2009

Analysis of a caliche stiffened pile foundation

Richard C. Stone Jr.

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ANALYSIS OF A CALICHE STIFFENED
PILE FOUNDATION

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A dissertation submitted in partial fulfillment of
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December 2009
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Analysis of a Caliche Stiffened Pile Foundation

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ABSTRACT

Analysis of a Caliche Stiffened Pile Foundation

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Cemented carbonate deposits (known locally as "caliche") in Las Vegas have been used to support shallow and deep foundations with relatively high bearing pressures compared to soil. Most high rise structures in Las Vegas are founded on either a large mat or a long pile foundation. Recently, a new foundation type consisting of a short pile system bonded to shallow cemented layers was utilized for a large high rise building in Las Vegas and settlements during construction were recorded. The bonding of caliche layers together with short piles forms a caliche stiffened pile (CSP) foundation.

The CSP foundation is unique since it derives stiffness from both piles and near surface caliche layers. This type of foundation is a new concept for building support, so this research formulates a method of analysis for the CSP foundation, and compares predicted to measured foundation settlement. The performance of the CSP foundation is also compared to a conventional pile foundation.

The settlement behavior of a spread footing, single pile and 4 pile group in a layered soil-caliche profile has been studied using both 2D and 3D finite element models. The results indicate that the presence of a thin layer of high elastic modulus in a soil profile
has a significant settlement reducing effect. Regarding a pile group in this profile, the settlement reducing effect due to the presence of caliche layers in a soil profile is greatest when stiff layers are present at both the top and below the pile tip. For a single pile in a caliche stiffened profile, the presence of the upper caliche layer causes an increase in the vertical stress adjacent to the pile due to the plate or beam effect. The analysis of a caliche stiffened pile (CSP) foundation system affects the load distribution and results in a more uniform stress distribution at the base of the lower caliche layer compared to a pile foundation in soil. The load distribution of a full scale pile load test in a soil/caliche profile was accurately predicted using both 2D and 3D finite element models. A case study building foundation was modeled using 2D and 3D models, and predicted settlements are compared to measured data. An analysis of the case study foundation indicates that increasing the pile length by 100 percent reduces the settlement by only 10 percent. Predictions of excess pore pressures and tensile stress in the caliche layer below pile tips were similar for both the 2D and 3D models. The settlement distribution along the building length including the building ends was reasonably predicted by the 3D model, but the model over predicted settlements where the upper caliche layer was thickest. The research indicates that the simpler 2D plane strain model can provide a reasonable initial prediction of settlement but limited information regarding anticipated differential settlements. Based on this research effort, guidelines for design of a CSP foundation are presented.
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ACKNOWLEDGEMENTS

I am pleased to take this opportunity to acknowledge and thank a number of people who have contributed to my success in completing this dissertation.

First, I want to express sincere thanks to my advisor and dissertation committee chair, Dr. Moses Karkouzian, for his guidance, support, and patience throughout this extended research and writing effort. I am very grateful for his constant encouragement to complete this project. I am also grateful to Drs. Ladkany, Rigby, Trabia, and Hodge for their participation and their valuable comments on the written material.

I thank the firm of Western Technologies, Inc. for providing access to the invaluable data that made this dissertation possible and for their financial assistance with education expenses while I was employed by them.

Additionally, I want to acknowledge Kleinfelder, Inc. for their permission to use the valuable data obtained from the NDOT project at I-15/U.S. Hwy. 95 in Las Vegas, Nevada.

Appreciation and thanks to my employer, Morris-Shea Bridge Company, Inc., for their interest in advancing the research of pile foundations. Morris-Shea provided the PLAXIS software, pile load testing, and instrumentation for the unique case study project.

I thank my wife, Karen, for her patience and assistance throughout the many years I worked on this dissertation.

This dissertation is dedicated to my mother, Nancy Holub and to my late stepfather, Carl Holub. They encouraged me to pursue a career in engineering and I am very thankful for their love and support.
CHAPTER 1
INTRODUCTION

1.1 Introduction

In an area that has seen some of the most vibrant growth in the country, construction in Las Vegas has encountered a variety of soil types such as sands, gravel, clays and cemented soils. The carbonaceous cemented soils, also known as caliche, commonly occur in most areas of Las Vegas and other arid regions (Wyman et al. 1993). Results of soil borings indicate that this material can be highly variable in thickness and presence at a site, resulting in difficulties during construction. Caliche deposits in Las Vegas have been used to support shallow and deep foundations, with relatively high bearing pressures (Cibor, 1983). Numerous high rise buildings have been built in Las Vegas since the 1980’s, including many 30 to 60 story condominium and hotel structures (e.g., WTI, 2002a and Langan, 2006).

In 2003, a new foundation type consisting of a short pile system bonded to shallow cemented layers was utilized for a large high rise building in Las Vegas (WTI, 2002a). The bonding of one or more caliche layers together with short piles forms a mat type foundation. Since the cemented deposits are continuous beyond the building area, the pile-mat system acts as a large laminated plate on elastic foundation, with the center of the plate system stiffened by the short piles. The caliche stiffened pile (CSP) system affects the load/stress distribution and results in more uniform stress distribution at the base of the caliche mat when compared to a conventional pile foundation. When compared to a thick mat foundation, the caliche stiffened pile foundation does not require mass excavation and removal of the cemented deposits, and utilizes the naturally stiffened laminate consisting of
caliche and soil layers. This combined with short, high capacity piles results in a very economical foundation compared to a conventional pile supported foundation.

Typical pile supported high rise building foundation systems in Las Vegas consist of isolated column pile caps and pile-raft type footings for shear wall loads (e.g., WTI, 2003a). All elements are usually connected by grade beams which are typically 2 to 3 foot square in cross section. A complete analysis of the foundation should consider the interaction between the isolated pile caps and the pile raft (core) areas, and variations in caliche thickness and soil conditions across the site. As such, a 3-dimensional finite element analysis is best suited for this purpose.

1.2 Research Objectives

The research will focus on analyzing the caliche stiffened pile (CSP) foundation system. The foundation will be analyzed using the finite element method due to its versatility and the lack of analytical solutions for the settlement of pile foundation in layered soil with high stiffness contrasts. Using the numerical model PLAXIS (Brinkgreve and Swolfs, 2007; Brinkgreve et al. 2008), the settlement characteristics of foundations established on profiles with cemented layers and short piles (pile groups) will be investigated. Model settlements of a case study building foundation will be compared to measured settlements. Model parameters such as soil and caliche elastic moduli will be determined by a back-calculation procedure based on pile load test data and a test fill constructed at the site.

Comparisons of settlements will be made between the CSP foundation and conventional pile raft foundation, with and without caliche layers. The interaction between adjacent CSP groups will be studied. It is anticipated that the result of the research will
give new understanding to utilizing naturally stiff materials as contributing foundation units. Using the derived model parameters, a settlement analysis will be performed and compared to the measured settlement data.

To achieve the research objectives, the effort is divided into four tasks, as follows:

1) Review and evaluate existing techniques for estimating ultimate side friction of bored piles in rock, and the construction/design of high rise buildings on soil profiles containing thin rock layers in Miami, FL (Kaderabek and Reynolds, 1981).

2) Develop a numerical model and evaluate the characteristics of foundations in soil profiles containing caliche layers.

3) Evaluate the results of field and laboratory tests, and perform a back analysis to determine parameters for use in the numerical model.

4) Compare measured data to calculated settlement data.

5) Propose a design method based on the results of the analyses.

The numerical results will be applied to a case study building in Las Vegas where settlements have been carefully monitored. Full scale pile load tests have been performed at the site to evaluate pile load transfer characteristics (WTI, 2002b). The measured settlement behavior of the building will be studied and related to the load transfer through the caliche stiffened pile foundation. An improved method of evaluating settlements of foundations on such soil profiles will be discussed and applied to the case study building.

1.3 Organization

A summary of the dissertation organization follows.

In Chapter 2, foundation systems used for high rise construction in Las Vegas are discussed. The Caliche Stiffened Pile (CSP) foundation system is explained. Application
to a high rise project in Las Vegas is detailed.

Chapter 3 discusses previous literature relevant to ultimate friction resistance of bored piles in rock. Aspects of foundation design on thin rock layers, such as in Miami, Florida, are also discussed. Other issues concerning to foundation design in rock are discussed.

In Chapter 4, the field and laboratory data produced during the design, evaluation and exploration of a case study high rise project are detailed. Relevant data from other sites in Las Vegas is also presented, including laboratory tests of caliche from three sites in Las Vegas.

Chapter 5 presents analysis of a typical spread footing, a single pile and a CSP pile group system by the commercial finite element program PLAXIS. The focus of the analysis is the effect of a stiff caliche layer(s) on the settlement and load distribution behavior. Comparisons are made with a 3D analysis (PLAXIS 3D Foundation, Brinkgreve at el., 2007).

Chapter 6 details the evaluation of existing data for caliche and the process of back analysis of the field data, to determine soil properties to be used in detailed settlement analyses. Back-analyses of pile load test and test fill data are performed using 2D and 3D finite element models. The back analysis focuses on in-situ properties of caliche and fine-grained soil.

In Chapter 7, the results of previous analyses are summarized into a model soil profile and a detailed settlement analysis of the case study foundation is performed using both 2D (PLAXIS v8) and 3D methods (PLAXIS 3D Foundation). The data from both types of analyses are presented with the measured data.
A comparison between the predicted (numerical) and measured settlement data is presented in Chapter 8. The differences between the results and the applicability of the settlement method based on back calculated field test data is discussed. The settlement behavior of the subject structure during its construction is also evaluated. Recommendations for further research of foundations in layered soil/caliche profiles are presented.
2.1 Geologic Profiles in Las Vegas

Las Vegas is located within the Basin and Range Province which consists of north-south trending mountain ranges separated by shallow valleys. The city is bounded on the west, south and east by mountains. The mountains to the west and east of Las Vegas are composed primarily of limestone and dolomite, while the mountains to the south consist of tertiary volcanics. Unconsolidated sediments of sand, silt, and clay, thousands of meters thick, are found in the center of the valley (Rodgers et al. 2006).

Mid-Pleistocene alluvial terraces exist at the base of the mountains surrounding the valley (Wyman et al. 1993). These areas are composed of granular materials. Significant deposits of cemented sand and gravel materials are found in the alluvial terraces which were exposed by late Pleistocene and Holocene erosion (Wyman et al. 1993). The cemented sand and gravel deposits in alluvial fan areas consist of granular materials and cementitious agents which are derived from the weathering of the adjacent mountains.

Cemented soils are found in most parts of the Las Vegas valley. These materials consist of sand and gravel particles cemented by calcium carbonate, or a finer-grained material consisting primarily of calcium carbonate (caliche). The cemented deposits are generally found in the central and western areas of the valley (Wyman et al. 1993).

Caliche in the Las Vegas is formed by lithification of fine grained sediments from evaporation of lime-rich (calcium carbonate) groundwater introduced through capillary action or precipitation (Cibor, 1983). The fluctuation of the ground water level and
intrusion of surface water into the soil has resulted in caliche layers at various depths and thicknesses. A low moisture condition is required to induce the precipitation process.

The presence of cemented deposits at a particular site in Las Vegas can be very erratic. The materials can vary widely in thickness, hardness and depth over a short distance. For example, at the Stratosphere Tower site, the cemented deposits ranged in thickness from about 10 feet at the Sky Tower location to ½ foot at the hotel structure location, over a distance of approximately 100 feet (WTI, 1994a). Aspects of engineering with cemented soils are discussed by Stone and Luke (2001), and Werle and Luke (2007).

Along the Las Vegas Strip area, where many high rise buildings are constructed, it is typical to have many near surface layers of caliche interbedded within a sandy clay (or clayey sand) material to great depth. The clay is usually very stiff and often includes gravel sized particles, making sampling difficult and resulting in very disturbed samples.

2.2 High Rise Foundations in Las Vegas

There are limited references regarding geotechnical engineering for foundations in Las Vegas. The papers by Wyman et al. (1993) and Cibor (1980) are likely the most referenced resources. Wyman et al. discusses many geotechnical issues involved with development in Las Vegas, and the Cibor article, in addition to basic geotechnical information, includes discussions on both shallow and deep foundations for high rise buildings. Most foundation investigation projects before 1980 were conducted by either Converse Consultants or Nevada Testing Laboratories (NTL). Prior to the early 1970’s, NTL provided most of the geotechnical design services for Las Vegas. The original data files from NTL dating back to the 1950’s, including boring data, are still housed at the
Las Vegas office of Western Technologies, Inc.

Central Las Vegas has a unique soil profile which consists of clays, sands and interbedded caliche layers. The soils are usually stiff or dense, and commonly mixed with gravel, making undisturbed sampling difficult. After sampling, the goal of geotechnical engineers is to establish the properties of the soils by traditional means, but there are limited sampling methods which are useful in this profile. Samples are commonly obtained by driving a split-spoon sampler (California modified ring sampler) filled with brass rings which fit directly into either an oedometer or direct shear testing device. Samples obtained with the modified California ring sampler are considered as disturbed samples (Mayne et al. 2001) and therefore generally not considered suitable for consolidation or shear testing. Shelby tube samples are rarely obtained since they usually become destroyed when pushing through stiff/dense soils with caliche gravel. Hammers with various weights between 140 and 300 pounds are used to drive the sampler, so published correlations for strength and stiffness properties of soil with SPT N-values (e.g., Bowles, 1996) may not be directly applicable.

Most high rise structures in Las Vegas are founded on either a mat or a pile foundation (Cibor, 1983). Both foundation systems utilize the caliche for foundation support, if it is present at or near planned foundation bearing elevations. Some structures (e.g., Rio Hotel) have utilized "extended" spread foundations which bear on caliche at depth, where the uncemented soil between the foundation and the underlying caliche layer has been replaced with low strength concrete.
Drilled shafts (or piers) have been used extensively in Las Vegas due to hard drilling conditions, although driven piles and continuous flight auger (CFA) piles have been utilized for some projects. Temporary casing for drilled shafts is typically not required since the foundation soils are often partially cemented or stiff in consistency. Additionally, the sandy clay or clayey sand soil which is commonly present in the center of the valley is mixed with ground water during drilling and acts as a suitable slurry to maintain hole stability. Driven piles are usually not used along the Las Vegas Strip due to near surface cemented soils. Driven piles have been used in early Nevada Department of Transportation (NDOT) bridge overpass applications, and more recently to support additions to the Clark County Sanitation District water treatment facility (GES, 1998).

The author of this dissertation has conducted approximately 25 geotechnical investigations for high rise buildings in Las Vegas since 1992. Most hotel towers utilized mat foundations throughout the early to mid 1990’s. As buildings became taller, deep foundations became more widely used as a primary foundation element. Boulder Station, Aladdin, and Stratosphere hotel developments were among the first projects since 1990 to use deep foundations as a primary support system. Piles were often used to support roller coasters, marquee signs and other specialty structures subject to large overturning loads (WTI, 1998).

High rise building settlements became a quantity of more interest in 1998 after a mat foundation for a 40+ story hotel experienced a settlement of approximately 20 inches in its central core. Combined with the increasing cost of concrete, this helped to popularize the use of deep foundations as a settlement limiting foundation, compared to the large mat foundation. Deep foundations are also common for hotel tower expansions
to control differential settlements between the previous (usually mat-supported) and the new portions of a tower (WTI, 2003a). Based on the author’s professional experience in Las Vegas, drilled pile foundations are now commonly used for high rise buildings.

Mat foundations in Las Vegas have typically been designed for allowable bearing pressures of 5 to 10 kips per square foot (ksf), and have average thicknesses of 5 to 10 feet (WTI, 2003b). For mat (or shallow) foundations bearing on caliche, bearing pressures on the order of 10 to 20 ksf have been used (WTI, 1997). The finish floor elevations of the lowest building level often dictate whether a mat foundation can be used, as it is desirable to bear the mat directly on top of the caliche deposit. Foundation elevations below the upper caliche deposit often require the use of piles, depending upon estimated total and differential settlements.

Pile foundations in Las Vegas are typically designed for friction, as most projects along the Las Vegas Strip have shallow water tables which render pile bottom inspection less reliable. Friction pile foundations may tip into caliche deposits, but it is generally not mandated due to the high variability of caliche presence at a site. Typical design friction values have ranged from 1 to 7 ksf in soil, and 10 to 20 ksf in caliche. For heavy loads, friction pile lengths of 50 to 100 plus feet are common. Figure 2.1 indicates general foundation applications in Las Vegas.
A few projects have utilized end-bearing straight (Showcase Mall) and belled (larger diameter at the pile tip) drilled piers (Caesars Hotel and Showcase Mall parking garages) which bear directly on caliche. In this application, shallow, above water, unweathered caliche deposits which are continuous across the site, are required. Design pile end bearing pressures of 20 to 60 ksf have been used successfully. These piles are usually less than 20 feet in length to allow for base inspection.

Pile load tests in Las Vegas today are common and have been performed at many high rise project sites. Load testing, as an integral part of the design process, began with the Boulder Station site in 1995, of which the author was the geotechnical engineer. Prior to that time, pile load testing was rare and typically only proof load (design load)
tests were performed rather than load tests to failure. Today, Osterberg load tests (e.g., Osterberg, 1989) are performed at most sites in Las Vegas for high rise building projects. The load test data is used to either confirm or increase pile capacities, and to a lesser extent, to provide meaningful data for settlement calculations. The presence of a significant caliche layer is considered a benefit to individual pile capacity, but it is not typically considered as a significant contribution to the overall foundation performance, such as providing the stiffening effect of a mat foundation.

In the last five years, micro-piles as ground improvement elements have been used to support high rise and other structures. This system combines the site improvement characteristics of a large micro-pile group with an overlying, separated mat foundation for total and differential settlement control (Vanderpool, 2002).

2.3 Caliche Stiffened Pile Foundation

The Caliche Stiffened Pile (CSP) foundation consists of short, high-capacity drilled piles which are well bonded to the upper layer of caliche at a site and are tipped or end-bear on a lower caliche layer. The piles bond the two stiff layers together and the system acts as a pile foundation which derives additional stiffness from the caliche layers. Since the caliche layers are continuous across the site, the system acts like a large, laminated plate on elastic foundation. This plate action provides additional settlement, controlling stiffness to the foundation system.

It is important that the upper caliche deposit has sufficient stiffness such that short piles may develop high capacities under minimal interface deflection (rough socket condition), as would be the case for augered piles in a hard rock deposit. Figure 2.2 depicts the CSP system in a typical Las Vegas soil profile. In Chapter 5, the CSP system
is demonstrated to be more efficient at settlement control with caliche layers at both the pile top and below the tip.

Pile caps are constructed above or slightly into the upper caliche layer. The settlement contribution from the first upper soil layer is minimized due to the pile action. One advantage of the CSP system over a mat foundation directly on the top of a caliche layer is that the large concrete volume required for a mat foundation is reduced and the settlement of the upper soil layer is significantly reduced. Another advantage of the CSP system over a typical long pile system is the short pile length, thereby, decreasing cost and construction time. Additionally, like a mat foundation, the differential settlement is more controlled due to the stiffness of the caliche layers. Figure 2.3 shows a typical single pile cap of a CSP foundation.

2.4 Application – Case Study Hotel Tower

A caliche stiffened pile (CSP) foundation has been implemented for one of the tallest hotels on the Las Vegas Strip. The case study hotel tower is 625-feet in height and has 51 structural levels, constructed of reinforced concrete with post-tensioned floor slabs. The hotel structure has a circular arc shape and is about 700 feet in length and 80 feet in width, as seen in Figures 2.4 and 2.5. An aerial photograph taken during construction is shown in Figure 2.6.

The soil conditions generally consist of an upper caliche layer with a thickness ranging between 8 and 16 feet, overlying a soil layer. The next lower caliche layer has an average thickness of about 5 feet. More details regarding the soil conditions are discussed in Chapter 4.
Figure 2.2  Typical CSP foundation system

Figure 2.3  Drawing of single CSP cap foundation system
Structural loads are usually termed as dead, or live loads. Dead loads (D) are those that are fixed, such as the building frame, while live loads (L) consist of non-permanent items, such as furniture, wind and seismic loads, and people. Dead plus live-column (D+L) loads for the case study building ranged between 5,000 and 9,000 kips for a typical column line which equates to a vertical contact pressure of 9.7 ksf for the 35 ft. by 80 ft. tributary area associated with each column line. The building includes two elevator cores and two isolated shear walls. Dead plus live-contact pressure loads in the elevator cores are about 14 ksf.

The building is supported on a CSP foundation consisting of the upper and lower caliche layers and 300 continuous flight auger (CFA) piles with a diameter of 1 meter. Piles had typical lengths of 30 to 35 feet below pile cap bottoms, and were tipped into the
lower caliche deposit a minimum of 2 feet. The structure was designed to accommodate a maximum settlement of 5 inches, and a differential settlement between columns of one inch. Settlements of the building were monitored by installing steel pins in the first floor columns. Settlement data was obtained monthly and the construction schedule allowed one floor per week to be completed. A detailed analysis of the settlement data is presented in Chapters 7 and 8.

Figure 2.5  Case study hotel tower boring plan (WTI, 2002b)
Figure 2.6 Case study site aerial photograph (from Clark County, NV GIS web site)
CHAPTER 3
RELATED LITERATURE

3.1 General

The success of the CSP foundation relies on adequate bonding of piles to a caliche deposit at the pile top and tip, thereby ensuring load transfer to (and thus benefit from) the plate stiffness effect of a caliche layer. Piles may be extended below the first upper caliche zone to further reduce settlements, as was done for a few piles at the case study site. A CSP utilizes piles that are relatively short with high capacities which develop in the upper caliche (rock) material. Under the design loads, typical friction piles in Las Vegas caliche material have average shear values much less than ultimate. A review of literature related to ultimate bored pile capacities in rock (or caliche) will be presented.

The author is not aware of any direct applications of this type of foundation system in the literature as high rise construction in a layered caliche-soil profile is unique to Las Vegas. However, as discussed in Section 3.3, there are similar applications in Miami, Florida, where, in some areas, there is oolitic limestone near the surface, underlain by loose sand.

3.2 Bored Pile Friction Capacity in Rock

The most common use of pile foundations is to transfer heavy building loads through more compressible overlying sediments to competent materials below. Often, the underlying competent stratum is rock, such as in Chicago where end-bearing drilled piers on rock are used (e.g., Baker, 1993). The piles may develop their capacity through either side resistance or end-bearing, or from a combination thereof.
In the case of piles bearing into or through rock formations, it is common to design for frictional capacity and neglect end-bearing effects. This is due to the need for inspection and cleaning of the pile base if an end-bearing load effect is included; however, the shaft bottom should always be partially cleaned of loose rock/soil (O’Neill and Reese, 1999).

Drilled piles installed in rock sockets have been shown to carry very high loads. In the last 30 years, there have been several studies regarding design of piles for ultimate friction (adhesion) capacity in rock and rock like materials. Most of these are based on the unconfined compressive strength (UCS) of the rock material, usually of the form

$$f_{su} = a \cdot (q_u)^b$$ (Zhang, 1999),

where,

$$f_{su} = \text{ultimate skin friction or bond capacity},$$

$$q_u = \text{unconfined compressive strength}, \text{ and } a \text{ and } b \text{ are constants.}$$

All relations generally accept that the controlling unconfined compressive strength is the weaker of the rock, or concrete. Most published correlations are for sedimentary rocks which have higher bond strengths than granitic and volcanic rocks (Ng et al. 2001). Williams and Pells (1981) have speculated that construction techniques in harder rocks may result in smoother sockets.

A summary of $$f_{su}$$ ($$f_{su}$$ and $$q_u$$ in MPa) relations for sedimentary materials, based on various researchers is shown in Table 3.1.
Table 3.1 Ultimate skin friction coefficients in rock

<table>
<thead>
<tr>
<th>Reference</th>
<th>a</th>
<th>b</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Williams et al. (1980)</td>
<td>0.44</td>
<td>0.37</td>
<td>based on 0.2-0.6 m drilled shafts in shale</td>
</tr>
<tr>
<td>Rowe &amp; Armitage (1987)</td>
<td>0.45</td>
<td>0.5</td>
<td>Clean sockets</td>
</tr>
<tr>
<td>Rowe &amp; Armitage (1987)</td>
<td>0.6</td>
<td>0.5</td>
<td>Rough sockets</td>
</tr>
<tr>
<td>Horvath &amp; Kenney (1979)</td>
<td>0.2</td>
<td>0.5</td>
<td>Smooth sockets; based on 0.4-1.2 m drilled shafts</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>in shale &amp; mudstone</td>
</tr>
<tr>
<td>Horvath &amp; Kenney (1979)</td>
<td>0.3</td>
<td>0.5</td>
<td>Rough sockets; based on 0.4-1.2 m drilled shafts</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>in shale &amp; mudstone</td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>in shale &amp; mudstone</td>
</tr>
<tr>
<td>Reese &amp; O’Neill (1988)</td>
<td>0.15</td>
<td>1</td>
<td>UCS &lt; 1.9 Mpa</td>
</tr>
<tr>
<td>Reese &amp; O’Neill (1988)</td>
<td>0.2</td>
<td>0.5</td>
<td>UCS &gt; 1.9 Mpa</td>
</tr>
<tr>
<td>Rosenberg &amp; Journeaux (1976)</td>
<td>0.375</td>
<td>0.515</td>
<td>based on 0.2-0.6 m drilled shafts in shale</td>
</tr>
<tr>
<td>Carter &amp; Kulhawy (1988)</td>
<td>0.2</td>
<td>0.5</td>
<td>N/A</td>
</tr>
<tr>
<td>Reynolds &amp; Kaderabek (1980)</td>
<td>0.3</td>
<td>1</td>
<td>Florida limestone</td>
</tr>
<tr>
<td>Gupton &amp; Logan (1984)</td>
<td>0.2</td>
<td>1</td>
<td>Florida limestone</td>
</tr>
<tr>
<td>Toh et al. (1989)</td>
<td>0.25</td>
<td>1</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Based on a numerical study and simplified Mohr’s circle relations, McVay et al. (1992) found that the best predictive results for Florida limestone resulted when the unconfined compressive strength was combined with the tensile strength from splitting tension tests, as shown below:

\[ f_{uw} = \frac{1}{2} \sqrt{q_u \sqrt{q_t}}, \text{ where,} \]

\[ q_t = \text{splitting tensile strength}. \]

Moreover, McVay states that numerical studies show that the ultimate bond strength is in close proximity to the rock’s cohesion value.
In his dissertation research, Zhang (1999) studied numerous load tests and concluded the power law relation provides better results with $b=0.5$, or

$$f_{su} = a\sqrt{qu} \text{ (MPa)}$$

(3.3)

where $a = 0.4$ for smooth sockets, and 0.8 for rough sockets.

Zhang and Einstein (1998) also studied the ultimate end bearing pressure ($q_{ult}$) for rock socketed drilled piles, and based on 39 load tests and recommended the following relation:

$$q_{ult} = 4.72\sqrt{qu} \text{ (MPa)}$$

(3.4)

AASHTO (2002) recommends unit shear values for rock based on the following chart which is based on work from Horvath et al. (1983) for smooth wall shafts.

![Figure 3.1 Ultimate side resistance of rock socketed shafts (from AASHTO, 2002)](image)
Kulhawy and Phoon (1993) evaluated ultimate side resistance in rock socketed piles based on a wide database which included data from Rowe and Armitage (1987) which had 80 load tests from 20 sites. Also included, were 47 tests to failure from 23 sites from Bloomquist and Townsend (1991), and McVay et al. (1992). Based on a linear regression of the data, a best fit expression was determined as:

\[
\frac{f_{su}}{p_a} = \Psi \sqrt[4]{\frac{q_u}{2p_a}}
\]

(3.5)

where,

\[p_a = \text{atmospheric pressure (0.1013 MPa)}.\]

The coefficient \(\Psi = 1\) for a lower bound, \(\Psi = 2\) for a mean value and \(\Psi = 3\) for an upper bound such as artificially roughened sockets. The averaged data from each site is plotted in Figure 3.2. According to the authors, the solid black squares on the plot are thought to represent artificially roughened sockets.

Kulhawy et al. (2005) re-evaluated the data including only load tests to failure using the same failure criteria. They recommend using Eq. 3.5 with \(\Psi = 1\) for ultimate side resistance of normal rock sockets from drilled shaft construction. Drilled shafts are constructed using numerous passes into and out of the hole, thereby, somewhat smoothing the hole sidewalls.
Figure 3.2 Normalized side resistance of rock socketed shafts (from Kulhawy and Phoon, 1993)

Note that the piles installed at the case study site are continuous flight auger (CFA) piles with a rock auger in which the auger is penetrated into the hole one time and the concrete is pumped as the auger is withdrawn. Therefore, the sidewalls in caliche are expected to be rough, as further discussed in Chapter 4.

Randolph and Leong (1994) performed finite element analyses of rock socketed piles with side shear only. They concluded that the ultimate shear stress was very dependent on the rock mass modulus. Additionally they state that the ultimate shear stress increases with both an increasing length to diameter ratio, and a decreasing socket diameter. In their analyses they concluded that the peak shear stress is reached at a displacement of 0.008D, where D = pile socket diameter. Horvath and Kenney (1979) also found that the displacement to peak shear was 0.5 to 1.5% of the socket diameter.
Radhakrishnan and Leung (1989) performed instrumented field tests on drilled shafts in rock and concluded, in agreement with Osterberg and Gill (1973), that at working loads, the behavior of the pile in rock is elastic. Based on the load tests, Radhakrishnan et al. (1989) concluded that the majority of the load is transferred in shaft friction in the upper two diameters, and additional pile length beyond two diameters does not appreciably increase the pile capacity.

Based on results from instrumented piles in a high rise building foundation, Radhakrishnan and Leung (1989) found that the rock socket side shear load observed in a pile load test reduces over time under the service loads which results in additional load in end bearing. The portion of end bearing load in one pile nearly doubled over time, under constant load. It was reported that this was due to pile creep, pile group interaction and the presence of a rigid pile cap, as mentioned by Cooke et al. (1979) and O'Neill et al. (1982) for piles in different soils. The end bearing percentage was expected to further increase over time. This behavior was also reported by Ladyani (1977). Leung and Radhakrishnan (1985) also observed similar behavior in the pile foundations for a 42 story building.

3.3 Foundations on Thin Rock Layers

In areas where foundations are constructed on rock, special geotechnical design conditions should be considered, especially if the supporting rock layer is underlain by a weaker material.

The geotechnical conditions in the Miami, Florida area are more similar to the geotechnical conditions in Las Vegas than in other areas of the country where high rise buildings are often constructed. In the Miami area, foundations are often constructed on
near surface limestone layers where the subsurface conditions are characterized by a near surface limestone deposit, termed “Miami Limestone”. This material is an oolitic, fossiliferous limestone formed by repeated precipitation of calcium carbonate around sand or shell particles (Kaderabek and Reynolds, 1981). A generalized subsurface profile form the Miami area is shown in Figure 3.3.

The limestone material used to support foundations in Miami can be very porous or well inundated. Typical values of unconfined compression, spitting tensile strength and elastic modulus are 31, 8, and 12,000 ksf (215, 56, and 84,000 psi), respectively. Due to its tensile strength, a thin limestone layer may act as a large “mat” on an elastic foundation when underlain by a less stiff soil (Kaderabek and Reynolds, 1981). This is very similar to foundations on a caliche layer in Las Vegas underlain by clayey soil, although the caliche may have higher strength. The mat effect for foundations supported on relatively thin rock like materials underlain by a softer soil would also be applicable to a soil/caliche profile in Las Vegas.

Kaderabek and Reynolds (1981) note that in designing foundations to be supported on the oolitic limestone “mat”, four criteria should be considered in design.

1) Punching shear
2) Local crushing due to high stresses from foundation contact
3) Beam tension failure
4) Settlement

Kaderabek and Reynolds (1981) also mention that when evaluating settlements of foundations bearing in the Miami limestone, the effects of adjacent loaded foundations should be considered due to the beam action of the rock layer. This beam action results
in stress overlap even for widely spaced foundations, so one could conclude that the “mat” action affects a stress spread over a large area. A field test of a footing on a thin rock layer was performed in an effort to induce a failure in bending. The estimated induced tensile stress was twice that measured in the laboratory based on splitting tensile tests and no failure occurred.

Sowers (1970) and Wyllie (1999) also indicate that shallow foundations on thin rock layers overlying a weaker material can fail by either punching shear for a thin rock layer, or flexure in the case of a thick rock layer. Hoek and Londe (1974) report the case of a tall building with a weight of 450,000 kips which punched through a 33 foot thick limestone layer that was underlain by a weaker material.

In regard to pile foundations in the Miami limestone, Kaderabek and Reynolds (1981) note that the limestone material has considerable shear strength and the load transfer from augercast piles is primarily by shear friction. This conclusion is based on rock anchor tests and very low settlements during pile load tests.

Frizzi and Meyer (2003) presented a summary of the settlement behavior of tall buildings supported on deep foundations in southeast Florida, primarily in the Miami area. They present a list of pile supported high rise buildings with settlements up to approximately 12 inches. Of the approximately twenty high rise projects presented, they conclude that most buildings taller than 20 stories will experience settlements exceeding 1/3 to 1/2 inch. The work of Frizzi and Meyer highlights that although pile load tests in interlayered rock/soil profiles verify the use of very high pile design capacities, it does not indicate that settlements of pile foundations will be minimal, nor does it represent the total building response. Additionally, an evaluation of the overall building settlement
response should consider pile group effects and all load cases.

Figure 3.3 Generalized subsurface profile in SE Florida (from Frizzi and Meyer, 2003)

3.4 Conclusions

Piles installed in rock that develop their capacity from side friction are commonly designed using empirical expressions based on the unconfined compressive strength of the rock or pile concrete whichever is weaker. For CFA piles installed in caliche, the sidewalls will be rough. The peak shear tends to increase with increasing L/D (length/diameter), and generally reaches a peak value at a deflection of about 1% of the socket diameter.

At service or working pile loads, the pile behavior in rock will be elastic. The
majority of the pile load will be transferred in the upper two diameters.

Field measurements of pile loads, after construction, have indicated that side shear in rock sockets tends to decrease with time as load is transferred to the pile tip.

Thin rock layers in Miami can act as mat on an elastic foundation with a beam action effect. The same rock behavior can be expected for caliche layers in Las Vegas. Several design criteria should be evaluated when designing foundations supported by thin rock layers. Some high rise buildings in Miami have experienced large settlements, although the pile load tests indicated high design capacities were available based on the results of pile load tests. The settlement evaluation of a building foundation should include the interaction between all pile groups.
CHAPTER 4

FIELD AND LABORATORY DATA

4.1 Introduction

The site of the case study project is located on the Las Vegas Strip, south of the downtown area. A large number of geotechnical investigations for various projects at the site have been performed which resulted in numerous borings and associated laboratory data. Some of the data that was acquired from laboratory and field tests (site exploration) at the site is discussed below.

4.2 Generalized Soil Profile

Since the 1970’s, over 75 borings have been performed at the site by various local geotechnical firms. The most notable geotechnical investigations were performed by Kleinfelder, Inc., (KI, 2001), Terracon Consultants Western, Inc. (TER, 1994), and Western Technologies, Inc. (WTI, 2002a).

A review of the boring data indicates the upper caliche layer (approximate elevation 2060–2050 feet) is continuous across the site. This layer has a depth of about 10 feet or greater in most borings and a 1.5 to 2 foot soil layer is typically encountered within the upper caliche zone, as indicated in Figure 4.1. During pile installation, pre-drilling was performed and the thickness logged at each of the 300 pile locations. A 3D surface plot of the pre-drilling data is shown in Figure 4.3. Note that at the time of the pre-drilling, all areas were excavated to the bottom of cap or core mat elevations. The thinner caliche in the core locations are observed in Figure 4.3. The data on the plot indicates the actual upper caliche layer thicknesses through which the piles are installed.

Below the upper caliche layer, a second layer exists at about elevation 2030 feet...
which is also continuous across the site. The presence of caliche zones becomes more random below the second layer.

Boring depths range from shallow up to 200 feet. The upper soils at the site consist of sands and gravels. Below the upper granular soils, the soils consist primarily of fine-grained clayey soils which may be classified as clayey sands in most instances (see Section 4.3.4).

![Figure 4.1 Photograph of upper caliche layer (by author)](image)
Figure 4.2 Soil profile based on select boring data
Laboratory Data

Soil samples were obtained from the case study site during the exploration phase of the project. Samples were obtained by using a standard split-spoon (SPT) sampler (2.0 inch O.D., 1.375 inch I.D.) and a modified California split-spoon (ring) sampler (2.5 inch O.D., 1.925 inch I.D.). Soil testing in the laboratory consisted of consolidation tests, Atterberg Limits and direct shear tests. Associated with these tests were density tests performed by three geotechnical firms, the results of which are shown in Figure 4.4. The moist densities range between 85 and 145 pcf. The lowest densities are in the 50 to 70 foot depth zone.
Consolidation Testing

Consolidation tests were performed by Kleinfelder during the initial case study site exploration in 2001. Samples were obtained using the ASTM D3550 ring sampler, and as might be suspected the laboratory tests appear to be from disturbed samples since the test data curve are relatively flat (Holtz and Kovacs, 1981). Such test data can sometimes be reconstructed to yield some information (Schmertmann, 1955).

The determination of the preconsolidation pressure is one of the most important results from a consolidation test. Unfortunately, disturbed soil samples do not provide an accurate assessment of this parameter as the data “knee” near the preconsolidation

Figure 4.4 Moist density test data (WTI, 2002a)
pressure is lost (Spangler and Handy, 1982). A typical disturbed sample consolidation
test result from the subject site is shown in Figure 4.5.

Figure 4.5 Typical consolidation test data from subject site (KI, 2001)

4.3.2 Atterberg Limits

Atterberg Limit and field moisture content tests were performed by Kleinfelder
and Western Technologies during the field exploration. Since high quality undisturbed
samples are difficult to obtain and in-situ testing is not commonly performed due to
caliche layers and hard soils, information regarding the in-situ soil stress state and compressibility was developed using correlations with the moisture content and Atterberg Limits test data (see Figure 4.6). From this data, the Liquidity Index (LI) is calculated as shown in Figure 4.7. An LI around 1 indicates the soil is near normally consolidated, whereas, a LI of zero is an indication of some overconsolidation (Bowles, 1996). It is noted that some field moisture contents are less than the plastic limit, indicating desiccation in the upper soils. Also, most of the field moisture contents are near the plastic limits. Most of the LI data lies between -0.2 and 0.5.

![Figure 4.6 Atterberg limits and moisture content data (WTI, 2002a)](image-url)
Figure 4.7 Site Liquidity Index data (WTI, 2002a)

Plasticity data relative to the Casagrande “A-line” is shown in Figure 4.8, with the corresponding Unified Soil Classification System (USCS) symbols. The case study site data is further plotted (as darkened circles) on the standard Casagrande plasticity chart as depicted in Figure 4.9. From this chart, most of the clay soils at the site have plasticity characteristics similar to glacial clays. Some clays at the site are classified as highly plastic (CH) type materials and these occur primarily in the 50 to 70 foot depth range (elevation 2000-2020 feet).
Figure 4.8  Subject site plasticity data (WTI, 2002a)

Figure 4.9  Casagrande plasticity chart (modified after Terzaghi et al. 1996)
The plasticity data may be correlated with the preconsolidation pressure and sensitivity. Some useful correlations with the plasticity data are shown in Figures 4.10 and 4.11. Based on the relationships shown in these figures, it could be inferred that the site soils have an average sensitivity of about 3 to 5 and a preconsolidation pressure between 4 and 40 ksf.

Figure 4.10  Preconsolidation pressure vs. LI (from U.S Dept. of the Navy, 1982)
Figure 4.11 Liquidity index vs. sensitivity (from Terzaghi et al. 1996)

Figure 4.12 shows a relationship between the drained friction angle ($\phi'$) and PI. Since most of the PI data for the low plasticity clay ranges between 10 and 30, the trend line would indicate a $\phi'$ of 28 to 32 degrees.

Figure 4.12 Relationship between $\phi'$ and PI (modified after Terzaghi et al. 1996)
4.3.3 Gradation Tests

Figure 4.13 shows the results of the gradation testing as the percent sand, gravel and fines (clay and silt) with depth. Most of the samples have fines contents (percentage passing the No. 200 sieve) between 10 and 50 percent, with measurable amounts of sand and gravel. The majority of all samples tested had some gravel content which decreases with depth. It is typical for Las Vegas Strip soils to have sufficient fines to behave as a clayey soil (such that they are plastic), but they may contain more than 50 percent sand/gravel. Few samples have over 50 percent fines and may be classified as a clay or silt. The common classification of the low plasticity soils with numerous fines is sandy clay.

4.3.4 Direct Shear Tests

Direct shear test data (ASTM D3080) for the subject site was obtained from the Kleinfelder (KI, 2001) geotechnical exploration report. Samples tested were obtained from ring samples. The rings from the split spoon sampler fit directly into the direct shear device after minor trimming.

The results indicating the drained friction angle (φ’) and effective cohesion (c’) with depth are shown in Figure 4.14. The average φ’ value is in the 25° to 30° range and the cohesion ranges from 0 to 255 psf.
Figure 4.13  Gradation test results with depth (WTI, 2002a)

Figure 4.14  Direct shear test data (KI, 2001)
4.3.5 Unconfined Compressive Strength - Caliche

During case study site exploration, cemented materials were identified by drilling techniques. Material hardness was logged, and the hardness values range between moderately hard and very hard, depending upon Standard Penetration Test (SPT) values and drill rod pressure. The materials are generally identified in the field as caliche or a matrix of cemented sand and gravel. When required for classification or laboratory testing, the cemented soils are sampled by rock coring techniques. Cemented deposits can be classified for quality using standard rock quality determination (RQD) techniques from rock mechanics.

The caliche deposits at the site were cored to retrieve samples for laboratory testing. Testing of caliche core samples consisted of unconfined compressive strength (UCS) testing (ASTM D2938). The coring and testing was performed by Kleinfelder during the initial site exploration in 2001. Percent recovery on the core runs ranged between 50 and 90 percent, but no rock quality designation (RQD) data was reported. The caliche data from the laboratory testing is shown below in Table 4.1. It is of interest to note that the upper caliche formation at the site was composed of an upper conglomerate type material, while the lower portion consisted of a fine-grained rock like material similar to limestone. UCS values indicate the lower material has approximately twice the compressive strength compared to the upper conglomerate caliche. A picture of the two phase upper material is shown in Figure 4.15.
Table 4.1 Caliche core UCS test data, case study site (KI, 2001)

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth, ft</th>
<th>% Recovery</th>
<th>UCS, ksi</th>
<th>UCS, MPA</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-7</td>
<td>17</td>
<td>80</td>
<td>0.98</td>
<td>6.7</td>
</tr>
<tr>
<td>B-7</td>
<td>20.7</td>
<td>80</td>
<td>14.56</td>
<td>100.3</td>
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<tr>
<td>B-7</td>
<td>21</td>
<td>80</td>
<td>15.16</td>
<td>104.4</td>
</tr>
<tr>
<td>B-13</td>
<td>14</td>
<td>90</td>
<td>3.95</td>
<td>27.2</td>
</tr>
<tr>
<td>B-13</td>
<td>16</td>
<td>90</td>
<td>11.42</td>
<td>78.6</td>
</tr>
<tr>
<td>B-13</td>
<td>17</td>
<td>90</td>
<td>2.67</td>
<td>18.4</td>
</tr>
<tr>
<td>B-14</td>
<td>13</td>
<td>85</td>
<td>8.45</td>
<td>58.2</td>
</tr>
</tbody>
</table>

Average: 8.2 56.3
Std. Dev. 5.8 39.8

Figure 4.15 Photograph of composite upper caliche (by author)

4.4 Field Data

During the site exploration, evaluation and design phases, various field testing was performed which consisted of additional soil borings, pressuremeter testing, five pile load tests and construction of an earthen embankment test fill of which deflections were monitored. Soil boring data was previously discussed.
4.4.1 Soil Borings

Penetration testing during the soil borings consisted of both standard SPT and modified California split-spoon (CASS) sampling. Ground water was encountered in the recent explorations at an elevation of about 2055 feet. A soil profile showing select boring data from the tower area is shown in Figure 4.1. Results of penetration sampling for select borings are shown in Figure 4.16. The data indicates the soils in the upper 50 feet at the site are dense or stiff and have lower SPT blow counts at depth. High blow counts can be indicative of isolated cemented zones.

Figure 4.16 Penetration testing data (WTI, 2002a & KI, 2001)
4.4.2 Pressuremeter Data

Pressuremeter testing (ASTM D4719) was performed during the Kleinfelder and Western Technologies explorations. The initial testing was conducted by Insitu Tech, Inc. at Boring B-12. The pressuremeter device was inserted and pushed into a pre-bored hole. The results of this testing are shown below in Figure 4.17. The reload modulus (Er or E+) is commonly used in elastic settlement analyses (ASCE, 1994).

<table>
<thead>
<tr>
<th>Test # &amp; Quality</th>
<th>Depth from QL ft</th>
<th>Soil Type</th>
<th>Net Pressure Limit</th>
<th>Initial Modulus</th>
<th>Reload Modulus</th>
<th>Er/E+ Ratio</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 11</td>
<td>28.5</td>
<td>Gravelly Silty Sand and Sands</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>ABORTED Hole Oversize</td>
</tr>
<tr>
<td>Test 13</td>
<td>32.5</td>
<td>Caliche</td>
<td>563</td>
<td>620</td>
<td>13.0</td>
<td>Approx. Su of 1.7 ksf</td>
<td></td>
</tr>
<tr>
<td>Test 14</td>
<td>41.5</td>
<td>Caliche</td>
<td>&gt;6000</td>
<td>-----</td>
<td>-----</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Test 15</td>
<td>57.0</td>
<td>Clayey Sandy Silt</td>
<td>12.7</td>
<td>165</td>
<td>620</td>
<td>13.0</td>
<td>Approx. Su of 2.1 ksf</td>
</tr>
<tr>
<td>Test 16</td>
<td>67.6</td>
<td>Silts</td>
<td>17.6</td>
<td>311</td>
<td>392</td>
<td>17.7</td>
<td>Approx. Su of 2.3 ksf</td>
</tr>
<tr>
<td>Test 17</td>
<td>78.7</td>
<td>Silts</td>
<td>25.3</td>
<td>535</td>
<td>1431</td>
<td>21.1</td>
<td>Approx. Su of 2.8 ksf</td>
</tr>
<tr>
<td>Test 18</td>
<td>86.5</td>
<td>Silts</td>
<td>20.4</td>
<td>654</td>
<td>1127</td>
<td>22.1</td>
<td>Approx. Su of 2.4 ksf</td>
</tr>
<tr>
<td>Test 19</td>
<td>101.5</td>
<td>Clayey Silt</td>
<td>33.0</td>
<td>828</td>
<td>1650</td>
<td>25.1</td>
<td>Approx. Su of 3.5 ksf</td>
</tr>
<tr>
<td>Test 20</td>
<td>131.2</td>
<td>Hard Silts</td>
<td>48.2</td>
<td>1175</td>
<td>2354</td>
<td>24.4</td>
<td>Approx. Su of 4.8 ksf</td>
</tr>
</tbody>
</table>

Figure 4.17 Insitu Tech, Inc. pressuremeter results (KI, 2001)

Pressuremeter testing during the WT exploration was performed by STS Consultants, Inc., in Borings P13 & P14. Results are shown in Figure 4.18.

A plot of both sets of pressuremeter data is shown in Figure 4.19. The high values in the data are due to isolated cemented (caliche) zones. The trend of the data between 60 and 100 feet indicate an increase in modulus with depth of about 10 to 20 ksf.
per foot. The undrained shear strength \((s_u)\) is obtained by correlation with the net limit pressure (Briaud, 1992), and generally increases with depth (at a rate of about 0.04 ksf per foot) as shown in Figure 4.20.

![Figure 4.18 STS pressuremeter results (WTI, 2002a)](image)

<table>
<thead>
<tr>
<th>BORING NUMBER</th>
<th>DEPTH (ft)</th>
<th>(P_d) (ksf)</th>
<th>(P_r) (ksf)</th>
<th>(P_i) (ksf)</th>
<th>(E_d) (kPa)</th>
<th>(E_r) (kPa)</th>
<th>(E_d/E_r)</th>
<th>(E_r/P_i)</th>
<th>(P_i/P_d)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-13</td>
<td>46.3-51.0</td>
<td>5.0</td>
<td>72.0</td>
<td>36.6</td>
<td>154</td>
<td>444</td>
<td>0.38</td>
<td>4.2</td>
<td>3.1</td>
</tr>
<tr>
<td></td>
<td>53.5-58.0</td>
<td>3.5</td>
<td>6.0</td>
<td>10.2</td>
<td>56</td>
<td>171</td>
<td>0.33</td>
<td>5.5</td>
<td>1.7</td>
</tr>
<tr>
<td></td>
<td>58.5-61.0</td>
<td>5.0</td>
<td>9.5</td>
<td>17.4</td>
<td>136</td>
<td>394</td>
<td>0.45</td>
<td>7.8</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
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<td>5.0</td>
<td>8.5</td>
<td>16.5</td>
<td>128</td>
<td>275</td>
<td>0.47</td>
<td>7.8</td>
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<tr>
<td></td>
<td>73.5-76.0</td>
<td>4.5</td>
<td>8.5</td>
<td>14.2</td>
<td>137</td>
<td>350</td>
<td>0.39</td>
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<td>6.0</td>
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<td>7.0</td>
<td>3</td>
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<td>360</td>
<td>0.29</td>
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<td>-</td>
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<td>58.5-61.0</td>
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<td>5.0</td>
<td>8.9</td>
<td>95</td>
<td>158</td>
<td>0.47</td>
<td>10.4</td>
<td>1.8</td>
</tr>
<tr>
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<td>6.5</td>
<td>11.3</td>
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<td>0.54</td>
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<td>1.7</td>
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<td>12.0</td>
<td>26.5</td>
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<td>0.35</td>
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<td>21.5</td>
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<td>544</td>
<td>0.35</td>
<td>8.9</td>
<td>1.6</td>
</tr>
</tbody>
</table>

\(^1\) Pressurized the test area to 29 ksf and did not reach the Pf.
\(^2\) Pressurized the test area to 41.2 ksf and did not reach the Pf.
\(^3\) Pressurized the test area to 39.2 ksf and did not reach the Pf.
\(^4\) Burst membrane pressurizing to 10.5 bars, hard and weaker layers in the test area.
\(^5\) Pressurized the test area to 29.5 ksf and did not reach the Pf.
\(^6\) Burst membrane pressurizing to 20 bars, hard and weaker layers in the test area.
\(^7\) Large test hole, gravel or rock in the test area.

Figure 4.18 STS pressuremeter results (WTI, 2002a)
Figure 4.19  Pressuremeter modulus with depth (WTI, 2002a)

Figure 4.20  Undrained shear strength based on pressuremeter data (WTI, 2002a)
As previously stated, the liquid limit and plasticity index data indicate that a compressible zone exists between 50 and 70 feet below the previously existing grade (elevation 2000 to 2020). To show the relationship between high liquid limit and moisture content data and the pressuremeter modulus data, Figure 4.21 includes the pressuremeter liquid limit and field moisture data (plotted inverse to pressuremeter data). Well-known correlations exist between soil compressibility parameters and liquid limit and/or moisture content (e.g., Holtz and Kovacs, 1981). The high liquid limit and moisture content data occur where the pressuremeter modulus is low relative to the other values. Low density data is also noted in the 50 to 70 foot depth zone from Figure 4.4. It is observed that the high liquid limit and moisture content data can be used to identify the low modulus or more compressible zones at the site. It is important to note that the soft zones were not identifiable using SPT data as most N-values in the 50 to 70 foot depth range were in the 30 to 100 blows per foot range.

4.4.3 Pile Load Tests

Five pile load tests were performed at the case study site during the design stage of the project. Two tests utilized the Osterberg load cell method (KI, 2001). The remaining tests were performed with the conventional beam and reaction pile arrangement (WTI, 2002b). One Osterberg type and three conventional pile load tests will be discussed below.
The Osterberg test data was analyzed by Loadtest, Inc., while the top-down load test data was reduced by the author. To determine the load distribution with depth, each test pile was instrumented with vibrating wire strain gages mounted on steel rebar (sister bars, Geokon Model No. 4911). For the Osterberg load test, the load in the pile section at each depth was determined as follows:

Following elastic theory,

\[ P = A E \varepsilon \]  \hspace{1cm} (4-1)

where \( P \) = load

\( \varepsilon \) = measured strain from gage
\( A \) = area of pile section at strain gage plane

\( E \) = composite Young’s modulus considering areas of steel and concrete in the pile section.

The concrete modulus was estimated from laboratory unconfined compressive strength tests performed on the day of the load test. From ACI 318-02 (ACI, 2002) for normal weight concrete (Section 8.5):

\[
E_c = 57000 \sqrt{f'_c} \text{ (psi)} \quad (4-2)
\]

The composite section modulus was determined from Equation 4-3:

\[
E = \frac{A_s E_s + A_c E_c}{A} \quad (4-3)
\]

where:

\( A_s \) = steel area

\( A_c \) = concrete area

\( E_s \) = steel modulus (29,000,000 psi)

For the conventional top-down test method, the pile elastic modulus was considered to vary with strain level, based on the tangent modulus method (Fellenius, 2001). This method is briefly discussed below.

To determine the load distribution with depth, gage data is recorded at each load increment, relative to the value at zero load. Loads in the pile section at each depth were determined as follows.
From elastic theory,

\[ P = A E(\varepsilon)\varepsilon \]  \hspace{1cm} (4-4)

where \( P \) = load,

\( \varepsilon \) = measured strain from gage

\( A \) = area of pile section at strain gage plane

\( E(\varepsilon) \) = Young’s modulus of the pile based on strain level

The steel modulus is well known, but the concrete modulus varies with load. Following the procedure by Fellenius, the strain gage data was first converted to micro strain, and plotted vs. the load. As increments of load are applied to the pile, the tangent modulus may be evaluated as:

\[ M_t = \frac{d\sigma}{d\varepsilon} = A\varepsilon + B \]  \hspace{1cm} (4-5)

where \( M_t \) is the tangent modulus and A and B are constants to be determined. This may be further written as:

\[ E_s = \frac{1}{2} A\varepsilon + B \]  \hspace{1cm} (4-6)

where \( E_s \) = secant modulus.

A typical plot of the strain gage vs. load data is shown below in Figure 4.22. The constants A and B are determined by drawing a best-fit line through the data which converges at high micro strain levels. As observed by the above equations, A is the slope and B is the y-intercept of the line.
The data that does not converge to the line indicates that sufficient yield in the soil has not yet occurred, as indicated by the gage(s) at that depth(s). This is due to shaft resistance effects which reduce the strain at that depth, resulting in high tangent modulus values. Once the soil reaches its ultimate friction value, the strain data changes only due to the applied top load and the calculated tangent modulus is representative of the pile itself. Using the linear approximation, an equation for the secant modulus is determined which allows calculation of the pile modulus at each strain level during the test. The pile load at each gage location may then be calculated using elastic theory.

The accuracy of strain gage data is often questioned due to many factors such as gage orientation in the pile, concrete modulus changes with strain level, residual load...
effects and assumed section shape in un-callipered bored piles (Hayes and Simmonds, 2002).

In evaluating the pile load distribution data, the effect of residual load in the pile was not considered because strain gage data during concrete curing was not available. In fact, in geotechnical practice, it is rarely considered or even mentioned. The construction process can cause residual strains and locked-in loads in the pile that are “zeroed out” when the load test begins (load test personnel conduct strain gage reading assuming zero load at the start of a test). According to Fellenius (2002), residual load from side shear developed during and after (due to reconsolidation) construction and concrete curing in drilled shafts can result in over-estimated loads down the pile shaft. For bored piles, this results in overestimating the load transfer with depth, or more load is taken in end-bearing than the strain gage data may imply. Locked in strains from concrete curing have been evaluated by Hayes and Simmonds (2002) which indicated tension followed by compressive shear stresses developed during the concrete curing period. It is not known what effect this would have on bored piles socketed into caliche. However, it is assumed that the calculated load distribution is “ball-park” correct.

The unit shear (load) transfer displacement data was calculated using the theory presented in Aurora and Reese (1976) and Vesic (1977), as follows. Figure 4.23 shows a single pile subjected to a axial load, $Q_o$, with displacement at the top, $w_o$, and load distribution, $Q_z$ from strain gage measurements. The load transfer distribution is $f_o(z)$, and it is desired to determine $f_o(w_z)$, where $w_z$ is the displacement of the pile section at depth $z$. 

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From elastic theory, the elastic compression is:

\[ \delta = \frac{QD}{AE} \]  

(4-7)

where AE = pile axial stiffness,

Q = load,

D = pile length

Consider an incremental section at depth z; the elastic compression of the section will be:

\[ dw_z = \frac{Q_z}{AE} \, dz \]  

(4-8)

or

\[ \frac{dw_z}{dz} = \frac{Q_z}{AE} \]  

(4-9)
Also, if the load is decreasing with depth, as is the case with a typical pile in soil, then

\[ dQ_z = -f_o Cdz \quad (4-10) \]

or

\[ f_o(z) = -\frac{1}{C} \frac{dQ_z}{dz} \quad (4-11) \]

where for circular piles, \( C = \pi D \) (pile circumference). This represents the load transferred to the soils at depth \( z \), and is the slope of the load distribution curve divided by the pile circumference. Note that

\[ w_o = w_z + \int_0^z \frac{Q_z}{AE} dz \quad (4-12) \]

and

\[ w_z = w_o - \int_0^z \frac{Q_z}{AE} dz \quad (4-13) \]

Based on the above equations, one can now plot \( f_o(z) \) vs. \( w_z \), as shown in the load transfer curves for the load test data in this chapter. Each soil has its unique load transfer curve, which, like a stress-strain curve, provides information on the amount of sidewall deflection to reach a failure in friction. The data can be used in t-z analyses of piles (Coyle and Reese, 1966).

4.4.3.1 Osterberg Load Tests

The Osterberg load test method (e.g., Osterberg, 1989) utilizes a single drilled shaft (pile) and a hydraulic jack placed at a pre-determined depth within the shaft, thereby, jacking the top portion of the pile upward and the remaining portion below the O-cell downward. The Osterberg load tests were located near the center of the subject
foundation tower plan. Data from Osterberg load Test 2 is not included because the results of that test were similar to Test 1. The test characteristics for Load Test 1 are indicated in Figures A.1 and A.2 in Appendix A. Load deflection, load distribution and load transfer curves are shown in Figures A.3 to and A.5.

The test data indicate the upper test section deflected about 0.3 inches and the lower test pile section moved downward about 0.1 inches under a maximum load of 5,500 kips each way. The upper 43 foot test shaft was reasonably bonded to the upper caliche layer(s) given the low upward maximum and net deflection. The maximum test load produced an average bond stress of 10 ksf for the upper test shaft.

Based on the load distribution data, Table 4.4 summarizes the test unit side shear developed at maximum load for the Osterberg load test. This data does not represent ultimate values, as indicated in Figure A.5.

Table 4.2 O-Cell load test side shear

<table>
<thead>
<tr>
<th>Load Transfer Zone (elevation, ft.)</th>
<th>Maximum Side Shear (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2070 to 2050</td>
<td>6.5</td>
</tr>
<tr>
<td>2050 to 2027 (O-cell)</td>
<td>13.3</td>
</tr>
<tr>
<td>2026 (O-cell) to 2010</td>
<td>18.9</td>
</tr>
<tr>
<td>2010 to 1995</td>
<td>3.4</td>
</tr>
<tr>
<td>1995 to 1977</td>
<td>2.3</td>
</tr>
<tr>
<td>1977 to 1961</td>
<td>0.9</td>
</tr>
</tbody>
</table>

4.4.3.2 Conventional Load Tests

The conventional top down load test program at the site consisted of three axial tests and one lateral load test (not discussed in this dissertation). The purpose of the test program was to determine ultimate failure parameters for the upper caliche deposit, the
soil zone immediately below the upper caliche deposit, and the load distribution and settlement of a full scale pile at the design load of 1,500 tons. All piles were fully instrumented with strain gages to aid in determining load distribution within the piles. The depth, diameter and test goal of each pile are shown in Table 4.3.

Table 4.3 Conventional Load Test Details (WTI, 2002b)

<table>
<thead>
<tr>
<th>Test Pile No.</th>
<th>Diameter (inches)</th>
<th>Installed Depth (feet)</th>
<th>Test Goal</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-1</td>
<td>39</td>
<td>55</td>
<td>Load Distribution</td>
</tr>
<tr>
<td>TP-2</td>
<td>24</td>
<td>37</td>
<td>Geotechnical Failure - Soil</td>
</tr>
<tr>
<td>TP-3</td>
<td>24</td>
<td>23</td>
<td>Geotechnical Failure - Caliche</td>
</tr>
</tbody>
</table>

The test pile site was located east of the tower perimeter at column line 15. The dedicated test piles were installed by Morris-Shea Bridge Company, Inc. on April 12 and 13, 2002. Reaction piles for all test piles were installed before the test piles. A 60 foot deep boring was performed in the load test area to determine the subsurface conditions. The deep boring was performed at the location of test pile TP-1. Based on the boring data, the soil profile at the test pile location consisted of 6.5 feet of soil fill above a 12.5-foot thick caliche deposit, underlain by uncemented soils. The upper caliche deposit included a 2 foot thick soil layer from 14 to 16 feet below grade. A second layer of caliche was encountered at a depth of about 40 feet below grade, which was 7.5 feet in thickness. The water level at the time of the boring was recorded at a depth of 19 feet. The upper 2.5 feet of the cemented deposit is logged as a cemented sand and gravel material which usually has a lower strength than the caliche.
Each test pile was instrumented with vibrating wire strain gages mounted on steel rebar (sister bars, Geokon Model No. 4911-4) which were attached to the rebar cage for the pile to provide information on load distribution with depth. The strain gages were placed at the following depths below existing grade, elevation 2069.

<table>
<thead>
<tr>
<th>Test Pile No.</th>
<th>Strain Gage Location (depth, ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-1</td>
<td>6.8, 12.8, 19.8, 24.6, 29.8, 34.7, 39.8, 43.7, 47.7, 53.7(2)</td>
</tr>
<tr>
<td>TP-2</td>
<td>20.2, 24.5, 29.5, 33.5</td>
</tr>
<tr>
<td>TP-3</td>
<td>7.0, 13.0, 18.9, 20.2</td>
</tr>
</tbody>
</table>

The piles were completed by pumping cement-sand/pea gravel grout through the hollow stem auger immediately following the drilling process. The grout was supplied by Nevada Ready Mix, mix number 6043, with a 28 day design strength of 9,000 psi. A steel rebar cage (10#10 bars) was placed in test piles TP-2 and TP-3. A cage consisting of 12 #10 bars and a single bar with strain gages was placed in the center of the test pile TP-1. A pile cap (4 foot square, 4 feet thick) was constructed on each test pile to aid in transferring load from the jacks to the test pile. Thin sheet metal casing was installed through the overburden soils above the caliche at TP-1 and TP-3, and the casing was installed through the overburden and upper caliche at TP-2. The actual depths of the casings for TP-1, TP-2, and TP-3 were approximately 5, 13 and 7 feet, respectively.

Eight reaction piles (24 inch diameter) were installed around each test pile. The reaction piles were approximately 32 feet in depth and reinforced with rebar cages. Two 1⅜-inch diameter high strength thread bars were placed in each reaction pile.
A load frame was situated over the test pile setup and the steel bars in the reaction piles were attached to the steel load frame. A steel plate was placed on the test pile cap. Two 1,000 ton hydraulic jacks were placed on the steel plate. The jacks were calibrated by Beerman Precision, Inc.

Following completion of the load frame set up, dial gauges were placed on the test pile to record defections during the test. The dial gages had a precision of 0.001 inch. Two gages were placed on the test pile. The gages were attached to steel reference beams installed across the top of the test pile and reaction piles. As applicable, a sunscreen and/or windscreen were placed above the reference beams to minimize disturbance. To establish test pile deflection from a reference point beyond the load test area, each pile test was monitored by reading deflections from a surveyor level. A small metal scale with a sensitivity of 0.01 inch was attached to the side of one jack for reference.

The tests were performed on the dates shown in the following table. Tests were performed in general accordance with ASTM D1143, quick test method. Based on laboratory test data, the grout strength on the day of the tests is indicated in Table 4.5.

### Table 4.5 Test pile concrete strengths (WTI, 2002b)

<table>
<thead>
<tr>
<th>Test Pile No.</th>
<th>Date Tested (2002)</th>
<th>Estimated Concrete Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-1</td>
<td>May 9 to 13 (hold); May 20 (lateral)</td>
<td>7,650; 8,000</td>
</tr>
<tr>
<td>TP-2</td>
<td>May 2; May 4; May 7</td>
<td>8,200; 8,500; 8,700</td>
</tr>
<tr>
<td>TP-3</td>
<td>April 26; April 27</td>
<td>8,400</td>
</tr>
</tbody>
</table>

The piles were loaded in typical increments of approximately 50 to 100 tons. During each load increment, data from the strain gages, reference level, and dial gages at
the surface was recorded. The piles were typically loaded to a maximum load of nearly 2,000 tons.

Test piles TP-2 and TP-3 were tested more than once for various reasons. The initial test for TP-3 was loaded with one jack to 1,000 tons due to equipment problems. The second test for TP-3 included both 1,000 ton jacks.

Each test pile was loaded to the maximum load of the system of approximately 2,000 tons, or to a failure load. The test results are summarized in the Table 4.6.

Table 4.6 Load test results (WTI, 2002b)

<table>
<thead>
<tr>
<th>Test Pile</th>
<th>Maximum Load (tons)</th>
<th>Maximum Deflection (inches)</th>
<th>Avg. Unit Friction (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-1</td>
<td>2,045</td>
<td>0.6</td>
<td>7.2</td>
</tr>
<tr>
<td>TP-2*</td>
<td>2,045</td>
<td>2.3</td>
<td>5.0**</td>
</tr>
<tr>
<td>TP-3</td>
<td>1,813</td>
<td>0.2</td>
<td>25.1</td>
</tr>
</tbody>
</table>

* Failure occurred
** For clay soil beneath upper caliche

Test pile TP-1 was held at the design load of approximately 1,500 tons for about 3 ½ days to evaluate creep. During that time, a creep of 0.14 inches was recorded. The deflection at the working load of about 1,500 tons was about 0.5 inches including the creep deflection. The load distribution within the upper caliche layer was less than realized in tests TP-2 and TP-3 which were smaller diameter piles. Since the auger was raised out of the hole for cleaning prior to grouting the pile, the sides of the hole in the upper caliche deposit were likely “smeared” with cohesive soils, thereby reducing the side friction. The drilling process was observed by representatives of Western Technologies and no noticeable difference between test pile TP-1 and the other piles was
noted. Additionally, a more typical value for load transfer in the lower caliche deposit was realized, which was not subjected to smearing. Therefore, it could be concluded that the withdrawal of the auger must have smeared the sidewalls of the upper caliche formation. Additionally, higher working friction values were obtained in the 24 inch diameter test piles which were not subjected to smearing. Hassan and O’Neill (1997) have shown that smearing in argillaceous intermediate geo-materials (IGM) during drilled shaft construction has a significant effect on reducing load transfer and ultimate side friction. Load deflection, load distribution with depth, load transfer, and unit shear transfer data are shown in Figures 4.24 to 4.27.

For test pile TP-2, it was intended to produce a failure in side friction within the uncemented soils below the upper caliche deposit. Although the upper soils and caliche were partially cased, a sufficient load to produce failure on the test section below the caliche was not obtained during initial testing.
Figure 4.24 Test Pile TP-1 load-displacement data (WTI, 2002b)

Figure 4.25 Test Pile TP-1 load distribution data (WTI, 2002b)
Figure 4.26  Test Pile TP-1 load transfer data (WTI, 2002b)

Figure 4.27  Test Pile TP-1 shear displacement data (WTI, 2002b)
It appeared that less than 10 percent of the applied top load was actually being applied to the test section due to friction in the upper soils and caliche. The caliche deposit area around the test pile was then fractured via air drilling closely spaced holes to reduce the frictional capability of the layer. The third test on TP-2 was successful in producing failure of the soil zone.

Following the air drilling process to isolate the pile from the upper caliche, test pile TP-2 exhibited a geotechnical failure in friction of the soil below the upper caliche deposit. The peak unit shear in this material was on the order of 5 ksf. An ultimate load transfer value of 25 ksf was obtained in the upper caliche zone following fracturing by pre-drilling, load deflection, load distribution with depth, load transfer, and unit shear-transfer data are shown in Figures 4.28 to 4.31.

Figure 4.28  Test Pile TP-2 load displacement data (WTI, 2002b)
Figure 4.29 Test Pile TP-2 load distribution data (WTI, 2002b)

Figure 4.30 Test Pile TP-2 load distribution data (WTI, 2002b)
Load test TP-3 was loaded to near the system capacity of 2,000 tons and exhibited only ¼-inch of top movement. The applied pile test load was distributed in the upper caliche, with the upper half of the deposit taking about 85 percent of the load. The average unit shear value in the caliche deposit was nearly 70 ksf in the upper half and about 15 ksf in the lower half. The shear displacement data indicate that failure (or yielding) of the caliche was not observed at a peak shear of 70 ksf, thus, the pile was very well bonded to the upper caliche. The partially cased upper soil/caliche material exhibited an ultimate shear of about 10 ksf. Load deflection, load distribution with depth, load transfer and unit shear transfer data are shown in Figures 4.32 to 4.35.

In Figure 4.32, the plot of the elastic line assumes no end bearing contribution and uses a value of $\alpha$ of 0.55 which corresponds approximately to a constant friction
distribution (Vesic, 1977). The $\alpha$ term reduces the load used to compute the elastic deflection by reducing the skin friction component of the total pile load. Unlike a column, the load in the pile reduces with depth due to the skin friction component.

The load displacement data relative to the elastic line indicates the pile response is elastic for applied loads below approximately 1,500 tons. For loads above 1,500 tons the pile response indicates a slight nonlinear behavior.

![Figure 4.32 Test Pile TP-3 load displacement data (WTI, 2002b) (Image: 67)]
Figure 4.33 Test Pile TP-3 Load distribution data (WTI, 2002b)

Figure 4.34 Test Pile TP-3 load transfer data (WTI, 2002b)
4.4.3.3 Summary – Load Transfer Curves

The three conventional load tests revealed shear data on shear transfer with deflection for both cemented materials and soil at the site. Summary plots are shown in Figures 4.36 and 4.37.

From the load transfer vs. displacement, it can be concluded that the soil shear generally peaks at a displacement of 0.2 to 0.4 inches, while the cemented material shear peaks at 0.1 inch or less.

Note that the two cemented material curves that peak in the 0.3 to 0.4 inch range were probably due to smearing in TP-1, while the TP-2 strain gage was located at a cemented/soil interface. Data from test pile TP-3 at a depth of 7 feet was from a strain
gage located at the bottom of the sheet metal casing that was installed to minimize load shedding above the upper caliche layer.

Photographs taken by the author during the test pile installation and load test program are shown in Figures 4.38 to 4.41.

4.4.4 Test Fill Embankment

A test fill embankment was constructed at the site in 2003 to aid in determining stiffness parameters for use in evaluating settlements of structures. The embankment was approximately 200-foot square (at the top) and 30 feet in height, constructed at the southwest corner of the site. Settlement plates were placed in the embankment and monitored during fill placement. The settlement plates consisted of a 2-foot square steel plate at the base, with steel pipe within a PVC pipe which extended up through the fill. The plate base was buried 1-foot below the existing ground surface.

Fill materials consisted of on-site silty sand materials. Numerous field density and moisture content tests were taken at each 5 foot lift during placement. The average soil moist density of the fill was 122 pounds per cubic foot. Photographs of the test fill are shown in Figures 4.42 and 4.43. The edges sloped at about 1:1 (H:V). Settlement data was obtained at the center and top edges, and at various points beyond the embankment. The data was recorded during and after fill completion. Figure 4.44 shows the settlement data. The average maximum settlement of the fill was 1.8 inches which was measured at two adjacent center points.
Figure 4.36 Summary of shear displacement data for soils

Figure 4.37 Summary of shear displacement data for cemented materials
Figure 4.38 Test pile construction (photographed by author)

Figure 4.39 Test pile TP-3 pile cap form (photographed by author)
Figure 4.40  Test frame for test pile TP-3 (photographed by author)

Figure 4.41  Two 1,000 ton jacks for test pile TP-3 (photographed by author)
Measurements at the top edges of the fill indicated about 1 inch of settlement, while measurements beyond fill indicated that some tilting toward the south and west occurred. The non-uniform displacement of the fill may be due to varying caliche
thicknesses in this area of the site. The settlement occurred as the fill was placed and the post-fill settlement was only about 0.2 inches, indicating the compression of the soils was essentially drained vs. undrained behavior.

Figure 4.44 Test fill settlement data (WTI, 2003c)

4.5 Building Settlement Data

Of the numerous high rise buildings in Las Vegas, only a limited number of projects have been monitored for settlement. Some buildings have been generally checked for settlement amounts at some point after completion, usually by elevation data of a finish floor relative to a site datum. On one of the author's projects, an elevation difference between the tower and adjacent connecting low-rise had been built to compensate for the tower settlement. The resulting difference in floor slabs between the structures then provides a rough settlement estimate. For most projects, there is no data available; only the conclusion that there were no (or minor) apparent settlement related
problems after the project completion. Monitoring on one 40+ story building was started late in the construction process when unusual settlement related movements were noticed.

Before the construction of the case study building, Turnberry Place Towers buildings were the only high rise buildings that were monitored for settlements during construction by regular surveys. The settlement occurred regularly with building height and the settlement stabilized about two to three months after top out.

4.5.1 Case Study Hotel Tower

The data collected for the settlement of the case study project was the most complete and detailed for a high rise building in Las Vegas at the time. Details of the project have been discussed in Chapter 2. There were approximately 88 settlement points established in the building shortly after the first floor columns were constructed (22 column lines with 4 columns per line). Movements were monitored by evaluating the elevation of each pin on a monthly basis. The benchmark for the surveying was located at the corner of Las Vegas Boulevard and Desert Inn Road. Data was recorded by an independent local surveying company. The initial benchmark data was recorded on February 26, 2003, and the first data set was recorded on March 24, 2003. Monthly monitoring began on May 5, 2003 and continued until December 31, 2004.

The data may be presented as the average settlement for each column line along the length of the tower footprint which includes the average data from two interior and two exterior columns, as shown in Figure 4.45. Some data points for the northern third of the tower were permanently not accessible at about 4 months after top out. Similar data for column line D is shown in Figure 4.46. The pattern of settlement along the building is indicative of the heavy loaded elevator cores at the north and south portions of the
building. The approximate locations of the cores are shown in Figure 4.45.

Whereas the settlement data in Figure 4.45 is the average of 4 columns per line, in Figure 4.46 the data is only for column line D, along the length of the building. This column line is one of the two heavily loaded interior columns. The relatively large recorded movement between 45 and 49 floors may be data related since there was little settlement observed for the following period. For a particular column line, the data may also be plotted with time, as shown in Figure 4.47 for column line D. Column line 2D is at the north end of the building, and line 20D is at the south end where the upper caliche is thinner (see Figure 4.3). As indicated on the plot, the building top out occurred at a time of 415 days. The settlement appeared to stabilize at about 3 months after top out.

Figure 4.45  Average column line settlement data
4.6 Additional Data From Other Las Vegas sites

4.6.1 General

Since the database of undisturbed soil/caliche laboratory and in-situ test data in Las Vegas is relatively limited, it was of value to document some additional soil/caliche test data from other sites. Soil tests and caliche core data from other sites in Las Vegas is discussed below. Tests performed on both soil and caliche at the Nevada Department of Transportation (NDOT) I15/U.S. 95 site consisted of triaxial testing and unconfined compressive strength. Additionally, in-situ dilatometer testing was performed at the NDOT site. At the Fremont Street Experience site in downtown Las Vegas, UCS, Indirect Tensile tests and Young’s modulus were performed on caliche samples.

![Figure 4.46 Settlement data vs. distance for column line D](image-url)

Figure 4.46 Settlement data vs. distance for column line D
Caliche cores were also obtained from the UNLV campus and subjected to density measurements and dynamic resonance tests in the laboratory.

4.6.2 NDOT US 95/I-15 Site

Kleinfelder (KI, 1996) performed consolidated undrained triaxial and unconfined compressive strength tests on soil and caliche samples. Unconfined compressive strength (UCS) tests were performed on soil samples from the site.

![Figure 4.47 Case study tower measured settlement data on D-line](image)

This represents the largest known test database for UCS tests on Las Vegas soils, as tube samples are difficult to obtain due to stiff soils, gravel content, and varying degrees of cementation. The UCS test results versus the dry density were plotted as shown in Figure 4.48. There is a general trend of increasing UCS with increasing soil dry
density. The tests indicate low values of UCS for a wide range of dry density values, so some samples may have been disturbed.

4.6.2.1 Caliche Core Testing

Tests on caliche core samples at this site consisted of UCS and triaxial testing, and unconfined compressive strength and triaxial tests on soil samples. The caliche core data is summarized in Table 4.7.

As seen in Table 4.7, the average UCS is 5.5 ksi and the standard deviation is 3.2 ksi. The standard deviation shown (or variation of 58.2%) is considered very high and may be attributed to natural variations in the material, weathering effects, sampling and testing procedures, and grain size or petrographic effects (Ruffolo and Shakoor, 2009). As shown in Figure 4.15, a caliche deposit may consist of distinctly different materials that are bonded together which will increase the variability of the test results. In addition, the cemented sand and gravel material contains large aggregates which also affects the

Figure 4.48 NDOT Site: UCS test data (KI, 1996)
Science Applications International Corporation (SAIC) performed triaxial tests on caliche cores from the NDOT site to determine values for the density, ultimate strength (US), Young’s modulus and Poisson’s ratio. The tests were performed at a single confining pressure of 14 psi. The test results are shown below in Table 4.8 and the test data is included in Appendix B. The ultimate strength (US) ranged between 7.9 and 10.7 ksi, and the elastic modulus ranged between 418,000 and 687,000 ksf. The average Poisson's ratio was 0.32 for the samples from Boring 5. The lowest elastic modulus value was from the sample with the highest strength, which is inconsistent with the other data in the table, and the fact that the stiffness generally increases with increasing rock or concrete strength (see Figure 4.49 for stress strain data).

4.6.2.2 Dilatometer Tests

As previously mentioned, very few in-situ tests have been performed in Las Vegas and include the pressuremeter, cone penetration (limited to uncemented zones), and dilatometer tests. Dilatometer tests were performed at the NDOT site. The tests were performed by Gregg In Situ, Inc. between March 20 and March 23, 1995, and data reduction was accomplished using the computer program DILLY4. As mentioned in the results report, cemented soils were encountered in the profile and the correlations used for various parameters were developed for uncemented soils. Therefore, for the reduced data, the classification of the soils based on the data are more coarse-grained than they actually are.

The reduced dilatometer test data is shown in Appendix C. It is of interest to note that this data represents the only known dilatometer data for Las Vegas soils and because
of the difficulty in obtaining quality samples, this likely represents some of the first information on in-situ lateral stress state and the overconsolidation ratio (OCR) for the Las Vegas Strip area.

Table 4.7 Caliche core test data, NDOT site (KI, 1996)

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth, ft</th>
<th>Elevation, ft.</th>
<th>UCS, ksi</th>
<th>UCS, MPA</th>
</tr>
</thead>
<tbody>
<tr>
<td>ES-7A</td>
<td>8</td>
<td>2016.2</td>
<td>5.23</td>
<td>36.1</td>
</tr>
<tr>
<td>ES-8B</td>
<td>16</td>
<td>2021.4</td>
<td>1.54</td>
<td>11.0</td>
</tr>
<tr>
<td>ES-15</td>
<td>19.5</td>
<td>2024.5</td>
<td>2.68</td>
<td>18.5</td>
</tr>
<tr>
<td>ES-15</td>
<td>20</td>
<td>2023.9</td>
<td>2.81</td>
<td>19.4</td>
</tr>
<tr>
<td>MLKS-6</td>
<td>14.1</td>
<td>2036.4</td>
<td>1.35</td>
<td>9.3</td>
</tr>
<tr>
<td>NMLK-9</td>
<td>25.7</td>
<td>2024.8</td>
<td>5.88</td>
<td>40.5</td>
</tr>
<tr>
<td>NMLK-13</td>
<td>40.4</td>
<td>2000.3</td>
<td>3.81</td>
<td>26.3</td>
</tr>
<tr>
<td>NMLK-13</td>
<td>41.6</td>
<td>1999.1</td>
<td>4.02</td>
<td>27.7</td>
</tr>
<tr>
<td>NW-2</td>
<td>10.6</td>
<td>2021.0</td>
<td>3.66</td>
<td>25.2</td>
</tr>
<tr>
<td>NW-2</td>
<td>11.1</td>
<td>2020.5</td>
<td>5.61</td>
<td>38.7</td>
</tr>
<tr>
<td>SE-2</td>
<td>20.1</td>
<td>2030.4</td>
<td>8.28</td>
<td>57.1</td>
</tr>
<tr>
<td>SE-2</td>
<td>20.8</td>
<td>2029.7</td>
<td>8.64</td>
<td>59.6</td>
</tr>
<tr>
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<tr>
<td>SE-7</td>
<td>21</td>
<td>2023.8</td>
<td>2.88</td>
<td>19.9</td>
</tr>
<tr>
<td>SE-9</td>
<td>30</td>
<td>2030.4</td>
<td>9.18</td>
<td>63.3</td>
</tr>
<tr>
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<td>30.5</td>
<td>2029.8</td>
<td>8.93</td>
<td>61.6</td>
</tr>
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<td>SE-10</td>
<td>44</td>
<td>2026.2</td>
<td>2.43</td>
<td>16.8</td>
</tr>
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<td>SWB-2</td>
<td>10.9</td>
<td>2029.8</td>
<td>12.03</td>
<td>82.9</td>
</tr>
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<td>SWB-2</td>
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<td>2029.1</td>
<td>12.26</td>
<td>84.5</td>
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<td>17.2</td>
<td>2033.3</td>
<td>4.40</td>
<td>30.3</td>
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<td>2.84</td>
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<td>73.6</td>
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<td>WB-1</td>
<td>15.5</td>
<td>2018.6</td>
<td>11.13</td>
<td>76.7</td>
</tr>
<tr>
<td>WRR-2</td>
<td>22.5</td>
<td>2014.9</td>
<td>4.92</td>
<td>33.9</td>
</tr>
<tr>
<td>WRR-2</td>
<td>23</td>
<td>2014.4</td>
<td>2.80</td>
<td>19.3</td>
</tr>
<tr>
<td>WRR-2</td>
<td>23.5</td>
<td>2013.9</td>
<td>4.66</td>
<td>32.1</td>
</tr>
<tr>
<td>WMS-1</td>
<td>16.5</td>
<td>2017.6</td>
<td>4.94</td>
<td>34.1</td>
</tr>
<tr>
<td>WMS-1</td>
<td>17.5</td>
<td>2016.6</td>
<td>4.44</td>
<td>30.6</td>
</tr>
<tr>
<td>WF-1</td>
<td>11.5</td>
<td>2016.6</td>
<td>2.49</td>
<td>17.2</td>
</tr>
<tr>
<td>WF-1</td>
<td>13.6</td>
<td>2014.6</td>
<td>2.07</td>
<td>14.3</td>
</tr>
</tbody>
</table>

Averages:  5.5 37.7
Std. Dev.:  3.2 22.2
Table 4.8  Caliche core triaxial test data, NDOT site (KI, 1996)

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth, ft</th>
<th>Density, pcf</th>
<th>US, ksi</th>
<th>US, MPA</th>
<th>E, ksf</th>
<th>E, GPA</th>
<th>v</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>15.6</td>
<td>159.5</td>
<td>7.9</td>
<td>54.2</td>
<td>687,000</td>
<td>33</td>
<td>0.17*</td>
</tr>
<tr>
<td>5</td>
<td>14.5</td>
<td>156.4</td>
<td>8.4</td>
<td>57.9</td>
<td>620,000</td>
<td>30</td>
<td>0.31</td>
</tr>
<tr>
<td>5</td>
<td>13.5</td>
<td>159.3</td>
<td>10.7</td>
<td>73.4</td>
<td>418,000</td>
<td>20</td>
<td>0.33</td>
</tr>
<tr>
<td><strong>Average:</strong></td>
<td></td>
<td></td>
<td>9.0</td>
<td>61.8</td>
<td>575,000</td>
<td>28</td>
<td></td>
</tr>
<tr>
<td><strong>Std. Dev.</strong></td>
<td></td>
<td></td>
<td>1.5</td>
<td>10.2</td>
<td>140,000</td>
<td>7</td>
<td></td>
</tr>
</tbody>
</table>

* - one lateral strain gage malfunctioned

4.6.3 Fremont Street Experience Site

During the geotechnical investigation for the Fremont Street Experience (WTI, 1994b), caliche cores were obtained and tested for UCS, Indirect (Brazilian) Tension (To) and Young’s modulus (E). There were distinct upper and lower layers at the site. The percent recovery and rock quality designation (RQD) were determined for the core runs, as shown in Table 4.9.

Tests to determine the Young’s modulus were performed by monitoring deflection with applied load. However, the tests were not performed in accordance with the ASTM procedure because a standard dial gage with a sensitivity of 0.001 inches was used instead of strain gages or a LVDT, and deformation measurements were not obtained in the lateral direction. The data (see Table 4.2) indicates an average E/UCS value of 400 which is less than the 505-570 range documented by Goodman (1980) for dolomite and limestone materials. Poisson’s ratio for the rocks from Goodman varied between 0.29 and 0.34. Splitting tensile strength was determined by the Brazilian test method (ASTM D3967). Most samples reached the ultimate UCS near 0.25% axial strain as indicated in Figure 4.49. The UCS and the splitting tension test results can be used to construct a Mohr’s circle which indicates a friction angle (\( \phi \)) of 38 degrees and an
average shear strength intercept, $S_i$, (cohesion) of 150 ksf (7 Mpa). Mohr’s circle plots from data are included in Appendix D. As a comparison to published data for limestone and dolomite rocks, Goodman reports $\phi$ and $S_i$ ranges of 35 to 42 degrees, and 146 to 500 ksf (7 to 24 Mpa), respectively. Jumikis (1983) indicates the friction angle of limestone varies between 35 and 50 degrees.

![Fremont Street & UNLV Core Data](image)

Figure 4.49 Caliche core stress-strain data
Table 4.9 Caliche core test data, Fremont Street site (WTI, 1994b)

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth, ft</th>
<th>Elev., ft.</th>
<th>Recovery, %</th>
<th>RQD, %</th>
<th>UCS, ksi</th>
<th>UCS, MPA</th>
<th>To, ksi</th>
<th>To, MPA</th>
<th>E, ksf</th>
<th>E, GPA</th>
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</thead>
<tbody>
<tr>
<td>B-4</td>
<td>6.3</td>
<td>2000.7</td>
<td>94</td>
<td>52</td>
<td>3.34</td>
<td>23</td>
<td>0.587</td>
<td>4</td>
<td>232,000</td>
<td>11</td>
</tr>
<tr>
<td>B-4</td>
<td>7.2</td>
<td>1999.8</td>
<td>94</td>
<td>52</td>
<td>2.73</td>
<td>18.8</td>
<td>0.221</td>
<td>1.5</td>
<td>185,000</td>
<td>9</td>
</tr>
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<td>B-10</td>
<td>16.3</td>
<td>1994.7</td>
<td>77</td>
<td>52</td>
<td>5.46</td>
<td>37.6</td>
<td>NA</td>
<td>NA</td>
<td>288,000</td>
<td>14</td>
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<tr>
<td>B-10</td>
<td>17.3</td>
<td>1993.7</td>
<td>77</td>
<td>52</td>
<td>3.26</td>
<td>22.5</td>
<td>NA</td>
<td>NA</td>
<td>77,000</td>
<td>4</td>
</tr>
<tr>
<td>B-10</td>
<td>18</td>
<td>1993.0</td>
<td>77</td>
<td>52</td>
<td>5.92</td>
<td>40.8</td>
<td>0.716</td>
<td>4.9</td>
<td>325,000</td>
<td>16</td>
</tr>
<tr>
<td>B-10</td>
<td>18.5</td>
<td>1992.5</td>
<td>77</td>
<td>52</td>
<td>1.64</td>
<td>11.3</td>
<td>NA</td>
<td>NA</td>
<td>145,000</td>
<td>7</td>
</tr>
<tr>
<td>B-10</td>
<td>19</td>
<td>1992.0</td>
<td>77</td>
<td>52</td>
<td>5.99</td>
<td>41.3</td>
<td>0.696</td>
<td>4.8</td>
<td>380,000</td>
<td>18</td>
</tr>
<tr>
<td>B-10</td>
<td>19.8</td>
<td>1991.2</td>
<td>77</td>
<td>52</td>
<td>2.25</td>
<td>15.5</td>
<td>NA</td>
<td>NA</td>
<td>210,000</td>
<td>10</td>
</tr>
<tr>
<td>B-10</td>
<td>20.3</td>
<td>1990.7</td>
<td>100</td>
<td>40</td>
<td>5.59</td>
<td>38.5</td>
<td>0.702</td>
<td>4.8</td>
<td>245,000</td>
<td>12</td>
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<td>B-10</td>
<td>20.8</td>
<td>1990.2</td>
<td>100</td>
<td>40</td>
<td>5.70</td>
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<td>NA</td>
<td>308,000</td>
<td>15</td>
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<td>B-10</td>
<td>21.8</td>
<td>1989.2</td>
<td>100</td>
<td>40</td>
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<td>355,000</td>
<td>17</td>
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<td>B-10</td>
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<td>1988.7</td>
<td>100</td>
<td>40</td>
<td>8.02</td>
<td>55.3</td>
<td>NA</td>
<td>NA</td>
<td>352,000</td>
<td>17</td>
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<tr>
<td>B-10</td>
<td>22.8</td>
<td>1988.2</td>
<td>100</td>
<td>40</td>
<td>4.83</td>
<td>33.3</td>
<td>0.513</td>
<td>3.5</td>
<td>260,000</td>
<td>12</td>
</tr>
<tr>
<td>B-16</td>
<td>16.8</td>
<td>2001.2</td>
<td>80</td>
<td>11</td>
<td>6.79</td>
<td>46.8</td>
<td>0.354</td>
<td>2.4</td>
<td>355,000</td>
<td>17</td>
</tr>
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</table>

Average: 5.0  34.2  0.5  3.6  258,615  12.5
Std. Dev. 2.0  14.1  0.2  1.3  88,987  4.2
4.6.4 UNLV Campus Site

During the construction of the Engineering Geophysics Test Site (EGTS) at the UNLV campus, rock cores of caliche material were obtained and select samples are shown in Figure 4.50. The test site is south of the Howard H. Hughes Engineering Building (see Tecle et al. 2003 for a detailed site map). The cores obtained from the UNLV campus had an average unit weight of 160 pounds per cubic foot from laboratory measurements.

One sample from the cores was tested for Young’s modulus in accordance with ASTM D3148 which had a value of 370,000 ksf at 1 percent axial strain. The stress-strain data is shown in Figure 4.49 with the Fremont Street core data. The UNLV caliche core exhibited higher stiffness compared to the Fremont Street caliche cores and showed additional stiffness with increased axial strain.

Compression and shear wave velocities of the caliche at the site were determined in the laboratory using free-free resonance testing methods, and in the field using the crosshole and downhole seismic test methods (Tecle et al. 2003). Based on the laboratory and crosshole testing, the average compression and shear wave velocity ranges were measured as 10,000 to 13,000 and 5000 to 8000 feet per second, respectively (Stone and Luke, 2001). For the caliche, the average Poisson’s ratio value inferred by the test data is 0.32.

Based on the downhole seismic tests, the average compression and shear wave velocity values in the caliche were 11,000 and 6,000 feet per second, respectively (Tecle et al. 2003). From the downhole test data, a Poisson’s ratio of 0.29 is calculated.
4.7 Conclusions

The Standard Penetration data (SPT) shows a wide scatter, but generally indicates that the soils in the upper 50 feet at the site are more dense or stiff than the deeper soils. This would be consistent with soil profiles that are overconsolidated near the surface (Bowles, Table 3-5, 1996). High blow counts (~100) are likely indicative of isolated cemented zones.

The field moisture, Liquid Limit and moist density data indicate that the most compressible soils at the site are between the depths of 50 and 70 feet (elevation 2000-2020 feet). The majority of the Liquidity Index (LI) data ranges between -0.2 and 0.5. The soft soils are more identifiable using PI and moisture content data instead of using SPT data.
From the Casagrande plasticity chart (Figure 4.9), most of the clay soils at the site have plasticity characteristics similar to glacial clays. Some clays at the site are classified as highly plastic (CH) type materials as discussed above.

Based on LI data and Figures 4.10 and 4.11, the site soils have an average sensitivity of about 3 to 5 and the preconsolidation pressure ranges between 4 and 40 ksf. Most of the PI data ranges between 10 and 30, and a correlation trend line would indicate a drained friction angle ($\phi'$) of 28° to 32°. Additionally, the direct shear tests (Figure 4.13) indicate an average $\phi'$ of 25° to 30° and a drained cohesion ($c'$) of 0 to 255 psf.

Gradation tests on soils at the site indicate the sand and gravel contents decrease with depth, most notably below a depth of 50 feet. Consolidated undrained (CU) triaxial tests were performed on a clayey sand soil sample from the NDOT site indicate a friction angle of 35° to 40°, and an effective cohesion value of 250 psf. An average UCS value of 8,200 psi was obtained from the caliche cores at the case study site.

Unconfined compression tests from the Fremont Street Experience and NDOT sites indicate an average strength of about 5,000 psi. The average Young’s modulus (E) and splitting tensile strength from the Fremont site were 260,000 ksf and 500 psi, respectively. The data (see Figure 4.49) indicates an average E/UCS value of 400 which is low compared to reported data on limestone and dolomite materials. Poisson’s ratio for these rocks (Goodman, 1980) varied between 0.29 and 0.34. Most of the samples tested for UCS from the Fremont Street site indicate a yield strain of 0.25 percent. Mohr circle plots based on two sets of UCS and splitting tension laboratory tests indicate an average friction angle of 38 degrees and a cohesion intercept of 150 ksf.
Caliche core measurements indicate the material from the NDOT site(s) has a unit weight of 160 pcf. Triaxial tests performed on the caliche cores indicate an average Young’s modulus and Poisson’s ratio of 575,000 ksf and 0.32, respectively.

Pressuremeter data between 60 and 130 feet indicates a best-fit increase in the reload modulus with depth of about 25 ksf per foot. Reload moduli ranges from 340 to 1,000 ksf for the 50 to 70 foot depth zone. The 50 to 70 foot depth range also has relatively high liquid limit and low moist densities. In this depth zone, the lowest pressuremeter modulus values were obtained, and the liquid limit and field moisture data can be used to identify the soft zones at a site. The undrained shear strength at the site generally increases with depth at a rate of about 5 psf per foot.

The Osterberg load test successfully held a load of 5,500 kips which indicates a working friction value of 10 ksf. The shear displacement curves indicate that a failure condition was not reached at all points along the shaft.

The load distribution data in the upper caliche for load test TP-1 may be unreliable due to smearing. Shear displacement data for test TP-1 indicates that the lower caliche has an ultimate friction value greater than 24 ksf. Load test TP-2 indicates that in the fractured state, the ultimate bond in the caliche is about 25 ksf and the ultimate shear in the soil below the upper caliche deposit is approximately 5 ksf. Load test TP-3 indicates that the augered pile achieves an excellent bond to the upper caliche, up to the test limit friction of 70 ksf.

Downhole seismic tests at the UNLV EGTS site indicate average compression and shear wave velocities in cemented soils of 11,000 and 6,000 feet per second, and a Poisson’s ratio of 0.29. For the caliche, the average Poisson’s ratio value inferred by the
dynamic testing is 0.30.

The caliche cores from both the UNLV and NDOT sites had an average unit weight of 160 pounds per cubic foot from laboratory measurements.

The photograph of the caliche cores indicates that the material can be highly variable in content. Some portions of the cores have large (compared to the core diameter) aggregates or gravel inclusions, and other portions of the cores show no inclusions. The Fremont Street caliche cores had no aggregate type inclusions.
CHAPTER 5

FOUNDATION BEHAVIOR IN A LAYERED SOIL/CALICHE PROFILE

5.1 General

This section presents the results of finite element analyses performed on typical foundation elements (shallow foundations and piles) to evaluate the response of a layered soil-caliche profile vs. a homogeneous profile. The commercially available programs; PLAXIS 2D and PLAXIS 3D Foundation (Brinkgreve et al. 2007) were used for the analyses.

5.2 Finite Element Analysis Using PLAXIS

PLAXIS is a finite element program used for deformation, stability, and dynamic analysis of geotechnical problems. Typical problems involve foundations, tunnels, excavations and slopes. Simulations in 2D may be either plane strain or axisymmetric. Plane strain problems are applicable to structures that may be considered continuous in the in-plane (z) direction, and displacements and strains in the z-direction are assumed to be zero. An axisymmetric model is useful for problems that are symmetric around a central axis.

The soil may be modeled using either 6 or 15 node triangular elements. The 15 node elements use a 4th order interpolation for displacements and have 12 Gauss integration points. The PLAXIS reference manual indicates that the 15 node elements have been shown to be very accurate for stress calculations of difficult problems such as collapse problems in incompressible soils. Although 4, six node elements have the same number of stress points as one 15 node element, the latter is more accurate than the former. The 15 node elements are used for all the calculations in this dissertation.
Models may include structural elements consisting of beam (plates) and geogrid elements. Plate elements are characterized by their axial and bending properties (per foot or meter), and are used to simulate walls and floors. Ultimate bending moments of plates may be introduced to simulate plastic behavior. Geogrid elements consist of line elements with axial stiffness and no bending stiffness, and may support only tension loads.

Interface elements may be used to simulate the interaction between structural and soil elements. These are special elements that allow for plastic behavior to occur between, for instance, soil and a wall, if the stress field is such that failure occurs, according to the Mohr-Coulomb (MC) failure criteria. A special interface factor ($R_{inter}$) is used to reduce the soil strength parameters ($c$ & $\phi$) at the soil structure interface. The physical thickness of the interface element is zero, but a virtual thickness is specified so specific soil properties may be assigned to the element. This virtual thickness is specified as a factor times the average element size, and the default factor of 0.1 is used in the calculations in this research.

5.2.1 Constitutive Models

A variety of soil constitutive models are available in PLAXIS, including the following:

1) Linear Elastic
2) Mohr Coulomb
3) Hardening Soil
4) Soft Soil and Soft Soil Creep
The linear elastic model, which is based on Hooke's law of linear elasticity, includes no failure criteria, and generally is not suitable for modeling soils except at very small strain levels. The linear elastic model is commonly used to model structural materials such as steel and concrete. The model requires two parameters; Young’s modulus (E) and Poisson’s ratio (ν).

The Mohr Coulomb (MC) model is an elastic-perfectly plastic soil model that is widely used for soil and rock. The model requires five parameters, namely E, ν, friction angle (φ), cohesion (c) and dilatancy angle (ψ). For highly over-consolidated clays, E is usually the initial modulus at low strain levels, whereas for sands and normally consolidated clays, it is better to assume E as the secant modulus corresponding to 50% strength. The observed soil stiffness related to the elastic modulus is dependent upon many factors, including stress level, stress path, and strain level.

The strength parameters c and φ in the MC model, as used in PLAXIS, are usually effective strength parameters. As stated in the PLAXIS users manual, when using the MC model for undrained analyses based on the drained strength parameters, the model does not predict the variation in shear strength with changes in pore water pressure as well as the advanced elasto-plastic models (Nos. 3 and 4 above). It is possible to use undrained strength parameters, but changes in strength, due to consolidation, are not realized. The PLAXIS user manual should be referred to for more details on undrained strength analyses. The MC model also includes the effect of volume change on plastic strains through the use of the dilatancy angle (ψ > 0 for φ > 30°) and a non associated flow rule. The non associated flow rule in the model stipulates that the plastic strain increment vectors are not normal to the yield surface which is a characteristic found when
modeling volumetric strains in most frictional soils. The MC model with effective strength parameters is used to model the soils for all subsequent analyses in this dissertation.

The Hardening Soil (HS) and Soft Soil (SS) models are advanced elasto-plastic constitutive models. The HS model is based on the well known Duncan-Chang hyperbolic model (Duncan and Chang, 1970) for soil, but differs from it in three ways. First, the HS model is based on the theory of plasticity vs. elasticity. Secondly, it includes the effect of dilatancy or volume change, and thirdly, by including a yield cap. The yield cap is not fixed in space and expands due to plastic straining. The initial position of the yield cap is set by the initial stress state. For undrained loading, the HS model is able to predict the reduction in mean effective stress for loose sands and normally consolidated clays, and the negative pore pressure increase associated with the undrained shear of dense sands and over-consolidated clays. The HS model is a versatile model that is suitable for both stiff and soft, normally consolidated soils. This model is most suitable for excavation problems and may also be used to model foundation settlement.

The soft soil (SS) model is a Cam-Clay (Schofield and Wroth, 1968) type constitutive model designed to simulate the logarithmic relationship between mean effective stress and void ratio. It is most suitable for modeling the compression of soft, near normally consolidated clays for both pre-primary and post-primary consolidation stress ranges. The PLAXIS SS model improves on the Cam-Clay model by solving the problem of over predicting the shear strength of over-consolidated stress states and by the introduction of a Mohr-Coulomb failure criterion to improve the model performance near
failure. This also allows more flexibility in choosing model parameters so that more realistic lateral stress states can be established. The PLAXIS software also includes the Soft Soil Creep (SSC) model to predict secondary consolidation. These models are used primarily to time dependent settlement problems.

5.2.2 In-Situ Stress State

When using the PLAXIS finite element program, initial stresses may be generated using either a gravity loading procedure or the Ko method. Gravity loading is used for non-horizontal soil and phreatic boundaries, whereas, the Ko method is applicable for horizontal stratigraphy. In the Ko method, the vertical stresses are generated to establish equilibrium relative to the self weight of the soil. The initial lateral stresses are then calculated based on the Ko proportionality factor. Alternatively, if one is interested in analyzing a weightless medium, the body force multiplier may be set equal to zero. For elasto-plastic models, it is important to establish the correct in-situ stress state so that the yield cap location is properly set.

5.2.3 Verification

The PLAXIS user manual set includes a verification document in which problems with known analytical solutions are compared to models performed with the software. These include both elastic and plasticity based problems for footings, plate and beam bending, shell elements, cavity expansion, interfaces, and groundwater flow. Additionally, Prakoso (1999) also performed a standard finite element check for stress calculation and convergence using the well known patch test (Cook et al. 1989). Prakoso examined the stress calculation under highly asymmetric meshes of six-node triangular plain strain elements to evaluate the mesh independency on the solution, and found the
software passed the test under shear, compression and tension loads. The tests indicate that the software is sufficiently accurate for the intended use.

5.3 Finite Element Analysis Using PLAXIS 3D Foundation

PLAXIS 3D Foundation is very similar to the 2D program described above, yet the analysis is extended to three dimensions. The software is most suited to the analysis of foundations such as mats, irregularly shaped footings, piles and pile groups, and pile supported rafts. It is also possible to model excavations using wall elements, although, it is generally more suitable to use a 2D plane strain analysis. All of the constitutive models mentioned above are available in the 3D program. Structural elements consist of vertical and horizontal beams, plates, walls, ground anchors, springs, volume piles, and embedded piles. Interfaces to model soil-structure interaction are automatically added to walls, but may be excluded for volume piles.

Boreholes are included in the program to define soil stratigraphy at any point at the top of the 3D mesh. It is possible to define different soil types and layer depths, as the model will linearly interpolate between boreholes to form the model layers. As with the 2D program, the mesh may be refined around points, lines or clusters, and vertical refinement is also possible (y-direction). Elements consist of quadratic 15 node wedge elements which may degenerate to 13 or 10 node elements in the case of non-horizontal geometry. The wedge elements provide a quadratic interpolation of displacements, and the Gaussian integration performed in each wedge element is based on 6 sample points.

5.3.1 Embedded Piles

In PLAXIS 3D, piles may be introduced into the model by means of volume elements, or embedded piles. The earlier version of the software included volume piles
which are volume clusters formed in the shape of a square or circular (or practically any shape) pile section. Structural properties are then assigned to the pile elements to simulate the presence of a pile. If an interface is not assigned to the volume pile, it will be adhesively connected to the mesh and no relative displacement between the pile and soil may occur. This is generally suitable for elastic analyses of piles in soils, but not used for modeling a pile load test to failure since the soil friction and end bearing properties are not explicitly specified. A disadvantage of volume piles is that they are composed of individual volume elements (which are 15 node wedge elements), thereby, largely increasing the number of elements for a single pile. This renders them less suitable for modeling problems with a large number of piles.

The most current version of PLAXIS 3D Foundation includes a new type of pile element which consists of a beam element that may be placed arbitrarily in any volume element without the adverse effect of a large increase in elements and nodes. Figure 5.1 shows how an embedded pile (dark black line) is placed in a volume element. Three pile (beam) nodes are added to each volume element that the beam crosses. The embedded pile in PLAXIS is intended to model the interaction with the surrounding soil by means of an interface at the pile perimeter and at the pile base. Ultimate skin friction and end bearing values are specified for each pile as limiting values.
The interface element consists of 3 node line elements with pairs of nodes. Three of the nodes are placed on the beam element while the remaining three nodes are placed on the edges of the volume element (see Figure 5.1). The beam element is linear elastic and the interface has elasto-plastic properties.

Skin traction forces are developed from the relative movements between the interface nodes on the beam element and the nodes at the edge of the volume element. The force-displacement relationship for the skin traction (ultimate soil friction values) may be defined as a linear distribution, multi-linear or layer property dependent. The force acting at the pile tip is determined by the relative displacement of the base spring which is elastic-perfectly plastic. An ultimate end bearing value is specified to represent a failure load. To prevent mesh dependency effects where elements are small enough to be inside the pile radius (Engin et al. 2009), integration points in this zone are forced to remain elastic. This gives the beam element the characteristics of a volume pile within
the zone defined by the pile radius. Ground anchors or tiebacks as structural elements in PLAXIS 3D Foundation consist of both an embedded pile for the grouted portion of a tieback and an elastic line element for the anchor rod.

Embedded piles have been shown to perform well in simulating actual pile load tests in both tension and compression (Engin et al. 2008; Brinkgreve and Swolfs, 2007). As shown in Chapter 6, the results from the embedded pile and a 2D axisymmetric analysis match well for a pile load test performed at the case study site.

5.4 Vertically Loaded Footing

The stiffening effect of a caliche layer at the surface was investigated by examining the behavior of a rigid footing model. An axisymmetric model of a perfectly flexible and rigid footing on a homogenous soil profile was initially evaluated with PLAXIS 2D and compared to the theoretical solution. The soil and footing parameters are shown in Table 5.1, where:

\[ Ec = \text{footing Young’s modulus,} \]
\[ Es = \text{soil Young’s modulus,} \]
\[ d = \text{footing diameter,} \]
\[ t = \text{footing thickness,} \]
\[ vc = \text{footing Poisson’s ratio,} \]
\[ vs = \text{soil Poisson’s ratio,} \]
\[ q = \text{pressure load,} \]
\[ I = \text{influence factors.} \]
Table 5.1 Footing model parameters

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5.4.1 Mesh Refinement

To evaluate the sensitivity of the finite element results to the mesh configuration, the maximum displacement, vertical stress and vertical strain of the model were evaluated for different levels of mesh refinement. The stress and strain from each model was evaluated on the footing centerline at a depth of 3 feet (one footing radius) below the upper model boundary. Mesh sizes ranged from 129 to 1,895 elements. Results for both a flexible footing (no footing) and a rigid footing are shown in Tables 5.2 and 5.3. Based on the mesh refinement study, there is a slight decrease in deflection for meshes finer than mesh 3, and there was no difference in the deflection for meshes 5, 6, and 7 to the nearest 0.0001 inches. The stresses and strains reach nearly constant values for meshes finer than meshes 4 and 5 for both the flexible and rigid cases, respectively. The range of mesh sizes are shown in Figure 5.2 and the results are shown in Tables 5.2 and 5.3. It can be concluded from the mesh sensitivity study that meshes finer than mesh 4 have no effect on the results for the footing settlement problem.

A finite element mesh with 1,522 elements and 12,486 nodes was used in the parametric studies that follow, as shown in Figures 5.6 and 5.7. It should be noted that
this mesh is finer than mesh 6, but coarser than mesh 7.

Table 5.2  Flexible footing problem mesh refinement results

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Table 5.3  Rigid footing problem mesh refinement results

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5.4.2  Settlement

For the flexible loaded area, the maximum vertical deformation resulting from the PLAXIS model was 0.614 inches, which compares exactly to a settlement of 0.614 inches calculated using the theoretical solution from Brown (1969), as referenced by Mayne and Poulos (1999). In addition, the theoretical solution for the vertical stress on the centerline of the footing at a depth of one radius is 6.46 ksf (Poulos and Davis, 1974) which compares well to a value of 6.47 ksf calculated by Mesh 7 in Table 5.2.
Figure 5.2  Meshes used for refinement study

The deformed mesh of the model and model dimensions are shown in Figure 5.3. Vertical displacement and strain contours are shown in Figures 5.4 and 5.5. Theoretical solutions are also available for the settlement of a rigid footing (Mayne and Poulos, 1999). For the case with the rigid footing, the maximum vertical deformation resulting from the PLAXIS model was 0.469 inches, compared to the theoretical solution of 0.485 inches. The deformed mesh for the case with the rigid footing is shown in Figure 5.6.
To evaluate the effect of a stiff (caliche, or thin rock) layer on the footing deformation, a stiff layer was placed at the top of the model, immediately below the rigid footing. The thickness of the stiff caliche layer was varied between 0.5 and 6 feet. Settlements were evaluated as before for the case with the upper caliche layer with thicknesses ranging from zero to one footing width or diameter. The deformed mesh of the model with rigid footing on an upper caliche layer (white layer) is shown in Figure 5.7.
Figure 5.4  Vertical stress for rigid footing case

Figure 5.5  Vertical strain for flexible footing case
Figure 5.6 Deformed mesh – rigid footing, elastic half space model

Figure 5.7 Deformed mesh; non-homogeneous footing model
The results of the analysis in non-dimensional form are indicated in Figure 5.8. The term $\frac{Sc}{S}$ represents the ratio of the settlement with a caliche layer to that without a layer, and $\frac{Hc}{B}$ equals the thickness of the caliche layer relative to the footing width (diameter).

The results indicate that the stiffening effect from the caliche layer directly below a footing is significant. A layer with a relative thickness of $\frac{1}{4}$ the footing width reduces the settlement by about 70 percent. A caliche layer with a thickness of one half of the footing width (3 feet thick for this example) reduces the settlement to 15 percent of the settlement of the homogenous model. The greatest benefit in settlement reduction is achieved with a caliche layer thickness less than one half of the footing width, as larger layer thicknesses have less additional effect on the settlement. In practice, it should be realized that for thin caliche layer thicknesses relative to the width, failure of the upper caliche layer in punching shear or bending can occur and should be a part of the design process.

The presence of the caliche layer also tends to smooth the deflection profile (and the differential settlement between adjacent footings) beyond the footing, and the deflection profile away from the footing extends out a greater distance than in the homogeneous soil case. The extended deflection profile may be attributed to the global stiffening or beam effect of a continuous cemented layer.

Vertical deflection contours for the case of $Hc=H$ are shown in Figure 5.9. The zero deflection contour is within the model, indicating the model width is sufficient to avoid boundary effects in the lateral direction.
5.4.3 Stress Distribution in Soil/Caliche Profile

The stiff over soft layer profile represents a multilayer vertical stress distribution problem investigated by Fox (1948), and published by Poulos and Davis (1974). The analysis of a multi-layer system is also applicable to the study of stresses in a pavement system (e.g. Croney and Croney, 1998). In this scenario, the vertical stress in the upper layer is distributed at a faster rate than for the homogeneous profile. A solution to this problem for an elastic modulus ratio (E1/E2) of 10 is shown in Figure 5.10, where E1 is the modulus of the upper layer. As indicated in Figure 5.10, at a depth of one radius the vertical stresses dissipate at twice the rate compared to the homogenous case for this modulus ratio. Poulos and Davis (1974) also present solutions to the displacements of a two layer system by converting it to a homogenous problem with an equivalent elastic modulus.

The layered elastic problem was evaluated with PLAXIS for the model footing used previously (6-foot diameter flexible loaded area). Vertical stress distributions were determined for caliche thicknesses of 1 to 9 feet, and for the homogeneous case. The results are shown in Figure 5.11 in non-dimensional form, where, $\sigma_v =$ vertical stress, $q =$ applied vertical load, $a =$ footing radius, $z =$ depth. As observed in the theoretical case (Poulos and Davis, 1974), the stresses in the caliche layer dissipate with depth at a faster rate when compared to the homogenous or Boussinesq case.
Figure 5.8 Effect of caliche layer on footing settlement

Figure 5.9 Vertical deflection contours for the case of $H_c=H$
For homogenous soil profiles, it is common to assume foundation stresses induced by a spread footing distribute according to the 1:2 (H:V) rule (e.g. Das, 2004). This assumption is generally valid for depths greater than one footing width. This method provides for a rapid means to calculate vertical stresses at depth from a loaded footing, as opposed to using the Boussinesq equation. Based on the author’s experience, it is common among geotechnical engineers in Las Vegas to assume that foundation stresses dissipate in cemented zones at a rate of 1:1 (H:V) which allows for lower stresses at depth compared to the common assumption for soil.
The data from this analysis indicates that for the case of a caliche layer greater than 1 foot, stresses dissipate at an average rate of 3.9:1 (H:V), or 1:0.26. This is greater by a factor of approximately 4 than typically assumed by geotechnical engineers in Las Vegas. As indicated in Figure 5.11, for the case of the upper layer of caliche being 1 foot or less in thickness, the rate of stress dissipation is reduced and failure conditions might occur in soils represented by an elasto-plastic constitutive model.

5.5 Vertically Loaded Pile in Soil

As a follow up to the footing problem, the pile response in a layered soil/caliche profile was investigated. An axisymmetric model of a compressible pile in a
homogenous soil profile was initially evaluated with PLAXIS and compared to the theoretical solution. The soil and pile parameters are shown in Table 5.4, where $E_p =$ pile Young’s modulus, $E_s =$ soil Young’s modulus, $L =$ pile length, $d =$ pile diameter, $v_{\text{caliche}} =$ Poisson’s ratio, for caliche, $v_p =$ pile Poisson’s ratio, $v_s =$ soil Poisson’s ratio, $q =$ pressure load.

Table 5.4 Pile model parameters

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5.5.1 Mesh Sensitivity - Pile Problem

To evaluate the sensitivity of the finite element results to the mesh configuration, the maximum displacement of the pile model was evaluated for different levels of mesh refinement. Mesh sizes ranged from 129 to 1,895 elements. Based on the mesh refinement study, there is a slight decrease in deflection for meshes finer than mesh 3, and there was no difference in the deflection for meshes 4 through 8 to the nearest 0.0001 inches. The range of mesh sizes are shown in Figure 5.12 and the results are shown in Table 5.5. It can be concluded from the mesh sensitivity study that meshes finer than mesh 4 have no effect on the results for the footing settlement problem. For the parametric studies that follow, a mesh with 2,551 elements and 20,799 nodes was used,
as shown in Figure 5.13.

Table 5.5 Pile problem mesh refinement results

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5.5.2 Pile Settlement

Similar to the shallow foundation study, the settlement of a single pile in an elastic soil was evaluated for homogeneous and layered soil/caliche cases. A 2 foot diameter pile with an axial load of 100 tons (200 kips) was considered. An interface between the pile and soil was not used since this evaluation was intended to study pile settlement in a linear elastic soil with no slippage or plastic deformation effects. The finite element mesh has a lateral dimension of 100 feet, and a depth of 80 feet.

For a compressible pile in a homogeneous, linear elastic medium, the vertical top deformation resulting from the PLAXIS model was 0.162 inches, which compares well to a pile settlement of 0.163 inches calculated using the analytical solution from Randolph and Wroth (1978). The deformed mesh of the model is shown in Figure 5.13. Vertical stress, vertical strain, shear stress and shear strain contours are shown in Figures 5.14 to 5.17. Vertical displacement and strain contours are shown in Figures 5.18 and 5.19.
The location of the zero displacement contour in Figure 5.14 indicates the model width is sufficient to avoid boundary effects. Additionally, it is noted that the distance to the point where shear stresses become negligible (limit of influence) for the subject pile is calculated to be 32.5 feet (or 32.5r, where r = pile radius) based on Randolph and Wroth (1978).
As shown in Figure 5.16, a slight vertical stress occurs at the pile top, but the vertical stress induced by the load is primarily around the pile tip as indicated by Sowers and Sowers (1970). The shear stress values shown in Figure 5.18 are in general agreement with the theoretical values presented by Poulos and Davis (1974).

To evaluate the effect of a caliche layer on the pile top deformation, the model settlement was evaluated for caliche layers at the pile top, pile tip, and both the top and the tip. The thickness of the caliche layer was varied between 0.5 and 6 feet. For the case of hard layers at the pile top and below the tip, the hard layer thicknesses were equal. The deformed mesh of a model with the caliche layer (white layer) at the top is shown in Figure 5.20, and with the hard layer below the tip is shown in Figure 5.21.

For the case of the caliche thickness equal to 6 feet, as shown in Figure 5.15, the zero deflection contour is within the model, indicating the model width is sufficient to avoid boundary effects.

The results of the analysis in non-dimensional form are shown in Figure 5.22. The term Sc/S represents the ratio of the settlement of a soil profile with a caliche layer to the settlement of a soil profile without a caliche layer. Hc/d equals the thickness of the caliche layer relative to the pile diameter. The results indicate that the presence of the stiff layer at the pile top also has a settlement reducing effect, although not as significant as the footing.
Figure 5.13  Deformed mesh for single pile problem

Figure 5.14  Single pile, vertical deflection contours for homogeneous case
The additional reduction in settlement for the footing case is expected since the footing case has more contact with the caliche than the pile, and can therefore derive more stiffness from the hard layer. A layer with a relative thickness equal to the pile width reduces the settlement of the homogenous model by about 50 percent. The results for the case of caliche at the pile top and tip are essentially the same for a stiff layer thickness less than one pile diameter. The benefit of the hard layer at the tip vs. the top is less for thicker layers, as shown in the plot. This is likely due to the additional load transfer to the hard layer that occurs for thicker layers at the pile top.
Figure 5.16  Vertical stress field around pile tip

Figure 5.17  Vertical strain field around pile tip
If a stiff layer is present at both the top and the tip, as is the case with a CSP foundation pile, additional settlement reducing effects result as shown. In conclusion, for piles installed in a soil profile with caliche layers, the least settlement would occur in a single loaded pile if the pile was installed through an upper caliche layer and tipped on a lower layer.
Figure 5.20  Deformed mesh, hard layer at pile top

Figure 5.21  Deformed mesh, hard layer below pile tip
5.5.3 Single CSP Analysis

The case of caliche at the pile top and tip is in fact a CSP type foundation. The load transfer characteristics of this case are investigated for the case of 2 foot thick hard layers at the top and below the bottom of the pile. The deformed mesh is shown in Figure 5.23. Contours of vertical deformation, stresses, strains, shear stress and tensile stress in the horizontal direction are shown in Figures 5.24 to 5.29.

When the load is applied at the pile top, the plot of shear stress contours shown in Figure 5.27 indicate that a portion of the pile load is transferred to both the upper and lower caliche layers. To a much lesser extent, some load is transferred to the intermediate soil layer through differential deflection between the pile and soil. The load transfer to the caliche layers results in the vertical deformation of the caliche layers and the observed vertical stress and strain distributions shown in Figures 5.25 and 5.26. There is some deformation (strain) in the intermediate soil layer, but the maximum strains are in the soil below the lower caliche layer. For this case, the maximum vertical strain at the pile tip is about 0.03 percent. For the case without caliche layers, the maximum vertical strain is 0.4 percent, which is an order of magnitude greater than the case with hard layers present. This magnitude of difference also occurs when evaluating shear strains.

The analysis also indicates that the shear strains extend out from the pile center about 20 diameters, as shown in Figure 5.28. For the case with no hard layers, the shear strains extend outward about 5 pile diameters. This difference is due to the beam action resulting from the presence of the hard layers.
A comparison of the displacement fields shown in Figures 5.14 and 5.24 resulting from the two cases indicate an observation similar to the discussion above, i.e., the displacement field is wider for the case with the upper caliche layer. It is well known that pile to pile interaction causes settlements in an adjacent pile due to the settlement of another pile in a group arrangement (e.g., Randolph and Wroth, 1979). Presumably, for the case of a pile with an upper hard layer, the group effect from a single pile on adjacent piles would be greater and encompass more piles compared to the homogenous case.

It is of further interest to view the magnitude of horizontal tensile strain in the caliche layers (see Figure 5.29), since they deflect similar to unreinforced pavement. In the design of pavements, tensile stresses are evaluated at the bottom of the pavement.
layer to avoid failure in flexure (e.g., Croney and Croney, 1998). Similarly, failure of a caliche layer in bending can occur if the stresses are excessive. The maximum tensile stress computed by the model is on the order of 175 psi. For a 4,000 psi concrete, the tensile strength based on ACI 363R is 470 psi. For this case, a factor of safety against failure in bending would be 2.7.

5.6 Vertically Loaded Pile Group in Soil/Caliche Profile

In this section, the single pile problem was recast as a 4 pile cap, and the pile cap response in a layered soil/caliche profile was investigated. The soil and pile parameters are identical to those in the previous section, and the pile cap has dimensions of 9 foot square by 4 feet in thickness. The pile to pile spacing is 3 diameters, or 6 feet. The load per pile of 200 kip, as used previously, was maintained for the pile cap response study.

A quarter plane model was used since the 2 x 2 pile cap is symmetric about its center. The model mesh has dimensions of 200 feet square by 200 feet in depth. A relatively large and deep mesh was required due to the plate effect from the caliche layers. As the thickness of the caliche increases, the boundary effects become evident as the point of zero vertical deflection nears the mesh boundaries. Pile and cap parameters are listed in Table 5.6. The finite element representation of the pile cap and volume pile is shown in Figure 5.30.
Figure 5.23  Deformed mesh, CSP 2 ft. caliche thickness

Figure 5.24  Vertical deflection, CSP 2 ft. caliche thickness
Figure 5.25  Vertical stress, CSP 2 ft. caliche thickness

Figure 5.26  Vertical strain, CSP 2 ft. caliche thickness
Figure 5.27 Shear stress, CSP 2 ft. caliche thickness

Figure 5.28 Shear strain, CSP 2 ft. caliche thickness
Figure 5.29  Tensile stress in x-direction, CSP 2 ft. caliche thickness

Table 5.6  Pile group model parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>E pile (ksf)</td>
<td>600,000</td>
</tr>
<tr>
<td>E caliche (ksf)</td>
<td>300,000</td>
</tr>
<tr>
<td>E soil (ksf)</td>
<td>1000</td>
</tr>
<tr>
<td>L (ft.)</td>
<td>20.0</td>
</tr>
<tr>
<td>dp (ft.)</td>
<td>2.0</td>
</tr>
<tr>
<td>v caliche</td>
<td>0.30</td>
</tr>
<tr>
<td>vp (ftg.)</td>
<td>0.20</td>
</tr>
<tr>
<td>vs (soil)</td>
<td>0.35</td>
</tr>
<tr>
<td>P (kips)</td>
<td>800</td>
</tr>
</tbody>
</table>
5.6.1 Mesh Sensitivity

For the pile cap model, the maximum displacement of the homogenous pile cap model was evaluated for different levels of mesh refinement. Mesh sizes ranged from 1,482 to 39,836 elements. Based on the mesh refinement results, each finer mesh causes the model to become more flexible, as expected. Each successively finer mesh only showed slight increases in deflection. Given that the converged solution is 0.360 inches, it is surprising that the deflection of the coarsest mesh is only 0.013 inches less than the finest mesh. This is likely due to the use of quadratic 15 node wedge elements which appear to behave well in this scenario. The mesh refinement study indicates that model performance is more sensitive to changes in the number of vertical elements compared to further refinement of horizontal elements.
Some selected mesh designs are shown in Figure 5.31 and the results are shown in Table 5.7. From the mesh refinement process, it can be concluded that meshes finer than mesh 5 have no significant effect on the results of the pile cap problem. For the parametric studies that follow, the mesh used is shown in Figure 5.33. It should be observed that the pile cap model study results in solutions that are non-dimensional and relative to the displacement with no caliche layers. Thus, if the same mesh is used for all calculations, the results may be considered to be accurate for the purpose of making relative comparisons.

Table 5.7 Pile group 3D model mesh refinement results

<table>
<thead>
<tr>
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<tbody>
<tr>
<td>1</td>
<td>1482</td>
<td>4549</td>
<td>0.348</td>
</tr>
<tr>
<td>2</td>
<td>3000</td>
<td>8869</td>
<td>0.349</td>
</tr>
<tr>
<td>3</td>
<td>5400</td>
<td>15523</td>
<td>0.353</td>
</tr>
<tr>
<td>4</td>
<td>7050</td>
<td>20088</td>
<td>0.354</td>
</tr>
<tr>
<td>5</td>
<td>9024</td>
<td>25541</td>
<td>0.355</td>
</tr>
<tr>
<td>6</td>
<td>17216</td>
<td>47611</td>
<td>0.360</td>
</tr>
<tr>
<td>7</td>
<td>24748</td>
<td>67939</td>
<td>0.360</td>
</tr>
<tr>
<td>8</td>
<td>39836</td>
<td>107295</td>
<td>0.361</td>
</tr>
</tbody>
</table>

5.6.2 Pile Group Settlement

The model of a single compressible pile in a homogenous soil profile was evaluated with PLAXIS 3D and compared to the theoretical solution and the results from the 2D model. The selected mesh configuration for this study has 14916 elements and 41392 nodes, as shown in Figures 5.33 and 5.34. It was possible to further refine the mesh near the pile and add more vertical elements to achieve the converged solution of meshes 7 and 8.
For a compressible single pile in a linear elastic medium, the vertical top deformation resulting from the PLAXIS 3D model was 0.160 inches, compared to 0.163 inches which is calculated using the analytical solution from Randolph and Wroth (1978).

For the pile group, the maximum vertical displacement for the cases of a single pile with and without a pile cap is shown in Table 5.8.

The group settlement ratio (or settlement factor) is 2.3 which is consistent with what is calculated using the simplified method for pile groups in sand proposed by Vesic (1977). Using Vesic's method, the settlement ratio would be 2. Displacement contours for the pile group with no stiff layers are shown in Figure 5.32.

The numerical model solution of the pile group with the cap effect can be compared to an approximate pile raft solution by Randolph (1994) which is based on the stiffness of the raft group and the stiffness of the pile group. In this analysis, the stiffness of the pile group is calculated including pile to pile or group effects, such that the group stiffness is less than the sum of the individual pile stiffness values. The raft stiffness is a dependent on the raft thickness and the elastic moduli of the raft and soil (e.g., Mayne and Poulos, 1999). The advantage of the approximate method by Randolph is its simplicity, and it provides information load sharing between the pile and the raft. Using this method, the computed displacement of the pile group is 0.34 inches (see Case 5 in Table 5.8) which compares well to the numerical solution (case 4).
Figure 5.31  Deformed meshes for 3D mesh refinement study for pile group model
The numerical solution for the case of no pile cap also compares well to the pile group solution by Chow (1986) which utilizes linear springs and considers pile to pile interaction. The cap is considered rigid, such that all piles experience the same deflection, although, the effect of the cap on the pile group stiffness is not included. Whereas the PLAXIS solution includes the entire half-space continuum, Chow's analysis consists only of the piles in the group. Each pile is modeled using beam elements which have elastic springs on the pile sidewall (t-z springs) for the soil and a spring on the pile bottom to simulate end bearing. Pile to pile interaction is accounted for using the Mindlin equation (Poulos and Davis, 1980). All spring stiffness elements are combined into a single stiffness matrix for solution. This analysis method was coded by the author using Matlab.
software, and the result is listed as Case 3 in the table. The Matlab code for Chow's analysis is included in Appendix G.

The comparison between the numerical solution and simplified, analytical solutions for the pile group with and without the cap indicates that the 3D PLAXIS model is sufficiently accurate for the intended study.

Table 5.8 Pile cap in homogeneous soil results

<table>
<thead>
<tr>
<th>Case No.</th>
<th>Description</th>
<th>Maximum Settlement, in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Single pile, no cap</td>
<td>0.160</td>
</tr>
<tr>
<td>2</td>
<td>FEM-4 piles, no cap</td>
<td>0.373</td>
</tr>
<tr>
<td>3</td>
<td>Chow soln., no cap</td>
<td>0.388</td>
</tr>
<tr>
<td>4</td>
<td>FEM-4 piles w/ cap</td>
<td>0.360</td>
</tr>
<tr>
<td>5</td>
<td>Approx. pile raft w/ cap</td>
<td>0.340</td>
</tr>
</tbody>
</table>

To evaluate the effect of a caliche layer on the pile cap deformation, the model was evaluated for caliche layers at the pile top only, pile tip only and both the top and the tip. The thickness of the caliche layer was varied between 2 and 10 feet at the pile top, and between 2 and 10 feet below the pile tip. For the latter case, the layer thicknesses at the top and tip are the same for each evaluation.

The results of the analysis in non-dimensional form are indicated in Figure 5.35. The term \( \frac{S_c}{S} \) represents the ratio of the settlement with a caliche layer to the settlement for the homogeneous case. The term \( \frac{H_c}{B_g} \) is the thickness of the caliche layer relative to the pile group width.
Figure 5.33 Mesh used for parametric study of pile group

Figure 5.34 Close view of mesh used for parametric study of pile group
The results for the case of caliche at the pile top and tip are very similar, although, at caliche thicknesses greater than half the pile group width, there is less settlement for the case of caliche at the pile top. As with the single pile, this is likely due to the additional load transfer to the hard layer that occurs for thicker layers at the pile top. Another contributing factor is the presence of the pile cap contact with the upper caliche layer.

The pile group model was also subject to a hard/stiff layer at both the top and the tip, as is the case with a CSP foundation. In this case, additional settlement reducing effects result, as shown in the plot, and the same behavior was observed for the single pile case. The data reflects that the greatest settlement reducing benefit is the case of \( H_c \) less than \( B_g \), so less effect of the caliche thickness on settlement is observed for greater values of \( H_c \). Qualitatively, the steeper the curve, the more benefit derived from the presence of the caliche.

The value of the CSP foundation is realized by comparing it to the settlement of the pile cap without the piles, or to a spread footing. Using the subject 3D model with a 2 foot thick caliche layer at the pile top and tip, the reduction in settlement of the CSP system vs. the spread footing is 30 percent. For a caliche layer at the top and the tip with a thickness equal to the pile group width (6 feet in this case), the settlement of the CSP group is only 20 percent of the same pile group in soil. The reduced settlement allows for greater pile design loads to be used, assuming the load capacity is satisfied.

The deformed mesh and vertical deflection contours for the 4 feet CSP case are shown in Figures 5.36 through 5.39. Also shown, is the distribution of horizontal tensile stress on the bottom of the lower caliche plate. In this case, the tensile stress is 63 ksf.
(438 psi). The numerical model is helpful in determining the stress under any load condition. It is of particular value in this case since excessive tensile stress could result in cracking of the lower caliche plate.

5.7 Conclusions

Using the PLAXIS 2D and 3D software, the effect of stiff layers in a soil profile have been studied. The cases of a spread footing, a single pile and a 4 pile group were examined. Soil properties were identical for all cases. To get a basic understanding of the behavior of the foundations, and because parametric studies using 3D models are time consuming, only linearly elastic behavior was considered. Pile to pile interaction in pile groups is over predicted using a linear elastic model since the nonlinear effects for pile deformation are ignored and the deformation is more localized at the pile soil interface (Randolph, 1979). Additionally, the interaction between piles decreases as the soil profile becomes less homogeneous, i.e., for a profile in which the stiffness increases with depth (Banerjee and Davies, 1977). The reduced interaction results in an increased tip load compared to a homogenous profile (Randolph, 1979). However, using the simplified soil model for the analyses gives valuable insight to the behavior of foundations in soil profile with stiff inclusions.

The spread footing evaluation indicated that the vertical stress distribution spreads at an average rate of 3.9:1 (H:V), or 1:0.26. For footings placed on caliche layers at the surface, a caliche layer with a relative thickness of ¼ the footing width reduces the settlement by about 70 percent.
The analysis of the single pile indicates that a caliche layer with a relative thickness equal to the pile width reduces the settlement of the homogenous model by about 50 percent. The shear stress distribution indicates that the load from the pile is transferred to the caliche layers which then results in the beam action in vertical direction.

As a comparison to a spread footing, a 2 foot thick caliche layer at the pile top and tip results in a 30 percent reduction in settlement of the CSP system vs. the spread footing. For the pile group, the data reflects that the greatest settlement reducing benefit is the case of $H_c$ less than $B_g$. 

Figure 5.35 Normalized settlement vs. caliche thickness, 4 pile group model
The analysis of a single CSP with a concentrated load indicates that the beam effect of the upper and lower caliche layers causes an increase in the vertical stress along the entire pile length. The effect of the caliche layers at the pile top and tip results in a vertical stress field that extends about 10 pile diameters from the pile edge (see Figure 5.25). For the case of a pile in soil with no caliche layers, the vertical stress field induced by the load extends laterally about 2.5 diameters from the pile edge.

Figure 5.36 Deformed mesh, 4 feet thick layer, CSP foundation model
Figure 5.37  Settlement contours, 4 feet thick caliche layer, CSP foundation model

Figure 5.38  Vertical deformation contours of caliche layer, CSP model
Figure 5.39  Horizontal tensile stress on bottom caliche plate, CSP model
CHAPTER 6
NUMERICAL BACK ANALYSIS OF FIELD DATA

6.1 Introduction

The purpose in a back analysis procedure is, given an output of a system or problem, to determine a set of model parameters by varying the model parameters where the output is matched when re-introduced to the model. It is required to have a model which calculates outputs (e.g., stresses, strains), and an algorithm that minimizes the error between the observed and measured quantities. Back analysis of geotechnical problems has been performed for a number of different cases such as piled retaining walls (Likar and Vukadin, 2003), tunnels (Fakhimi et al. 2004), rock mass moduli (Hoek and Brown, 1997), and deep excavations (Lee et al. 2004). Pavement stiffness properties are commonly determined from back analysis of falling weight deflectometer data using a layered elastic model for deflections. Small and Zhang (2000) used a back analysis of a pile load test to determine the soil elastic modulus. Reul and Randolph (2003) performed an analysis of building on a pile raft foundation based on a soil elastic modulus determined from a back analysis procedure.

The author routinely uses a back analysis or optimization procedure to determine soil parameters from pile load tests. One such example is the determination of lateral subgrade modulus from pile lateral load tests. From axial load tests, the soil shear modulus may be computed. This procedure allows for design of the most cost effective pile reinforcement.
Radhakrishnan and Leung (1989) determined the mass modulus \( (E_m) \) of relatively soft rock by back analysis of drilled shaft load tests. This compared well with the following empirical correlation with unconfined compressive strength presented by Rowe and Armitage (1984):

\[
E_m = 215 \sqrt{q_u} \text{ (MPa)} \tag{6.1}
\]

There are limitations to inverse analysis, especially for slope failures which often result in unconservative parameter estimates (Deschamps and Yankey, 2006).

The back analysis proceeds as shown in Figure 6.1, and initially the mass modulus of the caliche is determined by back analysis of Pile Load Test TP-3. Using this value of Ec, the stiffness of the soils is determined by back analysis of the test fill embankment. Both of these soil properties are then used in the case study tower model to predict settlements.

6.2 Back Analysis of Soil/Caliche Stiffness

The Mohr Coulomb (MC) model was used for the clayey sand/sandy clay material which is the dominant soil type at a site and extends to a great depth. By using the MC model, a linear increase in stiffness with depth (i.e., confining stress) is accounted for with one set of parameters. Alternatively, one would need to specify a number of layers with depth, each having a constant stiffness that increases with depth. It is worth noting that the Hardening Soil (HS) model could also be implemented for this material since the HS modulus is dependent upon both effective confining stress and strain.
Although the HS model is elasto-plastic and includes several parameters easily determined from conventional laboratory tests such as triaxial and consolidation tests, the current research will utilize the simpler MC model as those data are not available. Additionally, the back-calculation of soil stiffness is more reliable if less variables are required for the soil model. In comparing the results of using both the MC and HS constitutive models for a footing settlement problem, Anderson et al. (2007) found that the MC model performed better than the HS model when the elastic modulus was adjusted for the over-consolidation ratio (OCR).
6.2.1 Soil Profile for Analysis

When performing a back-calculation using manual iterations (direct method), considering that one could be faced with a multiple parameter optimization problem, the simpler the soil profile, the easier it will be to back analyze to determine soils stiffness properties. However, a sufficient amount of soil types must be retained to best match the real soils conditions. Therefore, the general soil profile chosen for analysis may be considered to consist of three to four soil types, of which, one would have variable stiffness/strength parameters.

As described in Chapter 2, the general soil profile along the Las Vegas Strip area consists of granular soil near or at the surface, underlain by inter-layered caliche and fine-grained soil. The grain size analyses presented indicate that these fine grained materials have appreciable amounts of sand and gravel, and are usually classified as a clayey or silty sand. The plasticity index (PI) lab data also indicates that these fine-grained sands, silts and clays may have relatively high degrees of plasticity. Additionally, the natural moisture content of these soils is usually between 20 and 40 percent. For the back analysis procedure, one soil type should consist of the fine-grained of low to moderate plasticity. This will be the dominant material type at the site, and its stiffness parameters will be determined during the back analysis procedure.

Next, the cemented (caliche) materials should be included in the profile since their presence largely affects foundation behavior. The stiffness parameters of this material would be fixed as discussed below in Section 6.3.
Uncemented gravel zones of significant thickness may sometimes be encountered. Therefore, it is simple to include a zone of this material with relatively high stiffness parameters which would be fixed.

A final soil type to be included in the simplified profile should be the high plasticity and high moisture content fine-grained soils. These soils, which tend to be classified as CH materials (based on the Unified Soil Classification System), are not always present at a site so the back analysis profile may often consist only of the two to three materials discussed above. The laboratory data does indicate the presence of the CH soils at the site which occurred within the south end of the building footprint, i.e., they are not continuous across the site. The stiffness parameter of the CH material would be fixed and selected based on correlations with moisture content and plasticity index test results. Thus, from a modeling perspective, we may represent the soil profile at the subject site (and the typical soil profile along the Las Vegas Strip) with four soil types, as follows:

1) Low plasticity and sandy soils (USCS - CL)
2) Caliche or cemented materials
3) Uncemented or partially cemented gravel (USCS - GP/GM/GC)
4) High plasticity clay soils (USCS - CH)

Based on the author’s experience with numerous high rise projects along the Las Vegas Strip, in general, most sites will not include the soft clay material type. A typical model of the simplified analysis profile is shown in Figure 5.1. Layer thicknesses, depths and lateral variability are adjusted according the actual site conditions.
6.2.2 Soil Properties

In the simplified model for back analysis, it is intended to determine the stiffness value(s) (as appropriate for the selected constitutive model) for the low plasticity clay/clayey sand to be used in deformation calculations via a direct trial and error iterative process. Therefore, properties of the remaining three soils types (as applicable to each site/model) need to be established apriori. These soil properties will be determined based on published correlations or from field testing such as a pressuremeter test. The important properties to be established are the stiffness parameters, whereas, the strength parameters are less important for deformation calculations.
6.2.2.1 Gravel

The gravel material encountered in Las Vegas typically consists of sand and gravel with some clay or silt fines. This material is usually defined to be very dense in consistency with SPT N-values exceeding 50 to 100 blows per foot. Thus, for a dense gravel, we can assign nominally high strength parameters and an elastic modulus. Typical values for the elastic modulus are shown in Table 6.1.

Table 6.1 Elastic modulus values for sandy grave from select sources

<table>
<thead>
<tr>
<th>Reference</th>
<th>E (ksf)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Winterkorn and Fang (1975)</td>
<td>2,000 – 4,000</td>
<td>dense sand &amp; gravel</td>
</tr>
<tr>
<td>Bowles (1996)</td>
<td>1,000 – 3,000</td>
<td>sand &amp; gravel, loose</td>
</tr>
<tr>
<td>Bowles (1996)</td>
<td>2,000 – 4,000</td>
<td>sand &amp; gravel, dense</td>
</tr>
<tr>
<td>Das (2004)</td>
<td>1,440 – 3,600</td>
<td>sand &amp; gravel</td>
</tr>
<tr>
<td>Coduto (1994)</td>
<td>1,000 – 2,000</td>
<td>dense sand</td>
</tr>
<tr>
<td>US Army Corps/ASCE (1994)</td>
<td>2,000 – 4,000</td>
<td>dense sand &amp; gravel</td>
</tr>
<tr>
<td>US Army Corps/ASCE (1994)</td>
<td>500 – 2,000</td>
<td>dense sand</td>
</tr>
</tbody>
</table>

Based on these published values, a modulus value ranging from 2,000 to 4,000 ksf would be appropriate, depending upon the estimated consistency (medium dense to very dense). A typical value of 2,000 ksf will be used in this research effort.

Regarding the strength parameters, typical published values of the drained friction angle for a dense gravel are 40+ degrees (Bowles, 1996). Also, Bowles states that a Poisson’s ratio for dense gravels between 0.3 and 0.4 would be suitable. It should be noted that the PLAXIS documentation recommends that granular materials be assigned a minimal cohesion value (such as 100 psf) to avoid numerical stability issues. This would also be appropriate considering that the local soils are often partially cemented.
Because the gravel zones are usually near surface and limited in thickness, it is not necessary to model change in stiffness with depth. Since the model properties are simple to estimate, the Mohr Coulomb (MC) soil model will be used, and the stiffness will be a constant value for this material.

6.2.2.2 Soft Clay

When soft clay soils are to be included in the model, the stiffness properties should be consistent with those determined by the pressuremeter test at the site, or at the site to be modeled, if available. These materials are best identified using plasticity index (PI) data, as previously discussed. The pressuremeter tests yielded a reload modulus value of about 400 ksf in the soft clay.

Considering that the PI for the highly plastic clay ranges between 50 and 70, a friction angle of 25 degrees may be estimated from Figure 4.12. Based on the direct shear tests at the case study site, a maximum effective cohesion of 200 psf would be suitable. As with the gravel material, the MC soil model will be used for this material.

6.2.2.3 Clayey Sand/Sandy Clay

For the low plasticity clayey sand (or sandy clay) material, strength parameters need to be determined for analysis. The stiffness parameters will result from the back-analysis procedure of the test fill. Since this is the dominant soil type at the site, the stiffness should include a variation with stress and strain levels. Thus, the PLAXIS Mohr Coulomb (MC) constitutive model will be used with a linear variation in modulus. Recall from Chapter 3 that the plasticity index (PI) for this material ranges between 10 and 30. And based on the data in Figure 4.12, the trend line would indicate a $\phi'$ of 28 to 32°. Therefore, a $\phi'$ of 30° would be considered suitable.
In addition to the tests from the case study site, numerous direct shear tests were performed on the near surface soils for the Las Vegas Monorail project (Ninyo and Moore, 2001) which spanned several miles along the Las Vegas Strip area. These tests indicate an effective cohesion ranging from 200 to over 800 psf, and effective friction angles of 25° to 38°. Detailed site data from direct shear tests should be used for analyses. If test data is not available, and considering that the soils are often partially cemented, a drained cohesion of 200 psf would be suitable.

6.2.2.4 Caliche

As shown in Chapters 3 and 4, the local caliche material has a compressive strength, unit weight, and compression wave velocity, similar to, and often exceeding, normal strength concrete. Additionally, the NDOT triaxial tests indicate an elastic modulus similar to concrete, based on ACI correlations. In evaluating the modulus of the caliche, one can make use of the expressions relating the UCS to E for concrete.

For the case study site, it should be noted that the upper caliche deposit often includes a 1.5 to 2 foot thick soil layer. Considering the caliche core test results in the upper caliche, if the soil layer is not explicitly included in the model, the stiffness of the upper layer should be reduced using a weighted average approach. This reduced the average UCS of the upper deposit from 8,200 to 6,700 psi, considering caliche thicknesses ranging from 10 to 16 feet. A picture of the two-phase upper caliche is shown in Figure 6.3.
From the field of rock mechanics, the concept of rock deformation modulus is intended to represent the deformation modulus of the rock mass and includes the effects of fractures and joints on a macro scale (e.g., Sabatini et al. 2002). Therefore, the rock elastic modulus to be used for foundation design is reduced from the laboratory or intact core value to account for scale effects and discontinuities in the rock mass. Since the Las Vegas caliche is formed by a sedimentary process and examination of cores does not reveal joints and fractures, per se, in this dissertation, the laboratory measured values are not reduced for the effect of discontinuities. However, it is prudent to consider that caliche deposits are subject to weathering and the thickness is variable in lateral extent (scale effects). Also, the near surface deposits are less weathered than the deeper deposits. Time rate drilling records performed during the case study site exploration show that the deeper caliche deposits are softer than the upper caliche deposit, although,
this is partly due to the drill rod weight effect (KI, 2001). Considering that the elastic modulus values from laboratory measurements or correlations thereof represent an upper bound stiffness, the mass modulus of the caliche to be used in the forthcoming deformation analyses will be based on a back analysis of Test Pile TP-3.

6.2.2.4.1 Elastic Modulus

The following are empirical expressions relating $f_c'$ (unconfined compressive strength of concrete at 28 days) to $E$ for concrete. American Concrete Institute, ACI 318-02, section 8.5; valid for $f_c'\leq6,000$ psi, and unit weight between 90 and 155 pcf (ACI, 2002):

$$Ec = 57000\sqrt{f_c'} \text{ (psi)}$$  \hspace{1cm} (6-1)

ACI 363R, Eq. (5-1), valid for $f_c'$ between 3,000 and 11,600 psi:

$$Ec = 40000\sqrt{f_c'} + 1E6 \text{ (psi)}$$  \hspace{1cm} (6-2)

Precast/Prestressed Concrete Institute (PCI) Design Handbook (PCI, 1999):

$$Ec = [40000\sqrt{f_c'} + 1E6]\left(\frac{w_c}{145}\right)^{1.5} \text{ (psi)}$$  \hspace{1cm} (6-3)

National Cooperative Highway Research Program Report 496 (NCHRP, 2003):

$$Ec = 33000\left(0.140 + \frac{f_c'}{1000}\right)^{1.5} \sqrt{f_c'} \text{ (ksi)}$$  \hspace{1cm} (6-4)
Upper and lower bounds of the NCHRP relation are obtained by applying the factors of 1.224 and 0.777, respectively, to the above equation.

Hughes et al. (2005) evaluated the in-place modulus of elasticity for high strength concrete from concrete cylinders and embedded fiber optic sensors in a prestressed concrete bridge structure. The UCS of the concrete they studied ranged from approximately 7,000 to 10,000 psi. In applying the above relations, they found that the ACI 318 equation over predicts the modulus, and the ACI 363R and PCI equations were the most accurate. All equations overestimated Ec for strengths above 9,000 psi, with the NCHRP lower range performing the best in that range.

ACI 363R, the committee report on high strength concrete, indicates that the PCI equation gives reasonable results over a wide range of strengths. The report further shows that the well known ACI 318 equation overestimates Ec for strengths above 6,000 psi. Irvani (1996) also shows that the ACI 318 expression over predicts the modulus for high strength concrete. Based on the data shown in the ACI 363R report, the PCI equation appears to be the best over a wide range of concrete strengths. A summary of the average UCS and E values using the above relations from the sites previously discussed is shown in Table 6.2.

Considering that triaxial tests have been performed on caliche samples from the NDOT site, it is worthwhile to compare the modulus measurements to the empirical equations. Note that these are the only known triaxial data for caliche from Las Vegas.
Using the method by Johnston (1985), which was applied to concrete testing by Anoglu et al. (2006), the effect of the triaxial test confining stress on the unconfined strength may be evaluated. As shown in Table 6.3, the confining effect is minimal for a confining stress of 1 atm.

To further compare the empirical equations with actual laboratory results, the $E-f'$c relationships were applied to test results from Pincus (1996) on Salem limestone, a similar rock. Pincus summarized the results of an inter-laboratory test program to determine variability parameters for certain test methods. Salem limestone is a calcite cemented limestone with fossil fragments which is commercially mined in Indiana. The commercial grade stone is called Bedford limestone.

For additional comparison, listed in Table 6.3 are UCS and elastic modulus properties of Braden breccia, a cemented breccia from a mine in Chile (Hoek, 1997). The breccia rock is similar in texture to a conglomerate, not unlike the cemented sand and gravel caliche deposits in Las Vegas which have gravel and cobble inclusions. The breccia rock is characterized by Hoek as a “massive weak rock” with very few joints and
similar to weak concrete. Both the Salem limestone and Braden breccia have similar constituents and strengths compared to caliche.

Table 6.3  Caliche & rock elastic modulus laboratory data compared to empirical equations

<table>
<thead>
<tr>
<th>Sample</th>
<th>Density,pcf</th>
<th>UCS,ksi</th>
<th>E, ksf</th>
<th>E, ksf</th>
<th>E, ksf</th>
<th>E, ksf</th>
</tr>
</thead>
<tbody>
<tr>
<td>NDOT B1</td>
<td>159.5</td>
<td>7,860</td>
<td>7,791</td>
<td>687,000</td>
<td>652,427</td>
<td>752,699</td>
</tr>
<tr>
<td>NDOT B5</td>
<td>156.4</td>
<td>8,400</td>
<td>8,331</td>
<td>620,000</td>
<td>669,731</td>
<td>750,246</td>
</tr>
<tr>
<td>NDOT B5</td>
<td>159.3</td>
<td>10,645</td>
<td>10,573</td>
<td>418,000</td>
<td>736,283</td>
<td>847,845</td>
</tr>
<tr>
<td>Salem LS</td>
<td>155.0^</td>
<td>NA</td>
<td>7,900</td>
<td>687,000*</td>
<td>655,960</td>
<td>724,975</td>
</tr>
<tr>
<td>Braden Breccia</td>
<td>155.0^</td>
<td>NA</td>
<td>7,400</td>
<td>625,000</td>
<td>639,494</td>
<td>706,776</td>
</tr>
</tbody>
</table>

* tangent E @ 25% of UCS
^ assumed values

The elastic modulus test result for the two samples from NDOT Boring 5 appears to be low comparing the UCS values to the sample from Boring 1. Additionally, for the two rock samples, the modulus values agree well with the ACI 363R equation. The data from Table 6.3 is plotted as shown in Figure 6.4. As seen in the plot, the PCI relationship and NCHRP lower bound relationships act as upper and lower bounds on the laboratory data, and the ACI relationship tends to represent the average values. The low value from the NDOT triaxial tests is not well represented by the empirical equations.

Goodman (1980) reports E/UCS values for various rock types. The average ratio for four samples of limestone (except the Solenhofen limestone for which the ratio is half the other limestone) and dolomite is 550, and using the ACI 363R relation, the average ratio is 560. This confirms that using the ACI 363R equation for caliche gives stiffness
values which are consistent with other published values for rocks. Therefore, the local caliche can be considered to behave like a typical limestone rock type.

Figure 6.4 Comparison of laboratory data vs. empirical estimates for concrete

Also listed in the Table 6.1 of Goodman’s book are Poisson’s ratio values. The average value for the discussed rock types is 0.30. This is consistent with the values measured in the NDOT triaxial tests which were 0.33 and 0.31 (the first test yielded 0.17, but one lateral strain gage malfunctioned).

Based on the comparison with laboratory test results above, the ACI 363R estimate appears to give the most reasonable elastic modulus estimate for intact high strength caliche, and a scale factor should be applied to determine a mass modulus for use in deformation and numerical analyses. The mass modulus for the subject tower site will
be determined by back analysis of a pile load test. This particular load test (Test No. 3) was performed on a slender pile embedded in the upper caliche layer.

6.2.2.4.2 Tensile Strength

The tensile strength of a rock core is commonly determined by performing a splitting tension test (or Brazilian tension test), as defined by the standard ASTM D3967. This test method is preferred over a direct uniaxial tension test because the latter is more difficult to perform, and the location of the tension failure plane cannot be controlled. While the splitting tensile strength is not the same as the tensile strength from a uniaxial tension test, it is a closer representation of the tension related failure seen in concrete structures (Wang and Salmon, 1985). An examination of various test results in the literature by Popovics (1998) indicates that the direct tensile strength (uniaxial), on average, is about 75 percent of the splitting tensile strength. Additionally, the flexural strength (modulus of rupture) is about 140 percent of the splitting tensile strength. It is generally accepted that cracking of a concrete slab on grade (or a caliche layer) in flexure is controlled by the modulus of rupture. Since caliche is a formed by sedimentary processes and may be highly variable in strength and uniformity, for the analyses in this dissertation, the tensile strength in bending will be represented by the more conservative splitting tensile strength.

As presented in Chapter 4, splitting tension tests were performed on caliche samples from the Fremont Street Experience site. In evaluating the tensile strength, we can examine empirical relationships which are dependent upon the UCS. Some of these are listed below (f'c in psi).
ACI 318-99:

\[ f_{sp} = 6.7 \sqrt{f'_c} \]  \hspace{1cm} (6-5)

ACI 363R:

\[ f_{sp} = 7.4 \sqrt{f'_c} \]  \hspace{1cm} (6-6)

The average UCS for the Fremont Street site is 5,000 psi, and the average splitting tensile strength is 68 ksf (0.47 ksi). This value is exactly what the ACI 318 equation predicts. Thus, the strength relationships applicable to concrete also appear suitable for caliche.

Anoglu et al. (2006) studied the relationship between splitting tensile strength and UCS for a wide range of concrete strength (580 to 17,400 psi). They found that the ACI equation underestimates the splitting tensile strength for concrete above 5,800 psi, and presented a relationship which provides satisfactory tensile strength estimates for a wide range of strengths and cement types (f’c in MPa):

\[ \frac{f_{sp}}{f'_c} = 0.387(f'_c)^{-0.370} \]  \hspace{1cm} (6-7)

A comparison of the empirical equations to the measured data is shown in Figure 6.5. There is no single correlation that works best in this case due to the scatter in the lab data. The laboratory data forms lower and upper bounds on the empirical relationships as shown on the plot, and as defined by the fitted equations. The splitting tensile strength for one laboratory data point is exactly estimated by both the ACI 363 and Anuglo relationships.
Figure 6.5  Splitting tensile strength data from Fremont Street site compared to empirical equations.

Low test results from the splitting tension test may result if the sample fails in compression on the ends before it fails in tension in the middle of the sample. For that reason, it is recommended that a cushion be used to reduce the contact stress. A cardboard strip was used as a cushion for these in the laboratory. Although there is scatter in the laboratory results, the three empirical equations have been shown to perform satisfactorily elsewhere for concrete. However, as previously mentioned, the average of all test results (UCS and splitting tension) follows the ACI 363R and Anoglu relationships. For the computations in this dissertation, the intact tensile strength will be estimated using the Anoglu or ACI 363R equations and the mass modulus factor will be applied for use in settlement computations.
6.2.2.4.3 Shear Strength (Cohesion)

For the isolated caliche layers, which are relatively thin compared to the entire model extent, the variation of stiffness with stress level will be insignificant and a constant stiffness value may be used with the MC soil model. However, an appropriate maximum tensile strength above and cut off criterion will be implemented (see Figure 6.5 below).

The cohesion of the caliche (and friction angle) can be determined from the NDOT triaxial tests to get a range of typical values. Due to the elastic-perfectly plastic, stress strain behavior of rock and its simple formulation, rock strength in terms of principal stresses is commonly evaluated using the Mohr-Coulomb (MC) criterion. As shown by Goodman, the failure of rock materials may be represented by the Mohr Coulomb criterion, and the straight failure line may also be replaced by a curved failure surface.

The MC failure criterion may be expressed as:

\[ \tau = c + \sigma_n \tan(\phi) \]  

(6.8)

where:

\( \tau \) = shear strength,
\( c \) = cohesion,
\( \sigma_n \) = normal stress, and
\( \phi \) = friction angle.
The MC relationships in terms of effective principal stresses at failure are:

\[
\sigma_{nf} = \left(\frac{\sigma_1' + \sigma_3'}{2}\right) - \left(\frac{\sigma_1' - \sigma_3'}{2}\right)\sin(\phi') 
\]

(6.9)

\[
\tau_f = \left(\frac{\sigma_1' - \sigma_3'}{2}\right)\cos(\phi')
\]

(6.10)

Note that the MC criterion is not dependent upon the intermediate principal stress \(\sigma_2\). Dropping the prime ('), rearranging and recognizing that \(\sigma_3 = 0\) for unconfined compression (UCS or qu or \(\sigma_c\)) tests:

\[
q_u = 2c\tan(45 + \frac{\phi}{2})
\]

(6.11)

and therefore,

\[
\sigma_{1f} = q_u + \sigma_3 \tan^2(45 + \frac{\phi}{2})
\]

(6.12)

which may be written as:

\[
\sigma_{1f} = q_u + k\sigma_3
\]

(6.13)

where:

\[
k = \tan^2(45 + \frac{\phi}{2}) = \frac{1 + \sin(\phi)}{1 - \sin(\phi)}
\]

(6.14)

This is a straight line in principal stress space as shown in Figure 6.6.
From the NDOT triaxial data on caliche core samples, and the above formulation, we can estimate the cohesion and $\phi$ as shown in Table 6.4. Also shown are the cohesion values using the McVay relationship, while estimating the splitting tensile strength using the Anuglo relationship (Eq. 6-7). Additionally, as stated in Chapter 3, McVay suggests the shear strength of rock is nominally the same as the ultimate bond strength. Recall that McVay’s work suggests that Equation 3.2 for ultimate bond strength (or cohesion). The cohesion calculated by this method is also shown in the table, along with the average of the values from both methods. As shown in the table, the average cohesion from the triaxial data and the McVay method are reasonably close.
Table 6.4  Cohesion and friction angle from NDOT triaxial data

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>$\sigma_1$, ksi</th>
<th>$\sigma_3$, ksi</th>
<th>k</th>
<th>$\phi$</th>
<th>c, ksf</th>
<th>UCS/c</th>
<th>To, ksf</th>
<th>McVay c, ksf</th>
<th>c, avg</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>7.87</td>
<td>0.014</td>
<td>5.9</td>
<td>45.3</td>
<td>231</td>
<td>4.86</td>
<td>119</td>
<td>183</td>
<td>207</td>
</tr>
<tr>
<td>B-5</td>
<td>8.41</td>
<td>0.014</td>
<td>5.9</td>
<td>45.4</td>
<td>246</td>
<td>4.88</td>
<td>127</td>
<td>195</td>
<td>221</td>
</tr>
<tr>
<td>B-5</td>
<td>10.66</td>
<td>0.014</td>
<td>6.1</td>
<td>46.0</td>
<td>308</td>
<td>4.95</td>
<td>160</td>
<td>247</td>
<td>277</td>
</tr>
<tr>
<td>AVG</td>
<td>9.0</td>
<td>0.014</td>
<td>6.0</td>
<td>45.5</td>
<td>262</td>
<td>4.9</td>
<td>135</td>
<td>208</td>
<td>235</td>
</tr>
</tbody>
</table>

For the splitting tension test, Cook shows data indicating the ratio \( \frac{UCS}{f_{sp}} \) varies from 10 to 16. Using a theoretical relationship from Jaeger and Cook (1976), the ratio \( \frac{UCS}{f_{sp}} \) may be shown to be 12. Based on this relationship and the McVay equation, the cohesion could be estimated to be 1.7 times the tensile strength. However, considering the above computations which are based on actual triaxial tests in caliche with UCS values similar to those at the subject tower site, the cohesion of the caliche will be assumed to be 230 ksf. This value will be reduced to account for scale effects based on the derived mass elastic modulus.

6.2.2.4.4 Friction Angle

As discussed in Chapter 4, the caliche core data from the Fremont Street site indicates an average friction angle \( (\phi) \) of 35° to 38°. Additionally, for limestone and dolomite rocks, Goodman reports \( \phi \) ranges of 35° to 42°, respectively. Jumikis (1983) indicates the friction angle of limestone varies between 35° and 50°. In the Federal Highway Association (FHWA) document on subsurface investigations, Mayne et al. (2001) reports friction angles of 32° to 49° for porous limestone. For basalt, limestone, granite and conglomerate rocks, Sabatini et al. (2002) lists friction angles of 34° to 40°.

The NDOT triaxial test data and the UCS test data on caliche allows one to
calculate $\phi$ as previously indicated. Considering the NDOT triaxial data as being the most representative of caliche, a friction angle of 45° will be suitable for the numerical modeling computations.

6.2.3 Summary

A summary of the soil parameters to be used for the back analysis is shown in Table 6.5. Based on the average UCS of 8,200 psi at the case study site, and using the ACI relationship since it is more conservative than the Anoglu correlation, a $f_{\text{up}}(\text{To})$ of 85 ksf is selected. It is reasonable to be more conservative regarding the caliche tensile strength due to the highly variable nature of the deposit.

Table 6.5 Summary of fixed soil parameters for to be used for back analysis

<table>
<thead>
<tr>
<th>Soil No.</th>
<th>Soil Type</th>
<th>$\Phi'$ ($^\circ$)</th>
<th>$c'$ (ksf)</th>
<th>$E$, ksf</th>
<th>$\nu$</th>
<th>$\text{To}$, ksf</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>sandy clay</td>
<td>30</td>
<td>0.2</td>
<td>variable</td>
<td>0.4</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>caliche</td>
<td>45</td>
<td>230</td>
<td>variable</td>
<td>0.3</td>
<td>85</td>
</tr>
<tr>
<td>3</td>
<td>gravel</td>
<td>40+</td>
<td>0.1</td>
<td>2,000</td>
<td>0.3</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>soft clay</td>
<td>26</td>
<td>0.1</td>
<td>300</td>
<td>0.4</td>
<td>0</td>
</tr>
</tbody>
</table>

Each property will be varied to meet the site conditions as appropriate. It should be recognized that in the numerical analysis of structures in rock, in addition to the elastic modulus, the strength properties of the rock mass are also adjusted to account for discontinuities, fractures, etc., and scale effects (Hoek et al. 1997). Table 6.6 shows reduction factors for pile friction in rock based on the rock mass modulus. In the table, $E_m$=mass modulus, $E_i$=intact modulus, $f_{\text{sm}}$=reduced friction value, and $f_s$=friction value.

Following the determination of the mass modulus ratio from the back calculation of Test
Pile TP-3, the caliche strength properties will be adjusted as indicated in Table 6.6. Although this table is considered applicable to bored pile friction capacity in rock, it is thought to be useful in regard to strength properties in general.

Table 6.6 Side resistance reduction based on rock mass modulus ratio (after O’Neill and Reese, 1999)

<table>
<thead>
<tr>
<th>Em/Ei</th>
<th>fsm/fs</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>0.5</td>
<td>0.8</td>
</tr>
<tr>
<td>0.3</td>
<td>0.7</td>
</tr>
<tr>
<td>0.1</td>
<td>0.55</td>
</tr>
<tr>
<td>0.05</td>
<td>0.45</td>
</tr>
</tbody>
</table>

6.3 Pile Load Test TP-3

Pile load test TP-3 (see section 4.4.3.2) was back analyzed to determine the elastic mass modulus (Ecm) of the upper caliche material at the subject site. The analyses were performed using both the PLAXIS 2D and 3D Foundation programs. The purpose of the 3D analysis is to examine the effect of the reaction piles on the pile deflection during the test, and to utilize that analysis if a significant difference exists.

As previously stated, the test pile was installed through a thick upper caliche layer to test the ultimate geotechnical capacity of the caliche. The test pile did not reach geotechnical failure and the load deflection data was basically elastic, with very small net displacement (0.025”). The test was very similar to a pull-out test in rock to determine pile ultimate friction or rock anchor capacity, and the pile response in rock is essentially elastic until a shaft friction failure occurs either by failing the rock material or the concrete. As shown in Section 6.3.2.3, the response of the pile is dominated by the
stiffness of the caliche. As installed at the site, the test and reaction pile layout is shown in Figure 6.7.

![Diagram of test pile TP-3 reaction pile layout](image)

**Figure 6.7 Test pile TP-3 reaction pile layout**

The soil layer and constitutive model parameters used to determine Ecm are indicated in Table 6.7 below. Soil stratigraphy is based on WTI Boring No. TP-1 (see Appendix E) performed within 20 feet of the test pile location, and the pile drilling time record for the test pile (Figure 6.8). Note that the upper 6.8 feet of the pile was encased in a greased aluminum can placed after pile construction while the concrete was wet. It was the intention to case the upper soils above the caliche to maximize the load transfer to the upper caliche layer. The boring performed for the load test area encountered cemented sand and gravel between 6.5 and 9 feet. To account for the presence of this cemented material, the thickness of the caliche layer was increased by 1 foot.
Table 6.7 Soil profile data for TP-3 back analysis

<table>
<thead>
<tr>
<th>Depth, ft.</th>
<th>Soil Type</th>
<th>Constitutive Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 4</td>
<td>sandy gravel</td>
<td>MC</td>
</tr>
<tr>
<td>4 - 8</td>
<td>sandy gravel</td>
<td>MC</td>
</tr>
<tr>
<td>8 - 14</td>
<td>caliche</td>
<td>MC</td>
</tr>
<tr>
<td>14 - 16</td>
<td>sandy gravel</td>
<td>MC</td>
</tr>
<tr>
<td>16 - 19</td>
<td>caliche</td>
<td>MC</td>
</tr>
<tr>
<td>19 - 40</td>
<td>sandy clay</td>
<td>MC</td>
</tr>
<tr>
<td>40 - 47</td>
<td>caliche</td>
<td>MC</td>
</tr>
<tr>
<td>47 - 51</td>
<td>sandy clay</td>
<td>MC</td>
</tr>
<tr>
<td>51 - 56</td>
<td>soft clay</td>
<td>MC</td>
</tr>
<tr>
<td>56 - 67</td>
<td>sandy clay</td>
<td>MC</td>
</tr>
<tr>
<td>67 - 70</td>
<td>soft clay</td>
<td>MC</td>
</tr>
<tr>
<td>70 - 150</td>
<td>sandy clay</td>
<td>MC</td>
</tr>
</tbody>
</table>

Figure 6.8 Test pile TP-3 drill time (DT) record (WTI, 2002b)
6.3.1 3D Analysis

The 3D analysis was performed before the 2D analysis to determine if there was any effect of the reaction piles. Given the stiffness and beam effect nature of the caliche, it was presumed that the reaction piles would significantly affect the test pile deflection. As shown in the following section, the 3D analysis with the reaction piles provides a more accurate model vs. the 2D analysis.

6.3.1.1 Model Parameters

The soil profile previously detailed was used in the 3D model. Also, the embedded pile structural element in PLAXIS 3D was used for both the test and reaction piles, with properties indicated in Table 6.8. Material properties for the embedded piles are shown in Table 6.8. In plan view, the model had a width of 200 feet in each direction, i.e., model boundaries were 100 feet from the test pile. The 3D Model is shown in Figure 6.9. Ultimate skin friction loads for the embedded piles are indicated in Table 6.9.

Table 6.8 Material parameters for embedded piles, TP-3 back analysis

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Installed Length, ft</th>
<th>Diameter, ft</th>
<th>Ep, ksf</th>
<th>v</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Pile</td>
<td>23*</td>
<td>2.0</td>
<td>7.9E5</td>
<td>0.15</td>
</tr>
<tr>
<td>Reaction Pile</td>
<td>32</td>
<td>2.0</td>
<td>7.9E5</td>
<td>0.15</td>
</tr>
</tbody>
</table>

* pile chipped down 4 ft prior to testing
Figure 6.9  TP-3  3D mesh for back analysis, plan view

Figure 6.10  TP-3  3D mesh for back analysis, side view
Figure 6.11 Structural elements for TP-3 back analysis, 3D model

Table 6.9 Embedded pile-soil properties

<table>
<thead>
<tr>
<th>Depth, ft.</th>
<th>Soil Type</th>
<th>fsu, ksf</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 4</td>
<td>sandy gravel</td>
<td>10</td>
</tr>
<tr>
<td>4 - 9</td>
<td>sandy gravel</td>
<td>10</td>
</tr>
<tr>
<td>9 - 14</td>
<td>caliche</td>
<td>130</td>
</tr>
<tr>
<td>14 - 16</td>
<td>sandy gravel</td>
<td>20</td>
</tr>
<tr>
<td>16 - 19</td>
<td>caliche</td>
<td>130</td>
</tr>
<tr>
<td>19 - 32</td>
<td>sandy clay</td>
<td>7</td>
</tr>
</tbody>
</table>
6.3.1.2 Mesh Refinement

For the 3D model, the effect of mesh refinement on pile cap deflection was examined as indicated in Table 6.10. All deflections shown are for a test load of 3,000 kips, for which the field test data indicated a deflection of the pile cap of 0.163 inches. The data indicates the calculated deflection is not very sensitive to mesh effects. This may be due to the additional nodes and integration points introduced by the use of an embedded pile. Mesh type D, as shown in Figure 6.14, was used for the calculations in the section. Mesh types A and C are shown in Figures 6.12 and 6.13.

Table 6.10 3D mesh refinement results for TP-3 model

<table>
<thead>
<tr>
<th>Mesh Type</th>
<th>No. 3D Elements</th>
<th>Test Pile Cap deflection, in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>3588</td>
<td>0.161</td>
</tr>
<tr>
<td>B</td>
<td>4752</td>
<td>0.161</td>
</tr>
<tr>
<td>C</td>
<td>6534</td>
<td>0.162</td>
</tr>
<tr>
<td>D</td>
<td>8228</td>
<td>0.162</td>
</tr>
<tr>
<td>E</td>
<td>10676</td>
<td>0.162</td>
</tr>
</tbody>
</table>

6.3.1.3 Sensitivity Analysis

Due to the increase in computation effort in performing a 3D analysis, a sensitivity analysis was not performed. However, the results of the sensitivity evaluation performed on a single pile in a 2D axisymmetric mesh can be considered relevant to the 3D problem. As shown in section 5.3.4.5, the solution to the axially loaded pile in a soil profile with stiff inclusions is most sensitive to the stiffness of the caliche vs. the soil.
Figure 6.12 Mesh refinement study for TP-3 back analysis, mesh type A

Figure 6.13 Mesh refinement study for TP-3 back analysis, mesh type C
6.3.1.4 Results

For a maximum load of 3,625 kips, the deformation contours of the model are shown in Figure 6.14. Note that the displacement at the model boundaries is approximately zero, indicating that the model dimensions are suitable. The results of the 3D analysis are indicated in Figure 6.15. Mesh type D with 8,228 elements (see Table 6.10) as shown in Figure 6.14 was used for the calculations.

The 3D analysis achieved a very good match to the data using the above parameters with elastic modulus of the caliche (Ec) of 280,000 ksf. The data match is less accurate for the high load range above 3,000 kips, where some nonlinearity or pile slippage occurred.

![Deflection contours for TP-3 back analysis model](image)

Figure 6.14  Deflection contours for TP-3 back analysis model
The data match up to 3,000 kips is considered acceptable because the important data range for back calculation is 1,500 to 3,000 kips where the load deflection plot is linear or the pile behavior is primarily elastic. There is some plastic deflection as the net deflection at zero load is not zero. Note that the initial part of the field data curve indicates some "seating" behavior, such that the incremental displacement per load increment decreases with load. This is also evident in the unit shear vs. displacement data in Figure 4.42.

Figure 6.15 3D model back analysis results for test pile TP-3
A comparison of the 3D cases with (case A) and without reaction piles (case B) is further evaluated in terms of displacement fields. Figures 6.16 and 6.17 show the displacement fields for the case with and without reaction piles, respectively. The displacement field is much smaller for the case with reaction piles, since the tension loaded piles surrounding the compressively loaded test pile effectively dampen the displacement field. For case B, the displacement field extends nearly to the model limits, as observed in a 2D model.

The x-direction tensile stresses in the caliche layers for each case are shown in Figures 6.18 and 6.19. For the case with reaction piles (case A), the stresses are more localized to the test pile compared to the case without reaction piles (case B). Note that the legends are the same for both figures and at the location of the test pile, the values of tensile stress are essentially the same.

Compressive stress contours in the vertical direction for each case are shown in Figures 6.20 and 6.21. As indicated, the vertical stresses for case A show minor changes when compared to case B. Due to beam action induced by the presence of the caliche layers in case B, there are noticeable increases in vertical stress to a depth of nearly 140 feet.

Based on the above results, the 2D analysis is not suitable for the back analysis of a top loaded pile load test in a soil/caliche profile due to the pile to pile interaction effects. Based on the 3D model analysis, the interaction effects between adjacent piles is greater in soil profiles with caliche layers. The top loaded load test is best simulated using a 3D model that includes all piles installed and actual loads applied.
Figure 6.16 Deflection contours for case with reaction piles, test pile TP-3

Figure 6.17 Deflection contours for case with no reaction piles, test pile TP-3
Figure 6.18  Tensile stresses in the x-direction for case A

Figure 6.19  Tensile stresses in the x-direction for case B
Figure 6.20  Vertical compressive stresses for case A

Figure 6.21  Vertical compressive stresses for case B
6.3.2 2D Analysis

Numerical modeling of piles is commonly performed with 2D axisymmetric models (e.g., Randolph and Wroth, 1978; Desai, 1974; Prakoso, 1999; Leong and Randolph, 1994; Fellenius et al. 1999). To evaluate the accuracy of the 3D model for the pile problem (with no reaction piles), a 2D axisymmetric model was developed based on the 3D model. Soil layer depths and properties and pile properties were identical to the 3D model. The pile was simulated by using volume elements vs. the embedded pile in the 3D model. Interfaces were activated in the soil clusters, but not in the caliche zones. Regarding interface use for concrete piles in rock, McVay et al. (1992) states that interfaces are not required since the failure is usually within the rock itself. Significant rock on the edges of the pullout samples has been observed.

6.3.2.1 Model Parameters

The 2D axisymmetric model has dimensions of 120 feet wide x 150 feet in depth. Model details of the pile cap and pile are shown in Figure 6.2. Material and soil properties are shown in Tables 6.11 and 6.12, respectively. A steel load cap was used on the top of the pile cap to minimize stress concentrations.
Figure 6.22 2D model pile cap details for TP-3 back analysis

Table 6.11 2D back analysis linearly elastic model parameters

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<td>γ_sat [klb/ft³]</td>
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Table 6.12 2D back analysis model soil parameters

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<th>2 Sand &amp; gravel</th>
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<th>4 Stiff Clay</th>
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<td>Drained</td>
<td>UnDrained</td>
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6.3.2.2 Mesh Refinement

To evaluate the sensitivity of the finite element results to the mesh configuration, the reference displacement of the model for a pile load of 3000 kips was evaluated for different levels of mesh refinement. Mesh sizes ranged from 323 to 3,429 elements (2,752 to 27,856 nodes). Based on the mesh refinement study, there is only a slight increase in deflection for meshes finer than mesh 1. Mesh 5 has twice the number of nodes compared to mesh 4, but the pile deflection was only 0.001 inches greater. Mesh 4 was used for the pile calculations. The range of mesh sizes are shown in Figures 6.23 and 6.24, and the results are shown in Table 6.13.
Table 6.13 2D Mesh refinement results for TP-3 back analysis

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Figure 6.23 2D Mesh refinement results for TP-3 back analysis, mesh 2
Figure 6.24 2D Mesh refinement results for TP-3 back analysis, mesh 4

6.3.2.3 Sensitivity Analysis

To determine the relative effect of caliche and soil strength and stiffness parameters, a sensitivity analysis was performed by varying the stiffness of both materials. This is an option within PLAXIS that allows any model parameter to be varied and the effect on deflection (or stress, for instance) to be realized at any point in the model. Model parameters which were varied are shown in Figure 6.25. The observed deformation at the pile top was sensitive to changes in the sandy clay cohesion and Poisson’s ratio, but to a much larger extent, the stiffness of the caliche. The sensitivity score (SS), as defined in the PLAXIS reference manual, is the sensitivity ratio (SR, change in function value/change in input variable) weighted by the ratio of the input range to the reference value, as follows:
\[ SR = \frac{(f(x^*) - f(x)) / f(x)}{(x^* - x) / x} \]  

(6.14)

and,

\[ SS = SR \frac{(\max(x^*) - \min(x^*))}{x} \]  

(6.15)

where \( x \) = parameter to be varied,

\( x^* \) = value of varied parameter.

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<th>caliche upper psi</th>
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Figure 6.25  Parameters for sensitivity analysis of 2D model, TP-3 back analysis

The plot of the total relative sensitivity is shown in Figure 6.26. This represents the sum of the sensitivity scores for each variable divided by the sum of all the sensitivity scores. As evidenced in the plot, the deflection of the pile is much more sensitive to the elastic modulus of the caliche than other parameters. Based on this result, the stiffness of the caliche is best determined from the back analysis of the subject pile load test.
6.3.3 Pile Load Distribution

As mentioned in Chapter 4, the load test piles at the case study site were instrumented with strain gages to determine the load distribution with depth. The pile load distribution for test pile TP-3 will be analyzed in this section using the 2D and 3D models discussed above. The load distribution data based on the strain gage data for test pile TP-3 is shown in Figure 6.27.
For the 3D model, the use of embedded piles allows the pile forces with depth as a direct output from the analysis. In the 2D model, a structural plate element must be added to the pile and then calculate pile forces based on the provided plate axial forces. The plate element is added to the axisymmetric model such that it represents a tube with a diameter of 1 foot (recall the pile has a diameter of 2 feet).

The field data compared to the load distribution data from the 2D and 3D models is shown in Figure 6.28. The data agrees well except for the upper layer (0 to 6 feet) where the load based on the field data indicates approximately 1,000 kips of load distribution above the upper caliche layer. This may be due to the presence of the pile cap in the upper 4 feet and the casing, and that the soils between approximately 5 and 9
feet are partially cemented sand and gravel. Therefore, it is reasonable to expect some load transfer in the zone. However, approximately 1,000 kips was transferred in the upper layer which seems excessive. As indicated in the plot, the load distribution predicted by the 2D and 3D numerical models agrees well.

Figure 6.28 Load distribution data predicted by 2D & 3D models compared to field test data

6.3.4 Results

Figure 6.29 below shows the comparison between the 3D model results with no reaction piles activated and the 2D model results, as well as, the 3D model with the reaction piles. The response of the 2D and 3D models are in excellent agreement which verifies that the 3D model is also suitable for modeling piles.
Vertical displacement contours are shown in Figure 6.30 for a load of 3,000 kips. The deflection is approximately zero at the upper edge of the model, as observed in the 3D model (case B).

Figure 6.29  Effect of reaction piles on test pile model for TP-3 back analysis
In conclusion, when modeling a conventional top-down load test in a soil/caliche profile, a 3D model is required so the effect of reaction piles may be included. Piles are commonly modeled using axisymmetric FEM models. Inclusion of reaction piles in an axisymmetric 2D model would actually model a continuous ring of piles similar to a large pipe pile.
6.4 Case Study Site Test Fill

As discussed in Section 4.4.4, a 30 foot high test embankment was constructed at the site to aid in determining parameters to be used for the settlement analysis of the man-made mountain structure at the site. Based on the back analysis of pile load test TP-3, the elastic modulus of the caliche (Ec) was used in the 2D PLAXIS model of the test embankment to determine the stiffness properties of the sandy clay soil at the site. A 2D axisymmetric model was used to back analyze the test embankment.

6.4.1 Model Parameters

As indicated in Section 4.4.4, the test fill embankment was approximately 200 feet square in plan dimension at the top, and 30 feet in height. Using total embankment load for the trapezoidal shaped embankment as the controlling parameter, radial dimensions of 76 and 112 feet at the top and bottom, respectively, were calculated for a truncated cone of equivalent weight.

Model dimensions, soil layers and parameters are indicated in Figure 6.31 and Tables 6.14 and 6.15. Soil stratigraphy is based on a 200 foot boring performed for the man made mountain project (WTI, 2003c), located at the west side of the site. The boring log is included in Appendix F.
Figure 6.31  2D test fill model details

6.4.2 Mesh Refinement

To evaluate the sensitivity of the finite element results to mesh configuration, the reference displacement of the model (1 foot below the ground surface) was evaluated for different levels of mesh refinement. Mesh sizes ranged from 415 to 4,019 elements (3,491 to 32,455 nodes). Based on the mesh sensitivity study, the model showed essentially no sensitivity (for deflection to the nearest 0.001 foot) to the fineness of the mesh. This is due, in part, to the use of 15 node triangular elements which have improved accuracy over 6 node elements. The range of mesh sizes is shown in Figures 6.32 and 6.33 and the results are shown in Table 6.16.
Table 6.14 Soil profile for test fill back analysis

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Table 6.15 Soil parameters for test fill back analysis

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<th>2 Sand &amp; gravel</th>
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</tr>
<tr>
<td>γ&lt;sub&gt;unsat&lt;/sub&gt; [klb/ft³]</td>
<td>0.16</td>
<td>0.12</td>
<td>0.13</td>
<td>0.13</td>
</tr>
<tr>
<td>γ&lt;sub&gt;sat&lt;/sub&gt; [klb/ft³]</td>
<td>0.16</td>
<td>0.13</td>
<td>0.13</td>
<td>0.13</td>
</tr>
<tr>
<td>E&lt;sub&gt;ref&lt;/sub&gt; [klb/ft²]</td>
<td>280,000</td>
<td>1,200</td>
<td>3,000</td>
<td>1,000</td>
</tr>
<tr>
<td>ν [-]</td>
<td>0.30</td>
<td>0.30</td>
<td>0.35</td>
<td>0.40</td>
</tr>
<tr>
<td>c&lt;sub&gt;ref&lt;/sub&gt; [klb/ft²]</td>
<td>230.0</td>
<td>0.10</td>
<td>4.5</td>
<td>0.20</td>
</tr>
<tr>
<td>φ [°]</td>
<td>45.0</td>
<td>40.0</td>
<td>44.0</td>
<td>30.0</td>
</tr>
<tr>
<td>ψ [°]</td>
<td>10.0</td>
<td>10.0</td>
<td>2.0</td>
<td>0.0</td>
</tr>
<tr>
<td>E&lt;sub&gt;inc&lt;/sub&gt; [klb/ft²/ft]</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>12.0</td>
</tr>
<tr>
<td>γ&lt;sub&gt;ref&lt;/sub&gt; [ft]</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>-16.0</td>
</tr>
<tr>
<td>c&lt;sub&gt;increment&lt;/sub&gt; [klb/ft²/ft]</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>T&lt;sub&gt;str&lt;/sub&gt; [klb/ft²]</td>
<td>85.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>R&lt;sub&gt;inter&lt;/sub&gt; [-]</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>0.8</td>
</tr>
</tbody>
</table>

190
Table 6.16  Mesh refinement results for test fill model

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>415</td>
<td>3491</td>
<td>0.150</td>
</tr>
<tr>
<td>2</td>
<td>2715</td>
<td>22047</td>
<td>0.150</td>
</tr>
<tr>
<td>3</td>
<td>4019</td>
<td>32455</td>
<td>0.150</td>
</tr>
</tbody>
</table>

Figure 6.32  Test fill model mesh refinement, mesh 1 with 415 elements
6.4.3 Sensitivity Analysis

To determine the relative effect of caliche and soil stiffness values, a sensitivity analysis was performed while varying the stiffness of both materials. This is an option within PLAXIS that allows any model parameter to be varied and the effect on deflection (or stress, for instance), to be realized, at any point in the model. Model parameters which were varied are shown in Figure 6.34. The observed deformation at 1 foot below the ground surface was not sensitive to changes in any other soil parameters.
Figure 6.3 Sensitivity score for test fill model

The plot of the total relative sensitivity is shown in Figure 6.35. As evidenced in the plot, the deflection of the embankment is much more sensitive to the elastic modulus of the soil vs. the modulus of caliche. Based on this result, the stiffness of the soil determined from the back analysis is not very sensitive to the caliche stiffness used in the model.

Figure 6.34 Total relative sensitivity for test fill model

<table>
<thead>
<tr>
<th>Run</th>
<th>Caliche upper</th>
<th>Clay - stiff, LV</th>
<th>$E_{\text{ref}}$</th>
<th>$U_y$</th>
<th>$U_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>SA1</td>
<td>2.5E5</td>
<td>1000</td>
<td>0.0532</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>SA2</td>
<td>1.0E5</td>
<td>1000</td>
<td>-0.15575</td>
<td>0.00887</td>
<td></td>
</tr>
<tr>
<td>SA3</td>
<td>7.6E5</td>
<td>1000</td>
<td>-0.14838</td>
<td>0.03464</td>
<td></td>
</tr>
<tr>
<td>SA4</td>
<td>2.5E5</td>
<td>500</td>
<td>-0.20818</td>
<td>2.4743</td>
<td></td>
</tr>
<tr>
<td>SA5</td>
<td>2.5E5</td>
<td>4000</td>
<td>-0.07247</td>
<td>0.61158</td>
<td></td>
</tr>
</tbody>
</table>
6.4.4 Results

Based on the back analysis of the field settlement data shown in Figure 4.44, a linear variation of soil stiffness of $E_s = 1,000 + 12z$ ksf (depth $\leq 19$ ft.) was obtained. Based on the boring data and the fact that the upper 19 feet at the site is predominantly caliche, the base modulus value of 1,000 ksf was selected to best represent the stiffness of the soils at a depth of 19 feet.

The back calculated modulus and the pressuremeter data are shown in Figure 6.36. The calculated modulus profile is on the upper bound of the pressuremeter modulus data for depths greater than 75 feet, with the exception of the isolated high measurements which are likely due to cemented soils. Note that the pressuremeter data shallower than 75 feet reflects the presence of the soft zone at the south end of the building area. The calculated modulus profile matches well with the data point at a depth of 130 feet. It is not unusual that the back analyzed modulus data is somewhat greater that the in-situ measured data. As with most in-situ measurements of soil stiffness, the actual values are usually greater than the field measurements due to the combined effects of anisotropy, stress release, soil disturbance and sensitivity to operator errors.

Deformation and vertical strain contours are shown in Figures 6.37 and 6.38. Displacements are effectively zero at the model edges which indicate the model extents are satisfactory.

6.5 Summary

The back analysis of the test fill and test pile TP-3 have resulted in elastic moduli for the sandy clay soil ($E_s$) and the caliche ($E_c$) of $E_s = 1,000 + 12z$ ksf (for $z < -18$ ft), and $E_c = 280,000$ ksf, respectively. These soil stiffness values will be used in the model
of the case study hotel tower to predict settlement of the CSP foundation.

Figure 6.36  Back calculated elastic modulus for test fill

The test fill model sensitivity analysis indicates that the settlement is most sensitive to the stiffness of the sandy clay vs. the caliche. The pile model indicates that for a pile installed primarily in a thick caliche layer, the deflection is most sensitive to the stiffness of the caliche vs. the surrounding soil.

The mesh sensitivity analyses indicate that PLAXIS 2D FEM mesh is sufficiently accurate in evaluating displacement using reasonably fine meshes.
From Table 6.2, the intact elastic modulus of the caliche based on the ACI 363R correlation is 670,000 ksf. The above analysis of TP-3 gives a mass modulus of 280,000 ksf, and therefore the mass modulus ratio is 0.4. Based on the strength reduction factors shown in Table 6.6, the caliche strength and pile ultimate friction properties used in the case study model will be reduced by a factor of 0.75.
The caliche mass modulus from the back analysis may be compared to the results of in-situ dynamic tests performed at the subject site. Spectral Analysis of Surface Waves (SASW) testing (Stokoe et al. 1994) was performed by the author and representatives of the UNLV Engineering Geophysics Laboratory (UNLV EGL) at the site prior to construction. The reduced shear wave data is available from the EGL web site (http://www.ce.unlv.edu/egl/lv_archives/). The results of the testing indicate average elastic modulus values for the upper caliche of 150,000 to 300,000 ksf. For the lower
caliche layer, the testing yielded average modulus values of 200,000 to 400,000 ksf. These are considered mass modulus values since the data is reflective of scale effects. The numerically derived caliche modulus value of 280,000 for the upper caliche agrees well with the SASW test results.
CHAPTER 7
CASE STUDY BUILDING SETTLEMENT ANALYSIS

7.1 General

The intent of Chapter 6 was to back analyze field tests to determine stiffness properties of the caliche and sandy clay (the two dominant materials at the case study site) for use in the tower settlement model. Soil properties for use in the numerical models have been established in Chapter 5. The profile used for analysis will be based on the borings performed in the tower area. A profile of the boring data is shown in Figure 4.2. The settlement analysis will be performed using both 2D and 3D models, and the results compared to the field measurements.

Figure 7.1 shows the overall dimensions, column line designations and foundation elements for the building. The typical column line consists of four pile caps; two 2 pile caps at the exterior of the building perimeter, and two 3 pile caps at the interior. The shear wall and core pile caps have thicknesses of 8 and 9.75 feet, respectively.

Figure 7.2 includes the dead plus live-loads (D+L) at the foundation level. The column lines with 4 pile caps have the same loads except line 1, as shown. Dead loads are 87 percent of the live loads, as specified by the structural engineer. All the piles (300 piles) in the actual foundation which were modeled using embedded piles in the 3D model are also shown in Figure 7.2.
7.2 2D Finite Element Analysis

The 2D plane strain model assumes no variability in soil conditions in the in-plane direction. A plane strain analysis is a valid assumption since the building length to width (L/B) ratio is approximately 10. A plain strain analysis is suitable for L/B ratios greater than 5 (Elhakim, 2005).
For the case study site and building conditions, a single model would provide a simplified estimate of the settlement. Therefore, to get an improved estimate of the settlement variation along the building length, four different 2D models were implemented. Each model had soil conditions (caliche thickness) deemed appropriate for its area based on the pre-drilling data. The areas modeled were the column line at the north end of the tower, north core, column line in the middle of the tower, and the south core. The previously discussed soft soil zones were included for the south core model only and were modeled as an undrained material.

7.2.1 Model Parameters

The model soil profile is essentially the same as used previously in the back-analyses, with the exception of the upper and lower caliche thicknesses. Soil parameters are based on the results of the back-analyses in Chapters 5 and 6.

In a 2D plane strain model, piles are simulated by means of continuous strips in the in-plane direction. In this case, it is necessary to make adjustments to the pile strip, elastic modulus, and the interface friction ratio. To analyze the piles using a plane strain model, the pile elastic modulus was adjusted as shown below to account for pile numbers, spacing and diameter, according to Prakoso (1999) and Desai et al. (1974).

\[
E_{eq} = \frac{n_p A_p E_p}{L_s B} = \frac{A_p E_p}{s B}
\]  

(7.1)

where,

\(E_{eq}\) = equivalent plane strain elastic modulus

\(n_p\) = No. of piles per row

\(A_p\) = cross sectional area of pile
\( E_p \) = pile elastic modulus

\( L_r \) = length of pile row

\( B \) = pile diameter

\( s \) = pile spacing

Relative to pile friction, the surface area of all the piles in the pile row will be smaller than the surface area of the plane strain pile strip. Accordingly, the interface pile strip side friction needs to be reduced by adjusting the interface coefficient as follows:

\[
R_{\text{inter}} = \frac{f_{s-eq}}{f_s} = \frac{n_p A_s}{2 L_r} = \frac{A_s}{2 s}
\]

where,

\( R_{\text{inter}} \) = PLAXIS interface friction coefficient

\( f_{s-eq} \) = equivalent strip friction

\( f_s \) = single pile friction

\( A_s \) = pile surface area

The soil parameters used in the 2D model are shown in Tables 7.1 and 7.2.

Table 7.1 Linear elastic parameters for 2D settlement analysis

<table>
<thead>
<tr>
<th>Linear Elastic Properties</th>
<th>Mat</th>
<th>1 pile row</th>
<th>pile cap</th>
<th>2 pile row</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drainage</td>
<td>Drained</td>
<td>Drained</td>
<td>Drained</td>
<td>Drained</td>
</tr>
<tr>
<td>( \gamma_{\text{unsat}} ) [klb/ft³]</td>
<td>0.15</td>
<td>0.15</td>
<td>0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>( \gamma_{\text{sat}} ) [klb/ft³]</td>
<td>0.15</td>
<td>0.15</td>
<td>0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>( E_{\text{ref}} ) [klb/ft²]</td>
<td>350,000</td>
<td>60,000</td>
<td>23,000</td>
<td>120,000</td>
</tr>
<tr>
<td>( \nu ) [-]</td>
<td>0.20</td>
<td>0.20</td>
<td>0.20</td>
<td>0.20</td>
</tr>
<tr>
<td>( R_{\text{inter}} ) [-]</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>
Table 7.2 Mohr-Coulomb soil parameters for 2D settlement analysis

<table>
<thead>
<tr>
<th>Mohr-Coulomb Parameters</th>
<th>1 Caliche</th>
<th>2 Sand &amp; gravel</th>
<th>3 Soft Clay</th>
<th>4 Stiff Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drainage</td>
<td>Drained</td>
<td>Drained</td>
<td>UnDrained</td>
<td>Drained</td>
</tr>
<tr>
<td>$\gamma_{unsat}$ [klb/ft³]</td>
<td>0.16</td>
<td>0.12</td>
<td>0.10</td>
<td>0.13</td>
</tr>
<tr>
<td>$\gamma_{sat}$ [klb/ft³]</td>
<td>0.16</td>
<td>0.13</td>
<td>0.12</td>
<td>0.13</td>
</tr>
<tr>
<td>$E_{ref}$ [klb/ft²]</td>
<td>280,000</td>
<td>3,000</td>
<td>300</td>
<td>1,000</td>
</tr>
<tr>
<td>$\nu$ [-]</td>
<td>0.30</td>
<td>0.30</td>
<td>0.35</td>
<td>0.40</td>
</tr>
<tr>
<td>$c_{ref}$ [klb/ft²]</td>
<td>173.0</td>
<td>0.1</td>
<td>0.10</td>
<td>0.20</td>
</tr>
<tr>
<td>$\phi$ [°]</td>
<td>35.0</td>
<td>45.0</td>
<td>24.0</td>
<td>30.0</td>
</tr>
<tr>
<td>$\psi$ [°]</td>
<td>5.00</td>
<td>10.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>$E_{inc}$ [klb/ft²/ft]</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>12.0</td>
</tr>
<tr>
<td>$y_{ref}$ [ft]</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>-16.0</td>
</tr>
<tr>
<td>$c_{increment}$ [klb/ft²/ft]</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>$T_{str.}$ [klb/ft²]</td>
<td>65.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>$R_{inter.}$ [-]</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>0.8</td>
</tr>
</tbody>
</table>

The core area model has dimensions of 1,000 feet wide by 500 feet deep and the column line models are half width. The building foundation components are located in the center of the mesh for the core models and at the mesh edge for the column line models. The 2D FEM mesh and loaded areas are shown in Figures 7.3 and 7.4. The 2-pile row has half the pile spacing of the 1-pile row so the equivalent stiffness is twice that of the 1-pile row.

Applied loads for the dead and dead plus live load cases are shown in Table 7.3. For the column line models, since the loads for column lines A through E differ, the average load of lines A and E was used for the exterior column line, and the average load of lines B and D was used for the interior column line. Analyses were performed for the structure at 28 and 51 floors (top out).

The models of the core areas include the full building width with pile caps on column lines D and E and either the north or south core. The column line models take...
advantage of symmetry about the building center and include an exterior and interior pile cap.

Table 7.3  Loads used for plane strain model

<table>
<thead>
<tr>
<th>Column Line</th>
<th>Cap Width, ft.</th>
<th>D+L Load, kips</th>
<th>Load/ft</th>
<th>D+L Pressure, ksf</th>
<th>D Load, kips</th>
<th>Load/ft</th>
<th>D Pressure, ksf</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>13</td>
<td>5070</td>
<td>141</td>
<td>10.8</td>
<td>4386</td>
<td>122</td>
<td>9.4</td>
</tr>
<tr>
<td>B</td>
<td>14</td>
<td>8751</td>
<td>243</td>
<td>17.4</td>
<td>7570</td>
<td>210</td>
<td>15.0</td>
</tr>
<tr>
<td>D</td>
<td>14</td>
<td>8577</td>
<td>238</td>
<td>17.0</td>
<td>7419</td>
<td>206</td>
<td>14.7</td>
</tr>
<tr>
<td>E</td>
<td>13</td>
<td>4710</td>
<td>131</td>
<td>10.1</td>
<td>4074</td>
<td>113</td>
<td>8.7</td>
</tr>
<tr>
<td>South Core</td>
<td>53</td>
<td>94827</td>
<td>702</td>
<td>13.3</td>
<td>82025</td>
<td>608</td>
<td>11.5</td>
</tr>
<tr>
<td>North core</td>
<td>53</td>
<td>96985</td>
<td>718</td>
<td>13.6</td>
<td>83892</td>
<td>621</td>
<td>11.7</td>
</tr>
</tbody>
</table>

Figure 7.3  2D model mesh for settlement analysis
7.2.2 Sensitivity Analysis

A sensitivity analysis of the 2D model was performed to determine the soil parameters that have the largest effect on the settlement. The parameters which were varied are indicated in Figure 7.5 and include the elastic modulus of the sandy clay soil, caliche, sandy gravel, and piles. Also varied were the cohesion of the sandy clay and caliche, and the interface strength on the piles. The total relative sensitivity of the parameters is shown in Figure 7.6.

![Figure 7.4 2D model foundation elements](image-url)
The sensitivity analysis indicates that the model deflection is most sensitive to the elastic modulus of the sandy clay soil, and to a lesser extent, the modulus of the caliche. This may be compared to the sensitivity analysis results for the back analyses performed in Chapter 6. Recall that the pile load test model was sensitive only to the elastic modulus of the caliche and the test fill model was most sensitive to the stiffness of the sandy clay.

For the subject model in this section, the total relative sensitivity of sandy clay modulus is twice that of the caliche modulus. Similar to the test fill back analysis model, the settlement is most dependent on the soil modulus since that is the dominant material (in terms of volume) in the model. The stiffness of the caliche is also important in this model since the caliche layers are part of the CSP foundation.

In conclusion, regarding the interface element use at the pile-soil contact, there is a negligible effect on the deflection from the interface properties. With the CSP model, the pile is constrained at both ends by the caliche layers, which reduces the relative deflection of the pile-soil interface. Therefore, it is not anticipated that the use of an interface would have a substantial effect on displacement of a CSP foundation.

7.2.3 Results

For the four different plane strain models, the results were combined to give a settlement profile along the building length. The settlement analysis results for the building with 28 and 51 floors are shown in Figures 7.7 and 7.8, respectively.
Figure 7.5 2D model sensitivity scores

![2D model sensitivity scores](image)

Figure 7.6 2D model total relative sensitivity results

![2D model total relative sensitivity results](image)
For the 28 floor case, the 2D analysis overestimated the settlement compared to the field measurements, as indicated in the plot. The error was more pronounced in the core areas compared to the column line areas. Since the core loads are much higher than the column line areas, it is expected that the settlements there would be larger. This trend holds true for the core 2D model, but the field measurements do not indicate a significant difference between these areas. It should be noted that the plane strain model does not capture the reduced settlement at the building ends due to the 3D effect.

The settlement estimate for the 51 floor case (building top out) is indicated in Figure 7.8. The measured data shown in the curve is for the period from building top out to 3 months afterward. For this case, the calculated settlement for the dead load case (black curve) matches the field measurement reasonably well, except for the north core area. It should be noted that the calculation includes the full dead load. One would expect that this load is reached at some point during the three month post top out period as the final cladding is applied to the building. It also takes about two to three months for dead load settlements to stabilize. For additional reference, the 2D model results for the dead plus live load case is also shown. As expected, the largest settlement would be calculated for this case. However, it only exceeds the measured data for the dead load case by about ½ inch.
Vertical deflection, strain and total stress contours for the south core model are shown in Figures 7.9 to 7.11. The 2D model results indicate that vertical deflections extend from the middle center to approximately 300 feet on either side of the building. Also, the vertical deflections are nearly zero below a depth of about 400 feet.

The vertical strain plot indicates that the maximum strain occurs in the sandy clay layers below the pile tips, and is on the order of 0.35%, or 0.0035. For the south core model, the soft clay layers contribute less to the total settlement than the sandy clay since they are modeled as undrained and have zero volume change upon instantaneous loading. Below a depth of about 150 feet, the strain is less than 0.1 percent.
The vertical stress after loading is shown in Figure 7.11. This figure shows the total vertical stress including the overburden stress (body forces). The applied model load results in a stress increase at the base of the model. However, the high soil stiffness at depth precludes any vertical strain as noted in Figure 7.10.

The horizontal tensile strains in the caliche layers are indicated in Figures 7.12 and 7.13. As shown in the figures, the maximum tensile stress occurs in the caliche layer below the pile tips and the analysis indicates the maximum tensile stress is on the order of 70 ksf (485 psi). For the caliche at the case study site with an average compressive strength of 8,200 psi, the tensile strength based on ACI 363R is 670 psi. Based on this...
information, a factor of safety against failure in bending for this case would be 1.4. It is interesting to note that the same approximate magnitude of horizontal tensile stress is present in the lower caliche layer for both the dead (D) and dead plus live load (D+L) cases. This implies that the load at the pile tips must be nearly the same for both cases so the additional applied load for the D+L case is distributed to the upper caliche layer.

Model calculated excess pore pressures in the soft clay layers are shown in Figure 7.14 which indicates a maximum value of about 8 ksf.

To determine the vertical stress increase solely due to the loaded configuration, the 2D Model initial (gravity) stresses were zeroed and the loads applied. This also required using linear elastic soil models to avoid reaching the failure criteria of the MC Model. The resulting vertical stress increase and strain contours are shown in Figures 7.15 to 7.17.

Figure 7.15 shows the dissipation of the stress field with depth. A close-up view of this figure is shown in Figure 7.16. This figure clearly indicates that a portion of the applied load is transferred through the piles to the lower soil layers. It is also observed that there is essentially no vertical stress increase in the soils between the piles. Considering that the average load applied over the building width is 11.75 ksf, the stress in the soil below the pile tip is about 8 ksf. Also note that the lateral extent of vertical stress increase from the foundation is about 100 feet from the foundation edge. This is due to the beam action from the upper caliche layer. This behavior is also observed in the vertical strain contour plot in Figure 7.17, where vertical strains are observed beyond the foundation. Based on this result, one could explore the benefit of additional piles beyond the limits of the foundation which would likely reduce the observed vertical strain.
Figure 7.9 2D model vertical deflection contours

Figure 7.10 2D model vertical compressive strain contours
Figure 7.11 2D model vertical total compressive stress contours

Figure 7.12 2D model horizontal tensile stress contours
Figure 7.13  2D model horizontal tensile stresses, south core, D+L load

Figure 7.14  2D model excess pore pressures for D+L load case
Figure 7.15 2D model vertical stresses with no body forces

Figure 7.16 Detailed view of 2D model vertical stresses with no body forces
7.2.4 Parametric Study of CSP Foundation

The efficiency of the 2D model affords the opportunity to perform a parametric study of the CSP foundation that otherwise would be too numerically intensive using the 3D model. The interests here are in the effect of additional pile length and the presence of the upper and lower caliche layers on the settlement. As previously mentioned, the upper and lower caliche layers provide the stiffening aspect of the CSP foundation, so the parametric study will provide information on the benefit of the CSP.

Figure 7.17 2D model vertical strains with no body forces
The typical purpose of increasing the pile length would be to reduce settlements. Additionally, at some sites, the upper caliche layer is not present or continuous in the building area and many projects have either basements or elevator cores which penetrate though the upper caliche layer. It is anticipated that this condition would result in additional settlement compared to the case of the CSP foundation.

For the CSP foundation at the south core area, the pile length was varied between 1 and nearly 4 times the actual pile length of 30 feet. The effect of increased pile length for the CSP foundation (case 1) and the cases with no upper caliche (case 2), no upper and lower caliche (case 3) are shown in Figure 7.18. In Figure 7.18, the settlement is normalized to the settlement of the CSP case for the south core area, and the pile length is normalized to the design length of 30 feet. Note that this area has the least amount of caliche in the building footprint. It could be expected that the removal of the caliche layers would have more effect at other areas of the building.

For case 1 (CSP foundation), the analysis indicates that for piles twice the design length, the reduction in settlement is only about 10 percent. This would represent a significant cost increase in pile construction for only a slight reduction in settlement. Each additional increment in pile length equal to the design length reduces the settlement by an additional 10 percent. To reduce the settlement in half, 150 foot long piles would be required. In the analysis, pile lengths greater than 70 feet were tipped on an existing caliche layer.

The analysis data for case 2 indicates that the settlement without the upper caliche layer is approximately 15 percent higher than case 1 for the design case of 30 foot long piles. This difference in settlement becomes less as the pile length is increased, or the
upper caliche has less effect on the settlement for longer piles. The results indicate that the settlement reduction of cases 1 and 2 due to increasing the pile length by 50 percent is insignificant.

The case 3 model indicates that the settlement of the south core pile foundation is about 50 percent higher than the CSP foundation. For pile lengths greater than 60 feet, the normalized settlement is very close to case 2.

For long pile lengths, all cases approach the same limiting settlement value which for the south core model is about 3 inches. The analysis indicates the benefit of the CSP foundation which has the lowest settlement and the shortest pile length.

![Figure 7.18 Normalized settlement vs. normalized pile length from CSP parametric study](image)
The effect of the CSP system on load distribution can be evaluated from the model with zero initial body forces and linear elastic properties to avoid a failure condition. Figures 7.19 and 7.20 show the vertical stress distributions as they vary with depth and in the lateral direction, respectively. Figure 7.19 shows how the vertical stress, due solely to the load, varies with depth for both the CSP foundation and the pile foundation without the upper or lower caliche layers. Both cases have the same loads (D+L) at the surface, and the plot begins at a depth of -19 feet which is at the bottom of the upper caliche layer. The analysis indicates the effect of the caliche layers in the CSP foundation results in lower stresses in the upper 350 feet. This aids the settlement reducing effect of the CSP foundation.

It is also of interest to examine the lateral variation in vertical stress beneath the lower caliche layer. Stresses were plotted at a depth of approximately 45 feet. Compared to the case without caliche, the smoothing effect on stresses of the CSP foundation can be clearly seen in the plot. High localized stresses for the case with no caliche are due to the presence of the pile tips. The beam or plate effect for the CSP case is also observed as the stresses are higher than the case with no caliche away from the foundation. The plot also shows the load determined by numerical integration of each stress distribution. The average load from the integration of each distribution is 1,081 kips, and the average applied load is 1,080 kips which are equal, considering the numerical error associated with the integration.
Figure 7.19  Vertical stress variation along centerline with depth

Figure 7.20  Vertical stress distribution on a horizontal plane at a depth of 45 ft.
7.3 3D Finite Element Analysis

A 3D model was created to evaluate settlements over the entire building area. One advantage the 3D model has compared to the 2D model is in evaluating the differential settlements between the cores and column areas.

The model includes all aspects of the foundation system except the grade beams which connect each pile cap which were excluded since they caused some elements to become too slender, increasing the chance for numerical issues. It is assumed that this would not affect the building settlement since they are for lateral load purposes rather than bending strength.

The model includes each pile that was installed in the building foundation (300 piles), each with ultimate skin friction properties depending upon where in the building it is located. The pile caps and core mats are simulated using structural plate elements. All pile tops are rigidly connected to pile caps or core mats. There are four different pile types with corresponding ultimate friction profiles that depend upon the location.

PLAXIS 3D Foundation includes an option for variable soils layering based on boring information that may be placed anywhere in the model. Soil layers are then linearly interpolated in both the x and z-directions. This option was utilized for two borings at the site to include the soft zones only present at the south end of the site, and the thicker upper caliche zone at the north end of the building.
7.3.1 Model Parameters

The building foundation elements and loads are shown in Figures 7.1 and 7.2. The model parameters include properties for embedded pile and soil properties, soil stratigraphy, soil properties, pile caps, core mats and core walls. The embedded pile properties are shown in Tables 7.4 and 7.5. An ultimate end bearing pressure of 100 ksf was used for all piles which were tipped in the lower caliche layer. This is value is based on Osterberg load tests at the NDOT sites which were tipped in thin caliche layers of similar thickness and failed in end bearing at a pressure of about 100 ksf (KI, 1996). Soil stratigraphy is shown in Table 7.7.

Table 7.4 Embedded pile ultimate skin friction properties

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>fsu, ksf</th>
</tr>
</thead>
<tbody>
<tr>
<td>sandy gravel</td>
<td>5</td>
</tr>
<tr>
<td>caliche</td>
<td>96</td>
</tr>
<tr>
<td>sandy clay</td>
<td>7</td>
</tr>
</tbody>
</table>

Table 7.5 Embedded pile material properties

<table>
<thead>
<tr>
<th>Pile Location</th>
<th>Installed Length, ft</th>
<th>Diameter, ft</th>
<th>Ep, ksf</th>
<th>v</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column Piles</td>
<td>34</td>
<td>3.3</td>
<td>800,000</td>
<td>0.15</td>
</tr>
<tr>
<td>Core Piles</td>
<td>30</td>
<td>3.3</td>
<td>800,000</td>
<td>0.15</td>
</tr>
</tbody>
</table>

The finite element mesh with plan and depth dimensions is shown below in Figures 7.21 and 7.22, respectively. When the 2D mesh is generated, prior to generating the 3D portion of the mesh, it is important that there are no very slender elements, as that can result in numerical problems, such as a singular stiffness matrix. Lines and nodes were added at various locations to force the elements to be more uniformly shaped.
Table 7.6 Soil profile for 3D Analysis

<table>
<thead>
<tr>
<th>Depth, ft.</th>
<th>Soil Type</th>
<th>Constitutive Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 7</td>
<td>sandy gravel</td>
<td>MC</td>
</tr>
<tr>
<td>7 - 10</td>
<td>caliche</td>
<td>MC</td>
</tr>
<tr>
<td>10 - 11</td>
<td>sandy gravel</td>
<td>MC</td>
</tr>
<tr>
<td>11 - 16</td>
<td>caliche</td>
<td>MC</td>
</tr>
<tr>
<td>16 - 40</td>
<td>sandy clay</td>
<td>MC</td>
</tr>
<tr>
<td>40 - 43</td>
<td>caliche</td>
<td>MC</td>
</tr>
<tr>
<td>43 - 85</td>
<td>sandy clay</td>
<td>MC</td>
</tr>
<tr>
<td>85 - 90</td>
<td>caliche</td>
<td>MC</td>
</tr>
<tr>
<td>90 - 108</td>
<td>sandy clay</td>
<td>MC</td>
</tr>
<tr>
<td>108 - 115</td>
<td>caliche</td>
<td>MC</td>
</tr>
<tr>
<td>115 - 127</td>
<td>sandy clay</td>
<td>MC</td>
</tr>
<tr>
<td>127 - 131</td>
<td>caliche</td>
<td>MC</td>
</tr>
<tr>
<td>131 - 136</td>
<td>sandy clay</td>
<td>MC</td>
</tr>
<tr>
<td>136 - 145</td>
<td>caliche</td>
<td>MC</td>
</tr>
<tr>
<td>145 - 152</td>
<td>sandy clay</td>
<td>MC</td>
</tr>
<tr>
<td>152 - 158</td>
<td>caliche</td>
<td>MC</td>
</tr>
<tr>
<td>158 - 167</td>
<td>sandy clay</td>
<td>MC</td>
</tr>
<tr>
<td>167 - 171</td>
<td>caliche</td>
<td>MC</td>
</tr>
<tr>
<td>171 - 190</td>
<td>sandy clay</td>
<td>MC</td>
</tr>
<tr>
<td>190 - 200</td>
<td>caliche</td>
<td>MC</td>
</tr>
<tr>
<td>200 - 400</td>
<td>sandy clay</td>
<td>MC</td>
</tr>
</tbody>
</table>

The model structural elements and embedded piles are shown in Figure 7.23, and the soil types are identified in Figure 7.24. Applied dead plus live loads are shown in Figure 7.25. As previously mentioned, dead loads (D) are assumed to be 87 percent of dead plus live (D+L) loads.

7.3.2 Results

Results of the 3D model analysis for 28 and 51 floors are shown in Figures 7.26 and 7.27. Note that the measured data was only available at certain points due to inaccessible or destroyed survey points. For the case of the building at 28 floors, the analysis results overestimate the settlement, as was the case for the 2D analysis. However, in this case, the overall settlement prediction is better than the 2D analysis.
Figure 7.21 3D model mesh for settlement analysis, plan view dimensions

Figure 7.22 3D model mesh for settlement analysis, side view
Figure 7.23  3D model structural elements

Figure 7.24  3D model soil types
It is unclear from the measured data when the final dead load was applied, although, it would be expected to be soon following the building top out. Additionally, given the rapid pace of construction, it is likely that some live loads were applied to the building during the three month period after top out. In evaluating the data, it is assumed that the total dead load was applied by the building top out and the dead plus live loads were applied by three months after top out.

For the case of the completed building frame at 51 floors, the calculated dead load case over estimates the settlement for the north half of the building, underestimates the settlement for the south core, and predicts the settlement reasonably well at the south end of the building. For the D+L case, the calculated settlement matches the measured settlement reasonably well in the south core area, but overestimates the settlement in the north half of the building. However, in this case, the shape of the settlement profile along the building is accurately predicted.
Figure 7.26  3D settlement analysis results for the structure at 28 floors

Figure 7.27  3D settlement analysis results for the structure at 51 floors
Figure 7.28 3D Model settlement contours for D+L load case

Figure 7.29 Settlement contours in building area for D+L load case
Figure 7.30  Cross section view of deflection contours for D+L load case

Vertical strain contours are shown in Figure 7.31. The maximum strain is about 0.4 percent which is very similar to that calculated by the 2D model.

The excess pore pressure distribution in the soft clay layer is shown in Figure 7.32. For reference, the model piles and core walls are also shown. The maximum excess pore pressure is 8.9 ksf, induced primarily by the load on the south core. This is slightly greater than the maximum value from the 2D model of 8.0 ksf. However, the 8.9 ksf maximum only occurs in isolated areas, as shown. The average value in the south core area appears to be about 8 ksf.

Vertical compressive stresses top of the caliche layer at the pile tips are shown in Figure 7.33. Areas not shown are in tension. These stresses shown represent the force at the pile tip when multiplied by the pile area of 8.55 square feet. The maximum stress is 110 ksf at column lines 18 and 19.
Figure 7.31  3D model vertical strain contours for D+L load case

Figure 7.32  Excess pore pressure contours for D+L load case
The average stress beneath the pile tip is about 30 ksf, or a typical pile tip load of 250 kips. The pile tip forces increase from the north to the south end of the building as the thickness of the upper caliche decreases. These are total stresses so the overburden stress of about 3 ksf is included.

Vertical compressive stresses on the bottom of the caliche layer at the pile tips are shown in Figure 7.34. The peak stress below the pile tips is about 50 ksf, while the average is on the order of 15 to 20 ksf.

Figure 7.35 shows stresses in the x-direction on the bottom of the caliche layer at the pile tips. The areas not shown between the pile caps are in tension. The average stress level (yellow areas) is about 90 ksf tension with some areas exceeding 200 ksf. The figure indicates that the tension stress increases from the north to the south as the upper caliche decreases in thickness. The tensile strength of the caliche may be estimated using the ACI 363R relation which is approximately 100 ksf for splitting tension. Figure 7.30 shows isolated areas on the lower caliche plate that exceed 100 ksf. If one computes the caliche rupture strength as 1.4 times the splitting tensile strength (as discussed in Chapter 6), there are seven isolated areas where this strength may be exceeded and cracking could occur. However, the true tensile strength is dependent upon the compressive strength of the lower caliche layer for which no information is available. As noted in Chapter 3, in the study performed by Kaderabek and Reynolds (1981), a field test which resulted in twice the laboratory tensile strength in a thin rock layer did not result in any apparent failure. The tensile stresses in the z-direction are shown in Figure 7.37. Since the z-direction peak values are lower than the x-direction, the latter would control.
Pile forces for a pile from a 3-pile cap in column line 17 (17B) are shown in Figure 7.38. Forces initially decrease in the upper caliche then show an increase with depth and then rapidly decrease due to the 2 foot embedment in the lower caliche layer. An observation of the vertical stress in the soil layer below the upper caliche shows an increase of 100 percent from approximately 2 to 4 ksf due to the applied loads. This additional stress is likely why the pile loads initially decrease and then show an increase with depth. Nevertheless, the load at the pile tip is largely reduced due to the lower caliche layer. A similar pile axial force distribution is shown in Figure 7.39 for a pile from the 3-pile cap at column line 5D.

7.4 Conclusions

Settlement analyses using 2D and 3D finite element models were performed for the case study hotel foundation. Four different models were used in the 2D analysis to provide a settlement profile along the building length. For the 28 floor case, the 2D analysis overestimated the settlement compared to the field measurements. For the dead load case at top out, the analysis data matches the field measurement fairly well, except for the north core area.

The sensitivity analysis indicates that the model deflection is most sensitive to the elastic modulus of the sandy clay soil, and to a lesser extent, the modulus of the caliche.

The presence of interface elements at the pile-soil region has a negligible effect on the model deflections.

The 2D analysis indicates that the maximum tensile stress on the bottom of the lower caliche layer is on the order of 70 ksf. The average tensile stress level based on the 3D analysis is about 90 ksf.
Figure 7.33  Vertical compressive stress on top of lower caliche layer
Figure 7.34 Compressive stress on bottom of caliche layer below pile tips
Figure 7.35  Horizontal tensile stress below pile tips in x-direction
Figure 7.36 Areas where the x-direction horizontal tensile stress below pile tips exceeds 100 ksf
Figure 7.37  Horizontal tensile stress below pile tips in the z-direction
Model calculated excess pore pressures in the soft clay layers indicate a maximum value of about 8 to 9 ksf for both the 2D and 3D models.
The parametric study of the CSP foundation indicates that for piles twice the design length, the reduction in settlement is only about 10 percent. The analysis indicates that the settlement without the upper caliche layer is approximately 15 percent higher and the settlement of the south core pile foundation without any caliche contact is about 50 percent higher than the CSP foundation for the same pile length.

The CSP foundation results in a smoothed vertical stress distribution on the base of the lower caliche layer compared to the case with no caliche layers. Additionally, the vertical stresses with depth from the bottom of the upper caliche layer are about 10 to 20 percent higher for the case without caliche layers compared to the CSP case.

For the case of the building at 28 floors, the 3D analysis overestimates the settlement. However, overall prediction is better than the 2D analysis.

For the D+L case at building top out, the calculated settlement using the 3D model matches the measured settlement reasonably well in the south core area, but overestimates the settlement in the north half of the building.

The maximum vertical strain calculated by the 2D and 3D models is 0.35 and 0.4 percent, respectively.

For the majority of the foundation area, the average stress beneath the pile tip based on the 3D model analysis is about 25 to 30 ksf, or a typical pile tip load of 200 to 250 kips.

The load distribution in the piles from the 3D model shows an initial decrease and then an increase in load with depth which is due to additional vertical stress from the beam effect of the upper caliche layer.
8.1 General

In Chapter 7, the settlement of the case study foundation was calculated using both the 2D and 3D models and the results compared to the measured data. In Chapter 8, the results of the 2D and 3D models is compared and further evaluated with regard to the measured data.

8.2 Measured and Computed Settlement Data Evaluation

The measured settlement data along D-line is shown in Figure 8.1, and the same data vs. time is shown in Figure 8.2. The initial data line, dated May 5, 2003, was measured for the structure at 6 stories. Since the floors were being added to the building frame at a consistent rate of one floor per week, the data prior to and including 20-stories (dated 8/05/2003) showed a constant accumulation of settlement. This corresponds to a linear settlement rate curve up to 190 days. Starting with the September 9, 2003 measured field data which corresponds to the building at a height of 24 levels at 218 days, the settlement rate increased. The rate increase is easily seen in Figure 8.2 since the slope of the curves is steeper. This was also the time when the settlement in the core areas started to exceed the settlement in the column line areas of the building.

After September 9, 2003, the settlement rate curve became steeper which is likely related to nonlinear soil/caliche behavior. There is a noticeable large movement in the data between the dates of February 2, 2004 and February 28, 2004 which corresponds to a building height of 45 to 49 stories, respectively. However, the settlement for the prior
period of December 30, 2003 to February 2, 2004 showed movement only in the south core area. It is evident that the settlement behavior shows nonlinear characteristics which is expected given the amount of settlement that occurred. The settlement stabilized at about three months after building top out. Additional consolidation settlement under constant load occurred later (neither shown nor considered herein), some of which may have been related to the construction of the mountain project, which incidentally started when the building was topped out.

Figure 8.1 Measured settlement data for column D
Figure 8.2  Measured settlement data vs. time for column line D

8.3 Comparison of 2D/3D Predicted vs. Measured Data

In Chapter 7, a comparison of the individual model results for both the 2D and 3D analyses to the measured data was provided. As mentioned previously, it is unknown when the total dead (D) or the dead plus live load (D+L) was applied. In evaluating the data, it is assumed that the total dead load was applied by time the building was topped out, and the dead plus live loads were applied by three months after top out.
The calculated 2D vs. 3D settlement and measured data for 28 floors is shown in Figure 8.3. Although both analyses over predict the measured settlement, the 2D and 3D results match reasonably well except at the building ends.

The calculated 2D vs. 3D settlement and measured data for 51 floors is shown in Figure 8.4. For the dead load analysis, the 2D model over predicts the settlement in the core areas, compared to the 3D model. In the column areas, both models had similar results. The results for the D+L analysis indicate that the 2D analysis matched the 3D analysis well in the core areas, but under predicted the settlement in the center of the building compared to the 3D result. This is due to the additional settlement of the center column line areas caused by the settlement of the cores which is not possible to capture with the 2D model.

As shown in Figure 8.4, the 2D analysis adequately predicts the maximum settlement in the cores at top out with all dead load applied. This is important, but provides limited information on the expected differential settlement, even with additional analyses for areas with different load and subsurface conditions. Regarding differential settlement (D+L data) between the center of the south core and the data point at 653 feet from the north end, the angular distortion calculated by the 3D model is on the order of 1/1,450, while the measured value is 1/1,000. The data indicates that the 3D model under predicts the differential settlement by 0.35 inches over a distance of 102 feet. This is considered a reasonable estimate given the magnitude of total settlement.
Figure 8.3 3D model calculated vs. measured data for the structure at 28 floors

8.4 Conclusions

The 2D and 3D models both predicted the maximum settlement in the south core area with reasonable accuracy, assuming the D+L loads were applied by three months after building top out. Settlements in the remaining areas of the building were over predicted. The over prediction of settlement is often the case since the true stiffness of the soil is not simply evaluated.

Among other parameters such as the stress path, the elastic modulus of a soil is a function of strain level, stress level and loading rate (Mayne et al. 2001).
The over prediction of the settlement by geotechnical engineers indicates that the design is somewhat conservative which is acceptable to ensure the foundation is safe and economical. The accuracy of settlement predictions is discussed by Coduto (1994), wherein he states that such an evaluation is faced with numerous uncertainties such as soil properties, the soil profile, errors in testing, unknown load conditions, and soil structure interaction effects. Anderson (2007) evaluated settlements of a spread footing using data from in-situ test methods including the standard penetration test, cone penetration test, dilatometer, and the pressuremeter. In addition, the engineering properties of the soils were evaluated using laboratory tests. Settlement predictions were
made by traditional and finite element methods, and compared to a field settlement test of the footing. The results indicated that the settlement was over predicted by all methods.

Prakoso (1999) conducted research on 2D plane strain analysis of pile rafts and showed that displacements are over predicted by 5 to 25 percent. This is due to the assumption of no strain in the out-of-plane direction which is overcome by the use of a 3D analysis to capture end effects. The 3D analysis is undoubtedly more accurate since it includes pile-pile, pile-upper and lower caliche, and pile-soil effects. For initial designs, the 2D analysis provides valuable information, and it is much simpler to perform parametric studies compared to the 3D analysis.

The CSP foundation system incorporates the settlement reducing effects of a pile foundation with the plate type stiffness effect from near surface caliche layers. This acts similar to a pile supported mat foundation. The settlement of a pile group in soil can be reasonably estimated using analytical (e.g., Randolph and Wroth, 1978, 1979; Scott, 1981; Shen et al. 2000; Chow, 1989; Guo, 2000; Lee and Xiao, 2001; Mylonakis and Gazetas, 1998), or numerical procedures (e.g., Poulos and Davis, 1968; Butterfield and Banerjee, 1971; Ottaviani, 1975; Banerjee and Davies, 1978; Chow, 1986). However, the analysis of a CSP foundation is more complex due to the combination of piles and high modulus caliche layers at the top and bottom, and can only be evaluated using numerical modeling techniques.

The 3D model used for the analyses in this dissertation included 300 individual piles which is considered numerous compared to typical pile foundation models in the literature. Some models used to analyze large pile foundations approximate the presence...
of the pile in soil through the use of a stiffened soil layer in the pile/soil zone (Rodriguez et al. 2009), or use an equivalent pier approach to approximate large pile groups with a single pier (Randolph, 1994; Poulos and Davis, 1980). In this case, the deformation of the foundation can be reasonably predicted, but the prediction of the pile load distribution and group effects may be less accurate. For the building core areas at the case study site, this would be an acceptable simplification, but for 2 and 3-pile caps spaced at 35 feet, it may not be appropriate.

In this research, modeling the foundation using individual pile elements has proved useful in determining the CSP pile load distribution which indicates the effect of the load in the upper caliche layer. This effect causes an increase in the pile load similar to a downdrag force (Fellenius, 2006). As indicated by Van Impe (2001) in a study of pile raft foundations, the presence of the raft (or upper caliche in this case) causes an increase in the normal stress at the pile soil interface which will increase the ultimate shaft friction capacity of the pile. This effect does not exist during the top load test of an individual CSP so the design load should not be based solely on this information. The design capacity should be reflective of additional loads that may result from this effect.

With the model piles in the exact configuration, it allowed for the determination of the tensile force in the caliche layer below the pile tips. It is important to evaluate the tensile stress in the design process to prevent excessive stress in the caliche layers of a CSP foundation.

Based on this research effort, the following are important points regarding the design of a CSP foundation for a high rise project in Las Vegas:
1) Perform borings and obtain samples at least every 5 feet.

2) Perform PI and moisture content tests on numerous samples to define the distribution vs. depth. This is the best method to determine the presence of isolated soft zones, and the relative compressibility of the soil. Do not rely solely upon the SPT data to determine the compressibility of soil layers.

3) Obtain cores and RQD of the upper and lower caliche layers deemed to be suitable for a CSP foundation. Perform UCS and splitting tensile tests on the cores.

4) Determine ultimate friction for a bored pile in caliche using Equation 3.5.

5) Perform a pile load test in the upper caliche layer and use a back analysis procedure to determine the caliche mass modulus. A 3D model is required if the load test is a top-down style with reaction piles.

6) Assume a linear variation in soil modulus with depth.

7) Use a 2D axisymmetric model to check the tensile stress in the lower caliche layer and compare to the splitting tension results.

8) Perform preliminary settlement analyses using a 2D plane strain model. Vary the parameters of the soil modulus and determine upper and lower bounds on settlement.

9) Use a 3D model to determine differential settlement, pile load at tip and resulting tensile stress in lower caliche layer. Revise the pile load as needed to avoid excessive stress of the lower caliche layer.
8.5 Recommendations for Further Research

The analyses in this dissertation utilized a simple elastic-perfectly plastic soil constitutive model. It would be desirable to include the effect of overconsolidation of the upper soils. The elastic modulus can be adjusted to account for an OCR > 1 (see Bowles, 1996). This would increase the modulus values in the upper part of the soil profile which may reduce predicted settlements.

To improve the settlement estimate, it would also be of use to evaluate the settlement using the PLAXIS hardening soil model and compare the results of both models. The soil stratigraphy of the 3D model could be improved by including more borings to better define the thickness of the upper caliche layer.

Based on the current research, a settlement analysis of the man made mountain structure at the case study site could be performed. This would require a 3D model due to the unusual shape of the mountain. It would be of interest to evaluate the effect of the mountain structure on the tower settlement.

Future research could include the evaluation of long term consolidation and creep settlement of the case study structure. There is a limited amount of measured data available for comparison since settlement monitoring points became inaccessible as the building was occupied. More research on the tensile stress that develops beneath foundations on thin caliche layers and the resulting stress distribution at depth should also be considered. Additionally, the effect on pile load of an adjacent vertical load on a caliche layer could be studied.
APPENDIX A

OSTERBERG LOAD TEST DATA

This appendix includes data from one Osterberg load test at the case study site.

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<thead>
<tr>
<th>Shaft:</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal shaft diameter: EL+2370 to EL+1942</td>
<td>= 48 inches</td>
<td>1219 mm</td>
</tr>
<tr>
<td>O-cell™ size (0014-6)</td>
<td>= 26 inches</td>
<td>660 mm</td>
</tr>
<tr>
<td>Length of concrete from break at base of cell to zero slipper (ground)</td>
<td>= 42.9 feet</td>
<td>13.1 meters</td>
</tr>
<tr>
<td>Length of concrete from break at base of cell to tip</td>
<td>= 85.1 feet</td>
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<tr>
<td>Shaft shear area above O-cell™</td>
<td>= 518.1 feet²</td>
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<tr>
<td>Shaft shear area from break at base of cell to tip</td>
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</tr>
<tr>
<td>Shaft end area</td>
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<tr>
<td>Weight of concrete from break at base of cell to tip</td>
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<td>Estimated shaft unit stiffness: EL+2370 to EL+1942</td>
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<tr>
<td>Elevation of ground surface</td>
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<tr>
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<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>None used</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Compression Sections:</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Elevation of top of teflon used for upper shaft compression</td>
<td>= +2079.0 feet</td>
<td>+631.9 meters</td>
</tr>
<tr>
<td>Elevation of bottom of teflon used for upper shaft compression</td>
<td>= +2098.5 feet</td>
<td>+618.3 meters</td>
</tr>
<tr>
<td>Elevation of top of teflon used for lower shaft compression</td>
<td>= +2026.4 feet</td>
<td>+617.6 meters</td>
</tr>
<tr>
<td>Elevation of bottom of teflon used for lower shaft compression</td>
<td>= +1942.2 feet</td>
<td>+602.0 meters</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Strain Gages:</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Elevation of strain gage Level 5</td>
<td>= +2040.9 feet</td>
<td>+624.7 meters</td>
</tr>
<tr>
<td>Elevation of strain gage Level 4</td>
<td>= +2031.2 feet</td>
<td>+621.7 meters</td>
</tr>
<tr>
<td>Elevation of strain gage Level 3</td>
<td>= +1995.2 feet</td>
<td>+608.1 meters</td>
</tr>
<tr>
<td>Elevation of strain gage Level 2</td>
<td>= +1977.1 feet</td>
<td>+602.0 meters</td>
</tr>
<tr>
<td>Elevation of strain gage Level 1</td>
<td>= +1961.2 feet</td>
<td>+597.8 meters</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Miscellaneous:</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Top plate diameter</td>
<td>= 38.0 inches</td>
<td>991 mm</td>
</tr>
<tr>
<td>Top plate thickness</td>
<td>= 1.5 inches</td>
<td>38.1 mm</td>
</tr>
<tr>
<td>Bottom plate diameter</td>
<td>= 42.0 inches</td>
<td>1067 mm</td>
</tr>
<tr>
<td>Bottom plate thickness</td>
<td>= 1.5 inches</td>
<td>38.1 mm</td>
</tr>
<tr>
<td>Water elevation</td>
<td>= +2995.0 feet</td>
<td>+626.7 meters</td>
</tr>
<tr>
<td>LYNWD™ radii</td>
<td>= 16.0 inches</td>
<td>406.4 mm</td>
</tr>
<tr>
<td>LYNWD™ orientation</td>
<td>= 0, 180, 270</td>
<td>degrees</td>
</tr>
<tr>
<td>Vertical re-bar size</td>
<td>= #11</td>
<td></td>
</tr>
<tr>
<td>Hoop re-bar size</td>
<td>= #4</td>
<td></td>
</tr>
<tr>
<td>Number of vertical bars</td>
<td>= 12</td>
<td></td>
</tr>
</tbody>
</table>

Figure A.1 Osterberg Load Test Summary data
NOTE:
- NOMINAL SHAFT DIAMETER 48"

GROUND ELEV.
CLAYEY SAND
CALICHÉ
Silty Sand
CALICHÉ
CALICHÉ
CLAYEY SAND

ECT2 17905, 17906

LVMOC 17898, 17897, 17898
26" O-Ceil (0014-6) TOP OF ECT1

SG4 20170, 20171 +2010.2

SG3 20166, 20167 +1995.2

ECT1 17901, 17902

SG2 20164, 20165 +1977.1

SG1 20160, 20151 +1961.2

CLANYEY SAND

BOTTOM OF ECT1
TIP OF SHAFT +1842.2

Figure A.2 Load Test 1 Schematic
Figure A.3  Load Test 1 Deflection data
Figure A.4 Test 1 Load distribution data
Lower compression was used instead of calculated downward movement in constructing this plot.

Net Unit Shear vs. O-cell Movement

Desert Inn Hotel / Casino - Las Vegas, NV - Test Shaft 1

Unit Shear (kips)

Movement (inches)
APPENDIX B

CALICHE TRIAXIAL TEST DATA

Appendix B includes data from triaxial tests performed on caliche (KI, 1996).

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Marker</th>
<th>Length (inch)</th>
<th>Diameter (inch)</th>
<th>Saturated Weight (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>01</td>
<td>31-215903 B-1 15.4' - 15.8'</td>
<td>4.680</td>
<td>2.053</td>
<td>648.738</td>
</tr>
<tr>
<td>02</td>
<td>31-215903 B-5 14' - 15'</td>
<td>4.283</td>
<td>2.002</td>
<td>555.296</td>
</tr>
<tr>
<td>03</td>
<td>31-215903 B-5 13.5' - 14'</td>
<td>4.290</td>
<td>1.998</td>
<td>563.061</td>
</tr>
</tbody>
</table>

Table 2. Static Data.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Marker</th>
<th>Confining Pressure (psi)</th>
<th>Ultimate Strength (psi)</th>
<th>Young's Modulus (psi)</th>
<th>Poisson's Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>01</td>
<td>31-215903 B-1 15.4' - 15.8'</td>
<td>14</td>
<td>7,860</td>
<td>4.77E06</td>
<td>0.17*</td>
</tr>
<tr>
<td>02</td>
<td>31-215903 B-5 14' - 15'</td>
<td>14</td>
<td>8,400</td>
<td>4.30E06</td>
<td>0.31</td>
</tr>
<tr>
<td>03</td>
<td>31-215903 B-5 13.5' - 14'</td>
<td>14</td>
<td>10,645</td>
<td>2.90E06</td>
<td>0.33</td>
</tr>
</tbody>
</table>

* Radial transducers displaying unequal strain. Both gauges plotted. 
* Calculated using radial gauge #1 strain = 0.001, v calculated using radial gauge #2 strain = 0.35.
31-215903  B-5  13.5' - 14'
TEST 03  KLEINFELDER

Differential Axial Stress - psi

Confining Pressure = 14 psi
Pore Pressure = 00 psi

Radial 1
Radial 2
Radial
Axial

Strain in/in

0
0.0025
0.0050

Science Applications International Corporation
An Employee-Owned Company
Dilatometer data from the NDOT site (KI, 1996).

### APPENDIX C

**DILATOMETER DATA**

**GREGG IN SITU**

PROJECT: US 95 AND I-15 INTERSECTION

RECORD OF DILATOMETER TEST NO. B-1 THRU B-5

**INITIAL CALIBRATION INFORMATION:**

DELTA A = .18 BARS  DELTA B = .50 BARS  GAGE 0 = .00 BARS

1 BAR = 1.019 KG/CM²  1.044 TFS = 14.51 PSI

ANALYSIS USES 2% UNIT WEIGHT = 1.00 T/M³

LOCATION: B-1

PERFORMED - DATE: 3-20-95

GWT DEPTH = 3.35 M

<table>
<thead>
<tr>
<th>Z (FT)</th>
<th>THRUST (KGS)</th>
<th>A (BAR)</th>
<th>B (BAR)</th>
<th>ED (BAR)</th>
<th>ID (BAR)</th>
<th>KD (BAR)</th>
<th>UO (BAR)</th>
<th>GAMMA (T/M²)</th>
<th>SV (BAR)</th>
<th>PC (BAR)</th>
<th>OCR</th>
<th>KD (BAR)</th>
<th>CU (BAR)</th>
<th>PHI (DEG)</th>
<th>M (BAR)</th>
<th>SOIL TYPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.29</td>
<td>5200.0</td>
<td>9.5</td>
<td>&gt;55</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.96</td>
<td>2200.0</td>
<td>1.49</td>
<td>5.04</td>
<td>105.0</td>
<td>2.05</td>
<td>2.13</td>
<td>.990</td>
<td>1.000</td>
<td>.667</td>
<td>1.36</td>
<td>1.58</td>
<td>.69</td>
<td>27.8</td>
<td>111.2</td>
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<td>SILTY SAND</td>
</tr>
<tr>
<td>6.50</td>
<td>5000.0</td>
<td>3.34</td>
<td>14.06</td>
<td>385.0</td>
<td>3.39</td>
<td>2.99</td>
<td>.314</td>
<td>1.900</td>
<td>.103</td>
<td>3.41</td>
<td>3.78</td>
<td>.57</td>
<td>32.3</td>
<td>530.0</td>
<td></td>
<td>SAND</td>
</tr>
<tr>
<td>9.23</td>
<td>12000.0</td>
<td>19.0</td>
<td>&gt;55</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.91</td>
<td>7000.0</td>
<td>19.0</td>
<td>&gt;55</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NEW QA** = .22  **NEW QB** = .55

12.65 2500.0  4.39  6.99  124.0  1.01  2.53  .142  1.400  1.412  2.00  1.41  .67  136.2  SLT

13.71 500.0  3.03  7.99  120.0  1.32  1.76  1.017  1.700  1.490  1.32  .68  .70  25.5  102.2  SANDY CLAY

14.93 3000.0  3.64  14.45  135.0  .46  5.38  1.136  1.600  1.588  7.43  4.69  1.29  102.3  SILTY CLAY

16.81 3000.0  4.80  —     No B reading, water in GMT blade, shorted out

**END OF SOUNDING**

LOCATION: B-2

PERFORMED - DATE: 3-21-95

**CALIBRATION INFORMATION:**

DELTA A = .18 BARS  DELTA B = .48 BARS  GAGE 0 = .00 BARS  GWT DEPTH = 3.66 M

<table>
<thead>
<tr>
<th>Z (FT)</th>
<th>THRUST (KGS)</th>
<th>A (BAR)</th>
<th>B (BAR)</th>
<th>ED (BAR)</th>
<th>ID (BAR)</th>
<th>KD (BAR)</th>
<th>UO (BAR)</th>
<th>GAMMA (T/M²)</th>
<th>SV (BAR)</th>
<th>PC (BAR)</th>
<th>OCR</th>
<th>KD (BAR)</th>
<th>CU (BAR)</th>
<th>PHI (DEG)</th>
<th>M (BAR)</th>
<th>SOIL TYPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.22</td>
<td>550.0</td>
<td>2.99</td>
<td>7.64</td>
<td>145.0</td>
<td>1.41</td>
<td>12.60</td>
<td>.900</td>
<td>1.800</td>
<td>.230</td>
<td>5.77</td>
<td>25.10</td>
<td>1.92</td>
<td>30.3</td>
<td>399.0</td>
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<td>SANDY CLAY</td>
</tr>
<tr>
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<td>1600.0</td>
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<td>12.65</td>
<td>282.0</td>
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<td>.900</td>
<td>1.900</td>
<td>.451</td>
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<td>30.48</td>
<td>1.41</td>
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</tr>
<tr>
<td>3.88</td>
<td>2400.0</td>
<td>6.24</td>
<td>20.00</td>
<td>472.0</td>
<td>2.40</td>
<td>8.41</td>
<td>.900</td>
<td>2.000</td>
<td>.585</td>
<td>18.63</td>
<td>27.19</td>
<td>1.32</td>
<td>32.5</td>
<td>1122.0</td>
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</table>

**END OF SOUNDING**
**LOCATION: B-3**  
**PERFORMED - DATE: 3-22-95**  

**CALIBRATION INFORMATION:**  
DELTA A = .18 BARS  
DELTA B = .48 BARS  
GAGE 0 = .00 BARS  
GWT DEPTH = 3.66 M

<table>
<thead>
<tr>
<th>Z (M)</th>
<th>THRUST (KG)</th>
<th>A (BAR)</th>
<th>B (BAR)</th>
<th>ED (BAR)</th>
<th>ID (BAR)</th>
<th>KD (BAR)</th>
<th>UO (BAR)</th>
<th>GAMMA (T/m)</th>
<th>SV (BAR)</th>
<th>PC (BAR)</th>
<th>OCR (BAR)</th>
<th>KO (BAR)</th>
<th>CU (BAR)</th>
<th>PHI (DEG)</th>
<th>M (BAR)</th>
<th>SOIL TYPE</th>
</tr>
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<tbody>
<tr>
<td>2.13</td>
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<td>5.60</td>
<td>125.</td>
<td>2.11</td>
<td>4.25</td>
<td>.000</td>
<td>1.60</td>
<td>.402</td>
<td>2.98</td>
<td>7.38</td>
<td>.90</td>
<td>29.5</td>
<td>214.0</td>
<td>SILTY</td>
<td></td>
<td>SAND</td>
</tr>
<tr>
<td>3.51</td>
<td>2.05</td>
<td>14.90</td>
<td>433.</td>
<td>2.91</td>
<td>2.51</td>
<td>.000</td>
<td>1.90</td>
<td>.253</td>
<td>1.76</td>
<td>3.89</td>
<td>.37</td>
<td>38.4</td>
<td>562.3</td>
<td>SAND</td>
<td></td>
<td>SAND</td>
</tr>
<tr>
<td>4.72</td>
<td>3.00</td>
<td>44.80</td>
<td>1142.</td>
<td>2.91</td>
<td>16.33</td>
<td>.104</td>
<td>2.150</td>
<td>.789</td>
<td>59.36</td>
<td>75.24</td>
<td>1.66</td>
<td>37.2</td>
<td>3243.9</td>
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<tr>
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<td>676.</td>
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<td>11.72</td>
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<td>2.100</td>
<td>.524</td>
<td>34.96</td>
<td>37.65</td>
<td>1.74</td>
<td>31.8</td>
<td>1760.5</td>
<td>SANDY</td>
<td></td>
<td>SILT</td>
</tr>
</tbody>
</table>

**END OF SOUNDING**

**LOCATION: B-4**  
**PERFORMED - DATE: 3-22-95**  

**CALIBRATION INFORMATION:**  
DELTA A = .18 BARS  
DELTA B = .48 BARS  
GAGE 0 = .00 BARS  
GWT DEPTH = 5.03 M

<table>
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<tr>
<th>Z (M)</th>
<th>THRUST (KG)</th>
<th>A (BAR)</th>
<th>B (BAR)</th>
<th>ED (BAR)</th>
<th>ID (BAR)</th>
<th>KD (BAR)</th>
<th>UO (BAR)</th>
<th>GAMMA (T/m)</th>
<th>SV (BAR)</th>
<th>PC (BAR)</th>
<th>OCR (BAR)</th>
<th>KO (BAR)</th>
<th>CU (BAR)</th>
<th>PHI (DEG)</th>
<th>M (BAR)</th>
<th>SOIL TYPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.37</td>
<td>1.55</td>
<td>6.50</td>
<td>229.</td>
<td>4.07</td>
<td>5.47</td>
<td>.000</td>
<td>1.80</td>
<td>.259</td>
<td>3.09</td>
<td>11.93</td>
<td>.82</td>
<td>36.5</td>
<td>452.7</td>
<td>SAND</td>
<td></td>
<td>SAND</td>
</tr>
<tr>
<td>2.59</td>
<td>3.35</td>
<td>24.70</td>
<td>754.</td>
<td>8.71</td>
<td>5.19</td>
<td>.000</td>
<td>1.90</td>
<td>.480</td>
<td>5.20</td>
<td>10.82</td>
<td>.00</td>
<td>43.7</td>
<td>1455.6</td>
<td>SAND</td>
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<td>SAND</td>
</tr>
<tr>
<td>3.66</td>
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<td>6.25</td>
<td>105.</td>
<td>1.11</td>
<td>4.98</td>
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<td>1.70</td>
<td>.670</td>
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<td>3.05</td>
<td>1.00</td>
<td>169.8</td>
<td>75.6</td>
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<td></td>
<td>SILT</td>
</tr>
<tr>
<td>5.03</td>
<td>4.00</td>
<td>1.45</td>
<td>4.55</td>
<td>6.89</td>
<td>1.70</td>
<td>1.68</td>
<td>.000</td>
<td>1.70</td>
<td>.658</td>
<td>1.03</td>
<td>.05</td>
<td>26.9</td>
<td>75.6</td>
<td>SANDY</td>
<td></td>
<td>SILT</td>
</tr>
<tr>
<td>6.25</td>
<td>2600.</td>
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<td>23.00</td>
<td>58.8</td>
<td>3.11</td>
<td>5.45</td>
<td>.120</td>
<td>2.000</td>
<td>1.80</td>
<td>15.68</td>
<td>11.88</td>
<td>.95</td>
<td>32.9</td>
<td>1150.5</td>
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<tr>
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<td>2800.</td>
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<td>19.80</td>
<td>497.</td>
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<td>4.24</td>
<td>.242</td>
<td>2.000</td>
<td>1.122</td>
<td>8.23</td>
<td>7.24</td>
<td>.34</td>
<td>31.7</td>
<td>671.7</td>
<td></td>
<td>SILTY</td>
</tr>
<tr>
<td>8.68</td>
<td>2200.</td>
<td>5.10</td>
<td>12.00</td>
<td>227.</td>
<td>1.42</td>
<td>3.75</td>
<td>.358</td>
<td>1.800</td>
<td>1.228</td>
<td>4.05</td>
<td>3.30</td>
<td>.90</td>
<td>27.4</td>
<td>352.0</td>
<td></td>
<td>SANDY</td>
</tr>
</tbody>
</table>

**END OF SOUNDING**
LOCATION: B-5  
PERFORMED - DATE: 3-23-95

CALIBRATION INFORMATION:
DELTA A = .20 BARS  DELTA B = .40 BARS  GAGE 0 = .00 BARS  GWT DEPTH = 3.66 M

<table>
<thead>
<tr>
<th>Z (M)</th>
<th>THRUST (KG)</th>
<th>A (BAR)</th>
<th>B (BAR)</th>
<th>ED (BAR)</th>
<th>ID (BAR)</th>
<th>KD (BAR)</th>
<th>UO (BAR)</th>
<th>GAMMA (t/M3)</th>
<th>SV (BAR)</th>
<th>PC (BAR)</th>
<th>OCR (BAR)</th>
<th>CU (BAR)</th>
<th>PHI (DEG)</th>
<th>M</th>
<th>SOL</th>
<th>TYPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.22</td>
<td>500</td>
<td>2.44</td>
<td>7.24</td>
<td>153</td>
<td>1.81</td>
<td>10.57</td>
<td>.000</td>
<td>.230</td>
<td>7.26</td>
<td>32.10</td>
<td>1.62</td>
<td>31.2</td>
<td>390.5</td>
<td></td>
<td></td>
<td>SELTY</td>
</tr>
<tr>
<td>2.29</td>
<td>9000</td>
<td>6.34</td>
<td>31.5</td>
<td>878</td>
<td>4.75</td>
<td>12.27</td>
<td>.000</td>
<td>.435</td>
<td>24.31</td>
<td>55.91</td>
<td>1.47</td>
<td>39.5</td>
<td>2368.3</td>
<td></td>
<td></td>
<td>SAND</td>
</tr>
<tr>
<td>3.51</td>
<td>2000</td>
<td>2.75</td>
<td>&gt;30</td>
<td>Over-inflated membrane, part of membrane likely in caliche</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>NEW DA = 0.18</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>4.71</td>
<td>5200</td>
<td>2.79</td>
<td>41.05</td>
<td>1372</td>
<td>48.96</td>
<td>.02</td>
<td>.299</td>
<td>1.000</td>
<td>.581</td>
<td>.31</td>
<td>.32</td>
<td>-1.18</td>
<td>45.0</td>
<td></td>
<td></td>
<td>1161.3</td>
</tr>
</tbody>
</table>

END OF SOUNDING
APPENDIX D

FREMONT ST. MOHR CIRCLES FOR CALICHE

This Appendix includes Mohr circles based on laboratory tests on caliche from the Fremont Street Experience site (WTI, 1994b).

Figure D.1 Mohr Coulomb Envelope for upper caliche, Fremont Street Experience
Figure D.2 Mohr Coulomb Envelope for lower caliche, Fremont Street Experience
APPENDIX E

BORING DATA FROM TEST PILE SITE

Appendix E includes a boring log from the case study site in the area where pile load tests TP-1, TP-2, and TP-3 were performed.

<table>
<thead>
<tr>
<th>DATE DRILLED:</th>
<th>LOCATION:</th>
<th>BORING NO. TP-1</th>
<th>ELEVATION: 2009</th>
</tr>
</thead>
<tbody>
<tr>
<td>4-5-02</td>
<td>3145 South Las Vegas Blvd.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>MOISTURE CONTENT (%)</th>
<th>DRY DENSITY (GUSC/LFT)</th>
<th>SAMPLE</th>
<th>BLOWS/PIT</th>
<th>DEPTH FEET</th>
<th>SPT</th>
<th>GRAPHIC</th>
<th>SOIL DESCRIPTION</th>
<th>MOISTURE</th>
<th>CONSISTENCY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>R</td>
<td>52</td>
<td>1</td>
<td></td>
<td></td>
<td>FILL-3&quot; AC over SANDY GRAVEL</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>R</td>
<td>57/11</td>
<td>2</td>
<td></td>
<td></td>
<td>SILTY SAND-trace gypsum, trace gravel</td>
<td>SI. damp</td>
<td>very hard</td>
</tr>
<tr>
<td></td>
<td></td>
<td>R</td>
<td>50/0&quot;</td>
<td>6</td>
<td></td>
<td></td>
<td>CEMENTED SAND &amp; GRAVEL-lt. brown</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>R</td>
<td>50/0&quot;</td>
<td>7</td>
<td></td>
<td></td>
<td>CALICHE-lt. brown</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>R</td>
<td>50/0&quot;</td>
<td>8</td>
<td></td>
<td></td>
<td>SANDY SILT-w/calcareous gravel, lt. brown</td>
<td>very stiff</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>R</td>
<td>50/0&quot;</td>
<td>9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>R</td>
<td>50/0&quot;</td>
<td>10</td>
<td></td>
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<td>50/0&quot;</td>
<td>11</td>
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<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>R</td>
<td>50/0&quot;</td>
<td>12</td>
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<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td></td>
<td>R</td>
<td>50/0&quot;</td>
<td>13</td>
<td></td>
<td></td>
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<td>R</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>ML</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>SANDY SILT-w/calcareous gravel, lt. brown</td>
<td>very stiff</td>
<td></td>
</tr>
</tbody>
</table>

NOTES: Water encountered at 19 feet. Elevation at 2068.59 feet.

DRIVING WEIGHT (LBS) 140

PROJECT: LE REVE HOTEL & CASINO
BORING LOG

WESTERN TECHNOLOGIES INC.
PROJECT NO. 4122J8051

PLATE A-2
<table>
<thead>
<tr>
<th>Sample</th>
<th>Depth (Feet)</th>
<th>Soil Description</th>
<th>Moisture</th>
<th>Consistency</th>
</tr>
</thead>
<tbody>
<tr>
<td>ML</td>
<td>16</td>
<td>Sandy Silty-w/calcareous gravel, lt. brown</td>
<td>Sl. damp</td>
<td>Very stiff</td>
</tr>
<tr>
<td></td>
<td>17</td>
<td>Caliche-lt. brown</td>
<td></td>
<td>Very hard</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>Silty Sand-lt. brown</td>
<td>Wet</td>
<td>Very dense</td>
</tr>
<tr>
<td></td>
<td>19</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>-w/clay lenses</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>21</td>
<td></td>
<td></td>
<td>Dense</td>
</tr>
<tr>
<td></td>
<td>22</td>
<td></td>
<td></td>
<td>Med. Dense</td>
</tr>
<tr>
<td></td>
<td>23</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>24</td>
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<td>25</td>
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<td>28</td>
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<td></td>
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<tr>
<td></td>
<td>29</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:** Water encountered at 19 feet. Elevation at 2068.59 feet.
<table>
<thead>
<tr>
<th>Depth (Ft)</th>
<th>Soil Description</th>
<th>Moisture</th>
<th>Consistency</th>
</tr>
</thead>
<tbody>
<tr>
<td>31-33</td>
<td>CLAYEY SAND-w/clay lenses, brown wet med. dense</td>
<td></td>
<td></td>
</tr>
<tr>
<td>33-38</td>
<td>SILTY SAND-brown</td>
<td></td>
<td>very dense</td>
</tr>
<tr>
<td>38-42</td>
<td>SANDY GRAVEL-brown</td>
<td></td>
<td></td>
</tr>
<tr>
<td>42</td>
<td>CALICHE-brown, l. brown</td>
<td></td>
<td>very hard</td>
</tr>
</tbody>
</table>

NOTES: Water encountered at 19 feet. Elevation at 2068.59 feet.

DRIVING WEIGHT (LBS) 140

PROJECT: LE REVE HOTEL & CASINO
BORING LOG

WESTERN TECHNOLOGIES INC.
PROJECT NO. 4122JS051

DATE DRILLED: 4-5-02
LOCATION: 3145 South Las Vegas Blvd.
BORING NO. TP-1 (Cont'd)
ELEVATION: 2069
<table>
<thead>
<tr>
<th>Depth (Feet)</th>
<th>Sample Type</th>
<th>Soil Description</th>
<th>Moisture</th>
<th>Consistency</th>
</tr>
</thead>
<tbody>
<tr>
<td>46</td>
<td>R</td>
<td>CLAYEY SAND-brown</td>
<td>moist</td>
<td>very dense</td>
</tr>
<tr>
<td>48</td>
<td>R</td>
<td>SANDY SILT-w/partially cemented lenses</td>
<td>very stiff</td>
<td></td>
</tr>
<tr>
<td>49</td>
<td>CL</td>
<td>SANDY CLAY-brown</td>
<td>wet</td>
<td>stiff</td>
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<tr>
<td>51</td>
<td>SC</td>
<td>CALICHE-lt. brown</td>
<td>moist</td>
<td>very hard</td>
</tr>
<tr>
<td>53</td>
<td>R</td>
<td>SANDY SILT-w/partially cemented lenses</td>
<td>very stiff</td>
<td></td>
</tr>
<tr>
<td>55</td>
<td></td>
<td>-w/within cemented lenses</td>
<td></td>
<td></td>
</tr>
<tr>
<td>56</td>
<td></td>
<td>-trace gravel</td>
<td></td>
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</tr>
</tbody>
</table>

**NOTES:** Water encountered at 19 feet. Elevation at 2068.59 feet.

**DRIVING WEIGHT (LBS):** 140

**PROJECT:** LE REVE HOTEL & CASINO

**BORING LOG**

<table>
<thead>
<tr>
<th>DATE DRILLED: 4-5-02</th>
<th>BORING NO. TP-1 (Cont'd)</th>
<th>ELEVATION: 2069</th>
</tr>
</thead>
<tbody>
<tr>
<td>LOCATION: 3145 South Las Vegas Blvd.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**WESTERN TECHNOLOGIES INC.**

**PROJECT NO. 4122JS051**

Plate A-5
APPENDIX F

BORING DATA FROM MOUNTAIN SITE

Appendix F includes a 200 foot deep boring log performed for the man made mountain project at the case study site.
<table>
<thead>
<tr>
<th>Moisture Content</th>
<th>Sample</th>
<th>Blows N</th>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Graphic</th>
<th>Soil Description</th>
<th>Moisture</th>
<th>Consistency</th>
</tr>
</thead>
<tbody>
<tr>
<td>15.6</td>
<td>119</td>
<td>50/50</td>
<td>16</td>
<td>R</td>
<td></td>
<td>CEMENTED SAND &amp; GRAVEL</td>
<td>dry</td>
<td>very hard</td>
</tr>
<tr>
<td>18</td>
<td>R</td>
<td>50/0'</td>
<td>17</td>
<td>R</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>R</td>
<td></td>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>21</td>
<td>R</td>
<td></td>
<td>22</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>R</td>
<td></td>
<td>25</td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<tr>
<td>26</td>
<td>R</td>
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<td>27</td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>R</td>
<td></td>
<td>29</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>23</td>
<td>R</td>
<td></td>
<td>23</td>
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<td></td>
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<td></td>
<td></td>
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<td>24</td>
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<td></td>
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<td>28</td>
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<td></td>
<td></td>
<td></td>
<td>29</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Notes:** Water encountered at approximately 23 feet. Elevation at 2077.6 feet by others.

**Driving Weight (LBS):** 140

**Project:** LE REVE MOUNTAIN

**Boring Log**

**Western Technologies Inc.**

**Project No.:** 4122JS169

**Plate:** A-17
<table>
<thead>
<tr>
<th>Date Drilled:</th>
<th>1-9-03</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location:</td>
<td>3145 Las Vegas Blvd S.</td>
</tr>
</tbody>
</table>

**Boring No. 2 (Cont'd)**

| Elevation: | 2078 |

<table>
<thead>
<tr>
<th>Soil Moisture Content (% of Dry Wt)</th>
<th>Dry Density (LB/FT³)</th>
<th>Sample Type</th>
<th>Sample</th>
<th>Blowcount</th>
<th>Depth (Feet)</th>
<th>USC</th>
<th>Graphic</th>
<th>Soil Description</th>
<th>Moisture</th>
<th>Consistency</th>
</tr>
</thead>
<tbody>
<tr>
<td>18.7</td>
<td>105</td>
<td>R</td>
<td>20</td>
<td></td>
<td>23.3</td>
<td></td>
<td>SC</td>
<td>CLAYEY SAND</td>
<td>damp</td>
<td>very dense</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>CL</td>
<td>SANDY CLAY-w/grav/ pale brown</td>
<td></td>
<td>very stiff</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>SC</td>
<td>CLAYEY SAND-w/grav/ pale brown &amp; pale red brown</td>
<td></td>
<td>dense</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>CEMENTED SAND &amp; GRAVEL-white-gray-brown-black</td>
<td>dry</td>
<td>very hard</td>
</tr>
</tbody>
</table>

**Notes:**
- Water encountered at approximately 23 feet.
- Elevation at 2077.6 feet by others.

**Driving Weight (LBS):** 140

**Project:** LE REVE MOUNTAIN

**Boring Log**

**Western Technologies Inc.**

**Project No.:** 4122 JS169

**Plate:** A-18
<table>
<thead>
<tr>
<th>MOISTURE (% of dry wt)</th>
<th>DRY DENSITY (lbs/cu ft)</th>
<th>SAMPLE TYPE</th>
<th>BLOW COUNT</th>
<th>DEPTH FEET</th>
<th>USC'S</th>
<th>GRAPHIC</th>
<th>SOIL DESCRIPTION</th>
<th>MOISTURE</th>
<th>CONSISTENCY</th>
</tr>
</thead>
<tbody>
<tr>
<td>29.9</td>
<td>58</td>
<td>R</td>
<td>46</td>
<td></td>
<td>CL</td>
<td>SANDY CLAY-w/gravel, pale gray</td>
<td>moist</td>
<td>very hard</td>
<td></td>
</tr>
<tr>
<td>21.5</td>
<td>88</td>
<td>R</td>
<td>53</td>
<td></td>
<td>CL</td>
<td>pale brown</td>
<td>damp</td>
<td>very stiff</td>
<td></td>
</tr>
<tr>
<td>38.9</td>
<td>26</td>
<td>R</td>
<td>58</td>
<td></td>
<td>CL</td>
<td>red brown</td>
<td>very moist</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:** Water encountered at approximately 23 feet. Elevation at 2077.6 feet by others.

**DRIVING WEIGHT (LBS):** 140

**PROJECT:** LE REVE MOUNTAIN

**BORING LOG**

**PLATE:** A-19

**PROJECT NO.:** 412235169

**WEATHER TECHNOLOGIES INC.**
**BORING NO. 2 (Cont'd)**

**DATE DRILLED:** 1-9-03  
**LOCATION:** 3145 Las Vegas Blvd S.

<table>
<thead>
<tr>
<th>MOISTURE (%)</th>
<th>DRY DENSITY (lbs/ft³)</th>
<th>SAMPLE TYPE</th>
<th>SAMPLE</th>
<th>BLOWS/FT</th>
<th>DEPTH (FT)</th>
<th>USCS</th>
<th>GRAPHIC</th>
<th>SOIL DESCRIPTION</th>
<th>MOISTURE</th>
<th>CONSISTENCY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>CL</td>
<td>very moist</td>
<td>very stiff</td>
</tr>
<tr>
<td>R-36.1</td>
<td>95</td>
<td>R</td>
<td>38</td>
<td>61</td>
<td></td>
<td></td>
<td></td>
<td>SANDY CLAY-w/gravel, pale brown</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:** Water encountered at approximately 23 feet. Elevation at 2077.6 feet by others.

**DRIVING WEIGHT (LBS)**: 140

---

**WESTERN TECHNOLOGIES INC.**

**PROJECT NO.:** 4122J8169

**PROJECT:** LE REVE MOUNTAIN

**BORING LOG**

**PLATE:** A-20
<table>
<thead>
<tr>
<th>SAMPLE</th>
<th>DRY DEN.</th>
<th>LC</th>
<th>SOIL DESCRIPTION</th>
<th>MOISTURE</th>
<th>CONSISTENCY</th>
</tr>
</thead>
<tbody>
<tr>
<td>76</td>
<td>77</td>
<td>CL</td>
<td>SANDY CLAY-w/gravel, lt. brown</td>
<td>very moist</td>
<td>very stiff</td>
</tr>
<tr>
<td>27.4</td>
<td>97</td>
<td>R</td>
<td>-w/sparse gravel, red brown</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>88</td>
<td>R</td>
<td>CALICHE-lt. brown-brown</td>
<td>dry</td>
<td>very hard</td>
</tr>
</tbody>
</table>

**NOTES:** Water encountered at approximately 23 feet. Elevation at 2077.6 feet by others.

**DRIVING WEIGHT (LBS):** 140

**PROJECT:** LE REVE MOUNTAIN

**BORING LOG**

**PLATE:** A-21

**PROJECT NO.:** 4122JS169
<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Soil Description</th>
<th>Moisture</th>
<th>Consistency</th>
</tr>
</thead>
<tbody>
<tr>
<td>91-92</td>
<td>CALICHE</td>
<td>dry</td>
<td>very hard</td>
</tr>
<tr>
<td>93-94</td>
<td>SANDY CLAY-w/gravel, red brown</td>
<td>damp</td>
<td>very stiff</td>
</tr>
</tbody>
</table>

**Notes:** Water encountered at approximately 23 feet. Elevation at 2077.6 feet by others.

**Driving Weight (lbs):** 140
**DATE DRILLED:** 1-9-03  
**LOCATION:** 3145 Las Vegas Blvd S.  
**BORING NO. 2 (Cont'd)**  
**ELEVATION:** 2078  

<table>
<thead>
<tr>
<th>MOISTURE</th>
<th>CONSISTENCY</th>
<th>SOIL DESCRIPTION</th>
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<tr>
<td>CL</td>
<td>DAMP</td>
<td>SANDY CLAY</td>
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<tr>
<td>SC</td>
<td>VERY STIFF</td>
<td>CLAYEY SAND-red brown</td>
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<td>VERY DENSE</td>
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<td>SANDY CLAY-gravel, red brown</td>
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**NOTES:** Water encountered at approximately 23 feet.  
Elevation at 2077.6 feet by others.  

**WESTERN TECHNOLOGIES INC.**  
**PROJECT NO.:** 4122JS169  
**PROJECT:** LE REVE MOUNTAIN  
**BORING LOG**  
**PLATE:** A-23  

**DRIVING WEIGHT (LBS):** 140
<table>
<thead>
<tr>
<th>DEPTH (FEET)</th>
<th>COHESIONLESS (BS styl.)</th>
<th>MOISTURE TYPE</th>
<th>CONSISTENCY</th>
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<tbody>
<tr>
<td>CALICHE-w/clay lenses, lt. brown</td>
<td>damp</td>
<td>very hard</td>
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<td>SANDY CLAY-red brown</td>
<td>dry</td>
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<tr>
<td>CEMENTED SAND &amp; GRAVEL</td>
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<tr>
<td>CALICHE-w/clay lenses</td>
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**Notes:**
Water encountered at approximately 23 feet. Elevation at 2077.6 feet by others.
<table>
<thead>
<tr>
<th>SAMPLE</th>
<th>BRITTLETY (BKRSFT)</th>
<th>SAMPLE</th>
<th>BLOWNTRY (BKRSFT)</th>
<th>DEPTH (FEET)</th>
<th>GRAPHIC</th>
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<td>SANDY CLAY-w/gravel, pale red brown</td>
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NOTES: Water encountered at approximately 23 feet. Elevation at 2077.6 feet by others.

DRIVING WEIGHT (LBS) 140

PROJECT: LE REVE MOUNTAIN
BORING LOG
**BORING NO. 2**

**DATE DRILLED:** 1-9-03  
**LOCATION:** 3145 Las Vegas Blvd S.  
**ELEVATION:** 2078

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<th>SAMPLE TYPE</th>
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<th>USCS</th>
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<th>MOISTURE</th>
<th>CONSISTENCY</th>
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</table>

**SPT:** STANDARD PENETRATION TEST  
**R:** RING SAMPLE  
**C:** CORE: %RECOVERY/ %RQD  
**B:** BAG  
**BN:** BULL NOSE

**NOTES:** Water encountered at approximately 23 feet. Elevation at 2077.6 feet by others.

**DRIVING WEIGHT (LBS):** 140

**PROJECT:** LE REVE MOUNTAIN

**BORING LOG**

**PLATE:** A-26

**PROJECT NO.:** 4122JS169

**WESTERN TECHNOLOGIES INC.**
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<thead>
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<th>LOCATION: 1-9-03 3145 Las Vegas Blvd S.</th>
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</table>

- Water encountered at approximately 23 feet. Elevation at 2077.6 feet by others.

WESTERN TECHNOLOGIES INC.
PROJECT: LE REVE MOUNTAIN
BORING LOG
PROJECT NO. 4122JS169
PLATE A-27
ELEVATION: 2078

NOTES:
Water encountered at approximately 23 feet. Elevation at 2077.6 feet by others.

PROJECT: LE REVE MOUNTAIN
BORING LOG

DATE DRILLED: 1-9-03
LOCATION: 3145 Las Vegas Blvd S.
BORING NO. 2 (Cont'd) ELEVATION: 2078

SPT: STANDARD PENETRATION TEST
R- RING SAMPLE
C- CORE: %RECOVERY/%RQD
B- BAG
BN- BULL NOSE

WESTERN TECHNOLOGIES INC.

PROJECT NO. 4122JS169
PLATE A-28

MOISTURE CONTENT (% by Wt.) UNCONSOLIDATED DENSITY SAMPLE TYPE SAMPLE BOREHOLE DEPTH FEET USC SOIL DESCRIPTION MOISTURE CONSISTENCY

CLAYEY SAND-gravel, red brown moist very dense

CEMENTED SAND & GRAVEL-brown-grey-black dry very hard

within clay lenses

DRIVING WEIGHT (LBS) 140
<table>
<thead>
<tr>
<th>DATE DRILLED: 1-9-03</th>
<th>LOCATION: 3145 Las Vegas Blvd S</th>
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<th>ELEVATION: 2078</th>
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</thead>
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<tr>
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<td>SAMPLE TYPE</td>
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<tr>
<td>199</td>
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**NOTES:** Water encountered at approximately 23 feet. Elevation at 2077.6 feet by others.

DRIVING WEIGHT (LBS) 140

PROJECT: LE REVE MOUNTAIN
BORING LOG

PLATE A-29
APPENDIX G

MATLAB CODE FOR PILE GROUP SETTLEMENT

Appendix G includes a Matlab program to calculate pile group displacement for a given load and pile configuration.

```matlab
% Pile Group Elastic soln.
% Based on Chow (1986)
% Pile-pile effects by Mindlin pt load soln;
% 1/09 Includes stiffness increase w/ depth
% 6/09 iterates displacement to give the desired cap load

clear all
close all;

npile = 4;  % no. of piles
pload=200;  %kip
sx = 6;  %pile x dir spacing, ft.
sy = 6;  %pile x dir spacing, ft.
sp=3.5;  %pile cap edge clearance, ft
t=4;  %cap thickness
np_sidey=2;  % square np_side x np_side group
np_sidey=2;

Es=1000;  %soil, ksf
vs=0.35;  %nu, soil
grate=0;  %ksf/ft increase in Es

Er=6e5;  %E - raft
L = 20;  % m or ft.
nel=20;  % no. elements for each pile
B = 2;  % diameter, m or ft
Ep = 6e5;  %E pile, ksf

I = pi*B^4/64;  % circle, m^4 or ft^4
A = pi*0.25*B^2;
g=.5*Es/(1+vs);
ro=.5*B;

%prescribed disp - for rigid pile cap effect
% intial disp guess
dd = 0.2;  %in
dd=-dd/12;  %ft.
%desired group load
pgrp=npile*pload;  %kip

if npile==1; igroup=0; else igroup=1; end
gem=zeros(npile,2);
gem(1,1)=0; gem(1,2)=0;
```
x=[0:sx:sx*(np_sidey)];
y=[0:sy:sy*(np_sidey)];

m=0;
for i=1:np_sidey
    for j=1:np_sidex
        m=m+1;
        geom(m,1)=x(j);
        geom(m,2)=y(i);
    end
end
if npile==1; geom=[0 0]; end
if npile==2; geom=[0 0; sx 0]; end
if npile==3; geom=[0 0; sx 0; .5*sx sx*sqrt(.75)]; end
a=2*sp+max(geom(:,1));
b=2*sp+max(geom(:,2));
Kr=4*Er*(1-.15^2)*b^t^3/(3*pi*Es*a^4);
r=sqrt((a*b)/pi);
Kr=(Er/Es)*(t/r)^3;
nnode=nel+1;
lng=L/nel;len=zeros(nel,1); len(:,1)=lng;
qinc=1;
qinc=qinc(:);
nloads=length(qinc);

%build stiffness matrix

d=zeros(nnode,1);
ff=zeros(nnode,1);
bload=zeros(nnode,1);
disp=zeros(nnode,npile);
f=zeros(nnode,1);
sx=zeros(nel,1);
sx1=zeros(nel,1);
sx1=zeros(nel,1);
sumf=0;
p=zeros(nel,2);
p(1,1)=1; p(1,2)=2;
for i=2:nel
    for j=1:2
        p(i,1)=p(i-1,2);
        p(i,2)=p(i,1)+1;
    end
end

if dd~=0
iter=0;
tolerr=5;
err=1e5;
while abs(err)>tolerr
iter=iter+1;
k=1;
kk=zeros(nnode*npile,nnode*npile);
for mm=1:nel
    index=p(mm,:);
    leng=len(mm);
    ke=buildpilistiff(Ep,A,leng);
    kk=feasmbl1(kk,ke,index);
end
for km=2:npile
    for i=1:nnode
        for j=1:nnode
            m=(km-1)*nnode+i; mm=(km-1)*nnode+j;
            kk(m,mm)=kk(i,j);
        end
    end
end
%

%if npile>1
% add group effects
[fd,kd]=mindlin(geom,Es,vs,nel,leng,npile,B,grate,igroup);
kk=kk+k;
%end
%

fi=qinc(k);
ff=zeros(nnode*npile,1);
ff(1)=fi;
mp(1,1)=1;
% add prescribed displacement
if abs(dd)>0; kk(1,1)=kk(1,1)+1e20; ff(1)=dd*kk(1,1); end
for i=2:npile
    m=1+(i-1)*nnode;
    mp(i,1)=m;
    ff(m,1)=fi;
    if abs(dd)>0; kk(m,m)=kk(m,m)+1e20; ff(m,1)=dd*kk(m,m); end
end
%

% solve stiffness matrix
dk=kk\ff;
d=zeros(nnode,npile);
m=1;
for i=1:npile
    d(1:nnode,i)=dk(m:m+nel,1);
m=m+nnode;
end

disp=disp+d;
for i=1:npile
    sigma=0; sx(:)=0;
    for mm=1:nel
        e1=d(mm,i); e2=d(mm+1,i);
        eps=(e1-e2)/len(mm);
        sigma=Ep*eps;
        sx(mm)=sx(mm)+sigma;
        fel(mm,k,i)=A*sx(mm); % force at each node with depth
    end
end

fem=zeros(nel,npile);
for j=1:npile
    for i=1:nel
        dd1=[d(i,j);d(i+1,j)];
        fe=ke*dd1;
        fem(i,j)=fe(1);
    end
    sumfe(j,1)=sum(fem(:,j));
end

% recover spring forces
df=kd*dk;

forcep=zeros(nnode,npile);
m=1;
for i=1:npile
    forcep(1:nnode,i)=df(m:m+nel,1);
m=m+nnode;
end
i=1:npile;
sump(i)=sum(forcep(:,i));
pile_load=sump;
group_load=-sum(sump)

pg(iter+1,1)=group_load;
pgw(iter+1,1)=dk(1)*12;
err=group_load-pgrp;
if err>0; dd=dd+(.002/12); elseif err<0; dd=dd-(.002/12); end
end
elseif dd==0;
k=1;
kk=zeros(nnode*npile,nnode*npile);
for mm=1:nel
    index=p(mm,:);
    leng=len(mm);
    ke=buildpilestiff(Ep,A,leng);
    kk=feasmbll(kk,ke,index);
end
for km=2:npile
for i=1:nnode
    for j=1:nnode
        m=(km-1)*nnode+i; mm=(km-1)*nnode+j;
        kk(m,mm)=kk(i,j);
    end
end
end

% if npile>1
% add group effects
[fd,kd]=mindlin(geom,Es,vs,nel,leng,npile,B,grate,igroup);
kk=kk+kd;
% end
% --------------------------------------------------

fi=qinc(k);
ff=zeros(nnode*npile,1);
% if dd>0; fi=dd*1e20; end

ff(1)=fi;
mp(1,1)=1;
if abs(dd)>0; kk(1,1)=kk(1,1)+1e20; ff(1)=dd*kk(1,1); end
for i=2:npile
    m=1+(i-1)*nnode;
    mp(i,1)=m;
    ff(m,1)=fi;
    if abs(dd)>0; kk(m,m)=kk(m,m)+1e20; ff(m,1)=dd*kk(m,m); end
end

% solve stiffness matrix
dk=kk\ff;

d=zeros(nnode,npile);
m=1;
for i=1:npile
    d(1:nnode,i)=dk(m:m+nel,1);
    m=m+nnode;
end

disp=disp+d;
for i=1:npile
    sigma=0; sx(:)=0;
    for mm=1:nel
        e1=d(mm,i); e2=d(mm+1,i);
        eps=(e1-e2)/len(mm);
        sigma=Ep*eps;
        sx(mm)=sx(mm)+sigma;
    fel(mm,k,i)=A*sx(mm); % force at each node with depth end
end
fem=zeros(nel,npile);
for j=1:npile
for i=1:nel
    ddi=[d(i,j);d(i+1,j)];
    fe=ke*ddi;
    fem(i,j)=fe(1);
end
sumfe(j,1)=sum(fem(:,j));
end

% recover spring forces
df=kd*dk;

forcep=zeros(nnode,npile);
m=1;
for i=1:npile
    forcep(1:nnode,i)=df(m:m+nel,1);
m=m+nnode;
end
i=1:npile;
sump(i)=sum(forcep(:,i));
pile_load=sump;
group_load=-sum(sump)
end

%forcep
%get stiffness for each pile
for i=1:npile
    Pgrw(i,1)=sump(i)/(g*ro*dk(mp(i)));
    stiffness(i,1)=sump(i)/dk(mp(i));
end
Pgrw
K=stiffness/12
dd*12

sumf=sumf+sum(df);
pav=sumf*npile;
for i=1:npile
    pload(i,1)=sump(i)/sumf;
    avload(i,1)=sump(i)/pav;
%iterstep(k,1)=iter;
topdisp(i,1)=disp(1,i);
end

% -----------------------------------------------------

n=length(topdisp);
ds=topdisp*12; % get d in inches
%fs=-fs/2;
for i=1:n
    dx(i+1,1)=ds(i);
    fx(i+1,1)=-pile_load(i);
dx(1,1)=0; fx(1,1)=0;
end
figure(1)
hold on
for i=1:npile
ffx=[0 fx(i+1)]; ddx=[0 dx(i+1)];
plot(ffx,-ddx);
% axis([0 max(fx) 0 max(dx)]);
end
hold off
grid
ylabel('Deflection, in')
xlabel('Load, kips')

x(1)=leng/2;
for i=2:nel
    x(i)=x(i-1)+leng;
end

% -------------------------------------
xx(1,1)=0;
for i=2:nel+1
    xx(i,1)=xx(i-1,1)+leng;
end

fem2=zeros(nnode,npile);
fem2(1,1:npile)=pile_load;
fem2(2:nnode,1:npile)=fem;

figure(2)
hold on
for i=1:npile
    plot(-fem2(:,i),-xx);
end
grid on
hold off
ylabel('Depth, ft.')
xlabel('Load, kips')

figure(3)
hold on
plot([0 0,0],'.')
line([0 geom(npile,1)+2*sp],0)
line([0 0],[0 geom(npile,2)+2*sp])
line([0 geom(npile,1)+2*sp]
geom(npile,2)+2*sp)
line([0 geom(npile,1)+2*sp
gem(npile,2)+3*sp],geom(npile,1)+2*sp,0])
%plot([geom(npile,1)+sp,geom(npile,2)+sp,']'.)
for i=1:npile
    xx=geom(i,1)+sp;
yy = geom(i, 2) + sp;
plot(xx, yy, '*')
str = num2str(-sump(i));
text(xx, yy + 0.3, str, 'FontSize', 7);
end
axis equal

figure(4)
plot(-pgw, pg)
grid on
ylabel('Group Load, k')
xlabel('Deflection, in')

%-----------------------------------------------------------

function [ke]=buildpilestiff(e,a,l)
ke(1,1)=e*a/l;
ke(1,2)=ke(1,1);
ke(2,1)=ke(1,2);
ke(2,2)=ke(1,1);

function [kk]=feasmbl1(kk,k,index)
% from Kwon & Bang book, 1997
%----------------------------------------------------------
% Purpose:
% Assembly of element matrices into the system matrix
% Synopsis:
% [kk]=feasmbl1(kk,k,index)
% Variable Description:
% kk - system matrix
% k - element matrix
% index - d.o.f. vector associated with an element
%----------------------------------------------------------
edof = length(index);
for i=1:edof
    ii=index(i);
    for j=1:edof
        jj=index(j);
        kk(ii,jj)=kk(ii,jj)+k(i,j);
    end
end
%----------------------------------------------------------

function [f,kd]=mindlin(geom,Es,vs,nel,leng,npiles,B,grate,igroup)
% reference: Smith & Griffiths (1988)
warning off MATLAB:divideByZero;
t=0;
tmax=1e10;

if igroup==0; f=zeros(npiles,npiles); kd=0; return; end

rho=1;
ro=B/2;
L=leng*(nel);
rm=2.5*rho*L*(1-vs);
rf=.9;
go=.5*Es/(1+vs);
zz(1,1)=0;
    for i=2:nel+1
        zz(i,1)=zz(i-1,1)+leng;
    end
n=nel+1;
f=zeros(n*npiles,n*npiles);

m=0;
for i=1:npiles
    for j=1:n
        m=m+1;
        p(m)=m;
    end
end

pk=zeros(npiles,npiles);
for i=1:npiles
    pk(i,1:npiles)=1:npiles;
end
for i=2:npiles
    pk(i,1)=i;
pk(i,i)=1;
end
if npiles>2;
    for i=3:npiles
        pk(i,i-1)=i-1;
    end
end

m=length(geom);
rr(1)=0;
for i=1:m
    %x1=geom(i,1);
    %y1=geom(i,2);
    for j=1:m
        x=geom(j,1)-geom(i,1);
y=geom(j,2)-geom(i,2);
        rr(i,j)=sqrt(x^2+y^2);
    end
end
% z=depth of nodes for pile where disp. eval. - i
% geom(1,1:2)=pile for which displacements are evaluated
% c=depth where P is applied - j

for i=1:n
    f(i,i)=1;
end
pp=0;
pn=0;
for k=1:npiles

    pp=pp+1;
    for i=1:n
        m=m+1;
        if i>n; ii=i-n; z=zz(ii); else z=zz(i); end
        z=zz(i); gsoili=go+grate*z;
        if i==1; gsoili=go+grate*.25*leng; end
        if i==nel; gsoili=go+grate*(z+.25*leng); end
    end

    m=0;
pn=pn+1;
    for kk=1:npiles
        kx=pk(k,kk);
        x=geom(kx,1)-geom(1,1);
        y=geom(kx,2)-geom(1,2);
        r=sqrt(x^2+y^2);
        r=rr(k,kk);

        for j=1:n
            m=m+1;
            c=zz(j); gsoilj=go+grate*c;
            if j==1; gsoilj=go+grate*.25*leng; end
            if j==nel; gsoilj=go+grate*(c+.25*leng); end

            r1=sqrt(r^2+(z-c)^2);
            r2=sqrt(r^2+(z+c)^2);
            g=.5*(gsoili+gsoilj);

            f(pn,m)=(1/(16*pi*g*(1-vs)))*((1/r1)*(3-4*vs)+(1/r2)*(8*(1-vs)^2-(3-4*vs))^2...+ (1/r1^3)*(z-c)^2+(1/r2^3)*((3-4*vs)*(z+c)^2-2*c*z)+(1/r2^5)*6*z*c*(z+c)^2);
            if i<n & j<n; f(i,j)=0; end
            if i>n & j>n; f(i,j)=0; end
end
end

end end
p=0;
for i=1:npiles
    for j=1:n
        j=j+(i-1)*n;
        for k=1:n
            k=k+(i-1)*n;
            f(j,k)=0;
        end
    end
end

if igroup==0; f=zeros(n*npiles,n*npiles); end

% add soil springs
for i=1:npiles
    gsoil=go+grate*.25*leng;
    xi=formxi(t,tmax,rf,rm,ro);
    m=i*n-(n-1);
    f(m,m)=f(m,m)+ xi/(pi*gsoil*leng);
    gsoil=go+grate*(zz(nel)+.25*leng);
    xi=formxi(t,tmax,rf,rm,ro);
    f((i*n)-1,(i*n)-1)=f((i*n)-1,(i*n)-1)+ xi/(pi*3*gsoil*leng);
    for j=2:n-2
        gsoil=go+grate*zz(j);
        xi=formxi(t,tmax,rf,rm,ro);
        j=j+(i-1)*n;
        f(j,j)=f(j,j)+ xi/(2*pi*gsoil*leng);
    end
end

%base
for i=1:npiles
    gsoil=go+grate*zz(n);
    f(n*i,n*i)=(1-vs)/((4*gsoil*ro)*(1-rf*t/tmax)^2);
end

kd=inv(f);
return

%---------------------------------------------
BIBLIOGRAPHY


American Society of Civil Engineers (ASCE). (1994). *Settlement Analysis*, Technical Engineering and Design Guides as Adapted from the US Army Corps of Engineers, No. 9, ASCE Press, NY.


Geotechnical Engineering, ASCE, 111(6), 730-749.


Kaderabek, T. J., & Reynolds, R. T. (1981). Miami limestone foundation design and 
construction. Journal of the Geotechnical Engineering Division, ASCE, 
127(GT7), 859-872.

Kleinfelder, Inc. (KI, 1996). I-15/US 95 Load Test Program, Project No. 31-215903- 
07A. Clark County, Las Vegas, Nevada.

Kleinfelder, Inc. (KI, 2001). Geotechnical Investigation, Proposed Desert Inn Hotel and 
Casino, Project No. 31-340501. Clark County, Las Vegas, Nevada.

Proc. On Conf. on Design and Performance of Deep Foundations: Piles and 
Piers in Soil and Soft Rock, ASCE Geotechnical Special Publication No.38, 172- 
183.

Ed. G. Chen et al., Anchorage.

Ladyani, B. (1977). Discussion of friction and end bearing tests on bedrock for high 
capacity socket design, by P. Rosenberg and N. L. Journeaux. Canadian 
Geotechnical Journal, 13, 324-333.

Langan Engineering & Environmental Services. (2006). Geotechnical Engineering Study, 
Fontainebleau Casino and Resort, Project No. 6114301. Clark County, Nevada.

settlement analysis in multiplayer soils. Canadian Geotechnical Journal, 38, 
1063-1080.

model and back analysis for flexible earth retaining walls. Computers and 
Geotechnics, 31(6), 457-472.

International Journal for Numerical and Analytical Methods in Geomechanics, 
18(1), 25-47.


Reynolds, R.T., & Kaderabek, T.J. (1980). Miami limestone foundation design and construction, Reprint No. 80-546, South Florida Convention, ASCE.


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