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Soil improvement using reinforced granular fills on earthquake zones: Three case studies

Asli Yalcın Dayıoglu
Istanbul Technical University

Mete İncecik
Istanbul Technical University

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ABSTRACT: In this study, three soil improvement applications performed on earthquake zones in Turkey using geosynthetic-reinforced granular fill are examined. The first two applications are both designed with industrial storage purposes and they are located in Marmara Region, where a strong earthquake with a magnitude of Mw=7.4 took place in 1999 and a large number of buildings were either damaged or collapsed. The first one is construction of two steel fuel tanks, where the soil profile consists of recent sea sediments and location of ground water table is close to the surface. The second case is designed as a sphere-formed steel ammoniac tank resting on the same soil profile as mentioned above. The third project was built in Aegean Region, which is also located on an extremely active fault zone in the western part of Turkey. Project consists of construction of six wind turbines and on the construction site alluvial soil deposits compose the existing soil profile. In addition to reinforced fill application, stone columns are constructed in this project to provide the required settlement criteria. In this context, detailed study on the design process and performance of soil improvement after the construction is executed.

1 INTRODUCTION

Turkey is a tectonically active country that experiences frequent destructive earthquakes. Due to this fact, many precautions have to be taken to provide adequate safety to meet both the bearing capacity and the settlement criteria under dynamic forces when starting construction projects on earthquake zones, especially if poor soil conditions exist.

To improve strength and consolidation parameters of weak soils, many improvement methods are nowadays available. Soil improvement using geosynthetics reinforced granular fills (GRSF) on foundations is based on the principle that the GRSF acts as a stiff soil layer overlying a softer soil deposit. The stiff GRSF acts like a beam and distributes the stresses from the footing over a larger area. This reduces the unit stress on the foundation soil from the footing. Figure 1 may be used to estimate the vertical stress exerted on the foundation soil when using a GRSF. This method has considerable potential as a cost-effective alternative to over-excavation and replacement, consolidation, densification and chemical stabilization.

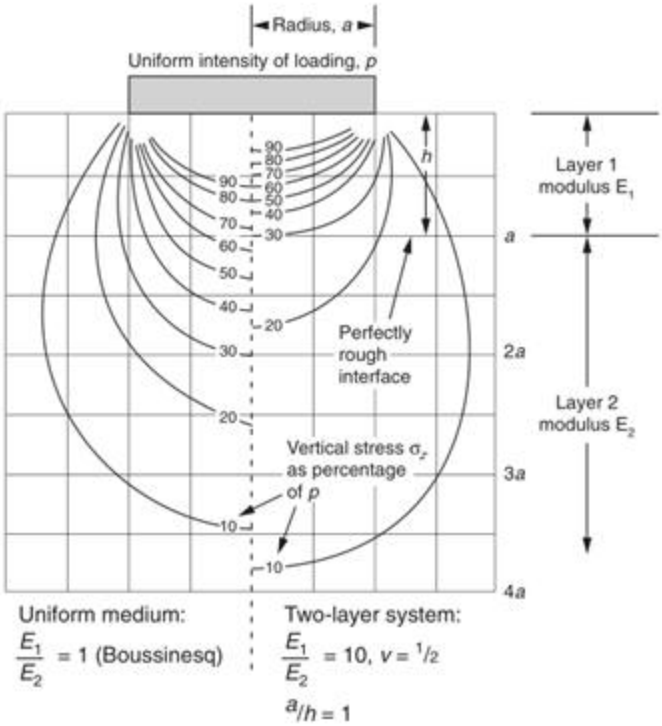

Figure 1. Westergaard stress distribution for a two-layer system (Munfakh et al., 2001).

2 CYLINDRICAL STEEL FUEL-OIL TANKS FOUNDATION PROJECT

Two steel cylindrical tanks were planned to be constructed in 1997 in the field of a synthetic fibre factory that was built many years ago near the seaside of Yalova city, Marmara Region and many chemistry plants having steel or reinforced concrete structural system are located in the field of the factory. In order to determine the soil profile of the site, several borings to the depth of about 20.0 m were drilled and laboratory tests were performed on the specimens taken from the site. Regarding the in-situ tests performed, soil profile is determined as follows: Beneath the fill layer with an approximate thickness of 2.5 m, medium to stiff, low plasticity silty sandy clay layer is located between -2.5 and -5.5 m. Following the clay layer, a medium dense-dense silty gravelly sand layer from -5.5 m to -11.5 m and stiff clay layer from -11.5 m to the end of the borehole are encountered. Groundwater elevations are measured at -1.5 m from the surface (Incecik, 1997). Soil layers composing the soil profile on site and results of the Standard Penetration Tests performed on site are given in Figure 2.

The diameter and the height of the tanks are 42 m and 12 m, respectively. Each tank weighs 17,000 tons when completely filled and is able to store a fuel of 16,000 m^3. The top of the tanks were covered by aluminium ceiling. During the foundation design, it is found out that the reinforced concrete shallow foundations of existing tanks constructed on the existing subsoil profile at the site do not have enough bearing capacity (Incecik and Iyisan, 2004). Hence, excavation of the existing weak subsoil, i.e. existing uncontrolled fill at the top and the underlying clay layer, to a depth of 5.5 m and placement of woven polypropylene geotextiles with a tensile strength of 40 kN/m on compacted granular fill material in every 50 cm was suggested to improve the bearing capacity of foundation soil (Figures 3 and 4). Granular fill layers were compacted with vibrated compactor of 35 tons and according to the results of the in-situ density tests 100% of the modified proctor maximum unit weight was obtained. The tanks were seated on an asphalt cover with a thickness of 6 cm.

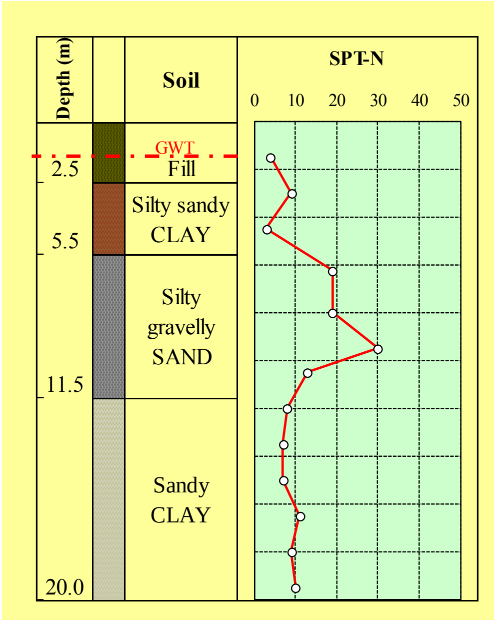

Figure 2. Soil profile based on the subsurface borings.

The use of geotextile reinforced earth increased the safety factor against failure and decreased total and differential settlements. On the planned foundation system, ultimate total settlement and differential settlement of the tanks were calculated as 20 cm and 10 cm, respectively whereas these values were estimated to rise to 35 cm and 17 cm, when the soil improvement was not taken into account.

Figure 3. Compaction of the fill material by a vibrated compactor.

After the construction of the tanks, the settlement of the soil was periodically observed at the site for one year, and the observed settlements at the edge of the tanks was in the range of 8-12 cm. A large percentage of the observed settlements were recorded in the first month during the test loading of the tanks.

Figure 4. Production of the reinforced granular fill layer.

An earthquake with a magnitude of Mw=7.4 took place at East Marmara Region on August 17, 1999, shortly after the cylindrical tanks were put into operation. The epicenter of the earthquake was located near Kocaeli, on the North Anatolian Fault Zone (NAFZ), which had produced many destructive strong ground motions in the past. Yalova was located at about 50 km west of the epicenter.

Although the new constructed tanks were half loaded during the earthquake, earthquake damage was not observed at the tanks which were located at about 50 km west of the epicenter and 1-2 km from the North Anatolian Fault Zone.

Earthquake caused some rupture on the ground surface and visible damage on structures near the tanks. In addition, failure of an adjacent pier due to loss of some of its piles has been observed. After the earthquake, site investigations indicated that additional settlement at the tanks did not occur, and neither cracks nor deformations on the steel body of the tanks were seen. The tanks are still being used after the earthquake.

3 SPHERICAL AMMONIAC TANK FOUNDATION PROJECT

After the cylindrical tanks mentioned in the previous section have survived the Marmara earthquake with a magnitude of Mw=7.4 in 1999 without any damage, construction of a spherical ammoniac tank in a very close location to the cylindrical tanks was planned in 2004. According to the results of the insitu tests, soil profile on site consisted of recent soil sediments as in previous project and groundwater table was located very close to the surface. Regarding the in-situ tests performed, soil profile was determined as follows: Beneath the fill layer with an approximate thickness of 2.0 m, a 3.5 to 4.0 m thick medium to stiff sandy silty clay layer is located. Following the clay layer, a medium dense-dense gravelly sand layer with a thickness of 6.0 to 6.5 m is observed and it’s underlain by a stiff sandy clay layer to -30.0 m elevation, i.e. the end of the borehole. Groundwater elevations are measured at -1.5 m from the surface.

The ammoniac tank, produced as a steel sphere with a diameter of 17.5 m is planned to be constructed on 11 pipe supports that shall be placed circular on a concrete ring foundation. Due to the project requirements the closest point of the sphere to the foundation shall be 1.5 m above the soil surface. During the loading tests, the static weight of the tank is determined as 35,000 kN when fully loaded.

Considering the existing soil conditions, superstructural loads and seismicity of the site and due to the fact that the required limit settlement values are quite low, foundation system of the subject structure is determined as a pile foundation by the project company. However, construction of a ring-formed reinforced concrete shallow foundation on a geotextile reinforced granular fill is decided to be a more appropriate solution since the soil layers with a high bearing capacity are located very deep from the surface. In this context, excavation of the existing weak soil layer to a depth of 5.50 m and replacement with a reinforced soil consisting of a granular fill layer with geotextile reinforcement of 40 kN/m tensile strength in every 50 cm is recommended as soil improvement method (Figures 5 and 6).

Figure 5. Excavation of existing weak subsoil (The adjacent cylindrical tanks are located in the background).

(a)

(b)

Figure 6. a) Layering of granular fill, b) Placement of the geotextile layers.

The ring formed reinforced concrete ring foundation that shall be placed upon reinforced fill layer, has an inner diameter of 11.5 m, an outer diameter of 25.5 m and height of 1.2 m. By means of this rigid foundation system total and differential settlements are planned to be minimized. Different construction phases of the foundation are shown in Figures 7 and 8. According to the results of loading tests performed after the construction of the foundation and assembly of the steel tank are completed, maximum value of total settlements are determined as approximately 8 cm. Differential settlement between two adjacent pipe supports is taken as the critical settlement value and is measured as 0.3 cm which is below the limiting differential settlement.

Figure 7. Installation of the reinforcement of ring foundation.

Figure 8. The sphere tank short before the final assembly.

4 WIND TURBINE FOUNDATION PROJECT

In 2009, a wind turbine project was planned to be constructed in Northern Aegean Region. In order to design a proper foundation system, in-situ tests were performed on construction site. In this section, recommended soil improvement method with geosynthetic reinforced granular fill and stone columns are examined.

Soil improvement with stone columns is recently a widely used method in order to improve strength and consolidation properties of soft and loose soils domestic and abroad. As a result of stone column application, bearing capacity of the weak subsoil shall be increased depending on the estimated project loads, site and soil conditions. Besides, required time for consolidation shall be reduced and potential of liquefaction or loss of strength is eliminated.

Soil improvement using stone columns is preferred both in cohesive and cohesionless soils. In the cohesionless soils, the main purpose is to compact the surrounding soil on columns and to transfer a certain amount of loads to lower soil layers that are stronger. In addition, amount of settlement in cohesive soils may be reduced by 50-60% via preloading and bearing capacity may be increased to higher levels. Stone columns application leads to successful results in liquefiable soils. It is generally recommended that stone columns under vertical loads should be socketed in a strong soil layer. Regarding method of installation, beside use of vibroflotation equipments or classical drilling options, compaction of soil through pipe driving and creation of the columns are also practicable.

On the subject site, construction of many wind turbines is planned while the first stage consists of six turbines. According to the results of in-situ tests performed and laboratory tests executed on disturbed specimens taken from boreholes, it is determined that soil profile under T1 turbine consists of vegetative cover and rock units, whereas the area where the other five turbines shall be constructed is located on an alluvial subsoil. Beneath the subsurface fill layer, these sediments are placed as sandy silty clay, clayey sand and sand layers in various thicknesses and elevations consecutively until the end of the borehole. In addition, laboratory tests showed that the clay layer located between the surface and the -10.0 m elevation contains a serious amount of organic material. Elevation of groundwater table was measured between -13.0 and -18.0 m, however, it is found out that on heavy rainfalls the GWT elevation closes to the surface.

Towers of the turbines shall be produced from steel and they are designed with a height of 85 m. Superstructural loads for each tower that shall be transferred to the subsoil is estimated to be approximately 3,500 kN. Reinforced concrete foundation of each tower shall be octagonal-shaped with a diameter of 20 m. In order to provide a safe and effective performance limit total and differential settlement values are required to be 2.0 cm and 3 mm/1000 mm, respectively.

Considering the amount of transferred superstructural loads and required limit total and differential settlement values, no soil improvement for the turbine T1 is estimated, since the soil profile in this area consists of weathered rock. Nevertheless, due to the fact that foundations of the other five turbines shall be placed on alluvial soil layers which can not meet neither the bearing capacity nor the settlement criteria, stone column application as an economical and fast soil improvement method is suggested (Incecik, 2009). Furthermore, the construction site is located on 1st degree earthquake zone and the subsoil of the area where the turbines 2 to 6 shall be placed falls within Z3 soil group according to “Specifications for structures to be built in the disaster areas”. By means of the mentioned soil improvement method, liquefaction potential shall be also reduced significantly.

Considering the above conditions, foundations of the T2-T6 turbines are decided to be placed at -2.00 m from the surface. A 3 m thick subsoil layer beneath this elevation shall be also excavated away and this area shall be filled with appropriate granular material. Placement of geotextile reinforcement, with a tensile strength of 40 kN/m in the fill layer for every 50 cm is accepted. Size range of the fill material is recommended to be 2-100 mm.

In order to perform production of stone columns on site with a spacing of 2.80 m in both directions and length of 12 m, pile pipe is recommended to be driven with vibrex pile equipments and to be filled with crushed stone with a grain size between 10-150 mm, where no significant settlement due to vibration in cohesionless or slightly cohesive layers during drilling is expected. Layout plan of the application is shown in Figure 9, whereas the cross-section is given in Figure 10.

Stone columns are estimated both to act as a structural elements in vertical direction and to increase strength and other engineering properties of the existing soil layer by providing fast drainage of the present groundwater within the soil. Allowable bearing capacity of the soil after the improvement has been performed successfully is predicted to be taken as q_{all}= 200 kPa under the turbines T2-T6.

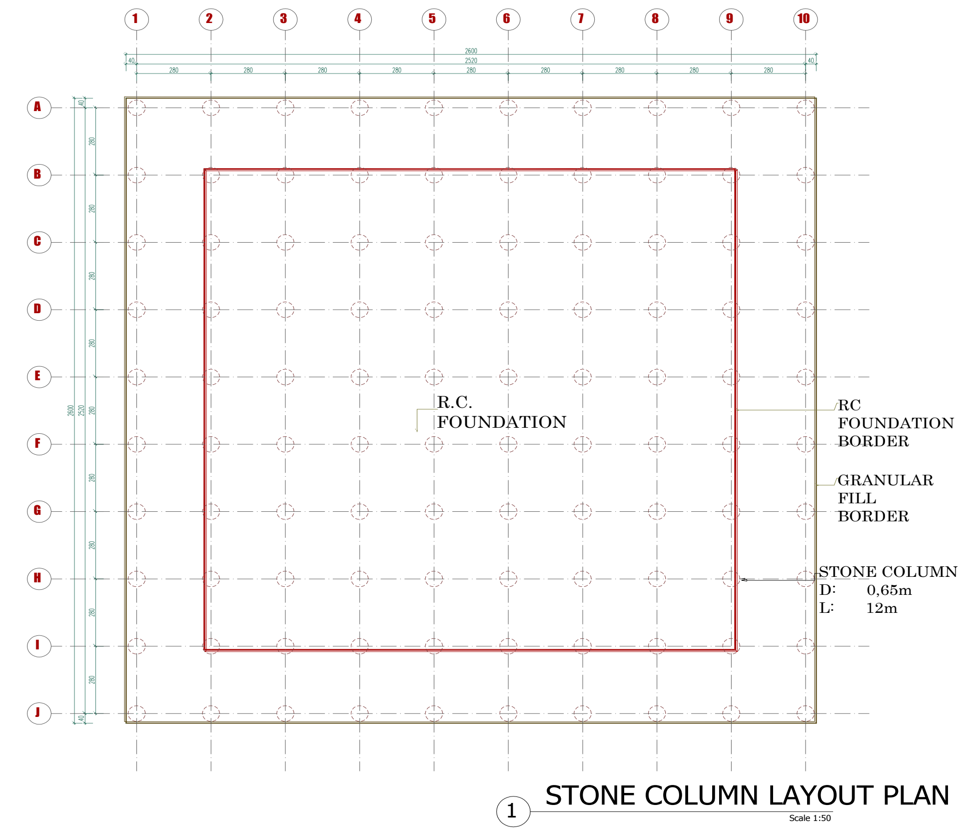

Figure 9. Stone column layout plan for a turbine foundation.

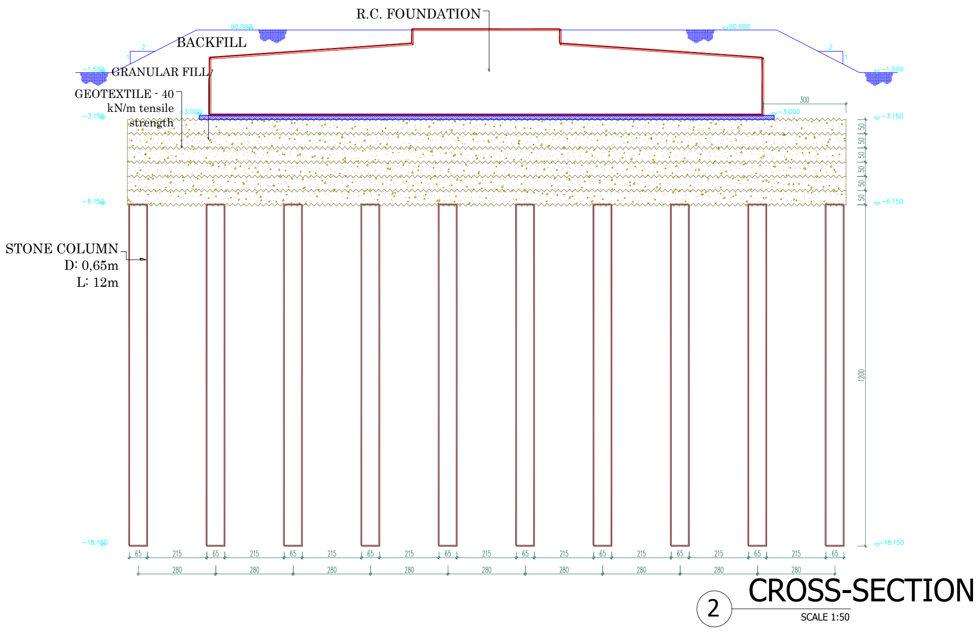

Figure 10. Cross-section of stone column application on a turbine.

5 CONCLUSIONS

In this paper, design process of three structures that are constructed in 1st degree earthquake zone in Turkey and located on weak soil profiles regarding bearing capacity and settlement criteria as well as their performance during and after construction is examined. Due to the fact that the characteristic soil profile in all three construction site consists of sea sediments, pile foundation was not preferred. Instead, geotextile reinforced granular fill layer is placed beneath foundation systems which is economical and is a fast soil improvement method. Shortly after the first project was completed and put into operation, Marmara earthquake with a magnitude of 7.4 took place on August 17, 1999, however, no significant damage was observed by the tanks due to earthquake. Although the other two projects have not survived an earthquake after they have been put into operation, the measured settlement values have remained within the desired and estimated range so far. Thus, in case pile foundations can not be constructed in earthquake zones due to weak soils located on construction site that extend to a certain depth, soil improvement applications using geotextile reinforced granular fill layers are observed to exhibit quite successful performances.

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