Geotechnical Design and Construction Aspects of the CityCenter

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The MGM MIRAGE CityCenter, currently under construction in Las Vegas, Nevada, is by many accounts the largest privately funded development project in the United States, with an estimated cost of $7.4 billion. This 76-acre site along the Las Vegas Strip includes more than 18 million square feet of retail, residential, hotel, and casino space, along with an elevated people mover, convention center, and Cirque de Soleil theatre. There are six high-rise towers ranging from 40- to 61-stories, with a maximum tower height of 650 feet, as well as several large-footprint parking garages, a 10-story central plant facility, and numerous low-rise retail buildings.

The development's signature structure, the Pelli Tower, is situated at the north-central end of the site, and is 1,000 feet in length along its primary axis and 80 to 120 feet wide. In addition to its central core, the tower's four wings range in height from 25 to 52 stories. Total service loads for the towers range from 100,000 to approximately 470,000 kips.

Foundation design and construction challenges have included the Las Vegas valley's unique geologic conditions, the intensity of foundation loading for the high-rise towers and parking garages, the disparity in loading between the high-rise and adjacent low-rise structures, and construction sequencing.

The foundation system for the development includes over 1,000 large-diameter, deep-drilled shaft elements for the high-rise and parking portions of the structure, and spread and mat foundations for lightly loaded parts of the structure.

Geotechnical Investigation

The design firm conducted a geotechnical exploration program to obtain adequate data within the anticipated "zone of influence" of the Pelli Tower foundations. This involved advancing over 125 borings to depths ranging from 25 to 250 feet below ground surface (BGS) using hollow-stem auger and mud-rotary drilling equipment. The selected depth of 250 feet is approximately twice the width of the towers; increases to the effective stress in the soil below this depth were not anticipated.

The design firm performed pressure-meter testing (PMT) and shear wave velocity measurements in the 250-foot deep borings within the tower footprint. PMT consists of placing a cylindrical probe in a pre-drilled hole, expanding the probe at discrete intervals of volume, and measuring pressure in the probe. The readings provide a cavity stress-radial strain response within the test zone. From the last few readings of the test, the design firm estimated a limit pressure and calculated a pressuremeter modulus. In turn, this data determined the shear strength and compressibility of the tested soil. Based on the results of the testing, the undrained shear strength of the soils ranged from 1.8 to 5.0 ksf, and pressuremeter soil moduli ranged from 235 to 925 ksf. Strength and compressibility data was used directly in the foundation design and settlement analysis.

Suspension P-S-velocity logging established the seismic shear-wave velocity measurements. P-S logging uses a 7-meter probe that contains a source and two receivers. The probe is lowered down the drilled

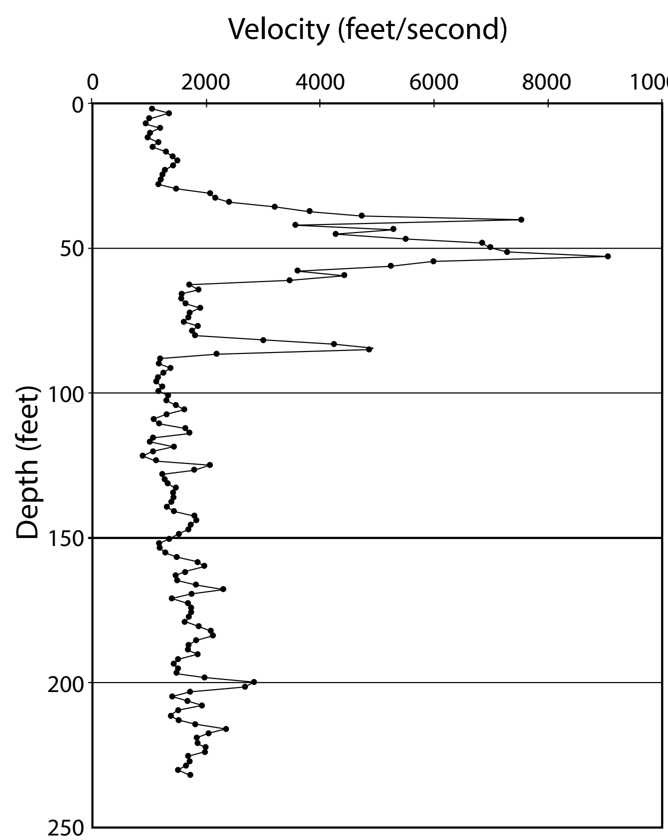

Figure 1a: Typical Shear Wave Velocity Profile.

Rendering of Project CityCenter.

hole, where the source generates a pressure wave in the drilling fluid within the hole. The pressure wave is converted to seismic P- and S-waves at the boring sidewalls and, at each receiver, the P- and S-waves are converted back to pressure waves. The elapsed time between wave arrivals at the receivers is used to determine the average velocity of a 1-meter-high column of soil. This process is repeated over the full depth of the boring to obtain a continuous log of the boring. The average shear wave velocity of the on-site soils provides a representation of the stiffness of the soil profile, which influences the determination of seismic design forces (base shear) in the IBC procedure. The average shear wave velocity can also be used directly in the development of site-specific response spectra. A typical shear wave velocity profile is shown in *Figure 1a*.

Based on the results of shear wave velocity measurements, it was determined that the average shear wave velocity for the on-site soils is approximately 1,600 feet (490 meters) per second. The upper gravelly and cemented soils, which have a relatively high shear wave velocity, are easily distinguished from the lower clayey soils, which have a relatively low shear wave velocity. Laboratory testing on selected samples focused primarily on defining the strength (direct shear, triaxial shear, unconfined compressive strength) and consolidation (one-dimensional consolidation testing) characteristics of the site's soils.

Geologic Conditions

The site is located in the central portion of the Las Vegas Valley, a broad sedimentary basin filled with primarily Quaternary alluvial deposits. Local tectonic subsidence, coupled with the active deposition of alluvial materials, resulted in a thick sequence of sediments underlying the central valley, which can be several thousand feet thick.

The upper native materials at the site consist of inter-fingering lenses of strongly cemented clayey gravel and sandy gravel; caliche (secondary calcium carbonate deposits); and soft to stiff clay, sandy clay, silty clay, and silt. The base of this shallow strongly cemented zone is highly variable and ranges in depth from approximately 55 to 65 feet beneath the proposed Pelli tower, and 20 to 55 feet beneath other structures at the site. The thickness of the strongly cemented units ranges from approximately 1 to 25 feet.

Beneath the shallow cemented zone, the soil units generally consist of soft-to-stiff clay along with some layers of sand or silty sand with varying degrees of cementation. Locally, thin discontinuous lenses of strongly cemented sand and gravel and/or caliche 5 to 20 feet thick were encountered within the deeper sediments. In contrast, very soft deposits were encountered in limited areas beneath the upper cemented zone. The very soft soil was typically at depths ranging from approximately 60 to 120 feet BGS, but also at 160 feet BGS.

Based on observations during drilling and laboratory test results, the sediments at the site below a depth of 60 feet were considered under-consolidated. There are several theories as to why this is the case. One theory is that the overlying caliche layers may be reducing the intensity of overburden loading by distributing stresses differently to the underlying soils. The hard, cemented materials that overlie these soft sediments locally connect in an interlocking lattice and, at specific depths, appear to extend laterally for sufficient distances to form a thick extensive layer, capping the softer, fine-grained materials, which could explain why normal consolidation has not occurred.

Field and Lab Data

The primary purpose of the field explorations was to develop generalized soil profiles (*Figure 1b*), which were then used in designing the foundations and evaluating foundation settlement under the loading prescribed by the project's structural engineer. To develop the general soil profile, several data sets were used to laterally and vertically correlate the sediments beneath the site. Composition of the materials encountered in the borings was the primary characteristic used for stratigraphic correlation and the subsequent development of geologic

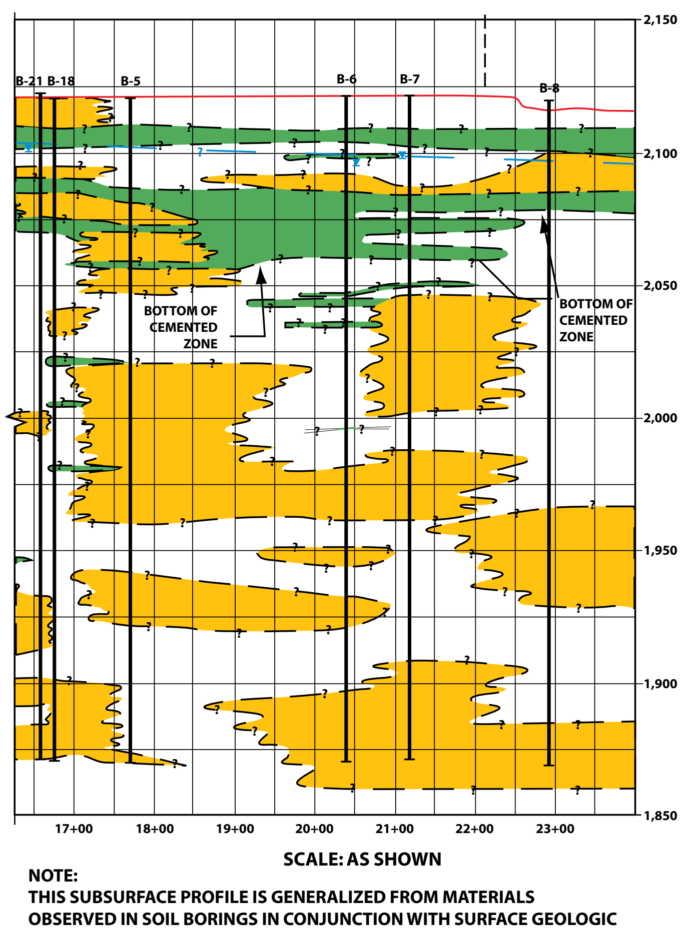

Figure 1b: Generalized Soil Profile.

Osterberg Load Cell.

cross sections. A secondary means of correlation included comparing field data such as sample blow counts and drill advance times with the data from the shear wave velocity profiling.

Typically, the majority of the engineering values obtained from lab data were tabulated by soil type and relative density or consistency. Where information was not available, the values were interpolated and/or estimated. Due to the difficulty in obtaining and testing samples, the engineering values for caliche, cemented sand and gravel, and very hard silt or clay layers were estimated based on the shear wave velocity.

Estimated and interpolated data allowed the assignment of engineering values to basically all layers within a boring where the soil type and relative density or consistency matched. This provided continuous engineering data when analyzing any single boring, specifically for settlement analysis and foundation design.

Deep Drilled Shaft Foundation Design and Settlement Analysis

Initial shaft design capacities were developed using conventional geotechnical design methods with both commercially available and internally developed software. The primary assumption made in the analysis was that load transfer would be entirely through side friction.

Osterberg Load Cell Testing

To further refine drilled shaft capacities, supplemental full-scale foundation shaft load testing was performed on two test shafts within the tower footprint. Each test shaft was 4 feet in diameter and drilled to a depth of 120 feet BGS. Load testing was administered by Load Test, Inc. of Gainesville, Florida.

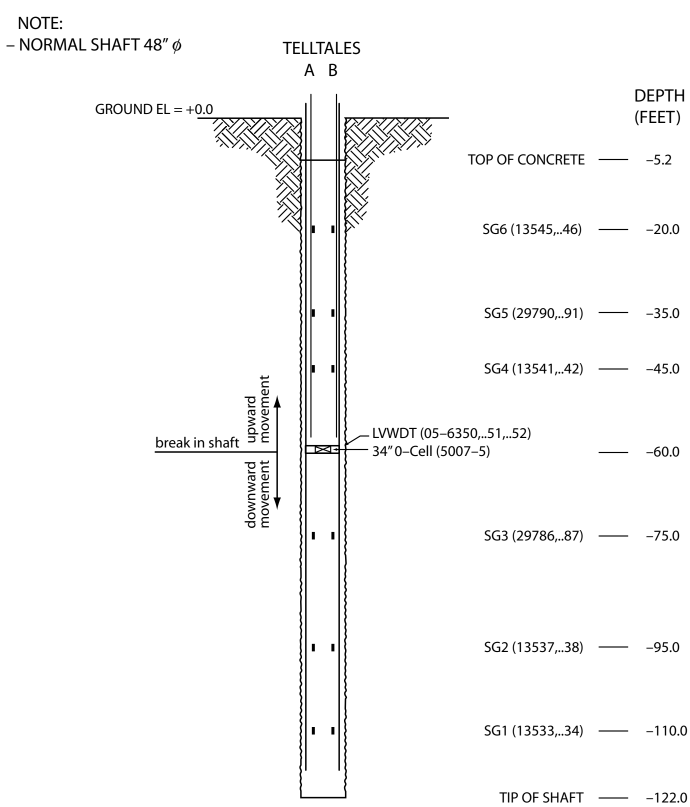

Figure 2: Osterberg Load Cell Test Shaft.

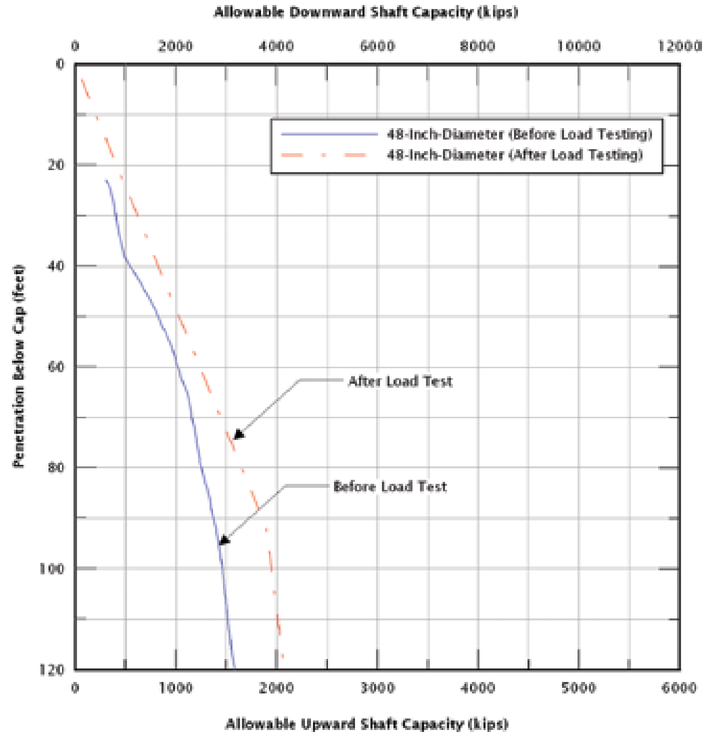

Figure 3: Revised Shaft Capacities.

In each test shaft, a pre-fabricated light-weight steel frame was used for installing strain gauges and associated wiring within the test shafts, instead of a full-scale reinforcement cage. Six pairs of strain gauges were installed at discreet depths along each shaft in an attempt to isolate the contribution of each soil layer to the total capacity.

The Osterberg load cell was installed at the midpoint of the pre-fabricated steel frame, which when set in the drilled shaft at mid-length, approximately 60 feet BGS. Concrete was placed to fill the shaft and allowed to cure prior to the initiation of the load testing. Each load cell consisted of hydraulic jacks sandwiched between 34-inch diameter, 1-inch-thick steel-bearing plates. The locations of strain gauges are shown in *Figure 2*.

The load testing involved gradually increasing the pressure within the load cell until the upper or lower portion of the shaft failed. Failure in this case was defined as movement of ½ inch or more. In each case, the upper length of the test shaft failed. The design firm used data from the strain gauges to develop an estimate of the ultimate side-resistance of the soils in each zone. These values included the computation of revised shaft capacities, which were typically 33% higher than the original shaft capacities (*Figure 3*).

The results indicated the subject shafts had ultimate capacities of 8,400 to 9,400 kips, respectively, and confirmed that all loading applied was dissipated in side friction, as significant strain was not registered in the lowest strain gauge for each shaft. Though the cost for the load testing was significant, the client benefited considerably by realizing a substantial reduction in the cost for the drilled foundation shafts.

Tower-Settlement Analysis

In contrast to a spread or mat foundation, where the stress distribution is generally well defined and well understood, a primary challenge in evaluating the settlement of drilled shafts is estimating the load transfer to the foundation soils. For Project CityCenter, the presence of widely variable foundation soils, primarily the cemented caliche and the under-consolidated fine-grained deposits, added another level of complexity to developing confident estimates of foundation settlement.

To evaluate the settlement of the tower foundations due to the anticipated loading, the distribution of the applied load through the caliche was varied at 2:1 and 3:1 (horizontal to vertical), instead of the more traditional Bosinesq distribution, which would generally be 1:1.

A secondary challenge involved identifying the depth within the foundation soil profile along the shaft length of where to actually apply the foundation load. After evaluating several scenarios, it was determined to be most appropriate to apply the foundation loading at the bottom of the caliche layer. In this manner, the intensity of the load applied to the underlying compressible soils was decreased, while the area of influence of the load was increased.

Construction Considerations

At Project CityCenter, the construction of high-rise towers adjacent to low-rise structures presents logistical challenges related to the potential for differential settlement between structures.

In most cases, the potential for differential settlement was lessened significantly due to the influence of the tower foundations extending 1 to 2 times the foundation width beyond the footprint. Thus, the settlement due to the tower loading will gradually transition beyond the tower footprint and greatly minimize the potential for significant differential settlement. In some cases, however, construction joints were considered to accommodate potential differential movement.

Key Design Considerations

- The unique and highly variable geologic conditions and the wide-range of structure-types for Project CityCenter presented several challenges to the design and construction of the subject development.
- Data from the full scale load testing allowed a more precise evaluation of the varying soil types' contributions to shaft capacity, as well as a more realistic estimated stress distribution through the stiffer soil layers.
- The use of shear wave velocity data and pressuremeter test results as a representation of soil stiffness provided more precise estimates of the compressibility of the foundation soil.
- Construction joints were generally not required to accommodate differential settlement due to the relatively large lateral influence of the towers.▪

As part of the geotechnical exploration program, more than 125 borings were advanced to depths ranging from 25 to 250 BGS.

*GeoDesign, Inc. provided geotechnical design services for Blocks A and C, which represents 80% of the project. Christopher Zadoorian, the principal geotechnical engineer in GeoDesign's Anaheim office, is the firm's project manager and lead design engineer for Project CityCenter. Susan Kirkgard, associate engineering geologist, has been the firm's lead geologist on the project, while Jaime Albornoz has served as project engineer. To contact Chris Zadoorian directly, send an email to **czadoorian@geodesigninc.com**.*