The application of Geotextile Encased Columns (GEC) beneath water in a highly seismic area for the construction of an up to 22 m high embankment

E. Guler, George Mason University, Fairfax, U.S.A.
O. Detert, HUESKER Synthetic GmbH, Gescher, Germany

ABSTRACT
A geotechnical construction project in the central Turkish region of Kirsehir is described. It comprises the construction of a highway link incorporating the re-establishment of a connection between two towns interrupted due to the reservoir of the Hirfanli dam. For this purpose an embankment of 430 m length was constructed across an existing reservoir. This embankment had to be formed over deep, soft deposits of clay, silts and sand lenses and is supported on a Geotextile Encased Columns (GECs). The unique aspect of the project is that this is the first time anywhere in the world that GEC construction has been carried out from and installed below water.

The construction of this section of highway has been a historically challenging one due to the difficulty in crossing the section of open water as well as the potential earthquake hazard. Previous designs had proposed either a bridge structure on pile foundations or an end tipped rock fill embankment. The design of pile foundations was not possible because the extremely soft alluvial deposit could not provide the necessary lateral stability. As a result both proposals were found to be either too expensive or technically inadequate for the constraints of the site.

The project environment, construction problems and typical solutions are described, as well as design and calculation philosophy and methods, with typical cross-sections and photographs illustrating the specific points. In respect to the hazard of earthquake loading, the behavior of GEC's under seismic loading will be discussed and research results presented.

RESUMEN
Se describe un proyecto de construcción geotécnica en la región central turca de Kirsehir. Consiste en la construcción de un enlace carretero que incorpora el restablecimiento de una conexión entre dos poblaciones interrumpidas por el embalse de la presa de Hirfanli. Para ello se construyó un terraplén de 430 m de longitud a lo largo de un embalse existente. Este terraplén tuvo que ser formado sobre depósitos profundos y blandos de arcilla, limos y lentes de arena y está soportado sobre una columna revestida de geotextil (GECs). El aspecto único del proyecto es que esta es la primera vez en el mundo que la construcción de GEC se ha llevado a cabo desde e instalado bajo el agua.

La construcción de este tramo de autopista ha sido un reto histórico debido a la dificultad de cruzar el tramo de aguas abiertas, así como al riesgo potencial de terremotos. Los diseños anteriores habían propuesto una estructura de puente sobre cimientos de pilotes o un terraplén de relleno de roca con extremo inclinado. El diseño de los cimientos de pilotes no fue posible porque el depósito aluvial extremadamente blando no podía proporcionar la estabilidad lateral necesaria. Como resultado, se consideró que ambas propuestas eran demasiado costosas o técnicamente inadecuadas para las limitaciones del lugar.

Se describen el entorno del proyecto, los problemas de construcción y las soluciones típicas, así como la filosofía y los métodos de diseño y cálculo, con secciones transversales típicas y fotografías que ilustran los puntos específicos. Con respecto al peligro de las cargas sísmicas, se discutirá el comportamiento de las cargas sísmicas de GEC y se presentarán los resultados de la investigación.

1. INTRODUCTION

In the last decades, soil improvement below embankments have become a common practice. One of the most preferred soil improvement schemes is the installation of stone columns. Installing ordinary stone columns in soft clayey soils is a cost and time efficient soil improvement technique. However it is known that their stability is predominantly based on the available lateral support that is provided by the surrounding soil (Hughes and Withers, 1974; Hughes et al., 1975). When implemented in extremely soft soils (s_u < 15 kPa), the columns usually fail in bulging due to lack of lateral support that the weak soil can offer. One way to overcome bulging failure is to encase the granular column materials with a reinforcing geosynthetic and thereby forming a geosynthetic encased column (GEC) which increases column performance by providing lateral confinement (Raithel and Kempfert 2000; Alexiew et al. 2005).

Another issue regarding the use of ordinary stone columns in soft clays are their behavior during earthquake loading conditions. Because there is no confinement around the stone columns, it is very difficult for the stone columns in soft soil to keep their integrity under seismic loading conditions. To shed light to the behavior of ordinary stone columns versus GECs under earthquake loading conditions Guler et al. (2014) conducted a finite element analysis using the Finite Element software DIANA to model GECs under the action of seismic input motions. They determined that encasing the stone columns significantly reduces the seismically induced settlements. And we have to consider, that due to the nature of the finite element analysis, the soil improvement column in any case remains intact and the fact that stones can be lost into the soft clay cannot be modeled. So this improvement observed in the finite element model was only due to the increased stiffness of GEC columns. Also Hasan and Samadhiya (2017) ran small scale laboratory tests and 3D finite elements analysis utilizing PLAXIS. The results indicated that ultimate load intensity and stiffness of the soft clay increased due to geosynthetic encasement of granular columns.

Guler and Cengiz (2018a,b) conducted a series of shaking table tests. They used both a rigid box and as well developed a new laminar box (Cengiz et al. 2019). As a result of the experimental work in which they modeled both ordinary stone columns and GECs in soft clay, they found that the vertical load carrying capacity of ordinary stone columns under seismic loads reduced by as much as 55%. This means that the static bearing capacity measured under static loads reduced to only 45% of the static capacity after the ordinary soil column, soft clay composite experienced earthquake loads. However for the same soil the vertical load carrying capacity of the GECs did not change significantly after the application of the same seismic excitation. So they conclude that ordinary stone columns may be good for static loading conditions, but if seismic loads are expected, they should not be used as a soil improvement scheme. The best solution will be replacing the ordinary stone columns by Geosynthetic Encased Columns, because they keep providing the bearing capacity without failure also under seismic loading conditions.

Guler and Cengiz (2018a,b) also measured the shear strain modulus of the soft clay they used in their shaking table models and looked into how the installation of the ordinary stone columns or GEC's influenced the overall shear modulus of the soil, which is one of the most important parameters when it comes to estimate the behavior of the foundation soil under earthquake loading conditions. The researchers measured the small strain shear modulus of the soft clay bed as 115 kN/m^2. The installation ordinary stone columns was able to increase the small strain shear modulus to 259 kN/m^2. However the GEC which had a geotextile of a stiffness of 1000 kN/m was able to increase the small strain shear modulus to 978 kN/m^2. This means that the overall shear modulus could be increased almost tenfold with the help of the geosynthetic encapsulation. Of course one has to remember that this tenfold increase in shear modulus has been achieved for a common area ratio and encasement stiffness. This improvement can be further enhanced by increasing the area ratio of the GECs and the stiffness of the encapsulation. This fact was validated by the authors, who concluded that the shear modulus of the soil + GEC composite increased with increasing encasement stiffness.

The researchers also observed that the additional strain demands brought about by the dynamic loads are distributed to the entire height of the column. Additional reinforcement strains as high as 3% is observed at a depth of about 8D from column head plane during earthquake loading conditions. This indicated that in the absence of a reinforcement material, like in the ordinary stone columns, bulging failure can occur not only at the top of the column but even at greater depths due to seismic loads. This fact also clearly shows that a geosynthetic enforcement is necessary to prevent the failure of ordinary stone columns under seismic loading conditions.

It is a well-known fact that the improved columns below the center of an embankment are exposed to mainly vertical loads. Hence the bearing capacity required from the columns is important. However towards the edges of the embankment the failure mode becomes a rotational edge failure. In those areas the soil improvement columns will undergo a risk of shear failure. To understand the contribution of the encasement geotextile Guler and Cengiz (2019) built a physical unit cell, like the one used in the analysis of the vertical capacity. The area replacement value was chosen on the rather low side as

6%. In general the minimum are ratio is 10%. However this time instead of applying a vertical load they sheared this unit cell. The residual static shear resistance derived from the pure clay model was increased by 90% with installation of GECs. Based on the measured values they determined that under static shear conditions the effective angle of internal friction of the clay soil was in the order of 23°. Upon installation of GECs with a reinforcement stiffness corresponding to the prototype equivalents of 1000 kN/m, the overall internal angle friction of the column-unit cell soil composites increased to an order of 35º. As can be seen, this is a very significant increase. The authors did not shear the composite unit cell only statically, but also applied cyclic stresses. In the later stages of cyclic shearing when shear displacement increased, the shear resistances of GEC installed unit cells were significantly greater than unenhanced and ordinary stone column installed unit cells. The authors indicate that the column acts as a dowel resisting the relative movement of the soil bodies on either side of the shear plane which causes buildup of soil pressure around the periphery of the column.

2. CASE STUDY

2.1 Background Information

The project site is located near the town of Sariyashi, in central Anatolia, within the region of Konya. The site of the embankment in question is at the location where the new 8.5 km link road crosses a manmade reservoir which services a hydroelectric power generation facility some 10 km downstream from the site.

The specific location of this road route is based around it tying into the regional north south highway which runs to the north east of the site, through the city of Kirsehir. This major highway has recently been upgraded, forming the main arterial connection between the Turkish cities of Ankara and Kayseri. The road under construction, which is the subject of this paper, forms a strategic regional link road to serve the rural communities to the south and west, as well as providing a link to the E90 highway which runs from Ankara to the southern city of Adana.

The Hirfanli dam located on the Kizilirmak river started accumulating water in 1959. The maximum capacity of the reservoir is 5,740,000,000 m3. The reservoir forms the natural boundary along which the boundaries for a number of the municipal districts namely, Kirsehir, Aksaray and Ankara. This has meant that the multi-jurisdiction of the embankment crossing site has posed political dilemmas as well as technical ones. In the recent past the section of the link road from the direction of Kirsehir to the north side of the site has been completed by Kirsehir district. This section of the link road has been completed previous to the dam reservoir accumulating water. Therefore a concrete multi span viaduct reaching the island located half way across the reservoir in Aksaray district was constructed easily (Figure 1).

Figure 1. Location of Proposed Embankment.

The southern part of the reservoir crossing from the central island to the south side of the reservoir had not been so easy to complete due to much more onerous ground conditions.

The embankment across the reservoir is approximately 550 m in length of which 400 m is across open water the remaining lengths being within the transition zones based on either shoreline.

2.2 Geotechnical Conditions

The recent depositional history of the sediments within the reservoir, which date within the last 40 years since the construction of the dam, means that they are very weak in nature, especially where in this backwater location there is no regular flushing currents to keep such sediments suspended. Typically they comprise sequences of inter-bedded soft clays and silts overlying sands and gravels formed of decomposed granite. Some sand lenses are also present within these layers (Figure 2).

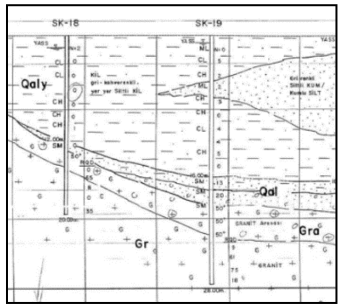

Figure 2. Extract from site investigation.

The depth of these sediments varies from 1 to 2 m at the shorelines to approximately 15 m deep in the center of the reservoir. Typically the SPT N values ranged from 0 to 5.

There were 5 boreholes which were conducted to get information on the soil properties. Based on these boreholes, the depth of these extremely soft sediments varies from 1 to 2 m at the shorelines to approximately 17 m deep in the center of the reservoir. The idealized cross section at the location of the deepest sediment accumulation can be summarized as follows:

- 0 – 17.8 m: Mud (extremely soft clay/silt/silty sand layers) SPT N = 1
- 17.80 – 29.50 m: Geological alluvium: (gravelly sand) SPT N = 18 – 22
- 29.5 – 32 m: Decomposed granite
- > 32 m: Granite

2.3 Alternative Solutions considered

Notwithstanding that these sediments were under a water depth of approximately 6 to 7 m, such was the weakness of these sediments that the options of installing either cast in situ or pre-cast driven piles was not an option given the lack of lateral support available. The geological profile meant that there were really only two options originally considered for the crossing of this area, namely a full span bridge some 400 m in length, so likely a suspension bridge or cable stay type, or else a rock fill embankment.

However with the costs of the former bridge option proving prohibitive and the technical concerns about the viability and stability of an end tipped rock fill embankment meant that the development of this link road remained in abeyance for some years.

For the construction of the embankment further measures are required to ensure sufficient stability due to the very soft sediments. Due to the thick sediment layer and the high seismic activity the installation of concrete piles was not feasible. Although the use of ordinary stone columns was due to the very soft sediments and seismic activity not feasible. The soft sediments are not capable to give sufficient lateral support to the stone columns, especially not under earthquake events.

It was primarily due to the awareness and foresight of the Turkish Federal highways Authority (TCK) which lead to serious consideration of the GEC system to be utilized. Representatives of the Research and Development Department of the TCK had visited a GEC site in Bremen northern Germany in 2010 and had seen the potential for the GEC foundation system for this particular project. Technical discussion began between TCK, Geoduvar and Huesker regarding the viability of using the GEC system for the support system for a rock and earth fill embankment across the reservoir.

3. DESCRIPTION OF GEC SYSTEM

The geotextile-encased column (GEC) foundation system was specially developed for earthwork structures built on weak subsoil. It comprises uniformly arranged columns, made from non-cohesive material enclosed in a geosynthetic sleeve, which transmit the structural loads to the bearing stratum. The overall loads and stress concentrations above the column heads induce outwardly directed radial horizontal stresses in the columns. The particularity of the GEC system is that these stresses are counteracted not only by the inwardly acting pressure of the soft soil, but also – most importantly – by the radial resistance of the stiff geotextile casing. The substantial circumferential tensile forces generated in the casing provide radial support to the columns and ultimately safeguard the equilibrium of the system, thereby allowing its use even in very soft soils, peats and sludges which offer negligible radial support, $s_u < 2$ kN/m².

The mobilization of ring-forces requires some radial extension of the encasement (usually in the range of 2 to 5 % strain), leading to some radial "spreading" deformation in the granular columns, and resulting, consequently, in vertical settlement at the top of column. The GEC system cannot therefore be completely settlement free. However, most of the settlement occurs during the construction stage and can be compensated by some increase of embankment height. Finally a state of equilibrium is reached, ensured by the strength and stiffness of sand or gravel, soft soil radial counter-pressure and the confining ring-force in the encasement geotextile.

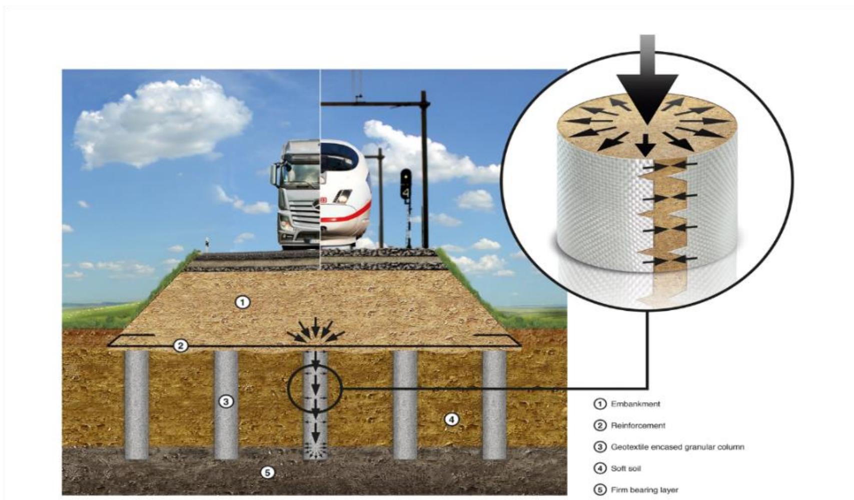

Figure 3. GEC System and its load transfer.

The GEC are arranged usually in a triangular grid pattern. Typical diameter of the columns is 800 mm and axial spacing of the columns is typically 1.7 m to 2.4 m, hence the resulting area of treatment ranges from 10 to 20%.

The arrangement of geotextile-encased columns produces a ductile bearing system that is immune to buckling under the incident column loads. The use of GEC considerably reduces both absolute and differential settlement, while enhancing structural stability both during construction and after completion. As the columns also act as filtration-stable mega drains, they speed up the settlement and consolidation process. Later settlement, e.g. caused by traffic loads, is low and can, if necessary, be largely offset by means of temporary cover fill.

4. DESIGN

4.1 Bearing Capacity Design

The design for the embankment on geotextile encased columns has been conducted in two steps. The first step is considered as vertical design of the columns to estimate the required tensile strength of the geotextile encasement. The design approach presented in the EBGEO (2010) was used, which is based on the research work of Dr. Raithel (1999).

4.1.1 Material Properties

For the design of the GEC's a polyester encasement was chosen. The characteristic ultimate strength of the geosynthetic was 500 kN/m. The long term ring tensile stiffness was about 4500 kN/m. The GEC columns chosen had a diameter of 0.8 m and a triangular pattern of installation was chosen, which gives the best positive impact in regard to soil improvement.

The soft soil was represented in this analysis with the following parameters: Effective strength parameters: $\phi' = 20^o$, $c' = 3\ kN/m^2$, unit weight $\gamma = 16\ kN/m^3$, poison ratio $\nu = 0.4$, Oedometric modulus at 100 k/m^2 normal stress: $E_{oed.} = 750\ kN/m^2$. The infill of the GECs were represented with the following mechanical properties: $\phi' = 34^o$, $\gamma = 19\ kN/m^3$.

The design resulted in a triangular pattern with a center to center spacing of 1.70 m. This caused a design with an area replacement ratio of 20 % (Area covered by columns in relation to the whole area).The length of the GECs were chosen as variable. It was assumed that at every location the GECs will extend until the geological alluvium, namely the gravelly sand which has an SPT N = 18 – 22. This formation was considered strong enough to provide an end bearing to the GECs.

4.2 Edge Bearing

For the edge bearing analysis a basal reinforcement was necessary. The configuration seen in Figure 4 was adapted. The details are given in the following sections.

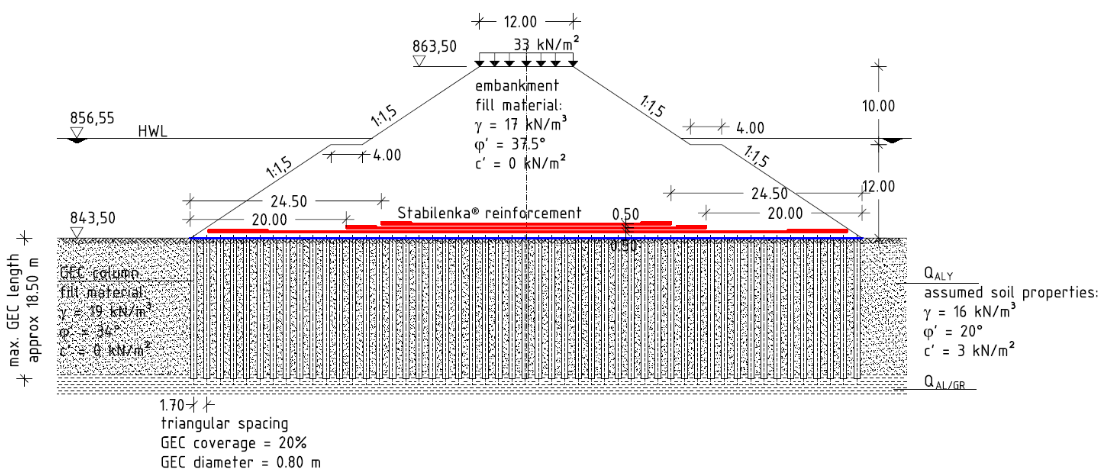

Figure 4. Cross Section of the embankment.

4.2.1 Stability Analysis for Static Condition

The second step is the horizontal design to check the global stability of the embankment. The design is done using the moment equilibrium method after Bishop. The contribution of the column encasement and the load concentration above the column is considered in the design by transferring the activated ring tensile strength into a so called equivalent cohesion (Raithel, 1999). Columns are represented in the software as 1 m wide vertical soil layers. Column diameter and column distance are assumed corresponding to area ratio a. Thus columns with area ratio a = 20% will be transformed to soil layers of a width b = 0.2 m and a column-to-column-distance of d = (1 - a/100) = 0.8 m. The basal reinforcement is necessary in order to increase the load redistribution, thus providing the equalization of settlements and increase of the overall global stability.

For the static design, the life of the reinforcement geosynthetics were taken as 120 years and hence creep properties were determined accordingly. A surcharge load of 33 kPa was chosen to represent the potential heavy traffic on the road. The result of the stability analysis can be seen in Figure 5.

Besides the bearing capacity, a settlement analysis was conducted and it was seen that settlements in the order of 1 m was estimated. This settlement was considered in the design, to accommodate that the road on top of the embankment does not fall below the maximum water level in the reservoir.

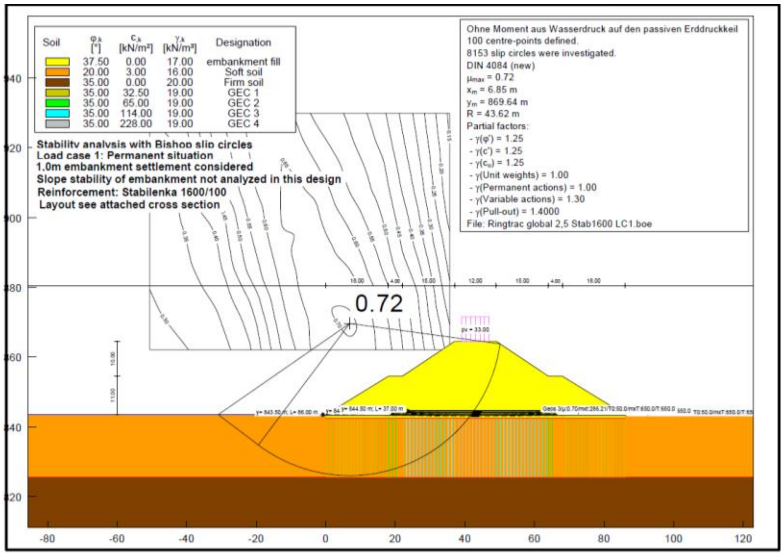

Figure 5. Slope Stability Analysis for static condition.

4.2.2 Stability Analysis for Earthquake Loading Condition

The area the project is considered as the first degree earthquake zone according to the Turkish Earthquake Map of the time. This means that the expected maximum earthquake acceleration is a_{max}=0.4g. Based on the earthquake design code of Turkey, which was valid at the time, the horizontal acceleration to be used in the pseudo-static analysis of embankments was chosen as k_h=0.16g. Due to the extreme boundary condition, especially for the earthquake situation, the design results in a required short term tensile strength for the basal reinforcement, made of Polyester, above the columns and at the bottom of the embankment, respectively, of 4800 kN/m. Those days the highest available strength for woven was limited to 1600 kN/m (nowadays strength of up to 3000 kN/m can be produced), so that three layers of this material was placed at the bottom of the embankment to secure the global stability. The installation and placement beneath the water level was done by divers.

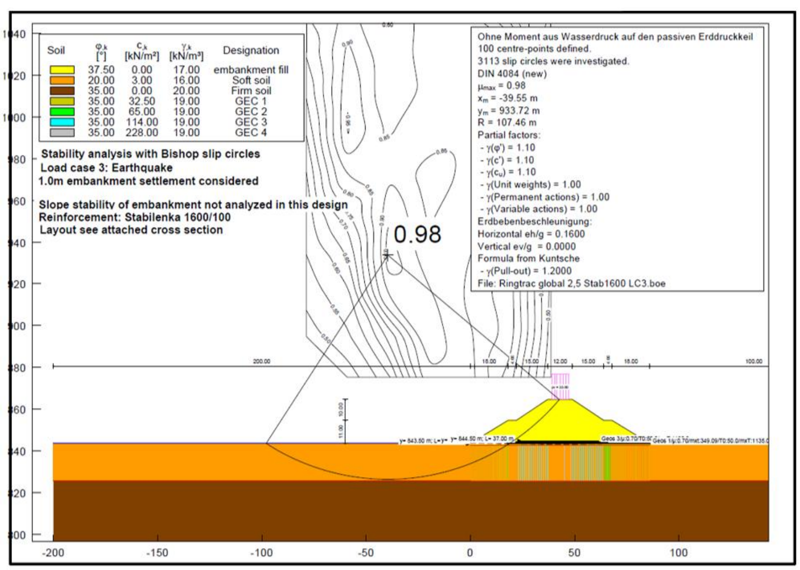

Figure 6. Pseudo static slope stability analysis for earthquake loading condition.

5. EXECUTION

The execution was quite special. This was the first site, where the GEC's have been installed below 7 to 10 m of water. The second issue was, that the installation equipment was not able to drive on top of the already installed columns, as it would be at an onshore installation sites.

Regarding the installation equipment two options have been analyzed. Option one included the use of a barge to accommodate the installation equipment, as it has been previously done at a construction site in Germany to enlarge the production facility of Airbus in Hamburg. The second option was to use a huge crane (Liebherr 1300) to allow the construction from land. This crane was capable to reach out with its cantilever to install columns 50 m in front of itself (Figure 7). The second option has been finally favored. The second issue was the installation below the water level of 7 to 10 m. To avoid wasting geotextile material a smart funnel has been developed by the construction company (Atlasyol) to allow an installation below the water at the level of the soft sediments. A special mechanism at the funnel allowed to adjust the free length in respect to water level above the sediment layers.

Figure 7. Installation equipment.

The installation sequence was as follows:

- GEC installation over a certain length in front of the crane
- Filling of the installed sleeve from a barge
- Installation of the three reinforcement layers by divers
- Placing embankment fill (granite “waste” from a nearby quarry)
- Walking the crane on the already installed embankment for further install of the GEC's

Due to the special set-up on this site the installation of the steel pipe for the GEC installation was not guided but free hanging. To assure the right position a GPS system has been used. Special attention had to be paid to the vertical installation of the columns. Figure 8 shows the finished embankment.

Figure 8. Finished embankment.

6. CONCLUSION

- An unique embankment design was conducted for an embankment in Turkey. The project was unique because it was constructed on an extremely soft clay deposit of 17 m thickness and approximately 7 m below the water table. The embankment height was in the order of 20 m.
- An additional difficulty was posed because the area was in a first degree earthquake zone. So the system had to remain stable not only under static loads, but also under seismic loading conditions.
- The GEC system has proven to be a very valid solution under extremely difficult boundary conditions such as very soft and thick soft bottom layers and seismic activities
- Literature data supports the efficiency and necessity of GECs for soil improvement under seismic loads. So it was a good engineering choice to use GECs in this project.
- The shaking table tests showed that for both vertical bearing capacity and edge failure GECs provide a good solution. Furthermore the sand infill in the shaking table did not liquefy, though extreme magnitude earthquake acceleration records have been applied to the models.

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