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THE USE OF EPS GEOFOAM LIGHTWEIGHT FILL IN HOLLYWOOD, FL

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ABSTRACT:

Expanded Polystyrene (EPS) Geofoam lightweight fill permitted elevation of a loading dock and access roadway to facilitate offloading of banquet/convention center and hotel supplies, products, etc. directly into the second floor of a development underlain with highly compressible soils, in Hollywood, Florida. Similar roadways had been previously constructed using conventional filling techniques and have resulted in continual maintenance problems and grade separation between pile supported structures and roadways supported on filled ground. The South Florida barrier island geology and interior mangrove swamp areas, which are reclaimed land formed through the use of dredging and hydraulic filling operations in the early 1960's, require specialized site preparation techniques. Typically, preloading, or surcharging, where feasible, is utilized to pre-compress the underlying compressible organic silts and peat deposits, and often structural relieving platforms/hollow filled structural ramps are constructed to prevent continual maintenance of critical on-grade supported appurtenances. In this instance, EPS Geofoam lightweight fill was ideally suited to accomplish the Developer's and Contractor's objectives with their fast-track schedule and site constraints.

This paper provides a brief description of the typical South Florida geology, compressibility characteristics of the underlying soft compressible organic deposits, and our settlement predictions, which showed the need for preloading, structural support or use of one of the first applications of lightweight fill in South Florida. An in-depth settlement monitoring and instrumentation program was conducted to confirm the expected behavior of the lightweight fill and the underlying subsurface behavior. Induced stresses through the overlying pavement and fill material are provided and the general construction procedures utilized are summarized.

INTRODUCTION

Within the largest hotel development constructed in South Florida, one of the first applications of EPS lightweight fill was successfully implemented for the support of an elevated roadway. Approximately 1,150 cubic meters (1,500 cubic yards) of Type II EPS lightweight fill was placed to raise grades up to 1.7 m (5 ½ ft) within an area of compressible ground conditions and documented on-going settlement related problems. The following sections provide a discussion of the pertinent design and construction related issues.

The site is located along the beach within the city limits of Hollywood, Florida. The project development is located both on the East (Ocean Side) and West sides (Intracoastal Side) of South Ocean Drive, with the subject site being located at the southeast end of the general development. The approximate site location is shown below in Figure 1.

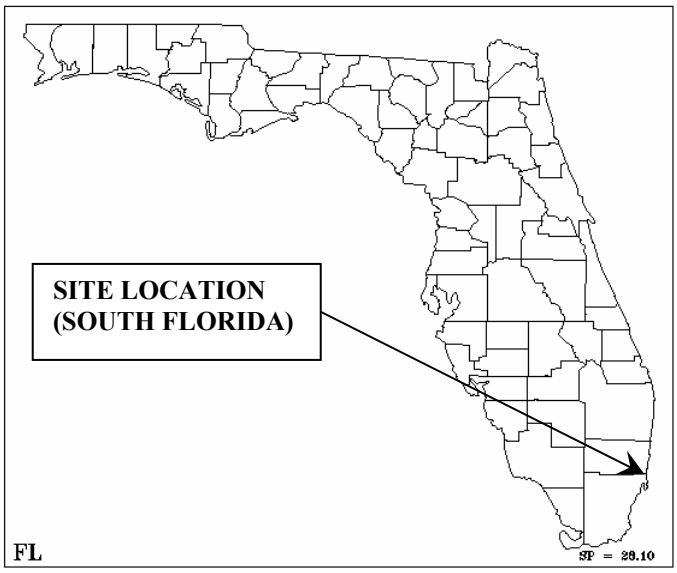

FIGURE 1- SITE LOCATION PLAN

SOUTH FLORIDA GEOLOGY

The South Florida geologic formations consist of near surface oceanfront/beachfront fine to medium grained sands of the Pamlico Formation and underlying organic silts and peats, which are swamp and tidal bay deposits. Both of these formations were deposited relatively recently in geologic terms as the sea level rose well above its current elevation during the post-glacial epoch. Beneath these deposits are sedimentary rock deposits of the Miami and Fort Thompson Formation. The Miami Limestone is a white to tan sandy oolitic limestone and is typically more consistent than the underlying older Fort Thompson Formation. The Fort Thompson formation is composed of sands, cemented sands, cemented sands and shells, limestones, and sandstones. This formation typically contains cemented zones which are significantly harder than the Miami Formation, and contains zones which are partially or completely uncemented in nature (Hoffmeister, 1974).

Of particular interest to this construction project are the more surficial sandy beachfront deposits and underlying compressible organic silts and peats. In many areas, to facilitate development and to provide as much waterfront property as possible, extensive dredging of canals and filling over mangrove swamps has been performed. The majority of this dredging and reclaiming of land was performed in the early to mid 1900's.

PROJECT DESCRIPTION

To provide vehicular access into the second floor of the banquet facility and allow offloading and loading of products, supplies, etc., an elevated loading dock and access roadway was required. The ground floor of the banquet facility would serve as ground floor and valet parking. An approximate 107 m by 12 m (350 ft by 40 ft) access roadway and loading dock were planned to be constructed. This roadway was to be located immediately adjacent to the project's recently completed 8-story Banquet Facility supported on augercast piles, and an existing neighboring one-level garage supported on driven precast concrete piles attached to a 15-story condominium. Research of available plans at the local building department indicated that the adjacent neighboring structure was supported on a system of grade beams and pile caps; however, the ground floor level slab was supported on grade at el + 1.5 m (el +5 ft). Visual inspection of this slab showed it to be in relatively good condition. It was determined that repair of the ground floor slab of the neighboring structure was unacceptable and a method must be designed so as not to adversely effect its slab on grade.

The proposed access roadway and dock required filling of the site up to 1.7 m (5.5 ft) above the previous existing site grades. Both immediate (primary consolidation) settlements and long term settlements associated with continual secondary compression were a concern with the underlying compressible organic silt and peat deposits. Methods of minimizing differential post-construction settlements between the pile supported Banquet Facility structure and the adjacent roadway were desired. Even further complicating placement of earth fill was the presence of settlement sensitive fiberglass encased chilled water lines, hot water lines, and electric lines encased in concrete (ductbanks), which had been installed within the area of the planned roadway. The figure below shows a cross-section of the originally planned construction.

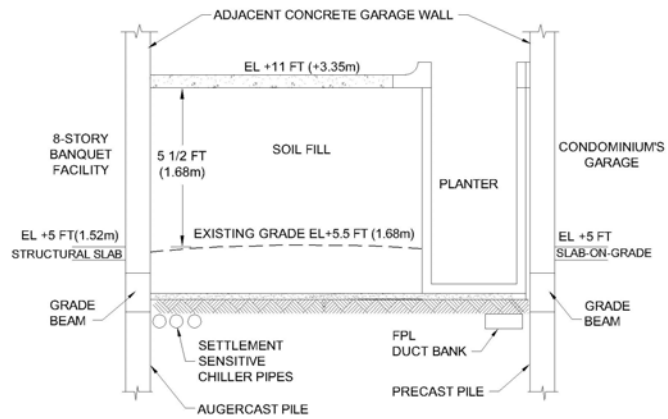

FIGURE 2- EXISTING CONDITIONS & PROPOSED SOIL EMBANKMENT (CROSS-SECTION)

SUBSURFACE CONDITIONS

Site specific subsurface conditions were determined through the use of mud-rotary drilling techniques and casing as a stablizing technique where necessary. Standard Penetration Tests were typically performed continuously for the upper approximately 3 m (10 ft) and at 1.5 m (5 ft) intervals thereafter. The standard penetration tests were performed using a 140 lb safety hammer falling 0.75 m (30 inches) in accordance with ASTM procedures.

The subsurface conditions generally consist of 10 ft of fine to medium grained sands underlain by 1.2 m (4 ft) to 2.1 m (7 ft) of organic silts and peats. The sands are typically loose to medium dense based on their range in SPT N-values from 8 bl/0.3 m to 20 bl/0.3 m and average of 15 bl/0.3 m. The organic silts and peats are typically soft to very soft based on their range in SPT N-values from 2 bl/0.3 m to 4 bl/0.3 m and average of 3 bl/0.3 m.

Laboratory natural water content determination tests were performed on the samples retrieved from the split-spoon sampler. A large variation of in-situ water content was

observed ranging from 90% to as high as 310% with a typical range between 100% to 170%. Typically, the upper zones of the organic deposit contain a lower percentage of peat and thus have a lower water content. No laboratory consolidation tests were performed due to the extensive information obtained by the author's firm regarding the compressibility characteristics of the South Florida organic soils through over 30 years of experience with preloading, preload monitoring, and extensive laboratory consolidation testing.

A generalized representation of the subsurface conditions along the alignment of the proposed roadway is shown in Figure 3.

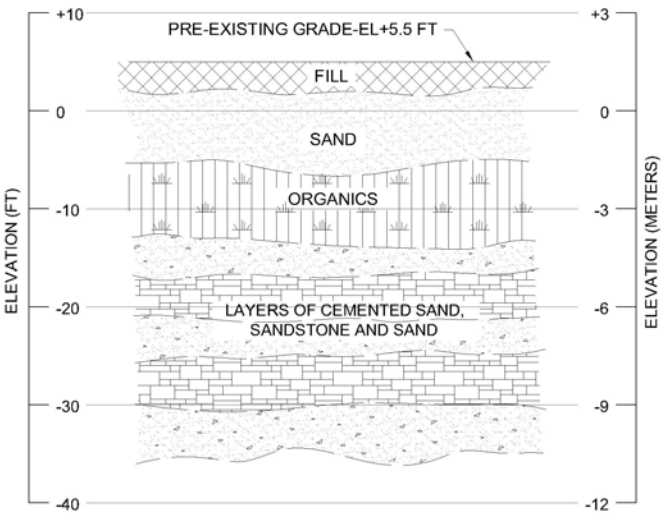

FIGURE 3-GENERALIZED SITE SPECIFIC SUBSURFACE CONDITIONS

COMPRESSIBLITY CHARACTERISTICS OF SOUTH FLORIDA ORGANIC SOILS

The organic silts and peats of South Forida are moderately to highly compressible in nature. Settlements are observed to occur over a relatively short duration upon application of surface loads, which is attributed to primary consolidation. In addition to primary consolidation, extensive and detrimental settlements have been observed over extended periods of time as a result of continual settlement associated with these organic soils, which is attributed to secondary compression.

Extensive information regarding the compressibility characteristics of the South Florida organic silts and peats has been obtained over the past 30 years of research by the authors' firm. In addition, an extensive preloading program was performed for the northernmost portion of the subject project, an adjacent golf course and country club development and adjacent 60 ½ Hectare (150 acres) site of single family two-story and townhouse structures designed by the authors' firm. This information allowed us to be able to accurately predict the resulting settlements of an earth embankment on these compressible ground conditions. Summarized below are some of the relevant properties of the organic silts and peats obtained through extensive laboratory consolidation testing (K.P.Yu, 1993). In addition, one back calculated data point is shown in each figure below based on the results of an instrumented preload performed within approximately 150 meters (500 ft) of the subject site.

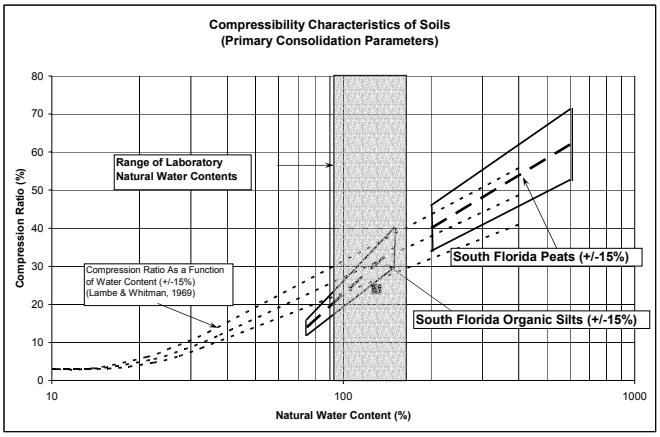

FIGURE 4- COMPRESSIBLITY PARAMETERS OF SOUTH FLORIDA ORGANIC SOILS

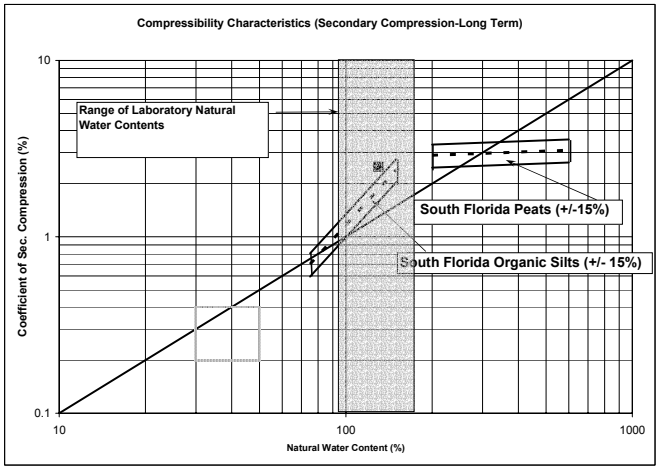

FIGURE 5- COMPRESSIBLITY PARAMETERS OF SOUTH FLORIDA ORGANIC SOILS

ROADWAY AND LOADING DOCK SUPPORT ALTERNATIVES & SOLUTIONS

The key geotechnical issues identified relative to raising of grades up to 1.7 m (5.5 ft) above previous existing grades were as follows:

- Post-Construction settlement of roadway embankment relative to pile supported Banquet Facility (i.e. abrupt differential settlements at transitions from on-grade to pile support);

- During and Post-Construction settlements of settlement-sensitive chiller lines and FPL ductbanks. Design requirements identified 1.25 cm (½ inch) of settlement as the tolerable level due to sensitive fiberglass utility line encasement system.
- During and Post-Construction settlements of the neighboring adjacent on-grade supported slab.
- Potential downdrag load which could be induced on the precast prestressed concrete piles supporting the wall of the adjacent Condominium's garage.

Predicted settlements as a result of placement of the roadway soil embankment were predicted utilizing conventional theories of elasticity and consolidation. Based on the settlement analyses, primary consolidation settlements were estimated to be in the range of 3.8 cm (1 ½ inches) to 5.1 cm (2 inches) within the center of the embankment and approximately 1.25 cm (½ inch) to 2.5 cm (1 inch) at the edge of the embankment adjacent to the neighboring condominium property wall. Contours of predicted settlement are shown below with respect to the anticipated filling scenario. It should be noted that with materials of this nature, secondary compression settlements (post-construction) can be significant. Secondary compression settlements on the order of 5.1 cm (2 inches) to 7.6 cm (3 inches) were predicted to occur after the completion of the filling process over the next 5 years to 20 yrs. Figure 6 shows the predicted contours of primary consolidation settlement and the table provided within the figure provides a summary of the predicted long term settlement over time.

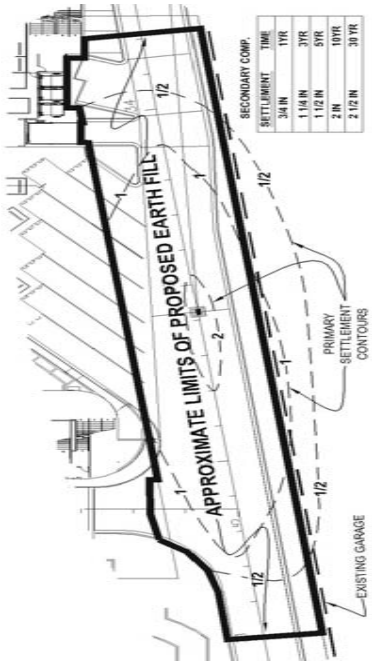

FIGURE 6- PREDICTED SETTLEMENTS DUE TO EARTH FILL

Due to the anticipated excessive settlements associated with both primary consolidation and secondary compression of the organic material, alternative support methods were investigated. Two alternatives consisting of a hollow structurally supported ramp/relieving platform and a geofoam lightweight fill were proposed to minimize construction settlements and limit post-construction settlements to tolerable levels. Due to the extensive network of installed utilities and the associated difficulty with installing pile foundations within and around these lines, the geofoam lightweight fill solution was chosen. The proposed EPS Geofoam lightweight fill roadway embankment construction consisted of removal/undercutting of the existing grade, placement of up to 1.2 m (4 ft) of EPS Geofoam lightweight fill and approximately 0.9 m (3 ft) of fill and pavement above.

EPS GEOFOAM LIGHTWEIGHT FILL EMBANKMENT DESIGN

Design of the geofoam lightweight fill embankment involved both external and internal stability issues, which is similar in nature to the design of a mechanically stabilized wall using geosynthetics. For this particular case, external stability of the geofoam embankment was not an issue as no soil fill was placed on either side of the geofoam embankment. The presence of the banquet facility structure on the north and the neighboring condominium's garage on the south prevented the presence of an external driving force. Design issues relative to transmitted lateral stresses to the adjacent walls, potential sliding of the pavement structure above the geofoam, and resistance to uplift during the design 100 year flood event were considered.

Internal stability of the geofoam was analyzed with respect to strain compatibility of the EPS geofoam under sustained and combined sustained and cyclic traffic loads. General design guidelines dictate that the induced strain within the geofoam should be within the range of 0.5% to 1.0% under sustained loading and should not exceed 1.0% under sustained plus cyclic loading. These guidelines have been established to limit creep deformations to a tolerable level and to maintain stresses within the elastic range of the geofoam behavior. See Figure 7 for a representation of the geofoam's characteristics under laboratory testing.

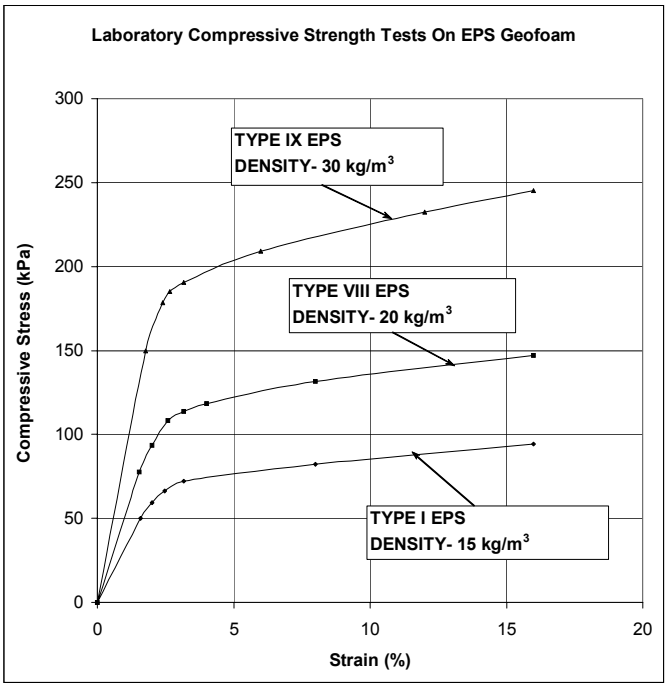

FIGURE 7- LABORATORY UNIAXIAL COMPRESSION TEST ON EPS GEOFOAM (BASF, 1995)

To allow the designer to specify the appropriate density of geofoam material, relationships have been developed for the initial tangent modulus (elastic modulus) based on the various geofoam densities. Figure 8 below shows this relationship graphically.

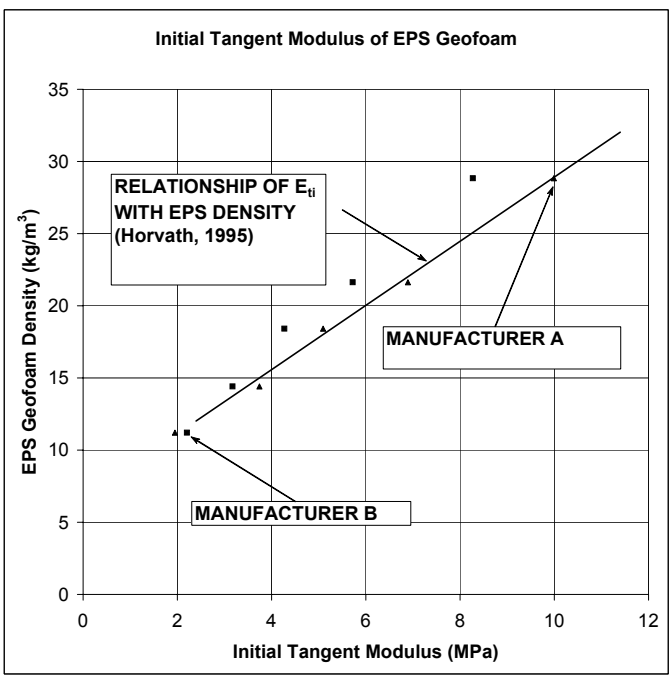

FIGURE 8- INITIAL TANGENT MODULUS RELATIONSHIP FOR VARIOUS EPS GEOFOAMS

EPS GEOFOAM LIGHTWEIGHT FILL CONSTRUCTION

In the fall of 2001, the EPS Type II Geofoam lightweight fill embankment was constructed as represented in Figure 9 below.

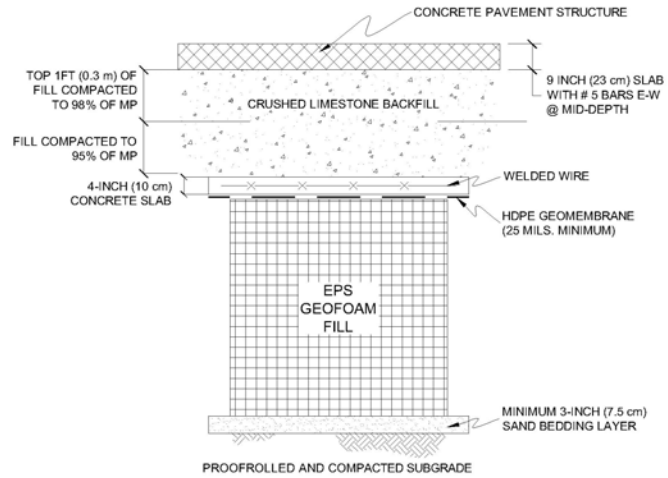

FIGURE 9- EPS GEOFOAM LIGHTWEIGHT FILL EMBANKMENT DETAIL (CROSS-SECTION)

Due to limited site access, the presence of an existing building on the south and the newly constructed banquet facility on the north, the geofoam construction occurred in two phases with the southern phase being constructed first. Prior to placement of the geofoam blocks, the existing grade was cut to 0.8 m (2.5 ft) below the previous site grade to provide minimal increase in stress within the underlying compressible organic stratum. The subgrade was then proofrolled with a large vibratory roller and a layer of fine to medium grained sand was placed and compacted to provide a level surface for placement of the EPS geofoam blocks. The blocks were then placed in such a manner that no joints/seams were aligned, and the uppermost blocks were positioned with the long dimension of the block being oriented perpendicular to the direction of traffic flow. The thickness of the EPS block embankment averaged 1.2 m (4 ft). After completion of placement of the EPS Geofoam embankment, a geomembrane and a concrete protection layer were constructed to allow subsequent filling operations to be safely performed and to provide protection from future contact with petroleum products over the life of the roadway. Utilities were either routed on top of the geofoam protection layer or beneath the bottom block of geofoam. After curing of the concrete protection layer, filling commenced to the planned pavement subgrade elevation using imported clean crushed limestone having a maximum dry density of 20.5 KN/m^3 (130 pounds per cubic foot), an optimum moisture content of 8%, and a Limerock Bearing Ratio in excess of 100 (i.e., Florida Test Method FM 5-515). The fill was placed in 0.3 m thick (1-ft-thick) loose lifts using light

construction equipment for the first lift and standard construction equipment for the subsequent lifts. To ensure that the geofoam was not overstressed during the construction filling process or under future traffic loading, an earth pressure cell was installed to obtain measurements of total stress on the surface of the geofoam and induced stresses from construction traffic. Each lift was compacted to at least 95% of the material's maximum dry density in accordance with the Modified Proctor Compaction Test and to at least 98% of the material's maximum dry density for the upper most lift of fill immediately beneath the concrete pavement.

The following construction photographs show various aspects of the construction process and the instrumentation installation procedure.

PHOTOGRAPH A- EPS LIGHTWEIGHT FILL PLACEMENT

As previously mentioned, an earth pressure cell (i.e. Geokon Model 4800 Vibrating Wire Pressure Transducer) was installed on top of the geofoam embankment prior to placement of the concrete protective layer to monitor the stresses induced within the surface of the geofoam embankment. A series of measurements were made under the dead weight of the compacted fill and under various construction traffic loadings. The photograph below shows the installation of the earth pressure cell.

PHOTOGRAPH B- PRESSURE CELL INSTRUMENTATION INSTALLATION

EPS GEOFOAM LIGHTWEIGHT FILL STRESS MONITORING PROGRAM

The two tables below summarize the results of the stress monitoring program and provide the details of the stress measurements obtained under the superimposed construction traffic loading.

EQUIPMENT	TOTAL EQUIP. LOAD	TOTAL CONTACT AREA	CONTACT AREA
D-3 Dozer	9.5 tonnes (21,000 lbs)	Two 2.75 m (9 ft) by 0.3 m (1 ft) wide tracks	Centerline of One Track
IR SD-70D Roller	7.1 tonnes (15,750 lbs)	One steel wheeled roller	Centerline of Roller
Fully Loaded Tandem Axle Fill Truck (i.e. 20 tons of Fill Soil)	31.3 tonnes (69,000 lbs)	Three Axles w/tires	Centerline of one side of back axle underneath tires
Fully Loaded Concrete Truck (i.e. 9 cubic yards of concrete)	29.9 tonnes (66,000 lbs)	Three Axles w/tires	Centerline of one side of back axle underneath tires
Concrete Pump Truck with Extended Boom	Not Available	Axles w/tires and Pad Supports	Centerline of Pad Support

TABLE 1- CONSTRUCTION EQUIPMENT INFORMATION

EQUPMENT	DEPTH[1]	INDUCED MEASURED STRESS
D-3 Dozer	0.25 m (10 in)	10.8 KN/m^2 (225 psf)
IR SD-70D Roller	0.71 m (28 in)	7.2 KN/m^2 (150 psf)
Fully Loaded Tandem Axle Fill Truck	0.71 m (28 in)	15.8 KN/m^2 (330 psf)
Fully Loaded Concrete Truck	0.71 m (28 in)	18 KN/m^2 (375 psf)
Concrete Pump Truck	0.71 m (28 in)	22.7 KN/m^2 (475 psf)

1- The depth is the distance from the underside of the contact area to the top of the earth pressure cell.

TABLE 2- MEASURED CONSTRUCTION INDUCED STRESSES AT TOP OF UPPERMOST GEOFOAM BLOCKS

EPS GEOFOAM LIGHTWEIGHT FILL QUALITY ASSURANCE/CONTROL PROGRAM

As a means of quality control, a two phase quality control/assurance program was implemented both at the manufacturer's plant and within the field at the project site. The first phase of the quality control program consisted of performing density tests on the manufactured geofoam blocks prior to shipment to the project site. The second phase of the quality control program consisted of obtaining samples from the geofoam blocks delivered to the project site and performing both density tests and unconfined compressive strength testing in accordance with ASTM C-303 and ASTM 165, Procedure A, respectively. The results of the testing program showed that the density of the EPS Type II geofoam varied from 0.25 KN/m^3 (1.6 pcf) to 0.27 KN/m^3 (1.75 pcf) and averaged 0.26 KN/m^3 (1.65 pcf). The results of the compressive strength testing showed the average compressive strength to be 155 KN/m^2 (22.5 psi). Both the compressive strength test results and density test results met the material requirements for EPS type II geofoam.

CONCLUSIONS

The following conclusions were drawn from the experienced gained with construction of the EPS Type II geofoam supported roadway in South Florida.

1. The current design procedures regarding stress distribution of traffic loads based on conventional theories of elasticity or design manual recommendations (i.e. 2V:1H distribution) are conservative based on the results of the stress monitoring program.

2. EPS geofoam provides a technically feasible alternative for support of roadways in compressible ground conditions.

3. The roadway has performed as expected based on the results of settlement monitoring. Post-construction settlements of less than 0.65 cm (¼ of an inch) have been measured up to 6 months after completion of the roadway and no signs of pavement distress have been observed up to 2 years after completion of construction.

4. The design and construction team member's continual efforts along with the assistance of the EPS geofoam manufacturer allowed the successful construction of one of the first geofoam supported roadways in South Florida.

5. Full-time field observation and monitoring of both induced stress within the uppermost block of the EPS Type II geofoam and settlement of the roadway embankment provided the design team with a reasonable assurance that the roadway would perform as intended.

ACKNOWLEDGEMENTS

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APPENDIX I.
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