SELMON EXPRESSWAY, TAMPA: CASE HISTORY OF DRILLED SHAFT DESIGN FOR EXTREME VARIABILITY

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The Selmon Expressway includes an elevated structure extending across several miles of weathered karstic limestone geology, and represents a challenge for design because of the high variability of the material and the difficulty in characterizing the strength. The nature of the epikarst in this region of Tampa is such that variability can be extreme within distances of only a few feet. The weathered rock includes extremely soft soil infill within solution features on and within the formation. The limestone is so weathered that coring typically does not yield useful samples for strength testing, and in-situ Standard Penetration Testing (SPT) tests often encounter refusal blow counts. Thus the challenge for foundation design includes two components of reliability risks: the uncertainty associated with the ground conditions at a given foundation location plus the uncertainty associated with the foundation performance at a given location even when the stratigraphy is determined.

This paper describes the design approach that has been implemented successfully for two projects at the Lee Roy Selmon Crosstown Expressway (LRS) in Tampa, Florida: the assessment and remediation of the reversible elevated lane (REL) foundations constructed in 2004, and the elevated expressways widening project constructed in 2012. Included in this design approach is a method developed for computing axial resistance in this local geology. Design for axial loading relies primarily on the development of side resistance, which was considered less subject to performance risks than base resistance. The calculation method incorporates a statistical analysis of SPT data, tempered by engineering judgment, to characterize stratigraphy at a given location into either soil, intermediate geomaterial (IGM) or rock.

Introduction

The subsurface stratigraphy of sites underlain by weathered limestone formations can be highly variable with respect to material layer thicknesses and strengths, even over short horizontal distances. This is the case for much of Florida, and particularly the Tampa area. These conditions present a challenging environment for the design of drilled foundations because of the need for a reliable determination of subsurface conditions at each foundation location, as they relate to the expected axial performance. Contributing factors include:

- Due to the variability in subsurface conditions, it is not always possible to anticipate stratigraphy based on borings even a short distance away, and sometimes even across the footprint of a single foundation unit.
- Highly weathered material is not well suited for typical investigation methods designed for soil or for rock.

- It is a challenge to assign material properties to a seemingly erratic stratigraphy that will produce meaningful correlations to the load testing data.

The most common subsurface investigation techniques used in central Florida and most of the United States are Standard Penetration Test (SPT) borings, Cone Penetration Test (CPT) soundings, and rock coring. None of these tests are fully capable of defining the in-situ strengths for design in weathered limestone.

CPT testing is well suited to delineating stratigraphy in soils but is not capable of penetrating rock (even weathered rock).

Typical design methods used by Florida DOT in limestone are based upon correlations with compression tests on rock core samples. However, in this epikarst region, rock coring typically yields very low to zero sample recovery in highly weathered limestone and strength testing of the few recovered samples that are large enough to be tested are not considered to be representative of the rock mass. Additionally, rock coring is a rather slow and expensive process, given the need for such high density coverage of exploratory borings.

Conventional borings can be advanced through highly weathered limestone with SPT testing in a relatively efficient manner. However, SPT tests often yield refusal blow counts in such material, and the test itself provides no direct indication of rock mass or IGM strength.

The method presented in this paper utilizes the relatively high speed and low cost of traditional SPT borings to simply identify materials as either soil, IGM, or rock. The unit side and base resistances of the materials identified through a statistical evaluation of the SPT data are determined from the results of full scale load tests conducted on site in these same materials, and are the primary focus of this paper.

The methods described in this paper include a simple statistical evaluation of the SPT data for a given shaft location, often utilizing multiple borings within close proximity to the given foundation, and must be tempered by engineering judgment. This approach has been implemented successfully for two projects at the Lee Roy Selmon Crosstown Expressway (LRS) in Tampa, Florida:

2004: Assessment and remediation of the reversible elevated lane (REL) foundations.

2012: Foundation construction for the elevated expressways widening project.

The location of these projects is shown below in Figure 1. Descriptions of how the method was applied for both these projects are included in the Case Studies sections of this paper.

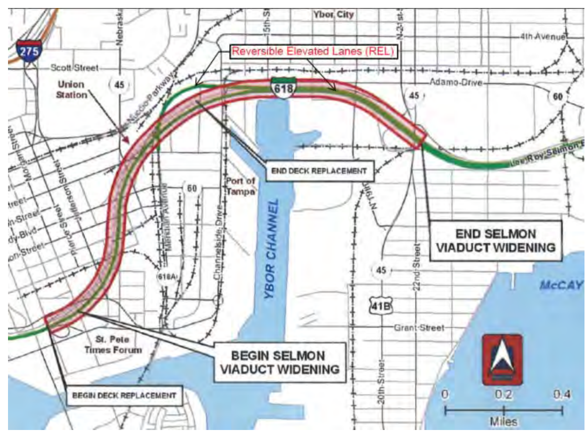

Figure 1. Project Location in Tampa, Florida

The Selmon Expressway is a twin set of elevated structures approximately 1.5 miles long that is currently being widened from 2-lanes in each direction to 3-lanes using small diameter drilled shafts (42-inch and 48-inch). The original foundations are pre-stressed concrete piling. The REL is a segmental box structure approximately 6 miles long with 3-lanes total, constructed in 2004 along the median for much of the alignment using large diameter drilled shafts (mostly 6-ft with some 8-ft). A typical arrangement of the structures is shown in Figure 2. In this picture, the SPT testing crew is working on a shaft location that will be used for the bridge widening, while the higher structure in the median is the REL.

Figure 2. Typical Arrangement of Structures

Geologic and Subsurface Conditions

The Selmon Expressway lies along the ancient Pamlico shoreline which roughly parallels the modern shoreline in Tampa Florida. The project is in close proximity to the bay and as a result the ground water is very shallow at only approximately 4 to 6 ft below ground surface. Although artesian pressures from the Floridan Aquifer are sometimes encountered in the limestone formation in Tampa, artesian pressures were not encountered at this site.

The site geology consists of three basic units:

1) **Recent Sand and Silt Sediments**: typically Pleistocene age or younger, and consist of mostly sand and silt with varying amounts clay and shells.
2) **The Hawthorn Group**: Oligocene to Miocene age marine deposited sediments, mostly clay or sandy clay, that are a greenish-gray or blueish-gray.
3) **Tampa Member Limestone**: Geologically, the Tampa Member is a Miocene age sub-unit of the Hawthorn Group; However, for foundation design the Tampa limestone is very different from the overlying Hawthorn clay. The Ocala limestone is at depth, but is below the bearing strata used for these shafts.

Thickness of these basic geologic strata varies greatly with location. The recent sediments at the LRS site generally consist of fine sand with varying amounts of silt and clay, and the thickness typically ranges from 10 to 30 ft. The Hawthorn sediments at the LRS site consist of sandy clay or clayey sand, and thickness typically ranges from 10 to 40 ft. Overall depth to the Tampa limestone at the LRS site typically ranges from 20 to 70 ft; however, in some locations limestone was not encountered at depths exceeding 100 ft below the surface.

The weathered limestone and limestone formations are highly variable and include inter-bedded lenses of calcareous materials, clayey sands, and clay. These soils infiltrate the limestone as erosion and infill occurred into the solution cavities and fissures. Much of this infilling is suspected to have occurred during the historic higher water level (Pamlico Shoreline). These weathered rock strata elements were mostly thin and discontinuous and tended to be horizontally bedded, but were not easily recognized as such by only a single boring. Figure 3 presents a basic schematic of the site stratigraphy.

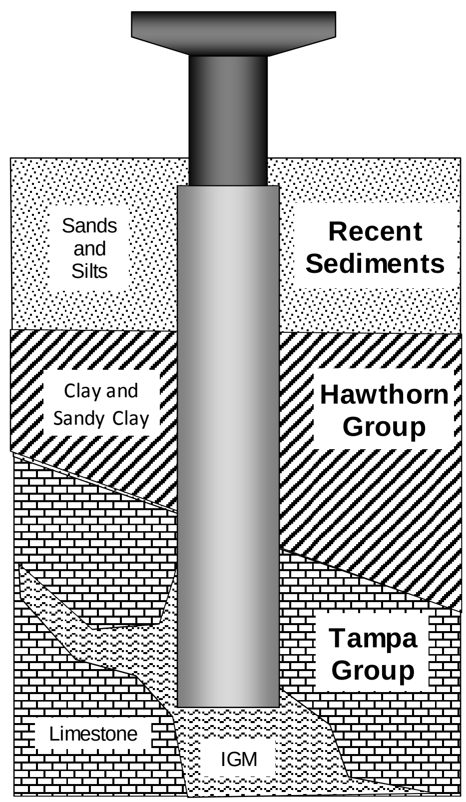

Figure 3 – Generalized Geologic Profile

The hydrologic conditions are divided by the low permeability Hawthorn clay. Above the Hawthorn aquitard is an unconfined aquifer within the recent sediments producing the shallow ground water table with tidal influence. Below the Hawthorn, a confined aquifer (Floridian Aquifer) can exist in Tampa, but was not encountered at this site at these depths. The Floridan Aquifer is typically contained within the vast network of interconnected limestone caves, cavities, fissures, and solution channels.

Additional geologic and hydraulic information can be found in: Randazzo and Jones (1997), Scott (1991), Handfelt and Attwooll (1988), and Miller (1986).

State of the Practice

The standard design procedures for axial analysis of drilled shafts for bridges in Florida are outlined in the Florida Department of Transportation (FDOT) *Soils and Foundations Handbook* (2013) and *Structure Design Guidelines* (2013). Consideration must also be given to the requirements of FDOT *Standard Specifications for Construction of Roads and Bridges* (2013). Much of the private industry in central Florida follow these same design procedures, as is noted by Kunhs et al. (2003).

In general, drilled shafts designed to bear in soil are designed using FHWA methods (Brown et al. 2010). Axial design of drilled shafts in Florida Limestone are typically designed using the procedure developed by McVay et al. (1992) which is based upon the unconfined compression strength of the rock cores adjusted by either the RQD or the Recovery, as described in Appendix A of the FDOT *Soils and Foundations Handbook* (2013). However, many of these highly weathered formations do not produce cores capable of laboratory strength testing to enable reliable application of this correlation, or the cores that are obtained are not representative of the rock mass.

Design Approach

The design approach described in the following paragraphs is based upon an intensive program of site exploration to define the stratigraphy and a given location, and empirical correlations to full scale load tests in this local geology. The implementation of the design also requires verification of conditions encountered during construction of each foundation.

Identification of Layers Using SPT Data

The layer delineation requires some judgment on the part of the designer, based on a working knowledge of the site geology. The writers can contemplate no way to circumvent this process; rational and informed interpretations of the geology cannot be simply programmed into a computer based on SPT data points alone. Rather the statistical evaluation of the SPT data presented is intended as a tool for use in these interpretations and resulting correlations to the load test data.

Each layer of geomaterial at a given shaft location is identified based upon a visual inspection of the material retrieved during the boring(s), as well as a statistical evaluation of the SPT data at the shaft location and any nearby borings. Of course the relevance of a "nearby boring" rapidly diminishes with distance away from the shaft location. Boring(s) within the shaft footprint and/or immediately next to the shaft location are required, and are weighted heavier in the evaluation primarily through judgment of the designer. Any nearby borings that are considered to be relevant should be within a distance that is comparable to that where major changes in the stratigraphy are anticipated.

It may be noted that the primary relevance of "nearby" borings are to identify potential zones of infill or extremely weak material within the limestone. These provide a trigger with which to temper the design based on the borings at the immediate foundation location. The presence of extremely soft material in nearby borings within the foundation bearing zone indicated by the specific foundation boring(s) alerts the designer to consider these risks in the determination of target foundation tip elevations.

The same method and criteria used for identification and delineation of the geomaterials at the load test shaft(s) should, of course, be repeated at all the production shafts to be designed based upon the results of the load testing program.

Because the SPT results are utilized in material that often produce refusal blow counts, consideration was included relative to the significance of the amount of penetration prior to refusal:

- <u>SPT refusals (N ≥ 50) for less than 6 inches of penetration are taken as N=100</u> for purposes of calculating the average N value for the layer. This "hard refusal" (with some allowance made for material that may have collected at the top) is considered to indicate more sound rock-like material.
- <u>SPT refusals (N ≥ 50) for greater than 6 inches but less than 12 inches of penetration are taken as N=80</u> for purposes of calculating the average N value for the layer. This value of 80 blows/ft is considered to represent a reasonable delineation between materials that behaved as IGM's vs. rock. Thus materials that allowed for significant penetration prior to experiencing refusal are considered as something between sound rock and IGM, and the surrounding SPT data points influence the identification of this material one way or the other.

Allowance for Site Variability

The site variability must be implicitly considered in the implementation of the computational methods for determination of axial resistance. The measured resistance determined from the load testing program includes uncertainty regarding the strata that were measured, and subsequent site specific correlations incorporate that uncertainty. The LRFD approach in AASHTO would appear to suggest that the interpretations of site specific correlations should be based on correlation to the mean parameters, and uncertainty incorporated into the resistance factors used for design. A high variability would presumably result in very low resistance factors applied to a mean correlation with a very high standard deviation about the mean. This approach was not used in a rigorous way for these projects.

The approach described in this paper accounts for variability by correlating the computed axial resistance to an interpreted lower bound of measured resistance from the load test drilled shafts. Resistance factors are then employed which are consistent with more conventional practice based on typical site variability. In the judgment of the writers, the correlation to a mean value in a geologic setting with extremely high variations could allow a few unusually good test results to influence the results in an unknown way, given the relatively limited number of full scale load tests.

In addition, the base resistance is neglected or greatly discounted for design. Base resistance is considered to be subject to greater uncertainty in these highly weathered and variable formations. Further, the consequences of over-reliance on the base resistance for a foundation design can subject the completed foundation to unusually high performance risk.

Side Resistance in Overburden Soils

Correlation of the load tested shaft resistance provided by the overburden soil materials (sands and clays) is made using traditional methods such as the FHWA Beta (β) methods and FHWA Alpha (α) methods. However, these resistance contributions are most often comparatively minor, and are not the focus of this paper.

Side Resistance in the Weathered Formation

Correlation of side resistance in the weathered bearing formation was characterized by the simple average of SPT data ($\bar{N}$) within a given layer. In a highly weathered and variable formation, this means that a given layer may contain a conglomeration or layering of materials such as calcareous clays, limestone (weathered to varying degrees), and perhaps even soil in-fills. All these materials within a given layer then contribute to the side resistance at amounts that are proportional to their occurrence within the layer, and the simple average appeared to capture this effect.

For purposes of correlating the side resistances from the load test(s), the layers in the bearing formation are divided into categories of soil, IGM, or rock based on the average SPT blow count ($\bar{N}$) of the layer, as shown in Table 1.

The unit side resistance of soil-like material is calculated using either the FHWA Alpha (α) or Beta (β) method, depending on the material description, as correlated to the load test results.

Table 1 – SPT Criteria for Side Resistance

Average $\bar{N}$ (bl/ft)	Geomaterial Category		Side Resist.
≤ 60	Soil	Cohesive	Alpha (α)
		Cohesionless	Beta (β)
60 to 80	IGM		Site Specific Unit Value
≥ 80	Rock		

The unit side resistance for IGM and Rock layers are set to a constant value that captures the lowest performance in the load test program. The values that are utilized for the case histories are as shown in Table 2.

Table 2 – Site Specific Unit Side Resistance

$\bar{N}$ (bpf)	Material	REL Project (ksf)	Selmon Widening (ksf)
60 to 80	IGM	7.2	9.0
≥80	Rock	11.0	14.0

Although both projects lie mostly along the same alignment, the unit side resistance values used for the widening project are slightly higher than those from the REL work. These differences are thought to be most likely an artifact of either/or:

1) SPT blow counts were systematically lower than the pre-bid borings even when corrections for the auto-hammers used were applied, a conclusion supported by literally hundreds of pre-bid and current borings. Obviously the ground conditions had not changed, so the same conditions were being represented by borings that had a trend of lower blow counts. In effect the SPT blow count criteria for meeting the category for IGM and Rock had been set a little higher in terms of actual energy delivered, and thus the new borings correlate to higher unit side shear values. Alternatively, the unit side resistances could have been kept the same, and the criteria ($\bar{N}$) lowered.
2) The beneficial effects of borehole roughness and concrete over-break are proportionally greater for the smaller diameter shafts of the widening project (3.5 and 4 ft) compared to the REL (6 ft and 8 ft).

Base Resistance in the Weathered Formation

Correlation of the base resistance in the weathered bearing formation was found to be characterized by the average minus one standard deviation of the SPT data ($\bar{N}$ - σ_N) during the REL work. Load test shafts during the widening project were constructed to eliminate the mobilization of base resistance during load testing. It is considered that thin layers of stronger materials within the bearing zone may fail in shear and simply displace into the weaker matrix without substantially contributing to the base resistance. The weaker matrix is considered to control the base resistance, and thus the measure of variability (standard deviation) was used to discount the material to the predominantly weaker material.

For purposes of correlating the base resistance from the load tests for the REL, the layers in the bearing formation are divided into categories of soil, IGM, or rock similar to that shown for side resistance in Table 1. However, for a given layer the average SPT blow count is reduced by one standard deviation ($\bar{N}$ - σ_N) for classification. Site specific unit base resistance is shown in Table 3. Standard FHWA methods are used to compute base resistance of soil layers ($\bar{N} \leq 60$), although foundations were not typically targeted to terminate in soil.

Table 3 – Site Specific Unit Base Resistance

$\bar{N}$ - σ_N (bpf)	Material	REL Project Unit Base Resistance (ksf)
60 to 80	IGM	72
≥80	Rock	108

A given layer then may be characterized differently in side resistance compared to the base resistance. For example, a layer with an average SPT ($\bar{N}$) of 87 b/f would be expected to exhibit a unit side resistance typical of rock; however, if the layer had a standard deviation in the SPT Data (σ_N) of say 10 b/ft, it would then be designed to exhibit a unit base resistance typical of IGM (87 b/ft – 10 b/ft = 77 b/ft).

To account for transition between layers, the estimated unit base resistance is taken as the average calculated for approximately 0.5 shaft diameters above and 1.0 shaft diameter below the shaft base, or the value calculated at the shaft base (whichever is less).

Construction and Verification

A shaft design is never complete until the foundation is built and evaluated, this is especially important in weathered and highly variable bearing formations. Construction in this geology is facilitated by the use of temporary or permanent casing to seal the extremely soft soil which often occurs atop the limestone, and some assumptions about the casing elevations affect the design. If the concrete is observed to drop significantly as the casing is removed, it is usually best to return the casing back to its original elevation and leave it as permanent. Otherwise there could be a risk of the loss of large volumes of concrete and/or a structural defect in the completed foundation.

Full-time, dedicated inspection is required. An inspector experienced with both the local geology and construction practices must diligently log the geomaterials being excavated.

The final step is that the engineer of record certifies each shaft for use by way of reviewing each shaft's construction, to include the excavation log, concrete placement, and any other testing performed. Often this must be done in "real time" as construction is proceeding, and the onsite inspection must have open lines of communication to the engineer of record.

For the Widening project, a robust testing regiment was conducted for each shaft which included bottom cleanliness inspection (mini-SID), as well as cross-hole sonic logging (CSL) and thermal integrity testing (TI).

<u>Case History: REL Foundation Assessment and Remediation</u>

The REL is an elevated 3-lane viaduct that extends for a distance of around 6 miles along the median of a large portion of the Selmon expressway. The REL is a segmental box girder structure resting on hammerhead piers supported by single shafts, typically 6-ft diameter. There are a few straddle bents where this alignment crosses over the regular expressway.

During construction of REL in 2004, one of the hammerhead piers experienced a sudden bearing failure, in which the pier sank approximately 11 ft into the ground (Figure 4). Fortunately there were no casualties or serious injuries. The cantilever construction of the segmental precast structure subjected the foundations to a relatively high axial load during construction, roughly comparable to the design service load of the completed bridge.

Figure 4. Pier 97 Failure During REL work.

The foundation that failed was one of 204 already constructed to support the new REL. The bridge structure at these foundation locations was at various stages of construction ranging up to a completed pier and bridge deck bearing on the shaft.

This event prompted an independent evaluation of the existing foundations, design of any necessary remedial foundation work, and redesign of the 14 shafts that had not yet been constructed. The design method presented in this paper was developed by the writers for these tasks.

Load testing conducted at the Selmon site have covered the gamut of techniques available. The REL design originally had eight (8) O-cell tests. Subsequent to the failure at Pier 97, eight (8) additional high strain dynamic tests were conducted on constructed piers using a large hydraulic hammer and dynamic measurements with signal matching (CAPWAP), shown in Figure 5.

Upon review of settlement data obtained during construction, it was noted that three of the piers with significant structure loading had exhibited unusually large vertical displacement compared to the other more typical behavior; these three piers were judged to be near to the geotechnical limit state, essentially constituted a static load test. Statnamic load testing was conducted latter during the widening project.

Figure 5. Dynamic Testing at REL.

Of the 204 shafts constructed:

a) 50 shafts were determined adequate.
b) 87 shafts were augmented with 4 to 10 micropiles depending upon the calculated deficiency.
c) 67 shafts were augmented with two additional 4ft diameter drilled shafts, connected to the existing foundation with a post-tensioned cap. This solution was employed when the calculated deficiency in axial resistance was in excess of 1000 kips. The added “sister” shafts had a clear space to the center (existing) shaft of only 2 feet to minimize the cap size and fit into the limited space available for the foundation footprint. The group was essentially designed as a barrette with the perimeter of the barrette capturing the outer perimeter of the shafts.

The assessment of axial resistance was made using multiple borings at or immediately adjacent to the drilled shaft location. For each drilled shaft, 3 to 5 borings were available within approximately 15 ft: typically one was conducted at the shaft prior to construction, and 2 to 4 others conducted around the shaft during the investigation subsequent to the event at pier 97. These SPT data were sorted by elevation and combined into one composite boring for design, as has been described.

A comparison of the load tested axial resistance to that calculated using the methodology and values presented are shown in Figure 6. The calculations for the dynamic tests were Class "A" predictions (made prior to performing the test).

This site specific unit resistances used for the REL site as was shown in Tables 2 and 3, and were derived based upon load testing results (both O-cell prior to the incident and subsequent dynamic testing) and the three piers thought to exhibit a geotechnical limit state.

It may be noted that the calculated values of resistance on Figure 6 include the calculated values of base resistance from Table 3. However, the design relied on only 1/3 the calculated value in order to reduce the reliance upon base resistance.

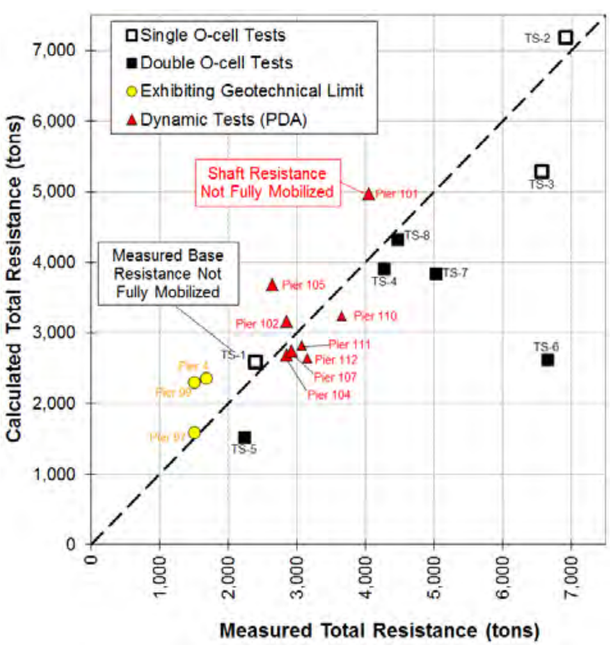

Figure 6. Total Axial Resistance from Load Tests at the REL

Upon completion of the remedial work, the REL project was completed successfully and without any issues related to foundation performance. The REL has been a financial winner for the Tampa-Hillsborough County Expressway Authority; the toll revenue from this project has provided the resources to finance additional expressway projects, including the current project described below.

<u>Case History: Selmon Lane Widening Project</u>

Foundation construction for a design build contract for widening and deck replacement of an approximately 1.5 mile elevated section of the original LRS structure occurred in 2012. The existing structure is supported on groups of square pre-stressed piles, or H-piles in isolated cases.

The new portion of the deck rests on hammer head piers, each supported on a single 3.5 ft or 4 ft diameter drilled shaft. Drilled shafts were selected because of their advantages in limited space available, since a single column can be supported on a single shaft without constructing a footing below grade. In addition, the work was constructed under restricted headroom conditions in many locations. In total, there are 237 individually designed production shafts on this project. The project is on an accelerated design-build schedule with many of the drilled shafts on the critical path.

The design-build team worked collaboratively to develop an approach to construction and design that was consistent and robust, with construction risks mitigated to the extent possible. One aspect of this approach was the decision to make no reliance on base resistance for the design of the foundations, based on the following considerations:

- The base resistance was considered to be subject to performance risks due to the uncertainty in base resistance related to the geology, and therefore the reliability is enhance by minimizing reliance on this portion of axial resistance,

- For drilled shafts of the sizes planned for the project, the base resistance could be offset by only a few feet of additional socket length into weathered limestone,
- The completed foundations would be less subject to performance risks associated with any real or perceived anomalies in integrity test results at the shaft base.

Temporary casing was planned, with installation of the casing to the top of rock using a vibratory hammer to refusal. This approach generally sealed off the very soft material that was often present atop the limestone and allowed the socket to be drilled using water. The casings were typically extracted during concrete placement.

In order to mitigate the potential risk of concrete loss during casing extraction, the design included a provision that the casing may be left in place should conditions in the field indicate the need. The design included an estimate of the casing tip elevation and included only the relatively low side resistance values anticipated in the overburden soils acting on the casing as if it would be permanent. This design approach allowed the flexibility needed during construction so that the risks associated with voids or soft infill could readily be accommodated.

Five Statnamic load tests were conducted on dedicated test shafts to verify the correlations. Since base resistance was neglected for design on this project, a 1 ft thick circular block of foam was place at the base of each test shaft to effectively eliminate base resistance during testing. In addition, one of the five test shafts was constructed using bentonite slurry in the event that the addition of slurry may be needed in some locations. The REL restricted the overhead room for much of construction as well as during load testing, as shown in Figure 7.

Figure 7. Statnamic Testing, Selmon Widening Project.

The site specific unit side resistance values used for the widening project were as shown previously in Table 2, and were derived based upon the load testing results. The test shaft locations were selected along the entire alignment, and representative of the range of ground conditions expected. A comparison of the load tested side resistance to that calculated using the methodology and values presented are shown in Figure 8.

The sensitivity of the side resistance in the overburden soils to various means of borehole stability during excavation was also investigated, as is annotated in Figure 8. In general, all test shafts utilized a temporary casing to rock and water as the drilling fluid, except for TS-5 which utilized a surface casing to the Hawthorn (clay or sandy clay) and bentonite slurry as the drilling fluid. The use of bentonite was not resorted to on any of the production shafts. The casing at TS-3 was intentionally left in place to the top of rock to assess its effect in the event that construction difficulties precluded the removal of a casing. Several of the production shafts had casings left in place, some at full depth some with partial extraction. TS-4 was the deepest test shaft, and with great fortune (from a designer's standpoint) experienced stability

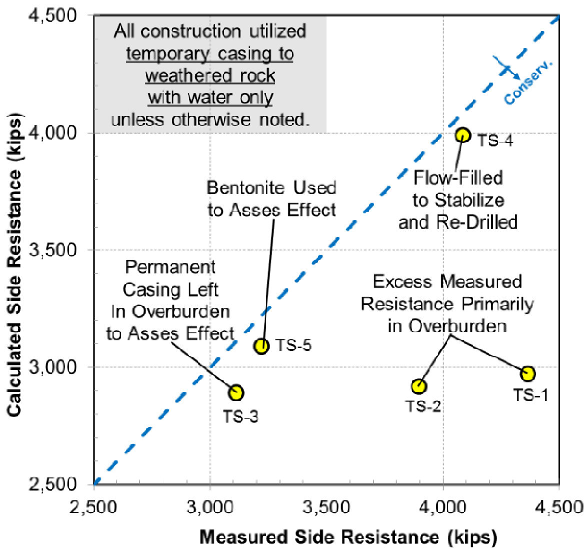

Figure 8. Measured Side Resistance from Load Tests at the Selmon Widening Poject.

problems that required backfilling with low strength sanded flow-fill. One production shaft required this technique, with a low strength concrete substituted for the flow-fill, and a program of settlement monitoring was implemented for that shaft. The design resistances were established such that they capture the measured resistance for the shafts that experience detrimental construction effects and appear somewhat conservative for those (TS-1 and TS-2) that do not.

A total of 388 borings existed prior to the widening work along the project alignment from the original construction, feasibility studies, and REL work. Additionally, a boring was performed at each shaft location for the widening project. At any given location, there were generally 3 to 5 borings in an approximately 200 ft radius, including the boring on location.

As differences in subsurface conditions can often be observed over 200 ft increments, the resistance was first estimated using the surrounding (existing) borings, and then the design finalized primarily with the boring performed on location with consideration given to the surrounding borings. An indication of the local variability at the shaft location is provided by comparing these resistance calculations. Additional consideration was also given to the nearby driven pile tip elevations of the existing structure, both in identifying the bearing zone as well as the expected variability at that location.

In some instances engineering judgment was used to establish drilled shaft design tip elevations were deeper than calculations indicated. This decision was typically driven by a sense that the deeper elevation would tip the shaft into a more appropriate bearing stratum to afford greater reliability on the resistance. Additionally, placing the drilled shafts into a harder, or more intact, stratum facilitated construction since the shaft excavation was easier to clean and the deeper foundation was less subject to the depth of temporary casing.

Conclusions

The design of drilled foundations in highly weathered limestone conditions requires consideration of the special challenges related to reliability in highly variable conditions. The case histories presented in this paper describe the approach that has been used successfully on two local projects in Tampa. The key components in the development of this approach are as follows:

- Site variability is addressed with borings at each foundation location. However the design approach is tempered with consideration of additional nearby borings or other information and does not mindlessly rely upon the results of a single boring.
- Correlations for design are based upon a substantial number of full scale load tests across a range of conditions. Correlations are based on the low end of the range of measurements, rather than the mean.
- Correlations utilize a relatively simple measure (SPT) of soil and rock strength with a large sample population so that variability can be assessed and strata grouped into simple categories.

- Foundation design relies primarily or even solely upon side resistance because of the greater uncertainty associated with base resistance. The design also incorporates flexibility with respect to the contractors use of casing. The design focus is to provide a robust and reliable solution which can be easily constructed.
- A rigorous program of quality assurance is required during construction to ensure that the as-built foundation is consistent with the design assumptions. The designer must be engaged during construction.

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