Ten Cate Geosynthetics

Ten Cate Geosynthetics has been at the forefront of reinforced soil technology for more than 30 years. During that time our Company has built formidable expertise in reinforced soil technology. Today, Ten Cate Geosynthetics Group’s reinforced soil business is focused as follows.

Worldwide coverage: Ten Cate Geosynthetics Group has worldwide coverage providing advice and delivery in all locations. Our geographical network can readily respond and provide quality solutions to create value for our clients.

Applications knowledge: Our expert personnel are at the forefront of reinforced soil applications knowledge. We not only provide applications advice and solutions for our clients but also serve on National Standards bodies that develop Codes of Practice on reinforced soil.

Value-engineered solutions: By combining our applications expertise with our extensive range of geosynthetic reinforcements we are able to provide innovative, value-engineered solution to many reinforced soil applications. This approach increases the value provided to our clients.

High performance reinforced soil materials: Ten Cate Geosynthetics Group manufactures and supplies a wide range of geosynthetic reinforcement materials specifically engineered for a wide range of reinforced soil applications.

High quality standards: Ten Cate Geosynthetics Group operates to ISO 9001 quality procedures, the best standards in the industry. Also, our geosynthetic reinforcements conform to specific National quality requirements.

Efficient reinforcement material delivery: Our Group is able to deliver our geosynthetic reinforcement materials efficiently to many diverse geographical locations. The materials are specifically packaged for ready storage on site if required.

Research and development: We operate extensive research and development programs in the fields of both reinforced soil applications technology and geosynthetic reinforcement engineering. This places us at the forefront of new reinforced soil developments.

Highly skilled personnel: As a pioneer in the field of reinforced soil using geosynthetic reinforcements, the history of the Group’s involvement stretches back to the mid 1970s.

- During the mid 1970s Ten Cate Geosynthetics Group first manufactured PET and PP woven geotextile reinforcements in Europe. These geosynthetic reinforcement materials were used for the efficient construction of basal reinforced embankments. In the late 1970s this technology was exported to the United States with great success, and later to Asia (mid 1980’s).
  - During the early 1980s PET geogrid reinforcements were introduced in the USA. These geosynthetic reinforcement materials were used for the efficient construction of reinforced slopes and walls. Later, this technology was exported to Asia (mid 1980’s) and to Europe (early 2000’s).
  - During the late 1980s PET geogrid reinforcement were introduced in Europe. These geosynthetic reinforcement materials are novel in that they combine strength and stiffness with the ability to dissipate pore water, and have been used in reinforced slopes and walls where conditions require these characteristics. In the early 2000’s this technology was exported to Asia.

Ten Cate Geosynthetics Group has also been at the forefront of reinforced soil applications technology for over 30 years. During this time our expertise has enabled innovative reinforced soil design procedures to be developed.

For reinforced soil applications a single layer, or multiple layers, of geosynthetic reinforcement are used to provide stability, and reduce deformations, in geotechnical structures. Geosynthetic reinforcement is used for a variety of reinforced soil applications, the most common are summarised below.

Basal reinforced embankments on soft soil
Here, a layer of geosynthetic reinforcement is placed at the base of an embankment constructed over soft foundation soils to improve the stability of the embankment. The presence of the geosynthetic reinforcement enables the embankment to be constructed higher, and with steeper side slopes, than would be the case if no reinforcement was used.

Basal reinforced embankments on piles
Here, a layer of geosynthetic reinforcement is placed at the base of an embankment over a pile foundation platform to improve the stability, and prevent settlement, of the embankment. The presence of geosynthetic reinforcement in combination with the pile foundation platform enables the embankment to be constructed to any height, at any rate, without instability and settlement problems.

Basal reinforced embankments spanning voids
Here, a layer of geosynthetic reinforcement is placed at the base of an embankment over a section that is prone to the formation of voids, to prevent instability and excessive localised settlements, to the embankment. The presence of the geosynthetic reinforcement ensures that foundation void formation does not lead to distress at the surface of the embankment.

Reinforced soft site closures
Here, a layer of geosynthetic reinforcement is placed across the surface of very soft deposits prior to the placement of fill and closure of the site. The presence of the geosynthetic reinforcement provides local stability, thus enabling a stable working platform to be constructed across the very soft deposit.

Reinforced soil walls
Here, multiple layers of geosynthetic reinforcement are placed in the wall to provide stability and limit deformations while placing and compacting the reinforced fill. The presence of the geosynthetic reinforcement enables stable slopes to be constructed to any height and at any slope angle.

Ten Cate Geosynthetics’ history in reinforced soil

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Ten Cate Geosynthetics’ reinforcement materials

Mirafi® PET, Geolon® PET and Polyfelt® WX woven polyester geotextile reinforcements

These geotextile materials are composed of high strength, high modulus polyester yarns woven into tensile strengths ranging from 100 kN/m to 1,600 kN/m. This large strength range, coupled with their very good long term load carry capability, makes these materials ideal for basal reinforcement applications where high tensile loads have to be carried for long periods of time.

Miragrid® XT and GX geogrid reinforcements

These geogrid materials are composed of high strength, high modulus polyester yarns embedded in a robust polymer coating and have tensile strengths ranging from 35 kN/m to 600 kN/m. These materials have good resistance to the effects of installation damage and are highly durable in a wide range of soil conditions. Consequently, they are almost always used for applications where tensile loads have to be carried for long periods of time.

Polyfelt® PEC and Bidim® PEC geocomposite reinforcements

These geocomposite materials are composed of high strength, high modulus polyester yarns assembled onto a continuous filament nonwoven geotextile support layer and have tensile strengths ranging from 35 kN/m to 250 kN/m. These materials have good resistance to the effects of installation damage and because of the incorporation of the nonwoven layer may be utilised to dissipate pore water from poorer quality fills.

Mirafi® HP and PP, and Geolon® PP woven polypropylene geotextile reinforcements

These geotextile materials are composed of high strength, high modulus polypropylene yarns woven into tensile strengths from 60 kN/m to 300 kN/m. This large strength range, coupled with their medium term load carrying capability and excellent durability, makes these materials ideal for short to medium term reinforced soil applications. These materials have good resistance to the effects of installation damage. Also, their specific gravity is such that they can be deployed over water where necessary.

International reinforced soil case studies

Ten Cate Geosynthetics has been involved in many reinforced soil applications, in many parts of the world, over the last 30 years. The selection of the case studies contained in this booklet give an appreciation of the diverse range of reinforced soil applications where Ten Cate’s geosynthetic reinforcements have been used. New reinforced soil applications are continually evolving.

Basal reinforced embankments on soft soil

Pacific Freeway, Chinderah, NSW, Australia.

Electrically double track, Ipoh to Padang Besar, Malaysia.

West Salym Communication Corridor, West Siberia, Russia.

Mine services corridor, Cape Preston, WA, Australia.

Embankment over old sludge lagoon, Interstate 670, Columbus, Ohio, USA.

Seawall construction, Brisbane Port expansion, Australia.

Runway overrun area, La Guardia International Airport, New York, USA.

Partially submerged dyke, Doeldock, Antwerp, Belgium.

Basal reinforced embankments on piles

Wat Nakorn-In bridge approaches, Bangkok, Thailand.

Atalanta dual carriageway, Dundalk to Newry, Ireland.

Bridge approach embankments, M74 Completion project, Glasgow, UK.

Basal reinforced embankments spanning voids

High speed railway track over karst foundation, LGV Est, Lorraine, France.

High speed railway track over karst foundation, Guadalajara, Spain.

Football field over old landfill, Barcelona, Spain.

Reinforced soft site closures

Wallasey Dock roll-on roll-off terminal, Liverpool, UK.

Mine tailings pond closure, Huelva, Spain.

Waste treatment sludge lagoons closure, Axis, Alabama, USA.

Reinforced fill slopes

Yeager Airport runway extension, West Virginia, USA.

Road realignment, Rodlauer Bridge, Styria, Austria.

Landslide restoration, Langkawi, Kedah, Malaysia.

Slope restoration, Maehongsong, Thailand.

Road widening, Chiangmai, Thailand.

Avalanche protection barrier, Diabach, Tyrol, Austria.

Highway earthworks widening, A3 Hindhead, Surrey, UK.

Railway embankment widening, Hamilton, Ontario, Canada.

Reinforced soil walls

Interstate 5/995 widening, San Diego, California, USA.

Structural walls, Antanara Qsar Al Sarab Desert Resort, Abu Dhabi, United Arab Emirates.

Hill side housing development, Batu Ferringhi, Penang, Malaysia.

Panipat Elevated Highway, Haryana, India.

Overpass abutments, M’Sila, Algeria.

Flyover abutments, Dakar, Senegal.

Coal mine dump wall, Sangatta, East Kalimantan, Indonesia.

Reinforced walls and slopes, Gwinnett, Georgia, USA.

Reinforced walls and slopes, Upper Harbour Corridor, Greenhithe, New Zealand.

Coal processing plant platform, Hetaoysu, Ganyang City, Gansu Province, China.

Segmental block wall with constrained reinforced fill, Paju, Korea.

Shear-key wall, Trump National Golf Course, California, USA.
Basal reinforced embankments on soft soil: Pacific Freeway, Chinderah, NSW, Australia

The Pacific Highway in Australia has undergone major upgrading between Sydney and Brisbane to turn it into a dual-carriage freeway. The construction between Yelgun and Chinderah in the North of New South Wales consists of a dual-carriageway freeway of some 36 km in length.

Approximately 10 km of this freeway were to be constructed in geologically old river valleys, and flood plains, where the foundation soils consisted of soft silty clays, with depths ranging from 5 to 15 m. The undrained shear strength of this soft silty clay layer ranged from 8 to 12 kPa, increasing with depth, with a 1 m thick overconsolidated crust of approximately 15 kPa. Embankment heights in these areas ranged from 2 to 5 m. The embankment geometry consists of a 30 m wide crest with 1V:2H side slopes.

To meet the construction time and performance requirements of the project it was decided to construct a basal reinforced, 1 m surcharged embankment in the areas where soft foundation soils were encountered. The basal reinforcement would provide adequate stability to allow the embankment to be constructed quickly to the full height, with the 1V:2H side slopes, and thus ensure the maximum time for foundation consolidation during the construction period. Foundation consolidation was accelerated by the installation of prefabricated vertical drains (PVD’s) into the soft foundation layer.

A Mirafi® 500X geotextile separator was placed directly over the grass vegetation on the soft foundation soil. Prior to its placement, trees and large vegetation were removed, but the grass was left in place in order not to disturb the surface of the soft foundation layer. The geotextile was overlapped 0.5 m to provide continuous geotextile separation coverage prior to placement of the bridging layer on top.

A bridging layer of 0.5 m thick of local clayey fill was placed on the separation geotextile. This bridging layer created a stable platform on which the PVD installation equipment could operate, and also enabled less granular material to be used for the drainage blanket. Following this, a 0.2 m thick drainage layer of crushed gravel was placed on the bridging layer. The gravel was obtained from crushing rock in cut sections of the freeway project. The drainage layer enabled the excess pore water from the PVD’s to be drained rapidly to the extremities of the embankment. The PVD’s were then installed through the drainage and bridging layers into the soft foundation on a square grid with spacings ranging from 1 to 3 m.

Mirafi® PET woven polyester geotextiles were placed across the top of the drainage layer to provide the basal reinforcement stability for the embankments. Depending on the height of the embankment sections, and the depth and strength of the soft foundation soils, different Mirafi® PET strengths of 200 kN/m, 400 kN/m, 600 kN/m and 800 kN/m were used. The Mirafi® PET geotextiles were installed across the width of the embankments to ensure a continuous length of basal reinforcement spanning across the width of the embankment sections. Along the length of the embankments the Mirafi® PET geotextile was overlapped by a minimum of 0.5 m.

The embankment fill was then placed on top of the Mirafi® PET basal reinforcement. The fill used was variable, ranging from overconsolidated clay to crushed rock, and was obtained from cut sections along the length of the freeway. To increase the rate of consolidation a surcharge of 1 m of fill was placed on top of the embankment. This surcharge, in combination with the PVD’s, enabled most of the embankment settlement to occur during the period of construction.

After 9 to 12 months the excess surcharge was stripped off the top of the embankments and the surface was graded and prepared for the placement of the freeway pavement. Once the concrete pavement had been constructed and the ancillary structures completed the freeway was opened to traffic.

Client: Roads and Traffic Authority, New South Wales, Australia.
Consultant: SMEC Pty Ltd, New South Wales, Australia.
Contractor: AbiGroup Ltd, New South Wales, Australia.
Peninsula Malaysia has a well-developed road transportation system but the railroad system has not been developed to the same standard. This has resulted in an overdependence on road transportation which accounts for over 90% while the current railroad system accounts for only 3% of total transportation.

To correct this transportation imbalance the Malaysian Government is upgrading the peninsular western railway line that runs from the Malaysian-Thailand border town of Padang Besar to Johor Bahru at the southern tip of Peninsular Malaysia. This railway line will eventually form part of the Trans-Asia railway line spanning from Singapore to Kunming in China.

The USD4 billion Electrified Double Track Railway Project covers the design and construction of the infrastructure and system works for a 330 km long electrified double tracking railway line between Ipoh and Padang Besar in the northern half of Peninsula Malaysia, passing through the Malaysian States of Perak, Penang, Kedah and Perlis.

This railway project involves laying two new parallel tracks, replacing the existing single-track. It includes extensive foundation improvement works as well as the construction of bridges and tunnels over a wide variety of geological and ground conditions. Construction of new stations and installation of modern electrification and signalling systems also form part of the project.

Approximately 180 km of the railway alignment passes over alluvial river valleys and low-lying coastal plains where normally or slightly overconsolidated alluvial and marine clay deposits predominate. Commonly, these soft soil deposits are of 20 m or more in depth, exhibit undrained shear strengths ranging from 5 kPa to 20 kPa, and are essentially normally consolidated. All of these areas required ground improvement works to ensure stability and deformations are maintained within specific limits for the railway embankments.

The ground improvement method used in the construction of the railway embankment over these soft clay foundation soils involved the use of prefabricated vertical drains (PVD's) and preloading the embankment by surcharging to ensure consolidation of the soft foundations occurs in a relatively short period of time during the construction project. To maintain stability of the surcharged embankment while the soft foundation soils were undergoing consolidation, a layer of Mirafi PET or Polyfelt WX geotextile reinforcement was placed at the base of the embankment prior to placement of the embankment fill. In areas close to associated structures, e.g. bridges, piling or stone columns were used for ground improvement depending on the ground conditions.

Construction of the basal reinforced railway embankments started with the stripping of vegetation and topsoil to provide a level surface. A Polyfelt® TS geotextile separator was placed on the soft ground surface prior to the placement of a 500 mm thick layer of sand. This sand layer acts as a drainage blanket to drain out excess pore water from the base of the embankment and also acts as a working platform to support the PVD installation equipment. The PVD’s were installed through the sand layer and geotextile separator into the soft foundation soils on a 1.2 m grid and to a depth that coincided with the bottom of the soft foundation layer. Following installation of the PVD’s a thin layer of sand was then placed over the sand platform prior to placement of the geotextile reinforcement.

The Mirafi PET or Polyfelt WX geotextile reinforcement was placed across the surface of the sand drainage blanket at right angles to the direction of the embankment. Depending on the strength and extent of the soft foundation soils and the height of the surcharged embankment different strengths of geotextile reinforcement were used varying from 100 kN/m to 800 kN/m in the machine direction.

After the basal geotextile reinforcement was installed, general fill was placed and compacted in layers to construct the surcharged embankments. Typically, the surcharged embankments were 5 m to 6 m in height, of which 2 m to 3m was surcharge. After the preloading period was completed, which was typically 3 to 6 months, the surcharge was removed from the embankments. Then the ballast and tracking works were carried out, followed by the electrification works.

Peninsular Malaysia is the northern half of Peninsular Malaysia, between Ipoh and Padang Besar in the 330 km long Electrified double track railway line spanning from Singapore to Kunming in China.

Basal reinforced embankments on soft soil:
Electrified double track, Ipoh to Padang Besar, Malaysia

Typical cross section through the railway embankments

Where possible, the alignment of the new double track ran alongside the existing single track. When this was not possible two alternatives could be adopted. One alternative involved the new embankment overlapping the existing single track. This involved a sequenced construction where one of the new tracks was built alongside the old single track so that trains could be diverted to run on this new track before the second new track could be constructed over the top of the existing single track. The second alternative involved total realignment of the track with complete new embankments.

The new railway alignments are designed for travelling at high speeds of 160 km/h. Along with systems modernization, travel times will be halved compared to the existing single track railroad system.

Client: KTMB, Kuala Lumpur, Malaysia.

Specialist Consultant: G&P Geotechnics Sdn Bhd, Kuala Lumpur, Malaysia.
A 50 km long, all-weather highway (the communication corridor) was required to service a number of oil drilling platforms in the area of the town of Salym in West Siberia. Ground conditions along the proposed highway alignment were very difficult, ranging from rock cuttings to very soft peat bog swamps varying in depths of up to 8 m. There was also a requirement to construct around 80 km of secondary roads connecting the various oil drilling platforms to the main communication corridor. The ambient temperatures in the region are extreme, ranging from well below freezing in the winter (-60ºC) to very warm to hot during the summer (30ºC). The proposed road system was to be constructed continuously over a 2 year period during all weather conditions, and it was critical that construction achieved an average rate of 300 m/day to meet the demanding goals established by the Client.

The road system had a required design life of 25 years after which it had to be dismantled and the area returned to its original pristine condition. This 25 year design life reflected the life time for oil extraction in the area. Further, after construction, the road system was required to have minimal maintenance in order to maintain traffic flows, and this lead Engineers to decide on the use of a compacted fine-crushed rock based course layer for the surface of the pavements. To maintain the stability of the boundary between the compacted base course layer and the subgrade formation, under all weather conditions, a Geolon® PP100S geotextile separator was used throughout the road system.

The construction of the communication corridor through the very soft peat bog swamps was particularly challenging. During winter the peat bogs are frozen, however, during the remainder of the year they are saturated with the groundwater level being at ground surface. Also, during the summer groundwater movement in the peat bogs can be as much as 5 m/day. Historically, the method of construction over peat bogs in the area was to place a layer of timber “corduroy” over the peat surface and then construct the roads on top. This was a slow and expensive process, and further, experience showed that the road would only last 1 to 2 years as the timber would sink into the peat and break up, and the road would fail. Also, any road construction required the extensive use of culverts to ensure adequate water flows from the peat bogs. The incorporation of culverts in these road embankments always gave problems with relation to differential settlements.

The solution adopted was to construct basal reinforced embankments across the peat bogs. Stability analyses were performed assuming a 3.5 m embankment height, with 1V:3H side slopes, and this resulted in a basal reinforcement strength requirement of 200 kN/m for short term stability. Geolon® PP200 geotextile reinforcement was chosen for the basal reinforcement because it met the short term strength requirement, and the material would perform well under all diverse weather conditions, including the large range in ambient temperatures.

In order to construct the basal reinforced embankments over the very soft peat bogs it was decided to place a timber “corduroy” platform over the peat bogs to provide an initial stable construction platform. While it was recognised that the timber platform would only provide a short term solution, it was enough for the construction of the basal reinforced embankments. This way the basal reinforced embankments could be constructed in the dry to good quality.

Once the timber corduroy platform had been installed the layer of Geolon® PP100S geotextile reinforcement was placed across the top and this was wrapped around the edges of the placed and compacted embankment fill. The embankment fill used was silty sand obtained from borrow areas nearby. A second layer of Geolon® PP120S geotextile reinforcement was wrapped around the edge of the embankment fill to ensure the fill is not lost due to edge instability of the embankments, or by erosion during the spring thaw and heavy rain. Where culverts were required, these were integrated into the base of the basal reinforced embankment in order to minimise the likelihood of later differential settlements.

The embankment fill was then constructed to full height (around 3.5 m) with side slopes of 1V:3H. Due to the year-round construction program some of these embankments were constructed during the summer and some during the winter.

Once the embankments had been raised to formation level the road pavement was constructed on top. As already stated, the road pavement consisted of a compacted fine-crushed rock base course as it was considered that this would perform well over time and require minimal maintenance. Before placement of the base course layer, a Geolon® PP100S geotextile separator was placed on top of the silty sand formation. The geotextile separator was used to ensure the boundary between the silty sand formation and the granular base course would remain stable throughout all weather conditions under traffic. During winter when there is snow and ice, and during summer and autumn when the embankment fill dries out the silty sand embankment fill can support the traffic loads exerted through the pavement. However, during the spring thaw and during periods of heavy rainfall local instability problems arise as the silty sand formation cannot support the pavement layer above when trafficked. The inclusion of the Geolon® PP100S geotextile separator is designed to account for these local instability problems. Following placement of the geotextile separator, the granular base course layer was placed and compacted.

The basal reinforced embankments have continued to perform well for 5 years since construction. Settlements have been less than expected, even after the loss of the effect of the timber working platform. The Client has estimated that the savings in cost of the basal reinforced embankments was around 50% compared to soft foundation replacement, and around 25% compared to complete (conventional) timber “corduroy” construction.

The fine-crushed rock pavements have also performed well over time. While there has been some pot-holing and change of shape, these are easily rectified by regrading and compacting.

**Client:** Salym Petroleum Development NV, Moscow, Russia.
The Sino Iron Project is a world-class, large-scale magnetite iron ore project located at Cape Preston, 100 km south west of Karratha, in Western Australia’s Pilbara region. This iron ore project is the largest planned magnetite project in Australia with an estimated 2 billion tonnes of identified magnetite ore. Mine development and infrastructure costs are estimated at USD 3.5 billion.

The project has an extremely tight time schedule, with construction beginning in mid-2008 and is due for completion at the end of 2010. In addition to the large open pit mine, major infrastructure items consist of a 450 MW power station, a 25 km long slurry pipeline, a 50 gigalitre desalination plant to supply fresh water, and a new deep water port with stockpile facilities. The port handles the import of heavy equipment for the mine site as well as the export of magnetite pellets.

A crucial component of the overall project was the construction of a 30 km long services corridor connecting the port to the mine site. This services corridor had to be completed in advance of other infrastructure items to enable the transportation of all heavy equipment for the mine site, power station, desalination plant and all other related facilities.

Part of this services corridor consisted of a 2 km long causeway constructed through a river estuary. The foundation conditions within the river estuary consisted of soft estuarine mud of approximately 4 m in depth overlying firm sandy soil. The estuarine mud consisted of a slightly overconsolidated crust of 1 m in thickness with an undrained shear strength ranging from 7 kPa to 10 kPa. Below this the undrained shear strength increased with depth from around 6 kPa to around 20 kPa at 4 m depth.

The causeway embankment ranged in height from 1 m to 7 m, with a crest width of 32 m, and consisted of mine waste rock fill. The side slopes of the causeway were maintained at 1V:2H. At the centre of the causeway a 200 m long reinforced concrete bridge was constructed to enable river flows during both normal and flood periods.

In addition to the tight construction schedule, the causeway embankment had to be designed to allow for heavy vehicle loadings from 240 tonne haul trucks as well as the safe transport of 1000 tonne giant grinding mills for the mining operations.

To construct the causeway a number of design and construction options were evaluated. These ranged from stage construction to soft soil replacement. The only solution that was economically and environmentally viable was to use basal reinforcement across the base of the causeway embankment to enhance stability and achieve the required factor of safety. A detailed analysis was performed using the limit equilibrium method to determine what strength properties the basal reinforcement should have to meet the stability requirements.

Once the basal reinforcement design loads were established, other factors such as the effect of the placement and compaction of the coarse mine waste rock fill on the basal reinforcement were also evaluated, as well as the design life over which the basal reinforcement was required. Taking all these factors into account, 3 layers of Mirafi® PET800-50 geotextile reinforcement were chosen as the basal reinforcement for its ability to meet all of the requirements. Mirafi® PET800-50 geotextile reinforcement is a woven polyester geotextile with a tensile strength of 800 kN/m at 10% strain in the longitudinal direction and a tensile strength of 50 kN/m in the cross direction. The polyester yarns used are of high tensile modulus and have an excellent resistance to creep.

The Mirafi® PET800-50 geotextile reinforcement was placed directly on the surface of the soft estuarine mud with the rolls of geotextile laid out 90 degrees to the direction of the causeway embankment. No geotextile joins were allowed in this direction across the width of the embankment. The first mine waste rock fill lift was placed on top of the geotextile reinforcement, spread out and compacted to construct an initial fill platform of 0.5 m thickness. On top of this fill platform the second geotextile reinforcement layer was placed and then a 0.3 m thick fill layer placed on top. Finally, the third geotextile layer was placed and then the embankment was constructed to its completed grade alignment.

Where the causeway embankments abutted the central bridge structure another three layers of Mirafi® PET800-50 geotextile reinforcement, placed coincidentally with the cross-wise layers, was used at the base of the 7 m high abutments to ensure adequate stability in the vicinity of the main river channel. These 3 layers were placed 40 m into the causeway to ensure the bridge abutments had adequate stability.

The use of basal reinforcement has enabled the causeway embankment to be constructed quickly, directly on the estuarine mud foundation, without soil replacement. Consequently, the impact on the environment has been reduced to a minimum. Further, the services corridor has been completed on schedule. No subsequent embankment deformations have been observed.

Client: CITIC Pacific Mining Management Pty Ltd, Perth, WA, Australia.
Consultant: Connell Wagner Pty Ltd, Perth, WA, Australia.
Contractor: MCC Mining (Western Australia) Pty Ltd, Perth, Australia.
Photographs courtesy of CITIC Pacific Mining Pty Ltd.
The extension of a 6-lane interstate freeway, I-670, into downtown Columbus, Ohio was required to eliminate a major highway bottleneck. The only alignment that was available was across a 1 km stretch of old gravel pits that had been filled with water softening sludge from an adjacent water treatment plant in the 1970’s. Probes identified the sludge to be up to 7 m deep.

The sludge is a by-product of the water treatment process and consists of aluminium sulphate, lime, soda ash and alum, and has the consistency of toothpaste. The sludge had a very high moisture content, ranging between 200% and 300%, and a pH = 10. Undrained shear strengths ranged between 200% and 300%, and a pH = 10.15 m surcharge for 3 years

The sludge layer was highly compressible from 5 to 10 kPa, increasing with depth. The extension of a 6-lane interstate

Embankment over old sludge lagoon, Interstate 670, Columbus, Ohio, USA

process was repeated for the second Mirafi® HP1500 geotextile reinforcement layer, with the third geotextile reinforcement layer placed on top of this. The geotextile reinforcement layers were placed without wrinkles and were seam to laterally into continuous sheets by means of onsite sewing.

Construction of the embankment to a maximum 12 m in height involved staged construction (even with the presence of PVD’s) where the embankment fill loading was matched with a gain in shear strength of the sludge, with the basal geotextile reinforcement providing the required short term stability. This staged construction was carried out with the aid of extensive instrumentation. The embankment fill, along with a 1.5 m surcharge, was placed in a controlled manner over a 15 month period. The surcharge was left in place for approximately 4 years (because there was funding issues that delayed the early completion of the freeway) before it was stripped off to the required grade level, and the concrete freeway pavements constructed.

Large settlements of up to 2.8 m have been recorded under the 12 m high embankment section prior to pavement placement. This corresponds to 35% of the original sludge thickness. At the same time geotextile reinforcement strains ranging between 1% and 4% have been recorded. While settlements have been large, five years after placement of the concrete pavement, the roadway section over the sludge is performing similarly to the pavements on non-sludge areas.

Client: City of Columbus/Ohio DOT, USA.
Consultant: Gale-Tec Engineering Inc., Minneapolis, Minnesota, USA.
Contractor: Kokosing Construction Co., Columbus, Ohio, USA.

Typical cross section through the basal reinforced embankment

Placement of embankment fill over Mirafi® HP1500 geotextile reinforcement

Embankment nearing completion

Interstate 670 embankment completed

![Typical cross section through the basal reinforced embankment](image)

Spreading sand working platform across Mirafi® FW402 geotextile separator

The geosynthetic reinforcement for the final embankment design was determined using a limit equilibrium approach. The design allowed for 3 layers of Mirafi® HP1500 geotextile reinforcement at the base of the embankment to develop the required short term stability. Mirafi® HP1500 is a high modulus, woven polypropylene geotextile with a tensile strength of 190 kN/m in the longitudinal direction. It was considered that a polypropylene geotextile would be better suited for this application because of the potentially harmful effects of the high pH sludge material on geotextile reinforcement durability.

To construct the embankment, a working platform was first constructed across the sludge material. The working platform consisted of a Mirafi® FW402 woven polypropylene geotextile separator installed across the sludge, with a 1 m thick sand layer on top to perform the dual role of a working platform and the drainage layer for the PVD’s. The Mirafi® FW402 geotextile separator was fabricated into wide panels onsite and then pulled across the sludge surface using ropes and small excavators. The sand fill was then spread across the geotextile separator to a thickness of 1 m using light weight equipment.

Following construction of the working platform, the PVD’s were installed, on a 2 m triangular pattern, through the working platform to the base of the sludge layer.

Following installation of the PVD’s, the first Mirafi® HP1500 geotextile reinforcement layer was placed across the width of the embankment, with 300 mm of sand fill placed on top. This reinforced embankments on soft soil:

Embankment over old sludge lagoon, Interstate 670, Columbus, Ohio, USA

embankment to develop the required short term stability. The geotextile reinforcement was chosen to perform the dual role of a working platform and the drainage layer for the PVD’s. The Mirafi® FW402 geotextile separator was fabricated into wide panels onsite and then pulled across the sludge surface using ropes and small excavators. The sand fill was then spread across the geotextile separator to a thickness of 1 m using light weight equipment.

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Embankment over old sludge lagoon, Interstate 670, Columbus, Ohio, USA
The Port of Brisbane is located at the mouth of the Brisbane River, and has seen rapid development over the last 20 years, and this growth is expected to continue in the future. To keep up with the pace of growth the Future Port Expansion Project was conceived. This project has the ultimate objective of allowing the Port to expand by reclaiming and developing an additional 230 ha of port land, including the extension of the current shipping quay by a further 1800 m. The reclamation will be formed from channel maintenance dredging materials. The first stage of this process involved the construction of a 4.6 km long and up to 7.5 m high perimeter seawall in order to contain the reclamation fill in an environmentally friendly and controlled manner.

There were significant geotechnical, environmental and construction risk issues associated with the project. These included, highly variable very soft, and soft, marine clays extending over 30 m below the seabed on the eastern wall alignment; the close proximity of the Moreton Bay Marine Park which could not tolerate any sediment contamination; and varying water depths and unpredictable sea conditions during construction. Preliminary analyses indicated the marine clay foundation to be generally too weak to support high embankment loadings without ground improvement works. A number of options were evaluated but the use of a high strength geotextile as the basal reinforcement for the seawall was ultimately assessed to be the most cost effective solution, and having the least associated risk.

The main geological formations across the project site can be summarized as Holocene deposits overlain by Pleistocene deposits, which in turn overlie basalt bedrock of the Petrie Formation. The Holocene alluvial deposit consists of two sub-layers. The upper sub-layer comprises mainly sands with inter-layered soft clays and silts. The lower sub-layer comprises very soft to firm compressible clay, generally normally consolidated from about 3 m depth below the seabed. Along the eastern seawall, the soft clay at shallow depth is weak, having undrained shear strength values of 3 to 5 kPa at the surface, and increasing towards the shoreline. The thickness of the soft layer varies from about 8 m to 30 m along the alignment.

The final seawall design required the use of basal geotextile reinforcement of tensile strength ranging between 400 kN/m and 850 kN/m, depending on the location and water depth. Polyfelt WX polyester geotextile reinforcement was used for the basal reinforcement. In shallow water (seabed at 1 m below low water level) Polyfelt WX geotextile reinforcement was placed on the seabed directly beneath the rock dyke seawall. However in deeper areas (seabed at 3.5 m below low water level), the seawall was designed with a wide-based sand embankment that was then topped up with the rock dyke. Polyfelt WX geotextile reinforcement was placed directly on the seabed beneath the sand embankment in this case.

To install the Polyfelt WX geotextile reinforcement below water level a shallow-draft barge was modified to enable both the geotextile reinforcement and the sand fill to be placed in a single operation. The geotextile reinforcement was sewn to the geotextile flat and help sink the geotextile reinforcement below water level. Polyfelt WX polyester geotextile reinforcement was placed on the seabed beneath the rock dyke in a single operation. The geotextile reinforcement was placed in a single operation. The geotextile reinforcement was placed in a single operation.

The placing of the rock fill upper layer of the seawall was carried out in the dry by end dumping the rock fill and then spreading using excavators. At the same time as the upper rock fill was placed the rock armour layers were also placed using end dumping and excavators. The armour protection was continued at the same time as the rock fill placement in order to protect the seawall from unforeseen storm activity during construction.

Following completion of the seawall the reclamation for the port expansion has progressed as planned. The spoil from the maintenance dredging operations has been used to build a land bank area for future port expansion programs. The seawall has also prevented any sediment contamination of the nearby Moreton Bay Marine Park.

Client: Port of Brisbane Corporation, Brisbane, Australia.
Consultant and Contractor: Alliance Partners, Brisbane, Australia.
A runway overrun area had to be constructed at the East end of runway 13-31 at New York’s La Guardia International Airport. This was due to several overrun incidents occurring that brought political and safety impetus to the construction of this overrun area. This impetus also dictated that the overrun area be completed in a short period of time.

The overrun area is constructed in a broad inter-tidal mud flat consisting of a 23 m thick layer of soft, normally consolidated organic clay. The undrained shear strength of this organic clay varies between 5 and 10 kPa at ground surface and increasing linearly with depth at a rate of 1.5 kPa/m. Below this organic clay layer are glacial deposits (dense sands and overconsolidated clays and silts) of thickness around 35 m. The challenge of this project was to place fill on the soft, normally consolidated organic clay without instability occurring.

The historical approach to land reclamation at La Guardia had been end-dumping of fill, which had created extensive, uncontrolled mud waves. Mud wave creation was deemed unacceptable for this project due to the close proximity of a federal shipping channel and community concerns about increased “low-tide” odour.

A number of design concepts were investigated for the overrun area. These included structural decking, pre-dredging and filling, or a geotextile reinforced construction. The latter solution was adopted because it was the most cost-effective and least disruptive construction methodology. In order to accomplish the filling in the inter-tidal area without creating mud waves, the design prescribed detailed stage construction procedures. This staged design incorporated the installation of a layer of high strength geotextile reinforcement across the surface of the soft organic clay prior to the placement of hydraulically pumped sand fill. The maximum tensile load and extension requirements for the geotextile reinforcement were calculated by evaluating the various geometric combinations and loadings.

The geotextile reinforcement used was Mirafi® PET200-150 which is a high strength, high modulus, woven polyester geotextile having an ultimate tensile strength of 200 kN/m in the length direction and 150 kN/m in the cross direction. The high cross-directional strength was required to ensure sewn seam strengths in this direction would meet the 60 kN/m design strength required.

The contractor elected to use 3 barges, coupled in tandem, with a total length of 230 m for the geotextile reinforcement deployment. It was planned to cover the whole soft organic clay area using two large fabricated sheets of geotextile reinforcement, overlapped by 15 m at their joins. Lengths of Mirafi® PET200-150 geotextile reinforcement were rolled out along the deck of the barges and the joins seamed to produce the 60 kN/m seam strength requirement. The sewn geotextile was folded in an accordion-like fashion on the barge decks for easy deployment.

During high tide the barges were manoeuvred close to the shoreline with the Mirafi® PET200-150 geotextile reinforcement unfurled off the barges and onto the shoreline. The geotextile was anchored in place and then the barges were slowly pulled from shore with the geotextile reinforcement unfurling into the water and progressively sinking onto the bay bottom, where it was secured with sand bags. The deployment of the two large fabricated sheets of Mirafi® PET200-150 geotextile reinforcement took place on two weekends when both runway closures and midday high tides coincided, with the unfurling process taking approximately 50 minutes per sheet.

The filling over the geotextile reinforcement was specified as hydraulically placed sand fill because this was the only placement method which could produce the low load levels and the flat slopes necessary for stability. The fill was placed in lifts no greater than 1 m, with the overall fill slope 1:20H.

With the interim overrun area in place, prefabricated vertical drains (PVD’s) were installed through the sand fill into the soft organic clay foundation to increase the rate of consolidation. The PVD’s were installed to an average depth of 25 m on a 1.2 m triangular grid and installation was performed during runway closures late at night. The performance of the PVD’s was good with foundation consolidation rates approaching laboratory predictions. It was anticipated that at completion of construction the final settlements would range from 3 m to 4.5 m at different locations in the filled area.

The overall performance of the Mirafi® PET200-150 geotextile reinforcement was excellent. There was no discernible mud waves created during the hydraulic filling and subsoil displacements were minimal compared to the large displacements typical with the previous end-dumping methods.

Client: La Guardia International Airport, New York, USA.
Consultant: Port Authority of New York and New Jersey, USA
Contractor: Yonkers Contracting Inc., Yonkers, New York, USA.

Basal reinforced embankments on soft soil: Runway overrun area, La Guardia International Airport, New York, USA
Basal reinforced embankments on soft soil: Partially submerged containment dam, Doeldok, Antwerp, Belgium

As part of the ever-increasing expansion of Antwerp Harbour, increased capacity of disposal facilities for dredged material and excavated soil have to be found. A solution to this problem has involved the construction of a partially submerged containment dam across an old, existing dock to contain spoil and other dredged material. The containment dam has a total height of 27 m, of which 19 m was constructed underwater. The major challenge for this project was that the containment dam had to be constructed on very soft sediments, of thickness approximately 9 m, in the base of the existing dock, and these could not be removed for environmental reasons.

In their natural state these very soft sediments have consolidated under their own buoyant weight only. Undrained shear strengths ranged from 2 to 4 kPa, increasing linearly with depth. Due to the very low bearing capacity and undrained shear strength of these sediments, it became clear that some kind of foundation layer reinforcement was required to ensure stability of the containment dam. The solution adopted was to combine deep soil mixing beneath the side-slopes of the containment dam with geotextile reinforcement in the outer (steeper) slope of the dam. The inner slope of the dam was constructed with a flatter side-slope, and thus only deep soil mixing was carried out here. The combination of the two treatments at the outer slope of the dam ensured there was adequate stability during the controlled construction of the containment dam.

Once the deep soil mixing had been carried out beneath the future side-slopes of the containment dam, construction of the dam was carried out using sand fill placed in stages to control stability. On the outer side of the containment dam stability was maintained by using large sand-cement segmental block facing units attached to layers of Geolon® PET200 geotextile reinforcement. The use of the segmental facing units enabled the outer side-slope to be constructed at a slope angle of 1V:2.5H, and also prevented erosion of the sand fill. The layers of Geolon® PET200 geotextile reinforcement are used to provide additional shear stability to the outer slope of the containment dam. Geolon® PET200 geotextile reinforcement consists of high modulus polyester yarns with an ultimate tensile strength of 200 kN/m. The use of polyester geotextile reinforcement was considered important as it would enable the geotextile to sink easily in water and thus facilitate placement. To provide the required stability, eight layers of Geolon® PET200 geotextile reinforcement were installed at 2 m vertical spacings, and extending continuously between 65 m and 100 m into the containment dam.

To enable efficient placement under water, large sand-cement segmental blocks and their Geolon® PET200 geotextile reinforcement attachments were fabricated on land prior to placement. The segmental blocks were fabricated in size 2 m by 3 m wide by 30 m long, with the Geolon® PET200 geotextile reinforcement attached to these blocks in widths of 30 m and to the continuous lengths required. The weight of these block units along with the rolled-up geotextile reinforcement approximated 380 tonnes and was lifted by a large floating crane that was used on the project. To facilitate lifting, high strength slings were placed around these blocks in widths of 30 m and to the continuous lengths required. The lifting of these units was allowed to consolidate for a period of 1 to 2 months. Following this, another block facing layer was placed with the Geolon® PET200 geotextile reinforcement and the sand filling procedure was repeated.

The construction of the containment dam was divided into two main phases. The first phase covered the construction of the dam up to water surface level. The second phase completed the construction of the containment dam to a height of 7 m above surface water level. The second phase only proceeded once adequate consolidation had occurred in the very soft sediments beneath the containment dam.

Another large block unit was then installed immediately behind the outer wall face, resulting in a total installed block size of 8 m width and 2 m height. The Geolon® PET200 geotextile reinforcement was then completely rolled out across the sand fill surface in one continuous sheet, to the length required, using a second floating crane.

The sand fill used for the filling operations was obtained from excavation works for the construction of a new dock nearby in Antwerp harbour. The sand was selected on the basis of its grain size distribution and fines content. The sand selection was important to ensure the placed fill in the containment dam met the shear resistance requirements assumed at the design stage. The sand was placed in layers 2 m thick using hydraulic filling. At each 2 m lift, the foundation was allowed to consolidate for a period of 1 to 2 months. Following this, another block facing layer was placed with the Geolon® PET200 geotextile reinforcement and the sand filling procedure was repeated.

The containment dam was constructed with a 1V:2.5H facing unit ensuring the outer side-slopes of the containment dam. Geolon® PET200 geotextile reinforcement were installed at 2 m vertical spacings, and extending continuously between 65 m and 100 m into the containment dam.

The sand fill used for the filling operations was obtained from excavation works for the construction of a new dock nearby in Antwerp harbour. The sand was selected on the basis of its grain size distribution and fines content. The sand selection was important to ensure the placed fill in the containment dam met the shear resistance requirements assumed at the design stage. The sand was placed in layers 2 m thick using hydraulic filling. At each 2 m lift, the foundation was allowed to consolidate for a period of 1 to 2 months. Following this, another block facing layer was placed with the Geolon® PET200 geotextile reinforcement and the sand filling procedure was repeated.
The Wat Nakorn-In Bridge and connecting road system is a major infrastructure project, and is part of a larger master plan to ease traffic congestion on the West bank of the Chao Phraya River in the Greater Bangkok area. The new bridge crosses the Chao Phraya River midway between the Rama VII and Northaburi bridges. The project also involved a network of connecting roads that necessitated the construction of other smaller bridges and traffic overpasses. Because of the overall project size, the project was awarded in five contracts, each involving the construction of bridges and embankments to handle up to 10 traffic lanes.

The foundations in the area consist of what is known as “soft Bangkok clay”, overlaid by a stiff clay layer. This soft clay layer has a thickness of about 15 m to 20 m in the Bangkok metropolitan area. Bangkok clay has low shear strength, and is highly compressible, as it is close to being normally consolidated. Typically, the soft Bangkok clay layer has water contents ranging from 80% to 140%, undrained shear strengths from 6 kPa to 15 kPa and bulk densities of 14 kN/m³ to 16 kN/m³.

Consolidation of the soft clay can lead to large differential settlements between embankments constructed directly on the clay, and any piled bridge structures. These differential settlements reduce riding quality and pose safety hazards. They also involve frequent maintenance works, which prove costly over time and cause unnecessary traffic disruption during the maintenance works.

The embankments approaching the Wat Nakorn-In Bridge were designed with pile support to provide stability as well as to prevent large differential settlements between the embankments and the bridge structures. The pile lengths were gradually increased as the embankment heights increased and as the embankments approached the bridges. Where the embankments met the bridge structures, the piles supporting the embankments were designed for end-bearing, similar to those supporting the bridge structures. This tapering of pile depth ensured a smooth road profile transitioning from the section unsupported by piles, over the entire embankment sections supported on piles, and across the bridge structures.

At the beginning of construction, all surface vegetation was removed from the site. Precast reinforced concrete piles, 100 mm square, were driven to the design depths using drop-hammer piling machines. Beneath the embankments the spacing between these piles ranged from 1 m to 2 m depending on the distance from the bridge. Pile caps and connecting beams were then constructed on top of the piles. Connecting beams were included because the foundation soil was very soft and it was thought that additional lateral restraint was required for stability purposes. This was followed by backfilling between the pile caps and connecting beams with sand to form a smooth platform.

Mirafi® PET1000-100 geotextile reinforcement, which has a tensile strength of 1,000 kN/m in the machine direction and 100 kN/m in the cross direction, was laid over this prepared smooth platform. The Mirafi® PET1000-100 geotextile reinforcement is designed to span across the pile caps and transfer the vertical embankment and traffic loads directly onto the them. In the application, the use of Mirafi® PET geotextile reinforcement ensures negligible load is carried directly by the soft foundation, and all load is carried directly by the piles.

The embankments were then constructed by placing and compacting sand fill to the required design heights. Right-of-way traffic restrictions meant that the embankments had to be constructed with steep side slopes in the vicinity of the bridge abutments. These steep slopes were constructed using Miragrid® SXT geogrid reinforcement at 0.5 m vertical spacings. The surface of the reinforced steep slope was then vegetated to provide a green finish to the embankment sides.

In other areas where the embankment heights were low and differential settlements were not an issue, expanded polystyrene (EPS) fill was used to construct embankments of low unit weight. This reduced the level of settlements occurring in these embankments.

Once the embankment earthworks had been constructed the pavements were constructed on top. Flexible asphalt pavements were used throughout.

Client: Public Works Department, Thailand.

Consultant: Norconsult Civil Engineering Co., Ltd, Thailand.

Contractor: Sumitomo – Italian Thai J/V, Thailand.
Basal reinforced embankments on piles: A1/N1 dual carriageway, Dundalk to Newry, Ireland

A section of the A1/N1 dual carriageway between Dundalk and Newry, forming the cross border link between the Republic of Ireland and Northern Ireland, has recently been constructed. The project was faced with many challenges, one being the crossing of the Flurry bog, a large peat bog combined with very soft silts of almost 1 km in length, with depths ranging up to 9 m. Beneath the peat bog was a firm stratum of gravel overlying rock.

The Flurry bog is low-lying, with groundwater levels at ground surface. The area is subject to periodic flooding from the adjacent Salmonoid River, and the area resembles more of a wetland than a bog. The peat has limited fibre strength making surface access very difficult, even on foot.

Due to the variable depth of peat along the highway alignment, two different foundation treatments were proposed to construct the dual carriageway embankments. In areas where the peat depth was relatively shallow the peat was excavated and replaced with granular fill. In areas where the peat depth could not economically justify this approach (along a 400 m length) a basal reinforced piled embankment solution was used, with the piles driven into the firm gravel stratum beneath the peat bog.

The basal reinforced piled embankment was designed according to BS8006, an internationally recognised design code, with consideration given to the variation in fill height along the length of the piled embankment. Due to alignment constraints, the embankment height approximated 3 m over the pile caps. Because of this low height, it was decided to preload the embankment with 1 m of surcharge in order to pre-strain the basal reinforcement, thereby reducing long term localised deformations in the embankment.

The final design incorporated Geolon® PET geotextile reinforcement across the tops of the pile caps. Depending on the embankment height two different geotextile reinforcement combinations were used. One combination consisted of 600 kN/m longitudinal and 700 kN/m transverse strengths, while the second combination consisted of 700 kN/m longitudinal and 800 kN/m transverse strengths. These strengths were determined based on allowable design loads, strains and required design life.

In order to gain access to the site, a working platform needed to be constructed. As the foundation soil was very weak, a Polyfelt® TS80 geotextile/Miragrid® GX35 geogrid stabilisation layer was placed in order to construct a reinforced working platform across the base area of the planned piled embankment. The geotextile/geogrid combination provided the strength and stiffness required for stability, allowed quick dissipation of seepage groundwater, and minimised the required thickness (and hence weight) of the working platform.

Approximately 2,700 precast concrete piles, spaced on a square 2.5 m grid, were installed for the embankment support. These piles were driven up to 3 m into the firm gravel stratum beneath the peat bog. Pile caps of 0.8 m square were cast on top of the installed piles and then the fill in the working platform was raised to coincide with the top of the pile caps.

The appropriate Geolon® PET geotextile reinforcement was then laid out across the top of the pile caps with the lower strength material placed longitudinally along the embankment alignment and the higher strength material placed transversely across the embankment alignment. Geotextile joins were made by simple overlap with the overlap amount established in order to meet load transfer requirements.

Granular fill obtained from a cutting further along the highway alignment was used to construct the embankment. This was placed and compacted to meet geometrical and compaction tolerances, including the 1 m surcharge. After 6 months the 1 m surcharge was stripped off and the pavements were constructed.

A comprehensive embankment monitoring program was performed. The results have shown that throughout the monitoring period the surface of the embankment has not settled at all. Settlement recordings at the base of the piled embankment show no settlements on top of the pile caps, with settlements of up to 100 mm on the geotextile reinforcement mid way between the pile caps. These results demonstrate a key feature of this technique where the basal geotextile reinforcement deforms between the pile caps thereby transferring the un-arched embankment loading onto the pile caps.

Client: DRD, Northern Ireland and Louth County, Ireland.
The M74 Completion project comprises the last stage of completing the motorway network in the Glasgow area. The project is 8.5 km in length, and continues from the existing M74 Motorway at Fullarton Road to the M8 Motorway south west of Kingston Bridge near Glasgow City Centre. The route comprises 4 major grade-separated junctions including a large motorway viaduct which is over 750 m long.

The M74 Completion project crosses predominantly brownfield land, some of which is heavily contaminated by past industries. Consequently, the design favoured above ground construction with only a small length of cut. Further, the motorway alignment lies on the southern side of the Clyde River where deep layers of soft alluvial clay foundation soils predominate. These clays range between 12 m to 35 m in depth, and overlie dense sand, glacial till and rock. Because of a tight construction schedule, extensive use had to be made of foundation treatment techniques.

Prior to the start of the contract a number of old industrial buildings along the alignment of the intended motorway were demolished. For environmental reasons, the building and foundation rubble was recycled by crushing and reusing within the embankments, and was not disposed of in landfill. The recycled material was also used in the construction of the piling platforms. Threading the route through an urban location meant isolated work sites and although much of the route was dereelct, numerous businesses were affected.

To enable rapid construction and prevent differential settlements, the 13 approach embankments to the grade-separated junctions have piled foundations with basal geosynthetic reinforcement. Each approach embankment was designed specifically in terms of pile spacing, pile cap size and basal geosynthetic reinforcement strength and extension. The design of the basal reinforced piled embankments was carried out in accordance with BS8006, an internationally recognised design code.

To support the approach embankment loads, 275 mm square precast reinforced concrete piles were used throughout for the foundations of the piled embankments. These were driven into the variety of formation types to depths of between 12 m to 35 m. The spacing’s between the piles varied according to the embankment loads, and ranged from 1.5 m to 2.2 m on a square grid at the different approach embankment sites.

Once the piles had been installed, they were capped with circular, cast in situ, concrete caps of 900 mm diameter. Next, between 0.05 and 0.15 m thickness sand infill was placed and spread to bring the ground level up to 0.05 m above the pile caps. This sand infill layer provided a smooth bedding for the Geolon® PET geotextile reinforcement across the tops of the pile caps.

Geolon® PET geotextile reinforcement was placed in two layers, at right angles to each other, across the tops of the pile caps over the base of the approach embankments. One layer of Geolon® PET geotextile reinforcement, with tensile strengths ranging from 100 kN/m to 200 kN/m, was placed along the length of the approach embankments. The second layer, with tensile strengths ranging from 400 kN/m to 1,600 kN/m, was placed across the width of the approach embankments. At the extremity of the piled embankment foundation, the geotextile reinforcement layer was wrapped around a rectangular anchor block and brought back into the embankment fill a required distance in order to develop adequate frictional bond resistance to support the tensile loads generated across the outer piles. The Geolon® PET geotextile reinforcement used is made of high modulus, high strength polyester yarns, resulting in a reinforcement material that combines high long term strength with low extension and low creep characteristics.

Once the Geolon® PET geotextile reinforcement had been installed, Zone F high friction granular fill material was then placed over the top to a thickness varying from 1.5 m to 2.5 m. Following this, general Class 1/Class 2 fill was used for the remainder of the embankment construction. As part of the monitoring process a regime of subgrade and surface monitoring at the pile/structure interface was included.

Finally, the pavement capping layer and base course and surface layer s were constructed, along with the necessary traffic ancillary items.

In other sections along the length of the motorway, foundation treatments included the use of Prefabricated Vertical Drains (PVD’s) to accelerate the rate of consolidation of the soft alluvial foundation deposits. Here, embankment fills ranged up to 9 m in height. To ensure that the majority of settlement occurred within the tight construction schedule, the PVD’s were installed through the base drainage layer on a 2 m square grid to the bottom of the soft foundation layer, which in some locations was as great as 35 m. This enabled 90% of primary consolidation to occur within 3 months of embankment construction. To ensure short term embankment stability Geolon® PET geotextile reinforcement with strengths of 200 kN/m to 1,000 kN/m was installed across the width of these embankments on top of the base drainage layer prior to the placement and compaction of the embankment fill material.

Funders: Transport Scotland, Glasgow City Council, South Lanarkshire Council, Renfrewshire Council, UK.

Client: Glasgow City Council, Glasgow, UK.

Principal Contractor: Interlink M74 JV comprising Morrison Construction, Morgan Est, Balfour Beatty and Sir Robert MacAlpine, UK.
The high speed railway line LGV Est is to connect Paris with the East of France (to Strasbourg) and then on into Germany to connect to the German high speed rail network. As such, it forms an integral part of the European-wide high speed rail network. Once completed, LGV Est has over 300 km of new constructed track, with trains travelling at speeds of 320 km/hr.

During construction of this high speed railway line, near La Croix-sur-Meuse in the East of France, the contractor discovered cavities in the foundation of a long cutting along the proposed track alignment. The cavities existed in a karst limestone layer that coincided with the base of the high speed track structure. These cavities consisted of fractures in the upper surface of the limestone anticline. The width of the joints in these fractures ranged between 0.15 m and 0.20 m.

To maintain the high speed performance of the rail track over these cavities it was necessary to arrive at a technique to neutralise the effects of their presence. Various design alternatives were investigated. The best design option arrived at was to include geosynthetic reinforcement to span across any potential foundation voids, and thus minimise any resulting surface differential deformations.

From a design perspective, it was assumed that the maximum possible cavity diameter that could develop was 0.5 m. Further, to maintain train speed, it was determined that the maximum surface deformation be limited to a maximum of 1 mm below the ballast level over any 0.5 m diameter void. From the viewpoint of earthworks construction it was also important to limit the thickness of the structural fill above the karstic limestone foundation. The French design method RAFAEL was used to analyse the problem and to provide an acceptable solution in terms of structural fill thickness and geosynthetic reinforcement strength and tensile stiffness.

From the design method a structural fill thickness of 1.05 m was chosen, along with Bidim® PPC75-75 geocomposite reinforcement as the corresponding basal reinforcement as the geocomposite reinforcement as the corresponding basal reinforcement, as it met the tensile strength and tensile stiffness requirements. Bidim® PPC75-75 geocomposite reinforcement is composed of high modulus polypropylene yarns and has an ultimate tensile strength of 75 kN/m in both the longitudinal and cross directions. It was considered that polypropylene would provide better long term durability in the higher pH conditions associated with the presence of lime and limestone.

The surface of the karst limestone layer was graded to the required foundation level. Across the top a layer of Bidim® PPC75-75 geocomposite reinforcement was placed running longitudinally through the railway cutting. The geocomposite reinforcement was laid all the way to the edges of the cutting to ensure that maximum geosynthetic reinforcement bond resistance could be achieved.

Across the top of the geocomposite reinforcement a 0.5 m thick lime-stabilised compacted platform was constructed. This platform consisted of fine-crushed limestone stabilised with 5% lime. This compacted layer was extended out to the extremities of the geocomposite reinforcement to provide maximum bond development coverage.

Following this, a 0.35 m thick granular capping layer was placed and compacted over the lime-stabilised platform. Then a 0.2 m thick granular subbase layer was placed and compacted. The total thickness of the lime-stabilised platform, granular capping layer and granular subbase layer being the required minimum thickness of 1.05 m.

Above these layers the rail ballast (approximately 0.25 m thick) and railway track structure were placed and tamped. LGV Est is now operating with trains running at 320 km/hr between Paris and Eastern France.

Client: S.N.C.F., Paris, France.
Contractor: GTM - Dechiron, Paris, France.
Basal reinforced embankments spanning voids: High speed railway track over karst foundation, Guadalajara, Spain

The construction of the Spanish high speed train from Madrid to the French border began in 1999 and is due to be completed before 2012. The length of the line is around 800 km and it passes through several important cities such as Guadalajara, Zaragoza, Lerida and Barcelona. The geological profile along the length of the track varies considerably, and in some areas the foundations are prone to collapse due to the presence of karst formations of limestone and gypsum soils.

In the area of Guadalajara, around 90 km northeast of Madrid, an exhaustive examination of the limestone foundation strata confirmed the presence of cavities due to the dissolution of the limestone. Depending on the diameter of the cavities, three different corrective techniques were employed: refill the cavities with concrete grout; excavation and construction of concrete slabs; and the use of geosynthetic reinforcement. All corrective techniques used had to ensure that no discernable surface deformations would occur if cavities later formed beneath the track support layers.

The geosynthetic reinforcement solution was employed in railway cuttings in this area where the likely foundation cavity diameters were quite small (≤ 0.5 m). Here, the limestone foundations were over-excavated by 1 m and a layer of Polyfelt® PEC100-100 composite reinforcement was placed over the excavated area. Polyfelt® PEC100-100 is composed of high strength polyester yarns in a composite structure and has an ultimate tensile strength of 100 kN/m in both the length and cross directions. Following placement of the Polyfelt® PEC100-100 composite reinforcement the excavation was refilled and compacted with granular fill material. This reinforced support platform prevented any differential surface movements from occurring if foundation cavities formed beneath the composite reinforcement layer.

Across the top of the reinforced support platform a capping layer 0.35 m thick was placed and compacted. On top of this a HDPE geomembrane layer was installed and directly connected to the surface drainage trenches on either side of the track structure to prevent surface water infiltrating into the karst foundation from above. Finally, the track subbase and ballast layers were laid, and then the track itself.

The high speed railway between Madrid and Zaragoza was brought into service in 2003, covering a distance of 318 km in 1 hour and 30 minutes. The first train arrived in Barcelona from Madrid in February 2008 covering a distance of 620 km in 2 hours and 30 minutes.

Other areas along the high speed railway route were also prone to the formation of foundation cavities, e.g. in the Figueres - Perpignan sector. Here, the size of the cavities encountered was greater than that in the Guadalajara area, and consequently, a higher strength and stiffer geotextile reinforcement was used in the reinforced support platform. Geolon® PET300 geotextile reinforcement with an ultimate tensile strength of 300 kN/m was used in this location.

Client: Gestor de Infraestructuras Ferroviarias, Madrid, Spain.
Consultant: Prointec, Madrid, Spain.
Contractor: Ute Gajanejos, Madrid, Spain.
RCD Español de Barcelona is one of the oldest La Liga football clubs in Spain. The construction of a new stadium to replace the existing one began in 2005. The aim was to create a football stadium that was safe and modern, and blended in well with the surrounding community. The proposed site for the new stadium was in the city of Cornella, close to Barcelona city.

A geotechnical investigation of the proposed stadium site showed that there was a substantial stratum of anthropic material varying in thickness between 4 m and 13 m over the site. This anthropic material was the remains of an old solid waste landfill site that consisted of solid industrial and construction waste (but no organic waste). This stratum has a history of localised differential settlements and collapse due to differential movements between the various solid waste components in the stratum. The soil layers above this anthropic stratum have a history of collapsing and forming sinkholes, or surface depressions, when subjected to groundwater entry. As the proposed stadium will consist of large areas where surface water can enter the foundation it was recognised that special treatment of the foundation would be required in order to prevent potential sinkholes and surface depressions from forming after completion of the stadium and its surrounding area.

A detailed analysis of the likely magnitude of the sinkholes and depression formations due to the anthropic stratum was carried out with 4 m diameter being the maximum likely. This size of void was used as the basis for the design of the foundation platform.

At the ground surface it was judged that the maximum allowable differential deformation be limited to 2% for any void forming in the anthropic stratum up to 4 m in diameter. This allowable differential deformation level is consistent with that allowed for road pavements, and was also considered appropriate for high quality football fields. To meet this surface deformation requirement three complimentary components were required for the foundation beneath the football field and surrounding areas. First, there had to be a basal reinforced support system in the foundation. Second, there had to be an impermeable layer to prevent surface water entering the foundation and triggering a collapse.

The treatment of the anthropic stratum was carried out as follows. Any anthropic material that was within 4 m of the final ground surface was removed. Next, a 0.5 m thick compacted clay capping layer was placed directly across the top of the anthropic stratum. This provided a local sealing layer as well as a bridging layer.

A 1.2 m thick compacted sand reinforced platform was constructed on top of the clay capping layer. This platform was designed to meet the surface differential deformation requirements of the foundation, given the likely design void diameter (4 m) in the anthropic stratum. This was calculated to require 5 layers of Geolon® PET600 geotextile reinforcement installed orthogonally (at right angles) to each other. In the upper level a single layer of Geolon® PET600 geotextile reinforcement was installed. Heavy compaction of this sand layer was specified in order to achieve the high density of 100% standard Proctor.

Above the reinforced platform was placed and compacted a 1.9 m thick layer of granular fill. This material was also subject to heavy compaction in order to achieve maximum dry density and deformation modulus greater than 20 MPa.

A 1.5 mm thick HDPE geomembrane was installed across the top of the compacted granular fill layer. This geomembrane fed into the surface and subsurface drainage systems in the stadium, and prevents surface water infiltration into the foundation layers.

Above this was placed and compacted a 0.4 m thick soil layer that also included the football stadium grass package on top. Included in this layer is the football field drainage system.

The stadium was completed in 2009 and is the most modern in all of Spain. It has since been awarded the top 4-stars rating by UEFA, the European Football Association.

Client: RCD Español de Barcelona SAD, Barcelona, Spain.

Consultant: Reid Fenwick Asociados y Gasulla Arquitectura y Gestión, Barcelona, Spain.

Contractor: FCC Construcción – Copisa JV, Barcelona, Spain.

Basal reinforced embankments spanning voids: Football field over old landfill, Barcelona, Spain

Football stadium following completion

Installing Geolon® PET600 geotextile reinforcement across the base of the excavation

Client: RCD Español de Barcelona SAD, Barcelona, Spain.

Consultant: Reid Fenwick Asociados y Gasulla Arquitectura y Gestión, Barcelona, Spain.

Contractor: FCC Construcción – Copisa JV, Barcelona, Spain.
During the late 19th and early 20th centuries the Wallasey Docks area on the River Mersey was one of Europe’s largest cattle handling facilities. However, by the end of the 1960’s this trade had seriously declined, forcing partial closure of the Docks. In 2001 it was decided to construct a new roll-on, roll-off ferry terminal on the site of the disused docks.

The dock basin has dimensions of 500 m by 100 m. The depth of the basin is between 2 m and 12 m, however, because of its disuse, there is up to 10 m depth of very soft sediments within the dock basin. A number of options were investigated on how to deal with these very soft sediments, including complete removal. The best option was to leave the sediments undisturbed, cap them, and fill the dock basin to the required height with sand fill. A geotextile reinforcement solution gave the best opportunity to cap the sediments in an undisturbed manner. The prime concern was how to spread the geotextile reinforcement across the dock basin without disturbing the soft sediments.

The water level in the dock basin was normally maintained at a low level, varying from below sediment surface level near the edges of the basin, to around 1.5 m depth in the natural drainage channel that had formed within the basin. It was decided to raise the level of the water in the dock basin and float the geotextile reinforcement across thereby leaving the soft sediments undisturbed. The water level in the dock basin was raised to around 1 m below the edges of the dock by blocking off the water exit culvert from the dock.

Geolon® PP120S was chosen as the geotextile reinforcement for two major reasons. First, the material has a tensile strength of 120 kN/m in both the length and cross directions, and low extension, and this would make the geotextile strong enough to resist any localised instability in the soft sediments that might occur. Second, the material was composed of 100% polypropylene and thus had a very low gravity less than water, and hence, would float naturally.

It was planned to float the Geolon® PP120S geotextile reinforcement across the 100 m width of the dock basin, using ropes to pull the geotextile sheets across. The geotextile was fabricated into 120 m long by 15 m wide sheets in the factory. The length of the geotextile panels were long enough to span the 100 m of the dock basin as well as to have around 10 m extra length on either side of the basin to account for later settlements of the soft sediments, etc.

On site the Geolon® PP120S geotextile reinforcement sheets were fabricated into wider panels at the edge of the basin with the ropes introduced at every 15 m join. The geotextile panels were then pulled across the flooded dock basin using the ropes connected to excavators on the opposite side of the basin. Once all the geotextile panels had been pulled across the water level in the dock basin was then lowered back to its natural level.

Marine sand dredged from the River Mersey was used as the fill material to bring the dock floor up to its desired level. Because of the low undrained shear strength of the soft sediments this filling was carried out in several layers in order to maintain stability. First, a sand layer varying in thickness from 400 mm to 700 mm was spread across the geotextile, and down to the base of the dock.

It was estimated that the settlement of the fill would approximate 1.4 m. To achieve this in a short period of time Prefabricated Vertical Drains (PVD) were installed over the dock basin at 2.1 m centres. These were installed from on top of the surcharge layer, down through the geotextile, and down to the bottom of the soft sediments. After a period of 6 to 7 months the settlement requirements were met and the surcharge layer above dock level was removed.

The pavements for the roll-on roll-off terminal were subsequently constructed on top of the sand fill.

Client: Mersey Docks and Harbour Company Ltd, UK.
Consultant: Bullen Consultants Ltd, UK.
Contractor: AMEC Capital Projects Ltd, UK.
Reinforced soft site closures: Mine tailings pond closure, Huelva, Spain

The Sotiel Coronada mining complex is located at Calañas in the province of Huelva (Andalucia), Southwest Spain. This was a huge deposit of pyrite mineral that had been extracted over centuries of mining in the area. The main mineral extraction activity was the processing of copper, lead and zinc, as well as the production of sulphuric acid.

Following completion of mining and processing activities in the area, the environmental company, EGMASA of the Regional Government of Andalucia (Junta de Andalucia) was made responsible for the reclamation and closure of the large pyrite tailings pond (surface area around 35 ha) from September 2005 to April 2006.

The reclamation and closure of the tailings pond required the movement and placement of large volumes of tailings and subsequent fill. The tailings were very soft and extended to a depth approximating 18 m. In many areas the surface of the tailings were saturated, while in other areas there was a crust on the surface of the tailings, but very low shear strengths existed at shallow depth below the surface.

Due to the extensive area to be reclaimed, the large volumes of fill to be moved, and the tight time schedule of the Client, large volume, heavy earth moving equipment had to be used. This posed a major technical problem as the heavy equipment had to traffic over the very soft tailings. The solution was to construct a 1.0 m thick stone-fill working platform over a layer of Polyfelt® PEC400 geocomposite reinforcement. Polyfelt® PEC400 is composed of high modulus polyester yarns in a composite structure and has an initial tensile strength of 400 kN/m in both the longitudinal and cross directions. The Polyfelt® PEC400 also acts as a geotextile separator preventing the intermixing of the soft tailings with the stone fill. The selection of this geocomposite reinforcement was determined on the basis of a detailed stability analysis of support for the heavy earth moving equipment over the very soft tailings deposit. Different possible loading scenarios were investigated before arriving at the final solution. Further, consideration was also given to the good resistance of this geosynthetic material to the effects of installation damage caused by the placement of large stones in the working platform.

The Polyfelt® PEC400 geocomposite reinforcement was placed starting at the edges of the tailings pond (the high points) and working inwards to the centre (the low point). Care was taken to ensure that any run-off water would not contaminate the new working platform.

The stone fill was back-dumped from the earth moving equipment and then bulldozed over the top of the Polyfelt® PEC400 geocomposite reinforcement to ensure there was minimal disturbance of the soft tailings below.

Across the top of the working platform a 0.5 m thick layer of compacted clayey material was placed. This layer provided a smooth, stable platform for the subsequent placement of the geomembrane liner. To prevent rainfall seeping into the contained mine tailings and prevent contaminants rising to the ground surface a barrier consisting of a composite geomembrane/ compacted clay liner system was employed. This consisted of a 1.5 mm thick HDPE geomembrane cushioned between two layers of Polyfelt® TS80 geotextile protection, with a 0.5 m thick compacted clay layer on top.

To drain any percolating rainfall a 0.2 m thick granular drainage blanket was installed on top of the composite liner system. First, a Polyfelt® TS20 geotextile separator was placed to ensure no intermixing of the clay layer and the granular drainage blanket occurred. Then, the granular drainage layer was placed, and finally, a Polyfelt® TS20 geotextile filter was placed on top. After this, a 0.5 m thick soil covering was placed and the area vegetated. Extensive surface run-off drainage was also installed.

Client: Empresa de Gestion Medioambiental, S. A., Junta de Andalucia (EGMASA), Spain.

Consultant: Empresa de Gestion Medioambiental, S. A., Junta de Andalucia (EGMASA), Spain.

Contractor: Ferrovial Agroman, S.A., Madrid, Spain.
Reinforced soft site closures: Waste treatment sludge lagoons closure, Axis, Alabama, USA

In conjunction with the closure of a fibre cellulose plant that produced rayon fibre the State of Alabama Environmental Agency also required the closure of two associated waste treatment lagoons that contained rayon fibre residues. The waste treatment lagoons were each 180 m by 180 m in area, and consisted of sludge 4 m to 5 m deep. The sludge was very soft, and was estimated to have an undrained shear strength of less than 0.5 kPa, and people could not walk on it.

To effect closure of these two treatment lagoons a design was initiated which required geosynthetic reinforcement beneath a 1 m to 2 m thick sand fill layer. The geosynthetic strength requirement was determined using limit equilibrium analyses accounting for fill height, fill placement and construction equipment loadings. Further, a specific gravity of around 1.0 was required for the geosynthetic reinforcement in order to effect deployment across the water in the lagoons. The geosynthetic reinforcement chosen was Mirafi® HP770PET, a woven geotextile with high strength polypropylene yarns in the length direction (105 kN/m) and high strength polyester yarns in the cross direction (105 kN/m). The polyester yarns in the cross direction enables high cross directional strength and very good seam efficiencies. This combination of polypropylene and polyester yarns also resulted in a geotextile with a specific gravity of approximately 1.0, which facilitated its floating across the lagoons.

Before geotextile deployment was commenced a 300 mm depth of water was pumped into the lagoons. This enabled enough water to remain in order to float the fabricated Mirafi® HP770PET geotextile reinforcement sheet across each lagoon.

The 4.5 m wide panels of geotextile reinforcement were sewn together in the field at the dry edge of the lagoons. Special seaming techniques resulted in a seam strength of over 50 kN/m, which was determined to be enough to withstand installation and insitu stresses. The geotextile panels were sewn together in concertina fashion to create a single sheet to cover the whole lagoon.

To ensure the leading edge of the geotextile reinforcement remained above water as it was pulled across the lagoon surface, polystyrene blocks were sewn into pockets in the leading edge of the geotextile. Polyester webbing was sewn to the geotextile to enable nylon ropes to be connected to the leading edge of the geotextile fabric. Equipment available on site (excavators and bulldozers) were used to pull on the nylon ropes, deploying the Mirafi® HP770PET geotextile reinforcement sheet across the lagoon. It took around 3 hours to completely deploy the geotextile sheet across each lagoon.

Once the geotextile sheet completely covered the lagoon, a small key trench 0.5 m wide by 0.5 m deep was dug all around the lagoon. The geotextile reinforcement was placed in the trench and then the trench backfilled.

Locally available, fine sand was used as the closure fill. Prior to placement of the fill, 100 mm diameter PVC pipes were placed at 15 m intervals across the geotextile and then tied to a header which drained the collected liquid to a lagoon wet well where the liquid was pumped back to the plant site. A light weight swamp dozer was used to spread an initial fill lift of 1 m across the lagoon.

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The use of the Mirafi® HP770PET geotextile reinforcement enabled the closure fill to be placed by conventional earthmoving equipment. The alternative would have been hydraulic placement of fill, or placement of fill using a conveyor system. Either alternative approach would have presented difficulties and been more costly.

Client: OLT of Alabama, USA.
Consultant: Environmental Strategies Consulting, Pennsylvania, USA.
Geosynthetics Consultant: Gale-Tec Engineering Inc., Minneapolis, Minnesota, USA.
Contractor: Remediation Services, Texas, USA.
Reinforced fill slopes: Road realignment, Rodlauer Bridge, Styria, Austria

Due to increased traffic loadings and seepage problems, a section of the B115 highway in the mountains of Styria, Austria was showing considerable distress, and was in danger of failure. The highway is used mainly by heavy trucks hauling timber, and restrictions had to be placed on these vehicles after large deformations were observed in the vicinity of an old 10 m high masonry retaining wall. The local authority was under pressure to find a technically viable solution to the problem while keeping within a limited budget.

One solution investigated was the provision of a bridge to span across the unstable area, but this was rejected as being too costly. Further, conventional retaining wall solutions were found to be non-viable because of the large height involved (34 m) and limited base area available for the wall.

A value solution was found by using a geosynthetic reinforced steep slope to realign the highway, making it more stable, and improving traffic safety. A number of important aspects had to be accounted for at the design stage. These included the provision of a natural looking slope face to blend in with the surrounding environment; the steep slope had to be constructed within a steep "V" shaped incline in the hillside that limited the extent of the layers of reinforcement and made construction difficult; and the cost had to remain within the authority budget.

Experience in the region had shown that to provide a permanent vegetated slope face, which did not require irrigation, limited the slope face angle to a maximum of 2V:1H. This maximum slope face angle, together with the realignment of the highway, made the requirement for the slope to be 34 m in height. Further, the steep incline of the existing hillside limited the embedment lengths of the reinforcement layers because of the existence of a rock stratum at, or near to, ground surface.

A design analysis of the steep reinforced slope was performed using a limit equilibrium approach taking due account of National design standards. Because of the limited base width available, good quality granular material had to be used throughout as the reinforced fill and this was compacted to good density specifications.

Depending on the location within the steep reinforced slope, layers of Miragrid® GX200/30 and Miragrid® GX110/30 geogrid reinforcement, having ultimate tensile strengths of 200 kN/m and 110 kN/m respectively, were used throughout. Miragrid® GX geogrid reinforcement is composed of high modulus, high strength polyester yarns encased within a robust PVC coating, and has excellent strength, extension and durability properties.

At the toe of the steep slope it was not possible to provide a graded earth foundation base, so a concrete foundation block was constructed into the rock stratum. This ensured a stable toe foundation for the slope.

To form a smooth surface at the slope face a steel mesh facing system was used. The steel mesh was bent to the required 2V:1H face angle and consisted of units 0.5 m high, which coincided with the vertical spacing's between the geogrid reinforcement layers. This steel mesh facing was relatively flexible which enabled settlement of the fill during slope construction without creating undue deformations of the slope face.

Inside the steel mesh facing an erosion protection grid made of glass fibres was installed. The role of this glass grid is to protect the soil face from surface erosion until surface vegetation growth has been established. The glass grid also provides long term local stability to the slope face.

Immediately behind the steel mesh and glass grid facing good quality top soil was placed to enable good vegetation growth, followed by the placement and compaction of the granular reinforced fill.

It had been observed that substantial quantities of groundwater seeping out of the hillside had contributed to the failure of the existing stone masonry wall. Consequently, extensive drainage measures were installed in the new slope to manage this groundwater seepage in a controlled manner. In the lower part of the slope a geocomposite drainage layer was installed at the rock face. In the upper part of the slope gravel drainage materials were used to intercept groundwater flows. The seepage water was then channelled by drainage pipes through the reinforced fill and out through the face of the reinforced slope where it was discharged into the adjacent River Enns.

The value of this reinforced slope solution has proven to be very good, with its cost being around 50% of the cost of the originally proposed bridge solution.

Client: Steiermärkische Landesregierung FA 18B, Austria.
Consultant: Ing.- Büro Eisner ZT GmbH, Graz, Austria.
Supervisor: Steiermärkische Landesregierung BBL Liezen, Austria.

Contractor: Lang and Menhofer, Liezen, Austria.

Reinforced Fill slope near completion

Construction in a confined space near the base of the reinforced slope

Steel mesh facing used to form a smooth slope face which aids in vegetation growth and aesthetics

Typical cross section through the reinforced slope at maximum height

Reinforced Fill slope 2 years after construction

Existing highway and proposed new highway alignment

Existing highway alignment

New highway alignment

Vegetated face

Granular reinforced fill

Geocomposite drain

Drainage pipe

Concrete foundation

Drainage pipe

Granular reinforced fill

Existing masonry wall

Up to 34 m

Rock stratum

Geogrid reinforcement at 0.5 m vertical spacings

Typical cross section through the reinforced slope at maximum height

Landesregierung BBL Liezen, Austria.
Langkawi is an island located around 30 km of the north-western coast of Peninsula Malaysia. It is a top tourist destination in the country and in June 2006 was accorded Geopark status by UNESCO.

Gunung Raya, at 811 m above sea level, is the highest peak in Langkawi. It is accessible right to the top using Federal Route 278, known as Jalan Gunung Raya. The peak houses a museum, a park and a satellite base station, and also offers spectacular panoramic views of the surrounding coastline, especially during sunset. It is a very popular tourist location.

In 2003 after heavy rain a major landslide occurred around the mid-section of Jalan Gunung Raya on the upper side of the road. The landslide exposed the granite outcrop and the debris completely cut off the road access. After reviewing several remedial options the decision was made to realign the road away from the failed slope as a permanent solution. A reinforced fill slope structure was designed to support the realigned road located 30 m away from the original alignment. The landslide exposed the granite outcrop on the upper side of the road and hence there was no danger of further landslides occurring again in this location. However, precarious parts were trimmed and a gabion debris control structure was constructed to retain the occasional debris flow.

The reinforced fill slope structure was designed to have a slope facing of 1.20:1H, built up in 6 m high reinforced soil tiers. At the toe of each tier a surface water catchment drain was located to drain away surface water from the slope face.

The landslide debris was removed from the site to clear the road and provide an adequate zone for the new reinforced slope. At the rear of the excavated zone a subsurface drainage layer was provided behind the reinforced soil zone to intercept ground water seepage from the natural strata. A Polyfelt® TS10 geotextile filter was used as the filter for this subsurface drainage layer. At the toe of the slope a low gabion structure is incorporated to enhance toe stiffness.

Polyfelt® PEC150 geocomposite reinforcement was used to reinforce the fill in the slope. This geocomposite reinforcement has a tensile strength of 150 kN/m. The length of the reinforcement was maintained at a constant 20 m throughout the height of the slope for construction simplicity, except for the upper tier where the reinforcement length was 24 m. The vertical reinforcement spacings varied between 0.6 m and 1.2 m depending on the vertical location in the slope. Soil-filled bags were used as forms to shape the steep slope profile and enable the compactor to work close to the slope face for good compaction of the reinforced fill. These soil bags also fulfilled the role of a growing medium for the slope vegetation following completion of the slope construction.

Sandy soil from a nearby borrow area was used as the reinforced fill, and this was placed in lifts and compacted using a 10 tonne roller to achieve a minimum of 90% of standard Proctor density. A green coloured geotextile mesh was used as a wrap-around (around the soil-filled bags) on the slope surface. This geotextile mesh reinforces the root matter of the slope vegetation and prevents surface erosion.

Once construction of the reinforced slope had been completed, hydroseeding was carried out to establish surface vegetation. It only took several weeks for the surface vegetation to become fully established.

Client: Public Works Department, Kuala Lumpur, Malaysia.

Consultant: KGA Consultants Sdn Bhd, Kuala Lumpur, Malaysia.

Contractor: Protab Construction Sdn Bhd, Langkawi, Malaysia.
Reinforced fill slopes: Slope restoration, Maehongson, Thailand

Northern Thailand, which is hilly, experiences distinctive wet and dry seasons throughout the year. Here, the Province of Maehongson experiences mean annual rainfall of about 1200 mm, the vast majority of which falls in the wet season between April and October. During the wet season, there are many failures of earth slopes and hill sides.

In this area, roads are often carved into slopes in some locations while the embankment sections are filled with the excavated soil. The fills that support the roads normally have little compaction. Consequently, during the wet season groundwater flows easily penetrate these fills, causing instability, with many slope failures resulting.

Pang Oong, in Maehongson Province, is a village that is accessible only by road through hilly terrain and is rated one of the most romantic tourist destinations in Thailand, known for the beautiful misty lake and mountain pines located adjacent to the village. A major fill slope failure occurred along the access road into the village during a period of heavy rainfall. An expedient solution to repair the slope failure using gabions was initially implemented as there was an urgency to protect the half of the road that had not failed. However, this solution also failed with a second slip failure occurring after further heavy rain. The client then decided to adopt a long term solution by fully restoring the failed slope section.

The most important constraint for the design of the slope restoration was that the single lane of the road remaining after the slope failure had to be kept functional as this road was the only access to many villages in the area. This also meant that any design involving excavation that could further jeopardize the integrity of the road above had to be excluded.

The final design resulted in a slope restoration in two parts. The lower part was rebuilt at a shallow slope angle of 1H:1.5V, with compacted residual soil benched into the existing good ground. This was done such that a 15 m wide platform would be created for the construction of the 13 m high upper part of the slope consisting of reinforced fill. Within the fill in the lower part of the slope horizontal drainage pipes were installed to drain out any accumulating groundwater at the rear of the compacted fill zone. At the base of the reinforced fill slope a horizontal drainage blanket was constructed using single sized aggregate sandwiched between two layers of a Polyfelt® TS geotextile filter.

The upper part of the restored slope consists of 3 benched tiers of reinforced fill, each having a slope face angle of 2V:1H. The lower tier is 4 m high and reinforced with a combination of 2 layers of Miragrid® GX300 geogrid reinforcement, 4 layers of Miragrid® GX250 geogrid reinforcement and 2 layers of Miragrid® GX130 geogrid reinforcement. These lower tier reinforcements span 13 m in length. Miragrid® GX geogrid reinforcements are composed of high strength, high modulus polyester yarns within a robust polymer coating.

The middle reinforced fill tier is 6 m high and reinforced with a combination of 4 layers of Miragrid® GX130 geogrid reinforcement and 8 layers of Miragrid® GX80 geogrid reinforcement. These middle tier reinforcements span 8.5 m in length. At the midlevels of the primary geogrid vertical spacing, Miragrid® GX80 geogrid reinforcement is used.

The upper reinforced fill tier is 3 m high and reinforced with 3 layers of Miragrid® GX80 geogrid reinforcement of 5 m reinforcement length.

At the face of the reinforced slope, the geogrid reinforcements are wrapped around soil bags and tucked back into the slope at the next reinforcement level. The soil bags serve as forms to shape the steep slope profile and enable fairly heavy compaction to be applied close to the slope face. The jute bags also serve to prevent surface erosion during the initial phase of surface vegetation.

To monitor the performance of the reinforced slope the Geodetect® fibre-optic strain monitoring system was incorporated into the slope with the geogrid reinforcement. The monitoring results 7 months after construction showed that horizontal strains were small, less than 1%. At 15 months after construction there was negligible difference in the horizontal strains.

Client: Bureau of Highway 1, Chiangmai, Thailand.

Contractor: Phatthanapanaphap Construction Co., Chiangmai, Thailand.

Typical cross section through the reinforced slope

Installation of the drainage blanket at the base of the reinforced slope

Spreading locally obtained reinforced fill over Miragrid® GX geogrid reinforcement

Completed reinforced slope with vegetation growth on the face
Reinforced fill slopes: Road widening, Chiangmai, Thailand

Chiangmai is the major city of Northern Thailand. Wat Phrathat (or Phrathat Temple) is Chiangmai’s most famous Buddhist temple and a major tourist attraction. It is located 17 km from Chiangmai in hilly terrain along Srivichai Road, which was first built in 1935. This is a steep road that aligns with the surrounding hilly terrain. Over the last few years the road has been upgraded and widened to cater for increased traffic and improved road safety standards. Specific locations along the route required specialist geotechnical solutions, including reinforced soil technology, to execute road widening works due to the rugged terrain.

One section involved the straightening of the existing road alignment. The most important constraint for the design was that the road had to remain open as this road was the principal access from Chiangmai to Wat Phrathat as well as to Phu Ping Palace and the surrounding green environment.

Due to space limitations, temporary steep excavations were necessary during construction. So as not to jeopardize the integrity of the existing road, soil nails were installed in a 1.5 m square grid pattern to provide stability to the steep excavations necessary for the reinforced fill structure to be constructed. Also, in this location a deep ravine had to be partially filled to allow for the reinforced fill structure to be constructed on top. To prevent differential settlements between the fill and the cut ground from damaging the reinforced soil structure the designer decided to provide a piled foundation support. Precast concrete square piles were driven 4 m into the ground and capped with a reinforced concrete raft prior to constructing the reinforced soil structure.

Above the concrete raft a drainage blanket was installed consisting of single-size stone wrapped with a Polymat® EM4 erosion control mat. This drainage blanket was wrapped around the soil bags and tucked back into the slope at the next reinforcement level. The soil bags also enable fairly heavy compaction of the reinforced fill close to the slope face without undue face deformations. The jute bags also serve to prevent surface erosion during the initial phase of surface vegetation. Once completed, rapid vegetation occurs on the reinforced slope face.

The local residual soil was used as the reinforced fill and was compacted in 0.25 m thick lifts. Once compacted, reinforced fill slope option was determined to be the most practical and cost effective as well as taking the least time for construction. The geogrid reinforced fill slope also has the advantage of allowing a vegetated surface to blend well with the surrounding green environment.

Soil bags were used to form the face of the reinforced fill slope. The Miragrid® GX100 geogrid reinforcement was wrapped around the soil bags and tucked back into the slope at the next reinforcement level. The soil bags also enable fairly heavy compaction of the reinforced fill close to the slope face without undue face deformations. The jute bags also serve to prevent surface erosion during the initial phase of surface vegetation. Once completed, rapid vegetation occurs on the reinforced slope face.

The regraded toe slope formed by infilling the ravine was covered with a Polymat® EM4 erosion control mat to prevent surface erosion during construction and to aid rapid vegetation growth for long term erosion resistance.

Client: Department of Highways, Thailand.

Contractors: Jirangkorn Co., Ltd. and One Development Co., Ltd., Chiangmai, Thailand.

Soil nailing to maintain stability of existing road foundation.
Reinforced fill slopes: Avalanche protection barrier, Diasbach, Tyrol, Austria

Historically, the side of the Diasbach alp in the Tyrol region of Austria had been prone to avalanches during Winter. These avalanches have caused major damage to the resort village of Kappl located in the valley below. A solution had to be found to prevent this problem from reoccurring in the future.

Avalanche protection systems are complicated by their high altitude location and the difficult terrain in which they must be engineered. Primary protection measures, such as foresting, are designed to prevent initial avalanche movement. Secondary protection measures, such as dykes and retaining walls constructed across the path and/or along the peripheries of avalanche movements, are designed to withstand the forces of avalanches with the aim of restricting, splitting and/or deviating the already moving mass. Secondary protection barriers can be constructed quickly provided the proposed site is accessible to construction equipment.

To contain future snow, ice and debris slides and direct them away from the resort village of Kappl below, it was decided to construct a large avalanche protection barrier across the side of the Diasbach alp. This option was considered to be the only feasible and cost-effective solution. The protection barrier had a total length of 650 m and a maximum height of 28.5 m, making it the largest structure of its type in Austria. The upward side of the barrier is reinforced at a slope of 2V:1H while the downward side is a natural slope of 2V:3H.

The construction of the barrier was carried out during the Summer months only (June to September) over a 4 year period. The first 5 m in height of the barrier was constructed using rock blocks in order to provide a stable foundation and adequate resistance and hydraulic conductivity for the large water flows emanating from the snow melt and passing along the barrier during the Spring thaw seasons.

The reinforced soil system used on the upward side of the avalanche protection barrier above the rock block platform consisted of 0.5 m high steel mesh facing units angled at 2V:1H with layers of Miragrid® GX geogrid reinforcement at 0.5 m vertical spacings. The 0.5 m high steel mesh facings coincided with the 0.5 m geogrid vertical spacings. In the lower part of the slope the Miragrid® GX geogrid reinforcement extended 14 m into the slope, while in the upper part the geogrid reinforcement extended 10 m into the slope.

Immediately behind the steel mesh and glass grid facing good quality top soil was placed to enable vegetation growth, followed by the placement and compaction of the granular reinforced fill. The granular reinforced fill was obtained locally as part of the avalanche protection barrier earthworks.

Because of the high altitude of the site special techniques were adopted, such as improving the durability of the steel mesh facings and using altitude-resistant plant mixtures on the reinforced slope facing. Studies were undertaken to determine the best type of vegetation growth that would occur at this altitude. This special type of vegetation was subsequently planted on the face of the reinforced slope.

Client: WLV – Die Wildbach und Lawinenverbauung, Imst, Austria.
Consultant: Geotechnik Henzinger – Zivilingenieur für Bauwesen, Grinzens, Austria.
Contractor: Streng Bau GmbH, Landeck, Austria.

Cross section through the avalanche protection barrier

Beginning of the reinforced slope in the avalanche protection barrier

Special plant species used on the high-altitude reinforced slope face

Wire mesh facing on the reinforced slope face with vegetation applied to the lower part of the slope

Avalanche protection barrier almost completed
**Reinforced fill slopes: Highway earthworks widening, A3 Hindhead, Surrey, UK**

Much of the land surrounding Hindhead lies within a designated area of natural beauty, a site of special scientific interest and a special protection area for the conservation of wild birds. Consequently, these designation areas place severe constraints on any local construction development.

To improve traffic dispersal through the area of Hindhead a major road works scheme was developed that accounted fully for the local environmental requirements as well as the provision of a dual two-lane carriageway throughout. The highway scheme is 6.8 km in length with a 1.8 km tunnel under a specific sensitive environmental area, and involves the movement and placement of over 1 million m³ of earth fill. To maximise the effectiveness of the earth fill embankments reinforced fill slopes were constructed for structural, economic and aesthetic reasons. Depending on the highway earthworks alignment geometry these reinforced fill slopes were divided into shallow and steep reinforced fill slopes according to their slope angle.

**Shallow reinforced fill slopes**

At specific locations along the length of the highway earthworks embankments shallow reinforced fill side slopes were constructed at slope angles of 1V:1.5H or 1V:1H depending on the slope requirements. These fill slopes were constructed up to 15 m in height depending on the earthworks geometry. The foundations for the reinforced fill slopes were prepared by top soil stripping, excavation and proof rolling. Any soft spots were removed and replaced with compacted general fill. The reinforced fill slopes were constructed using granular fill reinforced with Miragrid® GX geogrid reinforcements. The reinforced fill had to meet specific grading and durability requirements, and was placed and compacted according to specification. The Miragrid® GX geogrid reinforcement was placed at 0.6 m vertical spacings in the reinforced fill extending from the rear of the reinforced fill to the slope face where it was truncated. Depending on the height of the reinforced slopes, up to four different strengths of Miragrid® GX geogrid reinforcement was used.

**Steep reinforced fill slopes**

At other locations along the length of the highway earthworks embankments steep reinforced fill side slopes were constructed at a slope angle of 1.5V:1H. These fill slopes were constructed up to 9 m in height depending on the earthworks geometry. The foundations for the steep reinforced fill slopes were prepared by topsoil infill for one of the steep reinforced slope sections.

The slope facing consisted of a 200 mm deep geocell containing topsoil infill. The geocell was placed down the slope face and fixed to the reinforced soil slope surface by means of galvanised steel anchor pins of 750 mm in length. The topsoil was seeded and then placed within the geocell structure.

The steep reinforced fill slopes consisted of the layers of Miragrid® GX geogrid reinforcement wrapped around hessian bags filled with seeded topsoil. The Miragrid® GX geogrid was then embedded 1.5 m at the top of each reinforced soil lift. This provided a stable, structural facing for the steep slope, and enabled surface vegetation to grow quickly.

To maintain the alignment of the reinforced fill slope face during construction a timber shutter system was used. This shutter system enabled the hessian bag facing to be easily placed along with the Miragrid® GX geogrid wrap-around to the correct slope angle. It also enabled compaction of the reinforced fill immediately behind the hessian bags without deformation of the slope face.

**Environmental barrier**

Much of the land surrounding Hindhead is designated as an environmental barrier which is of special scientific interest and a site of special protection for the conservation of wild birds. The designation areas place severe constraints on any local construction development. Consequently, these designation areas place severe constraints on any local construction development.
Reinforced fill slopes: Railway embankment widening, Hamilton, Ontario, Canada

GO Transit, the Greater Toronto Area’s commuter rail/bus system operator is currently undergoing a dramatic expansion in service. Part of this expansion is the improvement in the rail service between Hamilton, Ontario and Toronto, some 50 km to the East. Additional track has been required in order to meet this growing demand.

In the area of East Hamilton, just South of the intersection of Highways 403 and 6, an existing railway embankment supports CN Rail’s twin track as it passes adjacent to Sunfish Pond. This non-engineered rail embankment was originally constructed around 1900, and currently supports the main CN Rail line running from Halifax to Chicago, as well as GO Transit traffic and other passenger and freight services. To meet the increasing traffic demand a third track had to be constructed along the embankment alignment and this required the embankment crest to be widened.

Sunfish Pond is part of an environmentally sensitive watershed in the area that is managed by the local botanical gardens authority. Consequently, any widening of the existing rail embankment was not allowed to impinge on the pond. Thus, construction of a conventional, widened, 1V:2H embankment slope would have had the slope toe encroaching well into the pond, and thus was not permitted. Therefore, an alternative solution which met both the track alignment requirements and the environmental requirements of Sunfish Pond had to be found.

After evaluating a number of options, the solution chosen was a combined steel sheet pile wall with a vegetated geogrid reinforced slope on top. The sheet pile portion of the steepened embankment slope was tied back using earth anchors or battered piles (depending on the location). The 5 m high sheet pile wall was constructed immediately adjacent to Sunfish Pond. As the sheet pile wall was constructed, an earthworks contractor followed closely behind placing and compacting a specified granular fill behind the sheet pile wall.

Above the sheet pile wall a 1V:1.4H geogrid reinforced fill slope was constructed along with a vegetated surfacing. The slope consisted of compacted granular fill with layers of Miragrid® 2XT geogrid reinforcement as the primary reinforcement placed at 1.0 m vertical spacings, extending 6 m into the slope. Miragrid® 2XT geogrid reinforcement was used as the secondary reinforcement to provide local slope face stability, and these were installed at 1.0 m vertical spacings immediately between the primary geogrid layers. The Miragrid® 2XT secondary reinforcement layers extended 2.0 m into the slope face. Miragrid® 7XT and 2XT geogrid reinforcements are composed of high strength, high stiffness, polyester yarns encased within a robust polymer coating, and have ultimate tensile strengths of 90 kN/m and 35 kN/m respectively.

In order to construct the new reinforced fill slope and obtain the necessary geogrid reinforcement embedment lengths, it was necessary to excavate into the existing embankment slope, which was subsequently nailed to provide temporary stability. The reinforced fill slope was then toed into the excavated, nailed embankment slope with the primary Miragrid® 7XT geogrid reinforcement extending the full width of the new slope.

After completion of the structural portion of the reinforced fill slope, the slope surface was covered with 100 mm of topsoil and then hydro-seeded with a mix of grasses. The slope surface was then covered with a geomat erosion protection layer to prevent erosion of the topsoil while vegetation was established, and to provide reinforcement for the vegetation’s root matrix. To prevent localised movement the geomat was stapled to the slope face at 1m intervals and was trenched into the toe of the slope to provide good stability. Full vegetation of the reinforced slope took around 3 weeks, which was very quick. Following this, the third rail track was constructed on top of the reinforced fill slope.

Client: GO Transit, Toronto, Canada.

Consultant: Isherwood Geotechnical Engineers, Mississauga, Ontario, Canada.

Contractor: Bermingham Construction, Hamilton, Canada.
Reinforced soil walls: Interstate 5/805 widening, San Diego, California, USA

To reduce traffic congestion and improve safety conditions in northern San Diego, the California Department of Transportation (CalTrans) has created additional lanes and a truck bypass at the Interstate 5/805 junction. This construction includes a unique plantable geosynthetic reinforced retaining wall system that transforms a simple slope into a vertical face thereby enabling additional lanes of traffic.

The rate of traffic increase at this freeway interchange has been dramatic over the last 10 years. On some days the traffic would be backed up for hours, and every day around 260,000 vehicles pass through this interchange. This freeway improvement project is the most expensive ever in San Diego County, and has taken 5 years to construct. At its widest point the reconstructed interchange has 23 lanes of traffic: 7 conventional lanes and 4 bypass lanes in each direction, plus a northbound carpool lane.

To support the additional traffic lanes CalTrans designed a geosynthetic wrapped retaining wall with a large concrete basket system at the face. The two part system allows a retaining wall constructed with layers of reinforced fill and geosynthetic reinforcement to be attached to a concrete facing system that protects the geosynthetic exposed at the face and holds loose plantable topsoil to facilitate vegetative growth.

To ensure stability and limit differential settlements, the foundation of the plantable wall was enhanced by the installation of a geogrid reinforced granular foundation platform. Two layers of Miragrid® XT geogrid reinforcement were placed within the layer of compacted gravel to support the retaining wall structure. To prevent future contamination by fines, the reinforced granular foundation platform was wrapped in a Mirafi® 140NC geotextile separator. The construction of the 20 m high structure proved demanding. CalTrans required extensive laboratory testing of the proposed geogrid reinforcements before they could be approved for use on the project. Detailed installation damage, creep and strength/extension data was required for submission for approval. The geogrid reinforcement chosen for the project was various strength grades of Miragrid® XT geogrid reinforcement. Miragrid® XT geogrids are composed of high strength, high modulus polyester yarns within a robust polymer coating.

Cross section through the wall at maximum height had to develop a system to maintain the geogrid wrapped face square, achieve the required compaction at the face, and keep the geogrid reinforcement taut in place during the whole procedure.

Achieving high compaction within the geogrid wrapped sections proved most challenging on this project. The contractor developed a set of timber forms that held the geogrid reinforcement square and in place while compacting the reinforced fill adjacent to the geogrid face. Only hand-held compaction equipment would fit between the concrete headers, which slowed construction down significantly and made achieving compaction more difficult. Once the compaction was complete the timber forms were removed to reveal a densely compacted geogrid wrapped face that was completely square and almost as hard as stone. This intricate process of wrapping geogrid between the concrete headers was repeated at 125 mm vertical spacings in the lower section of the wall and increasing to 500 mm vertical spacings at the top of the wall.

When completed, around 20,000 m³ of reinforced soil wall facing had been constructed in walls up to 20 m in height and approximately 1 km in length. The quantity of Miragrid® XT geogrid reinforcements consumed in the project was around 700,000 m³.
Reinforced soil walls: Structural walls, Anantara Qasr Al Sarab Desert Resort, Abu Dhabi, United Arab Emirates

Anantara Qasr Al Sarab is an Arabic desert retreat about 90 minutes from Abu Dhabi airport, and 10 minutes from the Saudi Arabian border. The resort is situated in the Liwa desert’s Empty Quarters or Rub‘-al-Khali, the largest uninterrupted body of sand in the world. Sand dunes four times taller than the Tower of Pisa promise rugged adventure and awe-inspiring beauty. The sand in this area is composed of silica and iron, which gives striking colour changes from gold to red.

The consultant and contractor were faced with the prospect of constructing over 4.5 km of retaining structures, from 2 m to 12 m in height with face angles between 85° and 90°, in accordance with the insitu performance of the system. The insitu performance of the system was evaluated for all the retaining walls between 6 m and 12 m high a geogrid reinforcement was used. For walls up to 6 m high Miragrid® GX80/30 geogrid reinforcement was used. For walls between 6 m and 12 m high a combination of Miragrid® GX80/30, Miragrid® GX110/30 and Miragrid® GX160/30 geogrid reinforcement were used.

Construction of the retaining structures started with laying a geocell in the desert sand at the toe of the wall to protect it from erosion during heavy winds, which is a common phenomenon in this vast open quarter of the desert. Once this was done the first Miragrid® GX geogrid layer was laid on top. Starting the structure 0.5 m below the existing sand level provided additional protection against sand erosion and achieved the required factory of safety against sliding. For walls up to 6 m high Miragrid® GX80/30 geogrid reinforcement was used. For walls between 6 m and 12 m high a combination of Miragrid® GX80/30, Miragrid® GX110/30 and Miragrid® GX160/30 geogrid reinforcement were used.

After the approval of the basic design a trial wall was constructed to verify the in situ performance of the system. The trial wall was 7 m high and on completion was loaded to 200 kPa overburden, which was double the design overburden stress of 100 kPa. Maximum vertical deformations during maximum loading were less than 7 mm. After the load was removed the plastic deformation was 3 mm.

The contractor has estimated that this reinforced wall system has provided excellent value and has saved 3,000 truck loads of construction materials, such as steel, aggregate, sand, cement, etc. With each truck load having a minimum round-trip journey of 500 km, savings in truck emissions and fuel costs alone amounted to 1,500,000 km.

Client: Abu Dhabi Tourism Development and Investment Corporation (TDIC), Abu Dhabi, UAE.

Consultant: Halcrow Yolles International, Abu Dhabi, UAE.

Contractor: Al Jaber Engineering and Construction (ALEC), Abu Dhabi, UAE.
Reinforced soil walls: Hill side housing development, Batu Ferringhi, Penang, Malaysia

Moonlight Bay is a luxury, gated, property development on Penang Island in the North of Malaysia. The development consists of 2 condominium blocks and 70 exclusive villas blending into the surrounding hill side, with breathtaking sea views of the renowned Batu Ferringhi beaches and the Andaman Sea. Batu Ferringhi is located on the North Western coast of Penang Island. This area is famous for its beautiful sandy beaches and is a popular destination for both Malaysian and International holiday makers.

At this location, the residual granite soil mantle can have a thickness of up to 20 m overlying granite rock strata. Granite boulders often exist within the weathered soil layer close to the surface. In general, the pristine slopes tend to be stable, with the decomposed granite residual soil having effective shear strength properties of friction angles of 27° to 33° and cohesions approaching 30 kPa. The combined gravel and sand fractions in the decomposed granite soil average 67% while the mean silt and clay fractions are 28% and 7% respectively. The Moonlight Bay development straddles a hill slope overlooking the Batu Ferringhi beaches. It has been planned as a terraced development set into the existing hill slope and has been designed to minimize the amount of slope earthworks. The access within the development is by means of an access road which snakes gradually up the contours of the hill side, with hair-pin bends at either side of the development. In this way, the environmental impact is optimized while the geotechnical stability of the hill side development is maintained.

To provide for the road carriageway alignments, a series of retaining walls have been designed on the hill side slope. The retaining walls support the road carriageways and provide enough horizontal access for the villas constructed in between adjacent road alignments.

As this is a luxury property development, retaining wall aesthetics have been a very important consideration. Consequently, a reinforced soil segmental retaining wall system was chosen for this development for very compelling reasons. The wall face provides very good aesthetics with the surrounding environment while the reinforced soil walls can be designed using geosynthetic reinforcements to satisfy stringent engineering standards.

To realize the slope road alignment, steep cuts had to be made into the existing slope to accommodate the construction of the retaining walls. Being in the tropics where rainfall is frequent and intense it is important to keep the time exposure of the temporary steep cuts to a minimum. Conventional reinforced concrete wall systems are time consuming because they require onsite formwork erection, steel reinforcement assembly, casting and curing of concrete, and formwork removal before final backfilling can occur. Another important advantage of the geosynthetic reinforced segmental wall system is that the temporary steep cuts exposure can be kept to a minimum as components like concrete fascia units and geosynthetic reinforcements are manufactured offsite and backfilling occurs as the walls are being built.

The segmental block units used for the retaining walls were Allan Block® AB3 units. These block units weigh around 34 kg and therefore can be easily hand placed. The block units have a height of 200 mm, a depth of 305 mm and a width of 460 mm. The block units have a feature that allows stacking to automatically achieve a vertical wall batter of 3°. Special corner units with two aesthetic faces are used for the 90° corners of the walls. Special cap units are used for the top of the block walls.

To construct the walls, slopes were excavated to the designed base level for each wall. A foundation platform of 150 mm minimum thickness was formed using crusher run material compacted to 95% Standard Proctor to provide a hard level surface on which to place the first course of blocks. All cavities within the facing blocks and a minimum of 300 mm behind the facing blocks were filled with aggregate.

Polyfelt® PEC geocomposite reinforcement was used as the geosynthetic reinforcement for the construction of the reinforced soil retaining walls. This material consists of high modulus, high strength polyester yarns embedded into a composite structure. The polyester yarns support the internal tensile loads of the retaining wall while the composite structure provides good installation damage resistance and dissipation of any internal pore water pressures. Depending on the wall heights involved, the Polyfelt® PEC geocomposite reinforcements used ranged in tensile strengths from 50 kN/m to 150 kN/m.

To ensure good stability, crusher run material was used for the reinforced fill behind the block wall face. Layers of the appropriate grades of Polyfelt® PEC geocomposite reinforcement were installed at specified levels within the walls in between compacted layers of the crusher run reinforced fill. The crusher run reinforced fill was compacted to a minimum of 95% Standard Proctor.

The retaining walls were designed in accordance with US National Concrete and Masonry Association (NCMA) Standards. The retaining walls were designed to withstand a wide range of loading conditions, both during site development and following completion. The total length of walls involved is approximately 1.4 km, and they range in height from 3 m to 13 m.

Client: Ivory Meadows Sdn Bhd, Penang, Malaysia.
Contractor: Ivory Associates Sdn Bhd, Penang, Malaysia.
National Highway 1 (NH 1) runs 450 km from the town of Wagsah in Punjab near the India-Pakistan border to the Indian capital city of New Delhi, passing through Amritsar, Jalandhar, Ludhiana, Ambala, Kurukshetra, Karnal, Panipat and Sonipat en-route. NH 1 is one of the longest and oldest highways of India.

Panipat is located 90 km north of New Delhi on NH 1. With a population of about 250,000, it is a small city by Indian standards, however, it is the centre for certain textile industries and other industrial businesses. Panipat today is amongst the most rapidly developing cities in India and has the highest per capita income in the country.

The Panipat Elevated Highway Project involved the upgrading of 10 km of highway along the NH1 corridor that passes through Panipat city. This upgrading was undertaken to ease the acute traffic congestion within Panipat city. Various highway structures were constructed, including a number of flyovers along the alignment of the elevated highway. The flyovers involved the construction of reinforced soil retaining walls to maximum heights of 9 m.

Panipat is situated on the western bank of the Yamuna River which has its source in the Himalayas and is the largest tributary of the Ganges River. Foundation soils in the area are commonly alluvial in nature which makes the construction of high structures difficult from the perspective of bearing capacity. Such was the case with the reinforced soil retaining walls of the approach embankments to the flyovers for the Panipat Elevated Highway Project. At 9 m in height the retaining walls do not have adequate stability against potential bearing failures, and thus a 1 m thick compacted sand platform reinforced with 2 layers of Polyfelt® WX300 geotextile reinforcement was constructed at the base of the retaining walls to improve bearing capacity.

The wall system adopted for the project was a geogrid reinforced soil retaining wall with a precast concrete, discrete panel facing. These precast concrete panels are supported on a concrete levelling pad of cross section 0.6 m wide by 0.2 m thick that is cast-in-situ. Various strengths of Miragrid® GX geogrid reinforcements, ranging from 40 kN/m up to 100 kN/m, were used as the soil reinforcing elements in the retaining wall. Sand was used as the reinforced fill, compacted to achieve 95% Standard Proctor density.

A vertical drainage layer was included behind the precast concrete panel facing. This gravel layer was separated from the reinforced fill using a Polyfelt® TS30 geotextile filter. A perforated PVC pipe was placed at the bottom of the gravel column to drain away any seepage behind the wall face.

The installation of the wall facing units was carried out in tandem with the reinforced fill placement and compaction, and the laying of the Miragrid® GX geogrid reinforcement. The precast concrete panel facings were prefabricated offsite and delivered to the project site as and when required. The standard panel has a height of 0.6 m, width of 1.38 m and thickness of 0.2 m, and formed of M25 grade concrete nominally reinforced to enable transportation, handling and placing without cracking. Each standard panel weighed approximately 450 kg and had cast-in lifting ears which made them easy to lift using light machinery.

The bottom edge of the facing panels is cast with a groove while the top edge has a tongue such that when the panels are placed on top of each other a tongue and groove connection results. This connection detail facilitates the proper alignment of the facing panels during the erection process. It also enables the Miragrid® GX geogrid reinforcements to achieve a good connection capacity with the panels.

The Miragrid® GX geogrid reinforced soil retaining walls for the Panipat Elevated Highway Project were designed in compliance with major international design codes. Approximately 14,000 m² of geogrid reinforced wall facing was used on this project. The completed elevated highway has opened to traffic and the stretch of retaining walls is today the longest along the NH1.

Client: National Highways Authority of India, New Delhi, India.
Consultant: L&T – Ramboll Consulting Engineers Ltd., New Delhi, India.
Contractor: Larsen & Toubro Ltd., New Delhi, India.
Specialist Wall Contractor: Z-Tech India Pvt. Ltd., New Delhi, India.
To improve traffic conditions between the towns of Constantine and Didouche Mourad in Eastern Algeria it was decided to construct an overpass structure at M’Sila, a town 200 km South East of Algiers. This overpass structure supports a new road alignment over a railway alignment. The concrete bridge part of the overpass is 44 m long, 11 m wide, and is 10 m high to allow train traffic to pass beneath.

The Client was faced with using a retaining wall solution for the overpass abutments as a traditional soil slope solution would have required acquisition of private properties adjacent to the overpass, the removal of trees and relocation of an existing road close to the overpass. Various retaining wall solutions were investigated, however, because of cost, aesthetics and construction speed a geosynthetic reinforced soil segmental block wall solution was adopted.

The abutment retaining walls varied in height up to 10 m. This resulted in abutment lengths of 225 m in the M’Sila Central direction and 180 m in the Bordj Bou Arreridj direction. The crest of the abutments had to support a two-lane carriageway (7.5 m wide), along with parapet structures, giving a total required width of 13 m. The Atalus® 120 block facing layout was designed in such a way that the wall facing angle approximated 74º. This resulted in a base abutment width of 16 m at the location of maximum wall height and this met the Client’s requirements of not encroaching onto adjacent private properties and adjacent tree areas. This is the highest Atalus® 120 block wall constructed without the adoption of intermediate berms.

The design of the reinforced soil wall abutments utilised Bidim® PEC120 geocomposite reinforcement extending 7.5 m behind the wall face. Where appropriate, the geocomposite reinforcement was extended all the way through the abutments from one wall face to the other. Bidim® PEC120 geocomposite reinforcement consists of high modulus polyester fibres manufactured into a composite structure, and has an initial tensile strength of 120 kN/m. The use of Bidim® PEC120 geocomposite reinforcement made it possible to use local alluvial gravelly sand deposits for the reinforced fill. This made the overpass structure very economical to construct.

The Atalus® 120 block units were cast on site using local sand and gravels. A minimum concrete compressive strength of 30 MPa after 28 days was required. The foundation pads for the wall facings consisted of concrete. The granular reinforced fill material was obtained from a borrow area nearby the construction site. This was placed and compacted in 0.25 m lifts. Top soil was placed inside the facing units along with vegetation seedlings.

Client: Société Nationale des Transports Ferroviaires (SNTF), Algiers, Algeria.

Consultant: Georoute Ingenierie, Champhol, France.
Increased traffic flows are one of the major problems in the Centre of Dakar, the Capital City of Senegal in West Africa. To alleviate this problem at two of Dakar’s busiest city intersections, grade-separated interchanges, consisting of traffic flyovers, were constructed. A lack of space to construct the flyovers and the presence of soft clayey-sand foundations at these two locations were two critical issues that had to be overcome.

The soft foundation strata at the two sites varied in depth from 8 m to 13 m. The water table occurred at a depth between 0.3 m and 1.6 m from the surface which would make any solution involving excavation difficult. Consequently, to solve the site problems the flyover abutments were designed and constructed using geosynthetic reinforced soil wall technology.

The flyover structures themselves were founded on piles. The flyover abutments were originally designed using a proprietary reinforced soil wall system utilising strip reinforcements and specially graded reinforced fill. However, a value-engineered solution was designed using locally available dune sand as the reinforced fill with geosynthetic reinforcements. The vertical reinforced fill face was obtained by wrapping the geosynthetic reinforcement around compacted laterite fill. Laterite is a locally available fill that has cementitious properties and which can be compacted to form a stable vertical face. The maximum height of the flyover abutments is 5.2 m. Finally, a permanent vertical facing consisting of full-height, prefabricated, concrete panels were used as the final finish for the flyover abutments.

The reinforced fill used for the flyover abutments was dune sand obtained from a nearby quarry and which is readily available in the area. When properly compacted, its properties render it suitable for reinforced fill with geosynthetic reinforcements.

The geosynthetic reinforcement used in the reinforced soil walls was Bidim® PEC125 geocomposite reinforcement. Bidim® PEC125 geocomposite reinforcement consists of high modulus polyester fibres manufactured into a composite structure, and has an initial tensile strength of 125 kN/m. The geocomposite reinforcement was placed at 0.4 m vertical spacings throughout the height of the abutments. Following the consolidation period, 0.13 m thick, full-height, prefabricated, concrete panels were installed. These sat on a concrete footing, and were reinforced by a tie beam.

The geocomposite reinforcement was folded back along the upper surface of the laterite fill, slightly tensioned, and then anchored in place with the completion of the fill lift of dune sand reinforced fill. Once the reinforced fill lift had reached the top of the external steel shutter, the shutter was extracted from the reinforced fill and then placed on top of the fill with the reinforced fill construction then continuing.

When the reinforced soil abutment walls were completed they were left until such time as the soft clayey-sand foundation had consolidated. Settlements of the abutments, prior to concrete panel installation, ranged up to a maximum of 0.1 m. Following abutment construction there has been a negligible increase in strain.

Client: Etat Senegalais, Dakar, Senegal.
Consultant: EGIS, Seyssins, France.
Contractor: Eiffage Senegal, Dakar, Senegal.
Reinforced soil walls: Coal mine dump wall, Sangatta, East Kalimantan, Indonesia

PT Kaltim Prima Coal located at Sangatta, on the East coast of Kalimantan, has been mining coal in this location since the early 1990’s, and is today one of the largest coal mining companies in Indonesia. At Sangatta, the company has developed a fully integrated and self-supporting mine with a series of open-cut pits and coal preparation and processing facilities, supported by a 10 MW coal fired power station.

The coal at the Sangatta mine is delivered to the coal crushing plant where it is crushed and, if necessary, screened and washed at the coal washing plant before it is placed onto a 13 km long overland conveyor belt for transportation to the shipping terminal.

To increase the handling and transportation capacity at the mine site a new vertical dump wall had to be constructed. To minimize earthworks, the vertical dump wall was to be constructed by excavating into a hill side slope. On top of the hill an access road would lead large dump trucks up to the top edge of the dump wall. A hopper chamber would receive the dumped coal and this chamber would funnel and deliver the coal at a constant rate onto the conveyor belt located at the foot of the vertical dump wall.

Various retaining wall options were investigated for the dump wall. The retaining wall has a maximum height of 17.5 m and is required to support the vertical loads of the heavy dump trucks working close to the top of the wall face. A reinforced soil retaining wall system was chosen using a rock-filled gabion facing with Polyfelt® PEC geocomposite reinforcements. The unusual feature of the wall was that the Polyfelt® PEC geocomposite reinforcements were not structurally connected to the gabion facing. Instead, the gabion facing was connected to the geosynthetic reinforced soil structure by the short embedment of wire mesh tails into the reinforced fill zone.

The foundation base of the reinforced soil retaining wall was stabilized, leveled and then compacted to achieve a minimum undrained shear strength of 200 kPa. The lowest level of the gabion facing was set 1 m below surrounding ground level. Polyfelt® PEC geocomposite reinforcements were installed at 0.5 m vertical spacings throughout the height of the wall. At the face of the reinforced fill zone, immediately behind the gabion facing, the Polyfelt® PEC geocomposite reinforcements were wrapped around the face of the compacted reinforced fill and anchored back into the fill to the required embedment length at the next reinforcement level. Polyfelt® PEC200 (200 kN/m tensile strength) was used for the lower courses of the reinforcement while Polyfelt® PEC150 (150 kN/m tensile strength) was used for the upper courses of the reinforcement.

At each gabion level, the wire mesh tails of the gabion units were laid 2 m into the compacted reinforced fill at each 1 m gabion height. The reinforced fill used for the construction of the reinforced soil retaining wall was a residual soil obtained from a borrow area within the mine site. The residual soil was of a silty sand gradation and was considered appropriate provided good compaction was carried out and good drainage measures were provided. Compaction was carried out using a 10 tonne compactor to achieve 90% Standard Proctor compaction.

A drainage blanket to intercept groundwater seepage at the rear of the reinforced soil wall was provided. This consisted of granular material wrapped in a Polyfelt® TS50 geotextile filter. At two levels within the wall a series of drainage pipes at 4 m horizontal spacings were installed to drain the water captured in the drainage blanket out through the face of the retaining wall.

The choice of Polyfelt® PEC geocomposite reinforcement enabled the use of the local residual soil as the reinforced fill material even though it contained a significant fine fraction. It was considered that if water penetrated the reinforced fill zone at a later time, then it could be dissipated out of the wall structure using the geocomposite structure of the Polyfelt® PEC reinforcement.

A 0.5 m thick reinforced concrete slab was cast on top of the retaining wall. This served as a reversing platform for the coal dump trucks.

The choice of a rock-filled gabion facing permitted the facing erection process to be done manually, without the use of lifting cranes. Besides being the most cost effective option for the client, the reinforced soil wall structure was successfully completed to good engineering tolerances without any contractual delays.

Client: PT Kaltim Prima Coal, East Kalimantan, Indonesia.
Consultant: Golder Associates, Brisbane, Australia.
Contractor: PT Petrosea, Jakarta, Indonesia.
Reinforced soil walls: Reinforced walls and slopes, Gwinnett, Georgia, USA

A baseball stadium had to be designed and constructed for the Gwinnett Braves baseball team to accommodate over 10,000 spectators. The ballpark had to blend in with the surrounding topography and this necessitated the construction of several retaining wall structures as part of the project earthworks.

To reduce costs and speed construction a number of value-engineered solutions were proposed and adopted. This resulted in the design and construction of several segmental block reinforced soil retaining walls and a steep reinforced slope.

Reinforced soil walls
Several retaining wall structures had to be constructed within the ballpark precincts. These ranged from the wall support for the vehicular bridge at the entrance to the ballpark to the 10 m high outfield wall. These walls were constructed using segmental block facings and Mirafi® PET HS geotextile reinforcement.

The wall facings consisted of Newcastle® standard blocks, which having a 100% positive mechanical connection with the geotextile reinforcement, enabled the wall heights up to 10 m to be constructed. Layers of Mirafi® PET HS geotextile reinforcement were used as the reinforcement because it was composed of high modulus polyester yarns and had the required strength and durability properties. Local residual soil was used as the compacted reinforced fill. A system of granular collector drains and drainage pipes collected excess groundwater and drained it to the front of the wall face.

A typical retaining walls design and construction phase for a project such as this would normally take around 6 months. However, with the value-engineered solutions the retaining walls part of this project was completed within 2 months.

Reinforced fill slope
Part of the site earthworks involved the construction of a reinforced slope along one of the boundaries of the ballpark. This slope was designed to replace an originally proposed reinforced concrete retaining wall. The reinforced slope was of maximum height 13 m and constructed at a 3V:1H face angle, and was considered to be a “green” solution compared to the originally designed concrete retaining wall.

The slope facing consisted of 0.45 m high galvanized steel mesh units aligned to a 3V:1H slope angle. Inside these mesh units a grid geotextile was placed to contain the reinforced fill placed behind the steel mesh. After completion, the face was vegetated with the slope face being covered with grass quickly.

In the bottom half of the reinforced slope layers of Mirafi® PET HS geotextile reinforcement were installed at 0.45 m vertical spacings. Above this, layers of Mirafi® PET HS geotextile reinforcement were installed at 0.9 m vertical spacings.

The steep reinforced slope solution enabled construction to be carried out quickly and resulted in a “green” solution versus the originally proposed concrete retaining wall.

Client: Gwinnett Convention and Visitors Bureau, Georgia, USA.

Consultant: Fitzpatrick Engineering Associates, Lawrenceville, Georgia, USA.

Contractor: Wall Technologies Company Inc., Atlanta, Georgia, USA.
The Upper Harbour Corridor was designed as a traffic dispersal system for the North Harbour region of Auckland, New Zealand’s largest city. The alignment of this highway dispersal system resulted in a number of grade-separated highway interchanges, while a number of steepened slopes were utilised to ensure land acquisition was kept to a minimum for the earthworks construction. A series of reinforced soil walls were constructed at grade-separated highway interchanges, and a series of reinforced fill slopes were constructed on the side slopes of the embankment fill sections.

**Reinforced soil walls**

The walls had to be designed to accommodate both static and seismic loadings, as the area is prone to seismic activity. For performance and economical reasons it was decided to use Anchor Landmark® segmental block units for the wall facings as these provided a full positive connection with the Miragrid® XT geogrid reinforcement, and were easy to install. The design was performed using a limit equilibrium approach taking into account static and seismic loadings, the specific properties of the Anchor Landmark® blocks and the Miragrid® XT geogrid reinforcements. Wall heights varied up to 9.5 m in height. Wall heights varied up to 8.5 m in height.

The granular reinforced fill used was a fine-crushed rock with high frictional characteristics, and this material was easy to compact under variable weather conditions. The wall toes were embedded 0.4 m below ground level to provide good toe stability. To facilitate good groundwater drainage a 0.3 m thick granular drainage blanket, encapsulated in a Mirafi® geotextile filter, surrounded the reinforced fill zone. This drainage layer was extended up behind the wall face to ensure no groundwater would seep through the Landmark® block facing.

Following completion of the retaining walls, on-ramp and off-ramp road exits were constructed on top of the walls.

**Reinforced fill slopes**

The reinforced fill slopes were designed using a limit equilibrium approach, taking into account both static and seismic loadings. The facings of the slopes consisted of large knitted socks filled with soil, mulch and ryegrass grass seed. The Miragrid® XT geogrid reinforcement was placed extending out through the face of the slope, wrapping around the soil filled socks, and extending into the slope 2 m. Slope face angles were maintained at 2:1H for all reinforced slopes. Slope heights varied up to 16 m in height. A major positive point for this type of construction was that the same fill used in the embankment earthworks could also be used as the reinforced fill for these reinforced slopes.

If the foundation was soft at the base of the embankment earthworks, the soft material was excavated and replaced with free-draining granular fill. Where required for surface run-off, concrete culverts were installed across the base of the embankment earthworks. At the toe of the reinforced fill slopes 1 m cube concrete blocks were installed up to the top of the concrete culvert level. These were installed to ensure negligible settlements would occur around the concrete culverts. Compacted granular fill was placed behind these concrete blocks.

The reinforced fill slopes were constructed on top of the concrete blocks. Layers of Miragrid® XT geogrid reinforcement were placed extending out through the face of the slope. The large knitted socks containing top soil, mulch and ryegrass grass seed were placed and shaped along the slope face. The reinforced fill was then compacted up to the face of the slope, and then the extended length of Miragrid® XT geogrid was wrapped around the face and brought 2 m into the slope face, prior to the placement of the next layer of Miragrid® XT geogrid.

Because of continual poor weather onsite it became impossible to adequately compact the embankment fill material. Consequently, this material was combined with 2% lime, and this enabled adequate compaction. The lime stabilised embankment fill material was also used as the reinforced fill in the reinforced fill slopes with the Miragrid® XT geogrids.

To prevent groundwater entering the lime stabilised reinforced fill zone a 0.3 m thick granular drainage blanket, encapsulated in a Mirafi® geotextile filter, ran down the rear of the reinforced fill zone into the free-draining granular fill at the base of the slopes.

Once the slopes were completed the highway pavement structure was constructed on top where required. Client: Transit NZ Ltd, Auckland, NZ. Specialist Consultant: Riley Consultants Ltd, Auckland, NZ.
**Reinforced soil walls: Coal processing plant platform, Hetaoyu, Qingyang City, Gansu Province, China**

China Huaneng Group is China’s largest power producer. Through subsidiaries, it develops and operates more than 85 thermal and hydro power plants. The client is also actively investing in coal resources to secure supply at stable prices.

The Company is developing a coal processing plant at Hetaoyu in Qingyang City, Gansu Province. The site is on the side of the Jinghe River valley. The soil deposit at the site consist of loess (silt and fine sand) which may be up to 100 m thick. The natural form is quite porous and can be easily eroded by water. However, local experience has shown that provided the loess can be well-compacted and protected from water ingress and erosion, then it can be used for reinforced fill. For this project, a design friction angle of 28° was used, while test results showed compacted friction angles as high as 38° to 40° at low moisture contents.

The Jinghe River is generally a shallow meandering stream within the loess floodplain for most months of the year but water levels can rise quickly during flash floods in the rainy season. Consequently, it was necessary to ensure the platform of the coal processing plant was constructed well above any potential flood level. To utilize maximum platform area, a large reinforced soil retaining wall was constructed along the side of the river valley as part of the coal processing plant platform.

The reinforced soil retaining wall is over 90 m long and averages 25 m in height but reaches a maximum of 35 m at its highest section. The soil used for the reinforced fill and the backfilling of the retaining wall was locally available loess which had to be carefully placed and compacted, and protected from subsequent water erosion.

The retaining wall is founded on uneven rock formation beneath the loess soil deposit. At locations where the formation is below ground level, excavation to expose the rock surface is carried out and then lean concrete is applied to level the rock surface prior to construction of the reinforced soil wall.

This retaining wall was constructed using layers of Mirafi® PET geotextile reinforcements laid horizontally between layers of compacted loess reinforced fill. In the lowest tier of the wall Mirafi® PET300-50 geotextile reinforcement was used. This is a woven polyester geotextile with a tensile strength of 300 kN/m in the longitudinal direction. In the middle tier of the wall Mirafi® PET200-50 geotextile reinforcement was used. This is also a woven polyester geotextile but has a tensile strength of 200 kN/m in the longitudinal direction. In the top tier of the wall Mirafi® PET100-50 geotextile reinforcement was used. This is also a woven polyester geotextile but has a tensile strength of 100 kN/m in the longitudinal direction.

The wall is essentially constructed as a reinforced soil wall with the soil bag facing wrapped around with the geotextile reinforcement at the front of the wall. The soil bags were used to form the shape of the wall facing profile.

To protect the compacted loess backfill from groundwater ingress, subsurface drainage was provided between the compacted backfill zone and the existing loess stratum. The subsurface drainage consisted of nonwoven geotextile filter wrapping a layer of aggregate, and was constructed as the reinforced soil wall was being built.

The reinforced soil wall was built in vertically faced segments until site elevation EL 896 m. For the vertical portions of the wall, a 0.5 m thick superficial reinforced concrete wall facing was provided for permanent protection. A reinforced concrete base is constructed at foundation level, with the concrete facing sequentially extended as the reinforced soil wall is built. The facing wall is structurally connected to the reinforced concrete base and is cast in place against the geotextile wrapping around the reinforced soil wall.

The portion of the wall above site elevation EL 901 m was constructed at a face angle of 4V:1H. For this sloping portion of the wall, a 0.15 m thick steel wire mesh reinforced shotcrete cover is provided for long-term protection. The reinforced shotcrete cover is structurally attached to the reinforced soil wall with the aid of steel dowel pins. A cast-in-place reinforced concrete edge capping unit 0.3 m deep and 1 m wide was provided at the crest of the reinforced soil wall.

A total quantity of 100,000 m² of Mirafi® PET100-50, 200,000 m² of Mirafi® PET200-50 and 250,000 m² of Mirafi® PET300-50 geotextile reinforcements were used for the construction of the reinforced soil wall at this site. The wall took just over 1 year to complete.

**Client:** China Huaneng Group, China.

**Consultant:** Design and Research Institute of Sinocoal International Engineering Group, Wuhan, China.

**Contractor:** China Gezhouba Group Corporation, China.
Reinforced soil walls: Segmental block wall with constrained reinforced fill, Paju, Korea

Paju City is located approximately 50 km to the North West of the capital city Seoul. The area consists of hilly terrain with many rock outcrops.

The landscaping and earthworks for an apartment complex in hilly terrain at Paju required the construction of several reinforced segmental block retaining walls. At one location a 130 m long retaining wall was required, ranging in height from 3 m to 8 m. The area where this wall was to be constructed consisted of partially decomposed quartzite rock in close proximity to the planned wall alignment. In order to minimize risk to an adjacent building, the owner decided not to excavate to construct a conventional geogrid reinforced segmental block wall, and instead a combination geogrid and anchor reinforced segmental block wall was constructed.

The surface of the quartzite stratum in the vicinity of the proposed retaining wall was inclined at approximately 60° to the horizontal. The design called for a wall of exposed maximum height 5.6 m with a crest width of 3.0 m. With this constrained geometry, the line of maximum reinforcement tension in the geogrid reinforcements coincides closely to the boundary of the reinforced fill zone and the quartzite rock stratum at the rear of the wall. Thus, to provide the required stability, the tensions generated in the geogrid reinforcement layers would have to be dissipated within the quartzite stratum at the rear of the reinforced fill. The use of anchors was adopted to dissipate these tensions within the quartzite stratum.

The horizontal stress distribution acting against the rear of the segmental block wall facing was determined using a wedge stability analysis, and the appropriate geogrid reinforcement vertical spacing was determined on the basis of the geogrid design strength and the connection capacity between the geogrid reinforcement and the segmental block facing. For this wall, it was planned to economize on the number of anchors by utilizing one row of insertion earth anchors for every two layers of geogrid reinforcement. The original design called for 260 insertion anchors to be installed into the partially decomposed quartzite back slope prior to anchor insertion.

During construction it was found that the quartzite stratum was harder than originally anticipated. Consequently, drilled rock bolts had to be substituted for the majority of the insertion earth anchors. A total of 208 rock bolts were used in addition to 52 insertion earth anchors.

Because of the confined working area, the construction could only utilise small construction equipment, and resulted in a more labour intensive process. However, the level of soil compaction, laying of Miragrid® SXT geogrid reinforcement and quality
Reinforced soil walls: Shear-key wall, Trump National Golf Course, California, USA

Trump National Golf Course, formerly named Ocean Trails Golf Course, is the only ocean-front golf course in Los Angeles County. The course sits high atop jagged cliffs with the Pacific Ocean below, and offers spectacular scenery from all 18 of its fairways.

Development of the ocean-front golf course had been in the planning and approvals stages for nearly 10 years due to the difficulties in obtaining approvals for any coastal developments in Southern California. The difficulty in obtaining permits was compounded by the presence of a known ancient landslide area (known as Landslide "C") and an environmentally sensitive coastal bluff reserve that is the home of an endangered bird species.

Once approvals were finally obtained, construction of the golf course proceeded as planned. However, near construction completion the 70,000 m³ area of Landslide C reactivated in a single, rapid event. The slide occurred as a large translational block and moved laterally (seaward) approximately 15 m and downswords approximately 3 m. At the base of the slide was an almost-horizontal thin layer of bentonite, between 10 mm and 75 mm in thickness, that when exposed to water becomes extremely slippery. The total slide mass had a maximum length of 550 m, a width of over 130 m, and a depth of between 20 m and 30 m. About 2.8 million m³ of soil moved. The slide took with it most of the 18th hole (fairway and green), bluff edge, pedestrian trails, and a portion of a major Los Angeles County Sanitation District sewer line. It is believed that liquid from a leak in the sewer line may have caused the bentonite layer to slicken, triggering the landslide.

A number of design options were investigated to restore the landslide. The best of these involved partial removal and rebuilding of the landward portion of the landslide as this would achieve the intended purposes, have the least alteration to the natural landform, and be the most feasible from a geotechnical engineering perspective. Crucial to this design option was the incorporation of a shear-key wall, embedded below the bentonite slip plane, at the toe of the excavation to withstand any future destabilising forces. A number of shear-key wall options were investigated but it was decided a reinforced soil wall would be the most effective from the viewpoint of cost and performance. The shear-key wall serves as a large stable soil block that prevents any future soil movement and protects the rest of the golf course and associated structures from being "lost" into the Pacific Ocean.

Construction had to be carried out in environmentally sensitive habitat areas along steep bluffs with often unstable soils and deep excavations. At the same time the golf course had to remain open. The first step in the restoration works involved the stabilisation of the seaward side of the landslide. This involved the installation of soil dowels at the base of the landslide. The dowels were 5 m long, 0.9 m in diameter, and consisted of hollow steel cylinders filled with high density concrete. Altogether, 116 soil dowels were installed at 8 m centres on the seaward side of the landslide.

Once the dowels were installed, the construction of the shear-key wall began by excavating the slip area (over 1.3 million m³ of soil) to a depth below the bentonite slip plane (approximately 30 m to 45 m depth). The excavation was carried out in 6 sections across the landslide to ensure stability during construction. Each section had its shear-key wall completed and backfilled prior to moving on to the next section for excavation.

The reinforced soil shear-key wall is around 30 m wide at its base, and over 30 m in height. The wall consists of angled steel mesh facings with a Mirafi® FW404 woven geotextile filter wrapped around the inside of the steel mesh prior to placement of the geosynthetic reinforcement and reinforced fill. The geosynthetic reinforcement consisted of layers of Mirafi® PET geotextile reinforcement with tensile strengths ranging from 600 kN/m to 200 kN/m depending on their location in the reinforced soil wall. The optimum vertical spacing for the geotextile reinforcement was 1.5 m. The fill used for the reinforced soil wall consisted of on-site granular fill that provided good shear resistance and good bond characteristics with the geotextile reinforcement.

Prior to approval, the Mirafi® PET geotextile reinforcement underwent a series of tests to demonstrate good long term strength and good geotextile reinforced soil bond characteristics.

For additional stability, the seaward face of the massive shear-key wall abutted another reinforced triangular block. This second reinforced fill mass in turn abutted existing slide material and native soils on the seaward portion of the landslide, which were left in place to maintain the natural environment and protect the shear-key wall structure from the forces of the Pacific Ocean.

As the reinforced soil shear-key wall was constructed, on-site fill material was placed and compacted in the unreinforced backfill zone. A clay cap was placed over the entire filled area to keep water out. A layer of topsoil was placed on the clay cap to facilitate vegetation growth and landscaping. The entire reinforced soil structure is now covered by a beautiful grass fairway, putting green and sand traps.

Now, as golfers enjoy the breathtaking view from the 18th hole, few will ever know that they are playing on one of the most expensive holes ever constructed. Even fewer will realise that this is one of the safest places on the coast, because of the many layers of geosynthetic reinforcement below.

Client: Trump National Golf Course, California, USA.
Consultant: Converse Consultants, California, USA.
Contractor: Sukut Construction Inc., California, USA.
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