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APPLICATIONS OF GROUND IMPROVEMENT FOR NUCLEAR POWER PLANT SITES

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INTRODUCTION

Siting of a nuclear power plant (NPP) requires geotechnical evaluation for a potential site. Due to certain constraints in the site location selection, problematic soil conditions, such as soft clays, deep deposits, loose sands and silts, etc., are commonly encountered during site evaluation. These problematic soil conditions may cause undesirable ground/foundation behaviours including 1) excessive or differential settlement under static structure loading, 2) insufficient margin of safety against bearing capacity failure, 3) high potential for excessive pore pressure and associated earthquake liquefaction and 4) high seismic demand due to local site amplification. Excavation-and-replacement is commonly used to address these issues because this approach is considered a prudent approach. Other ground improvement techniques, such as grouting or soil mixing, have also been used in the nuclear industry to improve ground conditions and structure behaviour. However, the applications of these techniques are not as widespread as in non-nuclear industries. The major challenges for the applications of these techniques are to meet nuclear quality assurance requirements and to obtain regulatory approval. Some general considerations for fulfilling nuclear quality assurance requirements are first discussed in this paper, followed by a number of successful case studies on the application of various ground improvement techniques, including: deep soil mixing/grouting, excavation-and-backfill, and Roller Compacted Concrete (RCC) fill for a number of NPP sites worldwide. The paper is concluded with some general recommendations on the applications of ground improvement for the NPP sites.

GENERAL CONSIDERATIONS

While there is no universal code for the application of ground improvement in NPP sites, NUREG-0800 Section 2.5.4 (USNRC, 2010) and IAEA Safety Guide No. NS-G-3.6 (IAEA, 2004) provide some general guidance. Based on the review of these documents, there are no express limitations or constraints on the ground improvement system adopted for a NPP site. However, the regulatory processes in the nuclear industry can be lengthy and require a thorough review. The length and level of scrutiny can be even higher for cases with innovative solutions. In order to fulfil nuclear quality assurance requirements and achieve regulatory approval, the following procedures may be generally followed:

1. Conduct comprehensive engineering analyses to determine the subsurface response for the prior-to improvement condition;
2. Develop a conceptual ground improvement plan;

3. Obtain regulatory approval for the conceptual plan;
4. Conduct comprehensive engineering analyses to assess the subsurface response for the improved condition;
5. Obtain regulatory approval on the effectiveness of the ground improvement plan;
6. Develop a detailed work plan for the field monitoring and testing program to finalize the ground improvement parameters and to verify the improved conditions in the field;
7. Implement the field monitoring and testing program according to the work plan; and
8. Develop a report on the final ground improvement parameters and recommendations or specifications for the implementation of the ground improvement for the actual construction.

Due to various uncertainties in the subsurface conditions and the ground improvement process, steps 6 and 7 are particularly important to ascertain that the targeted improvement goals are achieved. Regulatory approval in step 5 may be conditional on the completion of steps 6 and 7. In the following sections, a number of case studies are presented to demonstrate some of the technical aspects in the above procedures. The site-specific challenges and the effectiveness/advantages of the proposed foundation schemes are highlighted in each of the case studies.

VOGTLE UNIT 3 SITE

Vogtle Unit 3 Site is a deep soil site with subsurface composed of three strata: 1) Upper Sand Stratum predominated by sands, silty sands, and clayey sands with occasional clay seams, 2) Marl Bearing Stratum consisting of Blue Bluff Marl (BBM) formation and 3) Lower Sand Stratum comprised of fine-to-coarse sand with interbedded silty clay and clayey silt. The Upper Sand layer is approximately 90 ft thick and judged inadequate to meet the total and differential settlement criteria in AP 1000 Design Control Document (DCD) (Westinghouse LLC, 2011). The decision was to excavate the entire stratum and replace it with engineered compacted fill. Besides being a prudent approach, excavation-and-refill is particularly effective in reducing the potential building interaction under seismic loading. A site strata profile, after excavation and backfill, is shown in Figure 1.

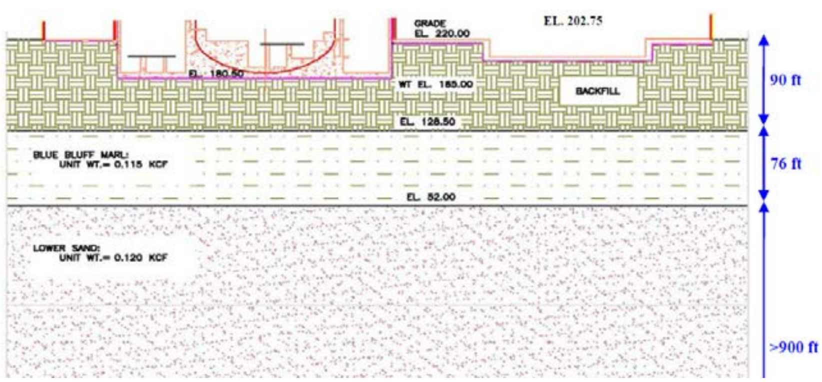

Figure 1. Vogtle Unit 3 Site Strata profile

The margin in settlement is anticipated to be small even with stringent quality control in the backfill placement, due to the large structure load (e.g. approximately 9 ksf for Nuclear Island [NI]). Therefore, settlement analysis and a proper settlement monitoring program become critical for fulfilling the DCD settlement criteria and for the successful construction and succeeding operation of building structures. The 3D finite element software PLAXIS 3D Foundation (Brinkgreve and Broere, 2008) was used to conduct the settlement analysis. In addition to its superior capability to model ground behaviour (e.g. excavation, dewatering, etc.), PLAXIS is capable of modelling structural elements and phased construction. This feature facilitates analysis of the evolution of foundation and superstructure rigidity as construction incorporates structural components; and enables a more accurate assessment of the differential settlement as construction progresses. The PLAXIS 3D model for settlement analysis is illustrated in Figure 2.

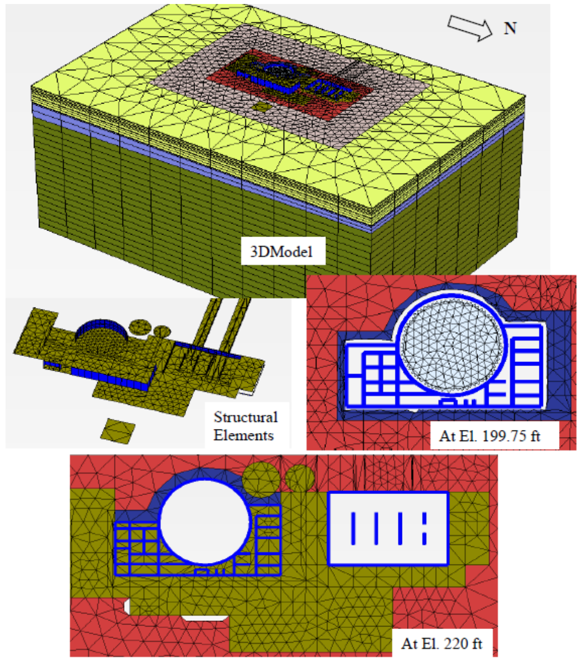

Figure 2. PLAXIS 3D model for settlement analysis

The excavation-dewatering-backfilling was conducted in five phases and completed in a time period of 32 months. 56 months were planned for building construction in eight phases. All these phases were modelled with PLAXIS in a consecutive sequence. A settlement monitoring program was planned for all the phases. The settlement monitoring results from earlier phases were used to calibrate the soil parameters, especially the stiffness, to be used in later phases. As the excavation-backfill and construction spanned a sufficiently long time period, 80 to 90 percent of the excess pore pressure was dissipated even in the least permeable BBM layer. As such, drained behaviour is assumed for all the layers and effective stress soil parameters are adopted for the settlement analysis. This assumption was validated from the pore pressure monitoring results.

To account for the potential nonlinear behaviour and plastic deformation of the soil strata, an advanced constitutive soil model (i.e. PLAXIS Hardening-Soil Model) was used in the settlement analysis. This model can capture the effect of the imposed loads on the soil stiffness. As a result, the pressure-dependent stiffness of backfill material can be modelled without additional material zonation and refinement in the finite element model. In addition to the settlement monitoring results from earlier stages, another useful source for backfill stiffness calibration is the shear wave velocity (Vs) measure. It should be noted that the modulus computed directly from Vs corresponds to the so-called small-strain modulus; the small-strain modulus should be reduced appropriately to be compatible with the strain level under static structure loading. A reduction factor of 0.15 was considered for the backfill material.

Two finite element models were considered in the settlement analysis: a BE model with best-estimate soil properties and best available loading and construction sequence information; and an AP1000 DCD limit case in which the BE model soil parameters are reduced such that the computed settlements approach the AP1000 DCD requirement. The latter is necessary for guiding the settlement monitoring program and re-evaluation of site-specific settlement if the measured settlement is close to the settlements computed in the DCD limit model.

LEVY NUCLEAR PLANT SITE

Subsurface at the Levy Nuclear Plant (LNP) Site consists of Quaternary age fluvial and terrace sediments, primarily silty fine sands, overlying the Avon Park Formation (Duke Energy Florida, 2016). The Floridan aquifer system in the Avon Park Formation at the LNP site consists of interbedded carbonate rock units such as fossiliferous limestone, dolomitized limestone and dolomite. The permeability of these interbedded units is generally high but varies due to rock porosity and many other factors. Different methods, including field observations, geophysical testing, and rock coring, etc., were used to quantify the vertical and horizontal dimension of the karst features at the LNP Site. The thicknesses of the karst features are typically limited to less than 5 ft and the average width-to-height ratio is 1H:5V. These existing karst features pose challenges for subsurface stability and structure construction.

To mitigate the potential excessive surface deformation associated with the cracks and voids in the rock units, a Roller Compacted Concrete (RCC) bridging mat concept was developed to bridge the postulated karst features. The sediments and rock beneath the nuclear island need to be excavated for the placement of RCC. The high groundwater table and high permeability of the Avon Park Formation at the Site are certainly unfavourable factors for the excavation. A "bathtub" dewatering scheme was developed, which is composed of a diaphragm wall on the perimeter of excavation and a grouted zone at the bottom of the excavation. This scheme will permit excavation at more than 60 feet below groundwater table. The upper 75 ft of the Avon Park Formation will be grouted to diminish its porosity and thus minimize the permeability. In addition to forming a "bottom of bathtub" to prevent up-flow of groundwater through the bottom of the excavation, the grouting can also protect the excavation base from heaving. The "bathtub" scheme will facilitate the excavation to be conducted in a more predictable manner.

RCC fill will be placed after excavation; the RCC fill will serve two main purposes: 1) replace the weakly cemented Tertiary sediments, thus creating a uniform subsurface with increased bearing capacity; and 2) bridge conservatively postulated karst features. The largest single potential karst feature identified has width of 3.9 ft. The RCC Bridging Mat was conservatively designed to span a 10 ft void beneath the Bridging Mat. The grouting, excavation and RCC refill plan is illustrated in Figure 3.

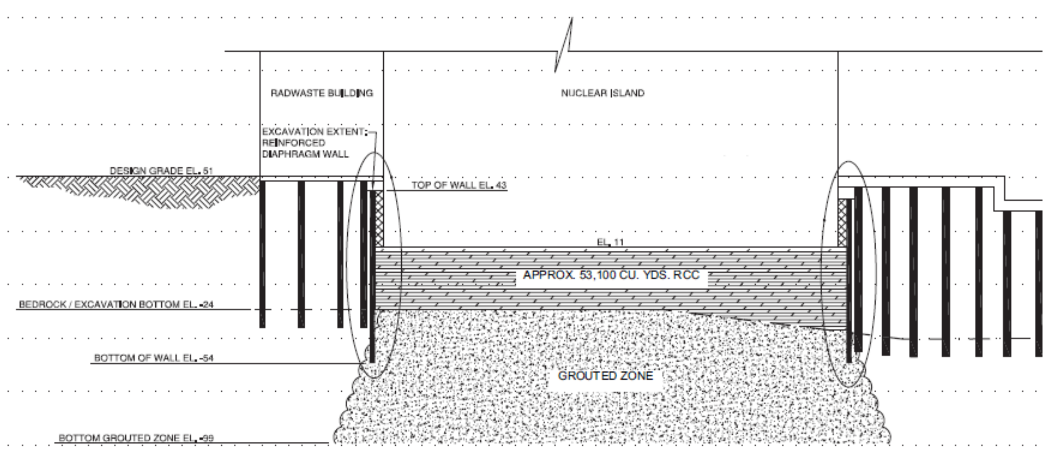

Figure 3. Levy Nuclear Plant Unit 1 grouting, excavation and RCC refill plan

Both field measurements and laboratory tests were planned to ascertain the properties of RCC fill meet the predefined acceptance criteria. Analyses show that both settlement and bearing capacity have relatively large margin with properly placed RCC fill. Although the grouting is expected to increase the stiffness and strength of the Avon Park Formation, these effects are not counted in the analyses such as bearing capacity, settlement, etc.

ANONYMOUS SITE 1

The target site is considered as a deep soil site with bedrock located at the depth of 1600 to 2000 meters. The subsurface consists of loam, fine silty sand, gravel sand, and stiff silty clay. Despite their different geological origins, the soil mechanical properties are relatively uniform, with a representative shear wave velocity of 422 m/s and representative deformation modulus of 135 MPa. Under the highest target foundation pressure of 910 kPa (19 ksf), the estimated settlement for natural soil condition is 84 cm, which is significantly greater than the limit requirement of 30 cm. A block-type of improvement with jet grouting and treatment thickness of 11 m was attempted first, which results in an estimated settlement of 60 cm, still significantly greater than the limit requirement.

Considering that further increase in the treatment depth for the block-type improvement may cause significant increase in the cost, a “lattice” type improvement was proposed as a conceptual plan. The “lattice” system will be composed of a grid of jet grouting elements with spacing of 4 m and inclusion of rigid concrete element (cast-in-place pile) at grid intersections, as illustrated in Figure 4.

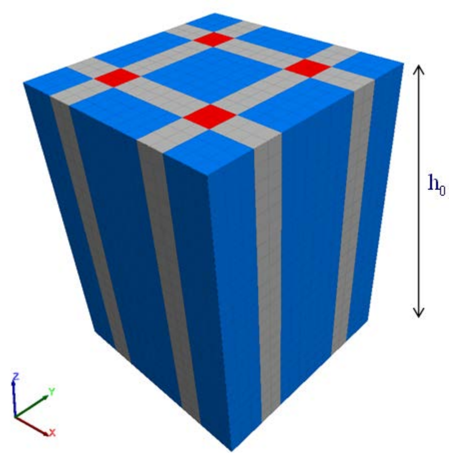

Figure 4. “Lattice” system (blue as in-situ gravel, grey as grouted grid and red as cast-in-place concrete)

The representative volume as shown Figure 4 was modelled as a macro element using FLAC 3D to obtain the equivalent elasticity modulus of the “lattice” system. The equivalent modulus is 602 MPa for the 11 m thick lattice system, and 731 MPa for a 14 m thick system. The estimated settlements (using an equivalent axisymmetric PLAXIS 2D model) for the 11 m thick and 14 thick systems are 64 cm and 53 cm respectively, still greater than the limit requirement of 30 cm.

The preliminary results indicate the grid should be denser, or the piles at the corners of the lattice be deepened to meet the limit requirement. New analysis suggested that the concrete inclusion elements should extend another 15 m below a 12 m thick lattice system. This system has an equivalent modulus of 930 MPa and results in an estimated total settlement of 26 cm.

Liquefaction is another concern for the site. The in-situ sands are judged to be liquefiable subject to the seismic margin earthquake (SME) with peak ground acceleration (PGA) of 0.49 g. It is anticipated that the proposed lattice system should be able to preclude the liquefaction concern. The equivalent shear wave velocity (Vs) of the lattice system may be used in the liquefaction initiation assessment. The equivalent Vs may be derived from the modulus of the representative volume as shown in Figure 4.

ANONYMOUS SITE 2

The target site is another deep soil site with bedrock located at depth of several kilometres. Although the subsurface soils are highly inhomogeneous and classified as various geological units, they are relatively uniform in terms of physical and mechanical properties. The reference elasticity modulus (PLAXIS hardening-soil model) of the bearing layers varies from 36 MPa to 73 MPa. The cohesion varies from 36 kPa to 45 kPa and the friction angle from 21 degrees to 24 degrees. For a reactor building load of approximately 600 kPa (12 ksf), settlement and bearing capacity analyses for the natural soil condition indicate neither settlement nor bearing capacity has sufficient margin against the limit requirement.

Deep soil mixing (DSM) was proposed to increase the overall subsurface stability. Settlement, bearing capacity and liquefaction initiation analyses were performed for both natural and post DSM improvement conditions. Four stages: dewatering, excavation, construction and rewatering, were modelled in the settlement analysis. These analyses show that settlement and bearing capacity have sufficient margin after DSM improvement. There is no liquefaction risk for the site with the weak surface layer excavated during construction.

Another concern for the target site is the high seismic demand; the 100 Hz spectral acceleration in the ground motion response spectrum (GMRS) is 0.44 g. It was hypothesized that DSM would also serve to reduce the seismic demand as the improved zone forms a stiff layer at surface with overall higher seismic impedance. This hypothesis was validated through a series of site response analysis (SRA) for the prior-to improvement condition and post improvement condition. One-dimensional random vibration theory (RVT) based equivalent linear analysis is used to conduct the SRA. It is argued that the one-dimensional analysis provides a good and acceptable estimate of the seismic response for the improved zone because the horizontal extent of the soil improvement is significantly larger than the thickness of DSM improvement. Equivalent linear approach is considered valid by inspection of the strain levels out of the analyses; they are well below 0.1% to 0.3%.

A site geotechnical model, consisting of shear wave velocity, mass density, layer thickness, low strain damping and strain-dependent dynamic properties for each soil stratum, was developed for the SRA. The natural soil condition was modelled as a soil column with total depth of 120 m. A scenario based controlling earthquake was developed based on the review of available regional seismic hazard studies in the literature, to be used as input motion for SRA. The amplitude of the input motion is highly uncertain and largely dependent on the assumed triggered soil nonlinearity level. Two “extreme” assumptions were made for the soil nonlinearity level, which results in two sets of input motions with very different amplitude, but both generated similar surface response with peak ground acceleration of 0.44 g for the natural soil condition.

These input motions were then used in the SRA sensitivity studies for the excavated conditions with and without DSM improvement; both modelled as a soil column with total depth of 110 m, but with various thickness and stiffness (or shear wave velocity, Vs) for the improved zone. The surface of the 110 m soil column corresponds to the foundation level of reactor building. The thicknesses for the improved zone are considered as 20 m and 30 m; and shear wave velocities of 700 m/s, 800 m/s, 1000 m/s and 1200 m/s are assumed for the DSM improved material.

The surface spectral acceleration ratios, relative to the base case without DSM, are presented in Figure 5 for an improved zone with thickness of 20 m and a range of target Vs values. It is observed that for frequencies higher than 3 to 4 Hz, the DSM improvement is an effective means to reduce the surface response. Larger Vs in the improved zone will result in better reduction in surface response. The surface peak ground acceleration (PGA) is plotted as a function of Vs values in the improved zone for both 20 m and 30 m improvement in Figure 6. It is noticed that the effect of reduction is more evident when the Vs increased to 700 m/s to 800 m/s; the reduction becomes insignificant with further increase in the Vs values. Also, it can be seen that the effect of 30 m improvement is slightly better than that of 20 m improvement.

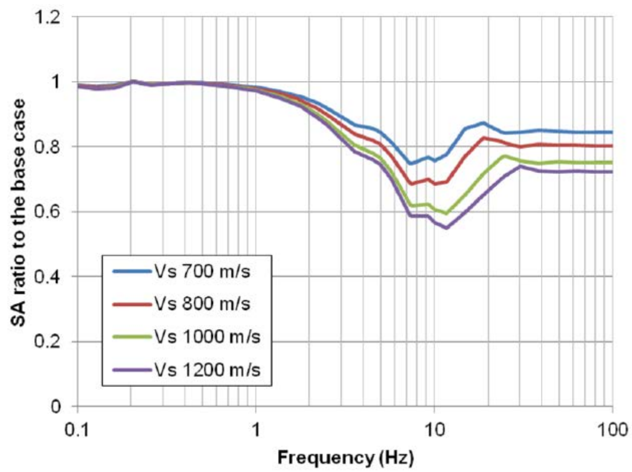

Figure 5. Spectral Acceleration Ratio to the Base Case for the improved zone with 20 m thickness
(Base case is prior to DSM improvement)

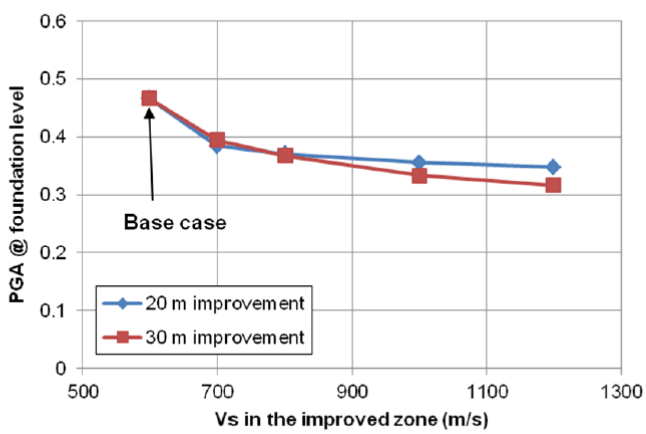

Figure 6. Foundation level PGA versus Vs in the improved zone
(Base case is prior to DSM improvement)

It is noted that the effect of reduction in the foundation level seismic demand is insensitive to the assumptions in the amplitude of the input motions or the triggered nonlinearity level in the soils, i.e. both sets of input motions yield very similar reduction effects as shown in Figure 5 and Figure 6. The mechanism of the reduction is that the improved surface zone serves as an energy barrier which can reflect the seismic waves over a broadband of frequency range. It is argued that this reduction in seismic demand is applicable for other ground motion improvement techniques which can form a stiffer surface layer with sufficient lateral extent. Readers are referred to Li at al. (2019) for a more detailed presentation of the SRA sensitivity studies for the DSM improved system.

CLOSING REMARKS

While there is an increasing trend for the implementation of various ground improvement techniques for NPP foundations in soft soil sites, the challenges are to go through the lengthy process of regulatory review and approval. Some general considerations towards better regulatory assurance are presented in this paper, along with some case studies on the technical aspects in the process. It is concluded that proper executions of the conceptual plan development, comprehensive engineering analysis, and field monitoring and testing programs, supplemented with appropriate quality procedures across all the processes, are crucial for fulfilling nuclear quality assurance requirements and achievement of regulatory approval.

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