activation ***************
4351 '  
4352 LOCATE 25,15
4353 PRINT "F1 to HOLD STRESS";
4354 ENTERFLAG=0
4355 KEY (1) ON
4356 RETURN
4357 '  
4370 ' ******* F2 TO SUCTION DRIVE De-Activation ***************
4371 '  
4372 ENTERFLAG=1
4373 KEY (2) OFF
4374 GOSUB 4070
4376 RETURN 7620
4377 '  
4380 ' ******* F2 TO SUCTION DRIVE Activation ***************
4381 '  
4382 LOCATE 25,34
4383 PRINT "F2 to DRIVE";
4384 ENTERFLAG=0
4385 KEY (2) ON
4386 RETURN
4387 '  
4390 ' ******* F3 TO MONOTONIC PULLOUT De-Activation ***************
4391 '  
4392 ENTERFLAG=1
4393 KEY (3) OFF
4394 GOSUB 4070
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4395 RETURN 9370
4396 '  
4400 ******** F3 TO MONOTONIC PULLOUT Activation ***********  
4401 '  
4402 LOCATE 25,47  
4403 PRINT "F3 to PULLOUT";  
4404 ENTERFLAG=0  
4405 KEY (3) ON  
4406 RETURN  
4407 '  
4408 KEY (12) ON:ON KEY(12) GOSUB 4414  'toggle to change capf GAIN  
4409 KEY (13) ON:ON KEY(13) GOSUB 4411  
4410 RETURN  
4411 GAIN(4)=1.1*GAIN(4)  
4412 LOCATE 7,43: PRINT USING P3$;GAIN(4)  
4413 RETURN  
4414 GAIN(4)=.9*GAIN(4)  
4415 LOCATE 7,43: PRINT USING P3$;GAIN(4)  
4416 RETURN  
4418 '  
4420 ******** toggle to turn on and off motors with f-keys ******  
4421 '  
4422 'II=1 :GOTO 4420  
4423 'II=2 :GOTO 4420  
4424 'II=3 :GOTO 4420  
4425 FOR II=1 TO 3 :GOSUB 4420 : NEXT II  
4426 IF MFLAG$(II)="start" THEN MFLAG$(II)="stop " ELSE MFLAG$(II)="start"  
4427 RETURN  
4428 IF CONTROL$(1)=1 THEN GOTO 4085  
4429 LOCATE 10,42  
4430 PRINT "Use up/down arrows to change WALLF GAIN"  
4431 KEY(11) ON:ON KEY(11) GOSUB 4434  
4432 KEY(14) ON:ON KEY(14) GOSUB 4437  
4433 RETURN  
4434 GAIN(2)=1.1*GAIN(2)  
4435 LOCATE 5,43: PRINT USING P3$;GAIN(2)  
4436 RETURN  
4437 GAIN(2)=.9*GAIN(2)  
4438 LOCATE 5,43: PRINT USING P3$;GAIN(2)  
4439 RETURN  
4440 IF CONTROL$(2)=2 THEN GOTO 4408  'toggle for capd gain  
4441 KEY(12) ON:ON KEY(12) GOSUB 4447  'toggle for capd gain  
4442 KEY(13) ON:ON KEY(13) GOSUB 4444  
4443 RETURN  
4444 GAIN(3)=1.1*GAIN(3)  
4445 LOCATE 6,43: PRINT USING P3$;GAIN(3)  
4446 RETURN  
4447 GAIN(3)=.9*GAIN(3)  
4448 LOCATE 6,43: PRINT USING P3$;GAIN(3)  
4449 RETURN  
4450 ******** print basic screen and collect readings********
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4460 '  
4461 LOCATE 11,1 :PRINT H3$;PRINT H4$  
4462 ROW=CSRLIN  
4463 LOCATE 21,20 :PRINT H1$;PRINT H2$  
4465 PTRFLAG! =0  
4470 VINFO =VINFOREAD ' get an initial input voltage  
4471 ROW=CSRLIN  
4472 LOCATE 21,20 :PRINT H1$;PRINT H2$  
4473 PTRFLAG! =0  
4480 GOSUB 3260 ' get an initial set of readings and convert to eng. units  
4490 '  
4500 '********print screen only ********  
4502 LOCATE 1,1  
4503 FOR I=1 TO 9 :PRINT BLKS :NEXT I  
4504 LOCATE 1,1  
4510 PRINT  
4520 PRINT " CURRENT READINGS TARGET VALUES GAIN RATES CONTROL  
SIGNALS "  
4530 PRINT " Walf = ";PRINT USING P3$;WALLD;PRINT " cm ";  
4533 IF CONTROL(1)< =1 THEN GOTO 4550  
4540 PRINT USING P3$;NEWWALLD;PRINT " cm ";  
4542 PRINT USING P3$;GAIN(1);PRINT " cm/volt-sec ";  
4544 PRINT USING P3$;MVOLTS(1);PRINT " volts";  
4550 PRINT  
4559 '  
4560 PRINT " Walf = ";PRINT USING P2$;WALLF;PRINT " kg ";  
4563 IF CONTROL(1)< =2 THEN GOTO 4569  
4566 PRINT USING P2$;NEWWALLF;PRINT " kg ";  
4567 PRINT USING P3$;GAIN(2);PRINT " kg/volt-sec ";  
4568 PRINT USING P3$;MVOLTS(1);PRINT " volts";  
4569 PRINT  
4570 '  
4574 PRINT " Capd = ";PRINT USING P3$;CAPD+WALLD;PRINT " cm ";  
4575 IF CONTROL(2)< =1 THEN GOTO 4586  
4576 PRINT USING P3$;NEWCAPD+WALLD;PRINT " cm ";  
4580 PRINT USING P3$;GAIN(3);PRINT " cm/volt-sec ";  
4585 PRINT USING P3$;MVOLTS(2);PRINT " volts";  
4586 PRINT  
4589 '  
4590 PRINT " Capf = ";PRINT USING P2$;CAPF;PRINT" kg ";  
4595 IF CONTROL(2)< =2 THEN GOTO 4610  
4600 PRINT USING P2$;NEWCAPF;PRINT " kg ";  
4602 PRINT USING P3$;GAIN(4);PRINT" kg/volt-sec ";  
4604 PRINT USING P3$;MVOLTS(2);PRINT" volts";  
4610 PRINT  
4611 '  
4620 PRINT " Airp = ";PRINT USING P3$;AIRP;PRINT" ksc ";  
4625 PRINT USING P3$;NEWAIRP;PRINT " ksc ";  
4632 PRINT USING P3$;GAIN(5);PRINT" ksc/volt-sec ";  
4634 PRINT USING P3$;MVOLTS(3);PRINT" volts";  
4649 '
4650 PRINT " FTOT = " ; PRINT USING P2$ ; FTOT ; ; PRINT" kg " ;
4655 PRINT USING P2$ ; OLDFTOT ; ; PRINT" kg " ;
4660 PRINT
4665 PRINT
4670 ROW=ROW+1 : IF ROW=20 THEN ROW=13
4680 RETURN
4690 ' 5000 '******** Re-initialize FTOT and SIGV ***********
5010 ' 5020 CLS : GOSUB 4463
5030 ROW=CSRLIN+1
5040 COLOR 0,7 : LOCATE ROW,11
5050 PRINT "Re-initialize FTOT and SIGV"
5060 COLOR 3,8
5070 PRINT "Please verify the following values"
5080 PRINT " -current air pressure, SIGV(ksc) = " ; AIRD
5090 PRINT " -target air pressure, SIGV(ksc) = " ; SIGV
5100 LOCATE 19,10 : PRINT SPACE$(50)
5110 LOCATE 19,10 : INPUT "Do you wish to change SIGV? (yes or no) " , A$  
5120 IF A$="no" THEN GOTO 5160
5130 IF A$="yes" THEN GOTO 5040
5140 FOR I=ROW+1 TO ROW+9 : LOCATE I,1 : PRINT BLKS : NEXT I
5150 LOCATE ROW+1,1
5160 INPUT "Enter the desired air pressure SIGV (ksc) " ; SIGV
5170 FOR I=ROW+1 TO ROW+9 :LOCATE I,1 : PRINT BLKS : NEXT I
5180 LOCATE ROW+1,1
5190 PRINT "Please verify the following values"
5200 PRINT " -current total caisson weight, FTOT=Wallf+Capf (kg) = " ; CAPF+WALLF
5210 PRINT " -target total caisson weight, FTOT=Wallf+Capf (kg) = " ; OLDFTOT
5220 LOCATE 19,10 : PRINT SPACE$(50)
5230 LOCATE 19,10 : INPUT "Do you wish to change FTOT? (yes or no) " , A$  
5240 IF A$="no" THEN GOTO 5280
5250 IF A$="yes" THEN GOTO 5210
5260 FOR I=ROW+1 TO ROW+9 :LOCATE I,1 : PRINT BLKS : NEXT I
5270 LOCATE ROW+1,1
5280 INPUT "Enter the desired total caisson weight, FTOT (kg) " ; OLDFTOT
5290 FOR I=ROW+1 TO ROW+9 :LOCATE I,1 : PRINT BLKS : NEXT I
5300 PRINT "Please verify your chosen values"
5310 PRINT **
5320 PRINT " -target air pressure, SIGV(ksc) = " ; SIGV
5330 PRINT **
5340 PRINT " -target total caisson weight, FTOT=Wallf+Capf (kg) = " ; OLDFTOT
5350 LOCATE 19,10 : PRINT SPACE$(50)
5360 LOCATE 19,10 : INPUT "Do you wish to change these values? (yes or no) " , A$  
5370 IF A$="no" THEN GOTO 890
5380 IF A$="yes" THEN GOTO 5350
5390 GOTO 5000
6000 '
7210 '******** Suction Drive ***********
7220 '
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7230 CLS : GOSUB 4463
7240 ROW = CSRLIN+1
7250 COLOR 0,7 : LOCATE ROW, 11
7260 PRINT "Suction Drive PARAMETER SELECTION"
7270 COLOR 3,8
7280 PRINT "This algorithm will drive the caisson walls at a constant rate"
7290 PRINT "while removing load from the cap equal to the wall load increase"
7300 PRINT "Please verify the following values"
7310 PRINT "-current caisson i.d.(cm)=", 2*SQR(CAPAR/3.14159)
7320 PRINT "-current air pressure = ", AIRP
7330 PRINT "-target air pressure = ", SIGV
7350 LOCATE 20, 10 : PRINT BLK$
7360 LOCATE 20, 10 : INPUT "Is it okay to continue (yes or no ) ", A$
7370 IF A$="no" THEN GOTO 890
7380 IF A$="yes" THEN GOTO 7350
7390 FOR I=ROW+1 TO ROW+9 : LOCATE I, 1 : PRINT BLK$ : NEXT I
7400 LOCATE ROW+1, 1
7402 TGAIN1=GAIN(1)
7404 GAIN(1)=-.0010205
7410 INPUT "enter the wall displacement rate (cm/sec) ", WALLDRATE
7440 SUCVTS1=WALLDRATE/GAIN(1)
7450 IF ABS(SUCVTS1)<.05 THEN PRINT "This rate is too slow for the gear setting"
7460 IF ABS(SUCVTS1)>4.9 THEN PRINT "This rate is too fast for the gear setting"
7470 GOTO 7540
7480 PRINT "You must change the rate or return to setup program"
7490 LOCATE 19, 10 : PRINT SPACE$(50)
7500 LOCATE 19, 10 : INPUT "Do you want to change rate (yes or no ) ", A$
7510 IF A$="no" THEN GOTO 890
7520 IF A$="yes" THEN GOTO 7540
7530 GOTO 7390
7540 FOR I=ROW+1 TO ROW+9 : LOCATE I, 1 : PRINT BLK$ : NEXT I
7550 LOCATE ROW+1, 1
7552 GAIN(1)=TGAIN1
7560 INPUT "enter the maximum wall displacement (cm) ", WALLDMAX
7561 DRIVEFLAG=1
7562 INPUT "Do you wish to start SUCTION DRIVING now? (yes or no ) ", A$
7564 IF A$="yes" THEN INPUT "Press ENTER to start SUCTION DRIVE ", A$ : GOTO 7620
7566 IF A$="no" THEN INPUT "SUCTION DRIVE parameters are set. Press ENTER to return to menu. ", A$ : GOTO 890
7567 IF A$="yes" THEN GOTO 7562
7610'
7620 '****** prepare to start driving ******
7622'
7625 MODULE=2
7630 CONTROL(1)=1 'wall disp control
7640 CONTROL(2)=1 'cap disp control
7650 STOPDEVICE=3 'keep wall and cap moving
7660 NEWAIR=SIGV 'maintain constant air press
7670 CLS : GOSUB 4450 'get initial readings
7680 LOCATE 25, 65 : COLOR 0, 7 : PRINT "SUCTION DRIVING": COLOR 3, 8
7720 LOCATE 10, 56 : PRINT "Target rate =": WALLDRATE; "cm/sec"
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7725 LOCATE 11,67: PRINT "Current rate"
7726 LOCATE 12,67: PRINT " cm/sec "
7750 GOSUB 4290 'set esc key
7760 GOSUB 4350 'F1 to HOLD STRESS
7770 GOSUB 4408 'capf gain activation
7791 WALLD0=WALLD
7793 OLDCAPF=CAPD
7794 GAIN(3)=.025
7795 VINFLAG=VINREAD
7796 ROW=12
7797 NO=TIMER
7798 MVOLTS(1)=SUCVTSL
7799 SUCFLAG=1
7800 WHILE (ENTERFLAG=0)
7801 OLDWALLF=WALLF
7805 GOSUB 3260 'take set of readings
7808 IF WALLD=WALLDMAX THEN ENTERFLAG=1: GOTO 7900
7809 ' ERRFTOT=FTOT-OLDFTOT
7810 ' DELWALLF=WALLF-OLDWALLF
7811 NEWCAPD=OLDCAPD-CAPD+WALLD0-WALLD
7820 GOSUB 2800 'control motors
7825 SUCFLAG=0
7840 IF TARGETFLAG=0 THEN GOTO 7900 ELSE
7850 GOSUB 4500 'print target/control
7860 IF ROW=20 THEN ROW=13
7870 LOCATE ROW+1,1:PRINT SPACES(75)
7872 LOCATE ROW,1:PRINT USING "####.#### ";
WALLD,WALLF,CAPD+WALLD,CAPF,AIRF,FTOT,(WALLD-WALLD0)/TINC
7873 LOCATE ROW,66:PRINT USING "###.#### ";(WALLD-WALLD0)/TINC
7874 TARGETFLAG=0
7875 WALLD0=WALLD
7900 WEND
7950 STOPDEVICE!=0 'stop all motors
7960 GOSUB 3110
7970 IF ENTERFLAG=2 THEN GOTO 890 'return to home
7975 DRIVEFLAG=0
7980 GOTO 2000 'hold stress
7990 '********* Monotonic Pullout *********
9010 '9020 CLS: GOSUB 4463
9030 ROW=CSRLIN+1
9040 COLOR 0,7: LOCATE ROW,11
9050 PRINT "Monotonic Pullout PARAMETER SELECTION"
9060 COLOR 3,8
9070 PRINT "This algorithm will pull out the caisson cap at a constant rate"
9080 PRINT "while matching the wall displacement with the cap displacement."
9090 PRINT "Please verify the following values"
9100 PRINT "current caisson o.d.(cm)= ";WALLDIA
9110 PRINT "current air pressure = ";AIRP
9115 PRINT "target air pressure = ";SIGV
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9120 LOCATE 20,10 :PRINT SPACE$(50)
9130 LOCATE 20,10 :INPUT "Is it okay to continue (yes or no) ",A$
9140 IF A$="no" THEN GOTO 9090
9150 IF A$="yes" THEN GOTO 9120
9160 FOR I=ROW+1 TO ROW+9 :LOCATE I,1:PRINT BLK$ :NEXT I
9170 LOCATE ROW+1,1
9180 INPUT "enter the wall displacement rate (cm/sec) (negative)";WALLDRATE
9185 CAPDRATE=WALLDRATE
9186 TGAIN1=GAIN(1)
9187 GAIN(1)=.000833
9190 PULVTS1=WALLDRATE/GAIN(1)
9191 MVOLTS(1)=PULVTS1
9192 GAIN(1)=TGAIN1
9200 IF ABS(PULVTS1)<.05 THEN PRINT "This rate is too slow for the gear setting";GOTO 9230
9210 IF ABS(PULVTS1)>4.9 THEN PRINT "This rate is too fast for the gear setting";GOTO 9230
9211 GAIN(3)=.00125
9212 PULVTS2=.5
9213 MVOLTS(2)=PULVTS2
9220 GOTO 9290
9230 PRINT "You must change the rate or return to setup program"
9240 LOCATE 19,10 :PRINT SPACE$(50)
9250 LOCATE 19,10 :INPUT "Do you want to change rate (yes or no) ",A$
9260 IF A$="no" THEN GOTO 9090
9270 IF A$="yes" THEN GOTO 9290
9280 GOTO 9160
9290 FOR I=ROW+1 TO ROW+9 :LOCATE I,1:PRINT BLK$ :NEXT I
9300 LOCATE ROW+1,1
9310 INPUT "enter the maximum wall displacement (cm) ";WALLMAX
9311 INPUT "enter the maximum caisson tension force, FTOT (kg) (negative)";FTOTMAX
9320 PULLFLAG=1
9322 INPUT "Do you wish to start MONOTONIC PULLOUT now? (yes or no) ", A$
9324 IF A$="yes" THEN INPUT "Press ENTER to start MONOTONIC PULLOUT ",A$;GOTO 9370
9326 IF A$="no" THEN INPUT "MONOTONIC PULLOUT parameters are set. Press ENTER to return to menu. ",A$;GOTO 890
9328 IF A$="yes" THEN GOTO 9322
9360 ' 9370 '****** prepare to start pulling ******
9372 ' 9380 CONTROL1(1)=1 'wall disp control
9385 MVOLTS(1)=PULVTS1
9390 CONTROL1(2)=1 'cap disp control
9392 MVOLTS(2)=PULVTS2
9400 STOPDEVICE=3 'keep wall and cap moving
9410 NEWAIRP=SIGV 'maintain constant air press
9420 CLS : GOSUB 4450 'setup screen
9430 LOCATE 25,60 : COLOR 0,7 : PRINT "MONOTONIC PULLOUT";COLOR 3,8
9460 LOCATE 10,50 : PRINT "Target rate =";WALLDRATE," cm/sec"
9462 LOCATE 11,67 : PRINT "Current rate"
9464 LOCATE 12,67 : PRINT " cm/sec ",WALLDRATE
9470 GOSUB 4431 'F5 to activate wall gain toggle
9480 GOSUB 4440 'F6 to activate capd gain toggle
9490 GOSUB 4290 'set esc key
9495 GOSUB 4350 'F1 to HOLD STRESS
9500 ZEDWALLD=WALLD
9501 WALLD0=WALLD
9502 CAPD0=CAPD+WALLD
9503 ZEDCAPD=CAPD
9504 OLDWALLD=WALLD
9505 OLDCAPD=CAPD+WALLD
9506 VINFLAG=VINREAD
9507 ROW=12
9508 WKPP=1
9509 WKPP=50
9510 WKV=20
9511 CKPP=1
9512 CKP=100
9513 CKV=0
9514 GAIN(2)=CKP 'temporary toggle for ckp
9515 GAIN(3)=CKPP 'temporary toggle for ckpp
9518 COUNT=1
9520 WHILE (ENTERFLAG=0)
9525 GOSUB 3260 'take set of readings
9526 IF WALLD<WALLDMAX THEN ENTERFLAG=1 : GOTO 9640
9527 IF FTOT<FTOTMAX THEN ENTERFLAG=1 : GOTO 9640
9530 IF COUNT=1 THEN GOTO 9560
9531 CKP=GAIN(2) 'temporary toggle for ckp
9532 CKPP=GAIN(3) 'temporary toggle for ckpp
9535 TARGWALLD=WALLD0+(TIMER-ZTIME)*WALLDRATE
9536 ' TARGCAPD=CAPD0+(TIMER-ZTIME)*CAPDRATE
9538 ' ERRWALLD=TARGWALLD-WALLD
9540 ERRCAPD=ZEDCAPD-CAPD
9543 ' ERRCAPD=TARGCAPD-CAPD=WALLD
9544 WALLV=(WALLD-OLDCAPD)/(MINC)
9545 CAPV=(CAPD+WALLD-OLDCAPD)/(MINC)
9546 'ERRWALLV=WALLDRATE-WALLV
9547 ERRCAPV=CAPDRATE-CAPV
9550 'pulVTS1=pulVTS1+WKPP*ERRWALLD
9552 'mVOLTS(1)=pulVTS1+WKV*ERRWALLD+WKV*ERRWALLV
9555 PULVTS2=PULVTS2-CKPP*ERRCAPD
9557 MVOLTS(2)=PULVTS2-CAPD*ERRCAPD-CKV*ERRCAPV
9560 GOSUB 2720 'control motors
9561 COUNT=COUNT+1
9562 OLDCAPD=CAPD+WALLD
9563 OLDCAPD=WALLD
9571 IF TARGETFLAG=0 THEN GOTO 9640 ELSE
9572 ' GOSUB 4500 'print target/control
9573 IF ROW=20 THEN ROW=13
9574 LOCATE ROW+1,1:PRINT SPACE$(?5)
9575 LOCATE ROW,1:PRINT USING " #.##### ";WALLD,WALLF,CAPD+WALLD,CAPF,AIRF,CAPD-ZEDCAPD
9576 LOCATE ROW,66:PRINT USING " #.##### ";(WALLD-OLDCAPD)/(MINC)
9577  TARGETFLAG=0
9640  WEND
9650  GAIN(2)=-.5  'temporary toggle reset
9655  GAIN(3)=.025 'temporary toggle reset
9690  STOPDEVICE!:=0
9700  GOSUB 3110  'stop motors
9702  INTTIME:=22
9705  MODULE:=0  'reset inttime
9707  OLDFTOT:=FTOT  'maintain last monopull flot
9710  IF ENTERFLAG=2 THEN GOTO 890  'return to home
9715  PULLFLAG:=0
9720  GOTO 2000  'hold stress