CHAPTER 17

Geotextile-Encased Columns

Case Studies over Twenty Years

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17.1 INTRODUCTION

The general scheme of the bearing system with geotextile-encased columns (GEC) is shown in Fig. 17.1(a). Due to the higher compression stiffness, the load concentration is at the top of the column, thus reducing the stresses and compression of the soft foundation soils. The vertical load on a GEC also generates horizontal radial normal stresses outwards and radial widening of the column. Consequently, this provokes a counterpressure from the surrounding soft soils and a confining resistance from the encasement (the latter is the key difference between these and "conventional" stone columns). The mobilized respectively allowable confining ring tensile force F (kN/m) in the encasement depends on its ring tensile stiffness (modulus J in kN/m) and ultimate tensile strength (UTS in kN/m), respectively design strength F_d (kN/m): the higher they are, the more intensive the confinement, the smaller the column widening and consequently its vertical deformation (settlement) thus reducing also the embankment settlement and the pressure on the soft soil between GECs.

Additional factors controlling the system's behavior are the choice of column fill (e.g., sand or gravel) and the percentage (area ratio) of columns, but the choice of encasement is a very powerful tool with the additional advantage of a made-in-plant controlled engineering component. The first design procedure was suggested by van Impe (1989), based on an ultimate limit state (ULS) approach without the possibility of a serviceability limit state (SLS)—say, settlements–analysis. The next step was a coupled SLS-ULS approach (Raithel, 1999; Raithel and Kempfert, 1999, 2000), considering the encasement ring modulus and the interaction between the column and the surrounding soft soils.

Embankments with Special Reference to Consolidation and Other Physical Methods
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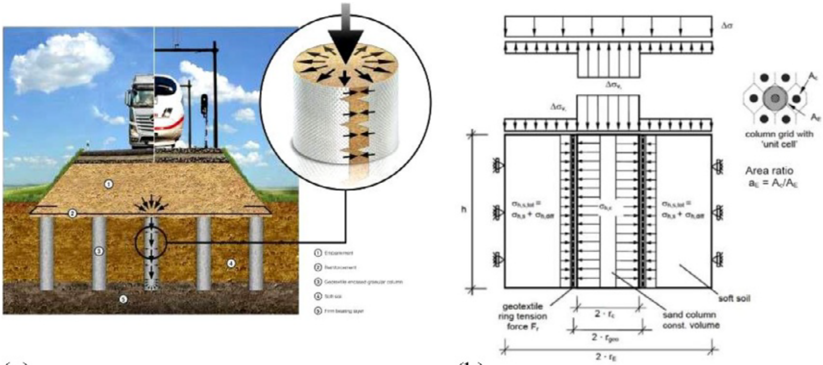

(a) (b)

Figure 17.1 (a) General scheme of GEC system. (b) Equilibrium of vertical and horizontal stresses in the so-called single cell as a base for design. *(Source: From Raithel (1999, 2000) and EBGEO (2011)).*

After further research and accumulation of experience and after marginal changes, this design procedure was included and codified in EBGEO (2011) and is the most used one today. The installation of GECs is now routine; they can be installed using a vibrator and an installation steel pipe with a temporarily closed tip with "flaps" (i.e., soft soil extraction from the pipe in the *displacement method*) or with a generally open tip (i.e., soft soil extraction from the pipe in the *replacement method*). Over the years a wide range of geotextile encasements was developed, thus allowing a system optimization. More details regarding design, installation methods, development, and so on, can be found in Alexiew et al. (2005), Alexiew et al. (2012), and Alexiew and Thomson (2013, 2014). For an overview of the method, see Tandel et al. (2012a,b).

17.2 RAILROAD EMBANKMENT AT WALTERSHOF, 1995

A heavy loaded railroad that runs to the harbor in Hamburg, Germany, required widening due to increasing traffic. The 5-m-high embankment is close to the Hamburg district of Waltershof. The subsoil consists of about 5–6 m of soft saturated clays and peat. The existing 5 m embankment had settled over the years by 1.2–1.5 m. There were two lots in the project. Due to logistic reasons Lot 1 had to be executed in a month and had a maximum time allowed for consolidation of four months. For Lot 2, a few months longer more was acceptable. Thus, it was decided to build the

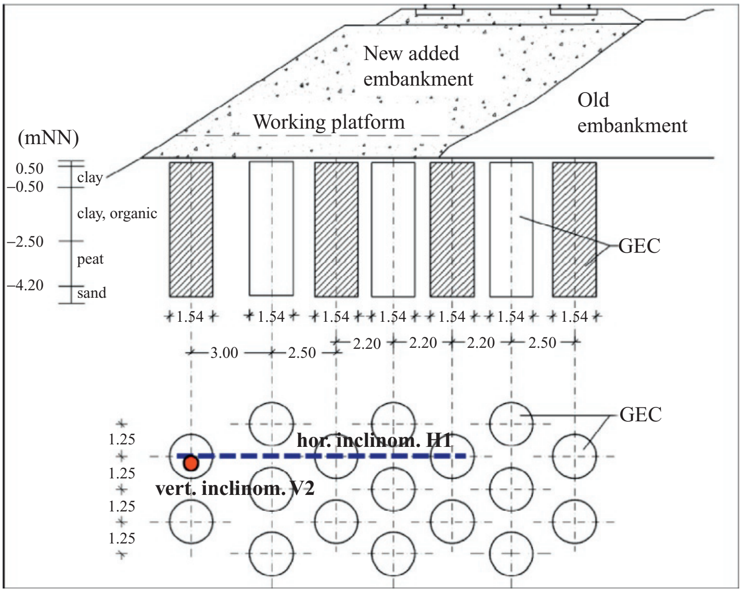

Figure 17.2 Typical scheme of the railroad embankment on GEC at Waltershof.

new embankment in Lot 1 on GECs and for Lot 2, to use a conventional solution with strip drains.

This was the first "real" project using the GEC solution. Until this time the technology had been tested on small testing areas only. At that time no SLS-related design procedure for GECs was available. A simplified design was performed based on conventional mineral columns, and an additional FEM design was performed to analyze the behavior of encasement, SLS, and so on. For more details, see Kempfert (1996) and Raithel (1999). In Fig. 17.2 a simplified scheme of the GEC-supported Lot 1 is depicted. The GECs had a diameter of 1.54 m; the area ratio was in the range of 20–30%. The encasement was produced from a geocomposite Comtrac 200/50 B 30 with an ultimate tensile strength (UTS) in the ring direction of 200 kN/m and an average ring tensile modulus of 1800 kN/m. The encasement had a vertical made-in-plant seam; at that time seamless encasements were still not available. The typical average soil parameters are given in Table 17.1.

First the working platform and the GECs outside the old embankment were constructed, then a part of the working platform at the old slope was elevated, and then an additional two rows of GECs were installed below the

Table 17.1 Average soil parameters of the subsoil layers at Waltershof

Soil layer	Position (mNN)	γ/γ_r (kN/m^3)	k (m/s)	E_s (MN/m^2)	φ' (°)	c' (kN/m^2)
Upper clay	$-0.5-0.5$	19/19	1×10^{-9}	2.6	29	8
Clay, organic	$-0.5-2.5$	13/13	$1{,}5 \times 10^{-8}$	0.8	25.5	16
Peat	$-2.5-4.2$	11/11	$1{,}4 \times 10^{-7}$	0.6	20.5	8.5
Sand	-4.2	19/20	3×10^{-5}	27	35	1

γ/γ_r: unit weight, natural/saturated; k: coefficient of permeability; E_s: oedometric compression modulus; φ': angle of internal friction, drained; c': cohesion, drained.

toe of the old existing embankment. After that, the new embankment was constructed step by step.

A measurement program was implemented comprising horizontal and vertical inclinometers and earth pressure cells (both over and between the GECs) and also piezometers in the soft soils. In Fig. 17.3(a) typical horizontal displacements of the first GEC at the toe of the new embankment are depicted, in a zone of generally intensive "spreading" of embankments on soft soils (see Fig. 17.2, V2); the final horizontal displacements of the toe amount to 9 cm, which is considered negligible under the given conditions. Figure 17.3(b) shows the settlements over a row of GECs across the embankment (see Fig. 17.2, H1); note the missing support by GEC at about

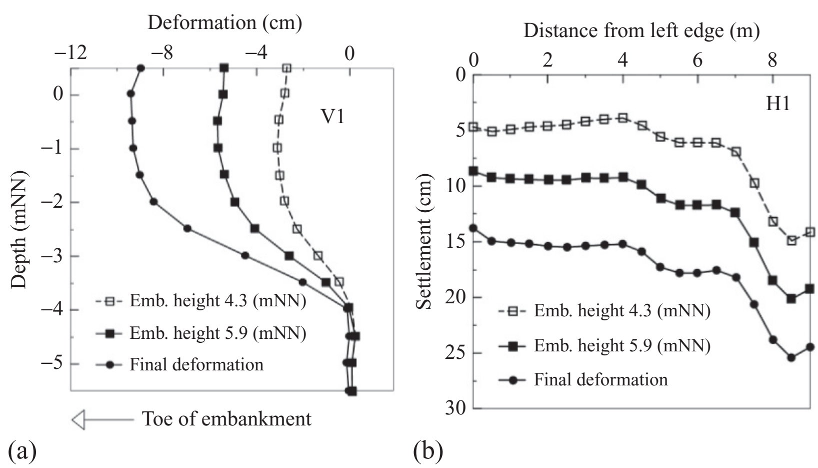

Figure 17.3 (a) Horizontal displacements of vertical inclinometer V1. (b) Settlements of horizontal inclinometer H1 (see Fig. 17.2).

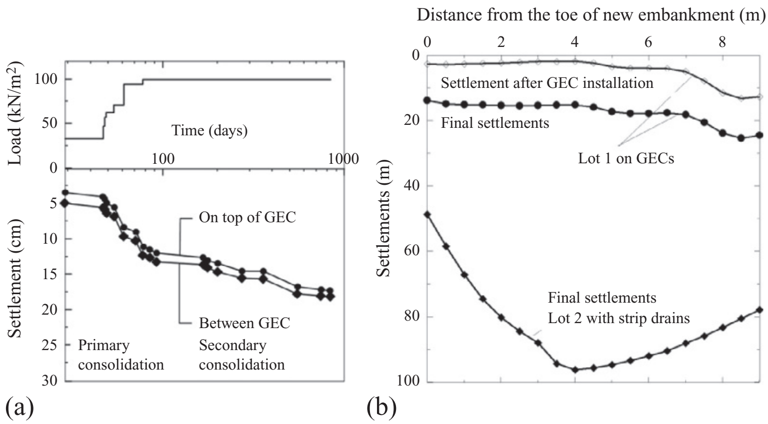

Figure 17.4 (a) Typical settlements in Lot 1 (on GECs). (b) Comparison of settlements between Lot 1 and Lot 2.

8 m from the toe (left edge) under full height embankment (see Fig. 17.2) resulting in significantly larger local settlement. In Fig. 17.3 "final deformation" means "after three years," as shown in Fig. 17.4.

Figure 17.4(a) provides time-dependent information on load development in Lot 1 and corresponding typical settlements on top of GEC and of the soft soil in-between over a period of about three years. Note that the postconstruction settlements (also under running heavy trains with iron ore) is only around 5 cm, and their increase tends to zero at the end of the period; this was the reason for the owner to stop further measurement sessions. Fig. 17.4(b) shows an interesting comparison of settlements across the new embankment between Lot 1 on GECs and the nonsupported Lot 2 with strip drains only; here "final settlements" means "at the end of the third year," as in Fig. 17.4(a). Note that geometry, subsoil, loads, and so on for both lots are practically identical; the only difference is that the construction period for Lot 2 lasted some months longer due to consolidation waiting periods (slower building up of new embankment).

The settlements between the GECs are a bit larger. This is controversial to the formal assumption of equal settlements in Raithel (1999), Raithel and Kempfert (2000), and EBGEO (2011), and was registered again later (see, e.g., the Houten project in Section 17.6). However, the difference is negligible and without influence on the correctness of design procedure (Raithel et al., 2012).

The evaluation of pressures on top of GECs and between them provided by the earth pressure cells resulted in a column efficiency

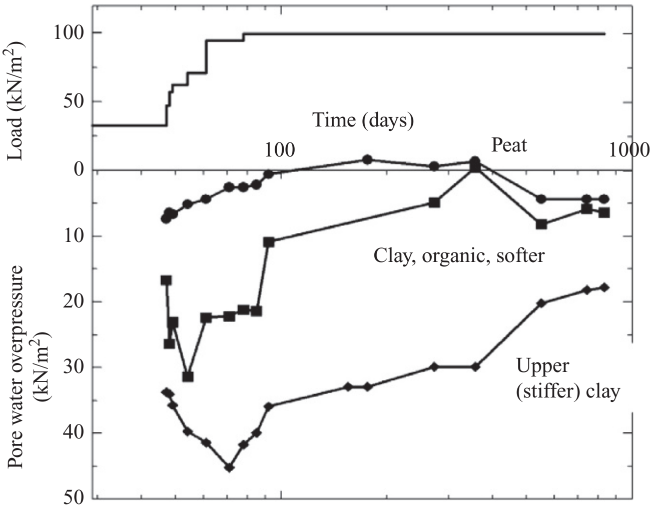

Figure 17.5 Typical development of pore water overpressure over time.

$E = 0.4$–0.6 varying over the time of construction and the GECs pattern; E is the ratio: load born by GECs/total load.

The development of pore water overpressure is depicted in Fig. 17.5. Three facts are of interest: (1) the maximum value ever registered (in the upper clay, see Fig. 17.2 and Table 17.1) is less than the half of applied load, (2) although the organic clay is in a disadvantageous position regarding draining, it consolidates quickly, and (3) the pore pressures in all soil layers start to decrease even under still increasing load before end of construction. The most plausible explanation is the huge draining capacity of the encased sand columns.

The project at Waltershof was completed and put in operation, meeting the time schedule and the cost estimations. It was a pioneering project for the real-life application of GECs and demonstrated most of the typical aspects of the system in the sense of philosophy and general behavior. This positive experience opened the door for further applications, for intensive research in Germany into a more precise design, and for further development of installation technique and geotextile encasements (Raithel, 1999; Alexiew et al., 2012; Alexiew and Thomson, 2013). The technological lessons learned led to the choice of smaller diameter GECs in the range of 0.6–0.8 m for future installations. The measurement program allowed for the first time a view into the inner life of the foundation system for a longer period.

17.3 EXTENSION OF AIRBUS SITE, 2000–2002

The site of the Airbus Company plant at Hamburg Finkenwerder at the Mühlenberger Loch on the River Elbe was enlarged by approximately 140 ha for new branches of production, in particular for the production of the new Airbus A380. The extension was carried out by enclosing an area of extremely soft soils and with tidal changes by a 2.4-km-long dike. The soft soil thickness with undrained shear strength S_U of only 0.4–10.0 kN/m^2 varied from 8–14 meters. The dike was founded on about 60,000 geotextile-encased sand columns of corresponding length with a diameter of 80 cm and a total installed length of about 650 km. It is to date the biggest single GEC job executed. The GECs were installed in about a year. A typical cross section of the GECs can be seen in Fig. 17.6. The characteristic drained consolidated soil parameters are summarized in Table 17.2. For more details, see Kempfert and Raithel (2002, 2004, 2014) and Raithel et al. (2002).

A geocomposite Comtrac where UTS = 500–1000 kN/m was installed on top of GECs combining high-strength and filter stability to accelerate

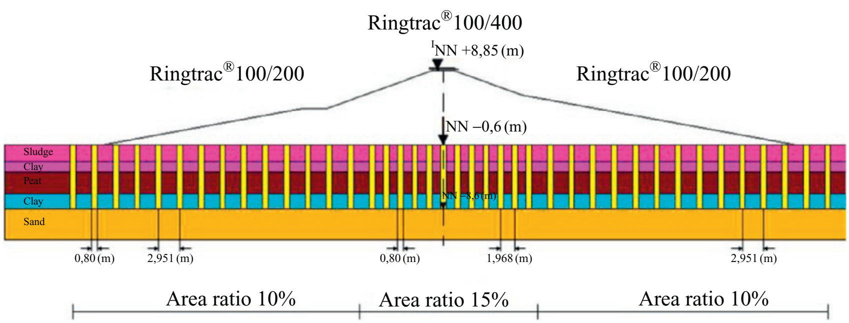

Figure 17.6 Typical dike cross section at the Airbus site at Mühlenberger Loch, Hamburg.

Table 17.2 Average soil parameters of the subsoil layers at Airbus site at Mühlenberger Loch, Hamburg

Soil layer	γ/γ_r (kN/m^3)	k (m/s)	E_s (MN/m^2)	φ' (°)	c' (kN/m^2)
Clay	6/16	2×10^{-10}	0.6	20.0	0
Sludge	4/14	2×10^{-10}	0.45	20.0	0
Peat	1/11	1×10^{-8}	0.55	20.0	0

γ/γ_r: unit weight, natural/saturated; k: coefficient of permeability; E_s: oedometric compression modulus; φ': angle of internal friction, drained; c': cohesion, drained.

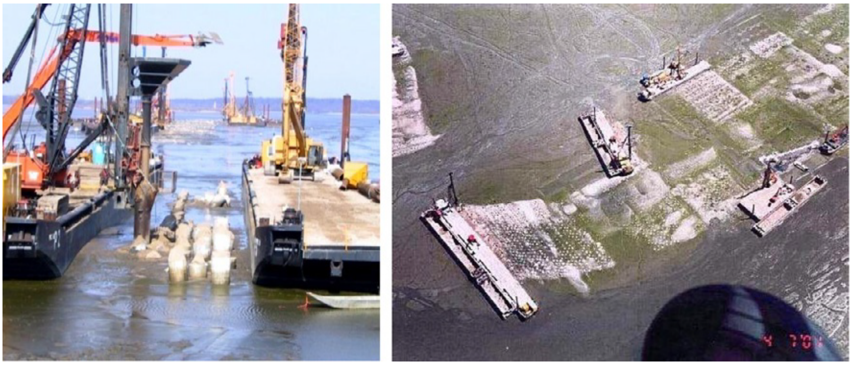

Figure 17.7 Overview photos of construction at the Airbus site, Hamburg.

Figure 17.8 Photos from the Airbus site, Hamburg: (a) GEC during high tide in streaming water; (b) high-strength geocomposite on top of GECs.

dike construction and to guarantee global dike stability, especially in the initial stage of construction. Figures 17.7 and 17.8 show the construction.

The stability and deformation predictions were verified by onsite measurements during construction. Figure 17.9 shows an example of development of settlements versus dike height and time. It can be seen again that the settlement ratio tends to decrease with time, despite the increasing height of dike. The most interesting point is at about day 300: although the dike height increases after that by about two meters, the settlement ratio drops down to almost zero. A possible explanation is the high degree of mobilization of GECs—higher than predicted by design assumptions. Most of the measurement instrumentation was designed for continued monitoring after completion of the dike. On the basis of settlement measurements over a period of 11 years, now the secondary or creep settlement can be studied (see Fig. 17.10).

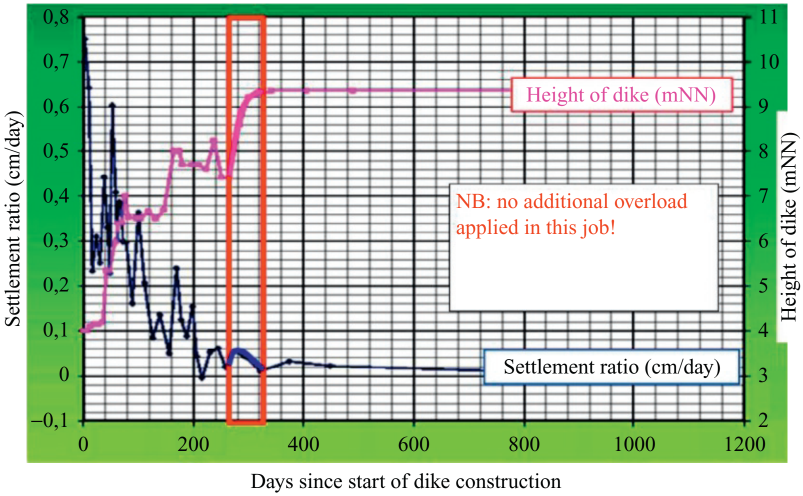

Figure 17.9 Example of measured settlement ratio vs. dike height.

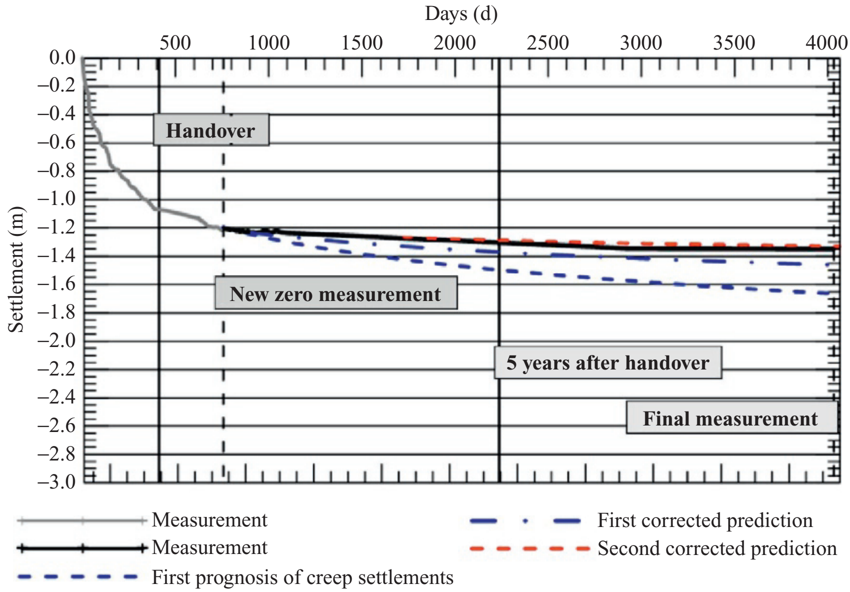

Figure 17.10 Results of long-term measurements and comparison with creep settlement predictions for the dike on GECs at the Airbus site, Hamburg.

The long-term settlement was first checked when primary settlement was practically complete, after roughly one year. A computational prediction was then made of further creep settlement. A further check in 2004 revealed significantly lower creep settlement than initially predicted.

A new prediction was then made using creep factors derived from the measurements by means of logarithmic regression functions. The predictions were revised again in 2013 on the basis of further settlement measurements over the last 11 years.

On the basis of comparisons between computational predictions and measurements, the reduction factor to be applied to the creep settlement for the unimproved subsoil is estimated at between 0.25 and 0.50, depending on the project parameters. In other words, GEC foundations achieve an approximately 50–75% reduction in creep settlement.

17.4 RAILROAD EMBANKMENT BOTHNIA LINE, 2001–2002

The Bothnia Line (Botniabanan) in Sweden is a 190-km-long, high-speed railway line in northern Sweden. Withstanding speeds of up to 250 km/h, this is also the highest-speed track in the country. The maximum axle weight amounts to 25 tons at 120 km/h (freight trains); lighter axle loads but speeds of 250 km/h are permitted for passenger trains. It is a single-track line. The Bothnia Line is the largest investment in the Swedish rail transport system since 1937.

At Bridge 4, the Bothnia line crosses a valley with very soft soils with a depth of up to 7.5 m. The embankments on both sides of the bridge have a height of 9–10 m. Due to the settlement requirements, a GEC foundation with a diameter of 80 cm using the displacement method was carried out in 2001. The GECs were arranged in a triangular pattern with 15% replacement ratio. Figure 17.11 shows a typical cross section.

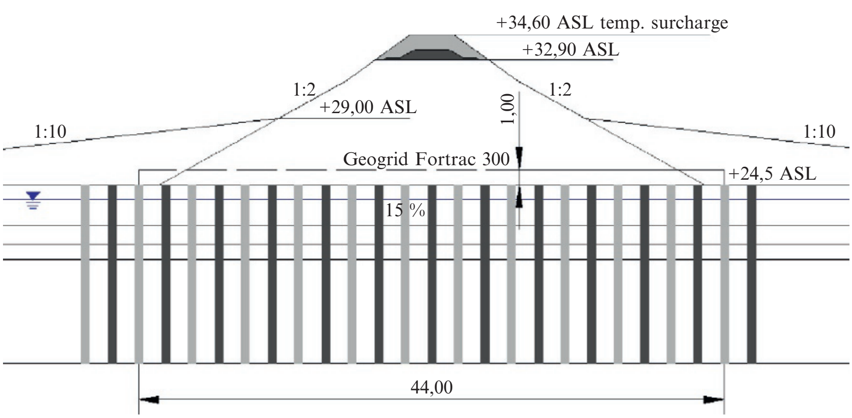

Figure 17.11 Cross section of the embankment with the GEC-column pattern.

The soft soil consists of alternating layers of silt and clay. The typical soil parameters were evaluated based on test data. An average of the soil sample depths was used to determine the boundaries of the layers for the design scheme and calculations. The oedometric modulus E_{oed} (E_s) was derived from the stress–strain curves of compression tests in stress-dependent form. The selected profile for design purposes is summarized in Fig. 17.12.

The design resulted in the choice of geotextile encasement Ringtrac 400 with UTS = 400 kN/m in the ring direction. At that time, seamless woven encasements had been developed and used, thus avoiding any weak zone in the ring (compare the first encasements in the previously described Waltershof project). However, for the first time, coarse crushed rock (basalt) was decided on for the column fill, as it was easily available in this region. A significantly higher damage of geotextile encasement (i.e., loss of strength) during installation and compaction had to be expected compared to the use

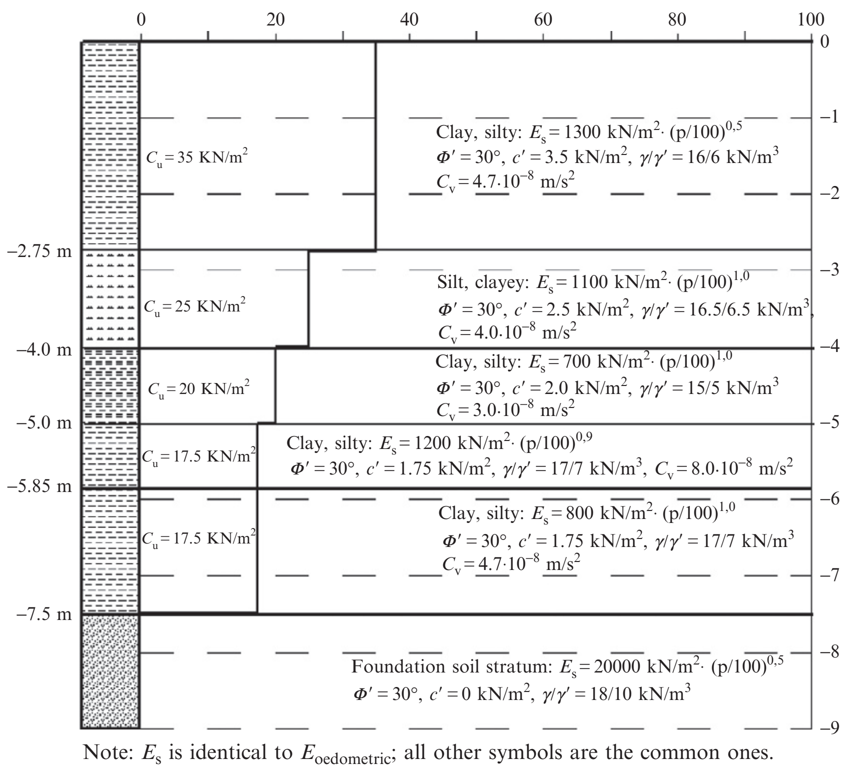

Figure 17.12 Characteristic values of subsoil layers at the Bothnia Line project.

of sands. For determination of the reduction factor $RF_{inst\ dam}$ for installation damage, the installation process was simulated at the Geotechnical Faculty of the University of Kassel.

For more details on this and about design strength in general, see, for example, EBGEO (2011). Crushed rock from Sweden was placed in two layers in the lower half of a 30 cm × 30 cm steel box. Each layer was compacted statically under 200 kN/m^2 for 60 seconds. Then the Ringtrac samples were installed followed by the upper half of the box filled with loose crushed rock. A cyclic load was applied by a steel plate on top of the system with pressure amplitude from 5–900 kN/m^2, a frequency of 1 Hz, and 200 load cycles. An average $RF_{inst\ dam} = 1.36$ was evaluated (in comparison to typically 1.05–1.15 for sands) and applied for estimation of the ring design strength F_d.

The design resulted in calculated settlements of about 60 cm. To be on the safe side, an angle of internal friction for the column fill $\phi' = 37.5°$ was assumed. After construction, the settlements of the embankment were measured for a period of about 250 days (Fig. 17.13) showing significantly smaller settlements. A postmeasurement design simulation (Fig. 17.13) demonstrated that $\phi' = 45°$ for the crushed fill is more realistic. This is the well-known dilemma of judgment of implicit or explicit "reserves" in

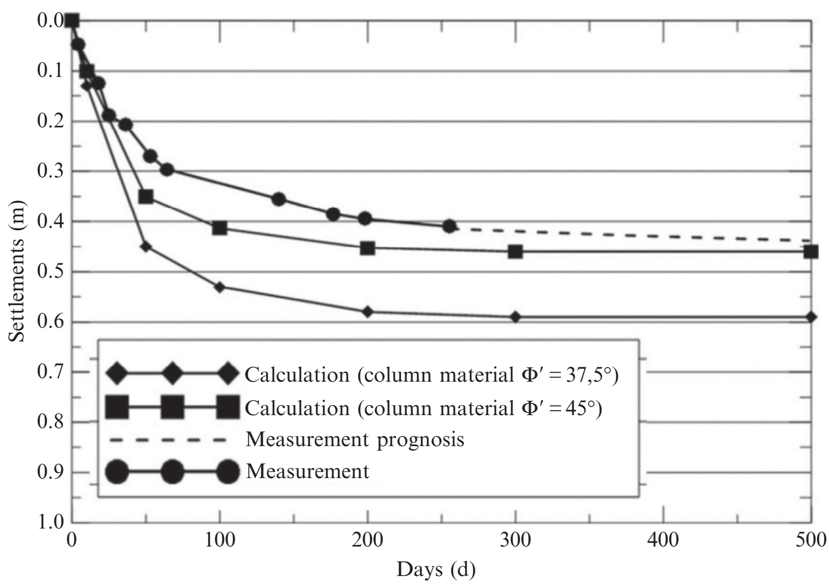

Figure 17.13 Bothnia Line: calculated and measured settlements.

geotechnical designs, especially for critical projects of high category (here a high-speed rail link). However, the general trend of settlement development over time looks correct. It can be seen again (see Fig. 17.4, Waltershof) that most (about 90%) of the primary consolidation settlements occur in the first three months after the start of construction.

The successful use of compaction of coarse GEC fill was demonstrated under realistic consideration of higher installation damage (i.e., reduced design strength) of encasement. Note that some years ago in a test field these aspects were additionally studied using a wide range of fills including, for example, crushed construction debris. A short overview concerning the interaction and optimization of the system components' area ratio, fill, and encasement can be found in Alexiew and Thomson (2013).

17.5 HIGH-SPEED RAIL LINK, 2002

At a place called Westrick near Breda, Netherlands, the new high-speed railroad link (HSL) from Paris to Amsterdam had to cross at over some hundred meters of former waste disposal. The Westrick landfill is an old sandpit that was filled from 1947–1959 with unsorted uncompacted municipal and industrial waste. The waste thickness is in the range of 4–6 m followed by sands with some thin clay interlayers (Fig. 17.14). All materials demonstrated a significant degree of contamination with oil, PAC, and heavy metals. The pH varied from 9.0–10.5 (i.e., strongly alkaline).

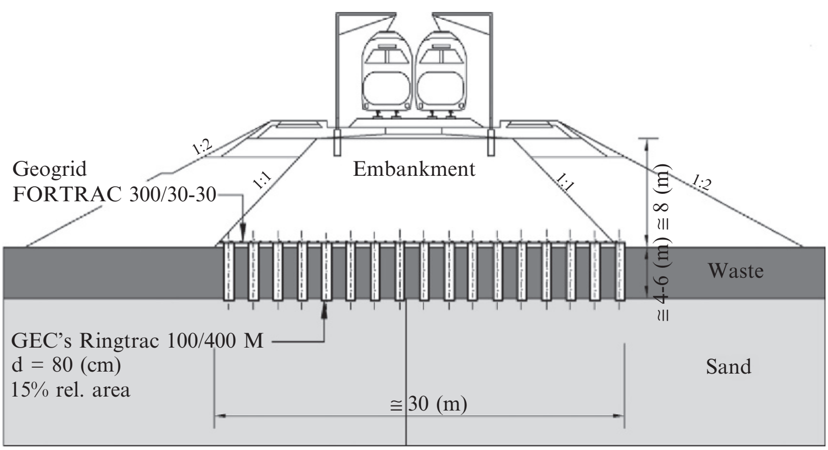

Figure 17.14 HSL Paris–Amsterdam at Westrick: typical cross section.

The composition, texture, and parameters of the old waste differed strongly, as was expected. There was the possible inclusion of, for example, old refrigerators or concrete blocks from construction debris and also the possibility of high chemical aggressivity.

In the beginning two options were under discussion: waste replacement by clean compacted sands or a reinforced concrete plate on piles of about 10 m in length. After additional studies, a solution on GECs (see Fig. 17.14) became the preferred option due to ecological, financial, and logistic reasons. For more details, see Nods and Brok (2003).

The design was relatively conservative due to the settlement limitations, the extreme inhomogeneity of waste, and the high chemical impact. However, due to economical reasons, it was decided not to install GECs below the shoulders of the embankment outside a load-spreading zone below the tracks (Fig. 17.14). The GECs had a diameter of 0.8 m with an average replacement ratio of 15% and a ring UTS of 300 and 400 kN/m. They were produced from polyvinylalcohol (PVA) as the high chemical resistance in a wide range of media and the high ring tensile modulus in the short- and long-term (after creep) ensured a more efficient reduction of settlements (absolute and differential).

There were three aspects concerning settlements in the GEC-supported zone: (1) to keep them less than about 10 cm, (2) to keep them as equal as possible despite the extreme waste heterogeneity, and (3) to achieve a quick consolidation during and after construction. Note that the modulus (tensile stiffness) of PVA is roughly two times higher than of, for example, even high-tenacity polyester (PET) (Alexiew et al., 2000; Alexiew and Thomson, 2013).

Additionally, for equalizing settlements and to increase global stability, a geogrid with a UTS = 300 kN/m was designed to be placed on top of the GECs (Fig. 17.14). In total, around 2200 GECs were installed in June and July 2002 using sand as fill. The displacement method was applied successfully, despite the problematic character of waste. The productivity varied from 40–80 GECs per day, depending on the local resistance of waste.

A measurement program was applied using flexible horizontal inclinometers level with the tops of the GECs, mainly to register settlement development and to decide when track installation on top of embankment could commence. Results for the left part of a typical cross section are shown in Figs. 17.15 and 17.16. The GEC-supported and unsupported zones are also shown for a better understanding. The most important facts are:

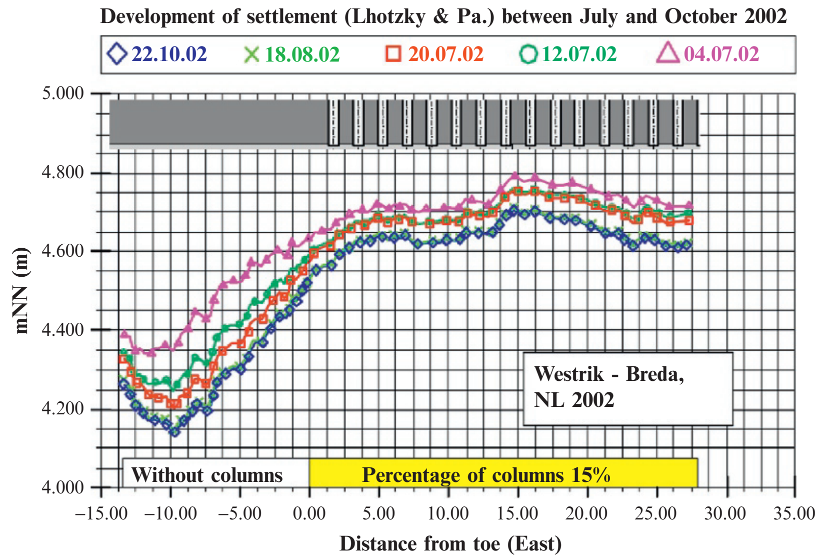

Figure 17.15 Westrick: development of settlements from July 4 (start of embankment construction) until October 22, 2002 (last measurement).

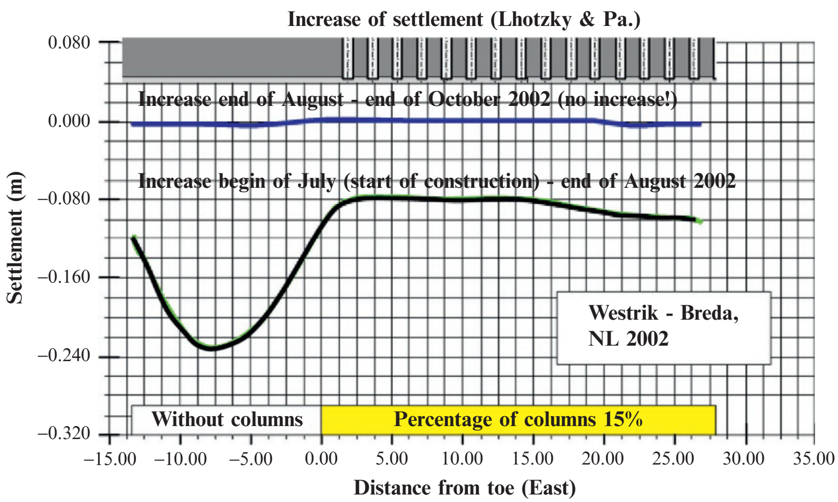

Figure 17.16 Westrick: increase of settlements from July 4 (start of embankment construction) until the end of August and from the end of August until October 22, 2002 (last measurement).

1. The settlements of the nonsupported shoulder are—despite the lower average load—approximately three times higher than for the GEC-supported full-height part of the embankment (Fig. 17.15, and especially Fig. 17.16).
2. The settlement profile in the GEC zone is not really even/smooth (Fig. 17.15); at the beginning the maximum absolute difference amounts to 15 cm. This is not surprising due to the strongly varying character of the heterogeneous waste; however, this difference is technically negligible and of no influence at higher levels in the embankment—the difference decreases later to 10 cm (although the settlement curve character remains the same), indicating a time-dependent equalization process.
3. The consolidation (and equalization) process is quick, especially in the GEC-supported zone (Fig. 17.16): after only two months from the beginning of embankment construction there is practically no further increase.
4. The total settlement in the GEC-zone was at a maximum of about 10 cm; this is a very modest value.

   Regarding the project at Westrick, some specific issues are worth bearing in mind: GECs can be installed even in such a problematic "subsoil" as heterogeneous waste, and both absolute and differential settlements can be controlled—this was a specific but hardly predictable aspect in this case.

17.6 BASTIONS VIJWAL HOUTEN, 2005

The area of Houten-Zuid in the Netherlands is one of the locations that the Dutch government had targeted as a growing area for housing development in the country. In two areas of Houten-Zuid, landscape embankments (the so-called Bastions West and East) were planned as connections between the residential area and the natural landscape around. Due to the soft subsoil, settlements of 1.6–1.9 m for Bastion West and 0.5–0.8 m for Bastion East were expected to occur.

The problems faced were not only the settlements, but also the long consolidation time and the possible lateral pressure in depth on the rigid pile foundations of the already-constructed adjacent buildings. For more details, see Brokemper et al. (2006). After considering different options, GECs were chosen as the optimal foundations to base the Bastions on. Figure 17.17

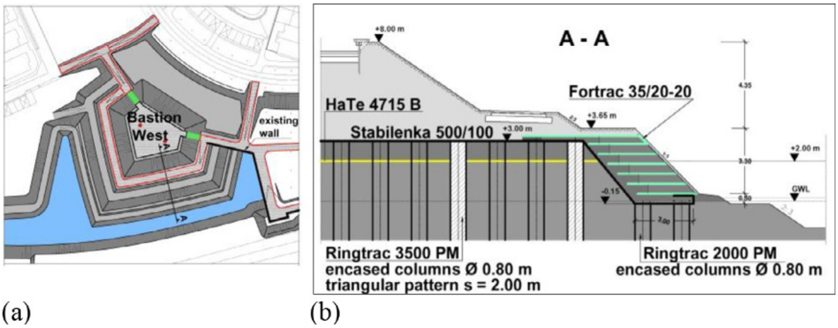

Figure 17.17 Bastions at Houten: (a) plan view of Bastion West with a height of approximately 6 m on approximately 8 m of soft soil; (b) a typical cross section with GECs, horizontal reinforcement and water level.

Table 17.3 Main data of the project Bastions at Houten

Embankment	Bastion West	Bastion East
Height	5.5 m	5.5 m
Fill material	$\gamma = 17$ kN/m^3 / $\varphi' = 20^\circ$ / $c' = 2$ kN/m^2	$\gamma = 17$ kN/m^3 / $\varphi' = 20^\circ$ / $c' = 2$ kN/m^2
Traffic load	20 kN/m^2	20 kN/m^2
Soft soil layer	Organic clay and peat	Sandy organic clay
Thickness	7.5 m	3.0 m
Properties	$\gamma = 14$ kN/m^3 / $\varphi' = 17^\circ$ / $c' = 2.5$ kN/m^2 $E_{s,pref} = 2000$ kN/m^2 ($p_{ref} = 100$ kN/m^2)	$\gamma = 17$ kN/m^3 / $\varphi' = 22.5^\circ$ / $c' = 2$ kN/m^2 $E_{s,ref} = 3000$ kN/m^2 ($p_{ref} = 100$ kN/m^2)
Ground water level	−2.0 m	n.a.
Foundation system	Geosynthetic encased columns	Geosynthetic encased columns
Geometry	s = 2.00 m, $dc = 0.80$ m	s = 2.30 m, $dc = 0.80$ m
Column fill	$\gamma = 19$kN/m^3 / $\varphi' = 32.5^\circ$ / $c = 0$ kN/m^2(sand)	$\gamma = 19$kN/m^3 / $\varphi' = 32.5^\circ$ / $c = 0$ kN/m^2 (sand)
Encasement	Ringtrac® 3500 PM — UTS = 200 kN/m, $J_K = 3500$ kN/m, $J_d = 2100$ kN/m	Ringtrac® 2000 PM — UTS = 130 kN/m, $J_K = 2000$ kN/m, $J_d = 1000$ kN/m
Basal reinforcement	Stabilenka® 500/100 — UTS = 500 kN/m	Stabilenka® 500/100 — UTS = 500 kN/m
Estimated settlements	≤0.40 m	≤0.15 m

* J_k = short term radial tensile stiffness; J_d = long term radial tensile stiffness (120 years)

shows the plan view and a typical cross section of Bastion West, and the main parameters of the project are given in Table 17.3.

A total of 780 GECs were installed. Two different types of encasement were used with different ring tensile stiffness (modulus) and strength, and in a different pattern (Table 17.3), both made of high-modulus PVA and with the same diameter of 0.8 m.

The Royal BAM Group operated at both Bastions with the same equipment: a 100-t piling rig and heavy vibrators, changing only the length

Figure 17.18 Bastions at Houten: (a) surface of working platform and heads of GECs; (b) GEC for load test; (c) Bastion West just after the end of construction.

of the steel installation pipes. The displacement method of installation was used. Up to 40 GECs per day were installed.

In Fig. 17.18 the surface of the working platform and the heads of installed GECs are shown and also the view of Bastion West after the end of construction. To ensure the appropriate compaction of the column fill material and the estimated load-bearing capacity of the columns, penetration tests inside the columns and simple short-term load tests were carried out (Fig. 17.19(a)). Due to financial restrictions, long-term settlement measurements on top and between the columns (using single settlement plates) were limited. Figure 17.19(b) shows typical results for Bastion West; after three months it was clear that the solution met the requirements, thus it was decided to stop the measurements. However, both the short-term single column load tests and the long-term settlement measurements fit very well the prognoses.

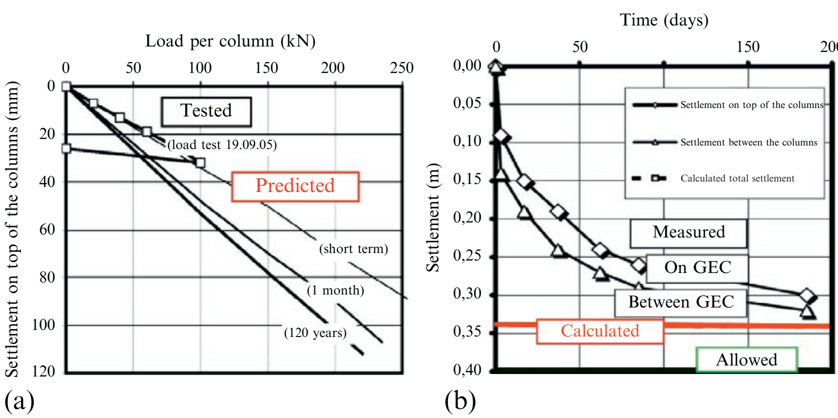

Figure 17.19 (a) short-term load test on a single GEC; (b) long-term settlements on top and between GECs.

A notable lesson from the Bastions Houten project was that GECs can be an efficient solution even for smaller projects with less than 10 km of total column installed length; before that, the GEC system had been mainly used for large-scale projects.

17.7 THYSSENKRUPP CSA STEEL PLANT, 2006–2010

The ThyssenKrupp CSA (TKCSA) steel plant had to be built on extremely soft soil lowlands in the Rio de Janeiro neighborhood of Sepetiba, Brazil. The plant included a large heavy-loaded stock yard for coal, coke, ore, and additives (Alexiew et al., 2010). Figure 17.20 shows a typical soil profile down to about 18 m below terrain. Very critical was the so-called "upper clay" of 8–10 m thickness. The stockyard included stock pile beds and runways (RW; a type of very large railroad) for the stackers/reclaimers (S/R) of 750 tons each (similar to the big excavators in mining pits), which were sensitive to any deformations of the RWs.

A wide range of factors had to be taken into consideration regarding the foundation of the RWs (Alexiew et al., 2010). The final decision was to found them on GECs in the coal/coke area. Key factors, among others, were the ductile behavior of the GEC system under lateral subsoil pressure combined with controlled behavior and constructability even in very soft soils. Some later RW maintenance in rail position corrections (if needed) was part of the client's concept in this extremely disadvantageous geotechnical situation.

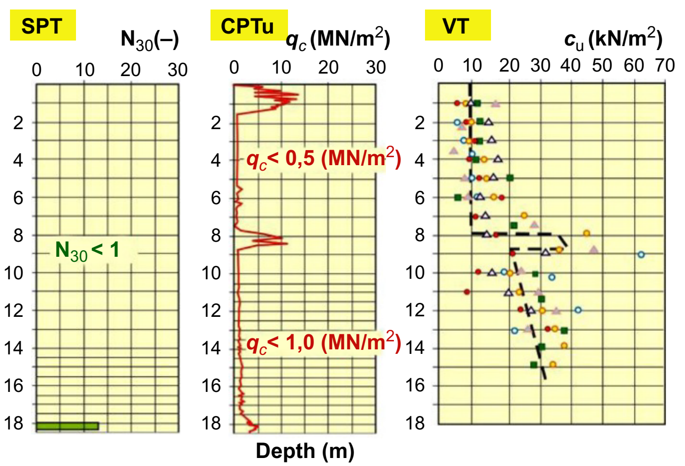

Figure 17.20 TKCSA steel plant: typical geotechnical profile.

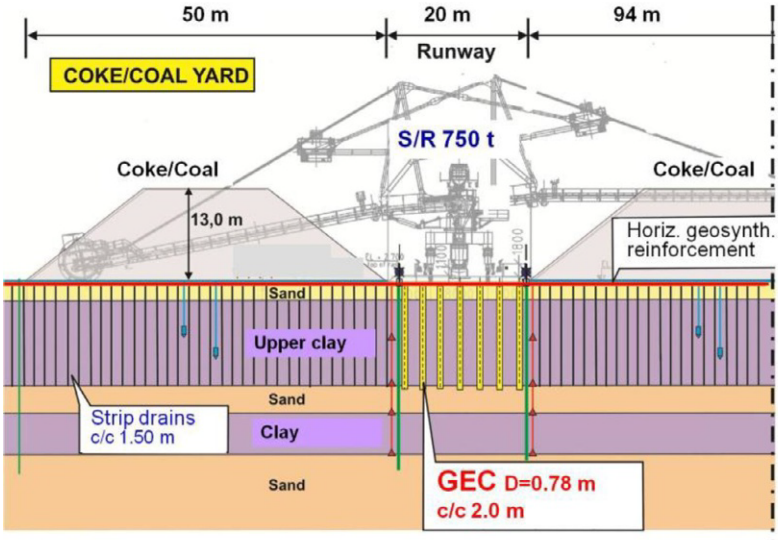

Figure 17.21 TKCSA steel plant: typical cross section (partial view).

Figure 17.21 shows a typical cross section of the system. A total of about 250 km of GECs were installed with a diameter of 78 cm, a center-to-center spacing of 2 m, average length of 10 m, and varying strength. Dredged sands were used as a fill.

In the early stage of stockyard construction, an instrumented load test was performed to study the behavior of a full stock bed and an adjacent GEC-supported runway on high-strength low-strain horizontal geosynthetic reinforcement from PVA with UTS of up to 1600 kN/m. Figure 17.22 shows some test results. Note that there are no GECs installed to the left.

Although the loading iron slag embankment and the horizontal reinforcement are symmetrical, due to the presence of GECs to the right, the lateral displacement at terrain level is zero, and the maximal lateral displacements and the zone of their influence are reduced to almost half in comparison to the left zone without GECs.

Loading tests were performed on the first runway on GECs under the first stacker/reclaimer (unpublished, private correspondence of first author). The consolidated settlements due to runway platform of 2–3 m thickness, ballast bed, sleepers, and so on, amounted to 20 cm after two months. A 750-t stacker/reclaimer kept in a fixed position for a week provoked

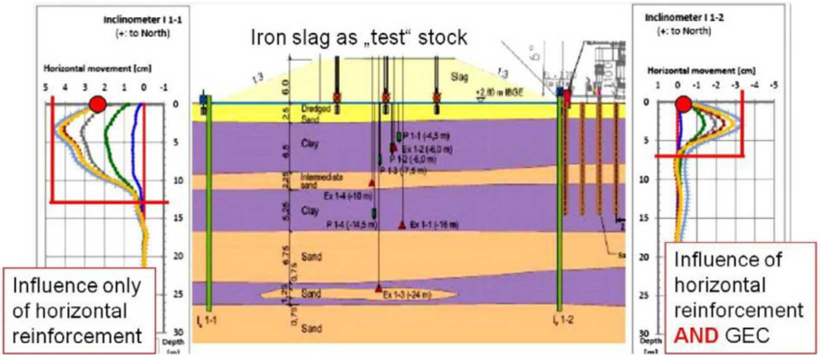

Figure 17.22 Different horizontal displacement behavior without (*left*) and with (*right*) adjacent GECs.

about 4 cm of additional settlement, and a similar test with rotated boom (i.e., under maximal load eccentricity) resulted in a cross-tilting of runway of <0.25%. These data met the requirements concerning the short- and long-term RW deformations, thus, these tests gave the final green light for using GECs for the runway.

17.8 A2 MOTORWAY EMBANKMENT, 2010–2011

The first state-of-the-art application of GEC in Poland was carried out in Jordanowo during the construction of the A2 motorway ("autobahn"). After consideration of many design options, the GEC system was chosen due to its capabilities in a complex geotechnical area such as Jordanowo. In the case of this section of the A2, some new developments were needed due to extremely long columns, installed in soft soil deposits down to 28 m under the level of a working platform (Figs. 17.23 and 17.24). Because of the quite flat final embankment geometry in combination with very stringent postconstruction settlement limitations, a temporary preload was applied to faster provoke a high degree of consolidation (Küster et al., 2012).

The general subsoil stratification is characterized by peat in the uppermost 5 m, followed by the so-called "gyttja" (a kind of sensitive problematic clay) down to the underlying bearing soil layer. In the so-called "peat area 15.2," an embankment height of 4 m was considered in the calculations. The maximum thickness of the soft soil here amounts to 22–28 m. Some properties of soft soil strata and sand used in the design of the embankment and the GECs are shown in Table 17.4.

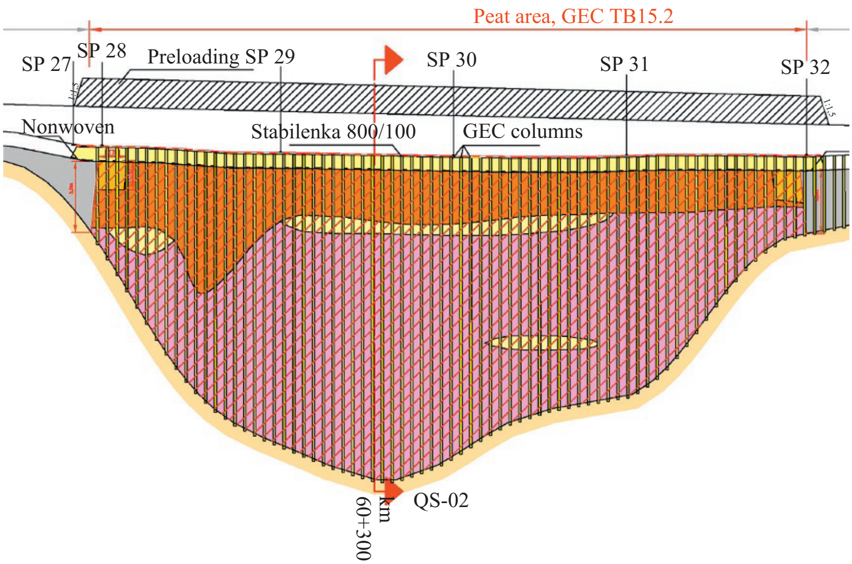

Figure 17.23 Longitudinal profile: embankment with preloading, km 60+225 to km 60+450.

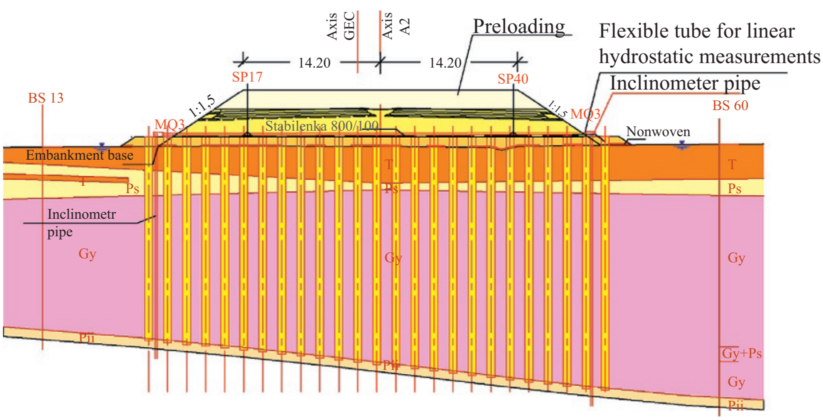

Figure 17.24 Cross section, embankment with preloading, km 60+300.

Between km 60+225 and km 60+450 the GEC system was designed and executed with 80-cm diameter columns in a triangle pattern (axial spacing 1.97 m, area ratio of 15%, total number of columns 3400). After installation of the columns, a sand-leveling layer with a thickness of 30 cm was placed and a strong basal reinforcement (woven Stabilenka® 800/100 with an UTS = 800 kN/m) installed.

Table 17.4 Properties of the soft soils and sand used for GECs and embankment

Soil type	Unit weight γ/γ' (kN/m)	Friction angle φ' (°)	Cohesion c' (kN/m^2)	Oedometer module E_{oed}' (kN/m^2)	Coefficient of consolidation $c_h = 5\times c_v$ (m^2/s)	Creep factor c_α (–)
Sand for embankment	19/10	35	(–)	(–)	(–)	(–)
Sand for columns	19/10	32.5	(–)	(–)	(–)	(–)
Peat	11/1	15	5	500*	1.59×10^{-7}	0.03
Gyttja	14/4	20	5	750*	3.96×10^{-8}	0.01
Deep sand	20/10	32.5	(–)	(–)	(–)	(–)

* E_{oed} for stress range 0–100 kN/m^2

Many of the GECs had to have a length of up to 28 m. The corresponding heavy equipment needed for their installation operated on wooden platforms (6.0 m × 6.0 m), which were stepwise moved and positioned over already-installed GECs using them as support. Thus, the new columns were installed ahead of the working front (Fig. 17.25(a)). The displacement method of installation was applied. For the longest GECs pipes, ones with attached "sacrificial" base plates were used (Fig. 17.25(b)). Instead of a classical flat clamp vibrator on the upper edge of these installation pipes, a moveable ring vibrator was used (Fig. 17.25(c)) to enable the step-by-step drawing down of the very long one-piece pipe.

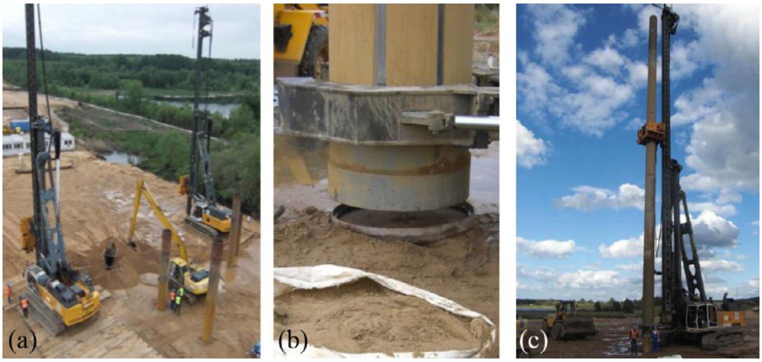

Figure 17.25 (a) Installation rigs standing on wooden platforms placed on already installed GECs. (b) Lost plate used at the base of the longest displacement pipes. (c) Moveable ring vibrator.

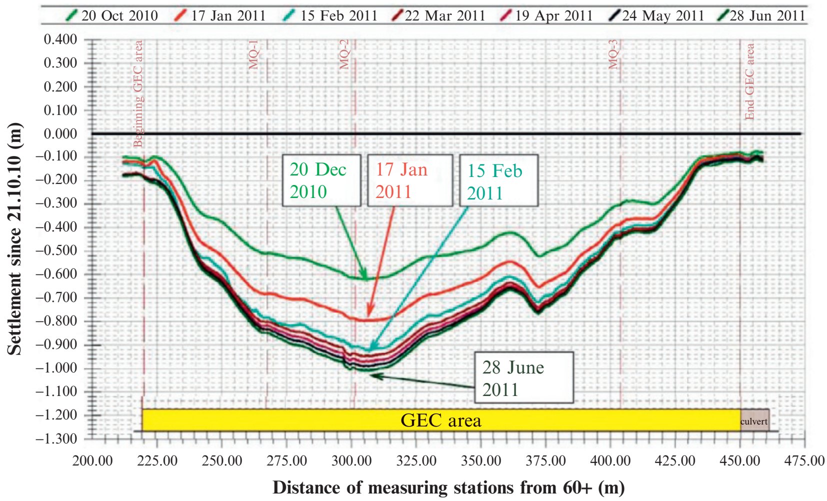

Figure 17.26 A2 Jordanowo: Embankment base settlements' longitudinal profile.

The project was accompanied by a comprehensive geotechnical measuring program (described in detail in Raithel et al., 2012) to verify the assumptions made in the design stage, to allow for comparison of design predictions and real behavior, and (most important) to allow a correct decision concerning the removal of the preload. The horizontal line 0.000 marked in Fig. 17.26 represents the start configuration of monitoring on October 29, 2010. The additional seven measurement sets present the deformed base of embankment along the installed hydrostatic lines. More than 80% of settlements took place in about two months only. The maximal settlement s measured on June 28, 2011 was $s = 1.05$ m. The predicted value of max settlement was $s = 2.3$ m, significantly higher.

A possible explanation for this difference is the underestimation of compression moduli in the geotechnical reports. Furthermore, the monitoring did not consider settlements caused by the weight of the working platform and the leveling layer, and also settlements under the weight of the heavy installation equipment with a slow change of position (i.e., very long columns, longer installation time per column in a constant position).

As mentioned earlier, there were stringent settlement limitations due to serviceability aspects (i.e., high driving velocity with up to 130 km/h). The client specified a maximal allowable postconstruction settlement difference

$\leq$15 mm between two points at 10 m from each other over a period of 30 years. The total postconstruction settlement has to be $\leq$10 cm over the same period.

Based on a detailed evaluation and conservative extrapolation of all the data collected, it was possible to prove that the requirements were met. The segment of A2 was put into operation in autumn 2011.

17.9 OTHER CASE STUDIES

Not all case studies believed to be of interest can be presented here. Some of them are handled only in unpublished reports; some have been described in published reports but are not discussed herein. The authors recommend the interested reader study at least three of the latter (see also References at end of chapter), all of which include results from measurement programs. They are, in chronological order:

- A precise project description of a highway on GECs in Brazil on quite inhomogeneous soils (De Mello et al., 2008).
- A very comprehensively instrumented test embankment on GECs in Germany (Raithel et al., 2012).
- An interesting application of GECs to protect rigid pile foundations against soft subsoil lateral pressure (Schnaid et al., 2014).

17.10 CONCLUSION

The GEC foundation system has reached the stage of maturity. Projects have been successfully executed worldwide. At least one verified and codified design procedure is available (EBGEO, 2011; sometimes a bit conservative about overestimating settlements) and two approved practice installation options are established as well. Installation techniques and equipment are quite simple and accessible for everyone; GEC lengths of up to 28 m have been installed but are not the limit. A wide range of noncohesive fills can be used, including sand (the most commonly used fill until recently), which can be a significant advantage, for example, in lowlands and/or on seashores, where other fills are rare or expensive. A wide range of geotextile seamless encasements from two polymers and with diameters of 40–100 cm are available. These are easy to transport made-in-plant engineered controlled elements. Consequently, an optimized solution is possible for many project circumstances.

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In all the projects described (and the ones not described) here, many competent colleagues (designers, owners, supervisors, installers, etc.) have been active, flexible, and enthusiastic. Their successful efforts are strongly appreciated.

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