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EVALUATION AND REPAIR OF A SUBTERRANEAN PARKING GARAGE TO RESIST HURRICANE FLOOD LEVELS

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ABSTRACT

Near-surface groundwater levels and high flood levels associated with flooding events impose significant hydrostatic forces on subterranean parking structures in Florida. Unique geologic conditions and the associated high hydraulic conductivities of the subsurface materials have precluded the use of conventional underdrain systems to provide hydrostatic relief. The case history presented here discusses the evaluation and repair of a subterranean parking garage of an existing office building that exhibited signs of distress including severe cracking of the ground floor slab, excessive quantities of water continuously seeping through these cracks and ponding water. Although various rehabilitation alternatives were evaluated, removal, replacement and re-design of the existing slab were chosen in order to provide additional tie-down restraint and implement a relatively maintenance-free, long-term solution.

This paper briefly describes geologic conditions, the results of site-specific subsurface investigations, historical groundwater information, and various regional and local subterranean design alternatives. The design and construction aspects of the implemented anchored hydrostatic uplift slab system are presented, including: anchor installation, performance and proof testing, construction dewatering, waterproofing issues, and chemical grouting of joints.

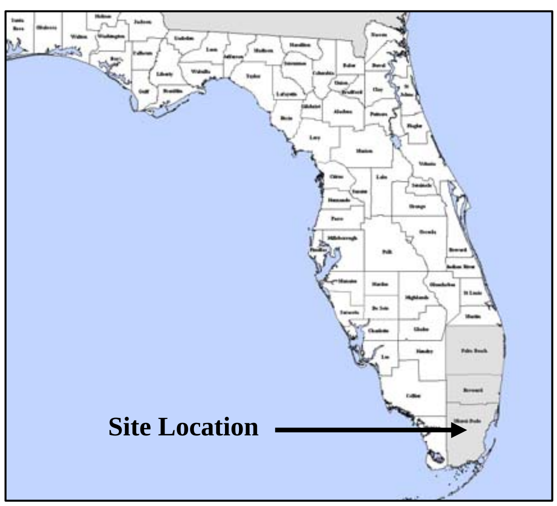

Fig. 1 – Site Location Map

INTRODUCTION

The project is located within the east central area of Miami-Dade County and in proximity to the Miami River. Street grades in the vicinity of the project are typically 4 ft (1.2 m) to 6 ft (1.8 m) above the typical static groundwater level. The project entailed the evaluating and assessing the subterranean level, determining subsurface and hydrogeologic conditions (design flood levels), and designing and constructing a hydrostatic-resistant slab system. Photograph A shows the project site, which includes a seven-level office structure and an elevated one-level parking podium.

Photograph A – Project site

The project site is occupied by a subterranean level with dimensions of approximately 150 ft (45.7 m) by 100 ft (30.5 m). A seven level structure overlies the central portions of the subterranean level, encompassing a footprint area of approximately 40 ft (12.2 m) by 90 ft (27.4 m). The perimeter portions of the subterranean level are overlain by a one-level open parking area and exit-entrance ramps providing access to the street level.

GENERALIZED GEOLOGIC CONDITIONS IN SOUTH FLORIDA

The geologic conditions in South Florida consist of interbedded and alternating layers or zones of soft sedimentary rock and granular soils. The sedimentary geologic formations that underlie the Atlantic Coastal areas of South Florida are among the youngest in the United States. The uppermost geologic strata that most closely resemble commonly accepted, rock-like material include the Miami and the Fort Thompson formations. These formations were deposited at the same time during the Pleistocene epoch which began about two million years ago. A generalized geologic profile is shown in Figure 2. The silica sands, cemented sand and shell (coquina), sandstone and limestone of the Fort Thompson Formation, the older of the two, is generally composed of relatively finer grained materials. The Miami Formation generally consists of a soft, relatively consistent rock formation (extremely weak to very weak rock in terms of uniaxial compressive strength, Brown 1981), and the Fort Thompson formation is typically interbedded and interlayered with materials of varying degree of cementation and hardness (typically very weak rock to weak rock with isolated zones of medium strong rock in terms of uniaxial strength, Brown, 1981) with soil-filled layers or zones. Sea level and possibly other environmental fluctuations likely contributed to the varied composition of this formation. As sea level rose during the most recent post-glacial epoch, low-lying mangrove swamps and tidal bays formed above the limestone. Along oceanfront areas, Holocene sands of the Pamlico Formation were subsequently deposited above the organic silts and peats (Hoffmeister, 1974).

The parent materials of the Miami and Fort Thompson formations have hardened over time as a result of successive deposition, partial exposure and cementation, and subsequent inundation and sedimentation. Despite this hardening, the complete metamorphosis into a relatively uniform rock strata has not occurred in the geologically short time period from initial deposition to the present. In this geologic setting, these varying interbedded materials can be classified in three ways. First, they can be classified by their appearance as soil, intermediate geo-materials and rock. Second, they can be classified by their relative consistency or degree of cementation as loose or soft, to very dense or typically very hard. Third, they can be classified in terms of uniaxial compressive strength as extremely weak to weak rock with some isolated zones of medium strong rock (Brown, 1981). Sand-filled vuggs are common within the rock-like zones. Karstic features are not typically present in the South Florida geology.

HISTORICAL HYDROGEOLOGIC INFORMATION

Surface hydrology is dominated by a series of lakes and water management canals, including the adjacent Miami River. The project site is underlain by the Biscayne Aquifer, which serves as Miami-Dade County's primary domestic water supply,

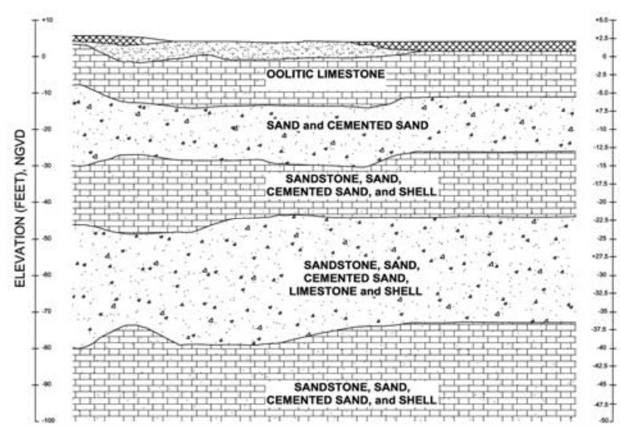

Fig. 2 – Generalized geologic profile

which has been designated as a sole-source aquifer by the U.S. Environmental Protection Agency (EPA) under the provisions of the SAFE Drinking Water Act. The aquifer is a highly permeable, shallow hydrologic unit of limestone, sandstone and sand about 120 feet (36.5 m) thick. The aquifer is unconfined and the transfer of water between surface waterways and groundwater reserves varies seasonally. Recharge occurs primarily from infiltration of rainfall, but also from canal water during the dry season. The groundwater table within the coastal areas of Miami-Dade County has a slight seaward gradient and generally ranges between el 0 to el +3 (0.9 m) based on the National Geodetic Vertical Datum of 1929 (NGVD) (Refer to the USGS groundwater monitoring database website for detailed information). Variations throughout the year amount to a variation of about 2 ft (0.6 m) higher in the rainy summer season as compared to the water levels in the dry fall and winter seasons.

The Miami Formation is very permeable. Available results of field aquifer tests and other field hydraulic conductivity tests performed in the region by the authors' firm indicate that the approximate range of hydraulic conductivity of this formation is highly variable and ranges from 3 ft/day (1 x 10^{-3} cm/sec) to 2,500 ft/day (1 cm/sec). The high variability of hydraulic conductivity is expected to be a result of the variability in the rock formation's constituents and structure including variable silt content, number of pores, extent of lateral connected channels, etc. The hydraulic conductivity of the formation has been documented to increase over the duration of a construction project as a result of migration of less cemented soft rock, soil or fines from within the overall rock structure.

The Fort Thompson formation is considered highly permeable. This formation is found thickening to the east until it becomes partly or completely interfingered with the Anastasia formation and occasionally interfingered with the Key Largo Limestone formation. The Key Largo Limestone formation is more prevalent within the lower portions of South Florida approaching the Florida Keys. The Fort Thompson formation consists of a series of marine, brackish-water, and freshwater limestone ranging from slightly to very porous. The Anastasia, Key Largo Limestone, and Fort Thompson formation constitute the bulk of the very highly permeable sediments of the Biscayne Aquifer in eastern Miami-Dade County. The average hydraulic conductivity of the three formations is much greater than 1,000 ft/day (4x10^{-1} cm/sec) and probably exceeds 10,000 ft/day (4 cm/sec) over much of the area (Fish et al. 1991). Well-cemented, consistent layers within the Fort Thompson and Anastasia Formations have been documented to have hydraulic conductivities significantly less than presented above; however, generally, the Fort Thompson Formation is considered to be highly permeable.

According to the Flood Insurance Rate Map (FIRM) of Miami-Dade County with latest revision of 1994, the project site is located within Flood Zone AE, which is defined as an area "of special flood hazard inundated by 100 year flood with base flood elevations determined," with a base flood elevation of el +9 ft (2.75 m), NGVD (FIRM, 1994).

The U.S. Geological Survey (USGS) groundwater level monitoring network in South Florida began in 1939 as a cooperative effort with the local governments to evaluate the affect of a drought on the groundwater supplies in this area. The most highly concentrated portion of the current USGS cooperative network occurs in Miami-Dade County in the Biscayne Aquifer where 146 wells are used to monitor groundwater levels. Extremely high groundwater levels associated with October tropical storms, hurricanes or flooding events have been recorded in the past. In the 25 years prior to 2001, the highest daily maximum water levels were recorded in wells located in the eastern portions of Miami-Dade County and are associated with Tropical Storm Fabian (October 1991), Hurricane Irene (October 1999) and Tropical Storm Leslie (October 2000). The highest daily maximum water level associated with these events ranged from el +5.5 (1.7 m) to el +10.16 (3.1 m). Subsequent to 2001, the highest groundwater level, of el +7.07 (2.15 m), was recorded during Hurricane Katrina in August of 2005.

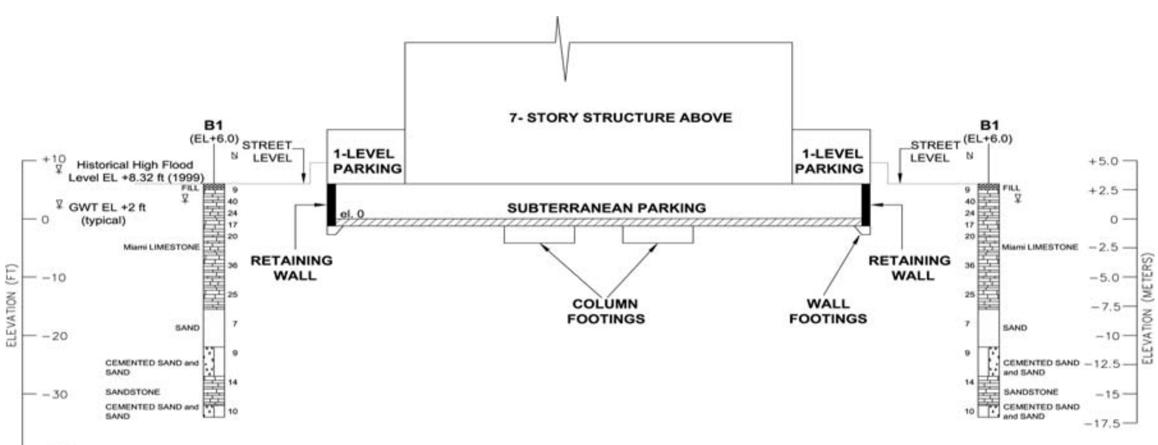

Fig. 3 – Site specific subsurface information and building section

SITE-SPECIFIC SUBSURFACE CONDITIONS AND HYDROGEOLOGIC INFORMATION

Because of limited access within the project site, borings consisting of conventional split-spoon sampling with standard penetration testing were performed around the subject property. Additionally, borehole permeability testing was performed to (1) evaluate the subsurface materials, (2) evaluate the feasibility of utilizing a sub-slab drainage system and (3) provide preliminary indications regarding the expected groundwater inflow rates during construction dewatering.

The subsurface conditions below the subterranean slab level were determined to consist of a 15-ft (4.6 m) thick layer of oolitic limestone of the Miami Formation with typical Standard Penetration Test (SPT) N-values, performed using conventional safety hammers, ranging from 15 blows per foot-bl/ft (blows/0.3 m) to 35 bl/ft, followed by a 12-ft (3.6 m) thick layer of sand with varying proportions of cemented sand and with typical SPT N-values ranging from 5 bl/ft to 10 bl/ft. Next, layers of sandstone and cemented sand of the Fort Thompson Formation were encountered to the termination depth of the borings at 40 ft (12.2 m) below grade. The authors' prior experience with unconfined compressive strength testing in the Miami Limestone suggest that the Miami Limestone at the project site would be expected to have unconfined compressive strengths ranging from 150 lbs/in^2 (1.0 MPa) to 250 lbs/in^2 (1.7 MPa). Borehole permeability testing, consisting of staged hydraulic conductivity testing to assess both vertical and horizontal permeability, was performed (Cedergren, 1989). The hydraulic conductivity of the Miami Limestone stratum was determined to vary from 1 x 10^{-3} cm/sec to 9 x 10^{-2} cm/sec. Hydraulic conductivity testing of the zones of sand and cemented sand beneath the Miami Limestone formation was attempted; however, a sufficient source of water could not be provided to maintain a positive head within the borehole, indicating that excessively high permeabilities are likely.

See Fig. 3 for a representation of the site-specific subsurface conditions and a schematic of the existing structure with its subterranean level.

HISTORICAL SUBTERRANEAN FOUNDATION SYSTEMS

In many areas along the east coast of the United States, subterranean parking structures are constructed either as individual projects or as a component of the development. Construction of parking levels below grade is considered architecturally desirable in urban settings to limit or eliminate above-grade parking structures. The below-grade parking structure approach allows the first above-grade level to consist of retail-lobby space, and subsequent levels above this to be habitable space (i.e., residential, rentable-leasable space). Other trends have included providing public parking areas below parks and greenways to facilitate efficient use within urban settings.

The design of subterranean structures must take into account the affect of static, seasonally high groundwater levels, and flood water conditions. For global stability, the structure should be designed to resist the hydrostatic uplift force with a suitable factor of safety (GEC #4, FHWA, 1999), through dead weight of the structure or a combination of dead weight and supplemental tie-down resistance. Alternatively, the design should provide contingencies to relieve hydrostatic pressure. Historically, in areas with soil or rock conditions with low hydraulic conductivities, hydrostatic relief systems have consisted of sub-slab drainage systems (underdrains), which are continually pumped to relieve hydrostatic pressure, or hydrostatic relief systems designed only for use during infrequent seasonal high groundwater levels or even more infrequent and extreme flood conditions. If hydrostatic relief systems allow flooding of the parking level to occur during times of infrequent seasonal high groundwater levels or extreme flood conditions, wet slab and inundated slab conditions will occur during times of high groundwater levels. These hydrostatic relief systems have been utilized for subterranean parking levels only, because the consequences of inundating habitable areas or inundating areas with costly mechanical equipment outweighs the cost-benefit of the relief system itself.

HISTORICAL SUBTERRANEAN REPAIR SYSTEMS

Locally implemented subterranean repair systems have consisted of (1) isolated patching, (2) topping with or without hydrostatic relief and restraint systems, and (3) full slab replacement in combination with hydrostatic relief and restraint systems. Local repair and rehabilitation of subterranean levels has also included less costly patching or patching in combination with supplemental slab restraint. Slab patching, without providing design contingencies to address the next flood event, is expected to require extensive supplemental repair as subsequent cracking occurs over time. Reportedly, more in-depth and extensive patching of subterranean levels has consisted of chemical grouting of cracks within floor slabs, around the interface of the floor slabs to walls, and around the interface of the floor slab to the columns. Continual pumping of water is also typically necessary for patched slabs to minimize the presence of ponded water. Subsequent grouting is likely to be required after each significant flooding event. Patching (using the existing slab without replacement) in combination with tie-down anchors, also less costly than full slab replacement has resulted in marginal success. Supplemental tie-down restraints must be spaced based on existing slab strength and cannot be optimally spaced for efficiency. Anchor-head connection details are also more challenging when using the existing slab system to provide a finished flat slab surface. Reportedly, the patched slab areas require numerous supplemental grouting treatments over time; however, the patched system in combination with tie-downs provides supplemental slab restraint which minimizes the potential for significant damage during subsequent flood events.

Slab topping in combination with supplemental slab restraint has not been used as a repair alternative for subterranean slabs because of code requirements for floor-to-ceiling clearance. Typically, the subterranean levels of structures constructed in the area have minimal tolerance for adding slab thickness and reducing floor-to-ceiling clearance based on local code requirements (FBC, 2003).

Full slab replacement and installation of either a hydrostatic relief system or supplemental slab restraint, while the most costly, results in the most reliable finished product. This system also allows under-slab waterproofing to be used, which may not have been used in the initial construction.

INITIAL BUILDING OBSERVATIONS AND ORIGINAL DESIGN SUMMARY

In late 2001, the authors' visited the subject project site and observed the conditions of the subterranean parking level. Extensive cracking and bowing-heaving of the ground floor slab were observed throughout the subterranean level. Additionally, the underside and exposed surface of the elevated concrete deck had extensive cracks. Ponded water was observed throughout the lower level and flowing water was observed through cracks within the slab and at the interfaces of the slab to the footings.

The structure was built in the late 1980's and consists of a seven-story office building centrally located over the subterranean parking level and surrounded on all sides by a one-level elevated parking podium. No damage was observed within the central portion of the subterranean level. The damaged slab areas were isolated to the perimeter areas, which were overlain only by a one-level elevated parking deck. The foundation system of the structure consists of shallow column and wall footings designed with an allowable bearing pressure of 6,000 lbs/ft^2 (287 kPa) and supported on Miami Limestone. The ground floor slab of the structure was designed and poured monolithically with the footings. No waterproofing was used beneath the floor slab of the original structure. The surface of the subterranean slab is located at el 0 based on the National Geodetic Vertical Datum of 1929 (NGVD).

Reportedly, the subterranean slab became damaged and seepage began entering the subterranean level after Hurricane Irene in 1999. Because the ground floor level is located 2 ft (0.6 m) below the typical static groundwater level and up to 8.32 ft (2.54 m) below the flood water level recorded in 1999, significant pressure is expected to have developed on the slab during the flooding event. The high uplift pressures apparently caused the ground floor slab to heave and crack, and the outer footings to be raised, causing the cracking of the elevated parking deck, which wraps around the perimeter of the taller, more heavily loaded office structure. See photograph B, C, and D. The actual developed pressure can only be approximated based on historical groundwater monitoring wells located in the general vicinity of the project site. See the attached Fig. 3 for a schematic of the expected flood level conditions imposed on the structure's ground floor slab.

Photograph B – Observed cracks on elevated deck

Photograph C – Observed seepage within subterranean level

Photograph D – Observed seepage within subterranean level

BUILDING ASSESSMENT AND EVALUATION

Various repair and rehabilitation alternatives were considered including (1) topping and installation of a sub-slab drainage system (2) topping of the slab and providing tie-down restraint, and (3) demolishing the slab, installing a tie-down system and constructing a new slab. According to local building code requirements, topping of the slab, which would result in reduced headroom (i.e., clearance from the surface of the slab to the underside of the elevated deck/mechanical equipment), was not feasible. Therefore, removal and replacement of the slab was deemed necessary.

An evaluation of the sub-slab drainage system versus the tie-down hydrostatic slab was performed. Because of the high hydraulic conductivities of the subsurface materials, a continually pumped sub-slab drainage system was not considered. An intermittent hydrostatic relief system, which only provided relief of water pressure during times of seasonal high groundwater levels or hurricane flood conditions, was also considered; however, this system would have resulted in a wet slab condition or inundated slab condition numerous times throughout the year according to historical groundwater information. Ultimately, the water-proofed, hydrostatic-resistant slab with a tie-down system was selected as the preferred alternative. The subsequent sections provide the details of the selected geo-structural systems as well as a discussion of the construction of the slab replacement.

SELECTED GEO-STRUCTURAL SYSTEMS

The hydrostatic-resistant system consisted of (1) a 12-inch-thick (30.5 cm) structural slab reinforced with #5 bars each way, top and bottom (2) locally thickened slab sections to facilitate connection of anchors (3) one hundred three (103) 22-ton (196 kN) 5 ½ inch-diameter (14cm), double corrosion protected 1-inch-diameter (2.54cm), grade 150 ksi (1,000 MPa) Williams bar rock anchors, and (4) bentonitic waterproofing beneath the ground floor slab. See Photographs E and F of the completed anchors and waterproofing.

Photograph E – Rock anchor head assemblies

The rock anchors were installed into the Miami Limestone bearing layer and consisted of a passive rock anchor system with an 8 ft (2.4 m) bond length and 2 ft (0.6 m) free strength length (smooth sleeved PVC) resulting in a total anchor length of 10 ft (3m). The double corrosion protection system consisted of a pre-grouted 1-inch-diameter (2.54cm) bar within a corrugated polyethylene tubing. Because of headroom restrictions, all of the rock anchors were spliced in the field and connected via a coupling, and subsequently corrosion-protected prior to heat shrink wrapping.

Photograph F – Underslab bentonitic waterproofing

The rock anchors were connected structurally to the overlying slab using a 1¼ inch-thick (3.2 cm), 7-inch square (17.8 cm) trumpet assembly which encompassed the upper portion of the free stress length and double corrosion protected bar, as well as being packed with corrosion inhibiting grease. Next, a galvanized washer and epoxy-coated hex nut were tightened to the surface of the bearing plate. Subsequently, the protruding Williams bar was saw-cut in accordance with the manufacturer's recommendations and cast within a locally thickened portion of the structural slab.

ROCK ANCHOR DRILLING AND INSTALLATION PROCEDURES

All rock anchors were installed with a low headroom Klemm drill rig to the required embedment into the underlying Miami Limestone bearing layer. See Photograph G showing the low headroom drill rig and installation of an anchor into the grout filled hole.

Photograph G – Low headroom anchor rig

Initially, several attempts were made to install the anchor assembly without the use of internal temporary stabilizing casing. After several attempts, installation techniques were modified to include the use of internal temporary 5½-inch (14 cm) O.D. casing. Drilling was accomplished using a downhole percussion hammer and cuttings were removed via compressed air. See photographs H and I below showing the downhole percussion hammer and internal casing system.

Photograph H – Downhole percussion hammer

Photograph I – Casing used during drilling operations

After completion of the drilling operation and cleaning of the drilled hole with rotary air techniques, the depth of the hole was verified by sounding techniques. The drilled hole was then filled with on-site batched 4,000 lbs/in^2 (28 MPa) design strength grout prepared with standard paddle mixers. Type II cement was used along with a water/cement ratio of 0.45 to limit the potential for future corrosion (ACI 543, 1988). The anchor assembly, with centralizers and post-grout tubes, was inserted in two pieces, with coupling occurring at approximately mid-length into the drilled hole. The casing was then subsequently extracted in 3-ft sections and the grout level within the casing was topped off after each casing section was removed to maintain the appropriate head of grout fluid. Typical grout factors averaged on the order of 1.9, excluding post-grouting volumes, with isolated anchors having grout factors in excess of 3 (2% of total anchors). Post-grouting was typically performed one to two days after initial installation and through post-grout ports placed near the bottom and at the approximate mid-length of the bond zone. Post-grouting pressures typically ranged from 300 lbs/in^2 (2.1 MPa) to 700 lbs/in^2 (4.8 MPa). In the event significant grout take was observed during post-grouting, the grout lines were flushed to facilitate secondary post-grouting several days later.

ANCHOR PERFORMANCE LOAD TESTING AND PROOF TESTING

Four rock anchors were installed, prior to installation of production anchors, and performance tested to 1.5 times the design load. See Photograph J. Creep tests (i.e., 50-minute hold under maximum test load) were also performed under the maximum test load. The measured deflection of the top of the bar ranged from 0.176 inch (0.45 cm) to 0.369 inch (0.94 cm). Under the maximum test load, the anchors were observed to creep between 0.001 inch (0.0025 cm) to 0.014 inch (0.0356 cm) during the 50-minute hold period. A graphical presentation of three of the performance load test results is presented in Figure 4.

The average mobilized bond stress for the soft rock to grout interface along the anchor bond zone was calculated to be 3.5 tons/ft^2 (0.34 MPa), under the maximum test load of 33 tons (294 kN). The resulting design bond stress used was 2.3 tons/ft^2 (0.22 MPa). The corresponding mobilized average transfer load along the bond length was determined to be 4.1 tons/ft (120 kN/m) with the design transfer load being 2.75 tons/ft (80 kN/m). These average mobilized load transfer values (4.1 tons/ft) are less than the values proposed for all categories of rock type material by various references for use in preliminary design (GEC #4, FHWA, 1999). The average mobilized bond stress is similar to the high end for fine to medium sand in a medium dense to dense condition (PTI, 1996) and in excess of PTI's preliminary ultimate bond stress values using 10% of the unconfined compressive strength of the rock.

All production anchors were proof-tested prior to construction of the replacement slab. Typically, the rock anchors were proof-tested three days to seven days after installation. The results of the proof testing showed the anchor movement averaged 0.2 inch (0.5 cm) under the maximum test load. Ninety percent of the anchors tested moved less than 0.25 inch (0.64 cm) under the maximum test load. No excessive creep movement in excess of PTI guidelines was observed for the majority of the anchors during the ½ minute to 5 minute hold period. Where anchors moved excessively upon application of the test load (1.5 times the design load), replacement anchors were installed. See Fig. 5 for a graphical representation of the proof tests.

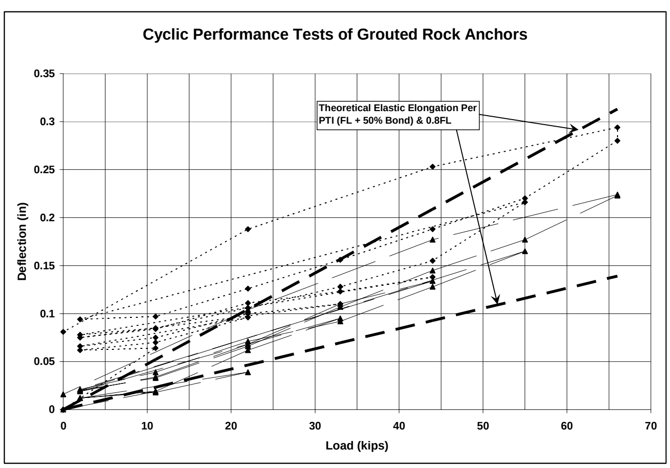

Fig. 4 – Plot of performance tested anchors

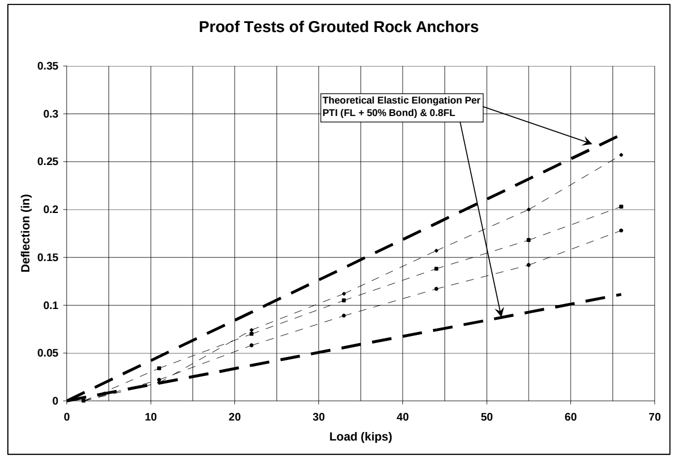

Fig. 5 – Plot of typical proof tested anchors

Photograph J – Anchor load test set-up

UNDERSLAB JOINT GROUTING

After constructing the replacement slab and cutting off the dewatering system, water leaks were observed at some cold joint locations where the original slab joined to the new replacement slab. These locations of observed seepage were sealed with chemical grout. The chemical grouting procedure consisted of injecting a proprietary chemical grout into injection ports, which were drilled and epoxied at the location of the observed seepage. The grout ports were installed at approximately 6 inches (15.2 cm) on center along the alignment of the observed seepage. See Photograph K.

Upon completion of the primary, secondary and tertiary grouting, no subsequent moisture or seepage was noted at the subject locations. Reportedly, three years after construction, no subsequent leaks or signs of seepage have been observed.

Photograph K – Underslab chemical grouting of joint

CONSTRUCTION DEWATERING

Dewatering of the project site was necessary because the interim construction grade after removal of the damaged slab was on the order of el -1 NGVD (approx 2 ft (0.6 m) to 3 ft (0.91m) below the natural groundwater level). Locally thickened slab zones were used to connect the rock anchor elements into the structural floor. The thickened slab zones were approximately 1 ft (0.3 m) deeper than the bottom of slab elevation (interim construction grade) resulting in the requirement to dewater to el -2 NGVD (approx 3 ft (0.91 m) to 4 ft (1.2 m) below the natural groundwater level). Dewatering in the Miami-Dade area is typically accomplished using either well points, lateral sock drains, or sump pumps. Traditionally, the use of well points and lateral sock drains is limited to the beachfront areas (barrier islands), where significant deposits of beach sands are present overlying the soft sedimentary rock at depth. The use of high capacity sump pumps has been used to dewater for construction operations within the soft sedimentary rock zones. Highly variable flow rates occur in the soft sedimentary rock of South Florida in what may be perceived as similar soft sedimentary rock formations. These highly variable flow rates are a result of variations of the rock formation. For sites with highly permeable rock conditions resulting in very high flow rates, dewatering is difficult. Significant construction planning-phasing is required to facilitate successful and timely completion of projects within highly permeable soft sedimentary rock formations. Additionally, significant planning is required in the urban setting of South Florida to determine means of discharging large volumes of dewatering effluent throughout the duration of the construction project.

For this project, a network of aggregate-filled lateral drains with geosynthetic encased perforated pipes were installed between column footings to facilitate conveyance of the groundwater to two extraction locations. A well point vacuum extraction pump was hard-connected to the lateral buried corrugated, perforated pipe at each extraction location. The flow rate required to maintain the project site in a dewatered condition varied but was typically on the order of 1,000 gallons per minute (gpm-0.063 m^3/sec). See Photograph L and M.

Photograph L – Lateral sock drain installation

The dewatering effluent was routed through pipes across several neighboring sites (with prior approval and authorization) to reach a sanitary sewer manhole with sufficient discharge capacity. This particular sanitary sewer manhole was close to an adjacent canal and contained an exit pipe-manhole structure sufficient to facilitate discharge of 1,000 gpm (0.063 m^3/sec) to 2,000 gpm (0.126 m^3/sec).

Photograph M – Extraction pit for lateral drain

CONCLUSIONS

The following conclusions can be drawn relative to the building assessment, design and repair of this subterranean parking garage to resist hurricane flood levels:

1) Flooding during the late 1990s resulted in the build up of hydrostatic pressure in excess of the resistive capacity of the slab and structure.

2) Use of a sub-slab drainage system was conceptualized; however, based on prior local experience as well as documentation of flow rates necessary to suppress the groundwater level only a few feet during construction, a continually pumped sub-slab drainage systems was not viable in the soft sedimentary rock formations with high permeability. Further, use of a hydrostatic relief system, would have been viable, but tenant's restrictions to provide limited flooding of the subterranean parking level resulted in the need to implement the waterproofed, hydrostatic tied-down structural floor system.

3) Repair of subterranean parking level, by providing supplemental tie-down resistance capacity to the ground floor slab, and installing a waterproofed structural slab has resulted in a system that has proven to be successful. Since repair completion, the structure has undergone several cycles of elevated groundwater level with no signs of distress.

4) Using a design grout to soft limestone bond of 2.3 tons/ft^2 (0.22 mPa), within the Miami Limestone, resulted in rock anchors meeting the performance requirements of the Post-Tensioning Institute (PTI, 1996). The authors do not suggest using local correlations for ACIP piles or drilled shafts for estimating design bond stress values as anchor installation procedures and performance criteria are not consistent with these other foundation systems.

5) Construction dewatering within the permeable soft sedimentary Miami Limestone Formation was successfully accomplished using a network of lateral drains and extraction pits.

6) Extreme care should be exercised and special attention needs to be paid to design details and construction implementation for slabs poured in phases (non-monolithically) below the groundwater level.

7) Chemical grouting proved to be an effective means of sealing deficiencies associated with waterproofing and slab joints. Multiple injections were required; but the final product proved to be water tight.

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Additionally, presentation and publication of this paper, for the first author, is of special significance, as the first author (Mr. Matthew Meyer) served as a teaching assistant to Professor James K. Mitchell in 1995 when Professor Mitchell moved from the University of California at Berkeley to Virginia Tech. Mr. Meyer would like to extend his thanks to Professor Mitchell for his mentorship.

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