Primera Conferencia Nacional de Geosintéticos-Honduras 2014
Tegucigalpa, Honduras 16 – 18 de Junio

Ingenuity in Geotechnical Design using Geosynthetics

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Zornberg, J.G. (2014). "Ingenuity in Geotechnical Design using Geosynthetics." Proceedings of the *First National Conference on Geosynthetics, Honduras 2014,* Tegucigalpa, Honduras, 16-18 June (CD-ROM).

ABSTRACT: Even though geosynthetics are now a well-established discipline within geotechnical engineering, ingenuity continues to be significant in projects involving their use. This is probably because of the ability to tailor the mechanical and hydraulic properties of geosynthetics in a controlled manner to address design needs in all areas of geotechnical engineering. This paper focuses on specific advances involving the use of geosynthetics in a wide range of geotechnical projects. Specifically, this paper addresses the creative use of geosynthetics in the design of earth dams, resistive barriers, unsaturated barriers, veneer slopes, coastal protection systems, foundations, bridge abutments, retaining walls, embankments, and pavements.

1 INTRODUCTION

Geosynthetics play an important role in geotechnical applications because of their versatility, cost-effectiveness, ease of installation, and good characterization of their mechanical and hydraulic properties. Probably because of these many attributes, the use of geosynthetics has often promoted ingenuity in multiple areas of geotechnical engineering. This paper discusses 10 (ten) cases of recent applications (or recent evaluations of pioneering applications) of geosynthetics in geotechnical projects. It updates the information provided by Zornberg (2012). For each type of geotechnical project, the following aspects are discussed: (i) some difficulties in their design, (ii) a creative approach to address the difficulties using geosynthetics, and (iii) a recent project illustrating the creative use of geosynthetics.

2 CASE 1: RECENT INNOVATIONS IN EARTH DAM DESIGN

2.1 *Some difficulties in the design of earth dams*

Filters are both expensive and critical components of large earth dams. The objective of drains and their associated filters is to lower the phreatic surface within the dam to prevent water from emerging from the downstream slope, where flow could trigger erosion that may endanger the integrity of the structure.

The configuration of the filter zones depends on the type of embankment. In a homogenous dam, the filter is generally placed as a blanket of sand and fine gravel on the downstream foundation area, extending from the cutoff/core trench boundary to the edge of the downstream toe. Instead, in a zoned dam the filter is placed between the core and the downstream shell zone. A longitudinal chimney drain collects the intercepted seepage flow and, via one or more transverse drains, conveys the water to the toe drains outside the embankment. Satisfying the filter requirements in the downstream drains may be particularly difficult in projects where the appropriate aggregate sizes cannot be obtained in sufficient quantities.

2.2 *A Creative Approach using Geosynthetics: Geotextile Filters*

Geotextiles can be used as filters in critical projects such as earth dams. They constitute a particularly attractive solution in projects where granular material is not readily available. While there has been significant resistance among dam designers towards the use of new filter materials such as geotextiles, the design base and experience in their use has continued to grow. For example, a recent re-evaluation of filter criteria was conducted, which confirmed the suitability of using geotextiles as filters in large earth dams (Giroud 2010).

The recent re-evaluation led to four criteria for geotextile filters: permeability, retention, porosity, and thickness criteria. Filtration is governed by the distribution of openings in the filter material. The characteristics of filter openings are their size, shape, density (number per unit area) and distribution. The four criteria address three of these four characteristics: the size, density and distribution. The shape of filter openings is not addressed in the four criteria, but is likely to be a minor consideration (Giroud 2010). On the other hand, the shape of openings may be a relevant issue in the case of some woven geotextiles and some other types of man-made filters. Ultimately, the four proposed criteria for geotextile filters form a coherent set that allows safe design of geotextile filters.

2.3 *The Recent Re-evaluation of a Pioneering Project: Valcros Dam, France*

The pioneering project described herein, and reevaluated in light of a recently re-assessment of filter design criteria, is Valcros Dam. This is the first earth dam designed with geotextile filters. It was constructed in France in 1970 using a geotextile filter under the rip-rap used to protect the upstream slope of the dam. Very importantly, a geotextile filter was also used in the downstream drain of the dam.

Valcros Dam is a 17 m-high homogeneous dam constructed with a silty sand having 30% by mass of particles smaller than 0.075 mm. Adequate sand filter could not be obtained for the downstream drain, leading to the use of a nonwoven geotextile as the filter. The construction of the downstream drain of the dam with a geotextile filter is shown in Fig. 1. The geotextile used in the downstream drain was a needle-punched nonwoven geotextile made of continuous polyester filaments, with a mass per unit area of 300 g/m^2. The performance of the drain has been satisfactory since its construction. This can be concluded from: (i) a constant trickle of clean water, (ii) a flow rate at the drain outlet that has been consistent with the hydraulic conductivity of the embankment soil, and (iii) no seepage of water ever observed through the downstream slope (Giroud 2010).

Fig. 1. Construction of the downstream drain of Valcros Dam (Giroud 1992)

The good condition of the geotextile filter was confirmed using samples of geotextile removed from the actual filter after 6 and 22 years of completion of construction. In fact, clogging was found to be negligible (only 0.2% of the pore volume of the geotextile). The good performance of the geotextile filter can be explained by a recent reassessment reported by Giroud (2010). It should be noted that the Valcros Dam filter was not designed using criteria derived directly from the classical Terzaghi's filter criteria. Instead, the geotextile filter was selected on the basis of limited experimental data available at that time (1970) involving the use of this geotextile under an experimental embankment constructed on saturated soft soil. The recent reevaluation of the use of a geotextile filter at Valcros Dam indicates that the geotextile indeed meets the current criteria for permeability, porosity, thickness and retention.

3 CASE 2: INGENUITY IN THE DESIGN OF RESISTIVE BARRIERS

3.1 *Some Difficulties in the Design of Resistive Barriers*

Conventional cover systems for waste containment involve resistive barriers, which may be particularly expensive when appropriate soils are not locally available. This includes the availability of topsoil, cover soil, drainage materials, and vegetation components. Additional costs include their annual operation and maintenance requirements, loss of revenue due to decreased landfill volume, and detrimental effects of post-construction settlements. In the case of steep landfill slopes, additional concerns involving the use of cover soils are erosion and stability along interfaces with comparatively low interface shear strength.

3.2 *A Creative Solution by using Geosynthetics: Exposed Geomembranes*

Many of the cost- and performance-related concerns associated with the construction of conventional cover systems can be minimized or eliminated by constructing exposed geomembrane covers. These covers are particularly suitable for sites where the design life of the cover is relatively short, future removal of the cover system may be required, the landfill sideslopes are steep, cover soil materials are prohibitively expensive, or where the landfill is expected to be expanded vertically in the future. In addition, the current trend towards the use of "leachate recirculation" or "bioreactor landfills" makes the use of exposed geomembrane covers a good choice during the period of accelerated settlement of the waste.

Key aspects in the design of exposed geomembrane covers are the assessment of the geomembrane stresses induced by wind uplift and the anchorage requirements against wind action. Wind uplift of the geomembrane is a function of the mechanical properties of the geomembrane, the landfill slope geometry, and the design wind velocity. Wind uplift design considerations involve assessment of the maximum wind velocity that an exposed geomembrane can withstand, of the required thickness of a protective layer that would prevent the geomembrane from being uplifted, of the tension and strain induced in the geomembrane by wind loads, and of the geometry of the uplifted geomembrane. Procedures for the analysis of geomembrane wind uplift have been developed by Giroud *et al.* (1995) and Zornberg and Giroud (1997). A number of exposed geomembrane covers have been designed and constructed using these procedures (Gleason *et al*. 2001).

3.3 *A Recent Project: The Tessman Road Landfill, TX*

The Tessman Road Landfill, located near San Antonio (Texas), was designed and constructed with an exposed geomembrane cover. In order to accommodate the wind uplift, the geomembrane requires high tensile strength properties. The good mechanical properties of geomembrane required by the design made it feasible to mount an array of flexible solar laminate panels. This led to the first installation of a solar energy cover (Roberts *et al.* 2009). The solar energy cover was installed during only a two-month period in early 2009 and is now generating about 120 kW of renewable solar power (Fig. 2).

Fig. 2. Aerial view of the exposed geomembrane with arrays of solar panels at the Tessman Road Landfill (Roberts 2010)

The solar power is tied directly into the existing "landfill gas to energy" system. The Tessman Road Landfill Solar Energy Cover allows generation of renewable energy, creates a revenue stream, and reduces maintenance requirements. The material selected for the Tessman Road Landfill Solar Energy Cover is a green, 60-mil, fiber-reinforced, flexible polypropylene– based thermoplastic polyolefin product. The product offers high strength, flexibility, and a relatively low expansion-contraction coefficient.

The flexible solar panels are less than ¼-inch thick and with a surface of about 23 ft^2. A total of 30 solar panels are arranged in rectangular sub-arrays. A total of 35 sub-arrays, with 30 solar panels each, fill about 0.6 acres, leaving room to expand the solar generation capacity over time. The 1,050 panels were adhered to the exposed geomembrane over a 5.6-acre project area, with flat areas (benches) separating the tiers. The panels are positioned parallel to final-grade contours with sideslopes angled about 15°. These panels were adhered to the geomembrane with an ethylene propylene copolymer designed for use on both the solar panels and the geomembrane surface. The Tessman Road Landfill Solar Energy Cover project is a good example of sustainable investment, with a high benefit-to-cost ratio, relatively low risk and increased energy efficiency.

4 CASE 3: INGENUITY IN UNSATURATED SOIL COVER DESIGN

4.1 *Some Concerns in the Design of Unsaturated Soil Cover Systems*

Resistive cover systems involve a liner (*e.g.* a compacted clay layer) constructed with a low saturated hydraulic conductivity soil (typically 10^{-9} m/s or less) to reduce basal percolation. While US regulations require resistive covers, they also allow the use of alternative cover systems if comparative analyses and/or field demonstrations can satisfactorily show their equivalence with prescriptive systems. Unsaturated soil covers are alternative systems that have already been implemented in several high-profile sites. Evapotranspiration, unsaturated hydraulic conductivity and water storage are parameters that significantly influence the performance of this system. The difficulty in adequately quantifying these important parameters has led to concerns regarding the long term performance of unsaturated soil covers. This has resulted in post-construction monitoring and in recommendations towards redundant measures such as additional capillary barrier systems.

4.2 *A Creative Approach using Geosynthetics: Geotextile Capillary Barriers*

The performance of evapotranspirative cover systems has been documented by field experimental studies (Anderson *et al.* 1993, Dwyer 1998), and procedures have been developed for quantitative evaluation of the variables governing their performance (Khire *et al.* 2000, Zornberg *et al.* 2003). However, recent studies have shown that the use of nonwoven geotextiles in a capillary barrier system provide superior performance than traditional coarse-grained soils (Zornberg *et al.* 2010).

The good performance of geotextiles as capillary barriers is shown in Fig. 3, which shows the water storage within a clay soil column as a function of time for columns involving geotextile and granular capillary barriers. This figure shows that the water storage increases as the infiltration front advances through the soil. Two values of water storage are shown as reference in the figure: the storage corresponding to a water content of 25% (the water content associated with free draining of the imposed impinging flow rate), and the water storage corresponding to saturated conditions. The water storage curves for Profile 1 (geosynthetic capillary barrier) and Profile 2 (granular capillary barrier) indicate that the clay stores water well in excess of the value expected from a freely-draining condition. Also, the results show that the geosynthetic capillary barrier outperformed the granular capillary barrier. In summary, geotextile capillary barriers provide higher water storage than granular soils. In addition, they also offer separation and filtration benefits that are necessary for a good long-term performance of capillary barriers involving granular soils.

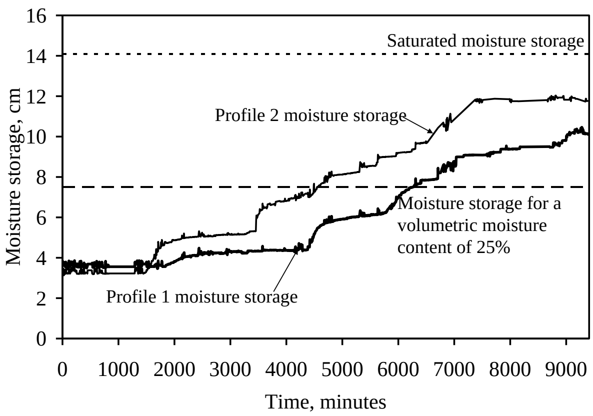

Fig. 3. Water storage in columns with geotextile (Profile 1) and granular (Profile 2) capillary barriers (McCartney *et al.* 2005)

4.3 *A Recent Project: The Rocky Mountain Arsenal, CO*

The Rocky Mountain Arsenal (RMA) is a Superfund site located near Denver (Colorado) that corresponds to one of the most highly contaminated hazardous waste sites in the US. One of the remediation components at the site involved the design and construction of alternative covers. The project includes over 400 acres of alternative covers. The climate in Denver is semiarid, with an average annual precipitation of 396 mm and an average pan evaporation of 1,394 mm. The wettest months of the year are also the months with the highest pan evaporation, which is appropriate for an evapotranspirative cover. The Record of Decision (ROD) for this hazardous waste site required a compliance demonstration to show equivalence of the alternative design with a prescriptive cover before construction of the final covers. The design and compliance of the covers at the RMA site are governed by a quantitative percolation criterion involving a threshold of 1.3 mm/year.

The compliance demonstration at the Rocky Mountain Arsenal involved a field demonstration, which was complemented with comparative numerical analyses (Kiel *et al.* 2002). Four evapotranspirative test covers were constructed on a rolling plain at the site in the summer of 1998. The instrumentation program involved monitoring of the basal percolation, precipitation, soil volumetric water content, and overland runoff in the four test covers. Basal percolation was collected in gravity lysimeters, which involved a geocomposite underlain by a geomembrane. Rain and snow were monitored using an all-season rain gauge. Surface water was collected in polyethylene geomembrane swales constructed around the cover perimeters. Water content reflectometer (WCR) probes were used to measure volumetric water content profiles.

While the test plots were well instrumented, the equivalent demonstration process initially focused almost exclusively on the lysimeter measurements. This was because the goal was that the water flux through site-specific soils under local weather conditions remains below the threshold of 1.3 mm/year. According to the lysimeter measurements, all test plots at RMA satisfied the quantitative percolation criterion over the period 1998-2003 of operation. However, subsequent evaluation of the water content records revealed that the presence of lysimeters had affected the flow of water due to the creation of a capillary barrier in the lysimeters. Even though this effect was not initially identified, the cover design was amended to include a capillary barrier.

The final cover design for the first group of alternative covers constructed at RMA is shown in Fig. 4. As shown in the figure, the cover includes a geosynthetic capillary barrier (Williams *et al.* 2010). Specifically, the final design of the first cover constructed at the site includes a nonwoven geotextile over a chokestone layer (coarse gravel) to form a capillary break at the bottom interface of the barrier soil. The geotextile also helps minimizing the migration of soil particles into the chokestone layer. The chokestone is underlain by a biotic barrier consisting of crushed concrete from a demolition site. The performance of the final cover is currently being monitored. It may be concluded that geosynthetic capillary barrier may act as an essential component that contributes to the adequate performance of the system.

(a)

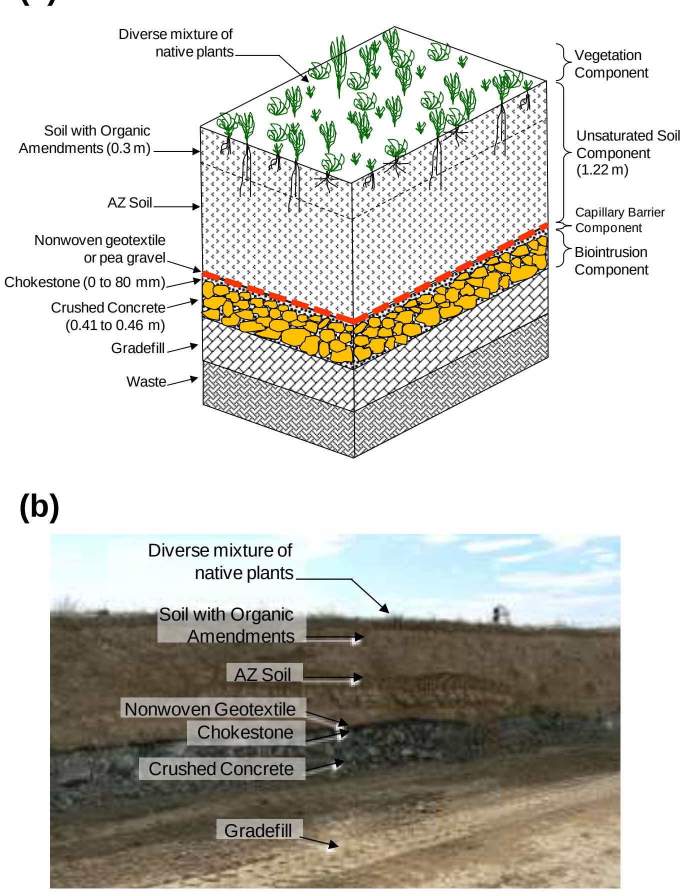

Fig. 4: Cross-section of the RMA Covers: (a) Schematic View; (b) Exposed cut in Shell Cover (Williams *et al.* 2010)

5 CASE 4: INGENUITY IN VENEER DESIGN

5.1 *Difficulties in the Design of Veneer Slopes*

The design of veneer slopes (*e.g.* steep cover systems for waste containment facilities) may pose significant challenges to designers. Considering the normal and shear forces acting in a control volume along the veneer slope (or infinite slope), and assuming a Mohr-Coulomb shear strength envelope, the classic expression for the factor of safety FS_u of an unreinforced veneer can be obtained as a function of the soil shear strength parameters. However, if the slope is comparatively steep or the veneer is comparatively thick, the designer is left with little options to enhance stability.

5.2 *A Creative Solution by using Geosynthetics: Anchored Reinforcements*

Geosynthetic reinforcement has been used as an alternative to stabilize veneer slopes. However, cases involving high, steep slopes lead to tensile requirements that are too high and for which reinforcement products do not exist in the market. A comparatively recent alternative involved the use of horizontal geosynthetic reinforcements, anchored in sound material underlying the soil veneer (Zornberg *et al.* 2001). In this case, the shear and normal forces acting on the control volume are defined not only as a function of the weight of the control volume, but also as a function of the tensile forces that develop within the reinforcements. In this case, the shear and normal forces needed for equilibrium of a control volume are defined by a formulation that depends on the tensile strength of the reinforcement and provides a convenient expression for stability evaluation of reinforced veneer slopes. Additional aspects that should be accounted for in the design of reinforced veneer slopes include the evaluation of the pullout resistance (*i.e.* embedment length into the underlying mass), assessment of the factor of safety for surfaces that get partially into the underlying mass, evaluation of reinforcement vertical spacing, and analysis of seismic stability.

5.3 *A Recent Project: North Slopes at the OII Superfund Site*

A cover reinforced using horizontally placed geogrids was constructed as part of the final closure of the Operating Industries, Inc. (OII) landfill. In 1986, the 60-hectare south parcel of the OII landfill was placed on the National Priorities List of Superfund sites. Beginning in 1996, the design of a final cover system consisting of an alternative evapotranspirative soil cover was initiated, with construction carried out subsequently from 1997 to 2000. Stability criteria required a static factor of safety of 1.5, and acceptable seismically-induced permanent deformations less than 150 mm under the maximum credible earthquake.

One of the most challenging design and construction features of the project was satisfying stability requirements for the North Slope of the landfill. The North Slope is located immediately adjacent to the heavily travelled Pomona freeway (over a distance of about 1400 m), rises up to 65 m above the freeway, and consisted of slope segments as steep as 1.5:1 (H:V) and up to 30 m high separated by narrow benches. The toe of the North Slope and the edge of refuse extends up to the freeway. The pre-existing cover on the North Slope consisted of varying thickness of non-engineered fill materials. The cover included several areas of sloughing instability, chronic cracking and high level of gas emissions. The slope was too steep to accommodate a layered final cover system incorporating geosynthetic components (*e.g.* geomembranes, GCLs).

After evaluating various alternatives, an evapotranspirative cover stabilized using geogrid reinforcements was selected as the appropriate cover for the North Slope (Fig. 5). Stability analyses showed that for most available evapotranspirative materials, compacted to practically achievable levels of relative compaction on a 1.5:1 slope (*e.g.* 95% of Standard Proctor), the minimum static and seismic stability criteria were not met. Veneer geogrid reinforcement with horizontally placed geogrids was then selected as the most appropriate and cost-effective method to stabilize the North Slope cover. The veneer reinforcement consisted of polypropylene uniaxial geogrids, installed at 1.5-m vertical intervals for slopes steeper than 1.8:1, and at 3-m vertical intervals for slopes ranging from 2:1 to 1.8:1.

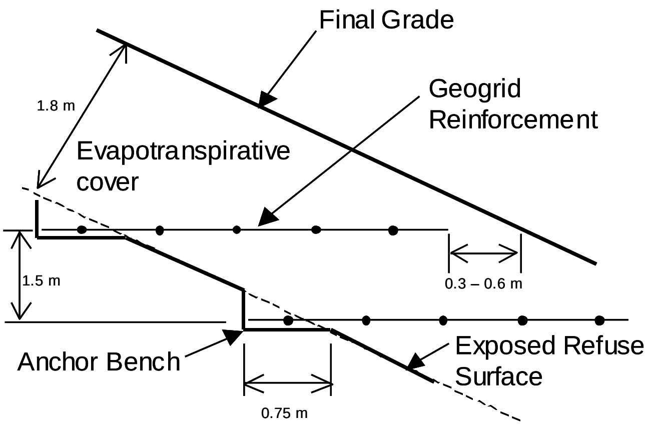

Fig. 5. Detail of the horizontal reinforcement anchored into solid waste (Zornberg *et al*. 2001a)

As shown in Fig. 5, the geogrid reinforcements are embedded a minimum of 0.75 m into the exposed refuse slope face from which the pre-existing cover had been stripped. Construction of the North Slope was accomplished in 12 months. Approximately 500,000 m^3 of soil and 170,000 m^2 of geogrid were placed, with a total area exceeding 9.3 hectares. The covers have shown good performance since its construction, illustrating that geosynthetic reinforcement led to a successful approach where many other stabilization alternatives were not feasible.

6 CASE 5: INGENUITY IN COASTAL PROTECTION SYSTEM DESIGN

6.1 *Some Concerns in the Design of Coastal Protection Systems*

Coastal protection is often achieved through rock armor, or riprap, which involves large rocks placed at the foot of dunes or cliffs. This approach is generally used in areas prone to erosion to absorb the wave energy and hold beach material. Although effective, this solution is unsightly and may be extremely expensive. Also, riprap may not be effective in storm conditions, and reduces the recreational value of beaches.

6.2 *A Creative Solution using Geosynthetics: Large Diameter Geotextile Tubes*

Coastal protection can be effectively achieved through the use of geotextile tubes (Lawson 2008). While geotextile tubes have been used for hydraulic and marine structures since the 1960s, the use of relatively large-diameter geotextile tubes is comparatively new. They involve the use of strong woven geotextiles as the tube skin (with no impermeable inner liner). The major advantage of this system is that a large encapsulated mass, a tubular structure, could be designed directly to meet many hydraulic and marine stability requirements. Geotextile tubes ranging in diameter from 1.0 m to 6.0 m have been used in hydraulic and marine applications.

Geotextile tubes are laid out and filled hydraulically on site to their required geometry. Hydraulic fill is pumped into the geotextile tube through specially manufactured filling ports located at specific intervals along the top of the tube. During filling, the tube, being permeable, allows the excess water to flow through the geotextile skin while the retained fill attains a compacted, stable mass within the tube. For hydraulic and marine applications, the type of fill typically used is sand or a significant fraction of sand. The reasons for this are that this type of fill can be placed to a good density by hydraulic means, it has good internal shear strength and, once placed, it does not undergo further consolidation that would change the filled shape of the geotextile tube (Lawson 2008).

The geotextile skin performs three functions that are critical to the performance of the filled geotextile tube. First, it should resist (with adequate tensile strength and stiffness) the mechanical stresses applied during filling and throughout the life of the units, and must not continue to deform over time. Second, it must have the required hydraulic properties to retain the sand fill and prevent erosion under a variety of hydraulic conditions. Finally, it must have adequate durability to maintain working conditions over the design life of the units.

6.3 *A Recent Project: Incheon Grand Bridge Project*

Geotextile tubes were recently used for the construction of an artificial island at Incheon Grand Bridge Project, Korea (Lawson 2008, IFAI 2011). The project includes the construction of a freeway connecting the island that holds the new airport to mainland Korea (Fig. 6). This bridge is the longest in Korea and the fifth-longest cable-stayed bridge in the world. An artificial island was planned in order to construct the freeway viaduct and associated toll gate facilities. This artificial island is to be left in place once the freeway viaduct is completed, as the area will later be enveloped by a large land reclamation scheme to build a new high-technology city.

Fig. 6. Geotextile tubes for the construction of an artificial island at Incheon Grand Bridge Project, Korea (IFAI 2011)

The foundation conditions where the artificial island is located consist of very soft marine clays to an approximate depth of 20 m. Also, the tide range in this area is high, with a maximum difference in level of 9.3 m. This results in exposure of the soft clay foundation at low tide and inundation to around 5 m at high tide. To address these difficulties, it was decided to construct a containment dike for the artificial island using geotextile tubes. This approach was selected over the alternative of using sheet-pile walls, considering the low shear strength of the soft foundation and the height to which the artificial island would have to be raised.

The sand fill for the geotextile tubes was brought to the site by barge, mixed with water, and then pumped hydraulically into the geotextile tubes. The base of the wall has two tubes side by side, with a third tube placed on top. A fourth tube was subsequently placed to heighten the final system. The performance of the geotextile tube structure used in this project was studied by Shin *et al.* (2008). The results show that the filled tubes underwent very little deformation once filled, confirming the adequacy of the geotextile tube system.

7 CASE 6: INGENUITY IN FOUNDATION DESIGN

7.1 *Some Difficulties in Foundations Design*

Foundations on very soft soils are always problematic. However, when the undrained shear strength is below some 15 kN/m^2, even solutions such as stone columns prove inadequate. This is because the horizontal support of the soft soil should at least be able to resist the horizontal pressures in the column.

7.2 *A Creative Approach using Geosynthetics: Geotextile-Encased Columns*

High strength geotextiles have been used to construct Geotextile Encased Columns (GEC), which may serve as foundation elements in very soft soils such as underconsolidated clays, peats, and sludge (Fig. 7). The columns involve the use of special geotextiles that encase granular material. The geotextiles provides radial support while the casing is strained by ring tensile forces (Raithel *et al.* 2005, Alexiew *et al.* 2011). The first projects involving this technology were successfully completed in Germany in the mid-1990s. Since their inception, over 30 successful projects have been completed in many countries including Germany, Sweden, Holland, Poland and Brazil.

Fig. 7. View of exposed Geotextile Encased Column (Alexiew *et al*. 2011)

Due to the presence of the geotextile casing, the soft soil can tolerate very low lateral support. This is because of the radial supporting effect of the geotextile casing, which depends in turn on the vertical pressure over the soft soil, which can be relatively small. To withstand the high ring tensile stresses, the geotextile casings are manufactured seamlessly. While the GECs also act as vertical drains, their main role is to transfer load to deep bearing layers. The GECs are arranged in a regular grid (Alexiew *et al.* 2011).

The vertical compressive stiffness of the GEC is lower than that of conventional deep foundation systems. Accordingly, the compacted vertical sand or gravel column settles under load due to radial outward deformations. The geosynthetic encasement, and to some extent the surrounding soft soil, provides a confining radial inward resistance, but some radial deformability is allowed. This deformability has been reported to provide better compatibility with the deformation of soft sols than more rigid systems. The use of geosynthetic reinforcements placed horizontally on top of the GECs (*e.g.* at the base of embankments founded using GECs) has also been used to reduce differential settlements between the columns and the surrounding soil.

7.3 *A Recent Project: Extension of Dockyards for the new Airbus, Germany*

Geotextile Encased Columns were used as part of the extension of the airplane dockyards in Hamburg-Finkenwerder for the production of the new Airbus A380. The area extension was conducted by enclosing a polder with a 2.4 km long dike, which was subsequently filled to provide an additional area of 140 ha. The main problem facing this project was the construction in very soft soils (undrained shear strength ranging from 0.4 to 10 kPa), with thicknesses ranging from 8 to 14 m. The original design involved the construction of a 2.5 km long sheet pile wall, driven to a depth of 40 m. Ultimately, a dike was constructed over a foundation involving installing approximately 60,000 GECs with a diameter of 80 cm. They were sunk into the bearing layers to a depth ranging between 4 and 14 m below the base of the dike footing. This dike is the new main water protection for the airplane dockyard.

This project was successfully implemented between 2001 and 2004. As part of the structural checks on the ground engineering concept, the stability and deformation predictions were verified by on-site measurements during construction. The comprehensive instrumentation included horizontal and vertical inclinometers, settlement indicators and measurement marks, as well as water pressure and pore water pressure transducers. Most of the measurement instrumentation was designed for continued monitoring after completion of the dike.

The dike surface was added to offset long-term settlement when much of the primary settlements were practically complete (after roughly one year). Additional predictions were conducted to estimate secondary settlements. An evaluation conducted in 2004 revealed significantly lower secondary settlements than initially predicted, confirming the soundness of the design involving GECs.

8 CASE 7: INGENUITY IN BRIDGE ABUTMENT DESIGN

8.1 *Some Difficulties in the Design of Bridge Abutments*

Conventional design of bridge abutments involve the use of a foundation approach to support the bridge (*e.g.* using a deep foundation) and a different type of foundation for the approaching roadway structure (*e.g.* foundations on grade). The use of two different foundation types for different components of the abutment has led to increased construction costs and times. In addition, vehicular traffic may not be smooth due to the development of a “bump at the bridge” caused by differential settlements between bridge foundations and approaching roadway structures.

8.2 *A Creative Approach using Geosynthetics: GRS Integral Abutments*

A comparatively recent approach involves the use of integral Geosynthetic Reinforced Soil (GRS) abutments, which support the bridge load by footings placed directly on a geosynthetic-reinforced wall, eliminating the use of traditional deep foundations altogether (Zornberg *et al.* 2001b, Keller and Devin 2003, Wu *et al.* 2006). Some additional advantages include their flexibility, and consequently added ability to withstand differential settlements and seismic loads as well as their ability to alleviate the bridge “bumps” commonly occurring at the two ends of the bridge. In addition, this approach eliminates the need of excavations specialized drilling equipment needed for deep foundations, leading to comparatively rapid construction.

8.3 *A Recent Project: Founders/Meadows Parkway Bridge, CO*

A GRS abutment for bridge support, the Founders/Meadows Parkway Bridge, was constructed on I-25, approximately 20 miles south of downtown Denver, CO. This was the first major bridge in the US built on footings supported by a geosynthetic-reinforced system, eliminating the use of traditional deep foundations altogether (Fig. 8). Phased construction of the almost 9-m high, horseshoe-shaped abutments, located on each side of the highway, began in July 1998 and was completed after only twelve months.

A comprehensive material testing, instrumentation, and monitoring programs were incorporated into the construction operations. Design procedures, material characterization programs, and monitoring results from the instrumentation program are discussed by Abu-Hejleh *et al.* (2002). Each span of the new bridge is 34.5 m long and 34.5 m wide, with 20 side-by-side pre-stressed box girders. The new bridge is 13 m longer and 25 m wider than the previous structure, accommodating six traffic lanes and sidewalks on both sides of the bridge. The bridge is supported by central pier columns along the middle of the structure, which in turn are supported by a spread footings founded on bedrock at the median of U.S. Interstate 25. Three types of uniaxial geogrid reinforcements were used in different sections of the wall. The long-term-design-strength of the various reinforcement products used in this structure is 27 kN/m, 11 kN/m, and 6.8 kN/m, respectively.

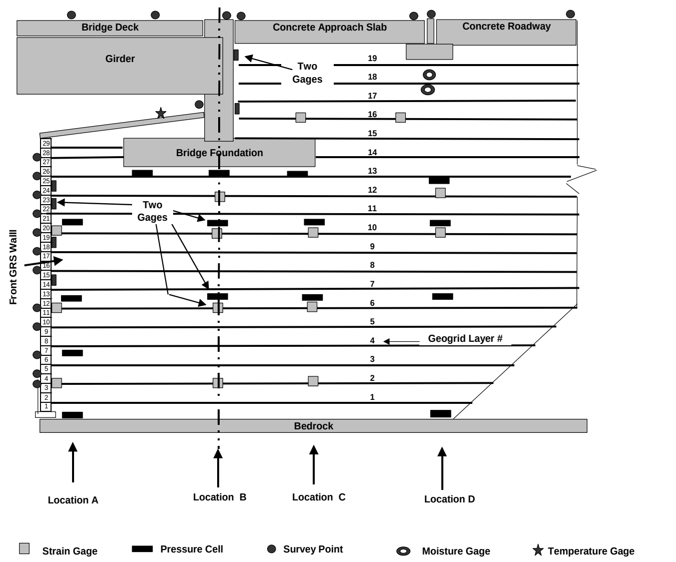

Fig. 8. Cross section of the Founders/Meadows Bridge abutment showing the geosynthetic reinforcements and instrumentation plan (Zornberg *et al*. 2001b)

Three sections of the GRS system were instrumented to provide information on the structure movements, soil stresses, geogrid strains, and moisture content during construction and after opening the structure to traffic. The instrumentation program included monitoring using survey targets, digital road profiler, pressure cells, strain gauges, moisture gauges, and temperature gauges. A view of the instrumentation plan for Phase II is also shown in Fig. 8. The figure shows the presence of the shallow footing resting on the reinforced soil mass.

Overall, the performance of the Founders/Meadows bridge structure, based on the monitored behavior, showed excellent short- and long-term performance. Specifically, the monitored movements were significantly smaller than those expected in design or allowed by performance requirements. Also, there were no signs of development of the “bump at the bridge” problem or associated structural distresses, and post-construction movements became negligible after an in-service period of 1 year.

9 CASE 8: INGENUITY IN THE DESIGN OF RETAINING WALLS

9.1 *Some Concerns in the Design of High Retaining Walls*

The flexibility of retaining walls is particularly relevant in the design of high (*e.g.* over 50 m) systems, when compared with conventional alternatives such as concrete retaining walls. This is important for an adequate long-term response, minimization of differential settlements, and achieving an adequate seismic behavior. In addition, the design of high structures using concrete retaining wall systems often requires deep foundations. Finally, and particularly in high walls, the time and cost requirements imposed by concrete retaining walls (*i.e.* formwork, placement of reinforcement bars, curing, removal of formwork) as well as technical limitations may be excessive.

9.2 *A Creative Solution using Geosynthetics: Optimized Flexible Wall Systems*

Geosynthetic-reinforced soil walls involve the use of continuous geosynthetic inclusions such as geogrids or geotextiles. The acceptance of geosynthetics in reinforced soil construction has been triggered by a number of factors, including aesthetics, reliability, simple construction techniques, good seismic performance, and the ability to tolerate large deformations without structural distress. Recent advances in the design of geosynthetic-reinforced walls and the availability of high strength products have led to systems that are particularly suitable for high walls. These systems include reinforcement systems with comparatively high tensile strength elements, low creep response, and flexible facing units.

9.3 *A Recent Project: Sikkim Airport, India*

An 80 m-high reinforced soil system has been recently constructed for the Sikkim Airport. The structure is a hybrid wall/slope system constructed in a very hilly road meandering along river Teesta, in the Himalayas region of India. This structure possibly constitutes the highest reinforced soil structure in the world built using geosynthetic reinforcements. Fig. 9 shows the front view and cross section of the recently constructed structure. The airport will provide connectivity to Gangtok, the capital of the state of Sikkim, which is nested in the Himalayas and remains often isolated during the rainy season. Site selection for an airport in this mountainous region required significant evaluation, as the airport’s runway and apron requires flat land due to operational considerations. The new airport will be able to handle ATR-72 class of aircrafts. Its runway is 1,700 m long and 30 m wide. Its apron will be able to park two ATR-72 aircrafts.

This innovative earth retention system involves the use of high strength geogrids as primary soil reinforcement with an ultimate tensile strength of 800 kN/m. The reinforcement vertical spacing ranges from 1.8 to 2.4 m. In addition, galvanized and PVC-coated wire mesh panels are used as secondary reinforcement (spaced every 0.6 m). A vegetated slope face was constructed in a significant portion of the reinforced soil system by installing tailored units as fascia elements.

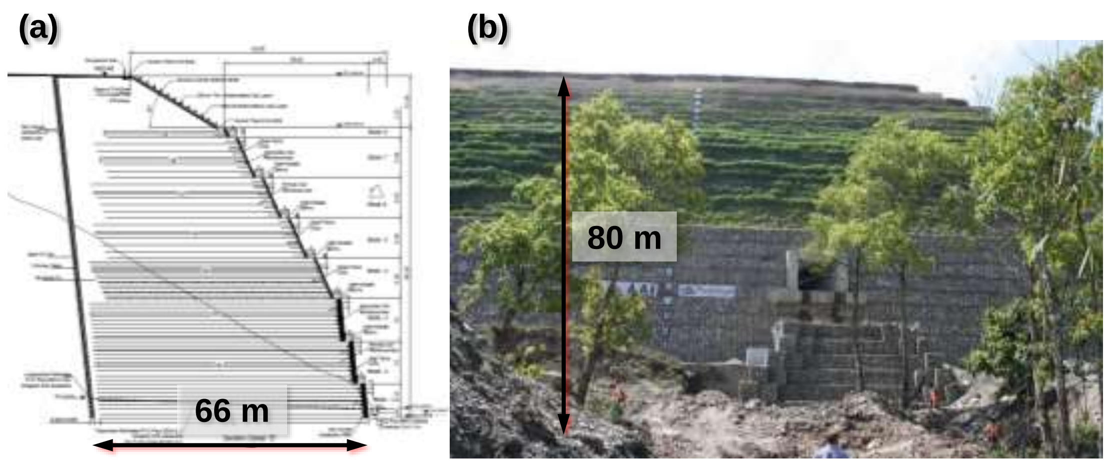

Fig. 9. High (80 m) geosynthetic-reinforced wall constructed for the Sikkim Airport: a) Cross-section, b) Front view (Zannoni 2011)

Seismic considerations played a significant role in the selection of the wall system. Indeed, the structure experienced a magnitude 6.8 earthquake during construction, with no signs of visible distress after the event. In addition, the selected system was able to accommodate well the available bearing capacity for the foundation soils. Finally, environmental considerations such as the reduced carbon footprint of this alternative in relation to those involving a concrete structure added to the decision to select a geosynthetic-reinforced system. Locally available backfill materials were used throughout the project. Sikkim is in a green valley with rich flora and fauna. Accordingly, the selected reinforced soil structure, with local stone and green fascia, blends well with the surroundings causing minimum adverse effects on environment.

10 CASE 9: INGENUITY IN REINFORCED EMBANKMENT DESIGN

10.1 *Concerns in the Design of Earth Embankments*

If fine-grained soils constitute the available backfill material for an engineered embankment, the engineer is limited to the use of unreinforced systems and, consequently, comparatively flat slopes. This is because granular soils have been the recommended backfill material for reinforced soil construction due to their high shear strength and ability to minimize the development of excess pore water pressures. Stringent specifications regarding selection of granular backfill are provided, for example, by the FHWA guidelines (Berg *et al*. 2009).

10.2 *A Creative Solution using Geosynthetics: Reinforcements with In-Plane Drainage*

A promising approach for the design of reinforced fine-grained soils is to promote lateral drainage in combination with soil reinforcement. This may be achieved by using geocomposites with in-plane drainage capabilities or thin layers of granular soil in combination with the geosynthetic reinforcements. This design approach may even lead to the elimination of external drainage requirements. The potential use of permeable inclusions to reinforce poorly draining soils has been documented (Tatsuoka *et al.* 1990, Zornberg and Mitchell 1994, Mitchell and Zornberg 1995).

The potential benefits of using marginal soils to construct steepened slopes are significant and include: (i) reduced cost of structures that would otherwise be constructed with expensive select backfill; (ii) improved performance of compacted clay structures that would otherwise be constructed without reinforcements; and (iii) use of materials, such as nearly saturated cohesive soils and mine wastes, that would otherwise require disposal. However, the significant benefits of using poorly draining soils as backfill material can be realized only if a proper design accounts for the adverse conditions. The adverse conditions and preliminary guidance are identified by Christopher *et al.* (1998) for the design of steep slopes using fine-grained soils.

10.3 *The Recent Re-evaluation of a Pioneering Project: Geotextile-Reinforced slope in Idaho National Forest*

A geotextile-reinforced slope designed as part of the widening of US Highway 93 between Salmon, Idaho, and the Montana state line (Barrows and Lofgren 1993). The reinforced structure is a 1H:1V slope located in Idaho's Salmon National Forest along Highway 93. Esthetics was an important consideration in the selection of the retaining structures along scenic Highway 93 (Parfit 1992). The 172 m-long and up to 15.3 m-high geotextile-reinforced slope is vegetated, causing a minimum environmental impact to the Salmon National Forest.

The slope was designed using geotextile reinforcements that not only were required to have adequate tensile strength but were also expected to provide appropriate in-plane drainage capacity to allow dissipation of pore water pressures that could be generated in the fill. In this way, an additional drainage system was not necessary even though indigenous soils were used as backfill and groundwater seeping was expected from the excavation behind the fill. An extensive instrumentation program was implemented to evaluate its performance.

On-site soil coming from excavation of the road alignment was to be used as backfill material. Subsurface drilling revealed that the majority of subsurface material on this project is decomposed granite. Although the project specifications required the use of material with no more than 15% passing sieve no. 200, internal drainage was a design concern. This was because of the potential seepage from the fractured rock mass into the reinforced fill, especially during spring thaw, coupled with the potential crushing of decomposed granite particles that may reduce the hydraulic conductivity of the fill. Widening of the original road was achieved by turning the existing 2H:1V unreinforced slope into a 1H:1V reinforced slope.

As shown in Fig. 10, the final design adopted two geosynthetic reinforced zones with a constant reinforcement spacing of 0.3 m (1 ft). At the highest cross-section of the structure, the reinforced slope has a total of 50 geotextile layers. A nonwoven geotextile was selected in the upper half of the slope, while a high strength composite geotextile was used in the lower half. The nonwoven geotextile, with an ultimate tensile strength over 20 kN/m, is a polypropylene continuous filament needle punched nonwoven. The composite geotextile, with an ultimate tensile strength over 100 kN/m, is a polypropylene continuous filament nonwoven geotextile reinforced by a biaxial network of high-modulus yarns.

Fig. 10. Geosynthetic-reinforced slope in the Idaho National Forest, illustrating the use of reinforcement with in-plane drainage capabilities (Zornberg *et al*. 1997)

The maximum geotextile strains observed during construction and up to eight weeks following the completion of slope construction are on the order of 0.2%. These are significantly low strain levels, mainly if we consider that extensometers report global strains, comparable with the soil strains obtained from inclinometer readings. The project was revisited in 2010, 17 years after its construction, in order to evaluate its post-construction behavior. The maximum strain in the geotextiles was measured to be only of 0.4%, that is, only 0.2% additional time-dependent strain. It is also possible that the post-construction reinforcement strains occurred due to settlement within the backfill material. The time-dependent strain behavior was found to be approximately log-linear.

Another means to evaluate the performance of the geotextile-reinforced embankment involved evaluation to determine the pavement condition index and the pavement condition rating. To provide a basis for comparison, two other pavement evaluations were conducted on earth structures of similar height in the same highway. Among the various retaining wall systems in the project, the pavement over the geosynthetic-reinforced slope was found to be the one with the highest pavement condition rating.

11 CASE 10: INGENUITY IN PAVEMENT DESIGN

11.1 *Concerns in the Design of Pavements over Expansive Clays*

The construction of pavements over expansive clay has often led to poor performance due to development of longitudinal cracks induced by moisture fluctuations. These environmental conditions are generally not fully evaluated as part of the design of pavements, which often focuses only on traffic loading conditions. Yet, volumetric changes associated with seasonal moisture variations have led to pavement heave during wet season and shrinkage during dry season.

The mechanisms leading to the development of the classical longitudinal cracks are expected to be due to tensile stresses induced by flexion of the pavement during settlements occurred in dry seasons. During the dry season, there is decrease in the moisture content of the soil in the vicinity of the pavement shoulders. This leads to settlements in the shoulder area, but not in the vicinity of the central line of the pavement, where the moisture content remains approximately constant throughout the dry season. On the other hand, during the wet season, the moisture content in the soil in the vicinity of the pavement shoulder increases.

11.2 *A Creative Solution using Geosynthetics: Base Reinforced Pavements on Expansive Clays*

Base reinforcement involves placing a geosynthetic at the bottom or within a base course to increase the structural or load-carrying capacity of a pavement system. Two traditional benefits are reported for reinforced pavements: (1) improvement of the pavement service life, and (2) equivalent pavement performance with a reduced structural section. Studies have been conducted to quantify the effectiveness of geogrids in pavements (Al-Qadi 1997, Perkins and Ismeik 1997, Zornberg and Gupta 2010). While field observations point to the good performance of geosynthetic-reinforced pavements, the actual properties governing the contribution of geosynthetics to the pavement reinforcement have not been clearly identified. A new application of basal reinforcement of pavements has been used in Texas with the purpose of mitigating the development of longitudinal cracks in pavements over expansive clays.

11.3 *A Recent Project: Low Volume Road over Expansive Clays in Milam County, TX*

A project involving the use of geosynthetic reinforcements in a pavement over expansive clays is the reconstruction of FM 1915 located in Milam County, Texas. In 1996, an extensive network of longitudinal cracks was observed in over a 4 km stretch of the pavement section. Accordingly, the pavement was reconstructed with 0.25 m of lime treated subgrade and an asphalt seal coat on top. Due to the presence of expansive clays, a geogrid was placed at the interface between the base and subgrade. In order to evaluate the actual effect of the geogrid on the required base course thickness, two geogrid reinforced sections were constructed. The first section (Section 1) included a 0.20 m-thick base course, while the second section (Section 2) involved a 0.127 m-thick base course underlain by the same geogrid. In addition, a control (unreinforced) section was constructed with a 0.20 m-thick base course (Fig. 11).

While falling weight deflectometer testing was conducted to quantify the pavement performance, the clearest evaluation was obtained based on condition surveys and visual inspection of the pavement. Specifically, the control section was found to develop significant longitudinal cracks only after a few months of use. On other hand, the two geogrid-reinforced sections were found to perform well, without any evidence of longitudinal cracking. Fig. 11 also illustrates the extent of the three experimental sections and details the performance of the three sections. An important lesson can be learned from this field experience: geosynthetic reinforcements have prevented the development of longitudinal cracks over expansive clays while unreinforced sections have shown significant cracking.

12 CONCLUSIONS

Geosynthetics can now be considered a well-established technology within the portfolio of solutions available for geotechnical engineering projects. Yet, ingenuity continues to be significant in geotechnical projects that involve their use. This is probably because of the ability to tailor the mechanical and hydraulic properties in a controlled manner to satisfy the needs in all areas of geotechnical engineering. This paper discussed 10 (ten) recent applications or recent evaluations of old applications in geotechnical projects involving geosynthetics.

The discussion of each application identifies specific difficulties in geotechnical design, the creative use of geosynthetics to overcome the difficulties, and a specific case history illustrating the application. Specifically, this paper illustrates the merits of using geotextiles as filters in earth dams, the use of exposed geomembranes as a promising approach for resistive covers, the use of geotextiles as capillary barrier in unsaturated soil covers, the use of anchored geosynthetic reinforcements in stabilization of steep veneer slopes, the use of geotextile tubes for challenging coastal protection projects, the use of geotextile encased columns to stabilize very soft foundation soils, the use of integral geosynthetic-reinforced bridge abutments to minimize the “bump at the end of the bridge,” the use of geogrids in the design of the highest reinforced soil wall involving geosynthetics, the use of reinforcements with in-plane drainage capabilities in the design of steep slopes, and the use of geosynthetic reinforcements to mitigate the detrimental effect of expansive clays on pavements.

Overall, geosynthetics play an important role in all geotechnical applications because of their versatility, cost-effectiveness, ease of installation, and good characterization of their mechanical and hydraulic properties. The creative use of geosynthetics in geotechnical practice is likely to expand as manufacturers develop new and improved materials and as engineers/designers develop analysis routines for new applications.

ACKNOWLEDGMENTS

The author is thankful to the many designers whose ingenuity involving the use of geosynthetics led to the success of the ten geotechnical projects described in this paper. The author is also appreciative to Hossein Roodi, from the University of Texas at Austin, for his help with the review of this manuscript.

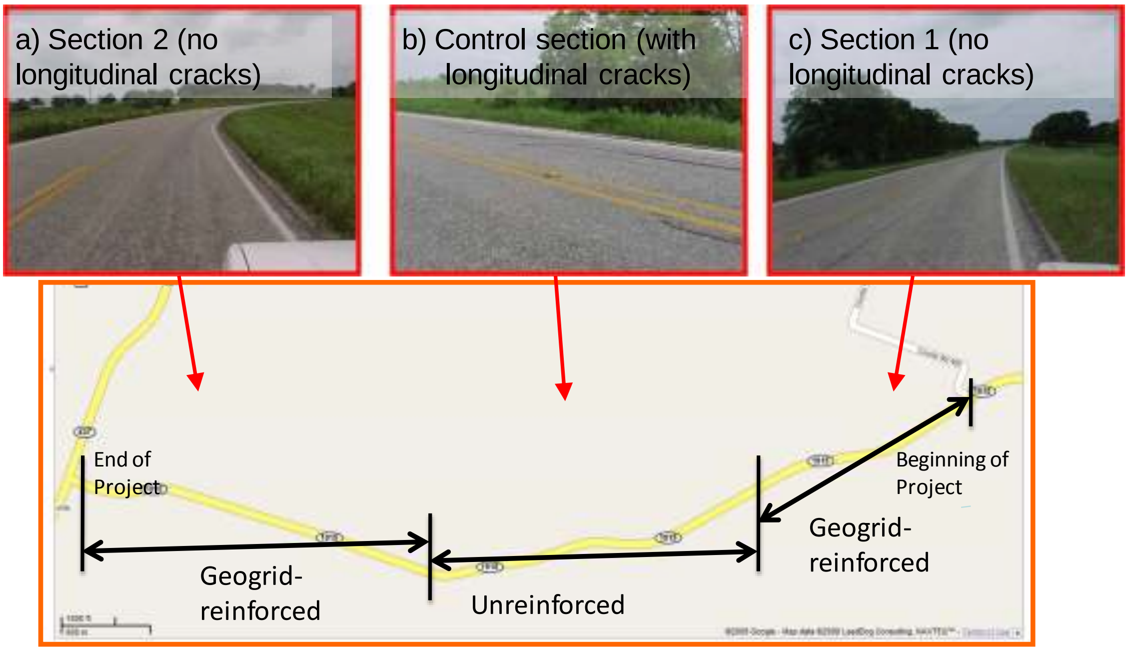

Fig. 11: Comparison of the performance of pavement sections over expansive clays: (a) Geogrid-reinforced Section 2; (b) unreinforced control section; (c) Geogrid-reinforced Section 1.

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