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THE USE OF STONE COLUMNS ON SETTLEMENT AND LIQUEFACTION SUSCEPTIBLE SOILS

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The new Paradisus Coco Beach Resort in Rio Grande (PR) is located on an old swamp area, which has been filled with relatively clean to silty sands. The swampy deposits consists of organic silts, peat and loose fine to medium sand and silty sand. The project requires the placement of 1.0 to 1.5 meters of additional fill together with the construction of light structures. The need for the fill will trigger the development of settlements in the underlaying weak ,and compressible stratum. Furthermore, the susceptibility of the loose sand to liquefaction during an earthquake was considered.

This paper describes the soil improvement by means of vibro-replacement, the purpose of which was threefold: reduction in total and differential settlement, acceleration of settlements during the surcharge period and densification of the loose sand to reduce its liquefaction potential. The predesign is presented together with relevant construction details of the preliminary trial areas from which the final column diameter and grid spacing were derived. Instrumentation together with settlement observations during the surcharge period are presented as well and compared with the initial predictions.

Introduction

Sol Melia's Paradisus Coco Beach resort is to be located at the town of Rio Grande, in the northeastern coast of the Puerto Rico Island. The 490 room resort, will be erected on 44 acres within Miquillo Point, a portion of land underlaid by swamp deposits and sand bars. The resort will consist of thirty (30) Villas and Bungalows (one-story), clubhouses, restaurants and a main Service/Entertainment Building (S/E Building) where the casino, administration, ballroom and other related facilities will be accommodated.

At the Miquillo Point, the natural topography is relatively flat. Flooding considerations, however, require the site to be upgraded 1.0 to 1.5 meters from existing grade. The need for this fill presented one of the major problems for developing the site, since this blanket-type loads would trigger large settlements in the known weak and compressible subsoil.

The swamp deposits and sand bars are composed of loose sand, organic silts and peat, all of these underlain by coralline rock. In general, the soil stratigraphy is comprised of a relatively clean beach sand horizon, over soft organic silts and loose silty sands, over cemented coralline rock. Towards the central portion of the farm, there are lens or pockets of fibrous peat and organic silts within the loose sand layer.

Typically, the uppermost sand layer is about 1.2 m thick (4 ft) and shows average N-values of 10 to 20 . The loose sand and soft sandy silt, underlie the relatively clean sands to an approximate depth of 9.1 m (30 ft). Most of the N-values in these soils ranged from 0 (weight of hammer) to 4. Natural moisture contents varied between 22% and 78%.

Towards the central portion of the farm, the loose sand and soft sandy silt show a sub-horizon of fibrous peat from 1.2 to 3.9 m (4 to 13 ft). Natural moisture contents within the peat horizon vary from 95 to 280 percent and N-values from 1 or 2.

The basement soil is made of coralline rock and/or cemented sand with average N-values of 60 . This zone, typically below 9 or 10.7 m (29 to 35 ft), posses good bearing capacity and low compressibility characteristics.

Observations made at the time of exploration indicate ground water levels varying from 0.6 to 1.2 m (2 to 4 ft), measured from prevailing ground surface elevation.

Liquefaction Considerations

Due to the loose density state observed in the sandy and silty soil deposits, and the high position of the water table, the susceptibility of the subsoil to liquefaction needed to be investigated.

Liquefaction is a phenomenon in which the strength and stiffness of a saturated soil is reduced by earthquake shaking or other rapid loading. Liquefaction susceptibility was evaluated by determining the soil compositional characteristics (particle size and gradation) and by evaluating cyclic shear stresses. These factors have been extensively investigated in the past and methods for evaluation have been developed by Seed and Idriss (1975).

Physical characteristics like grain size distribution and plasticity were evaluated. Most of the representative soil samples shown a clay fraction smaller than 15%, a D_{50} between 0.02mm and 2.0 mm, liquid limits (LL) below 35% and natural water contents greater than 0.90 LL. According to the compositional criteria, these characteristics make the loose sand and silt layer to have a tendency for liquefaction

For the case of evaluating liquefaction from the cyclic shear stress standpoint, a comparison between the earthquake loading (CSR) and the cyclic resistance ratio (CRR) was performed. From the expression F.S=CRR/CSR it was concluded that the sandy material extending to 9 m was susceptible to liquefaction. Factors of safety against liquefaction between 0.65 to 0.93 were obtained for a ground surface acceleration of 0.15g, which may correspond to an earthquake of Magnitude 7.5 in the Richter scale.

The probable settlement of the saturated sand deposit due to earthquake shaking was estimated using a relationship between the cyclic stress ratio, the N-value and the volumetric strain (Tokimatsu and Seed, 1987). Based on this simplified method of analysis, the predicted earthquake-induced settlement in the saturated sand deposit would be in the range of nine (9) inches.

Settlement Considerations

Another aspect that required evaluation was the settlement behaviour of the organic silt and peat under structural and permanent fill loads. Based on the load intensity resulting from the fill upgrading (~30Kpa) and mat foundations (28.7Kpa), primary consolidation settlements were estimated from 25 to 71 cm (10 to 28 inches). This amount of movement in the proposed structures would result in significant damage and loss of function, not to mention the effects of liquefaction-induced settlements, as these were estimated in 23 cm (9 inches).

Design Conclusions

Based on the results of liquefaction and settlement analyses, it was determined that a deep foundation system or a method of ground modification would be required for developing the site. This requirement brought into attention two main alternatives; 1. Pile foundations or, 2. Ground modification using Vibro-replacement (Stone Columns). Vibro-compaction and dynamic compaction were also considered, but the presence of silts and peat in the soil profile rendered them unsuitable alternatives.

Due to 1 or 1.5 m thick fill load (3.3 to 5 ft), both aforementioned alternatives would need to be assisted by the surcharge method of soil stabilization. Otherwise, the project site would suffer significant levelling problems between treated and non-treated areas. Geared by the need for mitigating the liquefaction potential and a tight time schedule that would not allow long surcharge stabilization periods, the alternative of using stone columns was selected. At the end, the benefits of the stone columns would be threefold: reduction in total and differential settlement, acceleration of settlements during the surcharge period and densification of the loose sand to reduce its liquefaction potential.

Liquefaction Mitigation

Stone columns mitigate liquefaction by means of two principal ways: 1. By increasing drainage which dissipates excess pore water pressure generated by earthquake loading and, 2. By increasing the SPT values which influence the Cyclic Stress Ratio.

The analysis of drainage for stone columns was carried out by evaluating, via finite element techniques, the extent of excess pore water pressure developed with respect to initial effective stress, as specified by the pore pressure ratio, r_u, as a function of drain geometry and spacing. Upon ground shaking, the pore pressure ratio r_u increases to a point where significant settlements can develop. After this point further increase of r_u will make the soil to loose shear stresses and liquefy.

The analysis method is very sensitive to the drainage characteristics of the in-situ soils and the drainage capability of the stone columns, therefore in-situ permeability tests were performed to establish these values. The in-situ test obtained permeability values of 1.6x10-4 cm/s for the silty soils ($5.25x10^{-6}$ ft/s) 8.2x10-2 cm/s and ($2.7x10^{-2}$ ft/s) for the stone columns. Due to the inherent installation process of stone columns, the permeability in the stone column resulted 60 times less than the typical permeability of clean, 3.8 cm gravel.

The finite element code evaluation showed that 0.91 m diameter (36 inches) stone columns spaced at 3 m c-c (10 ft) would be able to maintain the maximum pore water pressure ratio within acceptable values. Therefore, it was concluded that this geometry and spacing would be adequate to mitigate the liquefaction potential for a ground acceleration of 0.15g.

The reduction of total overburden pressure that results from the soil improvement and the favourable load distribution obtained from the installed stone columns was also investigated to determine how these affect the cyclic stress ratio. A simplified procedure introduced by H.J. Priebe, suggests a reduction in the CSR induced by an earthquake, by the ratio as shown in Figure B. For the case of 3 m (10') spaced stone columns, the reduction factor for the induced CSR corresponds to 50 percent. Upon the application of this reduction, the Factor of Safety against liquefaction rose above 1.3.

Settlement and Bearing Capacity Analyses

The settlement evaluations considered 1.5 m (5 ft) of permanent fill and approximately 28.7 Kpa (600 psf) of distributed structural load below Bungalows and Villas. For a surcharge ratio of one (1) and the assistance of 3 m c-c, 0.91-mt diameter stone columns, the time for stabilization would be in the order of 2 to 3 months. The average settlement expected at areas with peat was around 71 cm (28"). Zones not underlain by peat would suffer theoretical settlements of around 25 cm (10").

Nonetheless, due to the replacement effect of the stone columns, actual (field) settlement should be less than the theoretical values. Greenwood and Thomson (1984) suggest an empirical method for estimating field settlements after considering the replacement factor (Figure C). Based on this method, field settlements were estimated to be 50 percent less than the theoretical values, or from 13 to 35 cm (5 to 14").

Bearing capacity was estimated based on a formula given by Hughes et al. (1975). Since the resulting stone columns would easily be more than about 10 times stiffer than the surrounding weak soils, it was considered that the stone columns would carry the foundation loads with little or no contribution from the intermediate ground. For distributed loads at zones with no peat, it was concluded that the 3 m c-c- stone columns would safely sustain one-story Bungalows and Villas (28.7Kpa, or 600 psf) without bearing capacity or deep-seated settlement problems. At zones underlain by the peat layer, a closely spaced grid of 2 to 4 stone columns installed at 1.7 m c-c (5.5 ft) was considered necessary below spread footings for an allowable contact pressure of 96 Kpa (~2,000 psf). For strip footings at these critical zones, the stone column spacing was set at 2.75 linear meter (9 ft). Lightly loaded floor slabs did not present bearing capacity problems, thus a stone column array 3.35 m c-c (11 ft) was selected mainly for liquefaction mitigation and settlement acceleration.

Field Testing

At the beginning of the project, several test areas were selected in order to evaluate the degree of densification obtained by different stone column arrays. Three test areas were prepared with 2.75, 3.0 and 3.3 m c-c triangular arrays. After a waiting period of 72 hrs, one test boring was performed at the centre of each pattern.

The results of the testing program showed that the increase in density in the clean to silty sand, to a value necessary to mitigate liquefaction, was achieved for the three triangular arrays. Typical post-treatment borings recorded N-values between 11 and 30 , in contrast to pre-treatment values of 3 to 17 . All test columns were installed using a minimum average current of 150 amps.

Having these results at hand, and the analyses on bearing capacity and settlements, it was decided to use the 10 ft c-c triangular array for Bungalows and Villas, and 11 ft c-c triangular array below floor slabs at the S/E Building. Depending on the size of spread footings, special arrays consisting of 2 to 4 stone columns at 1.7 m c-c were chosen for the spread footings at the S/E Building.

Construction

A total of 8,033 stone columns were installed by three crane-suspended vibro-floats 360 mm in diameter. The top feed, wet method was used. The vibro-float was driven into the ground by a 130kW electric motor and water jets located at its tip. In this method the float is advanced to the required depth, and then retracted in 0.5-meter increments while 2.5 to 5 cm gravel backfill is dumped into the hole. During this process the vibrofloat compacts the gravel vertically and radially into the surrounding soft soil. The process of backfilling and compaction by vibration continues until a dense stone column, some 1.1 to 1.2 m in diameter, reaches the surface.

Stone Column Installation

Among the most important quality control measures during the installation of stone columns was to monitor the amperage reading during the densification process and to measure the amount of stone added to each column. Quality control required the vibrator to be re-penetrated repeatedly until the specified current of 150 Amps was achieved. The installation rigs were equipped with automatic recording units that recorded on a continuous basis the power drawn by the electric motor of the vibrofloat. A field technician assigned to each rig assured that the necessary current was achieved in each column and kept record of the amount of stone used for each column.

The amount of stone added to each column was determined by measuring full loader buckets. These measurements were used to determine the average column diameter and the total amount of stone used in each column for payment purposes. Each 9 m in average deep column required about 13 cubic meter of gravel back fill, resulting in an equivalent diameter in excess of 1.20 m after considering 15% due to losses and compaction.

Monitoring of Settlements

Several monitoring stations were installed throughout the project site to record settlements and establish the completion time of the stabilization period. Settlement stations installed at the area of Bungalows and Villas recorded maximum downward movements of 9.1 cm (3.5 inches) in a 2-month period. This maximum settlement resulted about 30 percent less than the predicted 5 inches but the total time for stabilization resulted just as anticipated.

At a particular area of the S/E Building, where the peat layer was found, settlements reached 19 cm (7.5 inches) in a period of 2 months. At the time of this Paper, the S/E Building area is still being monitored for further settlements. For the case of peaty soils, the settlements observed were 50 percent less than predicted. As planned, stabilization occurred in a period of 3 months.

Conclusion

The installation of 8,033 stone columns below mat foundations, floor slabs and spread footings resulted successfully both from the technical and time schedule standpoints.

From the technical point of view, the columns will safely provide adequate bearing and control of settlements as corroborated by the SPT program performed at the end of the vibro-replacement works. In addition, the increased density and drainage characteristics of the improved subsoil will mitigate the occurrence of liquefaction during a seismic event.

Regarding the project time schedule, the reduction in settlements and the acceleration of stabilisation time allowed the project maintain a tied calendar schedule without the need for time extensions associated to the surcharge method.

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