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Advances in Geosynthetics Materials and Applications for Soil Reinforcement and Environmental Protection Works

Ennio M. Palmeira
Professor, University of Brasília, Brasília, Brazil
palmeira@unb.br

Fumio Tatsuoka
Tokyo University of Science, Tokyo, Japan
tatsuoka@rs.noda.tus.ac.jp

Richard J. Bathurst
Professor, Royal Military College of Canada, Kingston, Canada
bathurst-r@rmc.ca

Peter E. Stevenson
Consulting Engineer, USA
Pete-geosyntheticssociety@charter.net

Jorge G. Zornberg
Professor, University of Texas at Austin, Austin, USA
zornberg@mail.utexas.edu

ABSTRACT

Geosynthetics have become well established construction materials for geotechnical and environmental applications in most parts of the world. Because they constitute manufactured materials, new products and applications are developed on a routine basis to provide solutions to routine and critical problems alike. Results from recent research and from monitoring of instrumented structures throughout the years have led to new design methods for different applications of geosynthetics. Because of the significant breath of geosynthetic applications, this paper focuses on recent advances on geosynthetics products, applications and design methodologies for reinforced soil and environmental protection works.

INTRODUCTION

Geosynthetics have been increasingly used in geotechnical and environmental engineering for the last 4 decades. Over the years, these products have helped designers and contractors to solve several types of engineering problems where the use of conventional construction materials would be restricted or considerably more expensive. There is a significant number of geosynthetic types and geosynthetic applications in geotechnical and environmental engineering. Due to space limitations,

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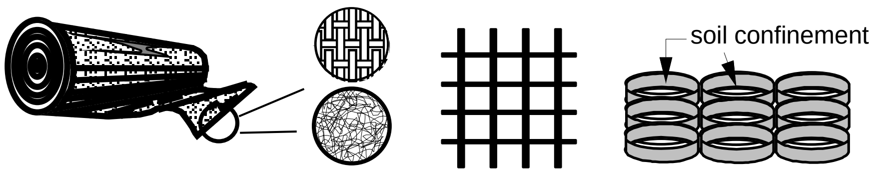
this paper will examine the advances on the use of these materials in reinforcement and in environmental protection.

Common types of geosynthetics used for soil reinforcement include geotextiles (particularly woven geotextiles), geogrids and geocells. Geotextiles (Figure 1a, Bathurst 2007) are continuous sheets of woven, nonwoven, knitted or stitch-bonded fibers or yarns. The sheets are flexible and permeable and generally have the appearance of a fabric. Geogrids have a uniformly distributed array of apertures between their longitudinal and transverse elements. These apertures allow direct contact between soil particles on either side of the sheet. Geocells are relatively thick, three-dimensional networks constructed from strips of polymeric sheet. The strips are joined together to form interconnected cells that are infilled with soil and sometimes concrete. In some cases 0.5 m to 1 m wide strips of polyolefin geogrids have been linked together with vertical polymeric rods used to form deep geocell layers called geomattresses.



(a) Geotextiles (b) Geogrids (c) Geocells

Figure 1: Geosynthetics commonly used for soil reinforcement (Bathurst 2007)

A wide variety of geosynthetics products can be used in environmental protection projects, including geomembranes, geosynthetic clay liners (GCL), geonets, geocomposites and geopipes. Geomembranes are continuous flexible sheets manufactured from one or more synthetic materials. They are relatively impermeable and are used as liners for fluid or gas containment and as vapour barriers. Geosynthetic clay liners (GCLs) are geocomposites that are prefabricated with a bentonite clay layer typically incorporated between a top and bottom geotextile layer or bonded to a geomembrane or single layer of geotextile. When hydrated they are effective as a barrier for liquid or gas and are commonly used in landfill liner applications often in conjunction with a geomembrane. Geonets are open grid-like materials formed by two sets of coarse, parallel, extruded polymeric strands intersecting at a constant acute angle. The network forms a sheet with in-plane porosity that is used to carry relatively large fluid or gas flows. Geocomposites are geosynthetics made from a combination of two or more geosynthetic types. Examples include: geotextile-geonet; geotextile-geogrid; geonet-geomembrane; or a geosynthetic clay liner (GCL). Geopipes are perforated or solid-wall polymeric pipes used for drainage of liquids or gas (including leachate or gas collection in landfill applications). In some cases, the perforated pipe is wrapped with a geotextile filter. Figure 2 presents schematically these products.

Because geosynthetics are manufactured materials, technological developments of the polymer and engineering plastics industries have been continuously incorporated in geosynthetics products, enhancing relevant engineering properties of these materials. Research results have also lead to the development of new and more powerful design and construction methods using geosynthetics. The combination of improved materials and design methods has made possible engineers to face challenges and to build structures under conditions that would be unthinkable in the past. This paper describes recent advances on geosynthetics and on the applications of these materials in soil reinforcement and in environmental protection projects.

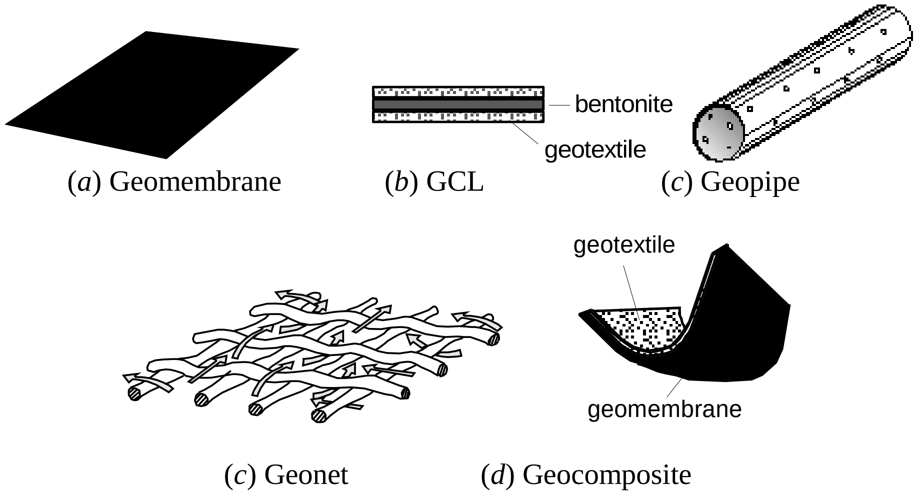


Figure 2: Schematic view of some typical geosynthetics used in environmental protection works (Bathurst 2007).

DEVELOPMENTS IN GEOSYNTHETICS MATERIALS TYPES AND APPLICATIONS

The axiom that there is nothing new under the sun regarding geosynthetics is simultaneously true and totally false. The truth is that the geotechnical problems that engineers use geosynthetics to solve are timeless: erosion, slope failure, poor bearing capacity etc. The products used to solve these problems could also be described as timeless as they derive from textile manufacturing techniques that date into antiquity. The falseness of this premise is revealed by the incremental advancements in the creation of geosynthetic solutions in the form of both product and geotechnical design. But what are the areas of incremental improvement in soil reinforcement and environmental applications? As the following capsules illustrate there is no end in sight for innovative application of geosynthetics.

For example, there are many developments in mechanically stabilized earth (MSE) walls and slopes and in basal stabilization. The MSE concept is essentially a uniaxial force problem and is served by the insertion of tensile members whose principal strength is uniaxial and that property is oriented to the expected forces of failure in the design. In 1993 a textile geogrid was employed using an ultra high strength polymer (the aramid known as Kevlar) to construct a road over karst terrain, as schematically shown in Figure 3. In 2001 a 15 meter wide sinkhole opened under the road which remained intact for more than one hour against a specification time of 15 minutes. Another textile geogrid application technology advance is the development of construction techniques that permit bridge abutments to be constructed where the sill beam rests directly on the GRS (geosynthetic reinforced soil) block while the GRS does not require a stiffening facing (Alexiew 2008). Textile geogrid reinforcement techniques are combined with other geosynthetic systems to build steep slopes on columns and piles, over geosynthetic encased stone columns and in piled embankments (Brokemper et al. 2006). Textile geogrid constructions mitigate landslides and debris flow and withstand storm surge exposure in a working platform. Yet another polymer, PVA, works in textile grid applications to withstand high alkali environments and especially the combination of lime and cement stabilizers and PVA grids in cohesive soils where there appears to be a synergistic effect resulting in higher strength and higher resistance to pullout failure (Aydogamus et al. 2006).

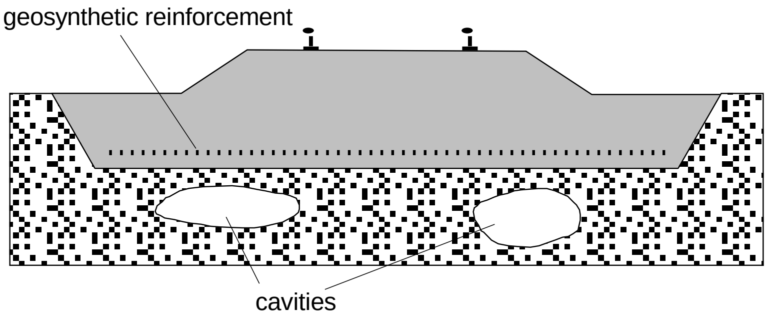


Figure 3: Reinforced embankment on unstable foundation soil.

Rigid grids have also experienced innovation with the development of new punching patterns that yield triangular shaped apertures after the stretching process. The new shape has several benefits in the product profile, rib thickness and in plane stiffness and this three dimensional structure is expected to offer improvement in confinement which will yield improved rut resistance and better load distribution (Tensar International 2008).

Soil reinforcement has seen the entry of a third type of geogrid, welded strapping (also described as strips or bars), which is rigid in structure. Produced in both polyester and polypropylene, the welded strapping grid is used in both uniaxial and biaxial applications. Properties of interest are strong junctions, excellent creep characteristics in the polyester form, and high chemical resistance. In the biaxial form two bars are employed in the cross machine direction giving a three dimensional structure to aid in confinement applications (Elias 2000).

Geogrids have been employed to resist or remediate reflective cracking in asphalt for many years (Fig. 4, Palmeira 2007). Nonetheless, innovation is present here in the continuing study and analysis of performance of these products. One claim is that bitumen coatings provide a superior bond to other polymers, enhancing grid performance in preventing crack propagation.

A three dimensional structure is a key to effective erosion control, what else is vegetation but a three dimensional structure that alters water flow characteristics? Efforts to impart three dimensional characteristics to erosion control products have been an important focus among manufacturers and one approach is embodied in the type of 3-D products developed by using combinations of yarns which have different shrinkage profiles. Products woven from these materials in two planes assume a three dimensional shape after exposure to heat in which some yarns contract in a controlled manner resulting in a three dimensional sheet (Propex 2007).

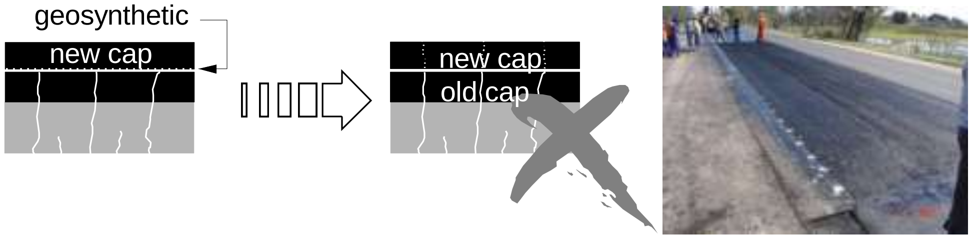


Figure 4: Geogrids to avoid reflective cracking in pavements (Palmeira 2007).

The three dimensional theme is carried forward in confinement applications by the development of three dimensional surfaces on geomembranes. Sliding failures, usually identified to occur at the interface between geomembrane and geotextile or geomembrane and soil, have been alleviated by the development of textured and embossed surfaces on geomembranes. Three dimensioned geomembranes, embossed surfaces for example, have consistent thickness, consistent asperity height and consistent properties and are easy to install and, most important, result in improved performance (better adhesion, better resistance to sliding) (Frobel 1996).

Electrokinetics and electroosmosis are techniques employed in manipulating pore pressure and plasticity indices of soils. Formerly hampered by difficulty in establishing suitable electrodes in soil structures, electrokinetics and eleectroosmosis are becoming viable technologies for soil reinforcement and environmental rehabilitation and geosynthetics are one of the means of introducing anodes and cathodes into a soil structure (Fig. 5), soil nailing is another. The concept of electrokinetics is the use of current to induce water flow. The technique can be used in environmental remediation wherein contaminants are recovered or removed from soil by causing groundwater to flow to a collection point. Anodes and cathodes are created from geosynthetics by using conductive materials such as carbon fiber, or by interlacing conductors (wire) in the textile. Other geosynthetic applications are mine tailing dewatering and sewage (perhaps contained in geotextile tubes) dewatering. Sports turf is managed by using current to draw off excess water, or by reversing polarity, delivering water to plant roots. The concepts of electrokinetics are applicable to slope stability, mechanically stabilized earth (walls), drainage and can result in cementation wherein ions precipitated from solution cement clays and the result is stiffer clays (Jones 2005).

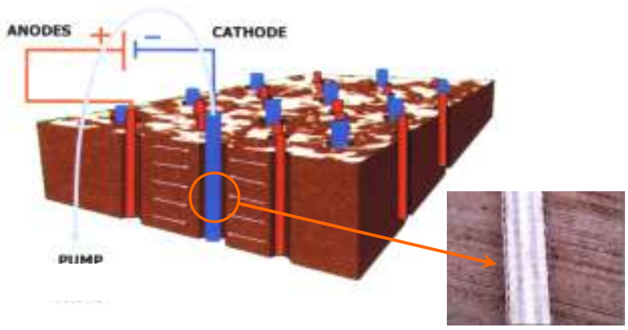


Figure 5: Electrokinetics geosynthetics for soft soil stabilisation (Jones et al. 2005).

Geocells have been used in innovative ways to stabilize aggregate while providing high volume drainage and working platform support. In an airport de-icing compound, the geocell confines the aggregate, improves the load capacity of the aggregate and the subgrade, contains large volumes of fluid in high volume events and drains fluid from the structure in a controlled manner. Another innovative use of geocells is as the facia on avalanche protection earthen mounds in Iceland (Bygness 2007). Five mile long barriers were raised 15 to 20 feet using multi layers of geocells with compacted soil filling as the facia resulting in an aesthetically pleasing alternative to conventional technique of concrete retaining walls.

Originating with applications in the containment industry, geosynthetic clay liners (GCL) continue to evolve in sophistication and improved performance (Fig. 6). In 1987, a patent was filed in Germany concerning a shear resistant mode of manufacture. This system used needled fiber to stabilize and strengthen the products structure. GCL applications continue to expand with applications as seals in substructures of earthen embankments, incorporation in hydraulic structures, and a host of additional applications. There are double layer GCL’s which give high assurance of desiccation proof impermeability in landfill caps. Composite structures of GCL’s and sand mats are produced for underwater installation. GCL’s are employed in waterproofing structures and the sealing of dam faces.

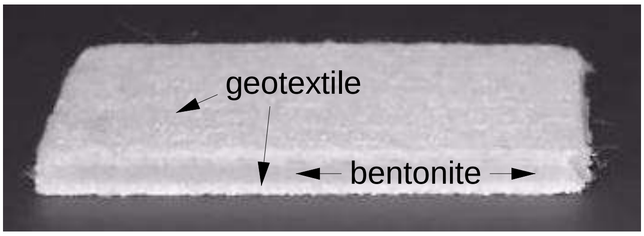


Figure 6: Typical example of a GCL (courtesy of M. Bouazza).

Construction on soft ground using geosynthetics is a well known theme that continues to evolve. As an example, a 16 meter high embankment was constructed over saturated soil in Germany employing a two layer system of 600 kN/m polyester fabric, with the result that a single layer of 1100 kN/m fabric is preferred due to non uniform loading of the two layers (Blume et al. 2006) . The construction scheme employed prefabricated vertical drains to assist in rapid dewatering. A different approach to construction on soft ground was used in Japan where a composite geotextile using a polyester fabric sheet (approximately 70 kN/m) combined with a pattern of woven textile tubes (714 kN/m) forming a lattice was installed and the tube lattice was then filled with pumped mortar (Yoshida et al. 2006). The result was greatly reduced settlements compared to the conventional construction on fabric over soft ground. Prefabricated vertical drains also benefit from innovation with improvement in composition and shape of the core as well as improvement in filter porosity resulting in greatly improved flow rates.

A very important aspect of innovation is the need for testing apparatus and procedures that reflect the product performance in situ and without undue influence. A simple example is the problem of tensile testing. In industry, testing is usually performed on a single unit (carbon fiber) with results extrapolated to a larger construct, perhaps an airframe. The geosynthetics industry has followed a different path in hopes of developing tests and tests methods which reflect properties developed in large areal applications. Tensile testing of geosynthetics has experienced apparatus testing one meter wide specimens, 8 inch samples, single ribs and individual yarns. Gripping devices include clamps, rollers, and devices that sense slippage and apply differential force to compensate. In every case slippage or perhaps more accurately, apparatus failure to avoid influencing results, is the problem causing the single rib method to differ from the wide width method which differs from yarn tests and strip tests. The use of grips which sense slippage in parts of a specimen and compensate, while expensive, are a major step in resolving the problems of apparatus influence on tensile data. Other testing developments include work to improve pull out testing apparatus, monotonic and cyclic loading evaluations, instrumentation studies and work in labs around the world to improve technique and equipment.

SOME ADVANCES IN SOIL REINFORCEMENT USING GEOSYNTHETICS

Advances in Soil Reinforcement in Asia

Construction of geosynthetic-reinforced soil retaining walls (GRS RW’s) and geosynthetic-reinforced steep slopes of embankments has become popular in Asia (e.g., Japan, Korea, China, Taiwan, Vietnam, Thailand, Singapore, Malaysia and India), following pioneering works in Europe and North America. Among the technologies used to construct these numerous geosynthetic-reinforced soil structures in Asia, a couple of unique ones that were developed in this region are reported herein.

GRS RWs Having a Full-Height Rigid Facing

Geosynthetic-reinforced soil retaining wall (GRS RW) having a stage-constructed full-height rigid (FHR) facing is now the standard retaining wall construction technology for railways in Japan (Tatsuoka et al., 1997a, 2007). Figure 7 shows a typical GRS RW having a FHR facing constructed in the center of Tokyo. This new type GRS RW has been constructed in more than 600 sites in Japan, and the total wall length is now more than 100 km as of March 2008. Very importantly, despite that railway engineers are generally very conservative in the structure design in civil engineering practice, the railway engineers in Japan have accepted this new type of retaining wall and this has become the standard retaining wall construction method for railways, including bullet trains.

This new retaining wall system has the following features:

The use of a full-height rigid (FHR) facing that is cast-in-place using staged construction procedures (Fig. 8). The geosynthetic reinforcement layers are firmly connected to the back of the facing. The importance of this connection for the wall stability is illustrated in Figure 9.

The use of a polymer geogrid reinforcement for cohesionless backfill to ensure good interlocking with the backfill, and the use of a composite of non-woven and woven geotextiles for nearly saturated cohesive soils to facilitate both drainage and tensile reinforcement of the backfill, which makes possible the use of low-quality on-site soil as the backfill if necessary.

The use of relatively short reinforcement.

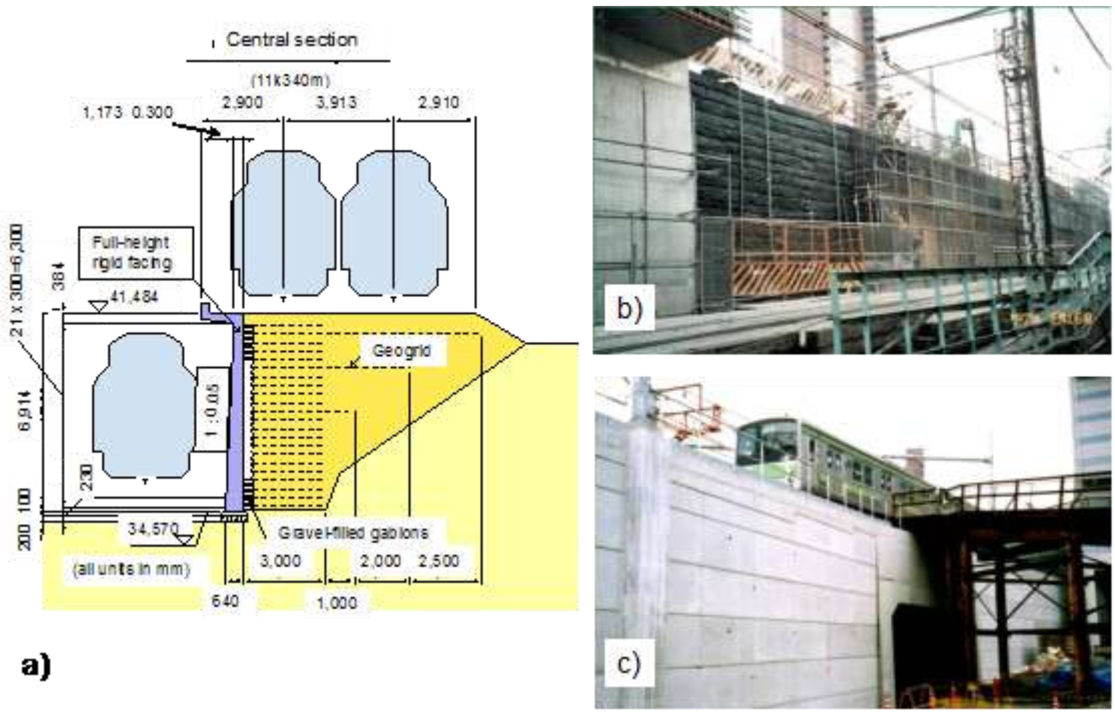


Figure 7: GRS RW having a FHR facing supporting one of the busiest rapid transits in Japan (Yamanote Line), near Shinjuku station, Tokyo (constructed during 1995 – 2000): a) typical cross-section; b) wall under construction; and b) completed wall

The staged construction method (Fig. 8), which is one of the main features of this RW system, consists of the following steps: 1) a small foundation element for the facing is constructed; 2) a full-height GRS wall with wrapped-around wall face is constructed by placing gravel-filled bags at the shoulder of each soil layer; and 3) a thin (i.e., 30 cm or more in the thickness) and lightly steel-reinforced concrete facing (i.e., a FHR facing) is constructed by cast-in-place fresh concrete directly on the wall face after the major part of ultimate deformation of the backfill and the subsoil layer beneath the wall has taken place. A good connection can be made between the RC facing and the main body of the wall by placing fresh concrete directly on the geogrid-covered wall face.

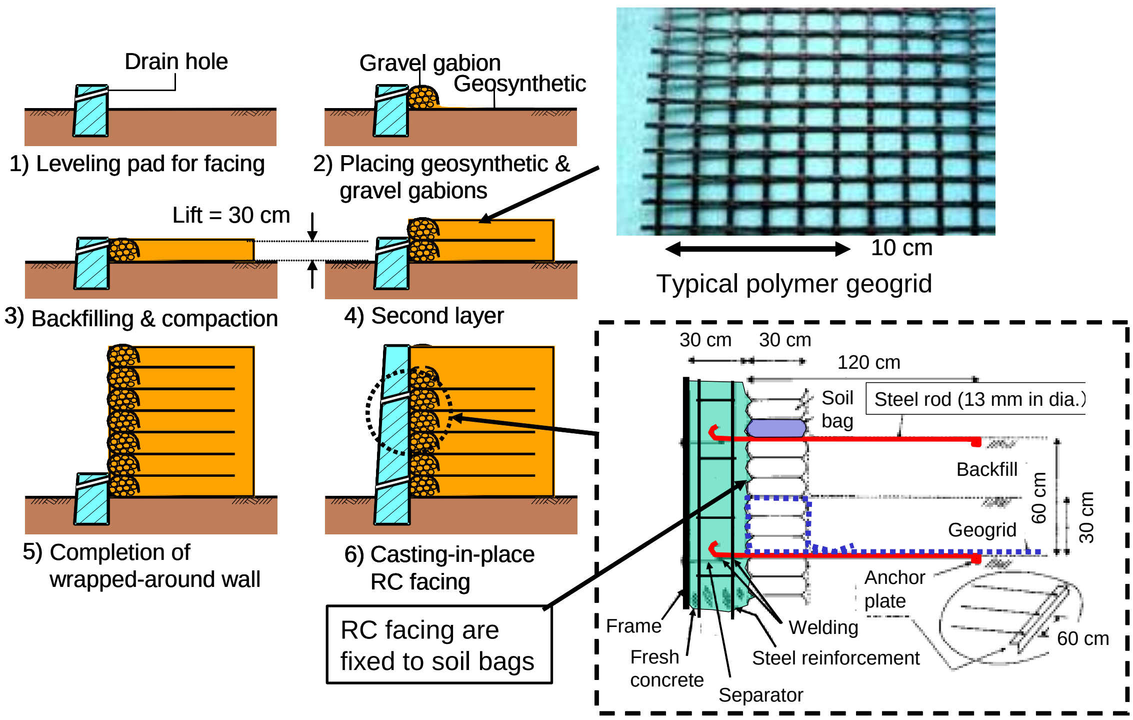


Figure 8: Staged construction of a GRW RW with a FHR facing.

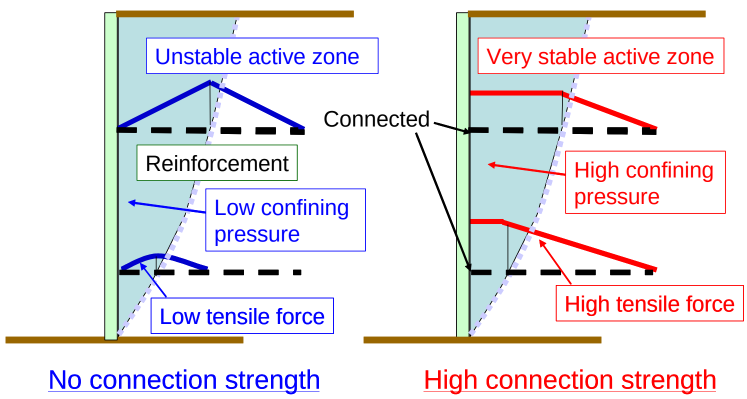


Figure 9: Effects of firm connection between the reinforcement and the facing (Tatsuoka, 1993).

The major structural feature of this new retaining wall is as follows. A conventional retaining wall type is basically a cantilever structure that resists against the active earth pressure from the unreinforced backfill by the moment and lateral thrust force activated at its base. Therefore, large internal moment and shear force are mobilized inside the facing structure while large overturning moment and lateral thrust force develop at the base of the wall structure. A large stress concentration may develop at and immediately behind the toe on the base of the wall structure, which makes necessary the use of a pile foundation in usual cases. Relatively large earth pressure, similar to the active earth pressure activated on the conventional retaining wall, may also be activated on the back of the FHR facing of GRS RW because of high connection strength between the reinforcement and the facing. This high earth pressure results in high confining pressures in the backfill, therefore high stiffness and strength of the backfill, which results in better performance than in the case without a firm connection between the reinforcement and the facing (Fig. 9) . As the FHR facing behaves as a continuous beam supported at a large number of points with a small span, typically 30 cm (Fig. 10), only small forces are activated inside the facing, resulting in a simple facing structure and insignificant overturning moment and lateral thrust forces activated at the bottom of facing, which makes unnecessary the use of a pile foundation in usual cases.

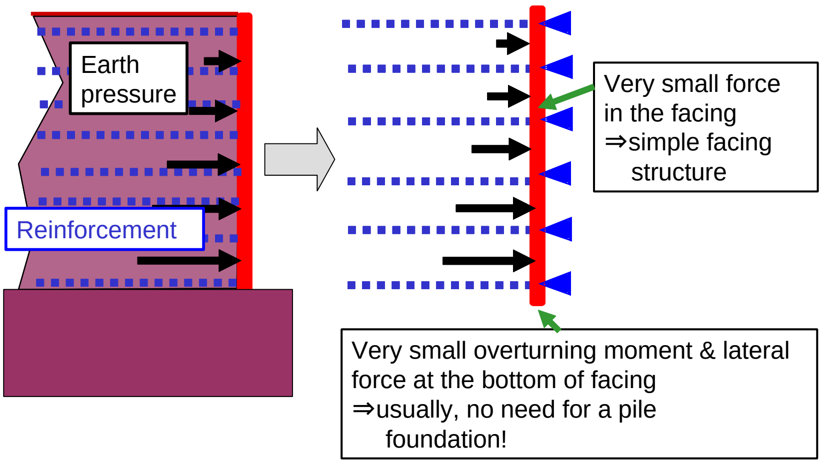


Figure 10: GRS RW with a FHR facing as a continuous beam supported at many points with a small span (Tatsuoka et al., 1997a).

A significant number of case histories until today have shown that the construction of GRS RW having a stage-constructed FHR facing is very cost-effective (i.e., much lower construction cost, a much higher construction speed and the use of much lighter construction machines), therefore a much less total emission of CO_2 than the construction of conventional types of retaining walls. Yet, the performance of the new type of retaining wall can be equivalent to, or even better than, that of conventional type soil retaining walls. The general trend of construction of elevated transportation structures in Japan is a gradual shifting from gentle-sloped embankments towards embankments supported with retaining walls (usually RC cantilever RWs with a pile foundation), or RC framed structures for higher ones, and then towards GRS RWs having a stage-constructed FHR facing (Fig. 11). It is expected that this new retaining wall technology is adopted and becomes popular in not only other countries than Japan in Asia but also many other countries outside Asia.

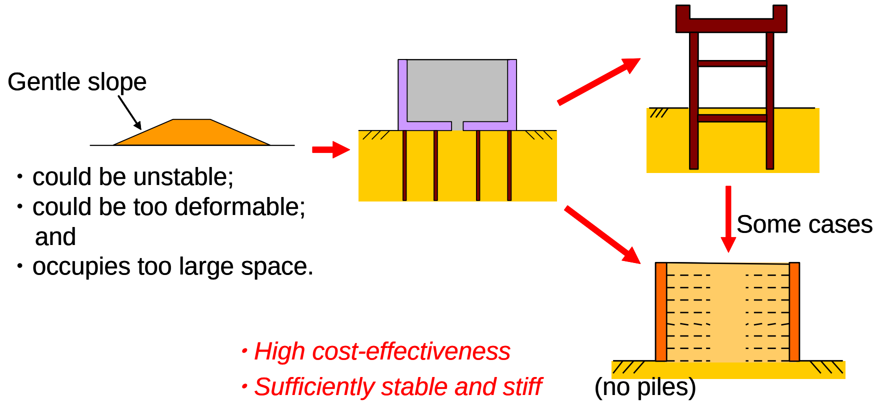


Figure 11: History of elevated railway and highway structures in Japan.

Reconstruction of Failed Embankments and Retaining Walls

Numerous embankments and conventional type retaining walls have collapsed due to flooding and earthquakes in the past in many Asian countries (Fig. 12). Previously, most of the collapsed soil structures were reconstructed to respective original structures despite that these conventional type soil structures have a substantially low cost-effectiveness with very low resistance against flooding and seismic loads. Since early 1990's, reconstruction of railway embankments that collapsed by flooding with embankments having geosynthetic-reinforced steep slopes or GRS RWs, having a stage-constructed FHR facing or their combination, started based on successful experiences of high cost-effectiveness and high performance of GRS RWs having a FHR facing, as described above. Figures 13(a) to (c) show a typical case of the above (Tatsuoka et al., 1997a; 2007). This reconstruction method was employed also in other similar cases after this event of flooding. It was after the 1995 Hyogo-ken-nambu Earthquake (the 1995 Kobe Earthquake) that gentle slopes of embankment and conventional retaining walls that collapsed by earthquakes were reconstructed using geosynthetic-reinforced steep slopes or GRS RWs having a stage-constructed FHR facing or their combination (Tatsuoka et al., 1996, 1977a & b, 1998). In particular, a very high performance of a GRS RW with a stage-constructed FHR facing at Tanata during the 1995 Kobe Earthquake validated a high-seismic stability of this wall type (Figs. 14a and b). Figures 15(a) to 15(c) show the reconstruction of one of the three railway embankments that totally failed during the 2004 Niigata-ken Chuetsu Earthquake using GRS RWs having a FHR facing. In this case, the new type of RW was chosen because of not only much lower construction cost and much higher stability (in particular for soil structures on a steep slope) but also a much shorter construction period and a significant reduction of earthwork when compared to reconstruction to the original embankments. The new type of reinforced wall is also much more cost-effective and needs a much shorter construction period than bridge type structures. During this earthquake, road embankments collapsed at numerous places in mountain areas and many of them were reconstructed using GRS RWs or embankments having geosynthetic-reinforced steep slopes. More recently, the March 25th 2007 Noto-hanto Earthquake caused severe damage to embankments of Noto Toll Road, which was opened in 1978. The north part of this road runs through a mountainous area for a length of 27 km. The damage concentrated into this part, where eleven high embankments filling valleys were extensively collapsed (Koseki et al., 2007). As schematically shown in Figure 16, the collapsed embankments were basically reconstructed using GRS RWs while ensuring the drainage of ground and surface water. The on-site soil that had originally been part of the collapsed embankment was re-used after lime-treatment for the construction of the upper fill.

Figure 12: Gravity type retaining wall without a pile foundation at Ishiyagawa that collapsed during the 1995 Kobe Earthquake (Tatsuoka et al., 1996, 1997b).

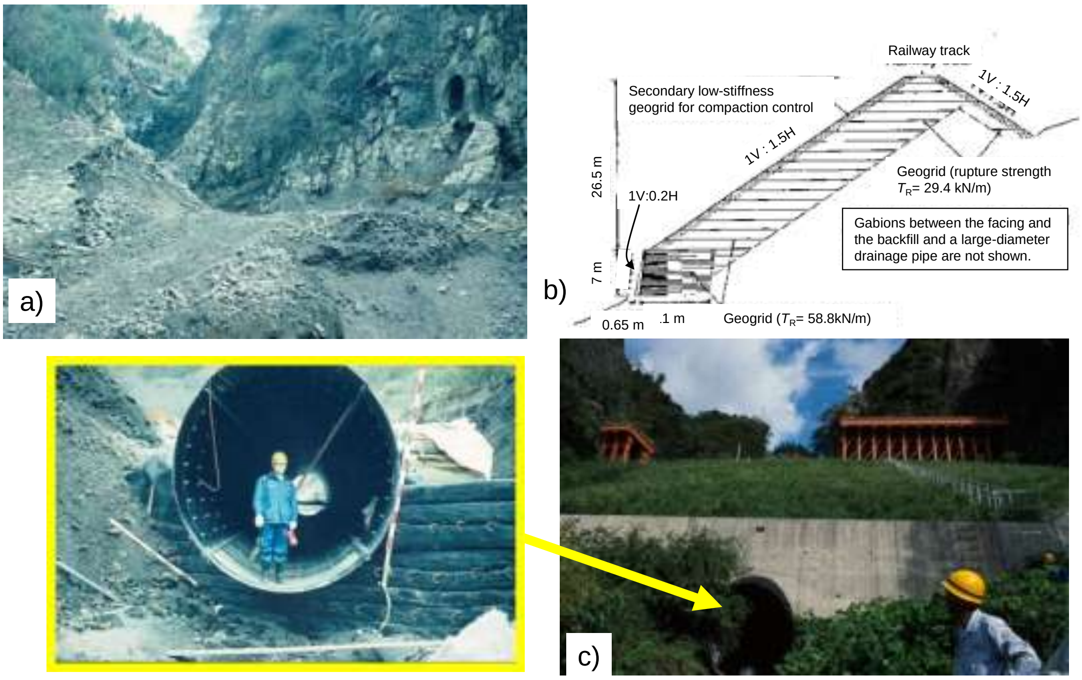


Figure 13: Typical section of a railway embankment damaged by rainfall in 1989 and reconstructed in 1991: a) before reconstruction; b) reconstructed cross-section; and c) after reconstruction (Tatsuoka et al., 1997a; 2007).

Figure 14: GRS RW having a FHR facing at Tanata, Japan; a) immediately wall completion; and b) immediately after the 1995 Kobe Earthquake (Tatsuoka et al., 1996, 1997b).

After a multiple successful case histories of geosynthetic-reinforced soil structures, as described above, when compared to two decades ago, GRS RWs and geosynthetic-reinforced steep slopes are now much more widely accepted as a relevant technology to reconstruct embankments and conventional retaining walls that have collapsed by floodings and earthquakes. This technology was also used to rehabilitate an old earth dam, having a crest length of 587 m and a height of 33.6 m, in the north of Tokyo (Fig. 17). When constructed about 80 years ago, this earth dam was the largest one in Japan.

The reservoir is exclusively for water supply in Tokyo, which will become extremely important in supplying water at the time of disasters, including seismic ones, because of its ability of sending raw water under gravity flow to several water treatment plants downstream. A 17 m-high counter-weight fill having a 1:1 steep slope was constructed on the down-stream slope of the dam aiming at a substantial increase in the seismic stability of the dam removing the possibility of vast disaster to a heavily populated residential area that had been developed in recent years close to the dam. Due to a severe space restriction, the slope of the counter-weight fill was very steep, which was possible by using HDPE geogrids installed over a total area of 28,500 m^2 in the fill.

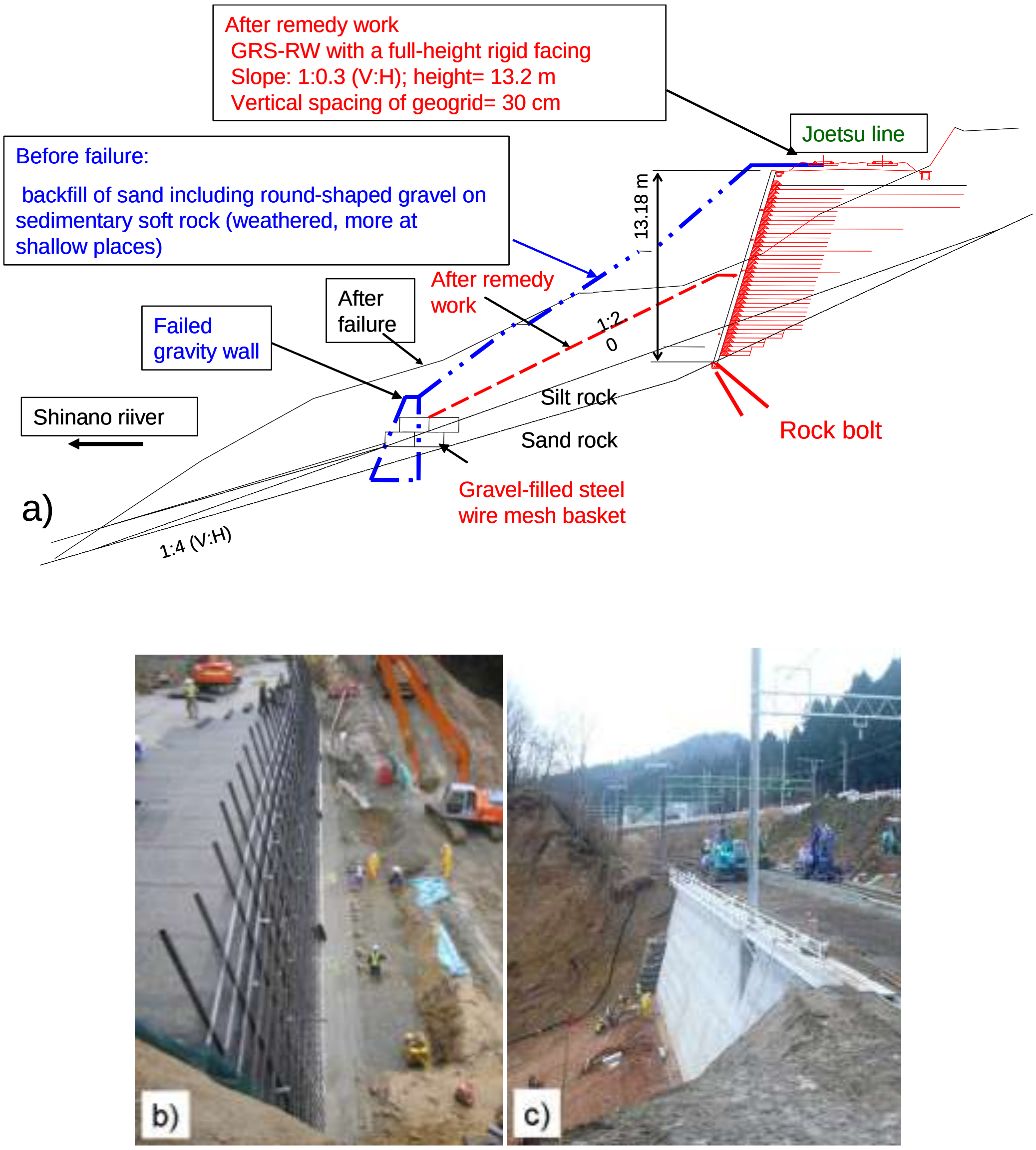


Figure 15: Railway embankment that collapsed during the 2004 Niigata-ken Chuetsu Earthquake and its reconstruction to a GRW RW having a FHR facing; a) cross-sections before and after failure and after reconstruction; b) wall during reconstruction; and c) completed wall (Morishima et al., 2005).

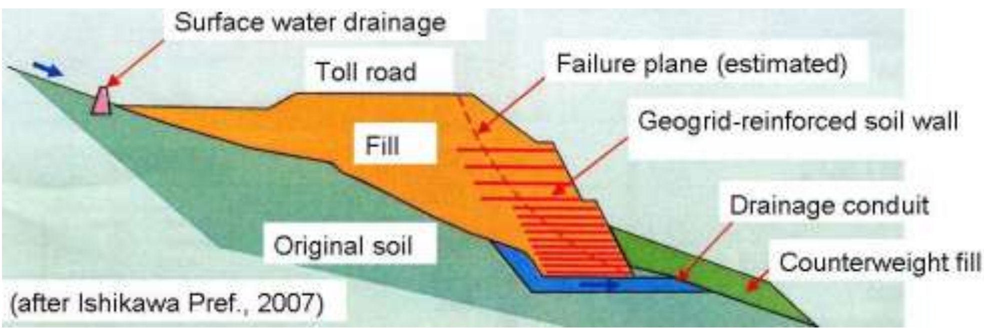


Figure 16: Schematic diagram showing reconstruction to GRS RWs of embankments damaged by the 2007 Noto hanto Earthquake (Koseki et al., 2007)

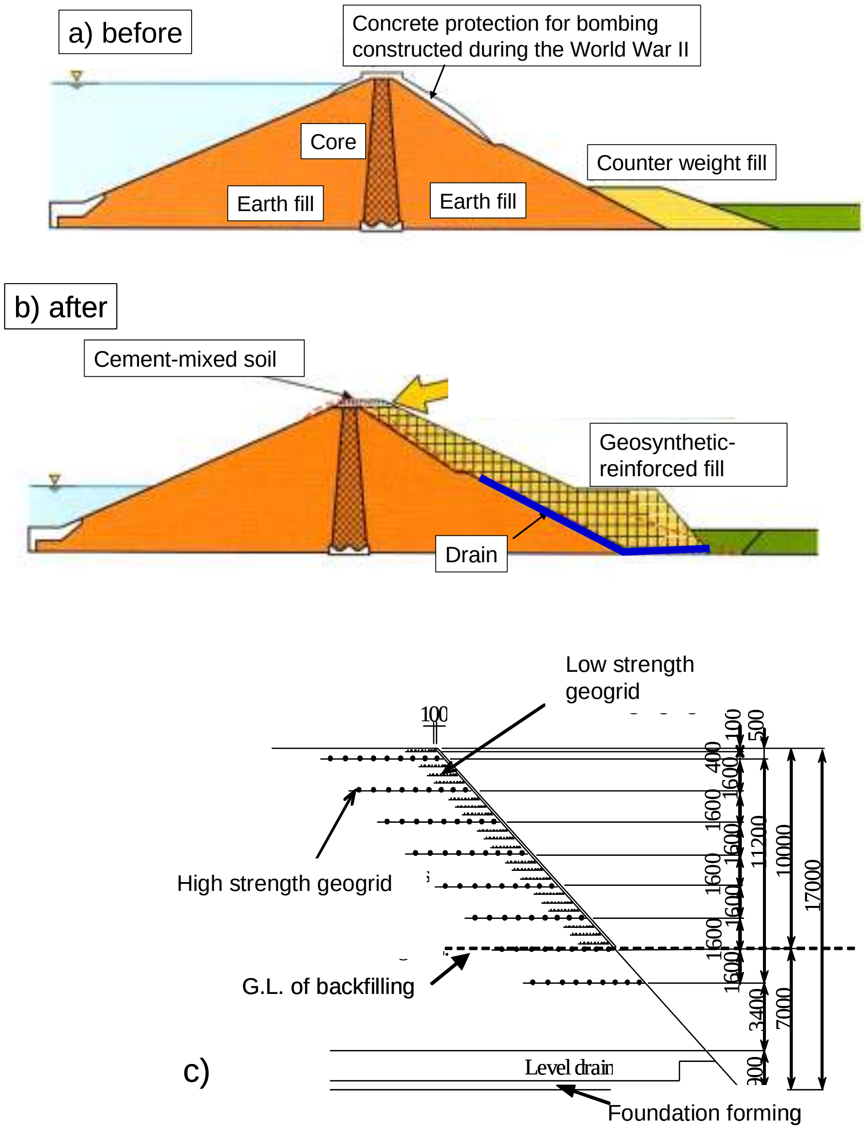


Figure 17: Shimo-Murayama dam in Tokyo: a) & b) dam before and after rehabilitation; and d) geogrid-reinforced counter-weight fill (Maruyama et al., 2006)

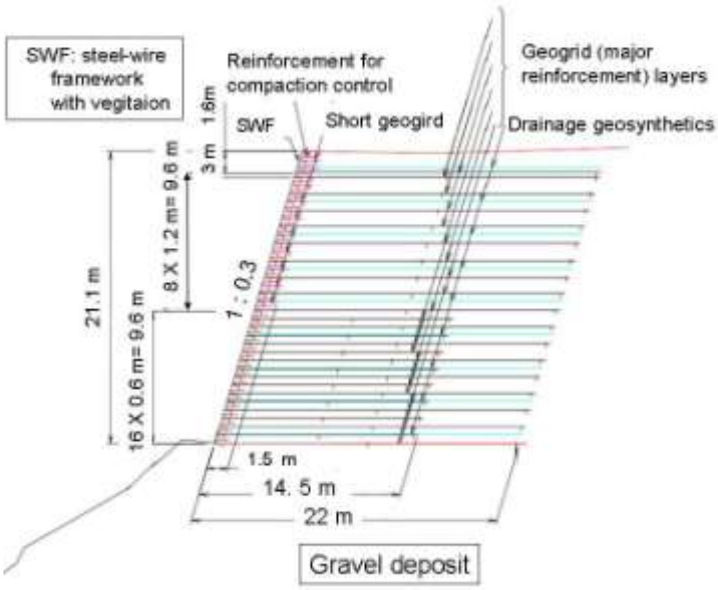
High Geogrid-Reinforced Wall

At the "Fujisan-Shizuoka Airport" in Japan, which is now under construction, two high GRS RWs (21.1 m and 16.7 m high) were constructed to preserve the natural environment, which consists of steep swamp areas, in front of the walls. These areas would be buried if gentle-sloped embankments were to be constructed (Fig. 18a). Figure 18b shows the cross-section of one of the two walls. As the walls support the east side of the airfield runway, minimum residual displacements at their crest are required. A sufficient high seismic stability is another important design issue. Well-graded gravelly soil was selected as the backfill and the backfill was compacted very well to an average degree of compaction higher than 95 % based on the maximum dry density obtained using compaction energy 4.5 times higher than the standard Proctor (Fig. 18c; Tatsuoka et al., 2008). The monitored deformations of the walls (Fig. 18d) showed very small deformations during construction and negligible post-construction deformations after wall completion, indicating high stability conditions. This case history indicates that long-term deformation of geosynthetic-reinforced soil structures can be restrained very effectively by good compaction of good backfill despite that significantly stiff reinforcement members are not used.

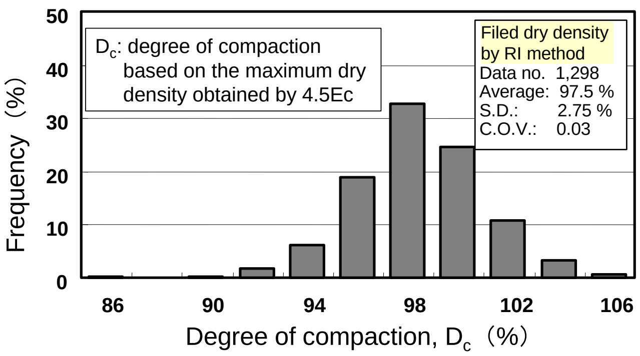
The recorded time histories of the tensile strain in the geogrid also exhibited nearly no increase after wall completion. Kongkitkul et al. (2008) analysed these data based on an elasto-viscoplastic constitutive model of the geogrid developed based on laboratory test results. They showed that the tensile load in the geogrid tends to decrease with time after wall completion, and creep rupture failure of the geogrid by the end of the wall design life is unlikely. The reduction of the tensile strain in the geogrid with time is due to not only the viscous properties of the geogrid but also because of compressive creep strains in the horizontal direction of the backfill caused by the tensile force in the reinforcement. This result indicates that the assumption in current practice that the tensile load mobilised in the geosynthetic reinforcement in the backfill is kept constant over-estimates the possibility of creep rupture failure of geosynthetic reinforcement. In fact, the rupture strength of geosynthetic-reinforcement used in the seismic design of GRS-RWs having a stage-constructed FHR facing is not reduced for creep rupture. Tatsuoka et al. (2006) proposed a new method by which the design rupture strength of the geosynthetic reinforcement to be used in both seismic and static designs of GRS RWs is not reduced for creep rupture.

(Figure 18 (*a*))

(*b*)



(*c*)



(*d*)

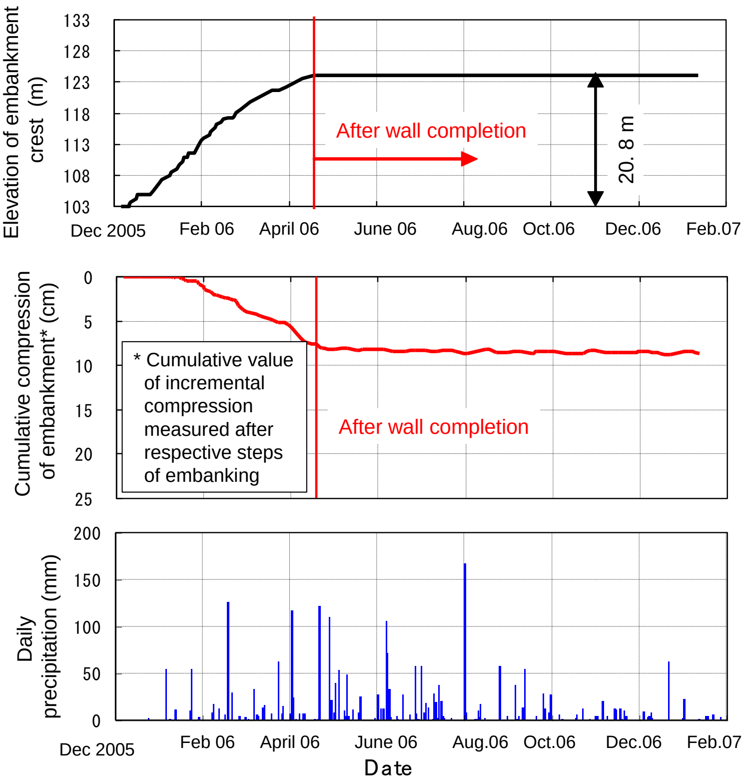


Figure 18: High GRS RW for Fujisan-Shizuoka airport; (*a*) wall in valley 1 (2 Nov. 2007); (*b*) cross-section of wall in valley 2; and; (*c*) measured degree of compaction of the backfill, wall in valley 2; (*d*) deformation of wall in valley 2 (Fujita et al., 2008).

SOME ADVANCES IN SOIL REINFORCEMENT IN NORTH AMERICA

This section is focused on developments in North America related to geosynthetic reinforced soil (GRS) walls. In North America the current common approach for the design and analysis of geosynthetic reinforced soil walls is the AASHTO (2002) Simplified Method. The approach is based on limit-equilibrium of a "tied-back wedge" for internal stability and its origins can be traced back to the early 1970's (Allen and Holtz 1991, Berg et al. 1998). The same allowable stress design (ASD) approach is proposed in the Canadian Foundation Engineering Manual (CFEM 2006) which is an important guidance document for geotechnical engineers in Canada. For segmental retaining walls constructed with discrete dry-stacked module concrete facing units, the most important reference is the guidance document published by the National Concrete Masonry Association (NCMA 1997). This document provides a full treatment for analysis, design and specification of these systems which continue to grow in popularity in North America. Nevertheless, this growth has been largest in the private sector compared to state, province and federal funded-projects. The experience of the writers is that specifications for backfill and modular facing components tend to be stricter for government projects and there continue to be reservations in some jurisdictions regarding durability of dry cast masonry modular facing units in harsh (freeze-thaw) environments.

Many suppliers of segmental retaining walls components (facing units and/or reinforcement materials) have developed computer design aids to facilitate design. However, generic programs are also available. Program SRWall 3.22 is a full implementation of the NCMA manual for static load environments and the seismic supplement (Bathurst 1998) for earthquake design of this class of structure. Program MSWE 3.0 (Leshchinksy 2006) allows the engineer to design complex geometries for geosynthetic reinforced soil walls using AASHTO (2002) for ASD, AASHTO (2007) for LRFD design and the NCMA (1997) (ASD) method.

A brief summary of developments related to geosynthetic reinforced soil wall technology and practice in North America follows below. This review does not claim to be comprehensive but highlights a number of developments that are familiar to the authors.

Cohesive-frictional soil backfills: The use of cohesive-frictional soils as a cheaper alternative to "select" granular fills continues to grow. This is due in part to increasing confidence as more projects are completed using these soils and the recognition that materials with a large fines content can be used as the backfill provided that adequate attention is paid to compaction control during construction and good drainage practice is carried out particularly at the backfill surface. Nevertheless, the use of these materials is largely restricted to private sector projects. A summary of recent experimental walls that have been monitored after being constructed with c-ϕ soils appears in the papers by Miyata and Bathurst (2007) and Bathurst et al. (2008).

Facing units: A very large number of proprietary masonry concrete units are available on the market today. The units vary in size and may be hollow or solid. They have a range of facing appearances and include concrete shear keys, pins or clips for alignment and is some cases for layer shear transfer (e.g. NCMA 1997). However, the use of larger modular block facing units formed from unreinforced wet-cast concrete is growing. The concrete is typically return concrete from wet concrete batch plants. These modular units are often 1 m^3 are larger (Figure 19). Most are solid with concrete shear keys but some systems are hollow to reduce the mass of concrete. The attraction of these systems to designers is that they are very stable and help to ensure a durable facing with good long-term facing alignment. A recent novel development that has appeared in the market place is a product that uses plastic molded shapes to entirely replace the concrete in conventional systems (Figures 20 and 21). The units lock together between courses and the interior components filled with granular soil. A range of different facing appearances are achieved by using different (patterned or textured) thin plastic panels that snap on to the internal molded unit.

Figure 19: Example of large wet-cast concrete modular block.

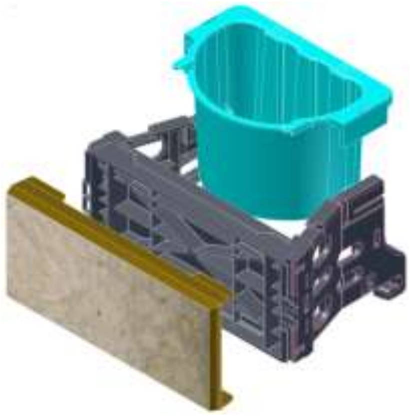


Figure 20: Geosynthetic modular "block" unit components (courtesy Robert Race).

Figure 21: Construction of GRS wall with geosynthetic modular "block" units (courtesy Robert Race).

Bridge abutments: Most GRS reinforced bridge abutments have been constructed with the bridge deck supported by piles taken to a competent foundation layer. Hence, the GRS wall has been required to primarily support only the backfill soil in the approach fill. A more cost effective solution is to have the bridge deck supported by spread footings placed directly on the reinforced soil zone. The first instrumented and monitored wall of this type was the Founders/Meadows structure constructed by the Colorado Department of Transportation in 1999 (Abu-Hejleh et al. 2002) (Figures 22 and 23). An additional advantage of this construction is that the bridge deck and approach fill settle together thus reducing the pavement bump that can occur at the fill-deck joint for conventional structures. A recent variation on this general approach is to place the bridge deck ends directly on the reinforced soil zone leaving a gap between the bottom of the bridge deck and the top of modular block facing. Examples of these structures have been reported by Adams (2008). An additional feature of these walls is the use of closely spaced reinforcement layers to ensure reinforcement capacity redundancy and to create a dense monolithic composite (gravity) mass comprised of the facing, reinforced soil and reinforcement layers.

Figure 22: Founders/Meadows GRS bridge abutment.

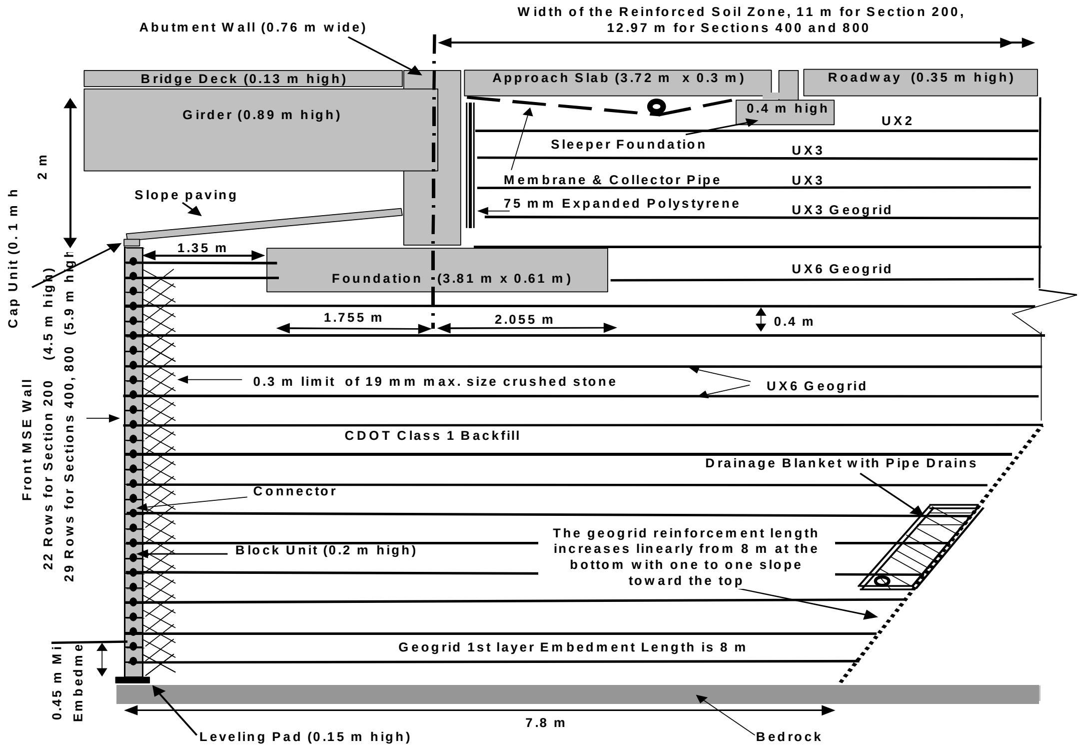


Figure 23: Cross-section view of Founders/Meadows GRS bridge abutment (Abu-Hejleh et al. 2002).

Alternative design methods: The consensus of experienced GRS wall designers is that current ASD-based design methods are conservative with respect to prediction of reinforcement loads under operational conditions. Quantitative evidence in support of this view has been reported by Allen et al. (2002, 2003). They showed that for walls with a hard facing the maximum loads in the reinforcement were (on average) three times higher than predicted values using the conventional AASHTO (2002) Simplified Method and there was no statistically significant relationship between predicted and measured loads. Furthermore, the distribution of maximum reinforcement loads was trapezoidal in shape rather than triangular as is the case for walls with uniformly spaced reinforcement layers supporting dead loads due to soil self-weight and designed using the AASHTO approach. The results of ongoing work have led to a new approach for working stress design for the internal stability design of reinforced soil wall structures called the K-stiffness Method. The origins of the method can be traced back to Allen et al. (2003) and have been implemented in the Geotechnical Design Manual by the Washington State Department of Transportation (WSDOT 2006). The original method was restricted to reinforced soil walls with granular backfill. The method has recently been extended to include c-ϕ soil backfills (Bathurst et al. 2008). The method includes an empirical expression for maximum reinforcement load (T_{max}) in a layer under operational conditions (i.e. a serviceability state characterized by a limit on post-construction wall deformations and a limit on reinforcement strains for the case of granular backfills):

$$T_{max} = \frac{1}{2} K\gamma(H+S)S_v D_{tmax} \phi_g \phi_{local} \phi_{fs} \phi_{fb} \phi_c \qquad (1)$$

Here: H = height of the wall; S = equivalent height of uniform surcharge pressure q (i.e., S = q/γ); D_{tmax} = load distribution factor that modifies the reinforcement load based on layer location. The remaining terms, ϕ_g, ϕ_{local}, ϕ_{fs}, ϕ_{fb} and ϕ_c are influence factors that account for the effects of global and local reinforcement stiffness, facing stiffness, face batter, and soil cohesion, respectively. The coefficient of lateral earth pressure is calculated as K = 1 – sinϕ with $\phi = \phi_{ps}$ = secant peak plane strain friction angle of the soil. The cohesion influence factor is of particular importance to walls with backfill soils having a significant and measurable cohesive strength component (c). The cohesion influence factor is calculated according to Miyata and Bathurst (2007) as:

$$\phi_c = 1 - \frac{\lambda c}{\gamma H} \qquad (2)$$

Where λ is a fitting coefficient from back-calculated data. The coefficient terms that appear in the above expressions have been back-calculated from a large database of instrumented walls. Quantitative comparison of measured to predicted loads shows that load prediction accuracy is greatly improved over the current AASHTO approach and variants. The first modular block GRS walls to be designed using this method were constructed and instrumented in Washington State. The walls were up to 11 m in height and were constructed with a granular backfill. Verification of the more efficient reinforcement layout and good agreement between measured and predicted reinforcement loads for the highest wall section (11 m) is quantitatively demonstrated by Allen and Bathurst (2006).

Limit states design: A recognized obstacle to even greater use of GRS walls in North America is the lack of an adequate transition to a limit states design (LSD) format (called load and resistance factor design (LRFD) in the United States. This situation is compounded by the observation that the design of retaining walls (at least in government projects) typically involves structural engineers who work in a limit states design environment. An initial step in this direction is the AASHTO (2007) bridge design code which is now fully LRFD. However, the general approach has been to fit limit state equations to ASD equations so that load and resistance factors matching a given reliability index value give the same factor of safety as in conventional practice. This is not an entirely satisfactory approach since it does not guarantee a uniform level of reliability for all possible limit states. Formal procedures to carry out rigorous calibration have only just begun for GRS walls. An example of the general approach described in a way that is familiar to geotechnical engineers can be found in a recent TRR circular (Allen et al. 2005). An advantage of the K-stiffness Method described earlier is that it can be easily recast into a limit states design format (at least for the calculation of internal reinforcement loads) since the underlying deterministic model has been calibrated using a statistical treatment of measured and predicted reinforcement loads. An initial step in this direction can be seen in the WSDOT (2006) design guidance document mentioned earlier.

SOME ADVANCES IN GEOENVIRONMENTAL APPLICATIONS USING GEOSYNTHETICS

Geosynthetics play an important role in environmental applications because of their versatility, cost-effectiveness, ease of installation, and consistency in their mechanical and hydraulic properties. Geosynthetics also can offer a technical advantage in relation to traditional liner systems or other containment systems. The use of geomembranes as the primary water proofing element at the Contrada Sabetta Dam, Italy (Cazzuffi 1987) and to keep an upstream clay seepage control liner from dessicating in the Mission Dam (today Terzaghi Dam), Canada (Terzaghi & Lacroix 1964) in the late 1950's represent applications that have been the precursors of today's usage of geosynthetics in containment systems. Both applications predated the use of conventional geosynthetics by some 20 years. Geosynthetic systems are nowadays an accepted and well-established component of the landfill industry (since at least early 1980's). Containment systems for landfills typically include both geosynthetics and earthen material components, (e.g. compacted clays for liners, granular media for drainage layers, and various soils for protective and vegetative layers).

The state of the art on the use of geosynthetics in waste containment facilities previous to this period has been documented by various important sources, which have set the path for the growth of geosynthetics in this field (e.g. Giroud & Cazzuffi 1989; Koerner 1990; Cancelli & Cazzuffi 1994; Gourc 1994; Rowe et al. 1995; Manassero et al. 1998; Rowe 1998; Bouazza et al. 2002, Junqueira et al. 2006).

This section focuses on some recent advances on the use of geosynthetics in environmental applications, including the design of geosynthetics in liquid collection systems and of reinforced cover systems.

The multiple uses of geosynthetics in the design of modern municipal solid waste landfills is a good illustration of an application in which the different geosynthetics can be and have been used to perform all the functions discussed previously. Virtually all the different types of geosynthetics discussed previously have been used in the design of both base and cover liner systems of landfill facilities. Figure 24 illustrates the extensive multiple uses of geosynthetics in both the cover and the base liner systems of a modern landfill facility (Zornberg & Christopher 2007). The base liner system illustrated in the figure is a double composite liner system. Double composite liner systems are used in some instances for containment of municipal solid waste and are frequently used for landfills designed to contain hazardous waste. The base liner system shown in the figure includes a geomembrane/GCL composite as the primary liner system and a geomembrane/compacted clay liner composite as the secondary system. The leak detection system, located between the primary and secondary liners, is a geotextile/geonet composite. The leachate collection system overlying the primary liner on the bottom of the liner system consists of gravel with a network of perforated pipes. A geotextile protection layer beneath the gravel provides a cushion to protect the primary geomembrane from puncture by stones in the overlying gravel. The leachate collection system overlying the primary liner on the side slopes of the liner system is a geocomposite sheet drain (geotextile/geonet composite) merging into a gravel layer. A geotextile filter covers the entire footprint of the landfill and minimizes clogging of the leachate collection and removal system. The groundwater level may be controlled at the bottom of the landfill by gradient control drains built using geotextile filters. Moreover, the foundation soil below the bottom of the landfill may be stabilized as shown in the figure using randomly distributed fiber reinforcements, while the steep side soil slopes beneath the liner could also be reinforced using geogrids. Different types of geosynthetics (e.g. geogrids, geotextiles, fibers) could have been selected for stabilization of the foundation soils.

The cover system of the landfill illustrated in Figure 24 contains a composite geomembrane/GCL barrier layer. The drainage layer overlying the geomembrane is a geocomposite sheet drain (composite geotextile/geonet). In addition, the soil cover system may include geogrid, geotextile, or geocell reinforcements below the infiltration barrier system. This layer of reinforcements may be used to minimize the strains that could be induced in the barrier layers by differential settlements of the refuse or by a future vertical expansion of the landfill. In addition, the cover system could include geogrid or geotextile reinforcement above the infiltration barrier to provide stability to the vegetative cover soil. Fiber reinforcement may also be used for stabilization of the steep portion of the vegetative cover soil. A geocomposite erosion control system above the vegetative cover soil is indicated in the figure and provides protection against sheet and gully erosion. Fig. 24 also illustrates the use of geosynthetics within the waste mass, which are used to facilitate waste placement during landfilling. Specifically, the figure illustrates the use of geotextiles as daily cover layers and of geocomposites within the waste mass for collection of gas and leachate. Geosynthetics can also be used as part of the groundwater and leachate collection well system. The use of geotextiles as filters in groundwater and leachate extraction wells is illustrated in the figure. Finally, the figure shows the use of an HDPE vertical barrier system and a geocomposite interceptor drain along the perimeter of the facility. Although not all of the components shown in Figure 24 would normally be needed at any one landfill facility, the figure illustrates the many geosynthetic applications that can be considered in landfill design.

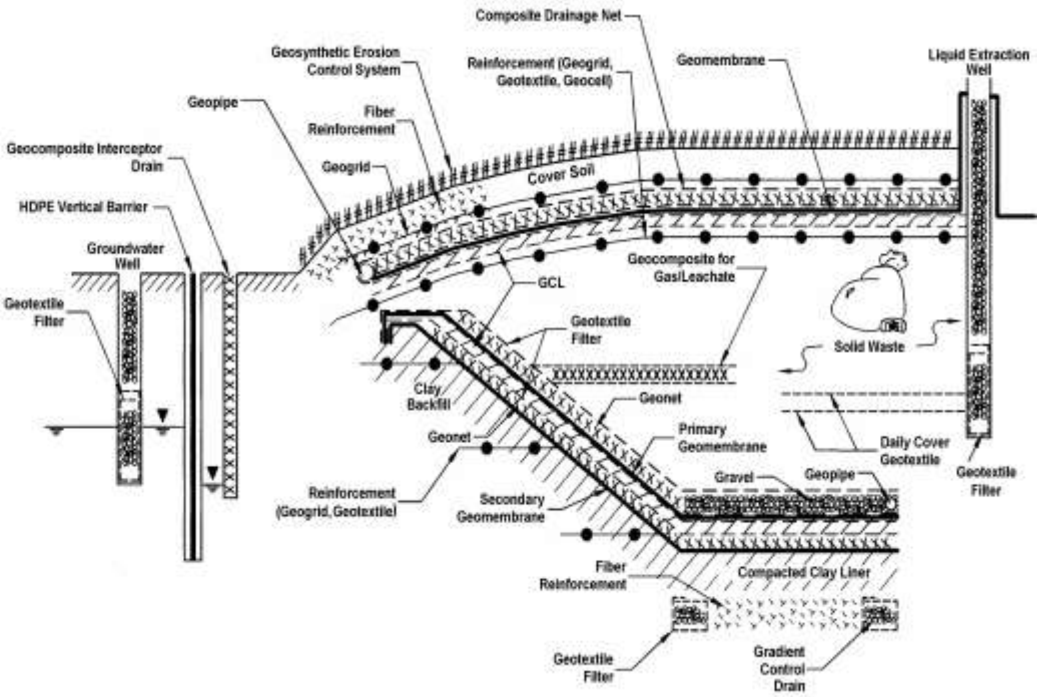


Figure 24: Multiple uses of geosynthetics in landfill design (from Zornberg & Christopher 2007).

Geosynthetics in Liquid Collection Systems

Calculating the thickness of liquid in a liquid collection layer is an important design step because one of the design criteria for a liquid collection layer is that the maximum thickness of the liquid collection layer must be less than an allowable thickness. The term “thickness” is used instead of the more familiar term “depth”, because thickness (measured perpendicular to the liquid collection layer slope), and not depth (measured vertically), is actually used in design.

The thickness of liquid in a liquid collection layer depends on the rate of liquid supply. A typical case of liquid supply is that of liquid impinging onto the liquid collection layer. Two examples of liquid collection layers with such a type of liquid supply can be found in landfills (Fig. 25): (i) the drainage layer of the cover system (Fig. 25a), where the liquid that impinges onto the liquid collection layer is the precipitation water that has percolated through the soil layer overlying the drainage layer; and (ii) the leachate collection layer (Fig. 25b), where the liquid that impinges onto the leachate collection layer is the leachate that has percolated through the waste and through the protective soil layer overlying the leachate collection layer (Giroud et al., 2000a). The terminology “liquid impingement rate” is often used in the case of landfills to designate the rate of liquid supply.

Equations are available (Giroud et al. 2000a) to calculate the maximum thickness of liquid in a liquid collection layer that meets the following conditions:

- the liquid supply rate is uniform (i.e. it is the same over the entire area of the liquid collection layer) and is constant (i.e. it is the same during a period of time that is long enough that steady-state flow conditions can be reached);
- the liquid collection layer is underlain by a geomembrane liner without defects and, therefore, liquid losses are negligible;
- the slope of the liquid collection layer is uniform (a situation referred to herein as “single slope”) ; and
- there is a drain at the toe of the slope that promptly removes the liquid.

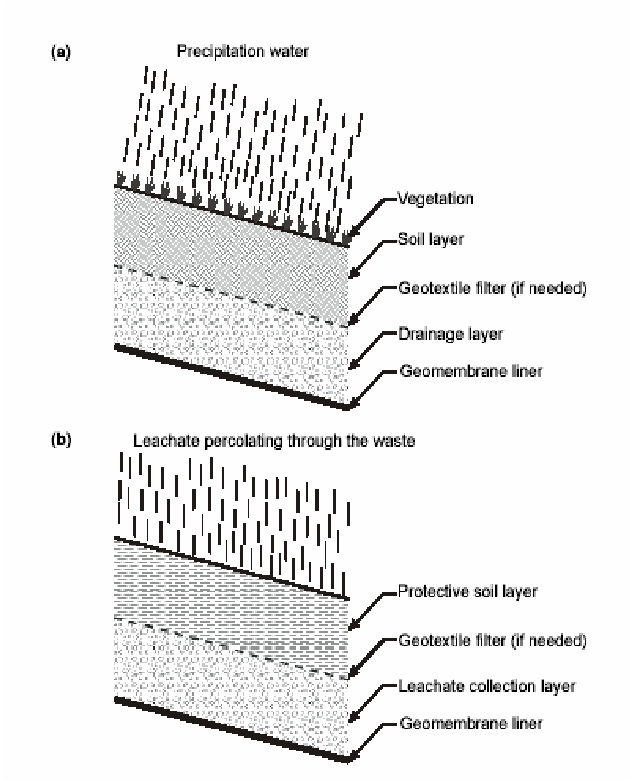


Figure 25: Examples of liquid collection layers subjected to a uniform supply of liquid in a landfill: (a) drainage layer in a cover system; (b) leachate collection layer (Giroud et al. 2000a).

The last two conditions are not met in cases where the liquid collection layer comprises two sections on different slopes, with no drain removing the liquid at the connection between the two sections; in those cases, the only drain is at the toe of the downstream section.

Regulatory equivalency between natural and geocomposite lateral drainage systems is currently based on equivalent transmissivity. However, Giroud et al. (2000c) have demonstrated that this practice is incorrect and non-conservative. An equivalency based solely on transmissivity will lead to selection of a geosynthetic drainage layer that may not provide adequate flow capacity and may result in the development of water pressure.

Equivalency between two lateral drainage systems must take into consideration the service flow gradients and maximum liquid thickness. Giroud et al. (2000c) have shown that, to be equivalent to a natural drainage layer, the minimum transmissivity of the geocomposite must be greater than the tranmissivity of the natural drainage layer. The minimum transmissivity of the geonet is obtained by multiplying the transmissivity of the natural drainage layer by an equivalency factor, *E*. For natural drainage layers having maximum flow depths of 0.30 m, *E* can be approximated as follows:

$$E = \frac{1}{0.88}\left[1 + \frac{t_{prescribed}}{0.88\,L}\frac{\cos\beta}{\tan\beta}\right] \tag{3}$$

where $t_{prescribed}$ is the maximum liquid thickness prescribed by regulations. The equivalency defined by Equation 3 is based on equal unconfined flow volumes in natural and geocomposite drainage systems. However, the very low heads associated with unconfined flow in a geocomposite lateral drain will result in a significantly reduced head acting on the underlying liner system, and therefore in a reduced potential leakage.

Reinforced Cover Systems

General considerations

The design of veneer slopes (e.g. steep cover systems for waste containment facilities) poses significant challenges to designers. The use of uniaxial reinforcements placed along the slope (under the veneer and above a typically strong mass of soil or solid waste) and anchored on the top of the slope has been a common design approach (Palmeira and Viana 2003). However, this alternative may not be feasible for steep, long veneer slopes. As the veneer slope rests on top of a comparatively stronger mass solid waste, alternative approaches can be considered. This includes use of uniaxial reinforcements placed horizontally (rather than along the slope) and anchored into the underlying mass. A second alternative includes the use of fiber-reinforced soil. A review of analyses for veneers reinforced using horizontally placed inclusions is presented in this section.

This section presents an analytical framework for quantification of the reinforcement requirements for reinforced veneers where reinforcements are placed horizontally and embedded into a comparatively strong underlying mass. Emphasis in this evaluation is placed on the assessment of an infinite slope configuration. This allows direct comparison of the different reinforcement alternatives.

Design criteria for reinforced soil structure have been the focus of significant debate (Zornberg & Leshchinsky 2001). Although different definitions for the factor of safety have been reported for the design of reinforced soil slopes, the definition used in this study is relative to the shear strength of the soil:

$$FS = \frac{\textit{Available soil shear strength}}{\textit{Soil shear stress required for equilibrium}} \qquad (4)$$

This definition is consistent with conventional limit equilibrium analysis, for which extensive experience has evolved for the analysis of unreinforced slopes. Current design practices for reinforced soil slopes often consider approaches that decouple the soil reinforcement interaction and do not strictly consider the factor of safety defined by Equation 4. Such analyses neglect the influence of reinforcement forces on the soil stresses along the potential failure surface and may result in factors of safety significantly different than those calculated using more rigorous approaches. 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, Equation 4 can be expressed as:

$$FS = \frac{c + (N/L)\tan\phi}{S/L} \qquad (5)$$

where N = normal force acting on the control volume; S = shear force acting on the control volume; L = length of the control volume; c = soil cohesion; and ϕ = soil friction angle.

From the analysis of equilibrium conditions, the classic expression for the factor of safety FS_u of an unreinforced veneer can be obtained:

$$FS_u = \frac{c}{\gamma\, T\, \sin\beta} + \frac{\tan\phi}{\tan\beta} \qquad (6)$$

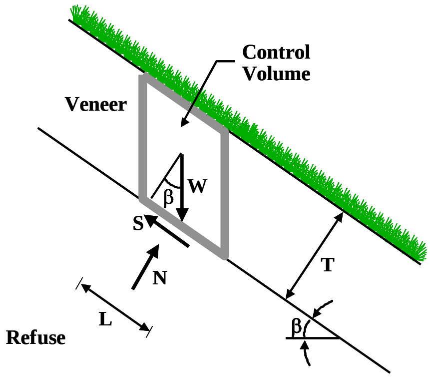


Figure 26: Unreinforced veneer.

Covers Reinforced with Uniaxial Geosynthetics Parallel to the Slope

Figure 27 shows a schematic representation of a cover system reinforced using uniaxial geosynthetics placed parallel to the slope. An infinite slope case is considered. In the case, the shear force needed for equilibrium of the control volume is smaller that the one in the unreinforced case. In this case, the shear force is defined by:

$$S = W \sin\beta - t_p\, L$$

where t_p = distributed reinforcement tensile stress of the reinforcement parallel to the slope.

When the geosynthetic reinforcements are placed parallel to the slope, the distributed reinforcement tensile stress is a function of the allowable reinforcement tensile strength (T_a) and the total slope length (L_T), as follows:

$$t_p = \frac{T_a}{L_T}$$

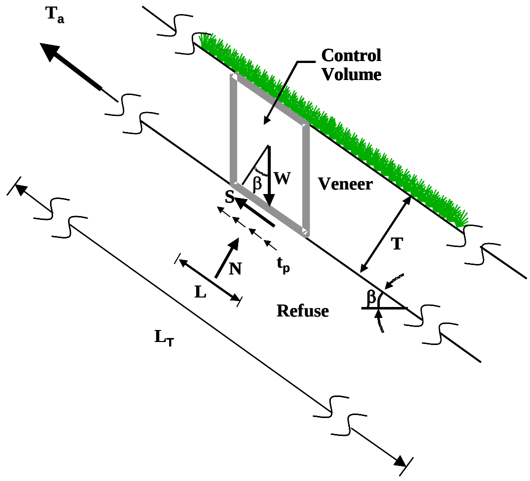


Figure 27: Schematic representation of a cover system reinforced using uniaxial geosynthetics placed parallel to the slope.

From limit equilibrium analysis, the factor of safety for the parallel-reinforcement case, $FS_{r,p}$, can be estimated as:

$$FS_{r,p} = \frac{\dfrac{c}{\gamma\ T\ \sin\beta} + \dfrac{\tan\phi}{\tan\beta}}{1 - \dfrac{t_p}{\gamma\ T\sin\beta}} \tag{9}$$

Equation 9 provides a convenient expression for stability evaluation of reinforced veneer slopes. It should be noted that if the distributed reinforcement tensile stress t equals zero (i.e. in the case of unreinforced veneers), Equation 9 leads to $FS_{r,p} = FS_u$.

Covers Reinforced with Horizontal Uniaxial Geosynthetics

Figure 28 illustrates a cover (veneer) reinforced using horizontal uniaxial geosynthetics. Also 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. For the purpose of the analyses presented herein, the reinforcement tensile forces are represented by a distributed reinforcement tensile stress t_h, which corresponds to a uniformly distributed tensile force per unit height. For a given slope with layers of reinforcement t_h can be expressed by:

$$\mathrm{t_h} = \frac{\mathrm{T_a}}{\mathrm{s}}$$

where T_a = allowable reinforcement tensile strength and s = vertical spacing.

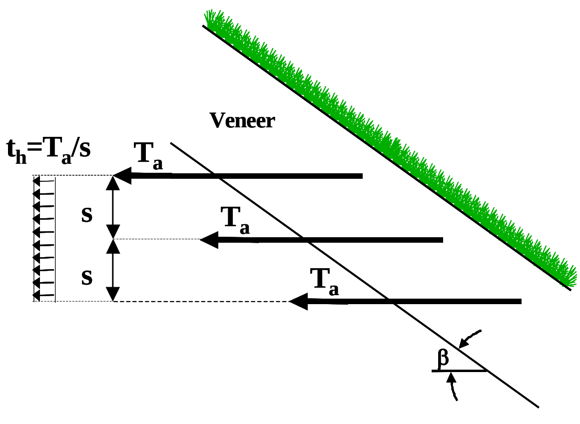


Figure 28: Veneer reinforced with horizontal uniaxial geosynthetics

From limit equilibrium analysis, the following expression can be obtained for the factor of safety $FS_{r,h}$ of a veneer reinforced with horizontal uniaxial geosynthetics:

$$FS_{r,h} = \frac{\dfrac{c}{\gamma\ T\ \sin\beta} + \dfrac{\tan\phi}{\tan\beta} + \dfrac{t_h}{\gamma\ T} \sin\beta\ \tan\phi}{1 - \dfrac{t_h}{\gamma\ T} \cos\beta} \tag{11}$$

Equation 11 provides an expression for stability evaluation of reinforced veneer slopes. It should be noted that if the distributed reinforcement tensile stress t_h equals zero (i.e. in the case of unreinforced veneers), Equation 11 leads to $FS_r = FS_u$

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 of the reinforced veneer.

Covers Reinforced with Randomly Distributed Fibers

A promising potential alternative for stabilization of steep landfill covers involves the use of fiber-reinforcement. Advantages of fiber-reinforcement over planar reinforcement in the stabilization of landfill covers are:

- Fiber-reinforcement is particularly suitable for stabilization of veneer slopes, as it provides additional shear strength under low confining pressures. A small increase of shear strength under low confinement has a significant impact on the stability of shallow slopes.

- Randomly distributed fibers helps maintaining strength isotropy and do not induce potential planes of weakness that can develop when using planar reinforcement elements.

- No anchorage is needed into solid waste as in the case of reinforcement with horizontal geosynthetics or at the crest of the slope as in the case of reinforcement parallel to the landfill slope.

- In addition to stabilizing the cover slopes, fiber reinforcement has the potential of mitigating the potential for crack development, providing erosion control, and facilitating the establishment of vegetation.

Relevant contributions have been made towards the understanding of the behavior of fibers. A soil mass reinforced with discrete, randomly distributed fibers is similar to a traditional reinforced soil system in its engineering properties but mimics admixture stabilization in the method of its preparation (Gray & Al-Refeai 1986; Bouazza & Amokrane 1995). Potential advantages of fiber-reinforced solutions over the use of other slope stabilization technologies have been identified, for example, for slope repairs in transportation infrastructure projects (Gregory & Chill 1998) and for the use of recycled and waste products such as shredded tires in soil reinforcement (Foose et al. 1996). Several composite models have been proposed in the literature to explain the behavior of randomly distributed fibers within a soil mass. The proposed models have been based on mechanistic approaches (Maher & Gray 1990), on energy dissipation approaches (Michalowski & Zhao 1996), and on statistics-based approaches (Ranjar et al. 1996).

Fiber-reinforced soil has often been characterized as a single homogenized material, which has required laboratory characterization of composite fiber-reinforced soil specimen. The need for laboratory characterization has been a major drawback in the implementation of fiber-reinforcement in soil stabilization projects. To overcome this difficulty, a discrete approach that characterizes the fiber-reinforced soil as a two-component (fibers and soil) material was recently developed (Zornberg 2002). The main features of this approach are:

- The reinforced mass is characterized by the mechanical properties of individual fibers and of the soil matrix rather than by the mechanical properties of the fiber-reinforced composite material

- A critical confining pressure at which the governing mode of failure changes from fiber pullout to fiber breakage can be defined using the individual fiber and soil matrix properties.

- The fiber-induced distributed tension is a function of fiber content, fiber aspect ratio, and interface shear strength of individual fibers if the governing mode of failure is by fiber pullout.

- The fiber-induced distributed tension is a function of fiber content and ultimate tensile strength of individual fibers if the governing mode of failure is by fiber breakage.

Figure 29 shows a schematic view of a fiber-reinforced infinite slope. The behavior of the fiber-reinforced soil mass depends on whether the failure mode is governed by pullout or breakage of the fibers. The governing failure mode of the fiber-reinforced soil mass depends on the confinement. A critical normal stress, $\sigma_{n,crit}$, can be defined for comparison with the normal stress σ_n at the base of the veneer. If $\sigma_n < \sigma_{n,crit}$, the dominant mode of failure is the fibers pullout. This is the case for cover system applications. In this case, the fiber-induced distributed tension t_f is defined by (Zornberg 2002):

$$t_f = \eta\, \chi\, c_{i,c}\, c + \eta\, \chi\, c_{i,\phi} \tan\phi\, \sigma_n \tag{12}$$

where $c_{i,c}$ and $c_{i,\phi}$ are the interaction coefficients for the cohesive and frictional components of the interface shear strength; η = aspect ratio (length/diameter) of the individual fibers, and χ = volumetric fiber content.

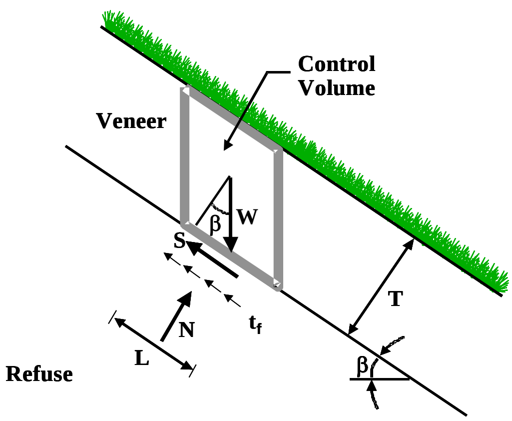


Figure 29: Veneer reinforced with randomly distributed fibers

Similarly, if $\sigma_n > \sigma_{n,crit}$, the dominant mode of failure is fiber breakage. Even though this is not generally the governing mode of failure for cover slopes the solution for this case is presented for completeness. The fiber-induced distributed tension t_f is defined by:

$$t_t = \sigma_{f,ult} \cdot \chi \tag{13}$$

where $\sigma_{f,ult}$ = ultimate tensile strength of the individual fiber.

In a fiber-reinforced veneer, the shear force needed for equilibrium of the control volume equals:

$$S = W \sin\beta - \alpha\, t_f\, L \tag{14}$$

where α is an empirical coefficient that accounts for preferential orientation of fibers. For the case of randomly distributed fibers considered herein α equals one.
From limit equilibrium analysis, the factor of safety for a fiber-reinforced veneer, $FS_{r,f}$:, is given by

$$FS_{r,f} = \frac{\dfrac{c}{\gamma\, T \sin\beta} + \dfrac{\tan\phi}{\tan\beta}}{1 - \dfrac{\alpha\, t_f}{\gamma\, T \sin\beta}} \tag{15}$$

Solutions were presented for the case of unreinforced, slope-parallel, horizontally-reinforced and fiber-reinforced veneers. As expected, additional reinforcement always leads to a higher factor of safety while increasing slope inclination would typically lead to decreasing stability. Yet, it is worth noting that increasing soil friction angle leads to increasing stability, when compared to the unreinforced case, only for the case of fiber reinforced slopes. Also, it should also be noted that increasing total height of the slope (or increasing total length) does not affect detrimentally the efficiency of horizontally placed reinforcements and of fiber reinforcement.

CONCLUSIONS

Geosynthetics have great potential to be used as cost-effective solutions for several engineering problems. This paper presented recent advances in geosynthetic products, on the utilization of these materials in reinforced soil structures and in environmental applications. Manufacturing of geosynthetics products allows incorporating recent advances in material sciences. Therefore, the expectation is that innovations in products, types and properties will continue to take place, adding to the already vast range of applications of these materials.

Geosynthetic reinforced soil retaining walls present better performance than traditional retaining walls under dynamic loadings and this has been demonstrated by a number of case histories of prototype structures that have withstood severe earthquakes. Thus, this type of structure can be cost-effective not only under static loading but also in regions where significant seismic activities are expected. New construction methodologies have also broaden the applications of geosynthetic reinforced soil retaining wall, which include new facing units and that reduces the construction time, costs and allow better aesthetic conditions for the final structure.

Investigations on the behaviour of large model reinforced walls built under controlled conditions, monitoring of real structures and theoretical studies have yielded the development of a practical method for the estimate of reinforcement loads, including the case of using cohesive backfill. This method is a significant advance on existing design approaches and will allow the construction of cheaper structures.

The use of geosynthetics has also led to major advances in environmental applications. While geosynthetics has been used in a number of applications in environmental project, this paper has described advances on the use of geosynthetics in landfills. Specifically, simple yet accurate formulations are now available for the design of liquid collection systems, which involve proper quantification of the thickness of liquid within drainage composites. Also, significant advances have taken place regarding the use of reinforcements for stabilization of steep cover systems. Approach include the use of geosynthetic reinforcements parallel to the cover slope, horizontal reinforcements embedded into solid waste, and fiber reinforcement of the cover soils. Overall, the use of geosynthetics has led to major advances towards the construction environmental systems that are cost-effective but that provide enhanced environmental protection.

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