GEOSYNTHETICS

Reinforced Soil Case Studies

TENCATE GEOSYNTHETICS

TenCate Geosynthetics

TenCate Geosynthetics has been at the forefront of reinforced soil technology for more than 40 years. During that time our Company has built formidable expertise in reinforced soil applications technology. Today, TenCate Geosynthetics Group's reinforced soil business is focused as follows.

Worldwide coverage: TenCate Geosynthetics Group has worldwide coverage providing advice and delivery to 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.

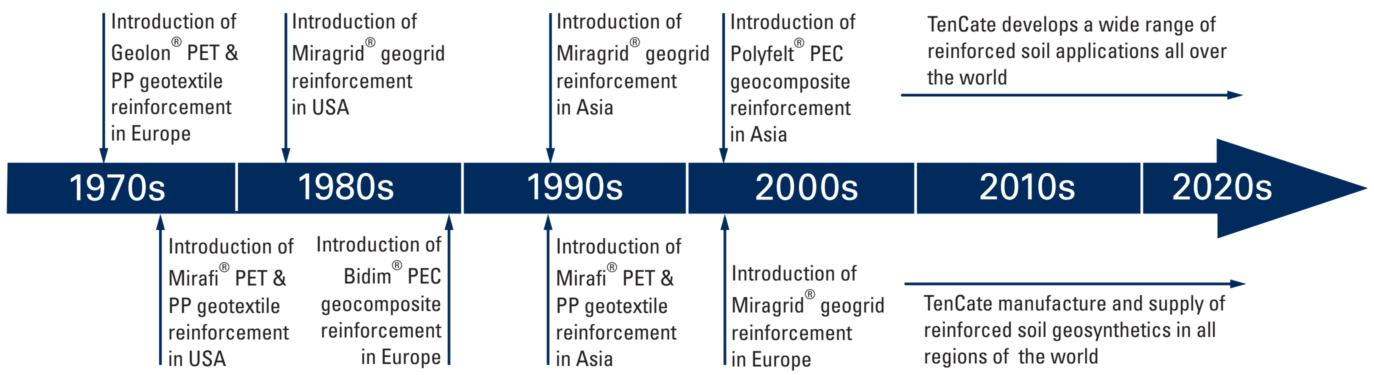
High performance reinforced soil materials: TenCate Geosynthetics Group manufactures and supplies a wide range of geosynthetic reinforcement materials specifically engineered for a wide range of reinforced soil applications.

High quality standards: TenCate 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.

TenCate Geosynthetics' history in reinforced soil



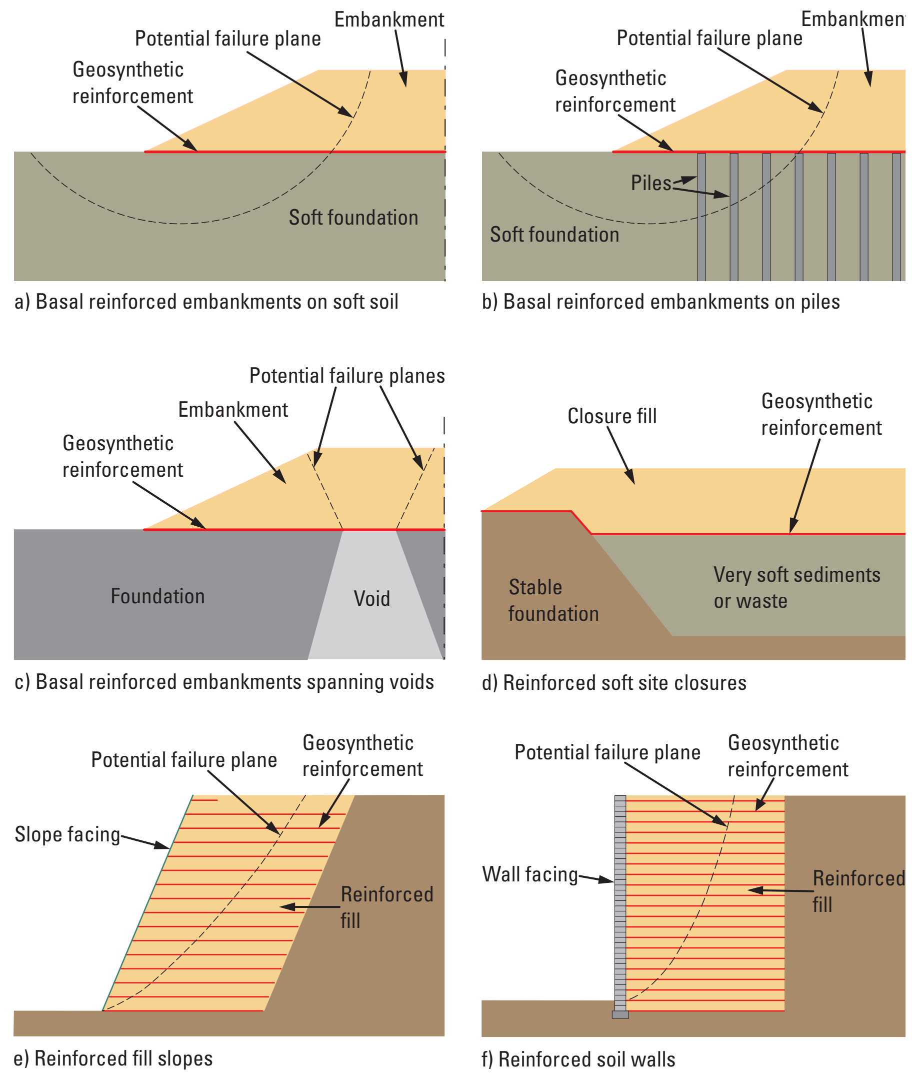
TenCate Geosynthetics Group is a pioneer in the field of reinforced soil using geosynthetic reinforcements. The history of the Group's involvement stretches back to the mid 1970's.

- During the mid 1970's TenCate 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 1970's this technology was exported to the United States with great success, and later to Asia (mid 1990's).
- During the early 1980's 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 1990's) and to Europe (early 2000's).
- During the late 1980's PEC geocomposite reinforcements 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.

TenCate 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.

Reinforced soil applications

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.



a) Basal reinforced embankments on soft soil

b) Basal reinforced embankments on piles

c) Basal reinforced embankments spanning voids

d) Reinforced soft site closures

e) Reinforced fill slopes

f) Reinforced soil walls

Typical reinforced soil applications

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 foundation 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 fill slopes
Here, multiple layers of geosynthetic reinforcement are placed in the slope 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, as well as a wide choice of slope facings.

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 walls to be constructed to a wide range of heights as well as the choice of a wide range of wall facings.

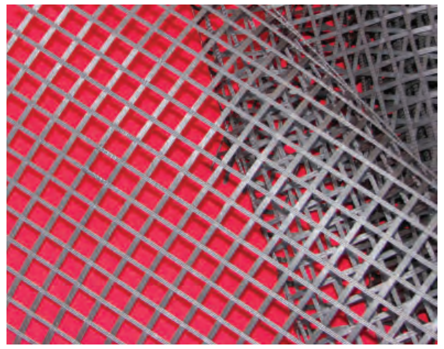
TenCate Geosynthetics' reinforcement materials

TenCate Geosynthetics has engineered a comprehensive range of geosynthetic reinforcement materials specifically suited to a wide variety of reinforced soil applications. All of these engineered materials exhibit excellent tensile load carrying capabilities at defined strains, along with very good durability characteristics.

Each of these materials may be supplied in several different roll sizes to make on site storage, movement and installation as easy as possible.

Mirafi® PET and Geolon® PET woven polyester geotextile reinforcements

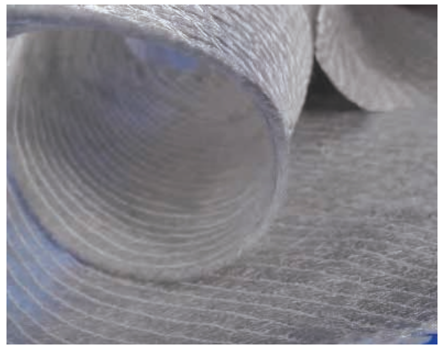
These geotextile materials are composed of high strength, high tensile modulus polyester yarns woven into tensile strengths ranging from 100 kN/m to 2,500 kN/m. This large strength range, coupled with their very good long term load carry capability, makes these materials ideal for basal reinforcement 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 tensile modulus polyester yarns embedded in a robust polymer coating and have tensile strengths ranging from 35 kN/m to 800 kN/m.

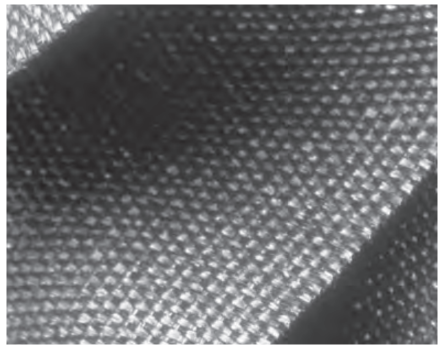
These materials have good resistance to the effects of installation damage and are highly durable in a wide range of soil conditions. Consequently, they are almost always used for applications where tensile loads have to be carried for long periods of time.



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® RSi and PP, and Geolon® PP woven polypropylene geotextile reinforcements

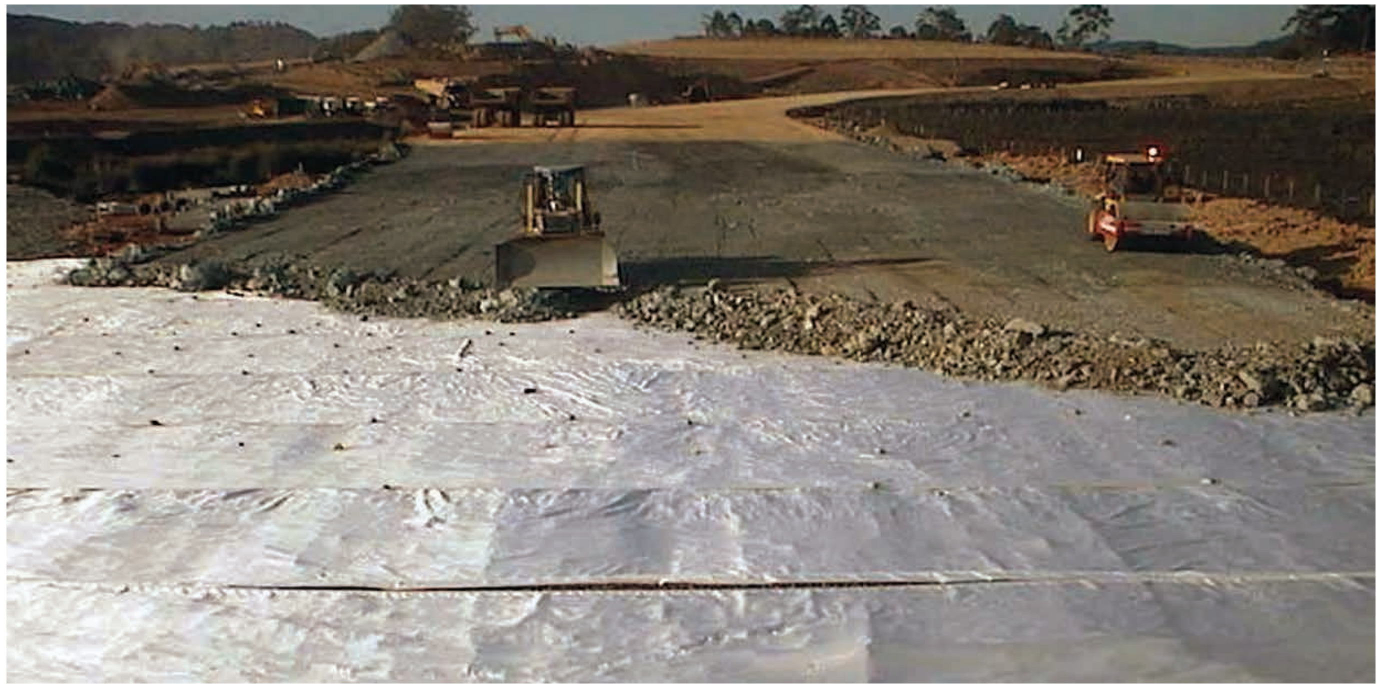
These geotextile materials are composed of high strength, high tensile modulus polypropylene yarns woven into tensile strengths from 60 kN/m to 300 kN/m. This large strength range, coupled with their medium term load carrying capability and excellent durability, makes these materials ideal for short 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

TenCate Geosynthetics has been involved in many reinforced soil applications, in many parts of the world, over the last 45 years. The selection of the case studies contained in this booklet give an appreciation of the diverse range of reinforced soil applications where TenCate's geosynthetic reinforcements have been used. New reinforced soil applications are continually evolving.

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 30 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.

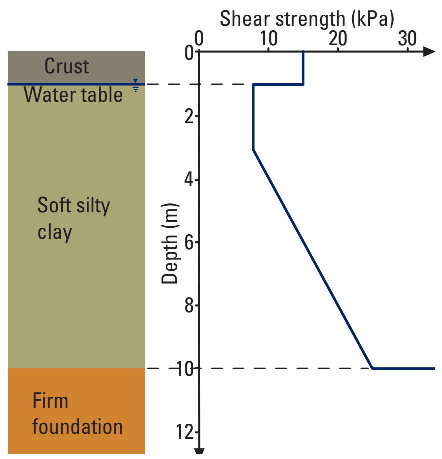
To meet the construction time and performance requirements of the project it was decided to construct a basal reinforced, 1 m surcharged embankment in the areas where soft foundation soils were encountered. The basal reinforcement would provide adequate stability to allow the embankment to be constructed 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.

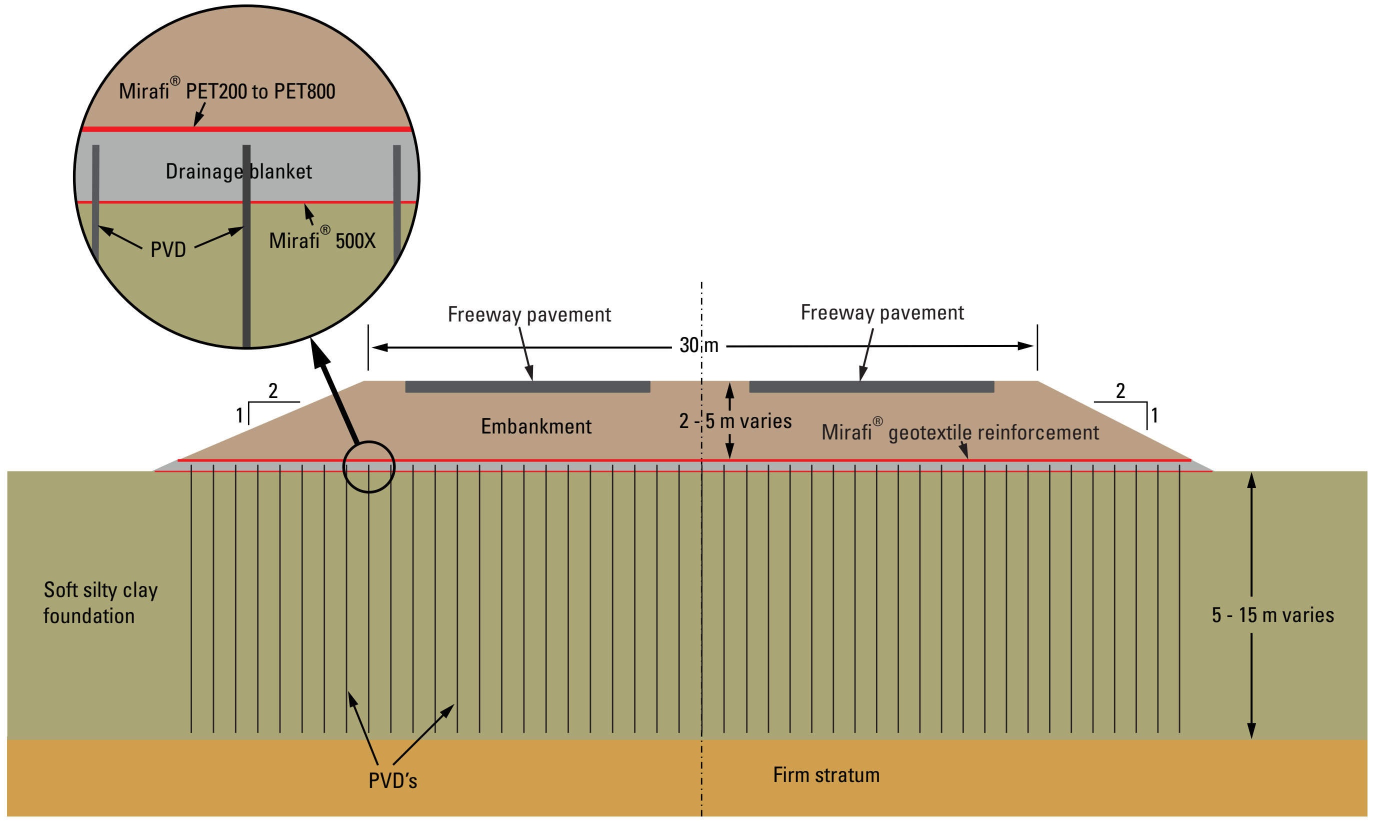
A bridging layer of 0.5 m thick of local clayey fill was placed on the separation geotextile. This bridging layer created a stable platform on which the PVD installation equipment could operate, and also enabled less granular material to be used for the drainage blanket. Following this, a 0.2 m thick 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,

Ground conditions at site



Foundation shear strength profile



Cross section through the basal reinforced embankments

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 spanned 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.

Placement of PVD's through gravel drainage layer

Embankment under construction

Placing Mirafi® PET geotextile reinforcement over drainage layer

Completed freeway

Client: Roads and Traffic Authority, New South Wales, Australia.

Consultant: SMEC Pty Ltd, New South Wales, Australia.

Contractor: AbiGroup Ltd, New South Wales, Australia.

Basal reinforced embankments on soft soil: Mine services corridor, Cape Preston, WA, Australia

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.

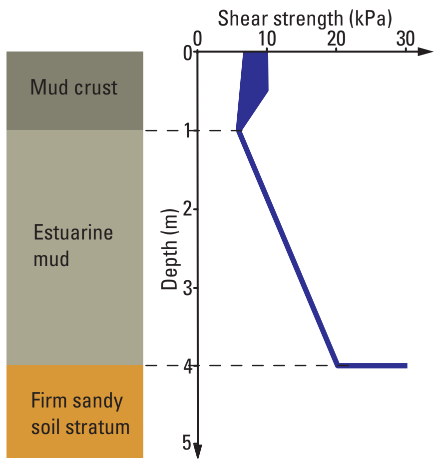
Part of this services corridor consisted of a 2 km long causeway constructed through a river estuary. The foundation conditions within the river estuary consisted of soft estuarine mud of approximately 4 m in depth overlying firm sandy soil. The estuarine mud consisted of a slightly overconsolidated 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.

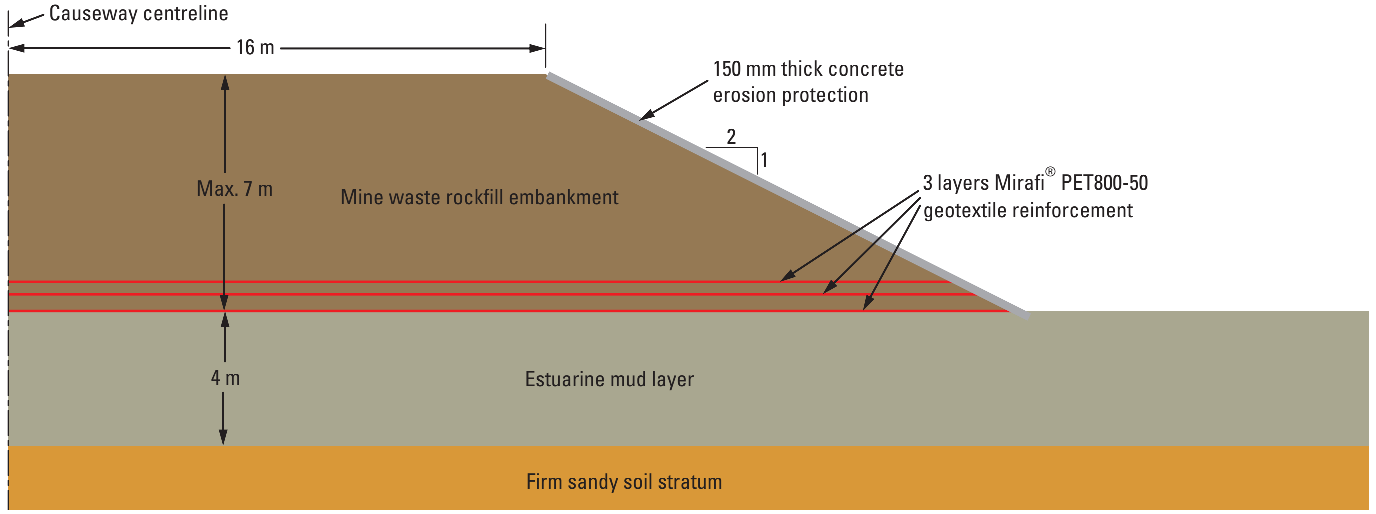
In addition to the tight construction schedule, the causeway embankment had to be designed to allow for heavy vehicle loadings from 240 tonne haul trucks as well as the safe transport of 1400 tonne giant grinding mills for the mining operations.

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



Foundation undrained shear strength profile



Typical cross section through the basal reinforced causeway

required. Taking all these factors into account, 3 layers of Mirafi® PET800-50 geotextile reinforcement was 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.

Layout of Mirafi® PET800-50 woven geotextile reinforcement on site

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.

Construction of basal reinforced causeway abutment

Completed reinforced causeway and bridge structutre

Transport of giant grinding mill along services corridor

Basal reinforced embankments on soft soil: Seawall construction, Brisbane Port expansion, QLD, Australia

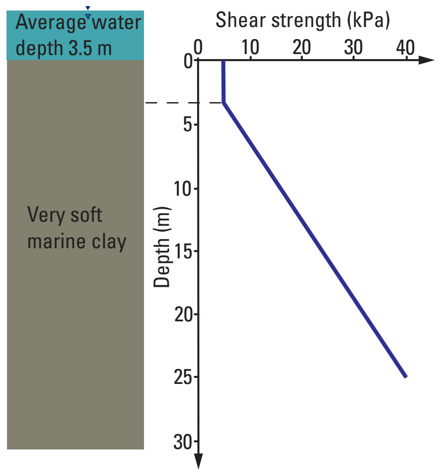
The Port of Brisbane is located at the mouth of the Brisbane River, and has seen rapid development over the last 20 years, and this growth is expected to continue in the future. To keep up with the pace of growth the Future Port Expansion Project was conceived. This project has the ultimate objective 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.

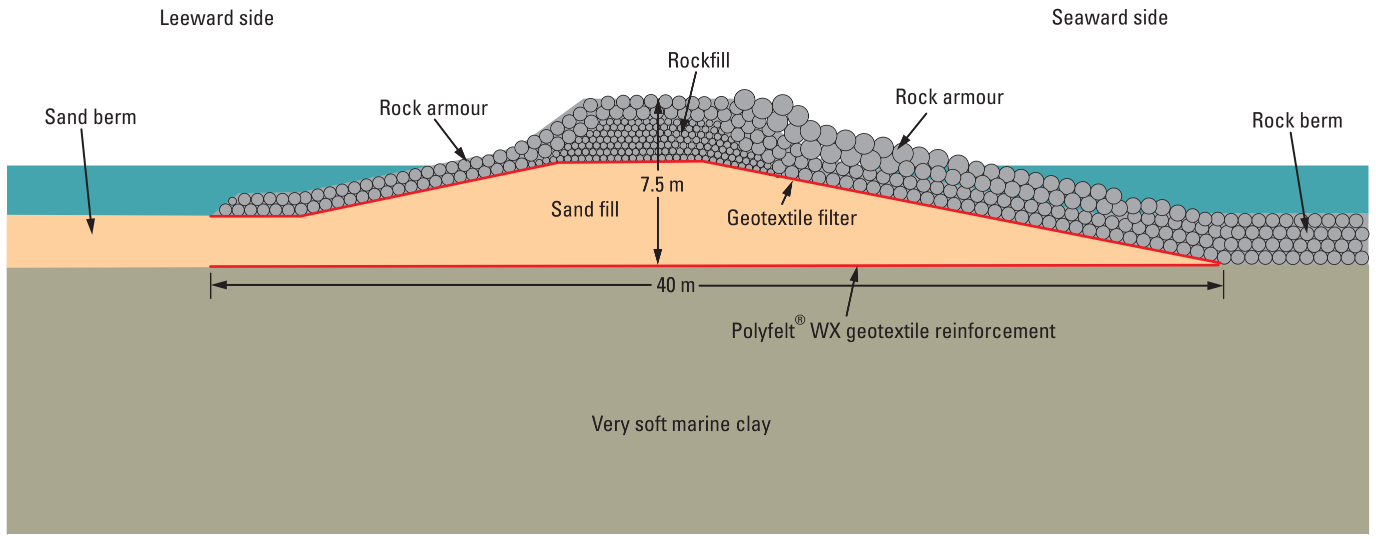
Preliminary analyses indicated the marine clay foundation to be generally too weak to support high embankment loadings without ground improvement works. A number of options were evaluated but the use of a high strength geotextile as the basal reinforcement for the seawall was ultimately assessed to be 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 overlying 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.



Foundation undrained shear strength profile



Typical cross section through the seawall

To install the Polyfelt® WX geotextile reinforcement below water level a shallow-draught 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 offsite to form panels up to 42 m wide by 100 m long. These were then rerolled for deployment from the barge. The geotextile reinforcement was unrolled, dropped over port side of the barge, and then taken beneath the barge past the starboard side by divers. To avoid the geotextile folding transversely 12 mm reinforcement bars were attached to the geotextile reinforcement with cable ties at 10 m spacing to hold the geotextile flat and help sink the geotextile to the seabed. Ballast was then placed to hold the geotextile reinforcement in place on the seabed.

Once the sand fill had been placed to the required levels the barge was then used to place the geotextile filter up the sides and over the top of the sand embankment prior to holding it in place with rock fill.

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 unforseen storm activity during construction.

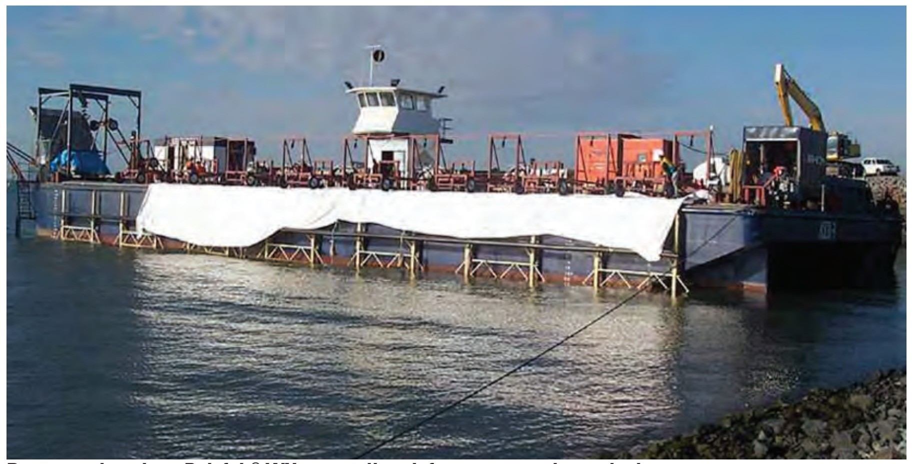
Following completion of the seawall the reclamation for the port expansion has progressed as planned. The spoil from the maintenance dredging operations has been used to build a land bank area for future port expansion programs. The seawall has also prevented any sediment contamination of the nearby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

Client: Port of Brisbane Corporation, Brisbane, Australia.

Consultant and Contractor: Alliance Partners, Brisbane, Australia.

End dumping and spreading rock fill to complete the core of the seawall

Placing and spreading the rock armour layers on the seawalll



Barge used to place Polyfelt® WX geotextile reinforcement on the seabed

Basal reinforced embankments on soft soil: Access road embankments, Pengerang Integrated Petroleum Complex, Johor, Malaysia

Pengerang Integrated Petroleum Complex (PIPC) is a large project development located in the Southeast tip of Johor, Peninsular Malaysia, opposite Singapore. It is one of the largest investments in the area and is located on a single plot of land measuring about 8,000 ha. The site offers strategic access to existing major international shipping lanes and an excellent road network to Singapore and other major ports in Peninsular Malaysia. Approximately, a 22 km access road network had to be constructed to fully realise the infrastructure requirements of the development project.

The ground conditions across the PIPC site consist of a very soft alluvial soil layer to around 4 m in depth. It can consist of silt, clay or peat and has an undrained shear strength of around 8 kPa and is constant with depth. The groundwater level (GWL) is around 1 m below surface level. Below this upper layer is a layer of medium stiff silt extending down to around 19 m below surface level. The undrained shear strength of this layer increases linearly with depth starting at around 20 kPa at the top of the layer. Below this layer is a firm foundation stratum.

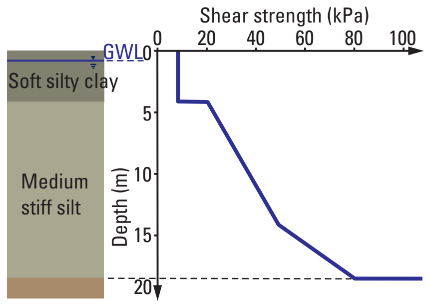
To construct the access road network a series of basal reinforced embankments was designed over the soft foundation soils. Based on the required road surface elevation, these embankment heights ranged from 1 m to 18 m.

Prefabricated Vertical Drains (PVD's) were proposed for around 50% of the access road embankments to accelerate the consolidation of the soft foundation soils. For the remaining 50% no PVD's were used with the Mirafi® PET geotextile reinforcement as it was considered that foundation consolidation and shear strength gain did not require this treatment in these locations. Further, embankment surcharging was also applied to accelerate foundation consolidation. A surcharge of up to 3 m was applied depending on embankment height with a view to it being removed after 6 months application.

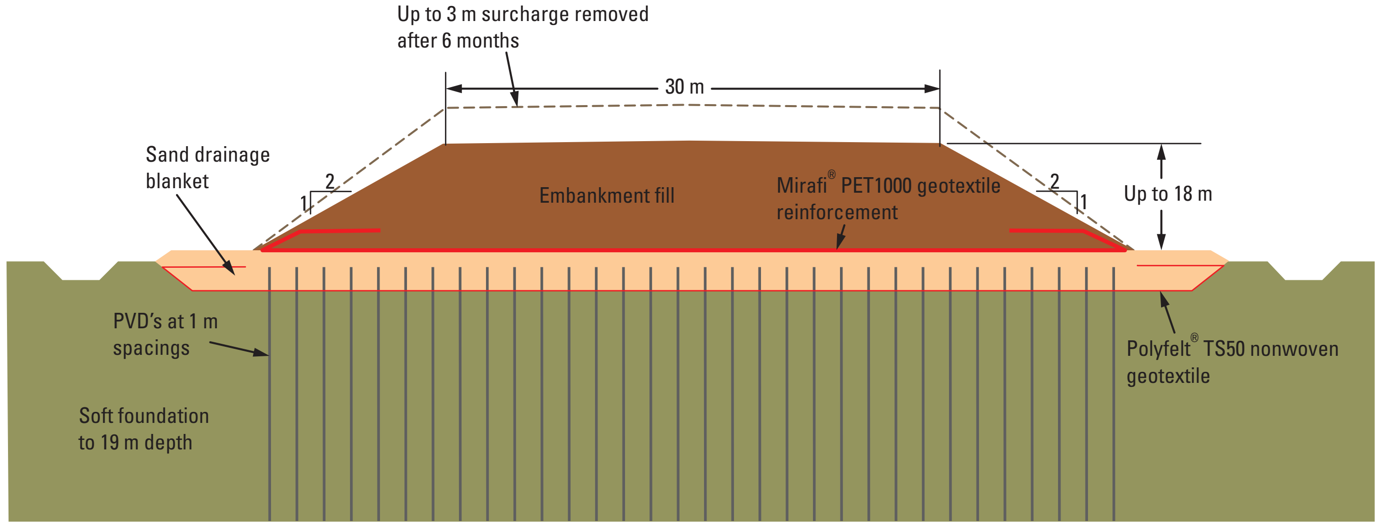
Soft foundation conditions at Pengerang site

In the areas where PVD's were used, a Polyfelt® TS50 nonwoven geotextile was placed on the soft foundation as a separator/filter geotextile to prevent the intermixing of the sand drainage blanket with the insitu soft foundation soil. Prior to placement of the Polyfelt® TS50 nonwoven geotextile large and sharp objects were removed from the surface of the soft foundation. The sand drainage blanket was then spread across the installed Polyfelt® TS50 nonwoven geotextile to create a combined drainage/working platform layer to support the PVD installation equipment.

A range of Mirafi® PET geotextile reinforcements were used for the various basal reinforced embankments throughout the site. Different Mirafi® PET grades and layers were applied depending on embankment geometry and soft foundation conditions. Single or multiple layers of Mirafi® PET200, Mirafi® PET400 and Mirafi® PET1000 were incorporated into the base of the embankments where appropriate. These Mirafi® PET grades were installed in two configurations depending if PVD's were used or not. First, if PVD's were used the Mirafi® PET grades were installed across the top of the sand drainage blanket prior to placement of the embankment fill. Second, if no PVD's were used the Mirafi® PET grades were installed directly on the soft foundation surface with embankment fill placed directly on top.



Typical undrained shear strength profile at Pengerang site



Typical cross section through access road embankments using PVD's

Select engineered fill was used to construct the embankments. This was spread in lifts and compacted to 95% Standard Proctor compaction. Embankments with more than a single layer of Mirafi® PET geotextile reinforcement was required to have 0.5 m of engineered fill placed between each layer.

The embankments with surcharging were constructed to completion. Following the required surcharging period, the surcharge was removed, with the road pavements constructed on top.

Placement of sand drainage blanket over Polyfelt® TS50 nonwoven geotextile

Client: Public Works Department (JKR), Malaysia.

Consultants: ZAQ Engineering Service Sdn Bhd and Jurutera Perunding MM Sdn Bhd (Geotechnical Consultant), Malaysia.

Contractors: Lambang Setia Sdn Bhd (Package 1), Kobako Trading Sdn Bhd (Package 2), NZ Bina Sdn Bhd (Package 3) and Naza Construction Sdn Bhd (Package 4), Malaysia.

Installing PVD's through the sand drainage blanket into the soft foundation

Placing and compacting embankment fill over Mirafi® PET geotextile reinforcement

Placing and compacting embankment fill

Construction of pavement on top of embankment

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

Peninsular 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 Peninsular Malaysia, passing through the Malaysian States of Perak, Penang, Kedah and Perlis.

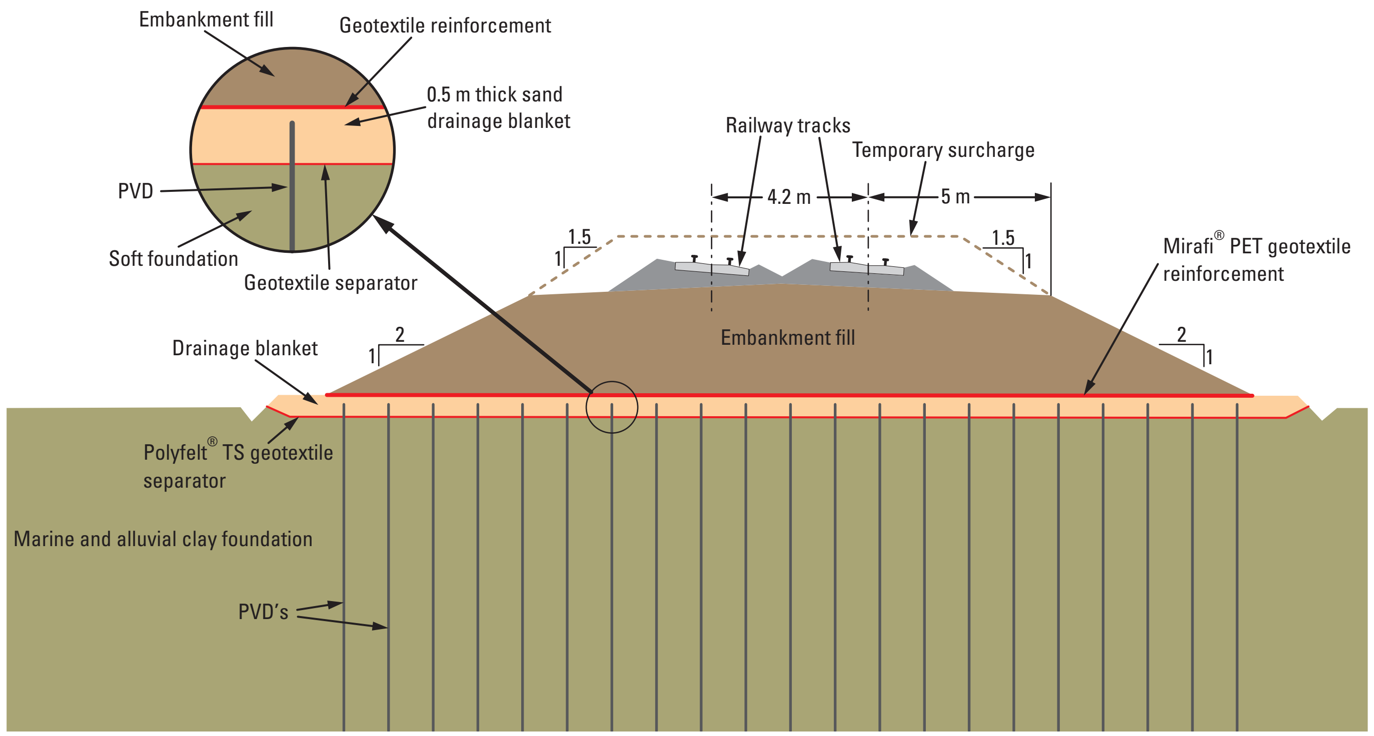
This railway project involves laying two new parallel tracks, replacing the existing single-track. It includes extensive foundation improvement works as well as the construction of bridges and tunnels over a wide variety of geological and ground conditions. Construction of new stations and installation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

Typical ground conditions across the alluvial valleys



Typical cross section through the railway embankments

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.

Placing Polyfelt® TS geotextile separator across the soft foundation

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.

Installing PVD's through the sand drainage blanket

Placing the embankment fill across the top of the Mirafi® PET geotextile reinforcement

Railway embankment completed

Specialist Consultant: G&P Geotechnics Sdn Bhd, Kuala Lumpur, Malaysia.

Basal reinforced embankments on soft soil:
Expressway embankments, Central Luzon Link Expressway, Philippines

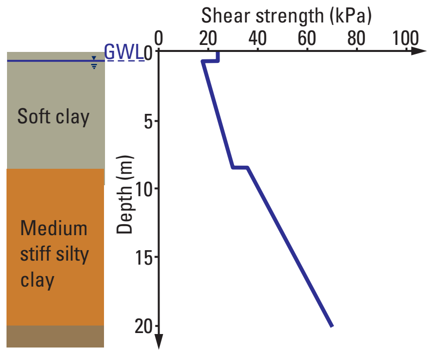
The Central Luzon Link Expressway (CLLEX) is a 30 km long, four-lane expressway linking several existing expressways in the Central Luzon region of the Philippines. The project aims to shorten the travel time from Metro Manila to Cabanatuan City which serves as the hub city for the Pacific Ocean Coastal Area Development. Once operational, the travel time from Tarlac City to Cabanatuan City is expected to be shortened from 70 minutes to 20 minutes.

The CLLEX traverses rural areas covered with agricultural farms where local rivers and farm roads intersect the CLLEX alignment. In these locations, abutment embankments are constructed for overpass bridge structures to minimise interruptions to the local communities.

The foundation soil profile along the length of the CLLEX mainly consists of a soft clay surface layer varying in depth from 2 m to 8 m. In many locations the ground water level (GWL) is near to the ground surface. The soft clay has an overconsolidated surface crust in the region above GWL. Below this upper soft clay layer is a medium stiff silty clay stratum.

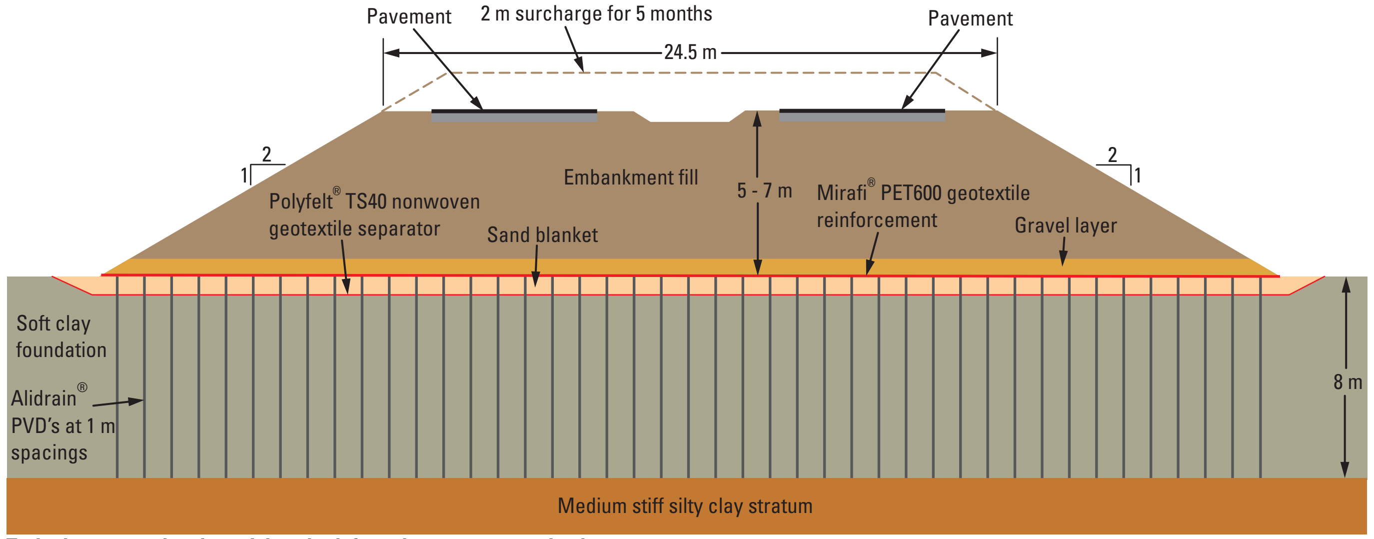
Embankment instability and excessive post-construction settlements over these soft foundation soils were a major concern for the project. Consequently, a combination of basal geosynthetic reinforcement and prefabricated vertical drains (PVD's) were used to improve initial embankment stability and accelerate consolidation of the soft foundation layer.

To provide the required embankment stability Mirafi® PET600 geotextile reinforcement was chosen as the basal reinforcement, while Polyfelt Alidrain® PVD's installed at 1 m spacings were chosen to accelerate the consolidation of the soft foundation. It was estimated that settlements of up to 1 m could be expected over a 5-month period with this PVD layout with a 2 m surcharge.



Typical foundation undrained shear strength profile along CLLEX

Installation of Polyfelt Alidrain® PVD's



Typical cross section through basal reinforced expressway embankments

In constructing the embankments, the surface vegetation layer was first removed and then a layer of Polyfelt® TS40 nonwoven geotextile separator installed. On top of this a sand blanket layer was constructed to remove the excess pore water from the base of the embankment and to provide a stable working platform for the PVD installation machines. After the PVD's had been installed, the Mirafi® PET600 geotextile reinforcement was laid in a continuous length across the width of the embankment. The geotextile reinforcement was sewn together onsite to provide a continuous coverage at the base of the embankments. Next, a 1 m thick gravel layer was placed over the Mirafi® PET600 geotextile reinforcement and compacted.

The embankment fill was placed in lifts and compacted to 95% Standard Proctor compaction. The expressway pavements were then constructed on top of the completed embankments. Finally, the side-slopes of the embankments were hydroseeded to provide vegetation growth.

Client: Department of Public Works and Highways, Philippines.

Consultant: Oriental Consultants Co, Ltd, Tokyo, Japan.

Contractors: Hunan Road & Bridge Const. (Package 1), China; China Road & Bridge Const. (Package 2), China; Ilsung Const. & Pacific Concrete JV (Package 3), South Korea; Qingdao Construction (Package 4), China.

Placement and compaction of embankment fill

Expressway section almost complete

Installing Mirafi® PET600 basal geotextile reinforcement above sand blanket layer

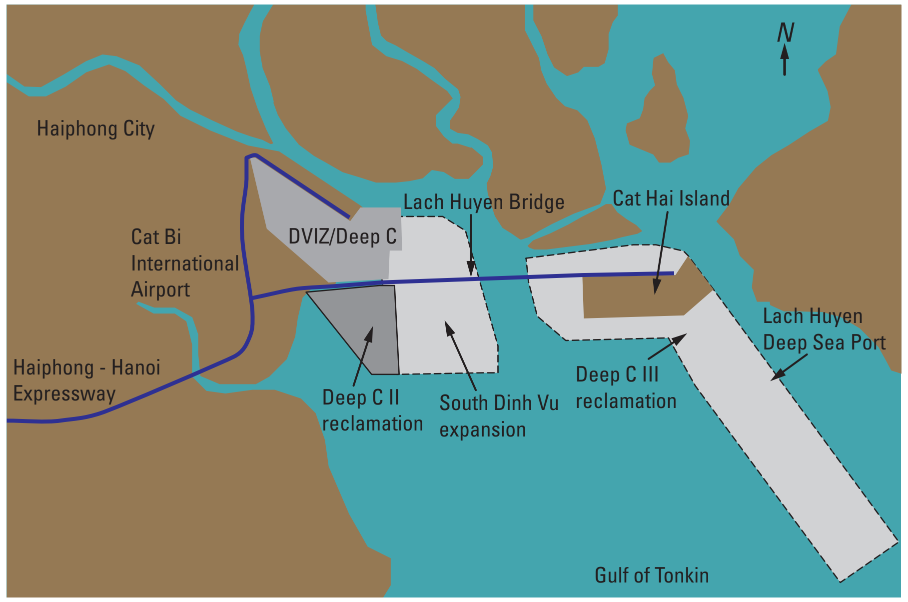
Basal reinforced embankments on soft soil: Deep C II reclamation dykes, Dinh Vu Industrial Zone, Vietnam

The Dinh Vu (Deep C) Industrial Zone (DVIZ) is located near the new Lach Huyen Deep Sea Port to the Southeast of Haiphong City, the major port area for Northern Vietnam. The various Deep C reclamations are being constructed to provide 3,000 ha of industrial development land, which is the largest industrial zone development in all of Vietnam. The Deep C II reclamation area has direct access to the new Lach Huyen Deep Sea Port via the newly completed Lach Huyen Bridge.

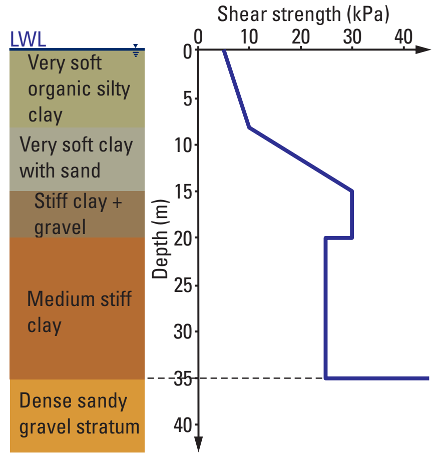
The hydraulic conditions at the Deep C II reclamation site consists of mean low water level (LWL) at CD+1.5 m, with mean high water level (HWL) at CD+3.0 m. Normal water currents are low, ranging from 0.2 to 0.3 m/s. However, the area is subject to extreme storm events from typhoons where water levels can rise to CD+4.5 m.

The seabed surface at the Deep C II reclamation site ranges from CD+1.5 m to CD+1 m thus, it is at or just below LWL. The upper 8 m of the seabed consists of very soft organic silty clay (marine clay) with the surface having an undrained shear strength of only 5 kPa. Below this is a 7 m thick layer of very soft clay with sand lenses. Below this are stiff clay layers, and finally at around 35 m depth a dense sandy gravel stratum. Because of the very low shear strength of the upper seabed layers the stability of any land reclamation structures is limited.

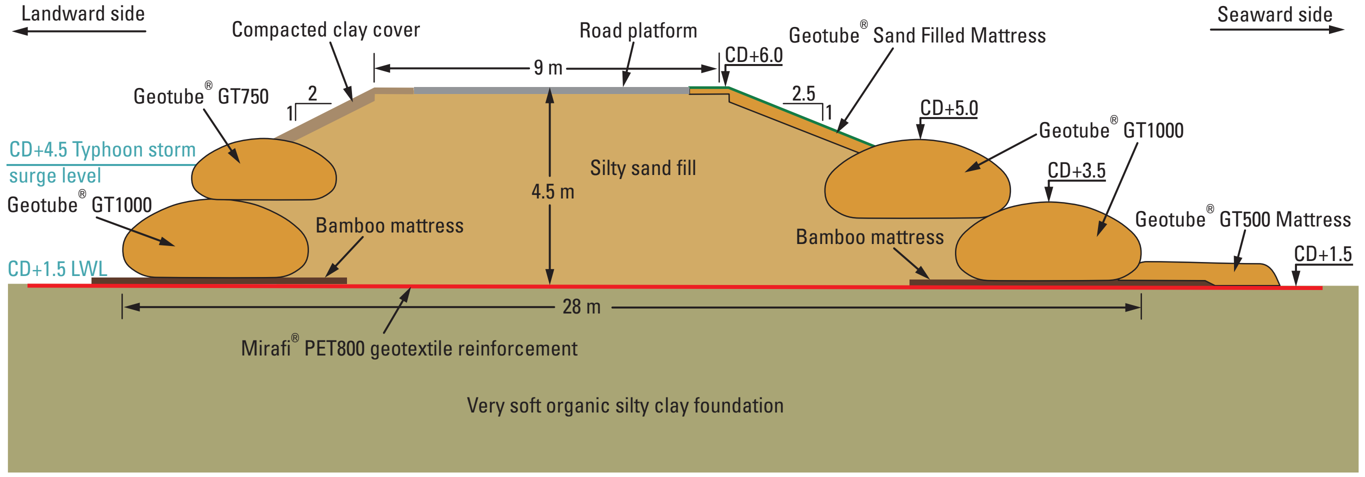
It was planned to construct sequentially several reclamation containment dykes extending out into the deep C II reclamation area. This was done to enable part of the site to be reclaimed and developed while waiting for further demand and expansion.



Location of Deep C II reclamation project



Foundation conditions at Deep C II reclamation site



Cross section through containment dykes at Deep C II reclamation

The design of the reclamation dykes consisted of locally dredged silty sand fill contained between outer Geotube® containment units. Geotube® units were chosen for the outer protection for the reclamation dykes for a number of reasons, namely:

- Geotube® units have an efficient contact surface with soft foundation soils and do not suffer from local shear failures as do rock fill.
- The sizing of the Geotube® units were such that they had adequate mass-gravity to resist water forces caused by extreme typhoon storms. The height of the Geotube® units surpassed that of the water height during typhoon storms.
- The Geotube® units to be provided had to have a long term design life in exposed conditions as it may be many years before any new expansion can cover the existing exposed dykes. This ensured there was adequate time for any reclamation expansion to cover over the previously installed Geotube® units. For the final outer dyke, rock armour is placed over the seaward side of the reclamation dyke to create a final, permanent protection for the reclamation.
- When expanding the reclamation area, it is not necessary to remove the Geotube® units as they can be just covered over with reclamation fill, and piled through during later development, unlike rockfill which would have to be removed during expansion.

To provide the required stability for the reclamation dykes a layer of Mirafi® PET800 geotextile reinforcement, which has an initial characteristic tensile strength of 800 kN/m, was placed across the base of the reclamation dykes on the seabed. The strength and strain characteristics of this geotextile reinforcement met the stability requirements for the reclamation dykes determined at the design stage. The Mirafi® PET800 geotextile reinforcement was installed in 35 m x 15 m panels at low tide and anchored in place using bamboo stakes.

To provide some bending rigidity bamboo mattresses were then installed beneath where the Geotube® containment units were to be located. Next, the bottom layer of Geotube® GT1000 containment units were laid out and filled with the locally dredged silty sand fill, with the area in between then also filled with the silty sand fill. This procedure was repeated with the second layer of Geotube® units, with the silty sand fill continuing up to the completed reclamation dyke level. Following this, an access pavement was constructed on top of the reclamation dyke to level CD+6.0 m. To prevent surface erosion of the dyke side slopes above the Geotube® units on the seaward side a 0.3 m thick Geotube® Sand Filled Mattress was used. To prevent toe instability on the seaward side of the dykes a 0.5 m thick Geotube® mattress was placed.

The Geotube® protected reclamation dykes have continued to perform as required. A number of them have remained in an exposed condition for over 4 years with no discernible loss of serviceability.

Client: Din Vu Industrial Zone Joint Stock Company, Haiphong, Vietnam.

Installation of Mirafi® PET800 geotextile reinforcement on very soft sea bed

Construction of the reclamation dykes

A completed reclamation dyke

Reclamation dyke 4 years after construction

Basal reinforced embankments on soft soil:
Embankment over old sludge lagoon, Interstate 670, Columbus, Ohio, USA

The extension of a 6-lane interstate freeway, I670 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 from 5 to 10 kPa, increasing with depth. The sludge was highly compressible with a compression index of 3.1 and a recompression index of 0.05. The permeability of the sludge was between 1×10^{-7} and 1×10^{-8} m/sec in both the horizontal and vertical directions.

Because of environmental concerns, the sludge could not be removed from site. Thus, a geosynthetic reinforced embankment, constructed across the top of the sludge, proved to be the only cost-effective option. The required height of the embankment ranged between 4 m and 12 m. To prove the design concept and derive accurate engineering performance parameters, a fully instrumented test fill was constructed with varying prefabricated vertical drain (PVD) spacings and varying geosynthetic reinforcement, and this was monitored for 2 years.

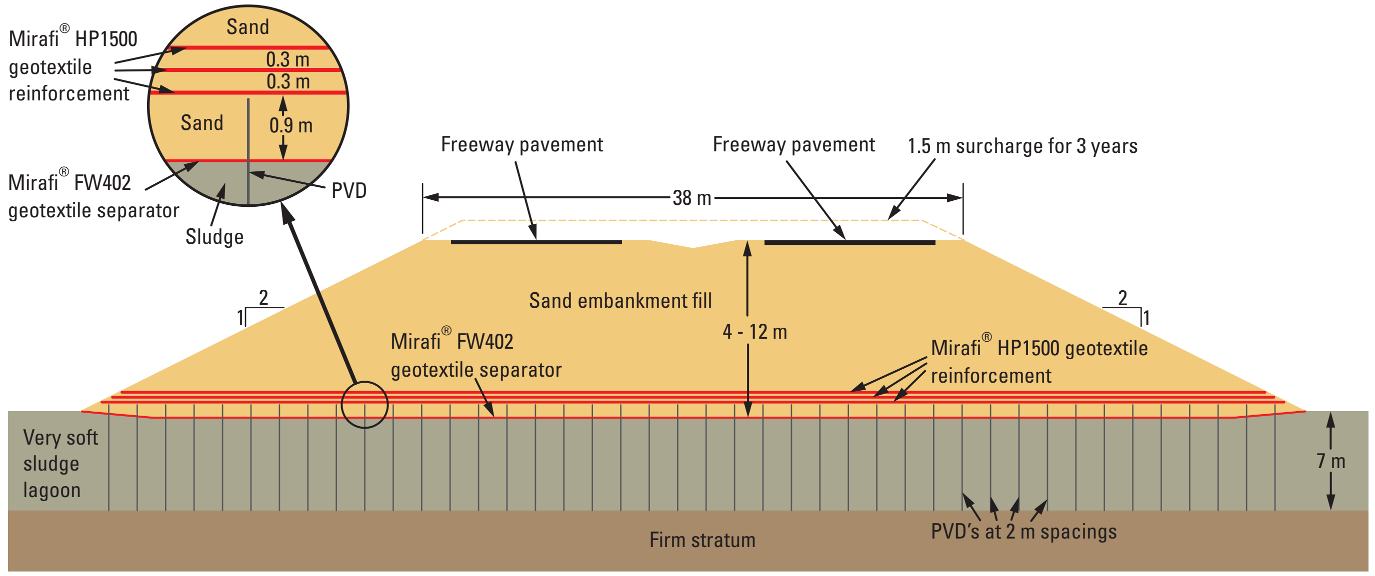
The geosynthetic reinforcement for the final embankment design was determined using a limit equilibrium approach. The design allowed for 3 layers of Mirafi® HP1500 geotextile reinforcement at the base of the embankment to develop the required short term stability. Mirafi® HP1500 is a high modulus, woven 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

Spreading sand working platform across Mirafi® FW402 woven geotextile separator



Typical cross section through the basal reinforced embankment

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 seamed together 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 over 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.

Installation of PVD's

Placement of embankment fill over Mirafi® HP1500 geotextile reinforcement

Embankment nearing completion

Interstate 670 embankment completed

Basal reinforced embankments on soft soil: Runway overrun area, La Guardia International Airport, New York, USA

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 project entailed the construction of a 150 m long by 230 m wide overrun area, which would be predominantly covered with grass, but would also have a jet blast pavement area as well as an emergency access roadway.

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.

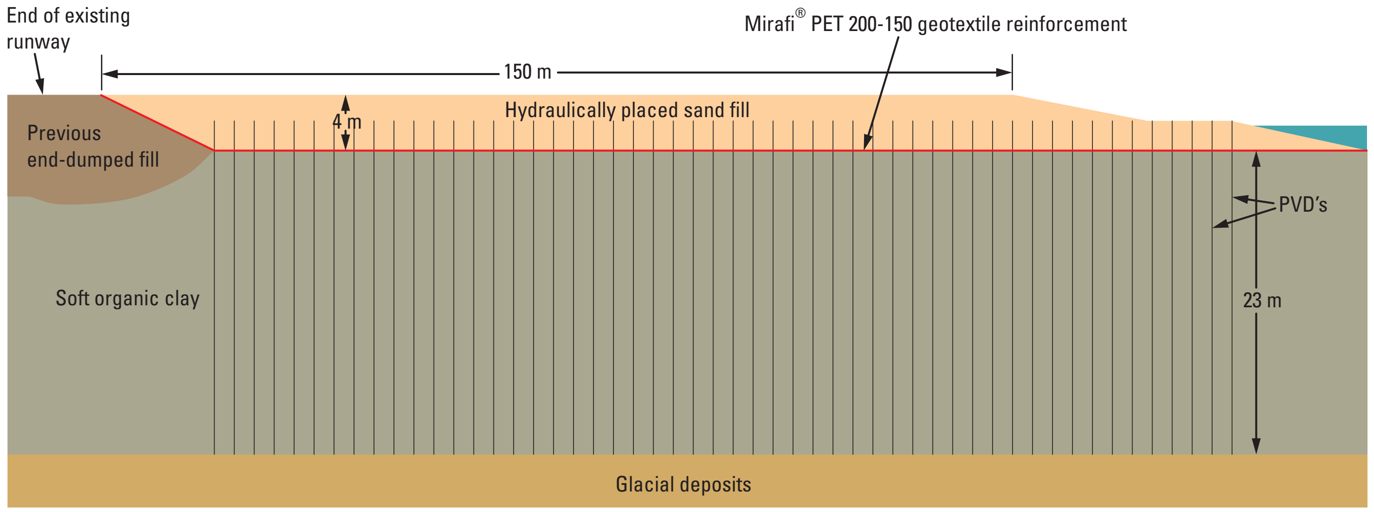
The historical approach to land reclamation at La Guardia had been end-dumping of fill, which had created extensive, uncontrolled mud waves. Mud wave creation was deemed unacceptable for this project due to the close proximity of a federal shipping channel and community concerns about increased "low-tide"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

Fabricating Mirafi® PET200-150 geotextile reinforcement into large sheets on barges



Typical cross section through the runway overrun area

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 90 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 1V: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.

Deploying Mirafi® PET200-150 geotextile reinforcement close to the shoreline

Deploying Mirafi® PET200-150 geotextile reinforcement into the water from barges

Sand fill placement to create the runway overrun area

Basal reinforced embankments on soft soil: West Salym Communication Corridor, West Siberia, Russia

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.

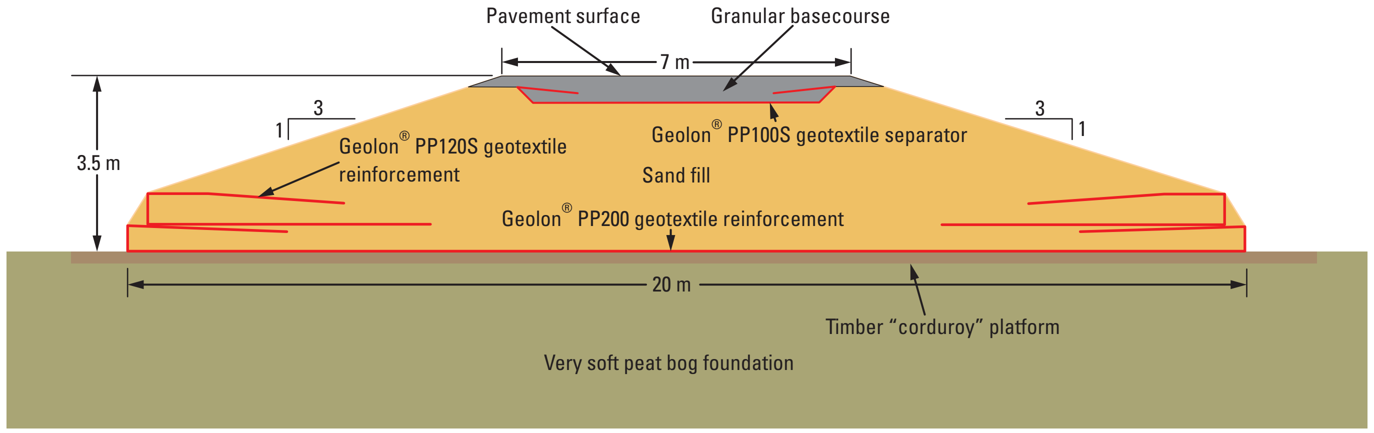
The road system had a required design life of 25 years after which it had to be dismantled and the area returned to its original pristine condition. This 25 year design life reflected the life time for oil extraction in the area. Further, after construction, the road system was required to have minimal maintenance in order to maintain traffic flows, and this lead Engineers to decide on the use of a compacted fine-crushed rock base 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

Very soft foundation conditions at peat bogs



Cross section through the basal reinforced embankments constructed over peat bogs

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® PP200 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 embankments were 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.

Embankment earthworks construction partially completed

Placing Geolon® PP100S geotextile separator prior to placement of road base course

Completing road base course layer on top of embankment

Completed communication corridor after 5 years of use

Basal reinforced embankments on soft soil: Partially submerged containment dam, Doeldok, Antwerp, Belgium

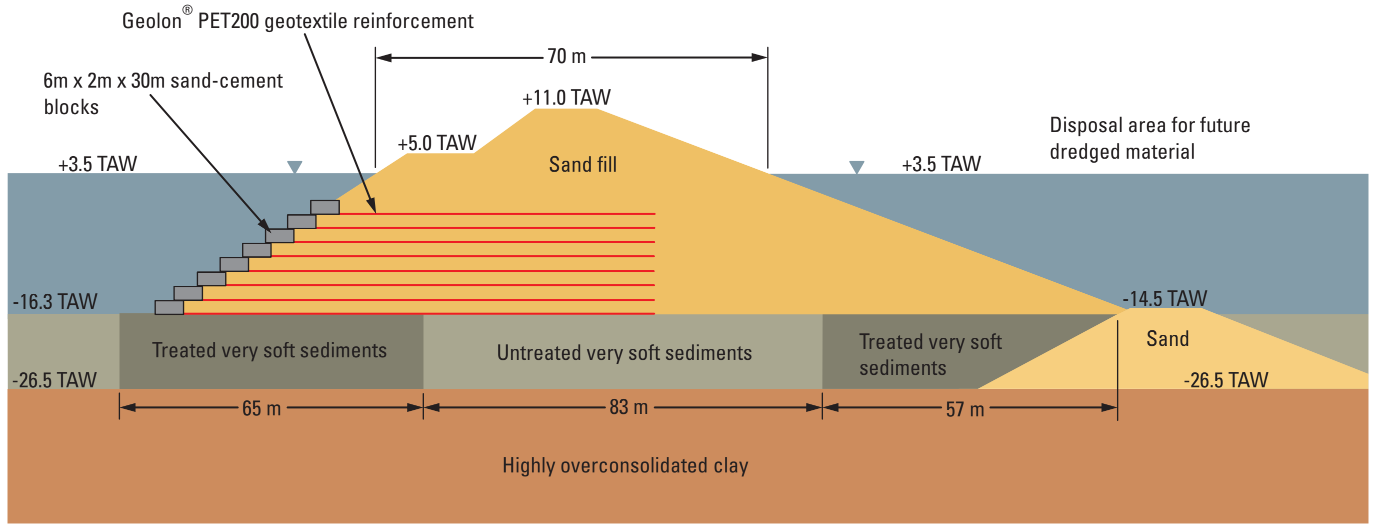
As part of the ever-increasing expansion of Antwerp Harbour, increased capacity of disposal facilities for dredged material and excavated soil have to be found. A solution to this problem has involved the construction of a partially submerged containment dam across an old, existing dock to contain spoil 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 bouyant 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 high 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 the block units prior to casting and the units were then lifted by means of a detachable steel loading frame.

On the outside of the wall, the large block facings with the attached geotextile reinforcement were installed using the floating crane. After partially unrolling the geotextile reinforcement, another large block unit was then installed immediately behind the outer wall face, resulting in a total installed block size of 6 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.



Typical cross section through the reinforced 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 fill 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 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.

Client: Ministerie van de Vlaamse Gemeenschap, Administratie Waterwegen en Zeewezen, Sint Niklaas, Belgium.

Consultant: Dredging International bv and Jan de Nul bv JV, Zwijndrecht, Belgium.

Contractor: Dredging International bv and Jan de Nul bv JV, Zwijndrecht, Belgium.

Formwork for sand-cement block

Fabricating the sand-cement block face units on land

Lifting a fabricated facing unit with attached Geolon® PET200 geotextile reinforcement for placement underwater

Fabricating on land the sand-cement block face with the 100 m long Geolon® PET200 geotextile reinforcement

Basal reinforced embankments on piles: Bridge approach embankments, M74 Motorway Completion, Glasgow, UK

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 derelict, numerous businesses were affected.

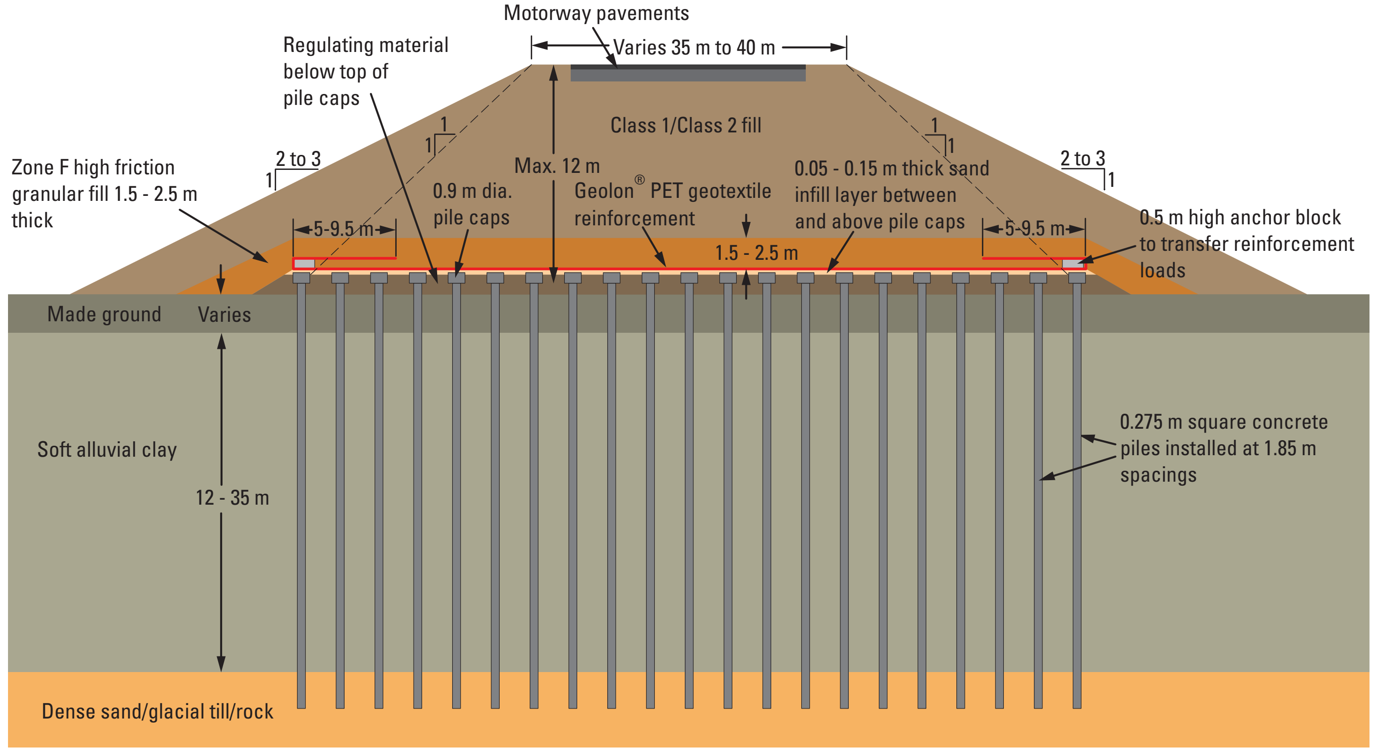
To enable rapid construction and prevent differential settlements, the 13 approach embankments to the grade-separated junctions have piled foundations with basal geosynthetic reinforcement. Each approach embankment was designed specifically in terms of pile spacing, pile cap size and basal geosynthetic reinforcement strength and extension. The design of the basal reinforced piled embankments was carried out in accordance with BS8006:2010, an internationally recognised design code.

To support the approach embankment loadings, 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 1.6 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 insitu, 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,

Placing Geolon® PET geotextile reinforcement across the top of the pile caps



Typical cross section through the approach embankments

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.

Lapping Geolon® PET geotextile reinforcement around anchor block at edge of piled area

One of the approach embankments under construction

One of the approach embankments nearing completion

Basal reinforced embankments on piles: Shallow basal reinforced piled embankment, Reeuwijk Bypass, Netherlands

Reeuwijk is a village in the South-Central part of the Netherlands, and historically was established as a centre for peat harvesting. Consequently, the ground in the area consists of thick layers of peat and organic clays, ranging around 8 m in depth. These are compressible and very soft. Further, the ground water level is high at 0.2 to 0.5 m below ground surface.

To improve the capacity of the existing road through Reeuwijk, but also limit traffic congestion in the village centre, a 3.5 km bypass road was constructed around the village. The new bypass road is a single carriageway in both directions with extra lanes at road junctions.

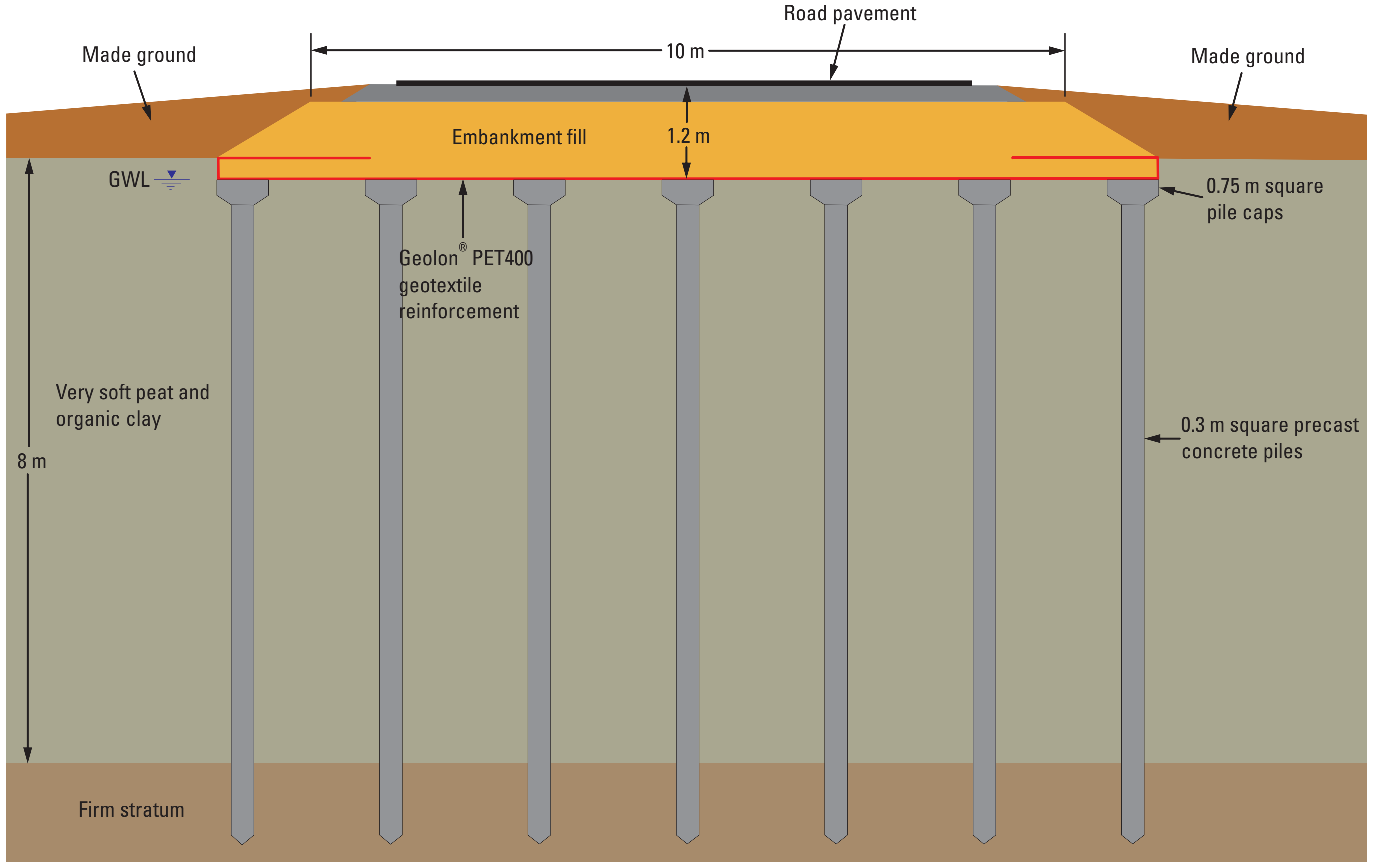
To construct the new bypass road over these very poor ground conditions, the construction method had to be selected carefully. Using embankment fills alone would have resulted in high settlements, instability and high post-construction maintenance costs. It would have also adversely impacted the underground services that existed along the bypass alignment. Further, one of the main conditions for the bypass road was that its surface level approximated the level of the surrounding ground (thus no embankment structures could be constructed). To construct a shallow embankment layer and at the same time prevent undue settlements led the designers to adopt the basal reinforced piled embankment technique for the bypass road.

To enable the basal reinforced piled embankment structure to meet the 100 year design life requirement 0.3 m square precast concrete piles were used to support the embankment. On top of these piles, 0.75 m square concrete pile caps were used.

To maximise the embankment height available for arching across adjacent pile caps it was decided to excavate the surface soil down to the water table level, around 0.2 m below ground surface. At the same time, the embankment height was raised to a level of 1.2 m (this embankment height included 0.25 m of pavement) so that the embankment would not be too thin in spanning across the pile caps. Also, to maximise the arching effect, highly granular embankment fill consisting of recycled crushed demolition waste was used.

The Dutch Design Guideline CUR226:2010 was used to design the pile foundation layout and the required strengths of the basal reinforcement in both the longitudinal and transverse directions. For a shallow embankment of this height (H = 1.2 m) CUR226:2010 allows a maximum pile spacing in the transverse direction of 1.9 m with a consequent basal reinforcement strength of 400 kN/m in both transverse and longitudinal directions, based on allowable design loads, strain and required 100 year design life. Geolon® PET400 geotextile reinforcement was selected as the basal reinforcement because it met the strength requirements and at the same time has high tensile stiffness to carry the tensile loads at low strain.

Placing Geolon® PET400 geotextile reinforcement across the tops of the pile caps



Cross section through the shallow basal reinforced piled embankment

The construction of the basal reinforced piled embankment structure involved first excavating down to the ground water level. After this, the piles and pile caps were installed at the required geometry. Then, the Geolon® PET400 geotextile reinforcement was installed across the tops of the pile caps in both the longitudinal and transverse directions. Next, the embankment fill was placed in layers and compacted. Finally, the pavement layers were constructed. To make the shallow embankment blend into the surrounding environment, made ground was filled to the height of the pavement surface.

Client: Municipality Bodegraven-Reeuwijk, Netherlands.

Consultant: Crux Engineering, Netherlands.

Contractor: Heijmans (Design and Construct), Netherlands.

CUR226:2010 Ontwerprichtlijn paalmatrassystemen, Stichting CUR, Gouda, Netherlands.

Construction of shallow basal reinforced piled embankment

Excavating down to water table level for the base of the embankment

Construction of the shallow basal reinforced piled embankment alongside the canal

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 BS8006:2010, 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.

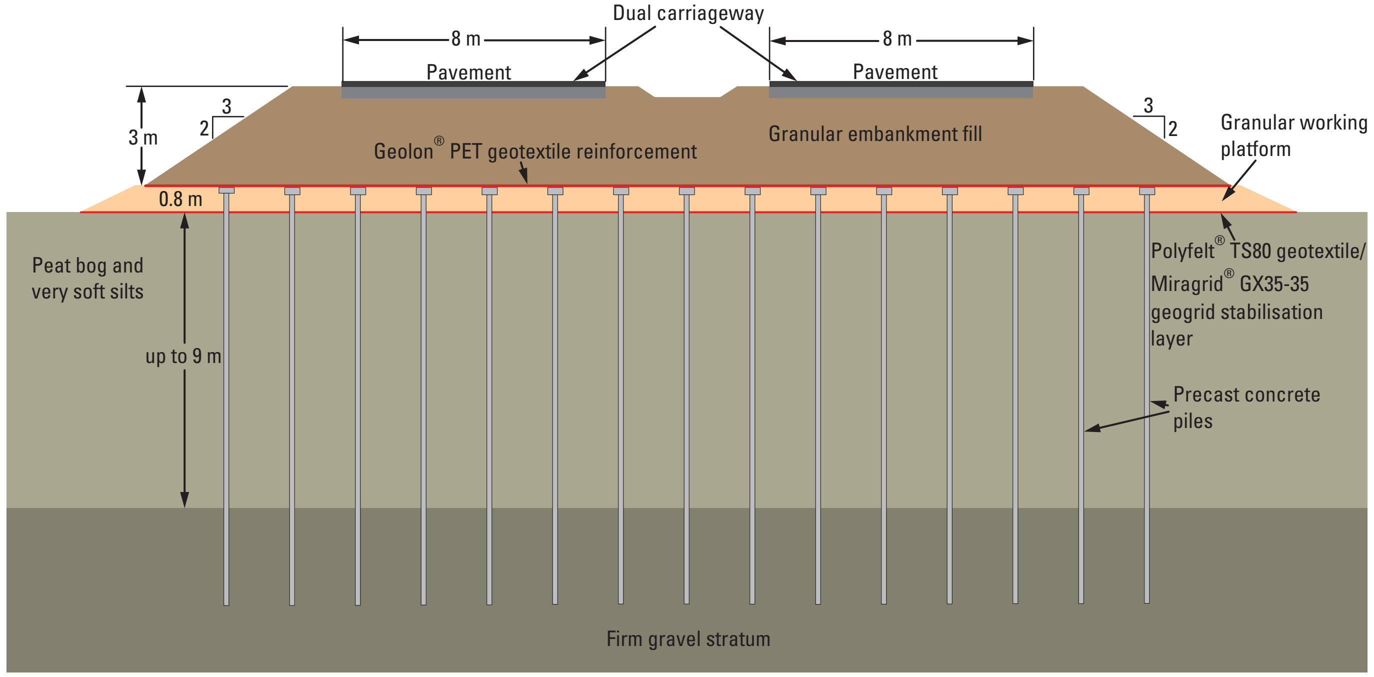
The final design incorporated Geolon® PET geotextile reinforcement across the tops of the pile caps. Depending on the embankment height two different geotextile reinforcement combinations were used. One combination consisted of 600 kN/m longitudinal and 700 kN/m transverse strengths, while the second 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® GX35-35 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

Polyfelt® TS80 geotextile/Miragrid® GX35-35 geogrid reinforced working platform across the peat bog



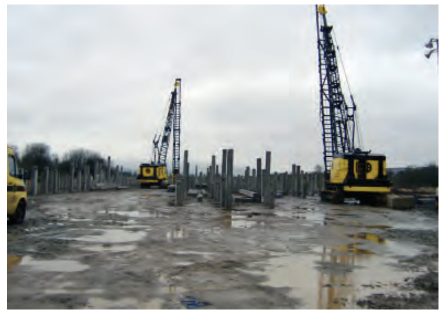
Typical cross section through the Geolon® PET geotextile reinforced piled embankment

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.



Driving concrete piles through the working platform

Consultant: RPS Consulting, Dublin, Ireland.

Contractor: SIAC Ferrovial JV, Ireland.

BS8006:2010 Code of practice for strengthened/reinforced soils and other fills, British Standards Institution.

Laying Geolon® PET geotextile reinforcement across the top of the pile caps

Placing and compacting embankment fill over Geolon® PET geotextile reinforcement

Completed piled embankment

Basal reinforced embankments on piles: Wat Nakorn-In bridge approaches, Bangkok, Thailand

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 Nonthaburi 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", overlying 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/m^3 to 16 kN/m^3.

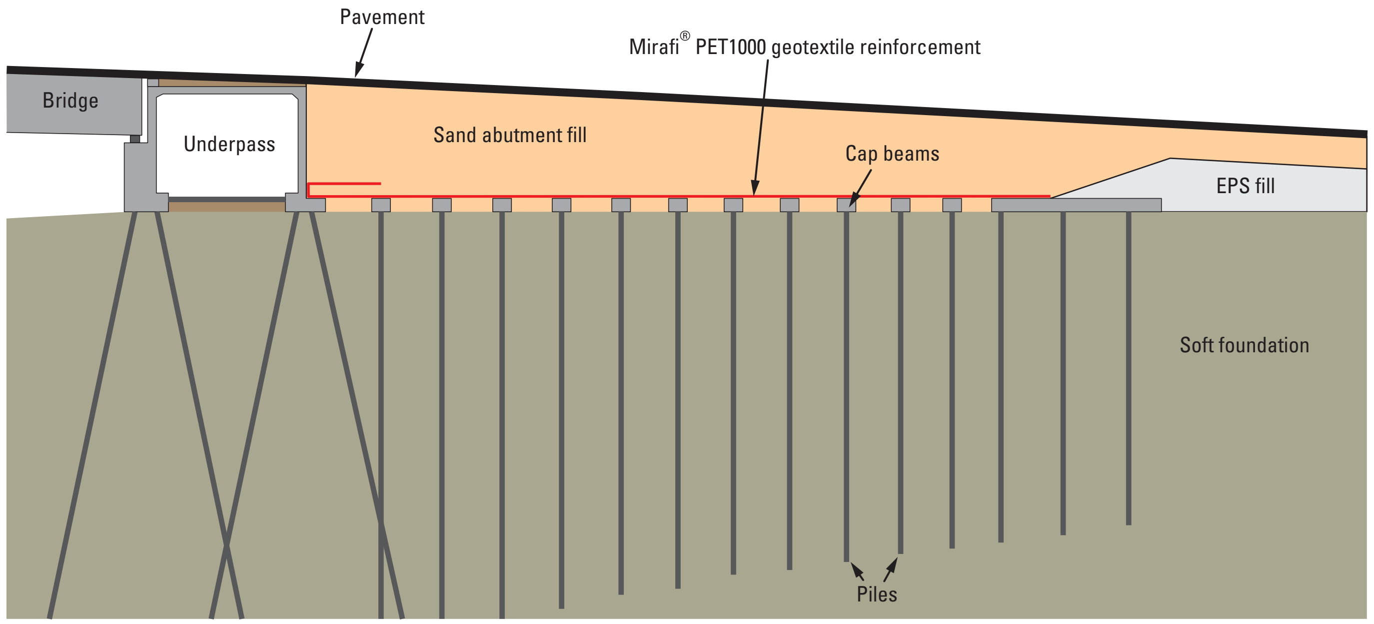
Consolidation of the soft clay can lead to large differential settlements between embankments constructed directly on the clay, and any piled bridge structures. These differential settlements reduce riding quality and pose safety hazards. They also involve frequent maintenance works, which prove costly 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 1m 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®

Piling the embankment foundation



Typical long section through the basal reinforced piled bridge approaches

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® 5XT 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.

Placing Mirafi® PET1000-100 geotextile reinforcement

Construction of pile caps and connecting beams

Embankment steep reinforced fill slope

Constructing asphalt pavement

Basal reinforced embankments spanning voids:
High speed railway over karst foundation, LGV East, Lorraine, France

The high speed LGV railway connection between Paris, Bratislava and Budapest constitutes a 1,500 km long railway corridor of major European importance. In France, the LGV East connects Paris to Strasbourg, and it will then continue on into Germany and then onto the Czech Republic and Slovakia. The first phase of the LGV East from Paris to Baudrecourt was completed in 2007. The second phase from Baudrecourt to Vendenheim, near Strasbourg, began in 2010.

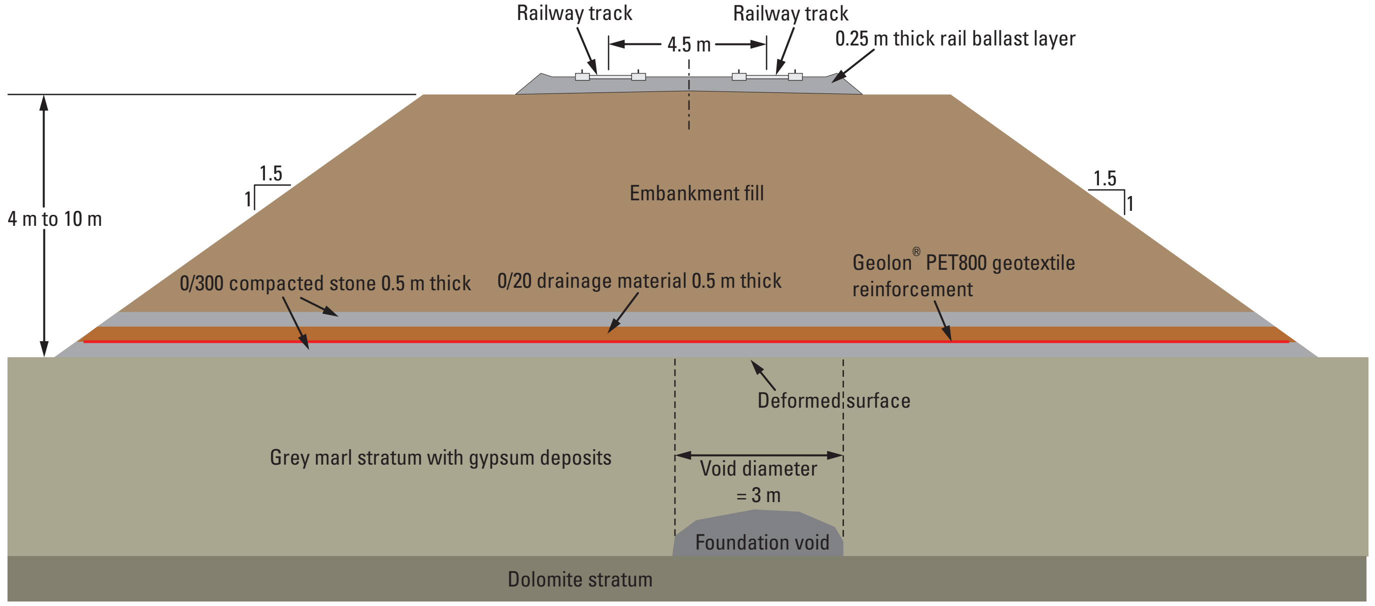
In 2011, on section 42 near Sarrebourg foundation subsidence was observed during preliminary earthworks along the railway alignment. At the foundation surface, voids of 3 m diameter were observed where an embankment varying in height up to 10 m was to be constructed.

The geology of the area displays a thin layer of silt at ground surface, under which lies a stratum of overconsolidated marls with agglomerations of gypsum throughout. Below this is a stratum of dolomite. Most of the gypsum deposition is located immediately above the dolomite stratum, and due to the presence of groundwater the gypsum has been dissolved leaving voids in many locations above the dolomite stratum. The presence of these subterranean voids has resulted in subsidence at the ground surface in many locations.

To prevent the detrimental effects of foundation subsidence and differential deformations propagating up through the embankment fill and impacting the performance of the rail track structure it was decided to install a combination of highly granular layers ($\phi' > 43°$) and geotextile reinforcement across the base of the embankment. The granular layers consisted of 0/300 mm stone sizes and were compacted into layers 0.5 m thick. These layers had high dilatancy and so would be expected to provide good resistance to any subsequent vertical deformations arising at the base of the embankment. The geotextile reinforcement sandwiched within this granular layer system was Geolon® PET800 geotextile reinforcement (initial tensile strength 800 kN/m) which provided the tensile strength and stiffness to maintain serviceable conditions at the base of the embankment. A 0/20 mm gravel drainage blanket of 0.5 m thickness immediately on top of the Geolon® PET800 geotextile reinforcement was used as protection for the geotextile.

For the design of the basal reinforced embankment system a design void diameter of 3 m was assumed as this was considered representative of the subsidence that would occur at the base of the embankment. The design analysis was carried out using the RAFAEL method which assumes a cylindrical failure at the sides of the void along with associated soil relaxation. The maximum deflection of the geosynthetic reinforcement under these conditions and then the allowable deformation is a function of the settlement at the ground surface. The RAFAEL analysis showed that to meet the embankment surface deformation limits the maximum tensile strain in the basal geotextile reinforcement should be limited to < 5% over a 100 year design life. The use of Geolon® PET800 geotextile reinforcement satisfied this criterion.

Subsidence found at the base of the railway embankment



Cross section through the basal reinforced embankment over the area prone to subsidence

The foundation preparation works involved a survey of cavities along the railway embankment alignment. In zones where there was a high concentration of cavities, it was planned to fill the voids by injection grouting after the embankment had been constructed. In the remaining areas it was planned to rely on the basal reinforcement for spanning any future subsidence.

After the Geolon® PET800 geotextile reinforcement had been unrolled over the 0/300 mm compacted stone layer at the base of the embankment a 0/20 mm gravel drainage blanket was spread on top and compacted to 0.5 m thickness. The purpose of the drainage blanket was to provide protection to the Geolon® PET 800 geotextile prior to placement of a further 0/300 mm compacted stone layer. Following compaction of this drainage layer, a trench was excavated in order to allow visual inspection of the geosynthetic reinforcement. It was observed that there was negligible apparent damage to the geosynthetic reinforcement.

Laying Geolon® PET800 geotextile reinforcement over stone arching layer

Following the placement of the basal granular layers and geotextile reinforcement the construction of the embankment was completed, with the railway structure constructed on top.

The use of geotextile reinforcement to span voids has become a common technique which can be applied to earthworks under deformation-sensitive structures like high-speed railway tracks, even for relatively wide cavities.

Client: RFF (SNCF réseau currently), Paris, France.

Consultant: Inexia Arcadis, Paris, France.

Contractor: Guintoli Valérian, Paris, France.

Placing granular protection layer over Geolon® PET800 geotextile reinforcement

Investigation of mechanical integrity of Geolon® PET800 geotextile reinforcement beneath granular protection layer

High speed railway track completed

Basal reinforced embankments spanning voids:

Roadway embankment, Bad Wünnenberg Bypass, North Rhine-Westphalia, Germany

Bad Wünnenberg is a town in the state of North Rhine-Westphalia, Germany. It is situated on the river Aabach, approximately 20 km south of Paderborn. The new section of the road B480n links to the Aftetal bridge, an 800 m long steel composite structure, which spans the Afte Valley at a height of almost 70 m and is the central feature of the Bad Wünnenberg bypass.

To withstand high traffic loads, Federal German roads require good quality foundations. This requirement was not achieved on a section of the B480n Bad Wünnenberg Bypass between the Rhenish Slate Mountains and the Münsterländer Chalk Basin, where the B480n passes over a wide chalk karst area. The underground cavities can rise to the ground surface, especially in cuttings, resulting in local deformations and subsidence.

Since ground subsidence could not be completely ruled out along the planned route, the application of basal geotextile reinforcement was applied as a form of insurance against possible future subsidence. If foundation voids were to arise in the future beneath the road structure then the geotextile reinforcement would be required to span across any formed depressions thus maintaining the road in a serviceable condition.

Along the planned route of the road bypass its alignment changes from cuttings to embankment fills up to 7 m in height. In the cuttings where most subsidence problems are expected to occur fill heights of 1.1 m were used.

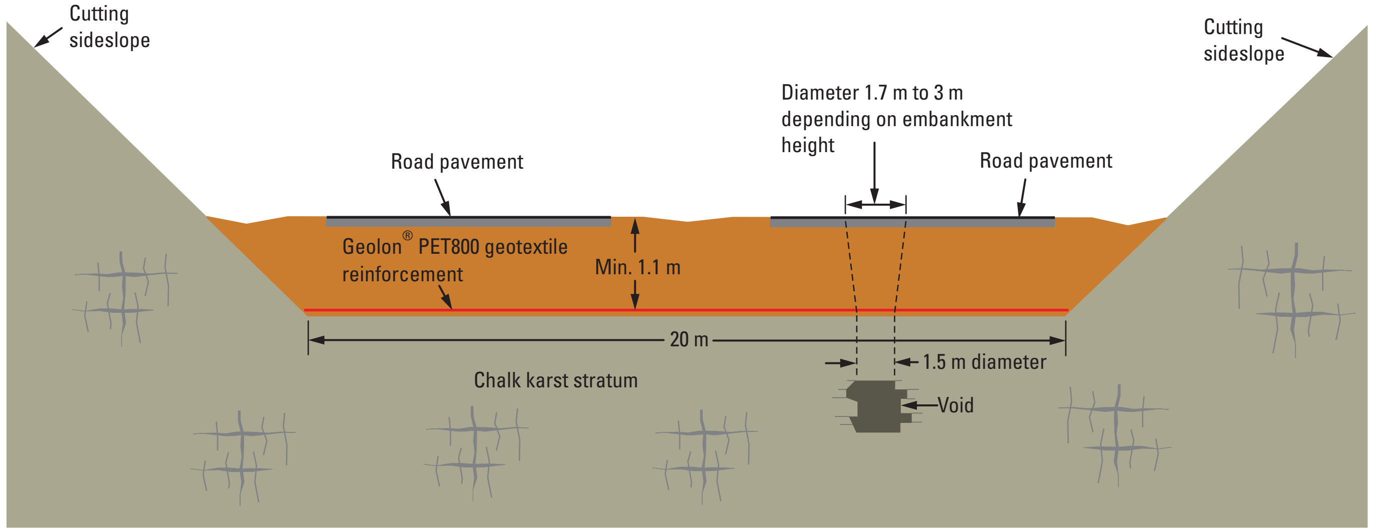
Based on the results of the ground survey and geotechnical analysis, it was found that initial void diameters of 1.5 m could be expected in the fissured foundation stratum beneath the roadway alignment. These voids would result in potential vertical subsidence beneath the earth fills.

In the cutting sections where 1.1 m of earth fill was used an analysis using the RAFAEL subsidence method showed that the expected subsidence diameter at pavement surface would be 1.7 m. In the embankment sections the same analysis showed the expected subsidence diameter at pavement surface to range up to 3 m depending on embankment height. The resulting maximum allowable geotextile reinforcement strains ranged from 2% to 6% depending on the height of earth fill on the basal geotextile reinforcement.

The required basal reinforcement properties were determined in accordance with EBGEO (2011). This resulted in a design tensile strength of at least 300 kN/m over a 100 year design life at the required strain levels. Geolon® PET800 geotextile reinforcement, with an initial tensile strength of 800 kN/m, fulfilled the long term design requirements and was subsequently used for the basal reinforcement.

The subgrade surface was first prepared by smoothing and compacting the ground surface. Next the layer of Geolon® PET800 geotextile reinforcement was installed in the direction along the length of the roadway. On top of the geotextile reinforcement granular fill material, having good dilatancy properties, was placed and compacted. Further lifts of granular fill were placed and compacted until the level for the pavement subgrade was achieved. Finally, the roadway pavements were constructed on top.

Site preparation works



Cross section through the cutting section of the road structure

Client: Landesbetrieb Strassenbau Nordrhein County, Germany.

Consultant: Dr Spang GmbH, Witten, Germany.

Contractor: Amand GmbH & Co. KG, Germany.

EBGEO (2011) Recommendations for design and analysis of earth structures using geosynthetic reinforcements, Wilhelm Ernst and Sohn, Germany.

Compacting embankment fill

Placing and compacting granular fill over Geolon® PET800 geotextile reinforcement

Basal reinforced embankments spanning voids: High speed railway embankment over karst foundation, Guizhou Province, China

Guizhou Province, located in the South West of China, consists of largely mountainous terrain with elevations ranging from around 590 m to 2,900 m. It is home to China's 500 m Aperture Spherical Radio Telescope (FAST) which is the world's largest single dish radio telescope. Guizhou Province aims to connect every city in the province with nearly 2,000 km of high speed railway lines by 2022. The 125 km long Anshun - Liupanshui high speed railway section was put into operation in 2020, reducing the travel time between Guiyang (the Guizhou Provincial capital) and Liupanshui (the border city with Yunnan Province) from 3.5 hours to 1 hour. The design speed of the Anshun - Liupanshui high speed railway is 250 km per hour.

Guizhou Provinces's geology consists of a sequence of limestone strata, thousands of metres in thickness. Around three-quarters of Guizhou Province is made up of karst landscapes, earning it the title ''Karst Province'' of China.

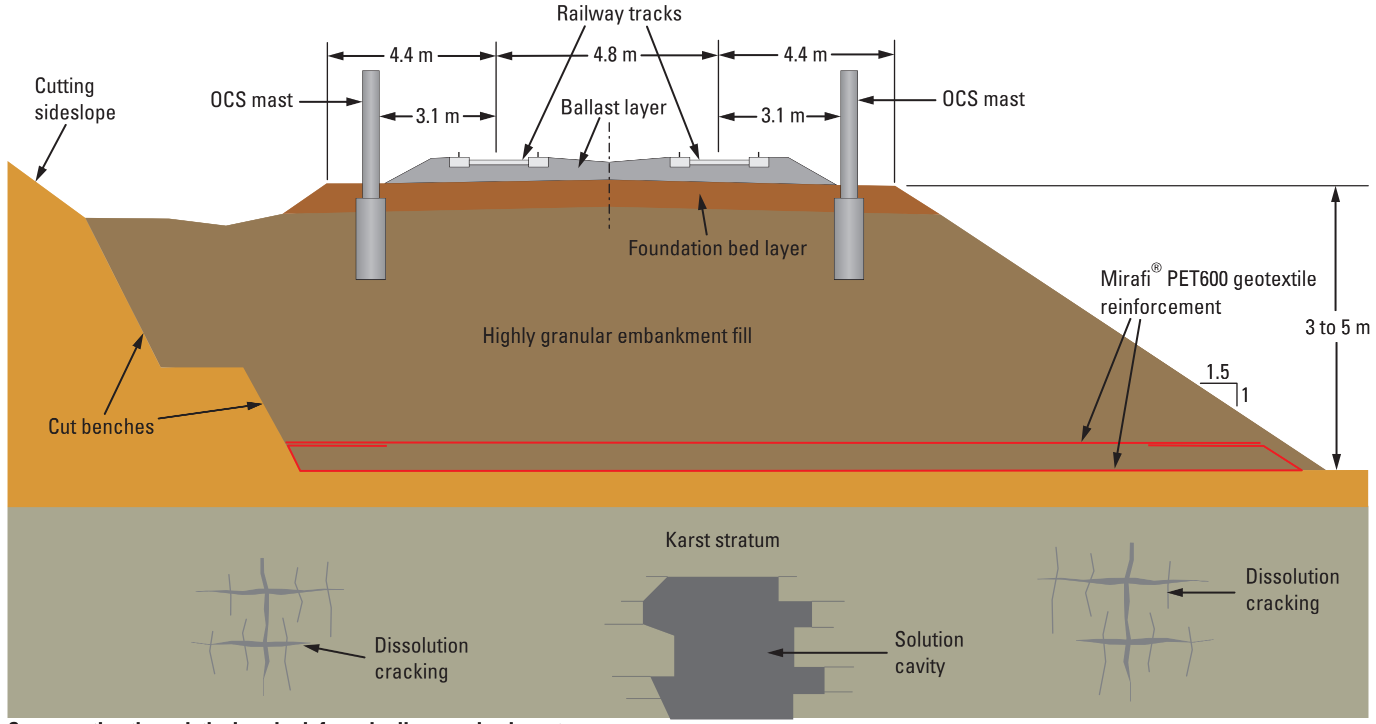
From a geotechnical standpoint, karst terrain is one of the most challenging ground conditions in foundation engineering. Above ground it is easy to see the unique shapes of limestone outcrops; some are massive while others are in the form of towers, pinnacles and cones. Therefore, it is easy to imagine that the subterranean soil/rock interface would reflect what is seen in the outcrops. As limestone is soluble, solution channels are common features in the foundation rock mass. Sometimes the roof of the solution channel collapses with the overburden soil falling into the solution channels. In more severe cases, total overburden soil collapse occurs, resulting in sinkholes becoming visible at ground surface. Such conditions make the ground highly variable in strength and prone to localised deformations and collapse over time.

The Anshun - Liupanshui high speed railway was constructed using a combination of viaducts, tunnels and embankments. In one section, the railway was constructed in a cutting where the subterranean karst features were close to the ground surface. Here, the ground was variable in terms of subgrade stiffness over short distances and there was a risk of subsidence occurring over the design life of the railway.

As high speed railways have strict differential settlement design limits for travel comfort and safety, the railway embankment in the cutting was designed using basal geotextile reinforcement. Two layers of Mirafi® PET600 geotextile reinforcement were included at the base of the embankment with the lower layer laid in direction across the embankment and the upper layer laid in direction along the embankment. Mirafi® PET600 geotextile reinforcement is composed of high modulus PET yarns and has an initial tensile strength of 600 kN/m. The Mirafi® PET600 geotextile reinforcement acts as insurance to limit the deformation at the base of the railway embankment should subsidence occur in the foundation beneath.

The construction procedure for the project was as follows. The ground surface was first levelled. The first layer of Mirafi® PET600 geotextile

Exposed karst formations in Guizhou Province



Cross section through the basal reinforced railway embankment

reinforcement was then laid in one continuous piece in the direction across the embankment. A 250 mm lift of gravel sand layer was then place over the first layer of Mirafi® PET600 and compacted to specification. The second layer of Mirafi® PET600 was then laid in one continuous piece in the direction along the embankment length. Granular embankment fill was then placed in lifts on top and compacted until the required embankment height was reached. Once that was achieved, the rail track superstructure was constructed on top.

Client: Guizhou An-Liu Railway Co., Ltd., Guizhou Province, China.

Consultant: China Railway Eryuan Engineering Group Co., Ltd., Sichuan Province, China.

Contractor: China Railway 12th Bureau Group Co., Ltd., Shanxi Province, China.

Spreading embankment fill across top of Mirafi® PET600 geotextile reinforcement

Compaction of embankment fill

Mirafi® PET600 geotextile reinforcement installed at base of railway embankment

Completed high speed railway in Guizhou Province

Basal reinforced embankments spanning voids: High speed railway 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.

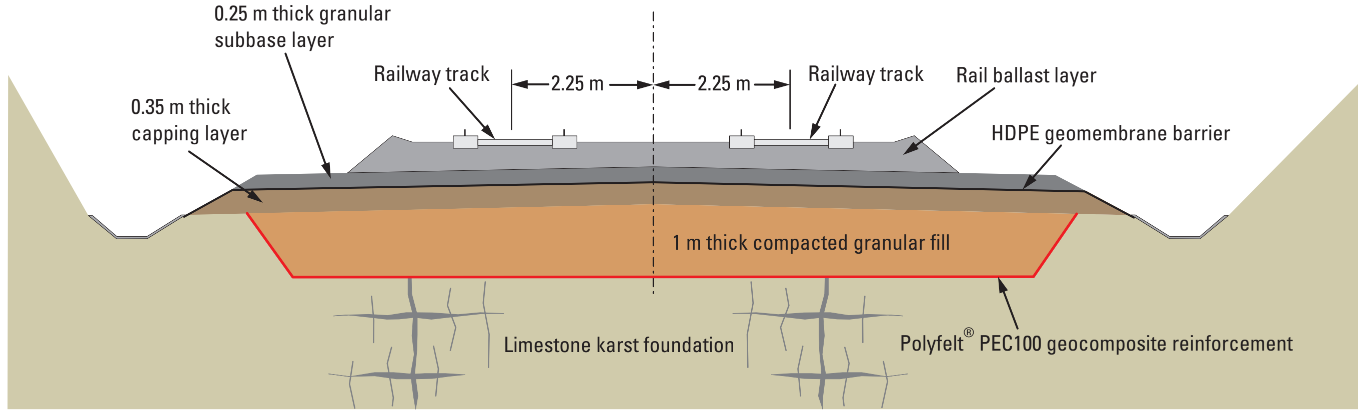
The geosynthetic reinforcement solution was employed in railway cuttings in this area where the likely foundation cavity diameters were quite small (≤ 0.5 m). Here, the limestone foundations were over-excavated by 1 m and a layer of Polyfelt® PEC100-100 composite reinforcement was placed over the excavated 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.

Excavation of limestone foundation prior to installation of Polyfelt® PEC geocomposite reinforcement



Typical cross section through the basal reinforced track structure

Client: Gestor de Infraestructuras Ferroviarias, Madrid, Spain.

Consultant: Prointec, Madrid, Spain.

Contractor: Ute Gajanejos, Madrid, Spain.

Placement of Polyfelt® PEC100-100 geocomposite reinforcement

Placing granular fill over Polyfelt® PEC100-100 geocomposite reinforcement

Placement and compaction of the granular fill layer

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

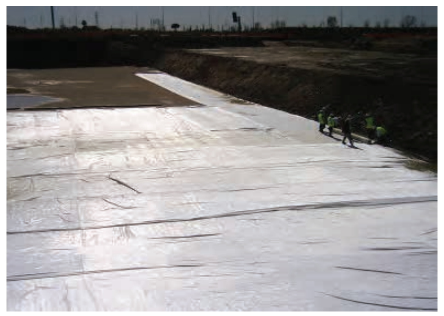
A geotechnical investigation of the proposed stadium site showed that there was a substantial stratum of anthropic material varying in thickness between 4 m and 13 m over the site. This anthropic material was the remains of an old solid waste landfill site that consisted of solid industrial and construction 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.

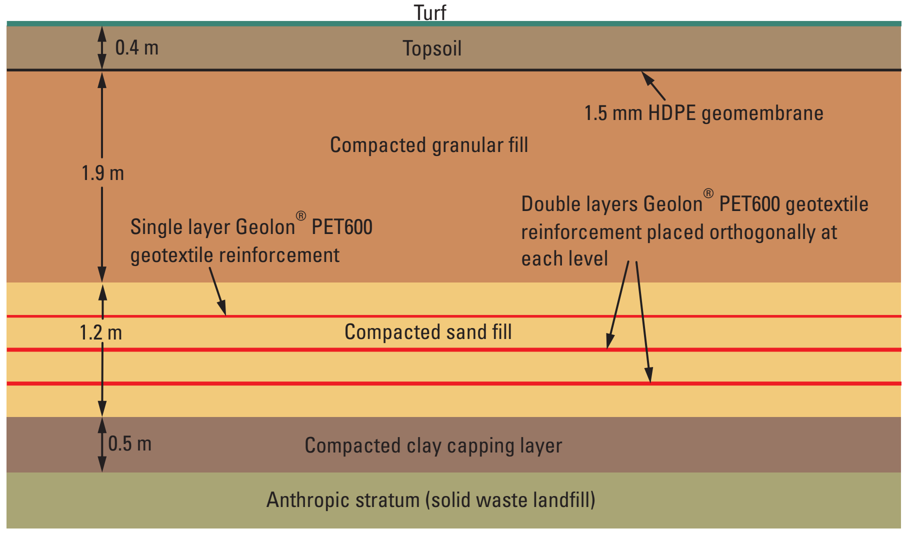
At the ground surface it was judged that the maximum allowable differential deformation be limited to 2% for any void forming in the anthropic stratum up to 4 m in diameter. This allowable differential deformation level is consistent with that allowed for road pavements, and was also considered appropriate for high quality football fields. To meet this surface differential deformation requirement three complimentary components were required for the foundation beneath the football field and surrounding areas. First, there had to be a minimum 4 m thickness of various well-compacted fills above the anthropic stratum. Second, there had to be a basal reinforced support system in the foundation. Third, 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 at 0.3 m vertical spacings within the compacted sand fill. The Geolon® PET600 has a tensile strength of 600 kN/m and is composed of high



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



Cross section details through the reinforced foundation

modulus, high strength polyester yarns giving excellent long term load carrying capability. In the bottom two levels, double layers of Geolon® PET600 geotextile reinforcement were 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.

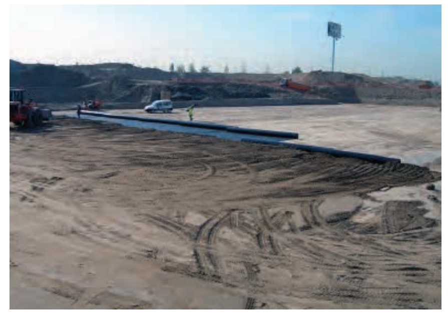
The stadium was completed in 2009 and is the most modern in all of Spain. It has since been awarded the top 4-stars rating by UEFA, the European Football Association.

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

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

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

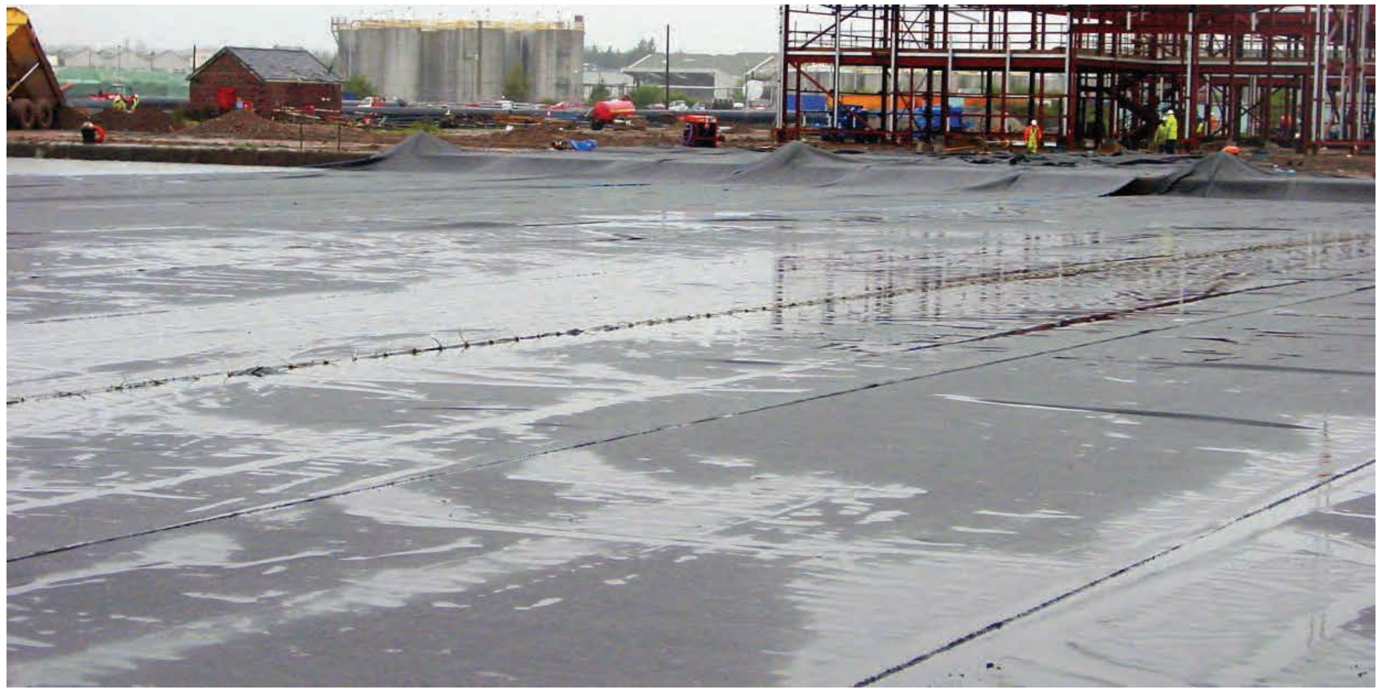
Placing and compacting granular fill over top of Geolon® PET600 geotextile reinforcement



Installation of HDPE geomembrane with cover soil layer

Completed football stadium

Reinforced soft site closures: Wallasey Dock roll-on roll-off terminal, Liverpool, UK



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.

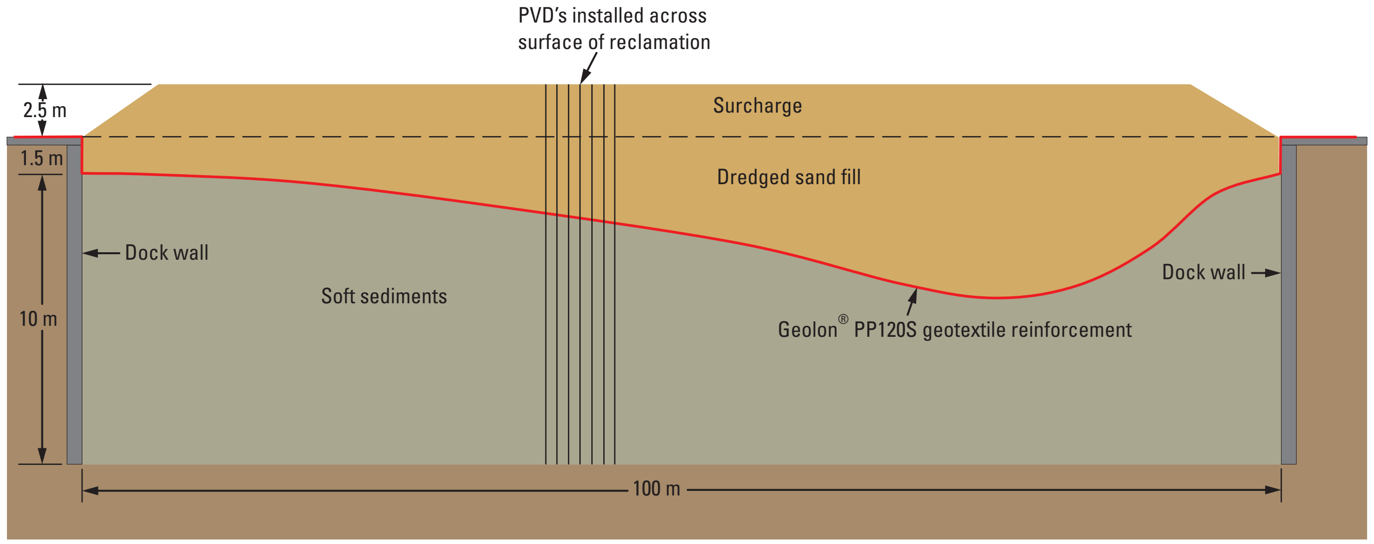
Geolon® PP120S was chosen as the geotextile reinforcement for two major reasons. First, the material has a tensile strength of 120 kN/m in both the length and cross directions, and low extension, and this would make the geotextile strong enough to resist any localised instability in the soft sediments that might occur. Second, the material was composed of 100% polypropylene and thus had a specific 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 hydraulically across the surface of the geotextile to ensure a stable base

Very soft sediments in dock basin at low water level



Typical cross section through the Wallasey Dock soft site closure

layer. Next, a second layer of sand was spread hydraulically to bring the fill level up to the height of the existing dock walls (around 1.5 m above the geotextile at the edges of the dock). Finally, the sand fill was raised to a level of 2.5 m above the surrounding dock level using a swamp dozer to spread the sand. This last layer of sand fill was to act as a surcharge to accelerate the consolidation of the soft sediments in 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.

Geolon® PP120S geotextile panel being pulled across flooded dock basin

Water level lowered in dock basin prior to placement of sand fill on top of Geolon® PP120S geotextile reinforcement

Geolon® PP120S geotextile fabricated into large panels onsite

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 traffick 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.

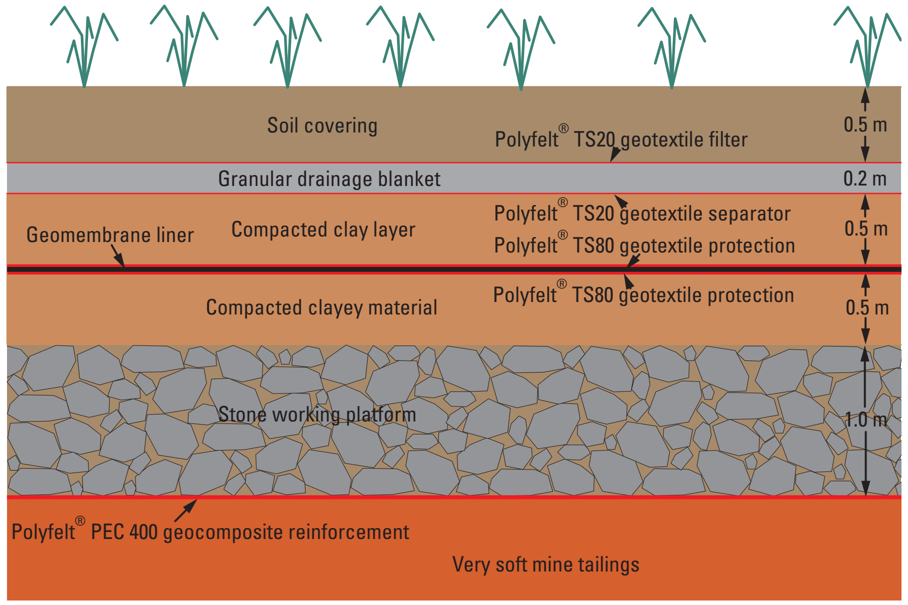
The Polyfelt® PEC400 geotcomposite reinforcement was placed starting at the edges of the tailings pond (the high points) and working inwards to the centre (the low point). Care was taken to ensure that any run-off water would not contaminate the new working platform.

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

Very soft, saturated tailings at certain locations in the tailings pond



Section through the mine tailings pond closure

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.

Type of stone fill used as the working platform on top of the Polyfelt® PEC400 geocomposite reinforcement

Partial closure of the mine tailings pond

Installation of the Polyfelt® TS20 geotextile separator prior to placement of the granular drainage blanket

Placement of the granular fill working platform over the Polyfelt® PEC400 geocomposite reinforcement

Reinforced soft site closures: Waste treatment sludge lagoons closure, Axis, Alabama, USA

In conjunction with the closure of a cellulose fibre 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 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.

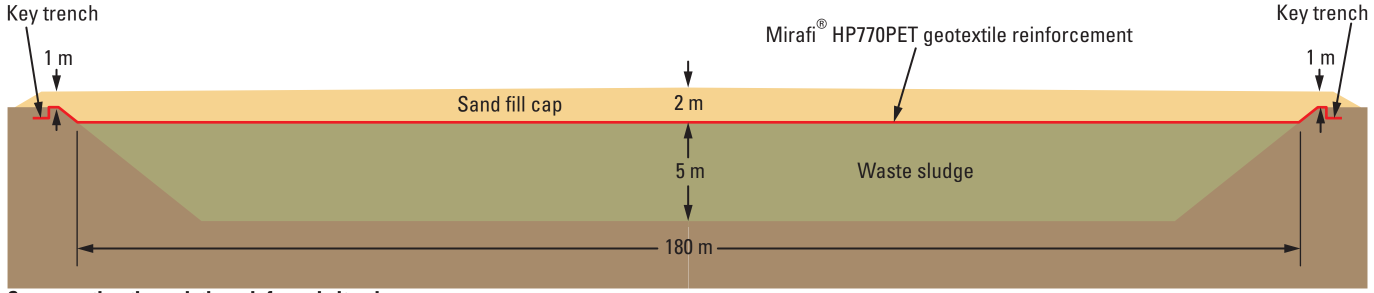
To ensure the leading edge of the geotextile reinforcement remained above water as it was pulled across the lagoon surface, polystyrene blocks were sewn into pockets in the leading edge of the geotextile. Polyester webbing was sewn to the geotextile to enable nylon ropes to be connected to the geotextile sheet at approximately 35 m intervals.

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

Condition of sludge lagoons prior to closure



Cross section through the reinforced site closure

spread an initial fill lift of 1 m across the geotextile reinforcement. Subsequent fill lifts of 0.5 m were spread across the closed lagoons. The total fill height varied from 1 m at the lagoon edges to 2 m at the lagoon centre. This was done to enable surface water run off.

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.

Pulling the sheet of Mirafi® HP770PET geotextile reinforcement across the sludge lagoon

Reaching the far side of the lagoon with the Mirafi® HP770PET geotextile sheet

Nearly completed sand cap prior to vegetation placement

Fabrication of Mirafi® HP770PET geotextile reinforcement into a single sheet prior to deployment

Reinforced soft site closures: Reinforced sludge capping layer, Harbin City, Heilongjiang Province, China

Wenchang Wastewater Treatment Plant in Harbin City generates sludge as a waste product of the wastewater treatment process. Over the years, the plant operator has been storing the waste sludge in an on-site sludge containment pond.

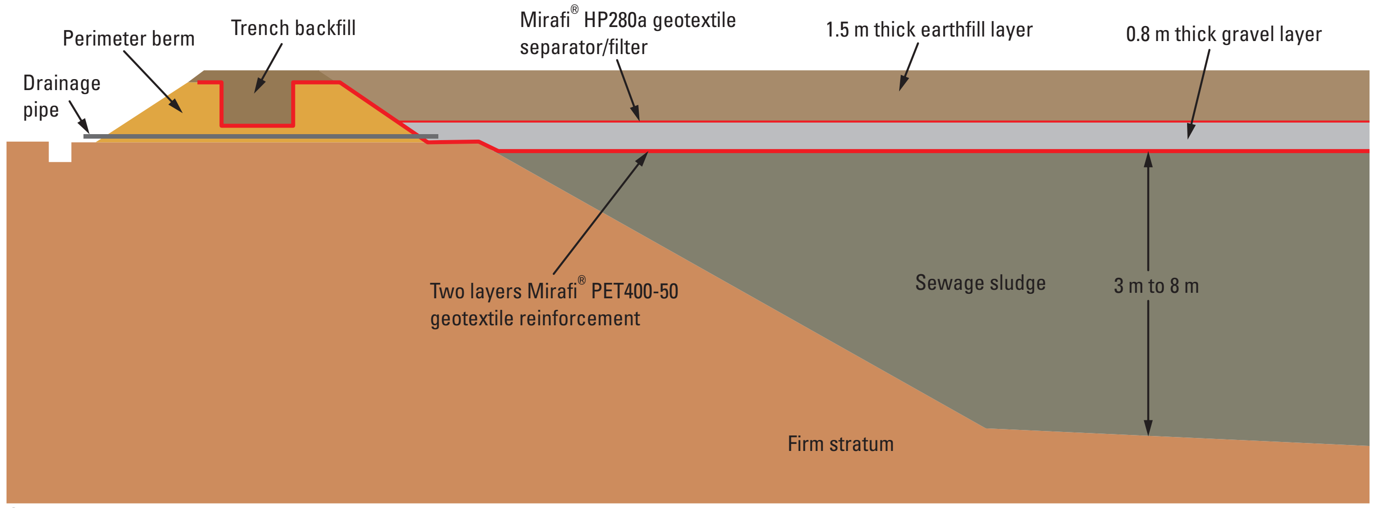
The sludge pond is located within the Wastewater Treatment Plant boundary. The sludge containment pond is trapezoidal shaped in cross section with a surface area of 70,000 m^2 and varies in depth from 3 to 8 m. The base of the sludge containment pond consists of firm ground. The wastewater treatment sludge consists of largely biosolids. The specific gravity of the solids ranges from 1.2 to 1.5, depending on which part of the wastewater treatment process it comes from. Therefore, the sludge in the containment pond is a mixture of very low density material of extremely low undrained shear strength. The sludge is generally thicker towards the bottom as the solids in suspension and colloids have settled out gradually over an extended period of time and have been subject to relatively higher overburden. However, even the bottom solids generally have only 25 to 30 percent solids concentration as a maximum. The undrained shear strength is estimated to be about 2 to 3 kPa at best.

One option investigated was to desludge the pond to extend its containment lifespan. However, this option was not deemed practical for the Wenchang Wastewater Treatment Plant as the city landfills would only accept dewatered wastewater sludge material that has attained a specific dryness. Also, over the years the land on and adjacent to the Plant premises had been fully developed, ruling out any dewatering of the sludge that would be necessary before the material could be sent to the landfills. Therefore, capping the sludge containment pond was the only practical option available. Safety and environmental requirements required the provision of an engineered soil capping layer over the sludge containment pond.

The soil capping layer consisted of a lower layer of gravel of thickness 0.8 m and an upper layer of earth backfill of thickness 1.5 m. A separation/filtration geotextile (Mirafi® HP280a) was used between the gravel and earth backfill layers. The gravel layer served as a drainage layer for the dissipating water from the sludge during the consolidation period. The earth backfill layer acts as both a surcharge and a load spreading layer. Because of the extremely soft nature of the sludge, the soil capping layer required geotextile basal reinforcement to provide both short term and long term reinforcement. The layer of reinforcement provided short term stability and prevention of loss of fill and prevented local shear and bearing capacity failures of the sludge.

Two layers of Mirafi® PET400-50 geotextile reinforcement, placed orthogonally over the sludge surface, was used for the basal reinforcement. Mirafi® PET400-50 is a uniaxial reinforcement with initial tensile strength of 400 kN/m in longitudinal direction and 50 kN/m in cross direction. By using a double layer, placed orthogonally, this would effectively result in a combined reinforcement layer of 400 kN/m in both directions.

Unrolling Mirafi® PET400-50 geotextile reinforcement across sludge pond



Cross section through reinforced capping of sludge pond

A raised earth berm 2 m in height was provided at the edge surrounding the sludge pond. An anchor trench was dug into the top of the earth berm to anchor the extremities of the Mirafi® PET400-50 geotextile reinforcement.

The construction of a capping layer over very soft material is always a challenge. This was particularly so for the wastewater treatment sludge pond because of the extremely low strength of the waste sludge. Movement directly on top of the sludge surface was impossible, both for construction workers and for earthworks equipment. The geotextile reinforcement had to be laid as a continuous panel in its longitudinal direction with the panel edges seamed together into a complete cover fabric. However, to prefabricate and lay the complete cover fabric can be very difficult by normal means; the prefabricated complete cover fabric would be too massive to handle, and besides there was no workspace at site to do on-site fabrication.

The contractor came up with a novel method of construction. This involved the fabrication of the cover geotextile reinforcement as the rolls of geotextile was being laid over the top of the sludge pond. To support workmen and sewing equipment over the sludge pond floating boards were used made from a combination of polystyrene and timber. These floating boards could be pulled across the sludge surface like snow sledge boards from the opposite bank.

Polystyrene board working platform beneath the Mirafi® PET400-50 geotextile reinforcement

Once the double layer of Mirafi® PET400-50 geotextile reinforcement had been installed and anchored in the surrounding anchor trench the granular drainage material was pushed out over the geotextile reinforcement by first constructing "fingers" of fill at predefined distances apart. These "fingers" were used to prestress the geotextile reinforcement preventing large differential deformations (mud waves) during filling. Once the fingers had been constructed subsequent infilling was carried out using light dumpers.

Once the granular drainage layer had been installed the separation/filtration geotextile was laid out over the site with the earth fill placed on top.

Client: Longjiang Environmental Protection Group Co., Ltd, Heilongjiang Province, China.

Consultant: China Nerin Engineering Technology Co., Ltd, Jiangxi Province, China.

Contractor: Shanghai Tongji Construction Technology Co., Ltd, Shanghai, China.

Anchor trench around edge of sludge pond

Creating a gravel "finger" across top of Mirafi® PET400-50 geotextile reinforcement

Using light dumpers to spread aggregate between fill fingers

Bringing fill onto edge of platform

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.

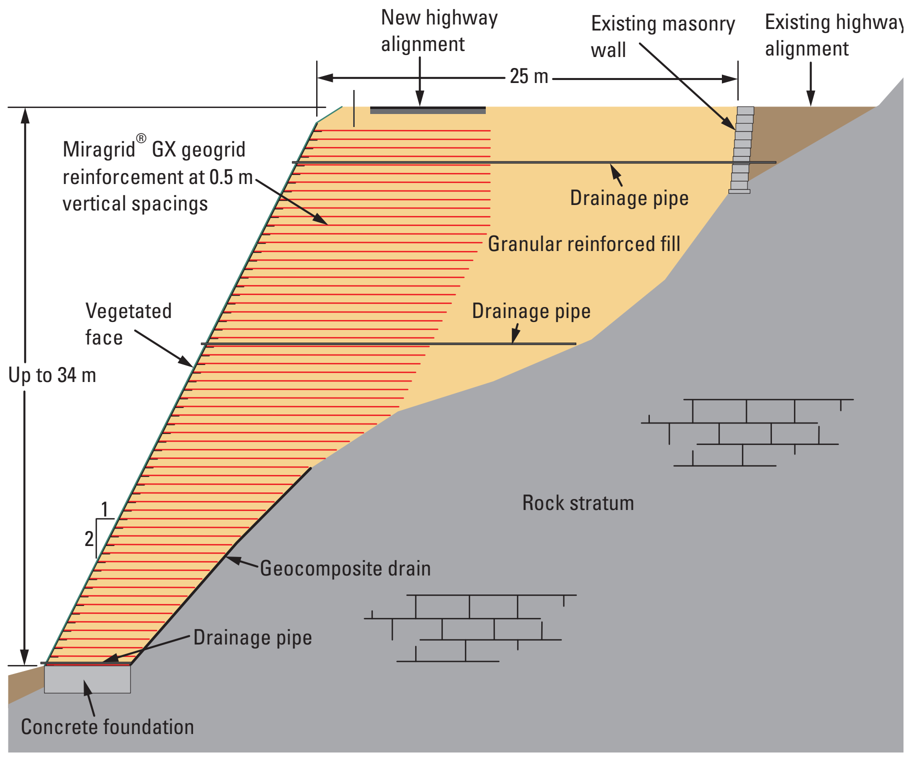
A design analysis of the steep reinforced slope was performed using a limit equilibrium approach taking due account of National design standards. Because of the limited base width available, good quality granular material had to be used throughout as the reinforced fill and this was compacted to good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

Existing highway and proposed new highway alignment



Typical cross section through the reinforced slope at maximum height

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.

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

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

Reinforced fill slope near completion

Reinforced fill slope 2 years after construction

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 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 26.5 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.

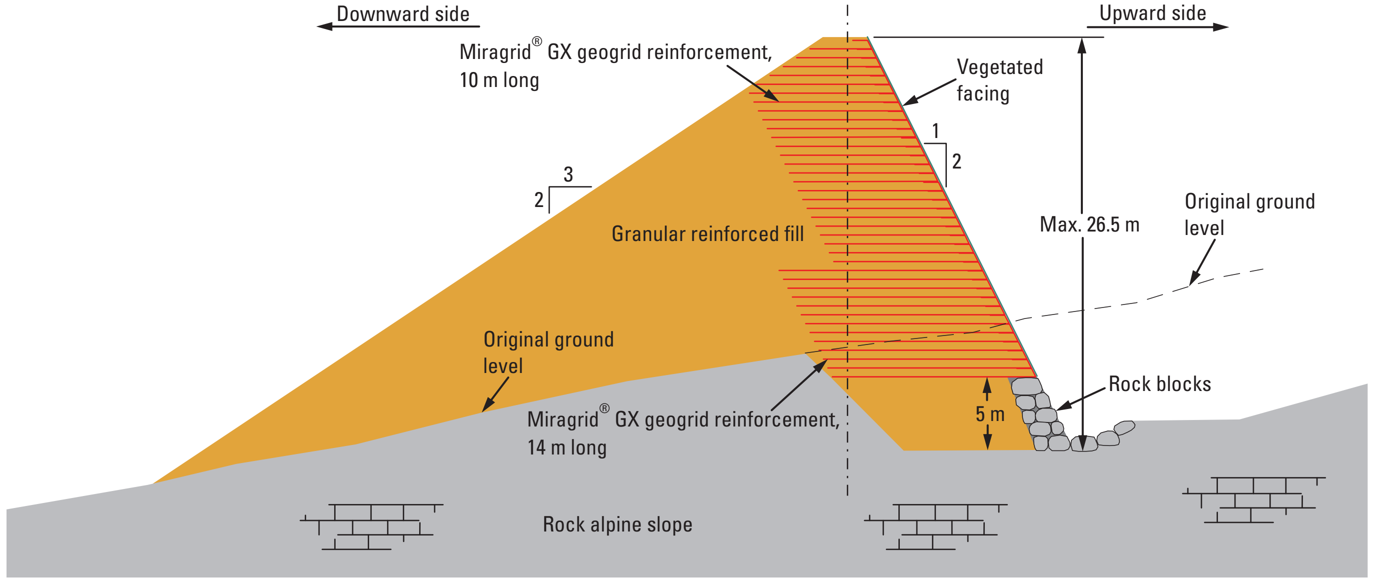
The reinforced soil system used on the upward side of the avalanche protection barrier above the rock block platform consisted of 0.5 m high steel mesh facing units angled at 2V:1H with layers of Miragrid® GX geogrid reinforcement at 0.5 m vertical spacings. The 0.5 m high steel mesh facings coincided 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.

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 such time as 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 vegetation growth, followed by the placement and compaction of the granular reinforced fill. The granular reinforced fill was

Rock block facing beneath the reinforced slope

Miragrid® GX geogrid reinforcement being placed in the reinforced slope



Cross section through the avalanche protection barrier

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.

Special plant specie 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

Beginning of the reinforced slope in the avalanche protection barrier

Reinforced fill slopes: Reservoir tsunami protection barrier, Axamer Lizum, Tyrol, Austria

The water storage reservoir 'Dohlennest' was built in the ski resort of Axamer Lizum, close to the Tyrolian capital of Innsbruck. Such reservoirs are located close to ski slopes to store water for artificial snow production prior to the starting of the ski season. The storage reservoir and the pumping station are situated in the Malgruben mountain, a location surrounded by steep slopes and rugged peaks of the Kalkkögel. To secure the storage reservoir against snow avalanche impacts, two avalanche protection barriers were constructed to deflect and/or prevent avalanches from entering the reservoir and creating a tsunami wave.

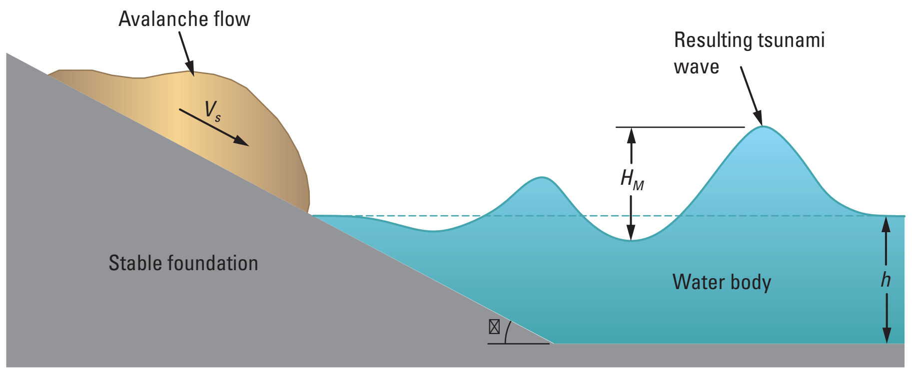
Various areas of avalanche danger were located, and scenarios were investigated and simulated to determine degree of risk. It was concluded that the most critical form of avalanche in the area was the 'snow slab' avalanche. This is where an extended weak layer is located within the snowpack and when this becomes unstable an avalanche can occur.

The slope area Southwest of the storage reservoir is characterised by steep slopes, rocky peaks and steep scree slopes with debris flow channels. The immediate proximity of the reservoir dam crest to the debris cones is the main cause for the prevailing avalanche hazard. The avalanche starting areas are in the upper, steepest part of these debris cones and reach up to the hollows between the rocky peak areas. The fall course is steep and flows directly into the storage reservoir. The fall elevation from the highest point of the cracks to the crown of the reservoir varies between 130 m to 170 m.

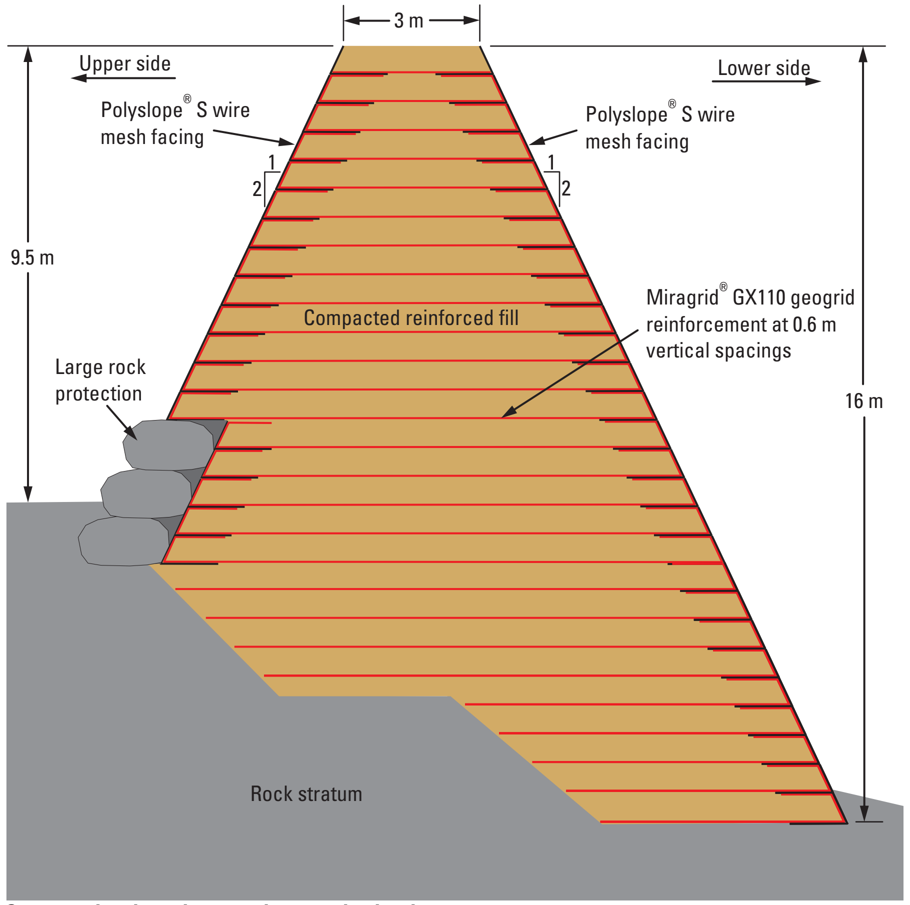
Even without avalanche simulations, it was evident that avalanches can damage the reservoir. Simulations of the design avalanche (using a 150-year return period) show that avalanches from the break-off areas can penetrate the reservoir. For this design return period, the avalanche flow velocity in the area of the reservoir crest is 22 m/s.

Avalanche flows into the storage reservoir cause a resulting tsunami wave to be generated which can spill over the reservoir crest if the freeboard is insufficient. The magnitude of the maximum generated tsunami wave height H_M is related to the depth of the water body h and its shape, and the momentum of the avalanche flow (being its mass and downslope velocity V_s). The resulting water spill causes flooding of the area below the storage reservoir and in the water courses of the streams below.

For the design 150-year return period, avalanches enter the reservoir unhindered. A consequent analysis yielded a maximum generated tsunami wave height of 3.1 m which would have resulted in significant water spill over the reservoir crest with subsequent below-crest flooding and damage.



Mechanics of tsunami generation due to avalanche flows



Cross section through reservoir protection barrier

To protect the reservoir from the effects of avalanche flows it was decided to construct two protection barriers, one along the Southwest side of the reservoir and the other along the Southern side. These protection barriers were constructed up to a height of 9.5 m on their upper sides and consisted of a Polyslope® S double-sided reinforced slope system which used Miragrid® GX110 geogrid reinforcement at 0.6 m vertical spacing in the reinforced slope. Miragrid® GX geogrid reinforcement is ideal for long term reinforced soil applications because it has well-defined strength and strain properties over long time periods. The reinforced fill used was local granular material. On the upper sides of the protection barriers large rock was used at the base of the slopes to protect from erosion and other damage.

The avalanche protection barriers were designed to primarily deflect avalanche flows away from the reservoir water surface. The protection barriers can prevent design 150-year return period avalanches from entering the reservoir. This has resulted in the greatest possible safety being achieved.

Reinforced protection barrier during construction

Polyslope® S facing on protection barrier

Completed protection barrier with vegetated surface

Client: Axamer Lizum Aufschliessungs AG, Tyrol, Austria.

Consultant: Ingenieurbüro Illmer Daniel e.U. and Geoechnik Henzinger, Tyrol, Austria.

Contractor: ALPENBAU GmbH, Terenten, Austria.

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.

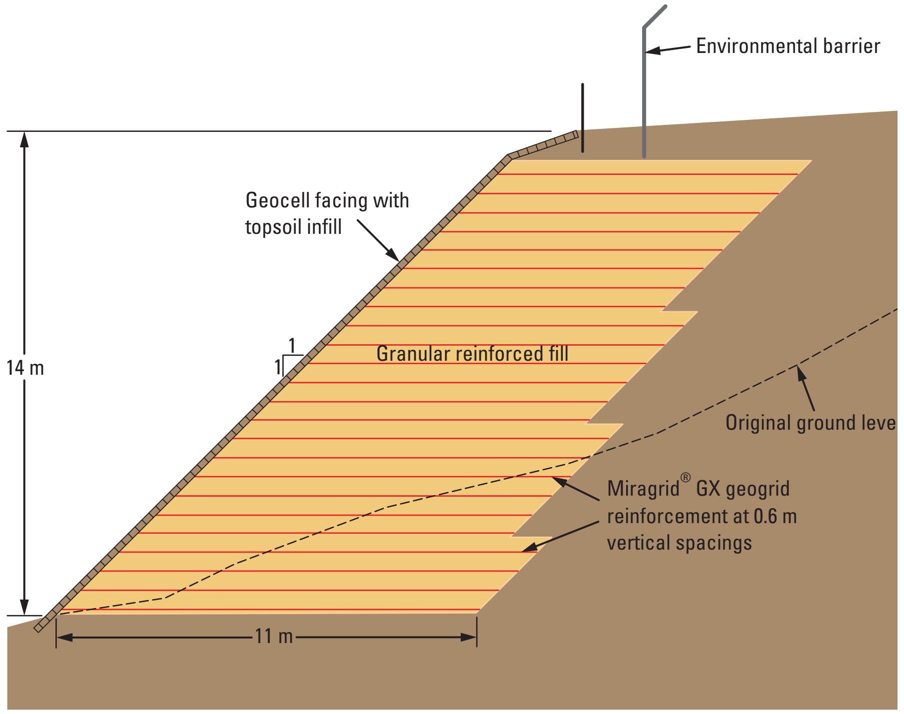
To improve traffic dispersal through the area of Hindhead a major road works scheme was developed that accounted fully for the local environmental requirements as well as the provision of a dual two-lane carriageway throughout. The highway scheme is 6.8 km in length with a 1.8 km tunnel under a specific sensitive environmental area, and involves the movement and placement of over 1 million m^3 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

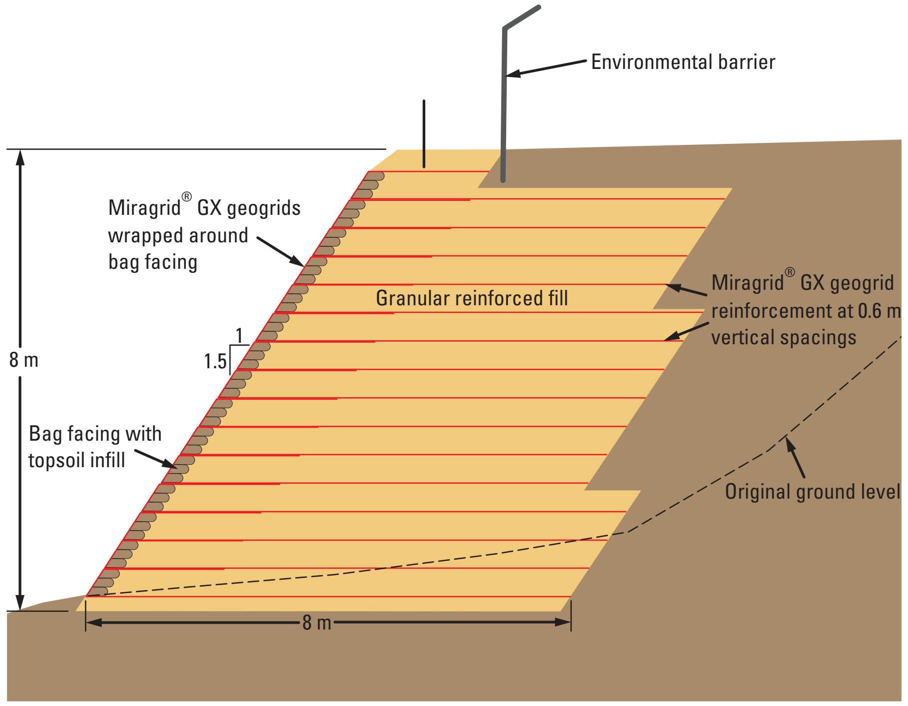
At specific locations along the length of the highway earthworks embankments shallow reinforced fill side slopes were constructed at slope angles of 1V:1.5H or 1V:1H depending on the slope requirements. These fill slopes were constructed up to 15 m in height depending on the earthworks geometry.

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.



Typical cross section through the shallow reinforced slopes



Typical cross section through the steep reinforced slopes

These were 35 kN/m (near the top of the slope), 55 kN/m, 80 kN/m and 110 kN/m strengths.

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.

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 in the same manner as for the shallow reinforced fill slopes to provide a firm, stable foundation platform.

The steep reinforced fill slopes were constructed using compacted granular fill with Miragrid® GX geogrid reinforcement placed at 0.6 m vertical spacings. The tensile strengths of the Miragrid® GX geogrid grades used varied from 35 kN/m to 110 kN/m depending on the location in the slope and the slope height.

The facing of the steep reinforced 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.

Use of timber shutter system to align the face for the steep reinforced slope sections

One of the steep reinforced slopes nearing completion

Grading the face of the shallow reinforced slopes

Placing geocell facing with topsoil infill for one of the shallow reinforced slopes

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.

Client: Highways Agency, UK.

Consultants: Atkins Consultants and Mott MacDonald and Partners, UK.

Contractors: Balfour Beatty Civil Engineering Ltd and J. McArdle Contracts Ltd, UK.

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.

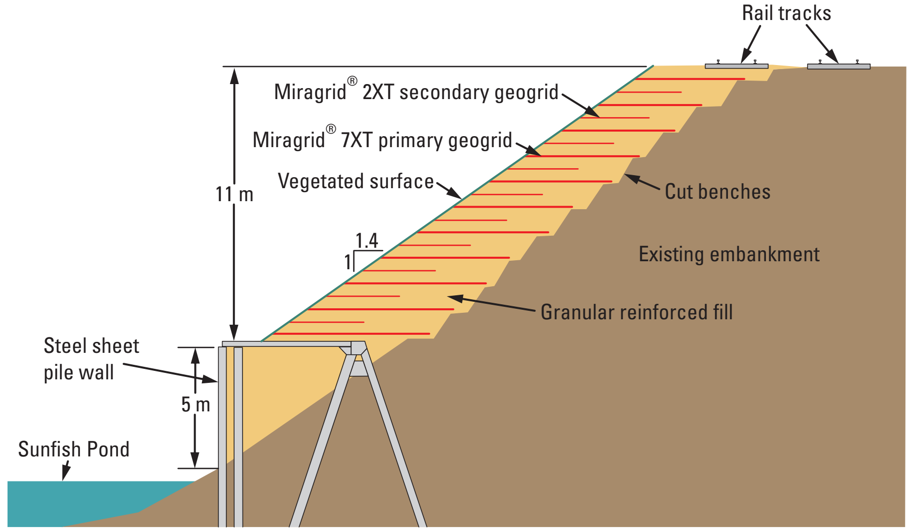
Sunfish Pond is part of an environmentally sensitive watershed in the area that is managed by the local botanical gardens authority. Consequently, any widening of the existing rail embankment was not allowed to impinge on the pond. Thus, construction of a conventional, widened, 1V:2H embankment slope 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® 7XT 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 intermediately 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

Clearing of vegetation from the existing embankment slope



Typical cross section through the reinforced fill slope

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 fill slope partially completed

Installed sheet pile wall and start of the reinforced fill slope construction

Placing reinforced fill over Miragrid® XT geogrid reinforcement

Geomat being placed on the slope surface to promote vegetation growth

Completed reinforced fill slope with vegetated surface

Reinforced fill slopes: Xinfeng MSE landfill expansion, Guangdong Province, China

Guangzhou is the third largest city in China, behind Beijing and Shanghai. With more than 10 million residents, Guangzhou is one of China's leading manufacturing and commercial centres and generates significant amounts of municipal solid waste (MSW). For many years Guangzhou's solid waste has mostly been sent to the Xinfeng MSW Landfill for disposal. Xinfeng Landfill is located around 40 km to the Northeast of Guangzhou City centre. It currently has a landfill capacity of more than 26 million m³ and is the second largest landfill in China in terms of volume and daily MSW intake. The Xinfeng Landfill was originally developed in 2003 with a design capacity of 19 million m³ based on 3,000 tonnes of MSW daily intake and had a projected collection life of 20 years. However, by 2010 Guangzhou was producing 8,000 tonnes of MSW daily, out of which 7,000 tonnes were sent to Xinfeng Landfill. Today, Xinfeng Landfill still handles on average 75% of the daily MSW generated in Guangzhou City.

By 2012 it was clear that Xinfeng Landfill was quickly running out of capacity. There was an urgent need to find a way to increase the capacity of the landfill, without which it would have to stop accepting waste. This would have had drastic consequences for Guangzhou City as other alternative new landfill and incinerator projects were still at the initial planning stage.

The decision was made to expand the capacity of the Xinfeng Landfill by constructing a 35 m high containment bund of over 300 m in length over the existing containment bund at the Southern end of the landfill. This would expand the MSW capacity by a further 7.5 million m³.

Due to site constraints and in order to gain maximum landfill capacity increase, the sides of the new containment bund were designed to a slope angle of about 1:1.1. The Mechanically Stabilized Earth (MSE) technique utilizing geotextile reinforcement layers in the compacted reinforced fill was used to construct the containment bund to this required slope angle.

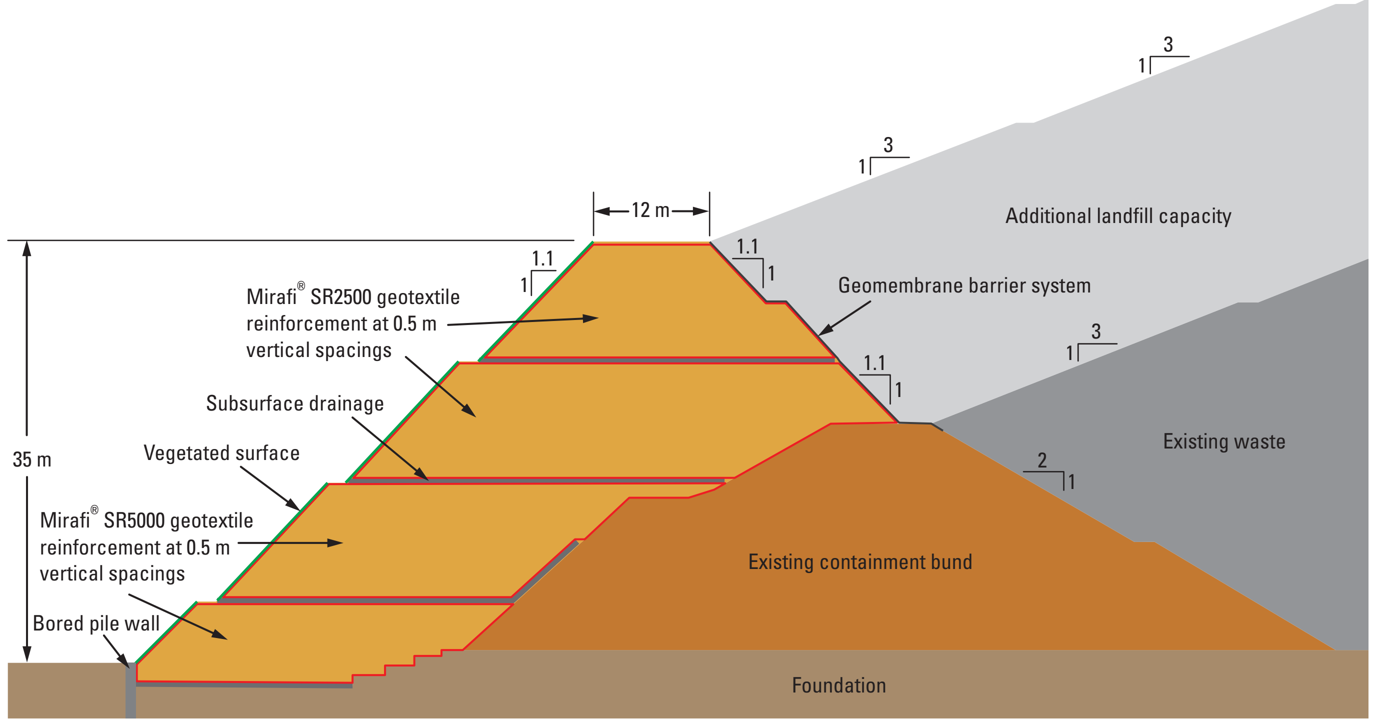
A row of 1.6 m diameter bored piles was designed to act as a shear key at the toe of the new containment bund to provide adequate stability against global failure. The bored piles typically ranged from 18 to 21 m in length.

The new containment bund has a maximum height of 35 m (from ground level), with an additional 2 m embedded below ground level. There are 4 MSE tiers in the containment bund, each set back 2 m from the tier below. The top of the containment bund is 12 m wide at the crest.

Mirafi® SR PET geotextile reinforcement was used throughout the reinforced bund because of its good long term performance in reinforced soil structures. Mirafi® SR5000 with an initial tensile strength of 500 kN/m was used at 0.5 m vertical spacings in the bottom two tiers of the bund, while Mirafi® SR2500 with initial tensile strength of 250 kN/m was used at 0.5 m vertical spacings in the upper two tiers. The reinforcement layers were continued for the full width of the bund. At both sides of the bund the geotextile reinforcement was wrapped around soil bags and extended back into the bund fill by 2 m.

At the base of each of the tiers a subsurface drainage blanket was

Placing layers of Miragrid® SR geotextile reinforcement at the lower levels of the MSE bund



Cross section through the Xinfeng MSE landfill expansion bund

constructed to enable easy exit of any seepage water. This consisted of a geotextile filter encapsulating granular drainage material.

Once the reinforced fill and geotextile reinforcement placement had been completed a geomembrane barrier system was installed on the inner side of the MSE bund. This barrier system connected into the existing side slope barrier system against the existing bund and continued up to the top of the new MSE bund.

On the outer slope of the MSE bund a topsoil layer was placed over the Mirafi® SR geotextile reinforcement wrapped facing and this was held in place by an erosion control mat that was fixed to the reinforced slope by means of steel pins anchored into the reinforced fill. This topsoil layer was then vegetated.

Client: Grantop Group Co., Ltd, Guangdong Province, China.

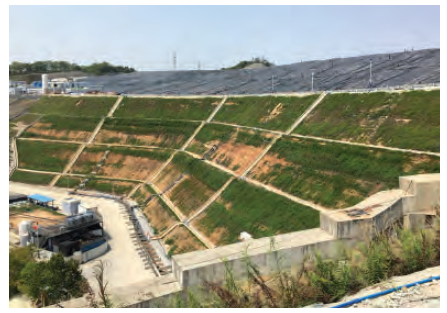
Consultant: The Architectural Design and Research Institute of Guangdong Province, China.

Contractor: Guangdong Foundation Engineering Group Co., Ltd, Guangdong Province, China.

Miragrid® SR geotextile reinforcement wrapped around soil bag facing units

Geomembrane barrier system being installed on the inner side of the MSE bund

Top soil covering placed on outer slope of the MSE bund prior to surface vegetating



Completed MSE expansion bund with vegetated surface

Reinforced fill slopes: Landslide restoration, Langkawi, Kedah, Malaysia

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

Gunung Raya, at 881 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.

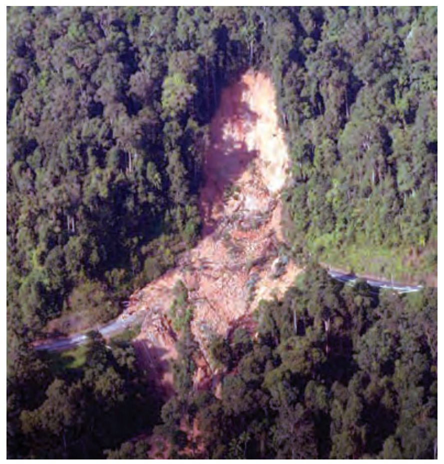
In 2003 after heavy rain a major landslide occurred around the mid-portion of Jalan Gunung Raya on the upper side of the road. The landslide exposed the granite outcrop and the debris completely cut off the road access. After reviewing several remedial options the decision was made to realign the road 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 1.2V: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.

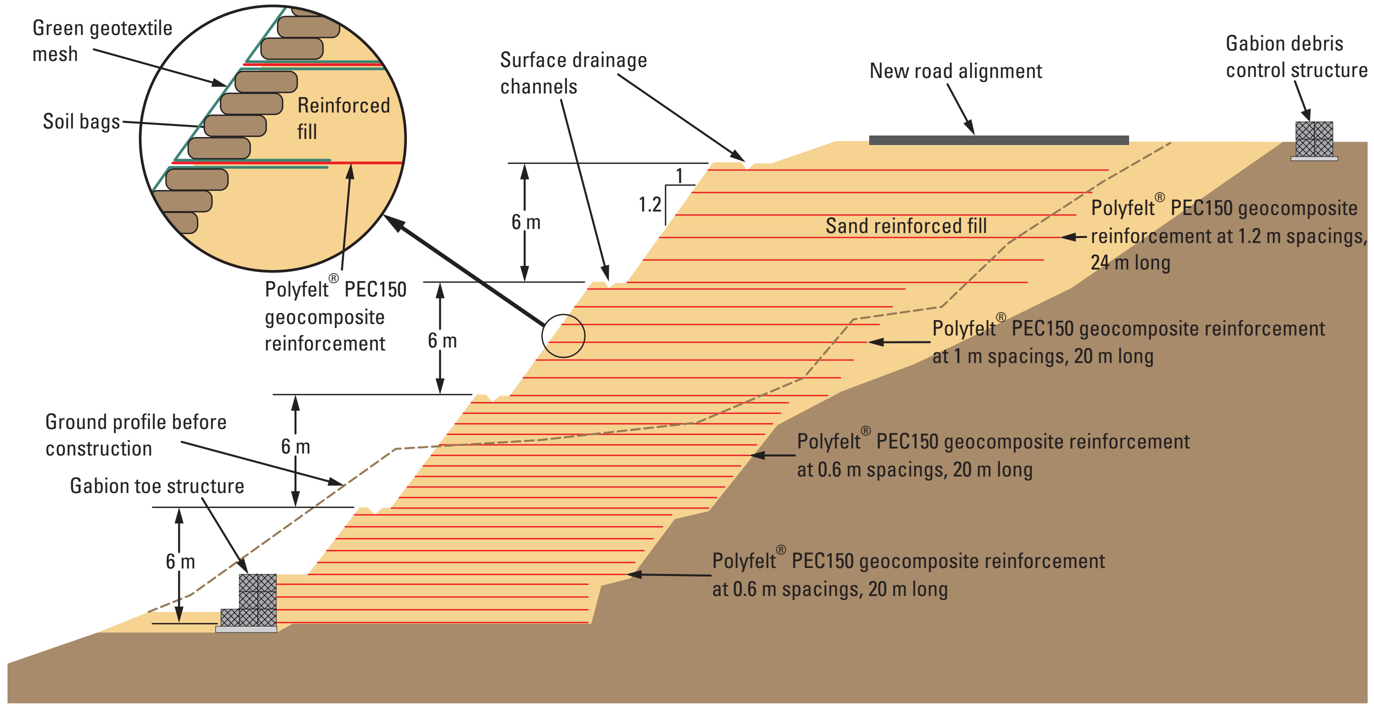
The landslide debris was removed from the site to clear the road and provide an adequate zone for the new reinforced slope. At the rear of the excavated zone a subsurface drainage layer was provided behind the reinforced soil zone to intercept ground water seepage from the natural strata. A Polyfelt® 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 fulfill 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



Landslide blocking Jalan Gunung Raya



Typical cross section through the reinforced slope restoration

root matter of the slope vegetation and prevents surface erosion.

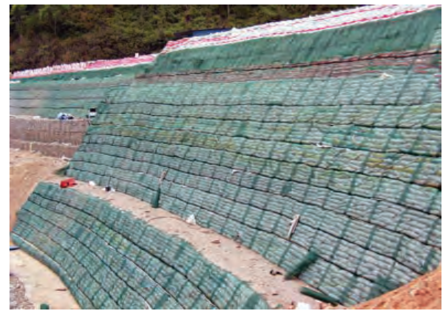
Once construction of the reinforced slope had been completed, hydroseeding was carried out to establish surface vegetation. It only took several weeks for the surface vegetation to become fully established.

Client: Pubic Works Department, Kuala Lumpur, Malaysia.

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

Contractor: Protab Construction Sdn Bhd, Langkawi, Malaysia.

Installing the gabion toe of the reinforced slope



Reinforced slope partially completed

Compacting the reinforced fill

Completed reinforced slope with vegetation being established

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.

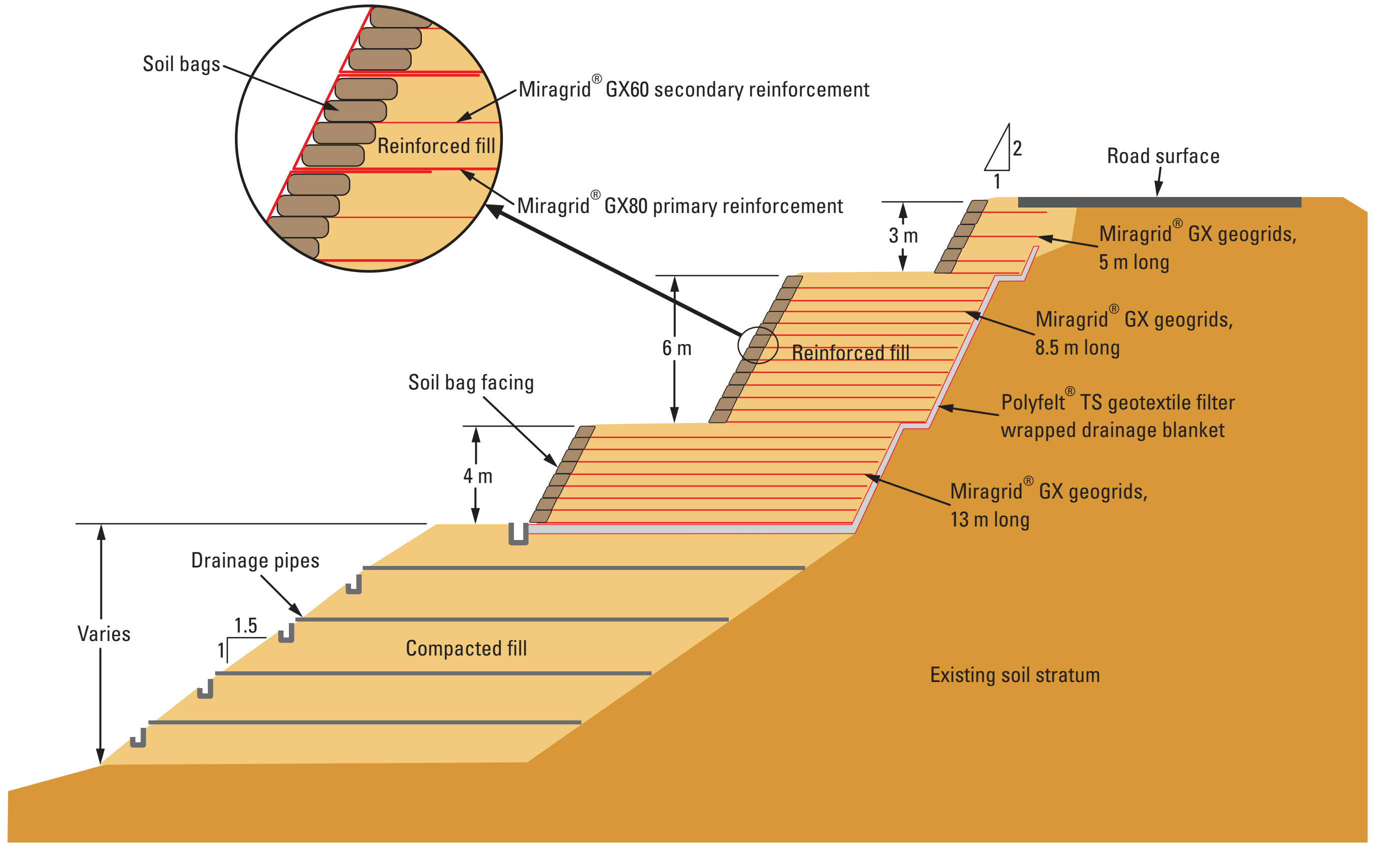
The most important constraint for the design of the slope restoration was that the single lane of the road remaining after the slope failure had to be kept functional as this road was the only access to many villages in the area. This also meant that any design involving excavation that could further 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 1V:1.5H, 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® GX60 geogrid reinforcement is used as secondary reinforcement. The upper reinforced fill tier is 3 m high and reinforced with 3 layers of Miragrid® GX80 geogrid reinforcement of 5 m reinforcement length.

Initial excavation at the toe of the failed slope



Typical cross section through the reinforced slope

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: Phatthananuphap Construction Co., Chiangmai, Thailand.

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

Spreading locally obtained reinforced fill over Miragrid® GX geogrid reinforcement

Soil bags with Miragrid® GX geogrid wrap-around facing

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 beyond. Prior to construction, various geotechnical options were evaluated. It was deemed impractical to widen the road by cutting back into the upslope. The only practical way was to widen the carriageway in the direction of the downslope. The geogrid 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.

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.

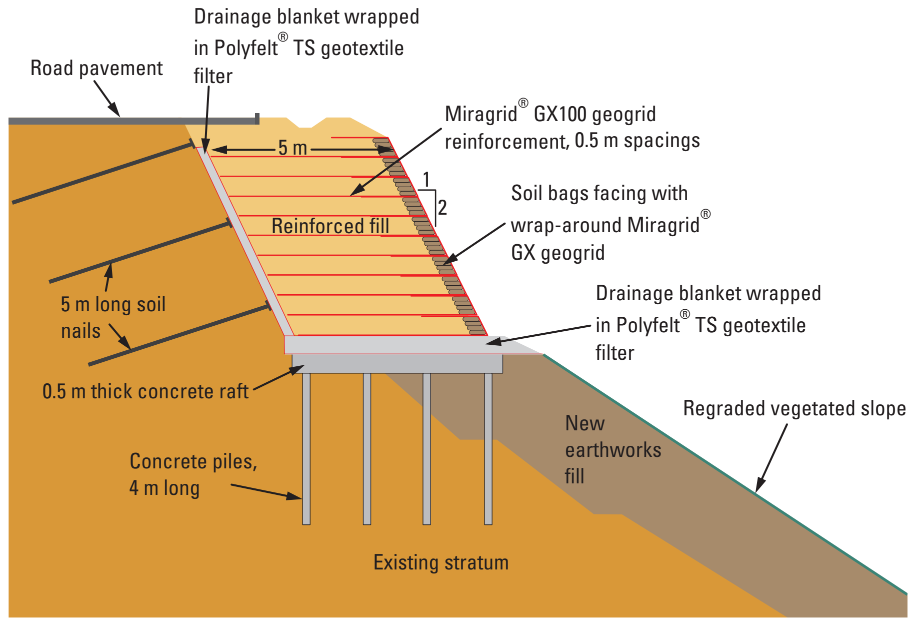
Above the concrete raft a drainage blanket was installed consisting of single-size stone wrapped with a Polyfelt® TS geotextile filter. This drainage blanket was continued up the rear of the reinforced slope to intercept groundwater flows emanating from the existing slope.

The reinforced fill slope was constructed to a height of 5 m at a slope angle of 2V:1H. Miragrid® GX100 geogrid reinforcement was used at 0.5 m vertical spacings. Miragrid® GX100 geogrid reinforcement is composed of high modulus polyester yarns within a robust polymer coating and has an initial tensile strength of 100 kN/m.

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 local residual soil was used as the reinforced fill and was compacted in 0.25 m thick lifts. Once compacted,

Soil nailing to maintain stability of existing road foundation



Cross section through the reinforced slope

this soil exhibits good reinforced fill properties.

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.

Piled concrete raft being constructed for the base of the reinforced slope

Placement of stone drainage blanket with Polyfelt® TS geotextile filter on top

Placing reinforced fill in the reinforced slope

Reinforced slope nearing completion

Reinforced fill slopes: Gabion faced reinforced slope, Kandy, Sri Lanka

Kandy is registered as a UNESCO World Heritage Site and is the second largest city in Sri Lanka after the capital Colombo. Kandy is both an administrative and religious city housing the Temple of Tooth which is one of the most sacred places of worship in the world. The city lies on the Kandy plateau which is mountainous and thickly forested.

With the population growth of Kandy it was necessary to construct a modern wastewater treatment plant in order for clean effluent water to be discharged into the city's waterways, and to prevent the existing practice of discharging untreated wastewater directly into these waterways. The wastewater treatment plant was located in a hilly area on the outskirts of Kandy. Thus, to provide the necessary flat platform areas reinforced slopes were utilised on site.

A reinforced slope of two tiers, consisting of gabion facing units with layers of geocomposite reinforcement, was used to enable the required level platform areas for the wastewater treatment plant and sludge drying beds. The reinforced slope was designed in accordance with BS8006:2010, a British Standard Code of Practice for Reinforced Soil. Rock filled gabion facing units, measuring 1 m wide by 1 m high by 1 m deep, were chosen for their hard, durable facing and aesthetic appearance. The slope facing angle was created by stepping each gabion layer half-way back at each level thus creating a slope angle of 65° (2:1). The main slope reinforcement was Polyfelt® PEC geocomposite reinforcement placed at vertical spacings of 0.5 m. Polyfelt® PEC geocomposite reinforcement has well-defined long term engineering properties which makes it ideal for reinforced soil applications.

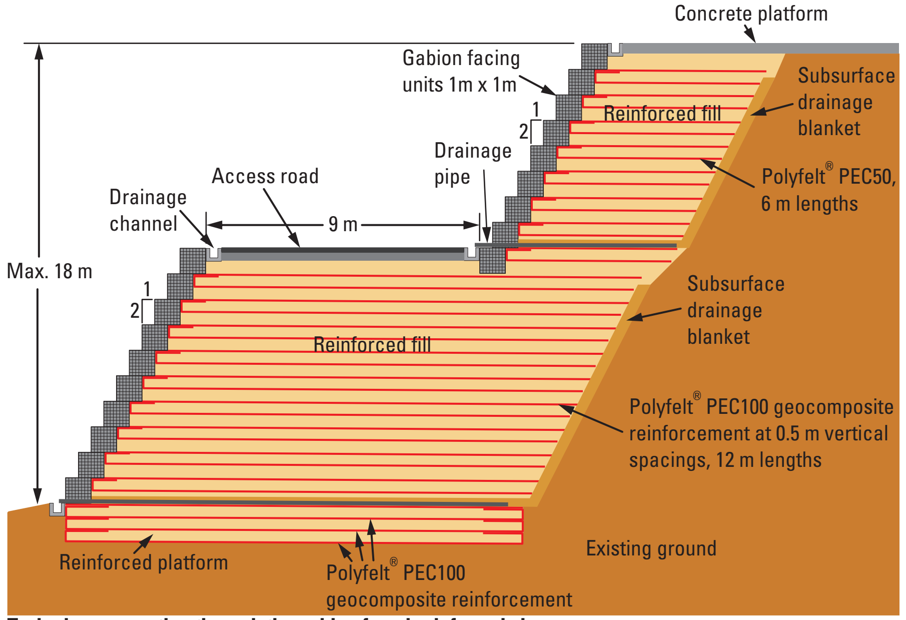
Site preparation works involved excavation of the hill slope to form the foundation platform for the reinforced soil structure. At the base of the slope below the first reinforced slope tier a reinforced platform, 1.5 m deep, was constructed using reinforced fill and three layers of Polyfelt® PEC100 geocomposite reinforcement. This was to provide a stable base for the construction of the two-tier reinforced slope.

At the base of each tier a subsurface drainage blanket was constructed consisting of granular drainage material wrapped in a Polyfelt® TS nonwoven geotextile filter. This drainage blanket was continued up the rear of the two reinforced soil zones to intercept any seepage water coming from the rear of the slope. A drainage pipe was installed to enable groundwater flows from the subsurface drainage blankets to pass into external drainage channels.

The reinforced fill used for the reinforced slope was a good quality residual soil. This was laid in lifts and compacted to 95% Standard Proctor density. The compacted reinforced fill lifts were 0.25 m in thickness.

The reinforcements used in the reinforced slope were Polyfelt® PEC100 in the bottom tier and Polyfelt® PEC50 in the upper tier. The reinforcements were installed at 0.5 m vertical spacings. Where a layer of Polyfelt® PEC coincided with the top and base of a gabion facing unit, it was continued between the gabion units thus creating a frictional connection between the reinforcement and gabion units. Where a layer of Polyfelt® PEC coincided half-way up a gabion facing unit, it

Site preparation works



Typical cross section through the gabion faced reinforced slope

was wrapped around the rear of the gabion unit and brought back into the reinforced fill. After laying the Polyfelt® PEC geocomposite reinforcement it is pulled taunt and reinforced fill placed on top and compacted.

A Polyfelt® TS nonwoven geotextile filter was placed behind the gabion facing units to intercept any seepage water emanating from the reinforced fill zone.

Following completion of the reinforced slope, a road pavement was constructed on top of the set-back platform to enable access to the wastewater treatment and drying beds area.

Client: National Water Supply and Drainage Board (NWS & DB), Colombo, Sri Lanka.

Consultant: Nippon Koei UK Co. Ltd, Tokyo, Japan.

Contractor: JFE Engineering Corporation, Tokyo, Japan.

BS8006:2010 Code of practice for strengthened/reinforced soils and other fills, British Standards Institution.

Laying Polyfelt® TS geotextile filter for the drainage blankets at the base of the reinforced slope tiers

Placing reinforced fill on top of Polyfelt® PEC geocomposite reinforcement

Compaction of reinforced fill

Gabion faced reinforced slope partially completed

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.

The concrete facing portion of the wall has tiers of headers that extend into the geosynthetic reinforced fill and stretchers that extend between the headers to form the front face of the wall. These stretchers, with the help of a geotextile filter bridging gaps between the stretchers, hold in loose topsoil so that vegetation can grow easily at the wall face.

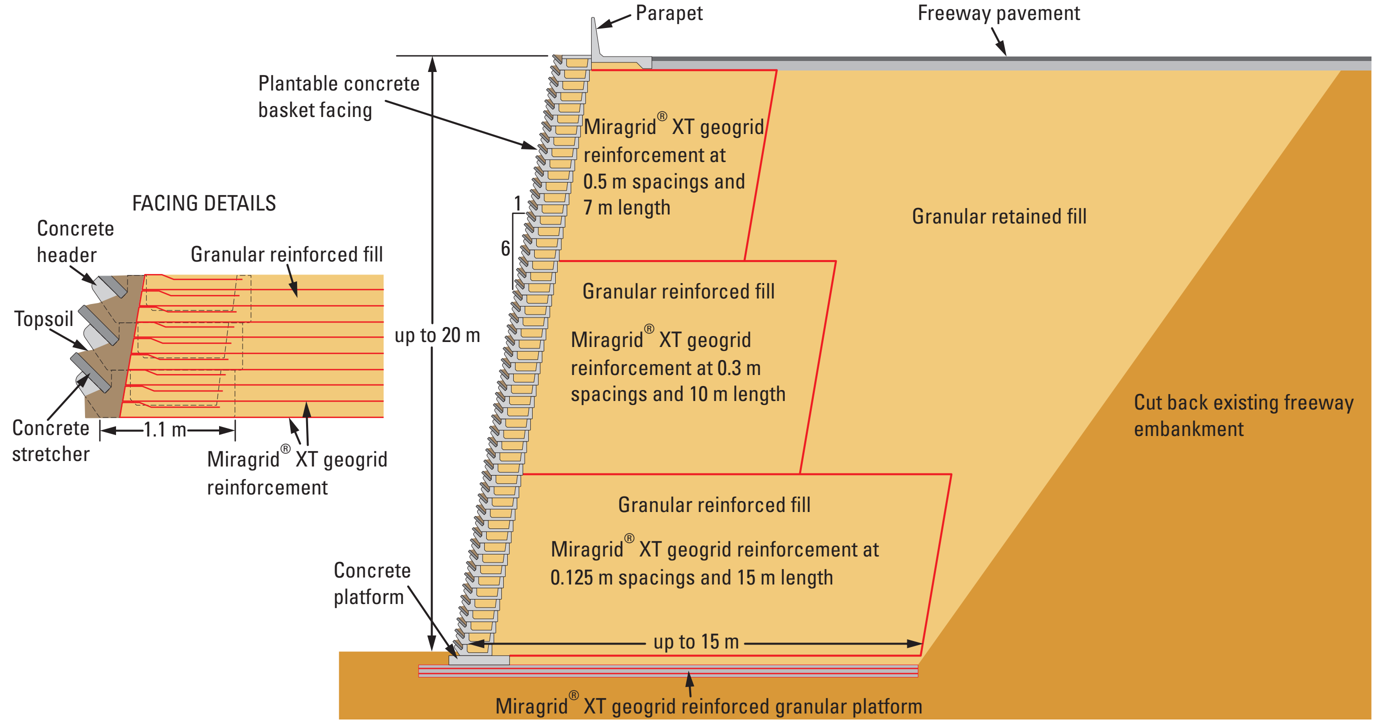
The soil forces generated behind the concrete tiers are sustained by layers of geogrid reinforcement that extend up behind the stretchers and then into the reinforced fill. The end result is a near vertical, maximum 20 m high retaining wall that will be completely vegetated.

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.

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® 10XT 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 specified very stringent requirements for the geogrid wrapped face, requiring high compaction of the reinforced fill, even adjacent to the wrapped face, to limit any differential settlements at the wall face. Further, the geogrid reinforcement had to be cut to fit around each concrete header. The contractor

Construction of Miragrid® XT geogrid reinforced granular platform



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 taunt and 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 m^2 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 m^2.

Client: California Department of Transportation, California, USA.

Consultant: California Department of Transportation, California, USA.

Contractor: E.L. Yeager Costruction Inc., California, USA.

Placing Miragrid® XT geogrid reinforcement in the granular reinforced fill

Plantable concrete wall facing system

Reinforced soil walls: MSE stress relief wall, US Bank Stadium, Minneapolis, Minnesota, USA

US Bank Stadium is a new enclosed, multi-functional stadium complex for the City of Minneapolis. The stadium has to deal with harsh winters, challenging wind and snow loads, while still capturing the essence of the local community. During the National Football League (NFL) season it is the new home of the Minnesota Vikings NFL team.

The stadium design included a perimeter basement wall around a portion of the stadium and locker-room areas. Here, the stadium architect proposed the use of a Mechanically Stabilized Earth (MSE) stress relief wall on the Southeast side of the stadium site to minimize the horizontal stresses acting on the external basement wall. By doing this, a lower cost Concrete Masonry Unit (CMU) wall could be constructed as the basement wall. The alternative would have been cast-in-place shear walls on footings to bedrock with rock anchors, a much higher cost option. The MSE stress relief wall option, along with the CMU basement wall, saved around USD 5.5 million on this site.

The 250 m long MSE wall was located in a constrained area of the site allowing minimal excavation width. Further, the top of the MSE wall was subject to high external loads consisting of point, line and area loads from construction equipment and service vehicles. Consequently, it was decided that a granular material having a friction angle of at least 35° was required for the reinforced fill to reduce the required reinforced zone width and support the high external loads on top of the MSE wall.

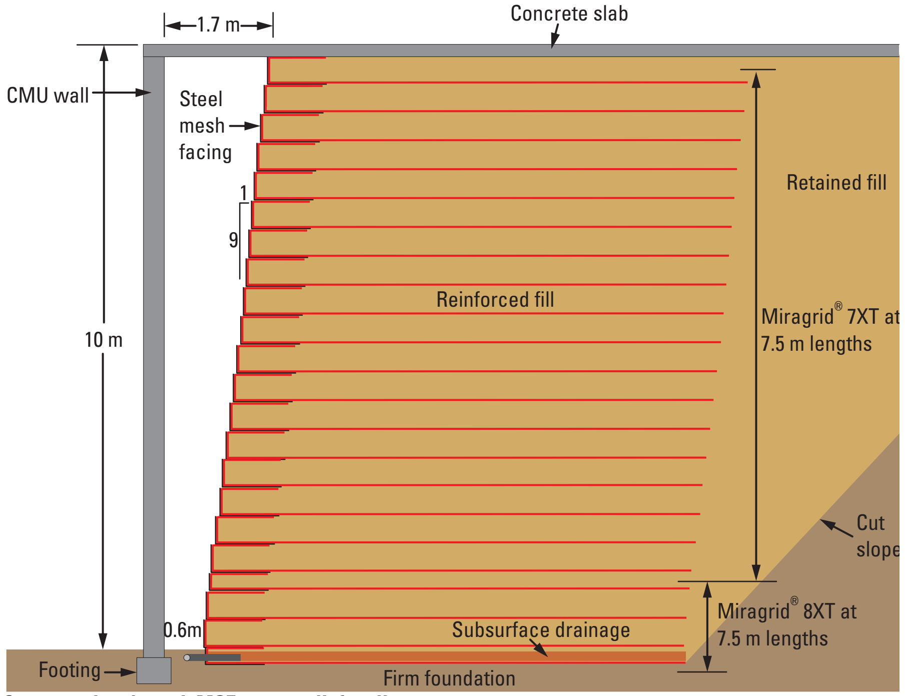
The 10 m high MSE wall was designed utilizing the Mirafi® Permanent Wirewall system which consisted of galvanised steel mesh facing units of constant 0.5 m vertical height. While individual facing units provided a vertical facing, successive units were stepped back around 25 mm so that the overall wall face would have an effective slope of 9:1. Thus, the gap left between the MSE wall face and the CMU wall ranged from 0.6 m at the base to 1.7 m at the top of the wall.

Layers of Miragrid® XT geogrid reinforcement were used at 0.5 m vertical spacings and 7.5 m lengths into the reinforced fill. Miragrid® 8XT geogrid reinforcement was required for the bottom four layers while Miragrid® 7XT geogrid was required for all succeeding layers. Miragrid® XT geogrid reinforcement is composed of high modulus PET yarns whose long term load carrying performance has been well-characterised thus, they are very suitable for long term reinforced soil applications.

There were a number of design and construction challenges for the MSE stress relief wall. These challenges involved changing wall alignments and allowances for storm and wastewater pipe utilities, inlets and manholes through the MSE wall. Also, the Miragrid® XT geogrid layout had to be adapted to allow for the presence of piles within the reinforced zone that supported an overhead pedestrian bridge. During construction, solutions were found to all of these challenges.

This project demonstrated that MSE stress relief walls are a most effective and economical solution to prevent imposed lateral loads acting on below-grade structural walls. This enables much cheaper solutions to be used for the structural walls.

Installing Mirafi® Wirewall retaining wall system



Cross section through MSE stress relief wall

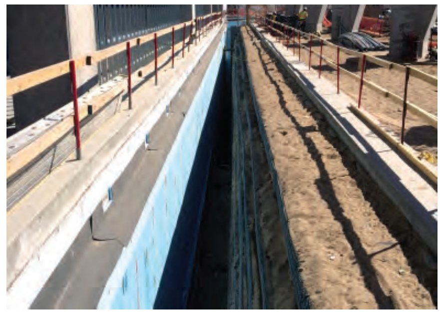
The stadium was completed in 2016 and in 2018 it hosted the NFL Super Bowl game. Since its completion it has been a great success hosting different team sports and functions.

Client: Minnesota Sports Facility Authority (MSFA), Minneapolis, Minnesota, USA.

Consultant (MSE wall): Gale-Tec Engineering Inc., Minneapolis, Minnesota, USA .

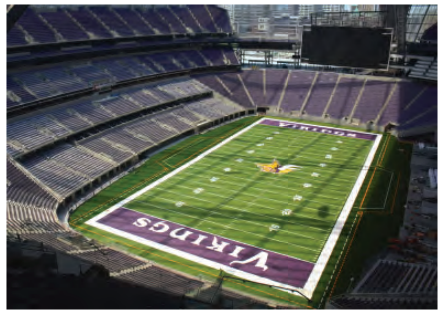
Contractor (MSE wall): Ames Construction Inc., Minneapolis, Minnesota, USA.

Almost completed MSE stress relief wall

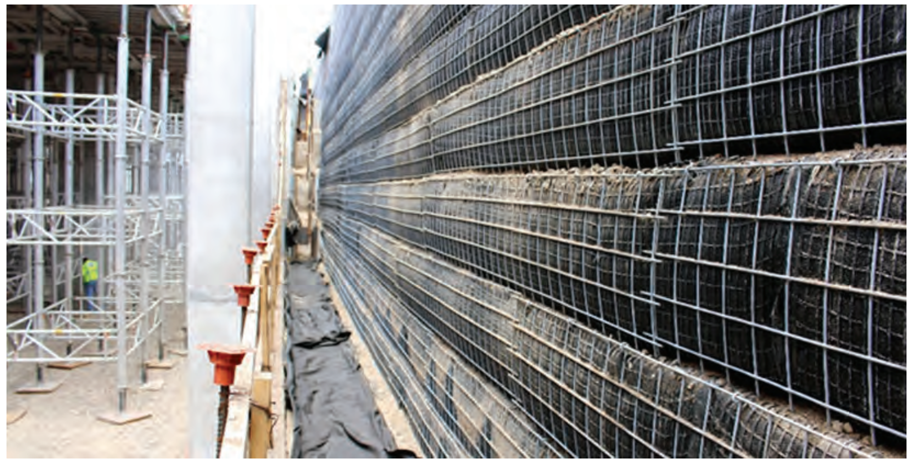


Gap between MSE stress relief wall and CMU wall

Completed US Bank Stadium



Inside the completed US Bank Stadium



Mirafi® Wirewall system stress relief wall under construction

Reinforced soil walls: MSE peripheral wall, Cherry Island landfill expansion, Delaware, USA

Cherry Island landfill has been used for municipal solid waste disposal since 1985. It is located at the confluence of the Delaware and Christina Rivers in Wilmington where the foundation conditions consist of a maximum 26 m of dredged spoil and alluvial deposits, which are soft and compressible, overlying a medium dense sand stratum. As Cherry Island landfill was nearing waste capacity the client waste authority had to develop a plan to extend the life of the landfill by an additional 25 years.

Because of its confined location, it was decided to construct a peripheral wall around the boundary of the landfill site to enable a vertical expansion of its landfill capacity. The peripheral wall was to be constructed from Mechanically Stabilized Earth (MSE) as this gave cost benefits, and also provided a flexible structure that could deform when the soft foundation soil was settling under the loading of the peripheral wall.

The problems of designing a peripheral wall up to 20 m in height on a soft foundation stratum became evident at an early stage of the process. It became clear that some form of foundation improvement treatment was required for the soft foundation stratum to support the loading of the peripheral wall. The use of cement-mixed columns inserted into the soft stratum was investigated but was found to be very costly. Consequently, an alternative of using prefabricated vertical drains (PVD's) to accelerate the consolidation of the soft stratum, and to accelerate its gain in shear strength, was adopted as this was of much lower cost. The rate of construction of the peripheral wall would be balanced with the rate in gain of shear strength of the soft foundation stratum.

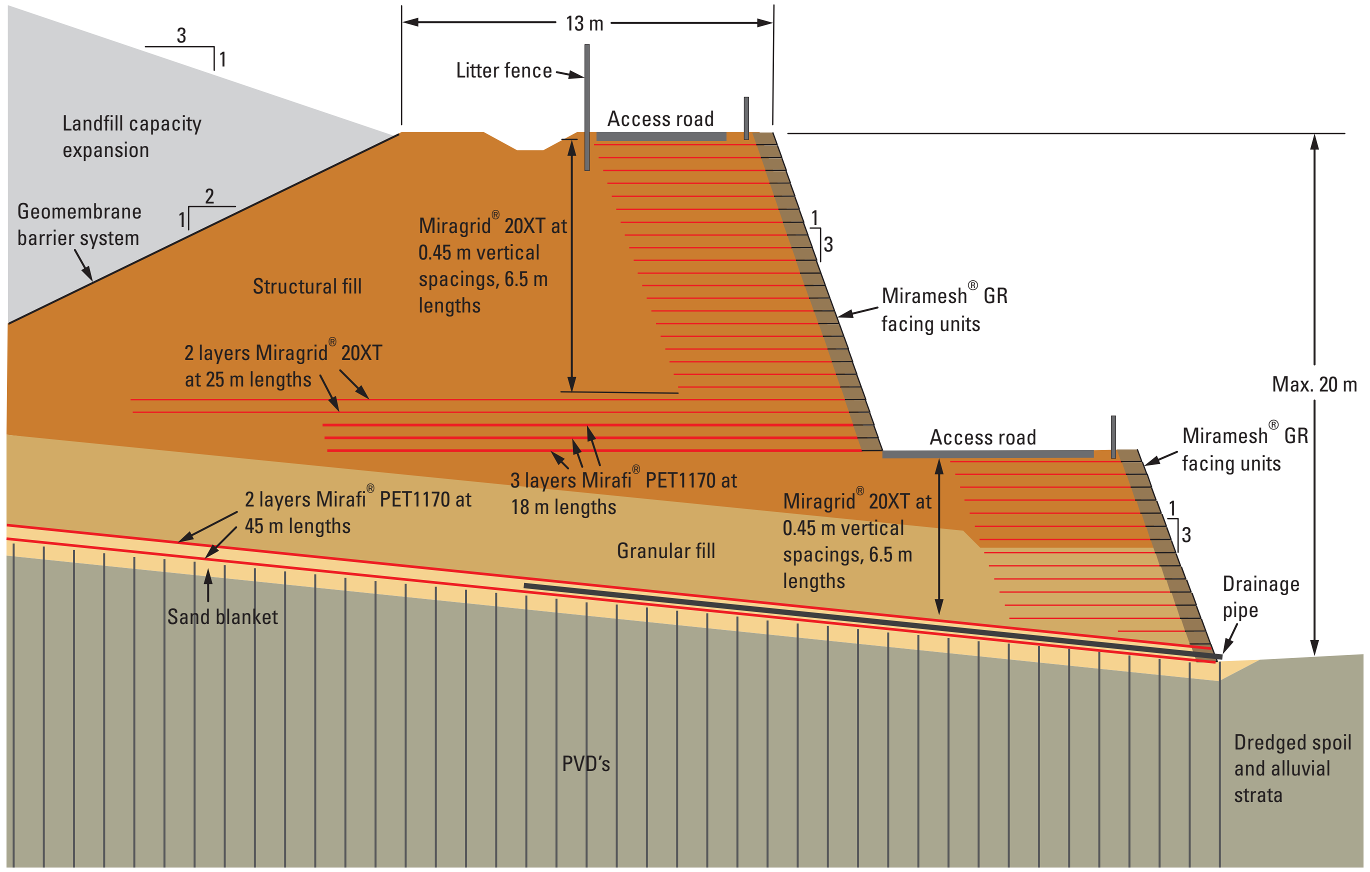
To provide additional stability during the construction of the peripheral wall, two layers of Mirafi® PET1170 geotextile reinforcement, with an initial tensile strength of 1,170 kN/m, was used as basal reinforcement across the base of the peripheral wall.

The construction of the MSE peripheral wall followed a stage construction procedure where the wall was increased in height in 3 m steps, then allowing the soft stratum to consolidate and gain in shear strength until it was safe to construct another 3 m height. Extensive foundation instrumentation was installed to monitor settlements and excess pore pressures for each 3 m lift as this was a major concern during construction of the peripheral wall. This procedure was followed until the peripheral wall was completed to the required maximum 20 m height.

A sand drainage blanket was constructed across the base of the peripheral wall. This drainage blanket also provided a stable working platform for the PVD installation rigs. The PVD's were next installed at 1.5 m spacings to the base of the soft stratum. Next, the two layers of Mirafi® PET1170 geotextile basal reinforcement was installed in 45 m lengths across the base of the peripheral wall, along with further drainage sand.

Installation of PVD's into alluvial stratum

Laying Mirafi® PET1170 geotextile reinforcement



Typical cross section through MSE peripheral wall

A layer of highly frictional granular fill was placed across the bottom half of the bottom tier of the peripheral wall to provide maximum internal frictional stability at the base of the wall.

The MSE wall was composed of Miramesh® GR steel mesh facing units with an internal erosion control blanket, stepped-back to achieve the overall 1(H):3(V) wall face angle. The layers of Miragrid® 20XT geogrid reinforcement were placed horizontally to the face of the wall, but were not structurally connected to the steel mesh facing units. The Miragrid® 20XT geogrid layers were located at 0.45 m vertical spacings and laid into the reinforced fill the required 6.5 m horizontal distance.

The Miramesh® GR face wrap enabled surface vegetation to grow on the MSE wall face.

At the base of the second MSE wall tier additional reinforcement was required for stability purposes. Here, three layers of Mirafi® PET1170 high strength geotextile reinforcement in 18 m lengths were used along with two layers of Miragrid® 20XT geogrid reinforcement in 25 m lengths.

Client: Delaware Solid Waste Authority, Delaware, USA.
Consultant: Geosyntec Consultants, Columbia, Maryland, USA.
Contractor: Sevenson Environmental Services, Inc., Niagara Falls, New York, USA.

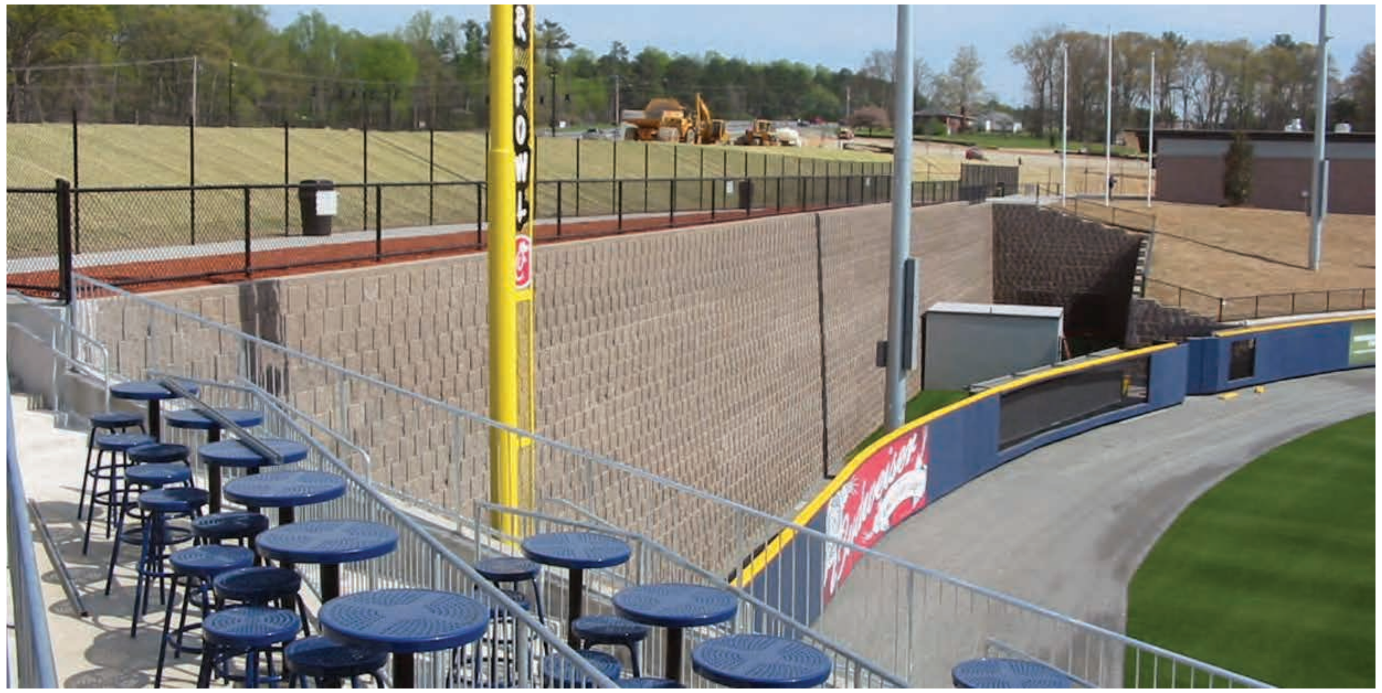
Installing layers of Miragrid® 20XT geogrid reinforcement for the MSE peripheral wall

Miramesh® GR steel mesh facing units with internal erosion control blanket

Completed MSE peripheral wall

MSE peripheral wall almost completed

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

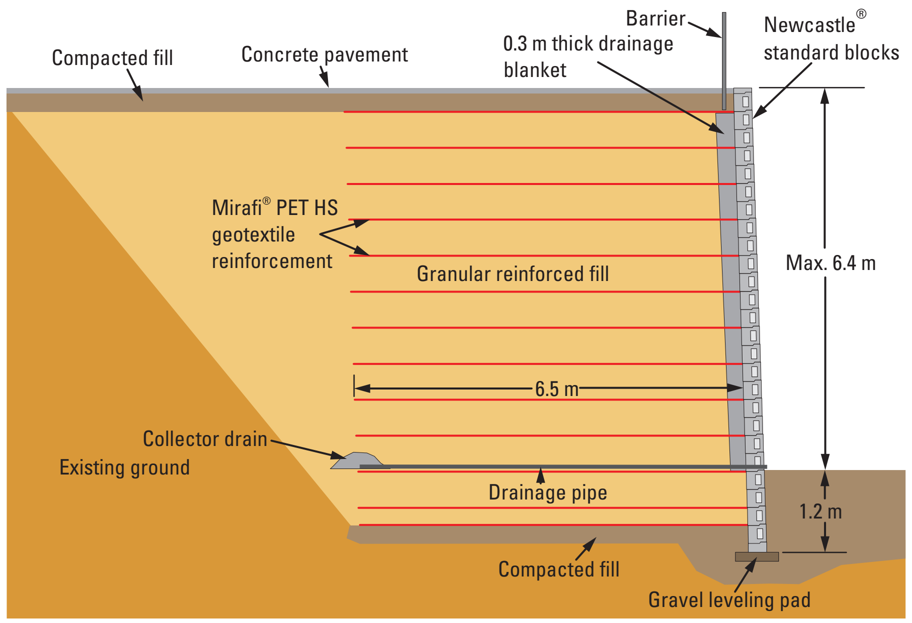
Several retaining wall structures had to be constructed within the ballpark precincts. These ranged from the wall support for the vehicular bridge at the entrance to the ballpark to the 10 m high outfield wall. These walls were constructed using segmental block facings and Mirafi® PET HS geotextile reinforcement.

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



Cross section through one of the reinforced segmental block retaining walls showing the Newcastle® block facing and the Mirafi® PET HS geotextile reinforcement

Placement of initial layers of Newcastle® blocks in the reinforced segmental block wall

Segmental block wall under construction

Segmental block wall near completion

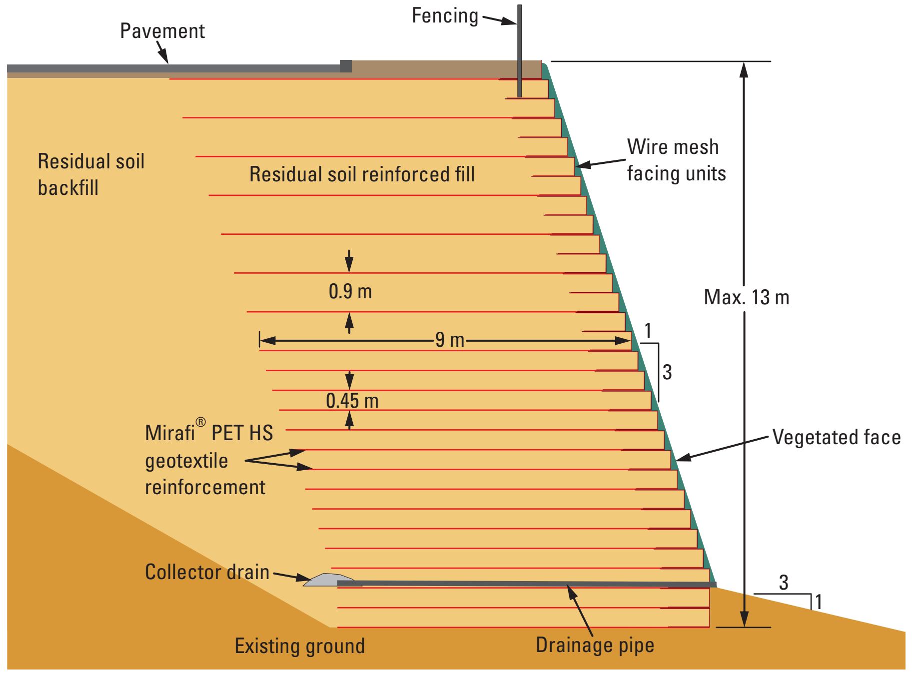
was considered to be a "green" solution compared to the originally designed concrete retaining wall.

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.



Typical cross section through the reinforced fill slope

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

Contractor: Wall Technologies Company Inc., Atlanta, Georgia, USA.

Reinforced slope under construction

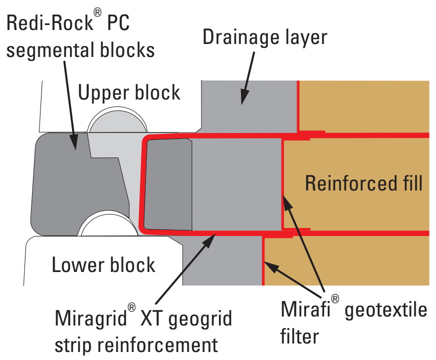
Reinforced slope showing the wire mesh facing

Completed reinforced slope with vegetation growth

Reinforced soil walls: Back-to-back MSE abutment walls, CN Rail, Montreal, Canada

Canadian National Railway (CN Rail) and the Montreal Metro were evaluating how to eliminate several at-grade railway crossings in the metropolitan Montreal area to ease congestion. The CN Rail main freight line crossed the Société de Transport de Montréal light commuter Metro line at several locations causing increased delays, scheduling challenges and added complications to the rail system.

These two rail lines run through a narrow corridor with heavy congestion and surrounding development, including residential, commercial and industrial. To eliminate the at-grade crossing at this location, plans were made to elevate the CN Rail line onto a bridge structure to overpass the lowered Metro line to enable it to pass under the new bridge overpass. This approach minimized the size and height of the bridge overpass and associated abutment structures.



Redi-Rock® PC segmental block connection details with Miragrid® XT geogrid strip reinforcement

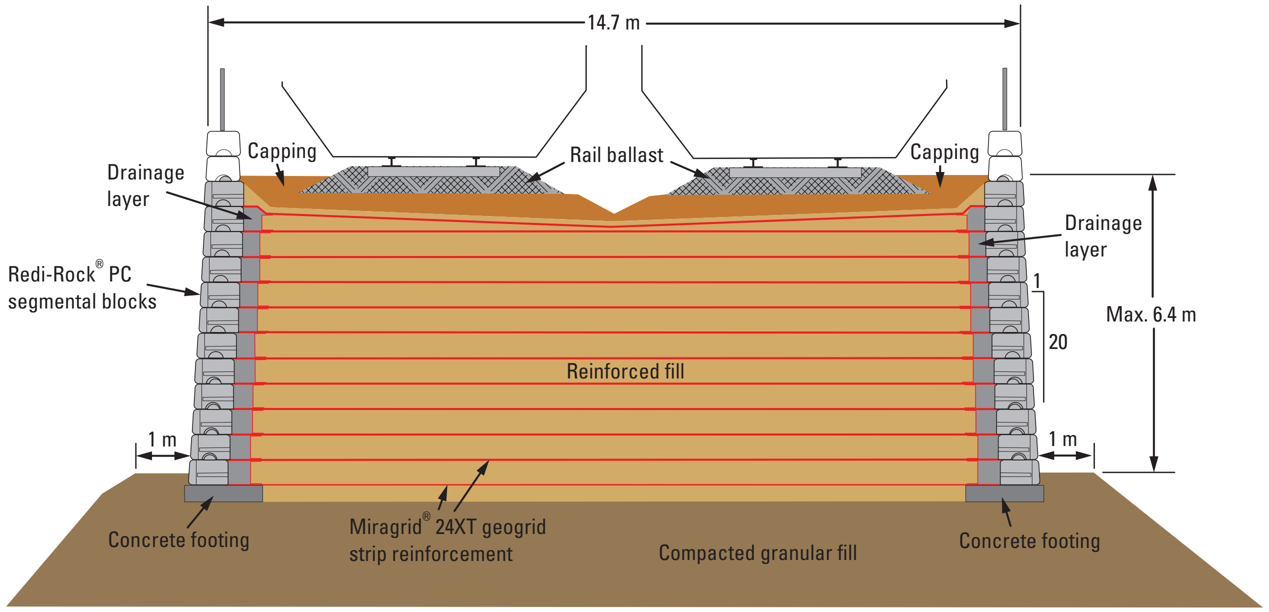
Due to the narrow right of way in the area, wide abutments for the CN Rail bridge overpass approaches were not possible. The only option was to construct vertical retaining structures for the abutments. Soldier pile and lagging and mechanically stabilized earth (MSE) wall systems were considered. A back-to-back MSE wall system was considered the best option from the viewpoint of cost, simplicity of equipment used and no construction noise on site.

MSE wall systems can utilize either metallic or geosynthetic reinforcements. For this site, the presence of high voltage electrical cables for the commuter line excluded the use of metallic reinforcements because of the possibility of stray electrical currents in the bridge overpass abutments. Further, the abutment walls would be subject to heavy dynamic loads due to the high loadings of the freight trains passing over the bridge overpass. Thus, an MSE wall system that provided a positive mechanical connection with the facing units and used geosynthetic reinforcements would only be considered.

The MSE retaining wall system that met all the designer's requirements was the Redi-Rock® PC segmental block system along with Miragrid® XT geogrid reinforcement. This segmental block system utilizes large, pre-cast, concrete block facing units 1.17 m long, 0.46 m high and 0.7 m deep, and has a novel connection system with the associated Miragrid® XT geogrid reinforcement, where 0.3 m wide strips of Miragrid® XT geogrid are laced vertically through the bottom to the top of the block and then extended into the reinforced fill the required distance to provide the necessary wall stability. This connection system enables almost a 100% load capacity to be achieved with the Miragrid® XT geogrid strip reinforcement.

To meet the high vertical external loading requirements for the back-to-back abutment walls at the site the


Installing the Miragrid® 24XT geogrid strips with the Redi-Rock® segmental block system



Cross section through the back-to-back MSE abutment walls

Redi-Rock® PC segmental block system along with Miragrid® 24XT geogrid strip reinforcement was used.

The reinforced fill used for the back-to-back abutment structures was a highly granular material that had high frictional properties. This meant that it could achieve its peak frictional behaviour at very low deformations.

The construction of the back-to-back MSE abutment walls comprised 4,200 m^2 of wall face. In addition to being able to withstand the high vertical loads acting on the abutments, the MSE system used for this project presented an aesthetic appearance that enabled it to blend in well with its surroundings. The back-to-back MSE abutment walls were installed quickly and without complications.

During and following construction monitoring of the horizontal movements of the MSE wall abutments were carried out with typical deformations being a maximum of only 5 mm along the wall length.

Client: CN Rail, Ottawa, Canada.

Consultant: Journeaux Associates, Quebec, Canada.

Contractor: CRT Construction, Montreal, Canada.


Layout of Miragrid® 24XT geogrid strips with Redi-Rock® PC segmental blocks


CN Rail overpass abutment near completion


Completed back-to-back MSE abutment in operation


Construction of the back-to-back MSE abutment walls

Reinforced soil walls: Reinforced avalanche barriers, Mittenwald, Germany

Mittenwald is located in the Northern foothills of the Alps in Bavaria, near the Austrian border, Southeast of Garmisch-Partenkirchen. In February 1999 an avalanche came very close to destroying houses and roads, and from that date community leaders pursued the goal of providing an effective avalanche barrier to protect against future avalanche flows.

The avalanche source area is approximately 4.3 ha, part of which has a very steep track of 73°. Large avalanches fall straight down towards the Mittenwald valley floor. Simulations have shown that an extreme avalanche can have a flow width of 100 m, a flow height of 4 m and a downslope velocity of 40 m/s. Such a destructive avalanche threatens various residential areas of Mittenwald, its road infrastructure and high voltage power lines.

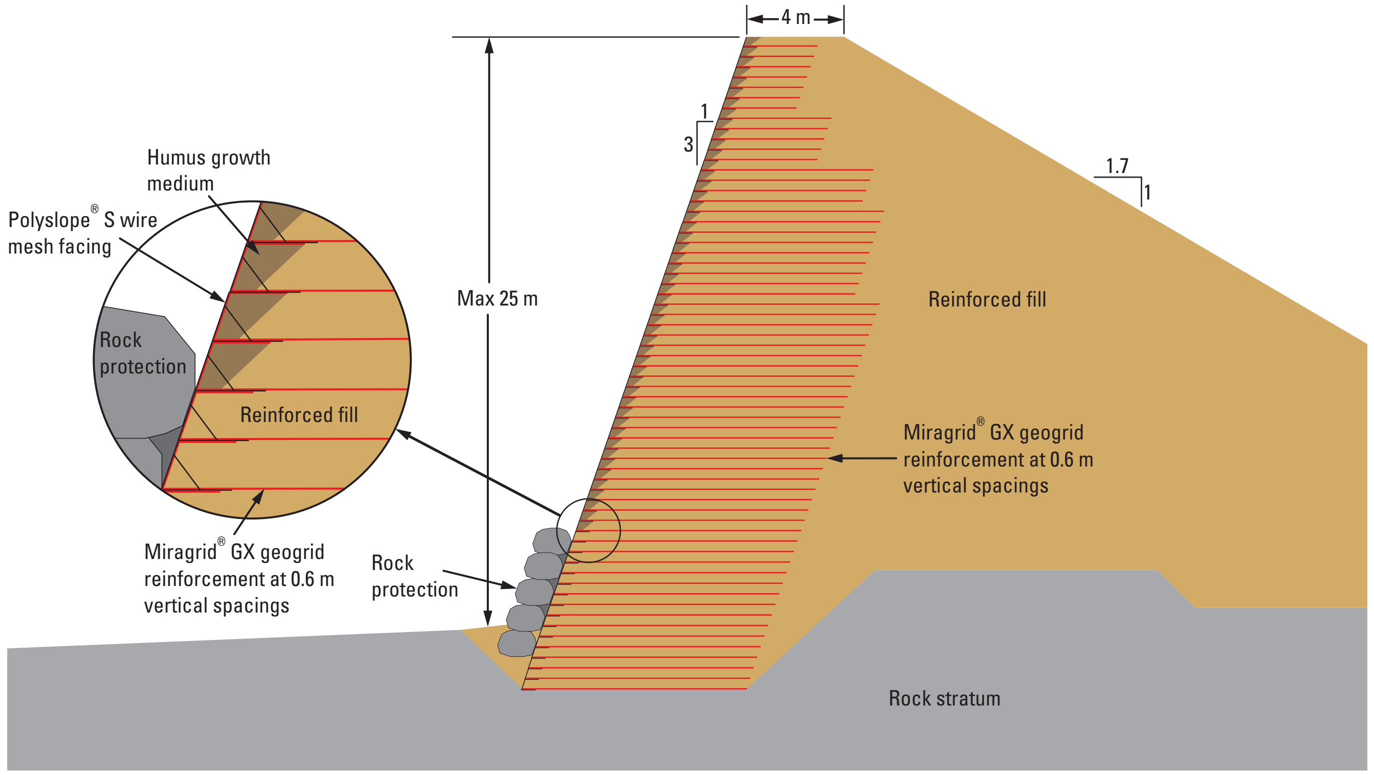
To ensure effective protection of Mittenwald, two protection barriers were constructed. One avalanche protection barrier was located at the foot of the avalanche fall line with a length of 310 m and a maximum height of 25 m with a 70° slope facing on the upper side. Another avalanche barrier was connected to the first one, with a length of 140 m and a height of 13 m with a 70 ° slope facing on both sides. Additionally, the valley station of the cable car was relocated away from any avalanche flows and a system for artificial avalanche triggering and automatic snow depth recording was installed in the catchment zone. The project was designed to prevent the flow part of the avalanche from reaching the federal road and the residential areas. The wet snow avalanche is deflected by the barriers and guided in a predetermined avalanche path to a designated deposition area.

Construction earthworks required a total volume of 150,000 m^3 of earth and rockfill removal. Only 16 workers were employed with three excavators, two bulldozers, a roller and two dump trucks. The bulk material was blasted out of the valley floor and processed on site, providing high-quality filling material. At the same time, space was also created for the snow masses expected in the event of an avalanche, which can be deposited in front of the avalanche barriers. For the Mittenwald residents, the construction remained almost hidden from view, as the cycle between extraction and recycling took place within the construction site boundary.

The protection barriers utilised the Polyslope® S reinforced wall system which included the wire mesh facing set at a 70° face angle, Miragrid® GX geogrid reinforcement and compacted reinforced fill. The construction of the two avalanche protection barriers resulted in large structures, but not at the expense of visual appearance. Planting vegetation on the face ensures that they are not perceived as "foreign bodies" in the landscape. In spite of the 70° steep face, a simple and quick implementation of surface greening was found by using locally grown willow cuttings. The insertion of these plants already resulted in vegetation growth during construction. As the willows grow, their root system reinforces the surface area of the wall facing.

The layers of Miragrid® GX geogrid reinforcement ranged in tensile strengths from 35 kN/m to 160 kN/m and were used in different embedment lengths depending on their levels within the barrier walls. The vertical spacings between adjacent reinforcement layers were a constant 0.6 m.

Site preparation works



Typical cross section through the reinforced avalanche barriers

The smaller protection barrier built parallel to the federal road was completed in 2016 with a total reinforced fill volume of 25,000 m^3. The larger protection barrier, which slows down and diverts the incoming snow flow was begun in 2017. It integrates into the steep terrain and has an arched curve to deflect the snow flow. A 3 to 5 m high rock scour protection layer was placed at the foot of the upper side of this barrier. In the event of an avalanche, this scour protection layer is expected to protect the barrier from damage.

Due to the simplicity of the Polyslope® S reinforced soil system, the construction work was completed during the summer of 2017 thus the safety of the population of Mittenwald could be guaranteed as early as the winter 2017/2018.

Within a few days over the New Year, the Northern side of the Alps experienced fresh snowfall of a magnitude not seen in decades. For the combined avalanche measures in Mittenwald it was the best opportunity to put their interaction to the test. Several times, the threatening snow depth was reached in the Karwendel and blasted off in a targeted manner. The snow masses were slowed down, successfully deflected and deposited by the avalanche protection barriers.

Client: Gemeinde Mittenwald/ Wasserwirtschaftsamt Weilheim, Germany.

Consultant: Klenkhart & Partner Consulting GmbH, Germany.

Contractor: Habau Hoch- und Tiefbaugesellschaft GmbH, Germany.

Installation of Polyslope® S mesh facing along with humus growth medium and Miragrid® GX geogrid reinforcement layer

One of the avalanche barriers completed

Avalanche barriers nearing completion

Aerial view of completed avalanche barriers

Reinforced soil walls: MSW incineration and power plant platform, Yuxi City, Yunnan Province, China

The proposed Yuxi City Municipal Solid Waste (MSW) Incineration Plant involves a total investment of USD 50 million. The site lies at an elevation of about 1,800 m above sea level and is located in rugged hilly terrain of Hongta District, some 30 km from the centre of Yuxi City and about 100 km from Kunming City, the provincial capital of Yunnan Province. When completed the incineration plant will manage the MSW not only from Hongta district but also the nearby counties of Eshan and Tonghai.

The incinerator is expected to handle 700 tonnes of MSW daily by 2020, increasing to 1,000 tonnes daily by 2030. The incinerator will operate at a temperature of 800°C reducing the MSW to ash with the associated heat energy created used to generate electricity using steam turbine generators. The electricity generated will be sold to the State Grid Corporation of China.

The development covers a land area of 7 ha and includes major earthworks, slope stabilisation works and construction of retaining walls to create level platforms for the incineration plant, power station and associated ancillary infrastructure. The natural topsoil layer consists 0.6 m of yellow, brown to red silty clay with traces of limestone gravel and plant roots. Below the topsoil is up to 12 m deep layer of weathered limestone residual gravelly soil. The weathered limestone residual soil sits above a weathered limestone rock formation. The ground terrain is higher in the North and lower in the South.

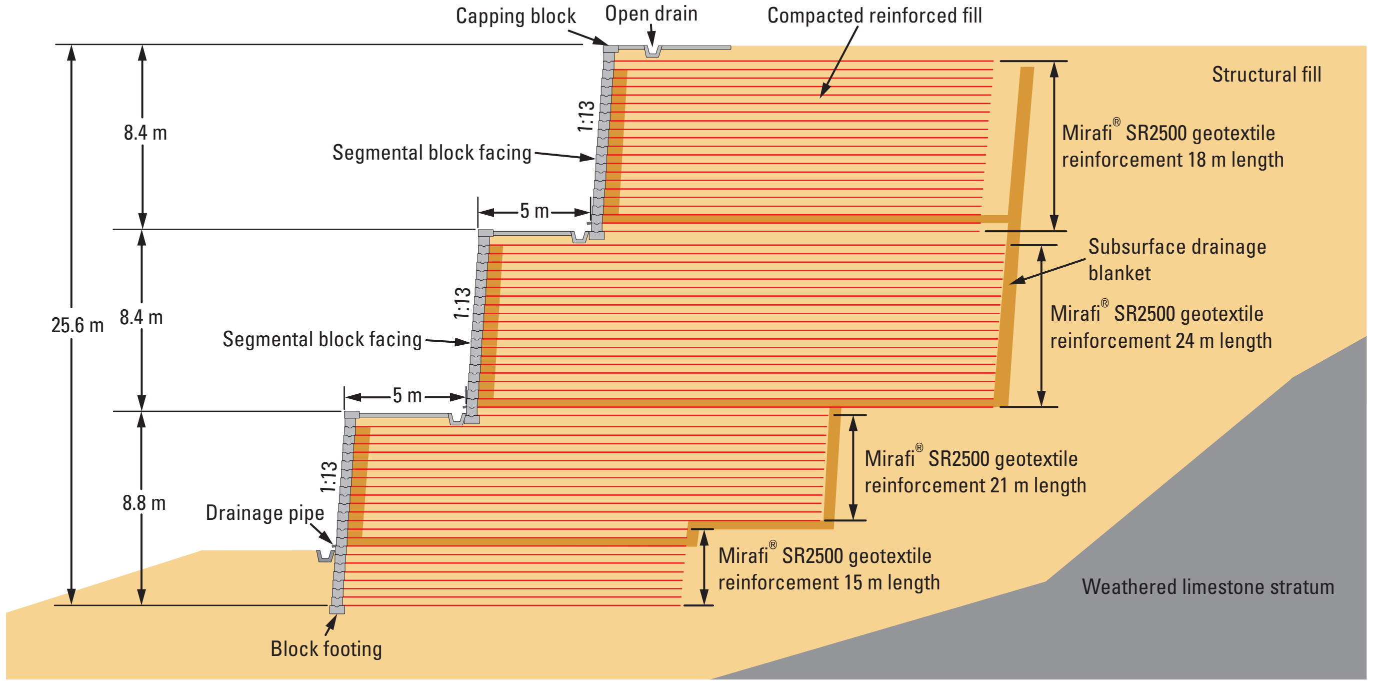
The retaining wall system adopted involved the use of a reinforced segmental block retaining wall system, while using the on-site limestone residual soil for the reinforced fill. The segmental block facing units consisted of wet-cast blocks 0.4 m wide, 0.4 m high and 0.5 m deep. Each block had tongue and groove features at the bottom and the top of each block to enhance the horizontal shear resistance between adjacent blocks and to provide a better connection capacity between the geotextile reinforcement and the block facing. Complementary modular footing and capping units are used.

The reinforcement used for the segmental block wall system was Mirafi® SR2500, a PET geotextile reinforcement having an initial tensile strength of 250 kN/m. Mirafi® SR geotextile reinforcements have good long term performance and is well-suited for long term reinforced soil applications.

At the Northern boundary of the development site a cut slope was formed with associated slope stabilisation works. At the Southern side of the development site a three-tiered retaining wall of about 25 m in total height and over 300 m in length was constructed. At approximately 30 m set-back further North a fourth single-tiered retaining wall of 8 m in height and over 350 m in length was also constructed.

The site is located in a remote area and was not easily accessed during site construction prior to construction of the paved access roads. Thus, the ability of the retaining wall system to utilize on-site material as the reinforced and platform fill was a huge economical advantage in deciding on the adoption of the retaining wall system. The choice of wet-cast segmental block facing units that could be cast nearby was another plus factor for the wall system

Precast concrete segmental blocks being used for the facing of the walls



Cross section through the first three tiers of the reinforced segmental block walls

adopted. The shape and size of the segmental blocks was also ideal for the project as they could be handled by two workers during placement, without the need for mechanical lifting equipment; but yet large enough to satisfy good facing construction quality and post-construction serviceability design and performance limits.

The silty clay topsoil was removed down to the weathered limestone residual stratum and excavated down to the base level of the bottom tier retaining wall. Here, the ground was levelled out and the segmental block footing units were then placed. Next, the base layer of Mirafi® SR2500 geotextile reinforcement was then laid over the segmental block footing units along with the required reinforcement length for laying behind the wall facing. The next facing block was lifted into place manually locking the lower geotextile reinforcement layer in place between the two adjacent blocks. The geotextile reinforcement was then spread out and pulled taut before the reinforced fill was placed over the geotextile reinforcement. The reinforced fill was then compacted to achieve a minimum of 95% of Standard Proctor dry density. When this was achieved, the process was repeated with further layers of geotextile reinforcement, facing blocks and reinforced fill. When the required wall height had been reached the wall was finished with the placement of capping units.

Placement of reinforced fill on top of Mirafi® SR2500 PET geotextile reinforcement

Once the first retaining wall tier had been constructed the second and third tiers were constructed in the same manner. These upper tiers were set back 5 m from the crest of each lower tier. After this three-tier retaining wall had been completed a fourth tier was constructed to form the structural fill platform for the incinerator and power station.

The rate of wall construction was very good with about 11,000 m² of wall completed within 7 months. Apart from normal earthworks and compaction equipment, no additional machinery was required for construction of the retaining walls.

Client: Yuxi Kelin Environmental Protection Technology, Yunnan Province, China.

Consultant: Beijing Urban Construction Design and Development Group, Beijing, China.

Contractor: Yunnan Construction and Investment Holding Group, Yunnan Province, China.

Laying out Mirafi® SR2500 geotextile reinforcement behind the segmental block facing

Reinforced segmental block walls showing drainage pipe exits

Reinforced soil platform complete and awaiting incineration and power equipment installation

Reinforced soil walls: Retaining walls for housing development, Bukit Pelali, Johor, Malaysia

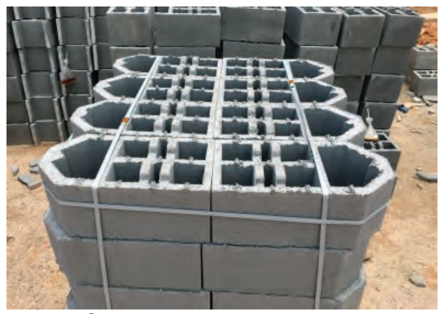
Bukit Pelali is a 150 ha housing development project located in Johor at the Southern tip of Peninsular Malaysia. It is nestled within an iconic hill that inspired its name and is located a short distance from the mega oil and gas hub Pegerang Integrated Petroleum Complex (PIPC) in Southern Johor state, which is one of the region's largest hubs for oil and gas, petrochemical industries, oil storage and trading activities, opposite Singapore.

This development is located in hilly terrain which involved major cut and fill earthworks to form the required housing development platform levels. The foundation conditions at the site consist of layers of stiff sandy clay overlying a firm stratum.

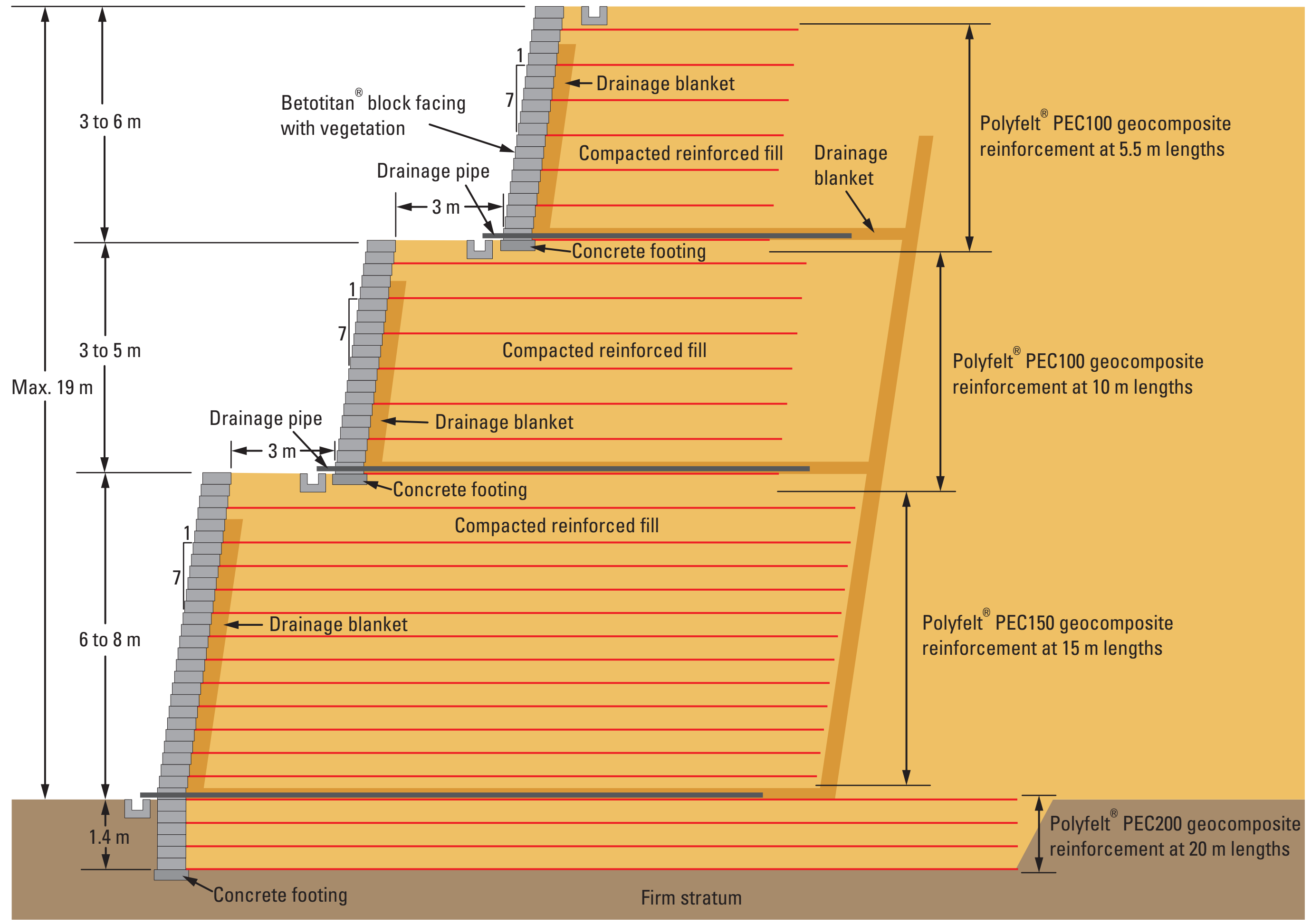
To create the flat platforms for the housing development two retaining wall structures were required on site. One retaining wall structure had to have a maximum height of 13 m while the other wall had to have a maximum height of 19 m. It was decided to use a reinforced segmental block wall system throughout the site. The maximum 13 m high wall was designed as a single-tier wall while the maximum 19 m high wall was designed as a three-tier wall. Total retaining wall length was approximately 830 m.

The reinforced segmental block retaining wall system chosen for the site consisted of a Betotitan® block facing with layers of Polyfelt® PEC geocomposite reinforcement. The Betotitan® block units consisted of dry-cast concrete blocks 0.25 m wide, 0.2 m high and 0.5 m deep. An advantage of the Betotitan® block units was that vegetation could be included in the block facing to give better aesthetics after construction of the retaining walls. The layers of Polyfelt® PEC geocomposite reinforcement were designed to different strengths, different lengths, and different vertical spacings (2 or 3 block heights), depending on their location in the retaining walls. The use of Polyfelt® PEC geocomposite reinforcement had two major advantages for use in these segmental block retaining walls. First, Polyfelt® PEC geocomposite reinforcement is composed of high modulus PET fibres where the long term strength and strain is well-defined making it ideal for reinforced soil applications. Second, the material is highly flexible which makes it easy for this material to conform to the surfaces between adjacent facing blocks giving good connection capacity between the reinforcement and the segmental blocks.

To construct the reinforced segmental block walls the foundation was excavated by 1.4 m to remove the softer surface layer down to a firm stratum. In this excavated area layers of Polyfelt® PEC200 geocomposite reinforcement, 20 m in length, were connected to the block facing units to create a strong and stable foundation for the retaining walls. At the bottom of each tier a concrete footing was installed to provide a strong and level base for the placement of the segmental blocks. For the bottom retaining wall tier layers of Polyfelt® PEC150 geocomposite reinforcement, 15 m in length, were located at 0.4 m (2 block height) or 0.6 m (3 block height) vertical spacings. The layers of Polyfelt® PEC geocomposite reinforcement were pulled tight to remove any wrinkles with the reinforced fill then placed and spread on top and compacted to 95% Standard Proctor compaction. For the upper tiers layers of Polyfelt® PEC100 geocomposite reinforcement, 10 m or 5.5 m in length, were located at 0.6 m (3 block height) vertical spacings throughout.



Betotitan® segmental block facing units



Cross section through the three-tier segmental block retaining wall

Extensive subsurface drainage measures were included throughout the retaining wall structures. Horizontal drainage blankets were incorporated at the base of each tier and these were extended up behind the reinforced soil zone. A vertical subsurface drainage layer was also included at the rear of the segmental block facing. The subsurface drainage layers consisted of granular drainage material wrapped with a Polyfelt® TS nonwoven geotextile filter. Drainage pipes were installed to enable easy exit of the seepage water through the wall facing.

Client: Astaka Padu Sdn Bhd, Saling Syabas Sdn Bhd, Johor, Malaysia.

Consultant: Jurutera JRK Sdn Bhd, Johor, Malaysia.

Contractor: JBB Builders (M) Sdn Bhd, Geopakar Engineering Sdn Bhd, Malaysia.

Completed single-tier segmental block wall with vegetation developing on face

Installing Polyfelt® PEC150 geocomposite reinforcement behind segmental block facing

Partial completion of Betotitan® segmental block wall

Completion of three-tier segmental block wall

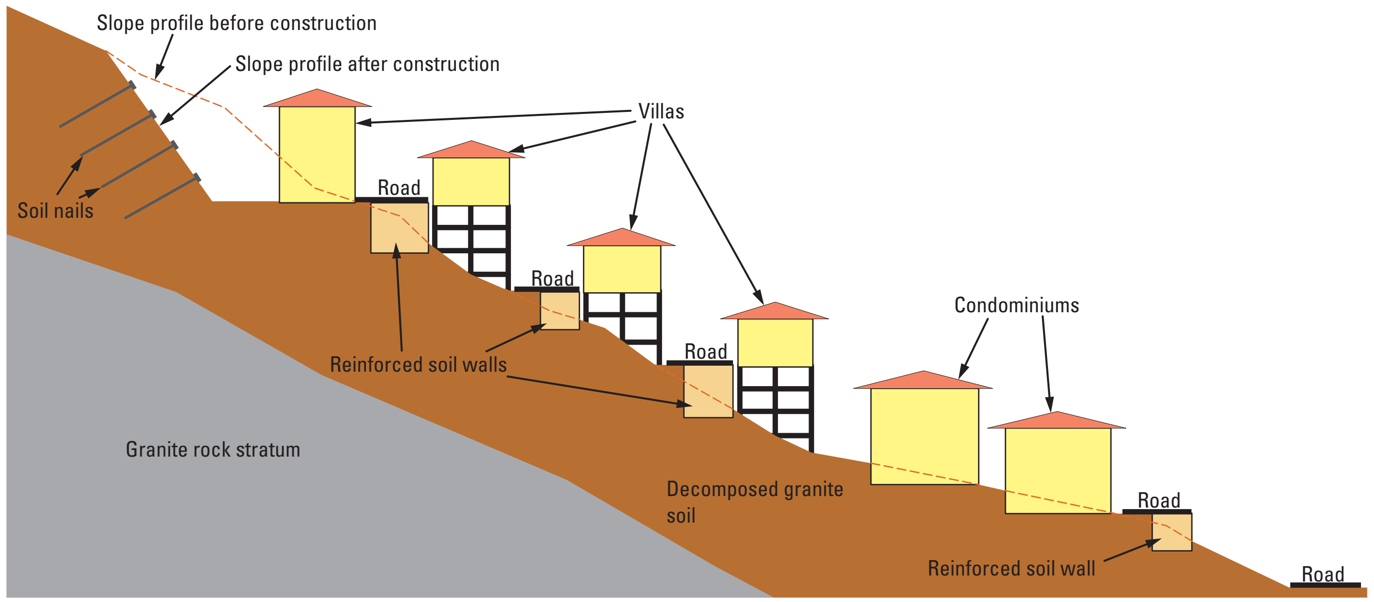
Reinforced soil walls: Hill side housing development, Batu Ferringhi, Penang, Malaysia

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.

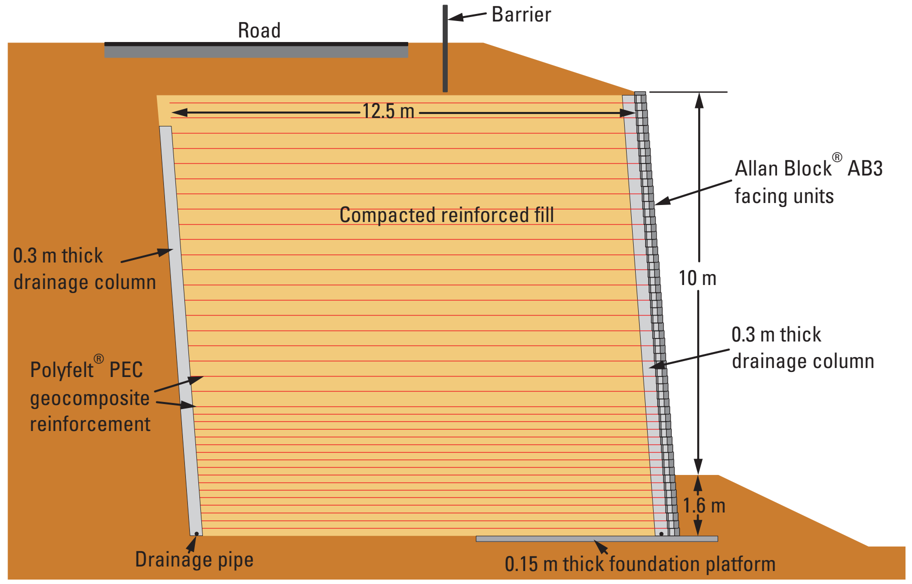
At this location, the residual granite soil mantle can have a thickness of up to 20 m overlying granite rock strata. Granite boulders often exist within the weathered soil layer close to the surface. In general, the pristine slopes tend to be stable, with the decomposed granite residual soil having effective 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 26% 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



Cross section through the terraced slope development showing the location of the reinforced soil walls



Cross section through a 10 m high reinforced soil wall on site

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 rainfalls are 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.

Placing of Allan Block® facing units

Construction of the reinforced segmental block walls on the hill side development

Completed villa on the development

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, Malysia.

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.

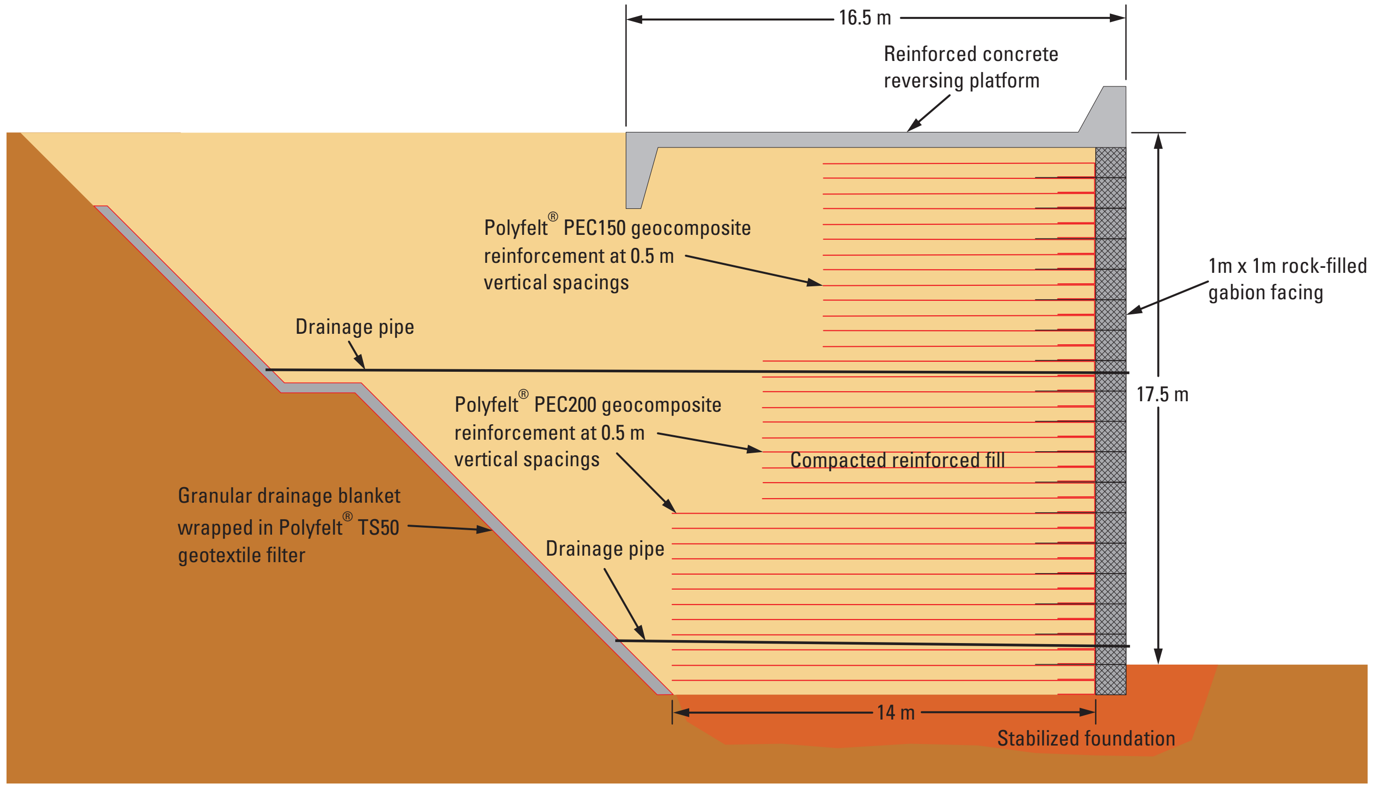
The coal at the Sangatta mine is delivered to the coal crushing plant where it is crushed and, if necessary, screened and washed at the coal washing plant before it is placed onto a 13 km long overland conveyor belt for transportation to the shipping terminal.

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.

Placing Polyfelt® PEC geocomposite reinforcement across reinforced soil wall



Section through retaining wall at maximum height

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.

Compacting granular drainage blanket wrapped in Polyfelt® TS50 geotextile filter

Construction of gabion facing

Reinforced soil walls: Reinforced segmental block culvert wall, Redbank Plains, QLD, Australia

The conversion of farmland into the development of the Redbank River Park Industrial Estate in Redbank Plains, Ipswich, Queensland required construction of Monash Road; a new primary road into the estate. The road alignment crossed a creek bed comprising soft ground which required the construction of a culvert and a vertical retaining wall with its highest point being at the culvert.

The designer chose a Keystone® Compac segmental block structure with Miragrid® XT geogrid reinforcement that encompassed a concrete pipe culvert passing through the base of the wall to allow surface water from the estate to flow out beneath the wall structure. The 230 m long structure with a maximum height of 7.2 m was designed in accordance withBritish Standard BS8006:2010 and Australian Standard AS4678:2002 to support high volumes of heavy vehicular traffic with a design life of 120 years.

Foundation conditions in the location of the creek bed comprised soft clay requiring excavation and replacement with a 1 m rock mattress foundation to support the culvert and prevent differential settlement between the culvert and the remainder of the retaining structure. To ensure water flowing through the culvert was discharged away from the base of the culvert wall, and to prevent the risk of local erosion, a stone filled wire mattress protection layer was installed within the creek bed beyond the wall.

To provide a stable base for the retaining wall a 0.3 m thick reinforced concrete foundation pad was constructed along the wall length. Keystone® Compac segmental blocks were used for the wall facing. Layers of Miragrid® 8XT geogrid reinforcement, having an initial tensile strength of 110 kN/m, were installed at 0.6 m vertical spacings and 8 m constant lengths. A 0.6 m thick vertical drainage layer with a nonwoven geotextile filter was installed immediately behind the block facing units.

The reinforced fill was a good-quality granular material and this was compacted to 95% Standard Proctor density using a sheepsfoot roller.

Two rows of guard rails were installed on top of the wall. The outer guard rail immediately behind the wall face was to protect people from falling over the wall. The anchorage for this guard rail along the top of the wall face consisted of PVC pipes placed down behind the facing blocks with the guard rails subsequently concreted in place. For the inner row of guard rails protecting the road edge were installed in a similar manner.

It was observed that there was no deformation of the retaining wall during construction and the wall remains in an excellent condition, even under heavy traffic loadings and flood events.

Council: Ipswich City Council, Queensland, Australia.

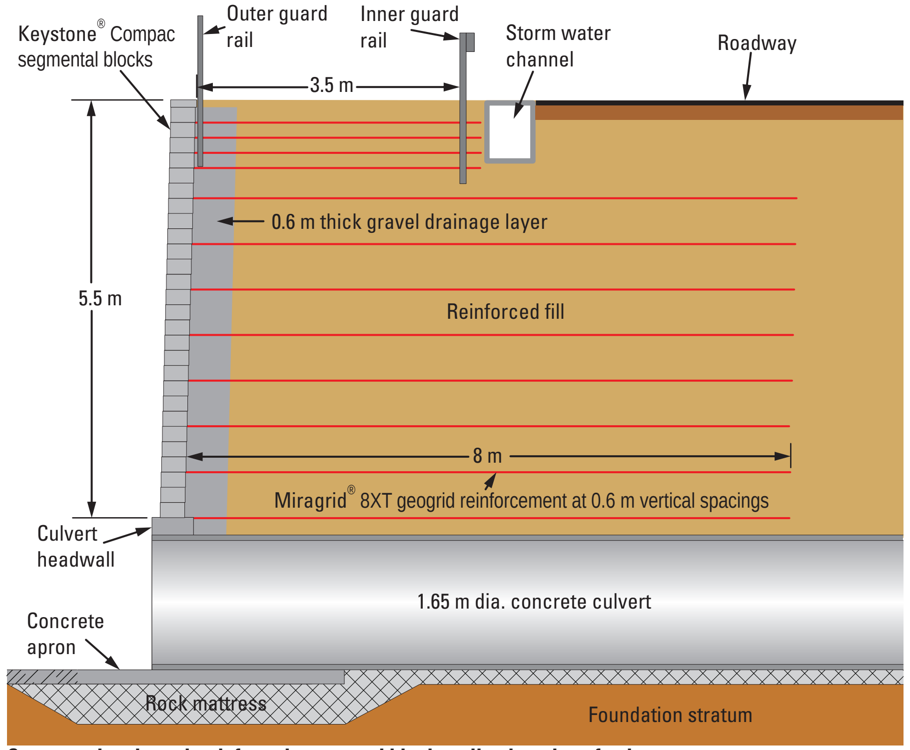
Main contractor: Shadforths Civil Contractors Pty Ltd, Queensland, Australia.

Wall consultant: Geoinventions Consulting Services Pty Ltd, Queensland, Australia.

Wall contractor: Concrib Pty Ltd, Queensland, Australia.

AS4678:2002 Earth-Retaining Structures, Standards Australia.

Soft soil conditions through culvert area

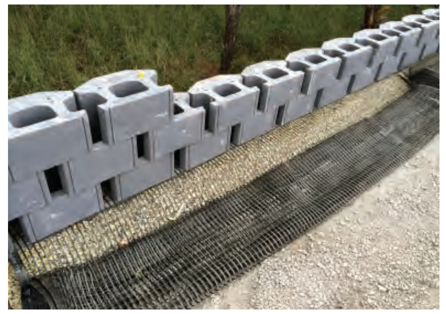


Cross section through reinforced segmental block wall at location of culvert

BS8006:2010 Code of practice for strengthened/reinforced soils and other fills, British Standards Institution.

Construction of the reinforced segmental block wall using Miragrid® 8XT geogrid reinforcement cut to design length

Keystone® Compac block wall at height of concrete culvert head wall



Reinforced segmental block wall showing Keystone® Compac block, Miragrid® 8XT geogrid and drainage layer components

Compacting the granular reinforced fill behind the segmental block wall

View of PVC pipe immediately behind block facing to house outer guard rail

Completed reinforced segmental block wall at culvert location

Reinforced soil walls: Reinforced walls and slopes, Upper Harbour Corridor, Greenhithe, New Zealand

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

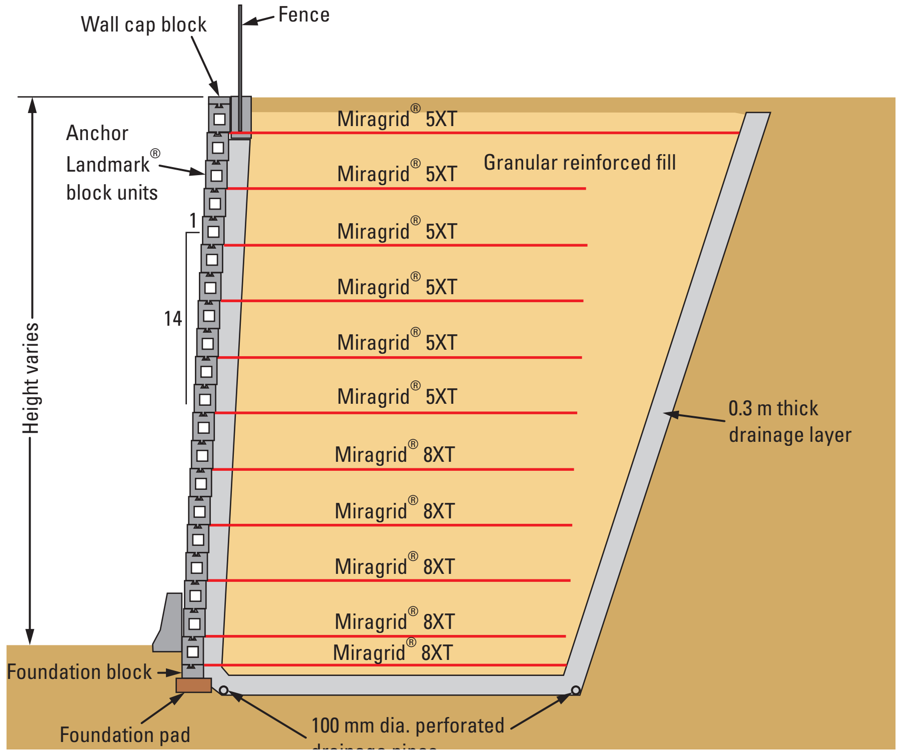
The walls had to be designed to accommodate both static and seismic loadings, as the area is prone to seismic activity. For performance and economical reasons it was decided to use Anchor Landmark® segmental block units for the wall facings as these provided a full positive connection with the Miragrid® 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.

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® 140NC 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 construction of reinforced fill slopes at the sides of the embankment earthworks sections enabled the embankment surface area to be maximised, while minimising the amount of land acquisition.

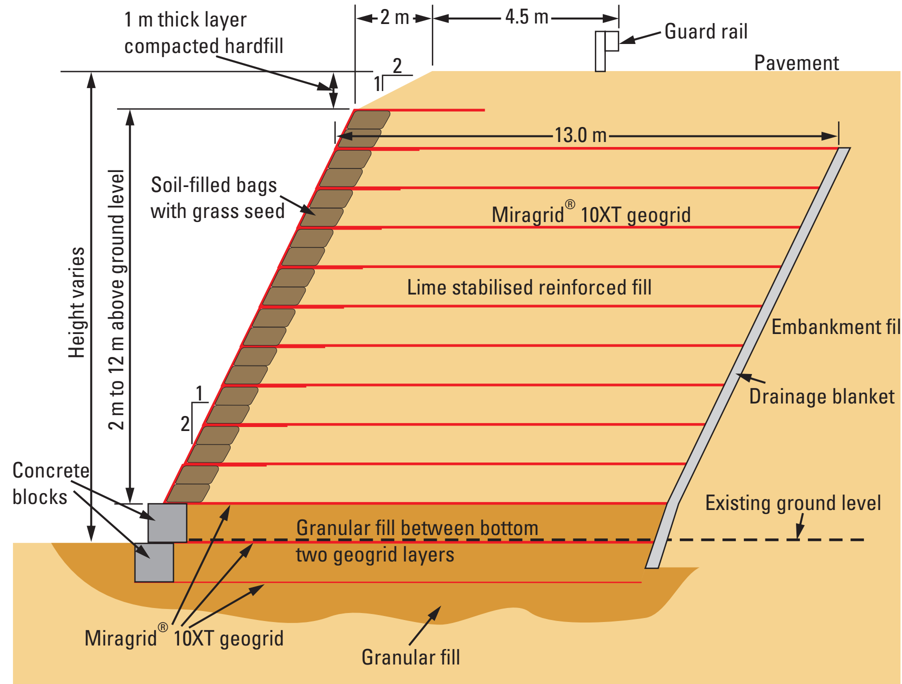


Typical cross section through the reinforced soil walls


Placing Anchor Landmark® segmental blocks for the wall facing


Installing the block facing to specific curve alignments


One wall near completion



Typical cross section through 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 extended out of the slope, wrapping around the soil filled socks, and extending into the slope 2 m. Slope face angles were maintained at 2V: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® 140NC geotextile filter, ran down the rear of the reinforced fill zone into the free-draining granular fill at the base of the slopes.


Placement of Miragrid® XT geogrid at base of reinforced fill slope


Construction of one of the reinforced fill 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: 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.

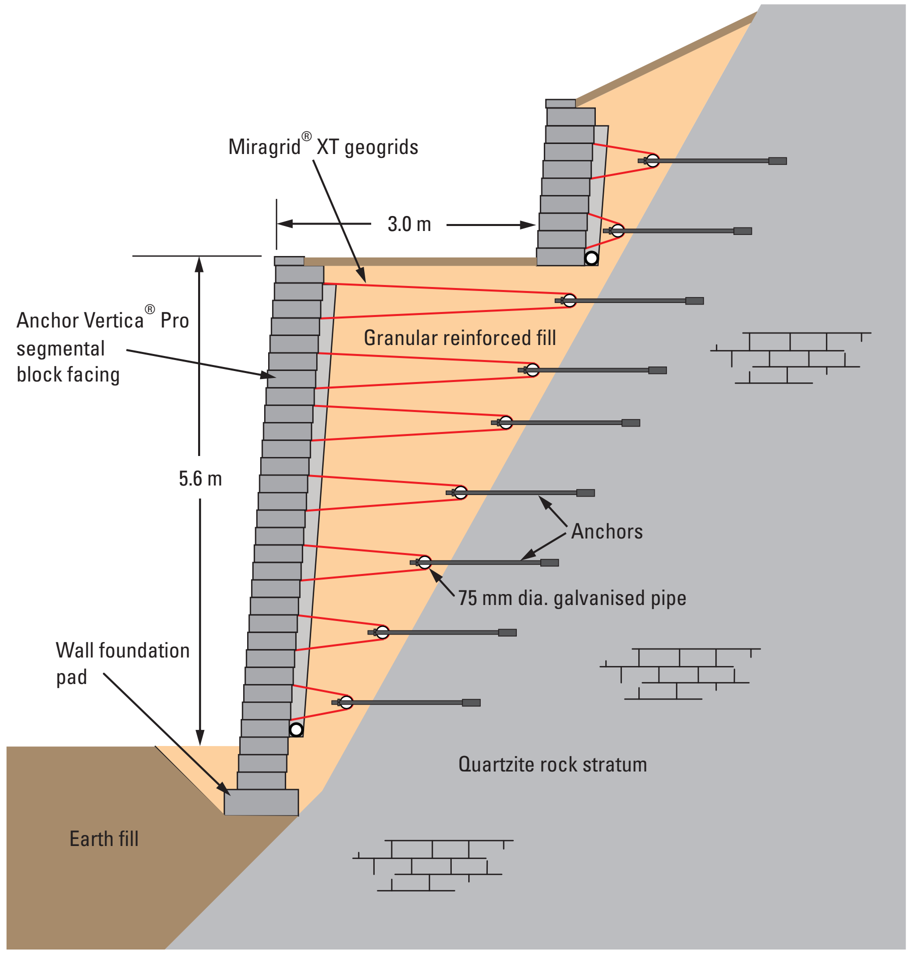
The surface of the quartzite stratum in the vicinity of the proposed retaining wall was inclined at approximately 60º to the horizontal. The design called for a wall of exposed maximum height 5.6 m with a crest width of 3.0 m. With this constrained geometry, the line of maximum reinforcement tension 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 stratum. These were to be installed by pre-auguring into the partially decomposed quartzite back slope prior to anchor insertion.

The facing for the retaining wall utilised Anchor Vertica® Pro segmental blocks. Miragrid® 5XT geogrids were used as the reinforcements because it met the long-term design strength requirements and had good bending flexibility to enable it to pass easily around the galvanised steel pipe without attracting additional tensile stresses. The Miragrid® 5XT geogrids were placed across the top of the Anchor Vertica® Pro block layers and held in place with another layer of blocks. At the rear of the wall, the Miragrid® 5XT geogrids were passed around a 75 mm diameter galvanised pipe, following which, the pipe was connected to the installed anchors by connecting bolts. Granular reinforced fill was then placed and compacted to the correct block height, with the Miragrid® 5XT geogrids then returned to overlap the block surface. The Miragrid® 5XT geogrids were connected into the blocks at every second block level.

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® 5XT geogrid reinforcement and quality

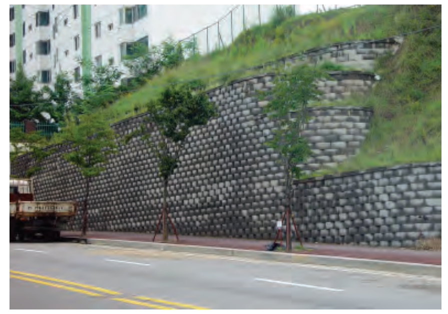


Cross section through the constrained reinforced soil wall

of the reinforcement connections to the facing blocks and anchors were not compromised. When completed, the retaining wall looked no different from that of a conventional geogrid reinforced segmental block retaining wall.

Surface condition of quartzite rock stratum

Various components of the combined reinforced soil and anchored wall



Completed constrained reinforced soil wall at Paju

Reinforced soil walls: Reinforced segmental block walls, Kanaka Durga Temple, Andhra Pradesh, India

Vijayawada, named as 'the city of victory', is strategically surrounded by the Bay of Bengal, the Rivers Krishna and Budameru, and the Indrakiladri Mountains. It is one of the fastest emerging commercial hubs in Andhra Pradesh State, primarily focusing on trading in agricultural products like rice paddy, sugarcane, red chili, etc.

Moreover, this city houses the temple for the Hindu deity Kanaka Durga, which is located on a hill shrine famously known as Kanaka Durga Hill. The temple is located at the top of the hill and is connected to the National Highway by a 1 km long hill road. The temple shrine attracts large numbers of pilgrims during the annual festival that falls in early autumn. The existing hill road has steep and narrow bends, and during the festive season these narrow bends get very congested seriously impeding traffic flow. The authorities decided to widen the hill road by extending the carriageway width and building parking spaces at specific locations along the hill road. Tall retaining structures were required to provide these platform areas.

The foundation conditions at the site consisted of a combination of a hard rock stratum and stiff to very stiff sandy silt soil overlying rock.

Site preparation works involved clearing existing buildings, uprooting trees and other vegetation. At some locations further slope cuttings were required to facilitate the installation of the geocomposite reinforcement to the required lengths. To optimize the cost, the locally available backfill obtained from the cut slopes was used in the construction for the reinforced fill and retained fill.

Widening of the hill road was an engineering challenge because of its close proximity to National Highway No. 5 which was always busy with traffic. The retaining wall structures required along the hill road alignment were a 42 m high, four-tiered reinforced soil retaining wall of 55 m in length and a 22 m high, two-tiered wall of 45 m in length. Each tier of the walls were set back 4 m from the tier below.

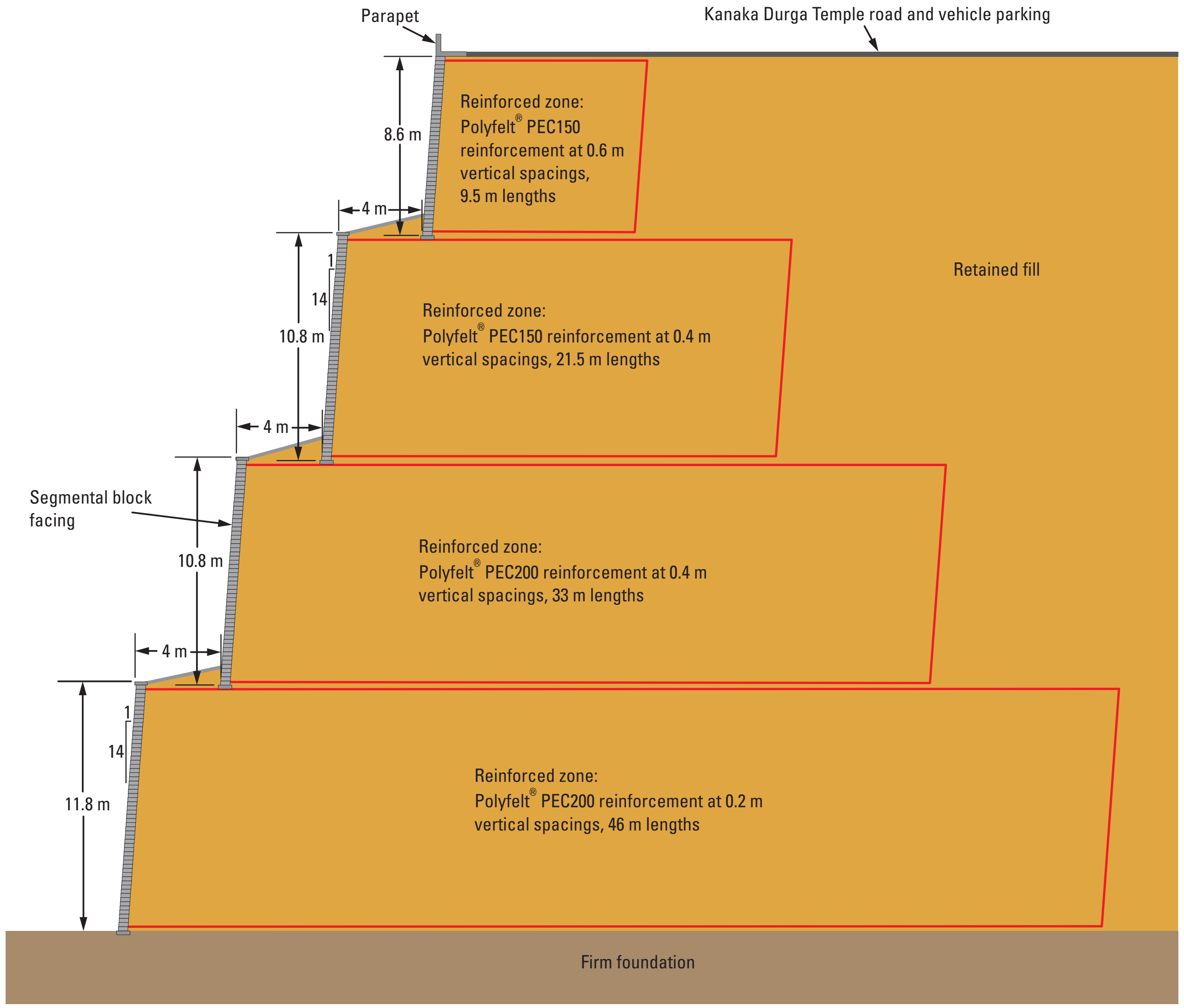
Reinforced segmental block walls were used as the retaining wall structures because of their cost, ability to tolerate differential settlements and speed of construction. The segmental block facing was made from wet-cast concrete of 35 MPa compressive strength. The block dimensions were 0.2 m in height, 0.45 m in width and 0.3 m in depth.

Polyfelt® PEC geocomposite reinforcements of varying tensile strengths were used as the layers in the reinforced fill. Polyfelt® PEC200, PEC150 and PEC100 were used at various levels up the height of the walls. Polyfelt® PEC geocomposite reinforcements have well-defined long term engineering properties which makes them ideal for use in reinforced soil applications.

A 0.5 m thick drainage layer was constructed at the rear of the segmental blocks to capture local seepage water. Also, a 0.5 m thick drainage blanket was located at the rear of retained fill where it abutted the existing bedrock stratum at the rear of the retaining walls.

To provide additional friction at the steep interface of the existing bedrock and the retained fill at the rear of the walls, 3 m long steel dowels were

Removal of loose material for site preparation works



Typical cross section through the 42 m high, four-tier reinforced segmental block wall

inserted 1.5 m into the bedrock stratum at 1.5 m spacings.

Client: Endowments Department, Government of Andhra Pradesh, India.

Consultant and contractor: GeoSol Associates Pvt Ltd, India.

Completed four-tier segmental block wall

Installing segmental block facing units and Polyfelt® PEC geocomposite reinforcement

Beginning the second tier of the segmental block wall

Widened platform area above retaining walls

Reinforced soil walls: Panipat Elevated Highway, Haryana, India

National Highway 1 (NH 1) runs 450 km from the town of Wagah 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 NH1. 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.

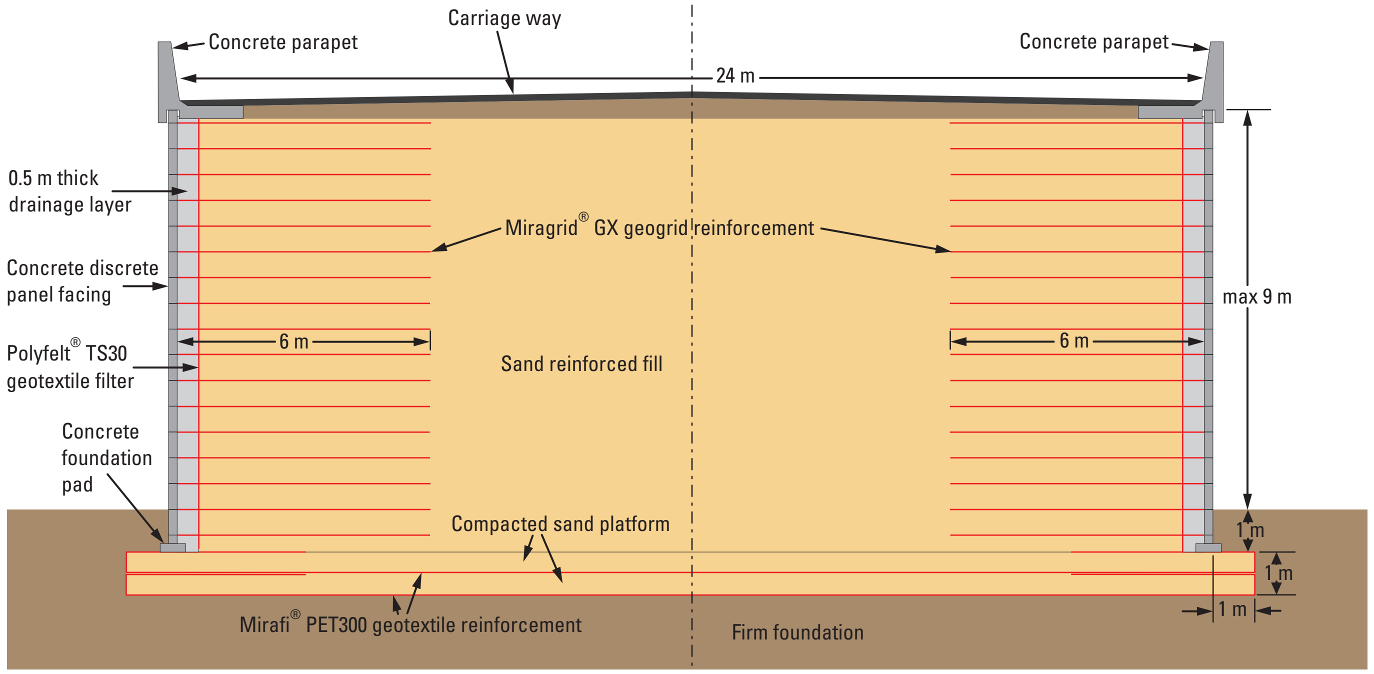
The Panipat Elevated Highway Project involved the upgrading of 10 km of highway along the NH1 corridor that passes through Panipat city. This upgrading was undertaken to ease the acute traffic congestion within Panipat city. Various highway structures were constructed, including a number of flyovers 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 Mirafi® PET300 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.

Installing the Mirafi® PET300 geotextile reinforcement in the compacted sand platform



Typical cross section through the reinforced soil walls

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 M35 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.

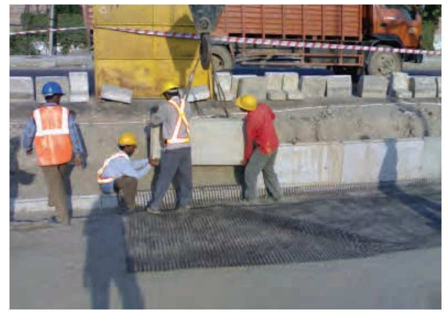
Approximately 14,000 m² of geogrid reinforced wall facing was used on this project. The completed elevated highway has opened to traffic and the stretch of retaining walls is today the longest along the NH1.

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 Pte. Ltd., New Delhi, India.



Installing the concrete discrete panel facing units

Granular drainage layer with Polyfelt® TS30 geotextile filter

Reinforced soil walls: Gabion faced retaining wall, La Réunion, France

On the Indian Ocean island of La Réunion the new coastal highway is 12 km in length and is set offshore from the existing coastal highway. This major infrastructure project supports a 30 m wide pavement area with 6 lanes dedicated to vehicles and for a future railway. Different geosynthetic solutions were applied in this project ranging from geotextile filters to geotextile and geogrid reinforcement within the various structures.

Historically, the existing coastal highway between Saint-Denis and La Possession had been regularly exposed to rockfalls and storm waves breaking over the highway making traffic movements very difficult, if not impossible.

The new, secure coastal highway alignment consists of alternating dykes and viaducts along its length. Its elevation profile is some 13 m above mean sea level (NGR) and in some locations is as high as NGR+30m at the viaduct abutments. The seabed along the new alignment ranges between NGR-5m and NGR-10m.

Due to its location along the coastline, the project faced important environmental issues such as the presence of marine mammals (humpback whales, dolphins), corals, endemic avifauna, etc. The protection of the natural environment was therefore a daily concern of all stakeholders both during construction and following completion of the new highway.

The geometry of the dyke was designed to adapt to the constraints of the site. Its location is far enough from the base of the cliff to not be impacted by mass landslides. Also, the crest of the dyke was designed to have an elevation to avoid the effect of storm and cyclone swells. On the seaward side of the dyke the foundation consists of 1 to 500 kg rockfill and this is topped by a 10 m to 15 m high reinforced concrete seawall. Protection against the waves is ensured by a concrete armour unit carapace ranging from 4 to 11 m³ in volume, below which is a secondary armour layer of rock between 0.2 and 3.8 tonnes.

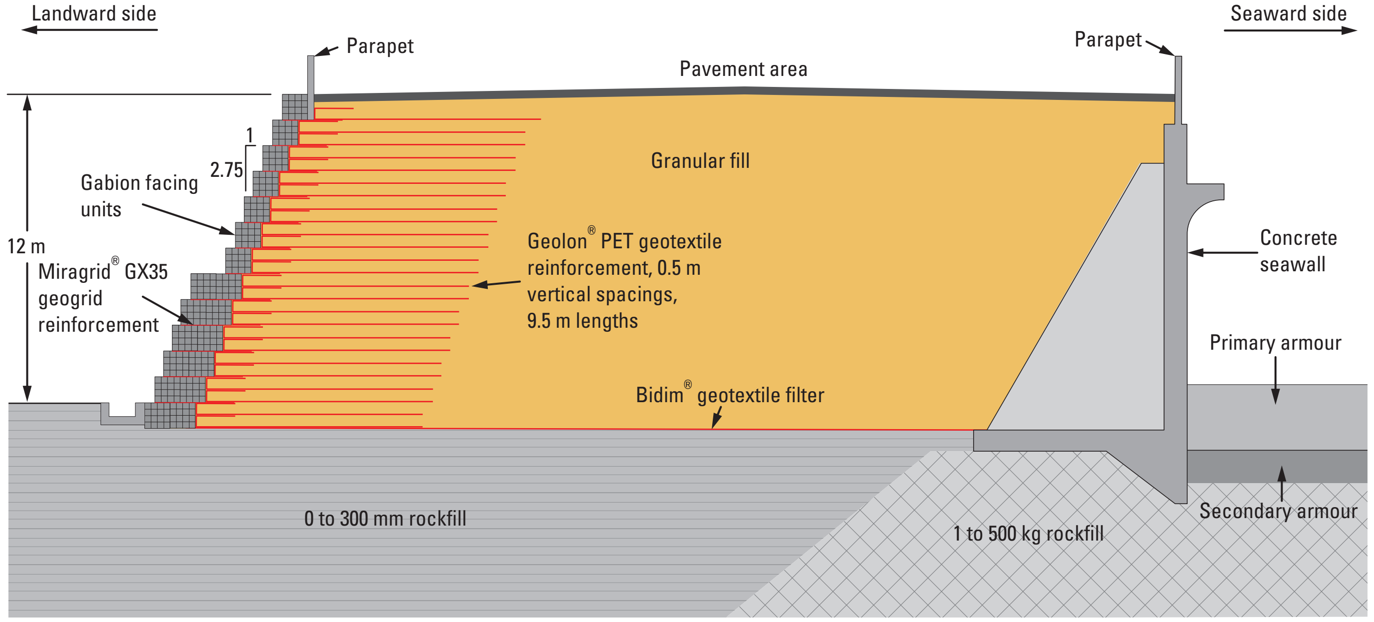
On the landward side of the dyke the foundation of 0 to 300 mm diameter rockfill. The upper part on this side of the dyke is supported by a stiffened retaining wall consisting of gabion facing units stepped back at an angle of 1:2.75 (70°) and is reinforced by layers of Geolon® PET geotextile reinforcement and Miragrid® GX35 geogrid reinforcement. This retaining wall acts to support the highway and also isolates any rocks in the event of rockfalls. This retaining wall varies in height between 9 and 15 m and is 6 km in total length.

The design of the reinforced soil retaining wall was carried out in accordance with the Eurocode approach considering ultimate and serviceability limit states. The performance criteria are set out in French Standards NF P 94-270 and NF G 38064. Additional design criteria such as seismic loadings and cyclonic swells were also considered.

Different grades of Geolon® PET geotextile reinforcement ranging in strengths from 100 kN/m to 400 kN/m

Rockfall blocking existing coastal highway

Storm waves breaking over existing coastal highway



Cross section through the reinforced dyke

at a constant vertical spacing of 0.5 m were used in the retaining structure. The length of the Geolon® PET geotextile reinforcement was kept constant at 9.5 m (≈0.8*H*) throughout the height of the wall. The Geolon® PET geotextile reinforcement layers were wrapped around behind the gabion facing units to fully support the stability of the reinforced face. To provide a structural bond between the gabion facing and the reinforced soil block a layer of Miragrid® GX35 geogrid reinforcement was inserted through adjacent gabion units and into the reinforced soil zone.

The gradation of the local material proposed for the reinforced fill was highly granular. Consequently, on site installation damage tests were carried out to confirm that the actual installation damage resistance of the Geolon® PET geotextile reinforcement grades met the requirements of the design. This was duly confirmed.

Installing Geolon® PET geotextile reinforcement

The use of the reinforced soil technique to design and construct the retaining wall structures along the new coastal highway enabled the construction program and budgetary requirements to be met in full.

Client: Région Réunion, France.

Consultant: Egis, Guyancourt, France.

Contractor: Consortium SBTPC - GTOI - Vinci Grands Travaux, France.

Compacting the granular fill

Partial completion of the gabion faced retaining wall

New coastal highway nearing completion

Reinforced soil walls: Structural walls, Anantara Qasr Al Sarab Desert Resort, Abu Dhabi, United Arab Emirates

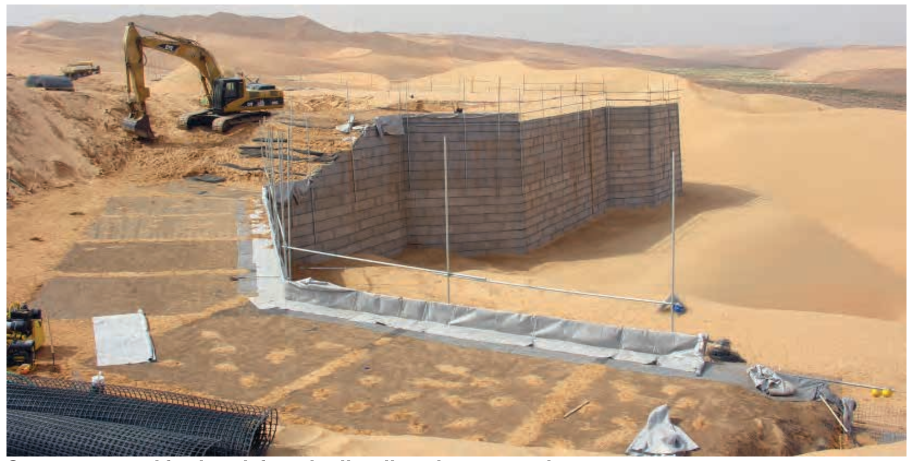
Anantara Qasr Al Sarab is an Arabic desert retreat about 90 minutes from Abu Dhabi airport, and 10 minutes from the Saudi Arabian border. The resort is situated in the Liwa desert's Empty Quarters or Rub'-al-Khali, the largest uninterrupted body of sand in the world. Sand dunes four times taller than 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 a very ambitious work program set by the client. Further, the location was extremely remote making transportation cost and time for materials prohibitively high. To save on cost and create maximum value a reinforced soil wall system, using locally available desert sand, was evaluated for all the retaining walls on site.

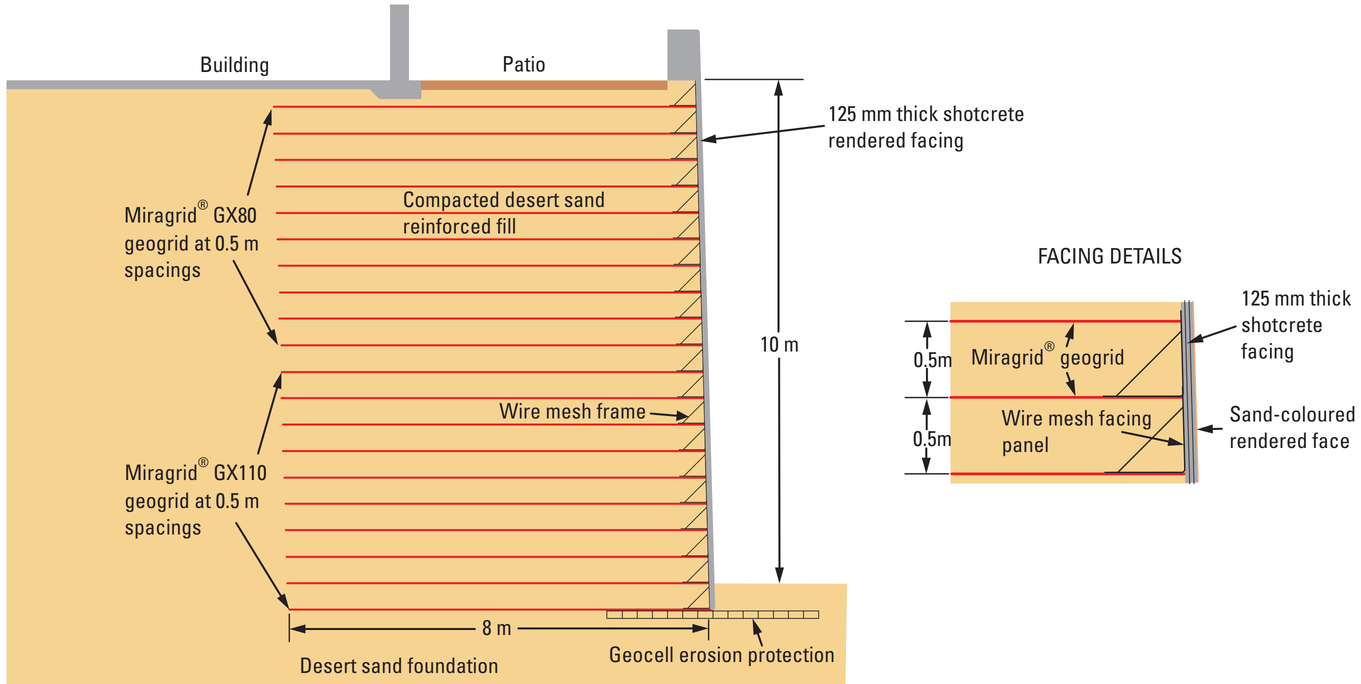
The reinforced soil wall system utilised Miragrid® GX geogrid reinforcements with a galvanised steel mesh facing to construct the retaining structures. These materials are light and easily transported to the remote site. In conjunction with readily available desert sand the contractor was able to construct the retaining walls extremely quickly. To retain the desert sand inside the wire mesh facing a layer of Polyfelt® TS60 geotextile separator was used. Subsequently, the geosynthetic reinforced retaining wall faces were shotcreted and rendered to give the appearance of an old Arabic fort.

The Engineers had to be mindful of some challenging design parameters while evaluating the design approach for this project. In addition to the local geotechnical conditions they were required to accommodate aggressive wind erosion and other challenging environmental factors such as seismic loadings and high static load design parameters. Being a hotel resort, it had complex architectural alignments, landscapes, underground utilities, swimming pools, plantations, etc.

After the approval of the basic design a trial wall was constructed to verify the insitu 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.



Components used for the reinforced soil walls under construction



Cross section through a 10 m high retaining wall, showing facing details

After the load was removed the plastic deformation was 3 mm.

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. The vertical spacings were maintained constant at 0.5 m with anchorage lengths ranging from 80% to 100% of the height of the walls.

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.

Retaining wall construction for the main resort earthworks

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.

Retaining wall structures for the resort villas

Completed retaining structures with another under construction

Completed shotcrete and colour rendered reinforced soil retaining walls blending into surrounding environment

Reinforced soil walls: Overpass abutments, M'Sila, Algeria

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 segmental blocks used were Atalus® 120 concrete blocks, which weigh 120 kg each. These blocks are cylindrical in shape, and allow soil to be placed within the cylindrical block facilitating vegetation growth on the face of the wall. Further, these blocks can be installed to a very low corner radius if required.

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.

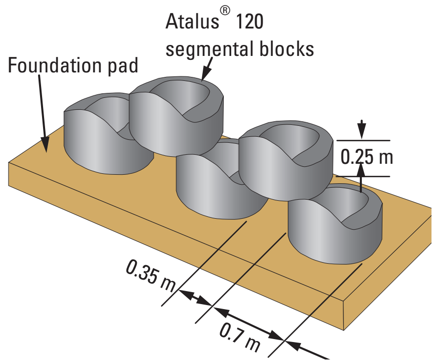
The design of the reinforced soil wall abutments utilised Bidim® PEC120 geocomposite reinforcement extending 7.5 m behind the wall face. Where appropriate, the geocomposite reinforcement was extended all the way through the abutments from one wall face to the other. Bidim® PEC120 geocomposite reinforcement 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.

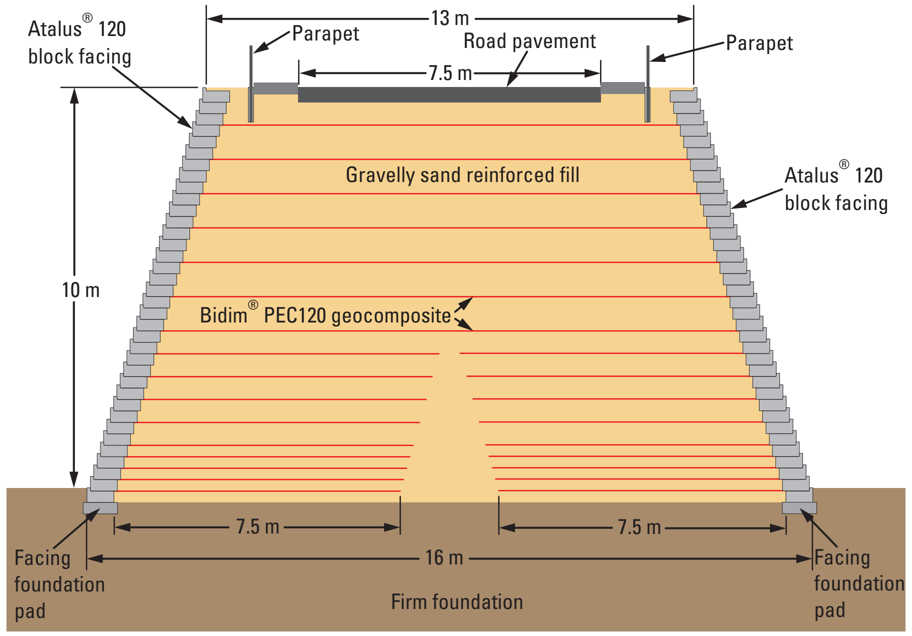
Consultant: Georoute Ingenierie, Champhol, France.

Contractor: Travomed, Algiers, Algeria.



Details of Atalus® 120 segmental blocks

Placement of Atalus® 120 segmental blocks for the wall facing



Cross section through abutment walls

Casting of Atalus® 120 segmental blocks on site

Placement of Bidim® PEC120 geocomposite reinforcement between segmental block layers

Wall construction prior to installation of the parapets

Segmental block walls nearing completion

Reinforced soil walls: Northern Interchange walls, Ouagadougou, Burkina Faso

Enabling the smooth movement of urban traffic is always a challenge anywhere in the world. This makes the development of city road infrastructure a key feature of efficient urban economic growth.

In the city of Ouagadougou, the capital city of Burkina Faso, the Northern Interchange project aimed to enable a good restructuring of the local road network and relieve traffic congestion. This interchange project was supported by the Burkinabe State in the framework of the SCADD (Strategy for Accelerated Growth and Sustainable Development). This interchange will improve traffic flows within Ouagadougou by extension of the Boulevard Circular through the Tanghin district in the direction of the international airport.

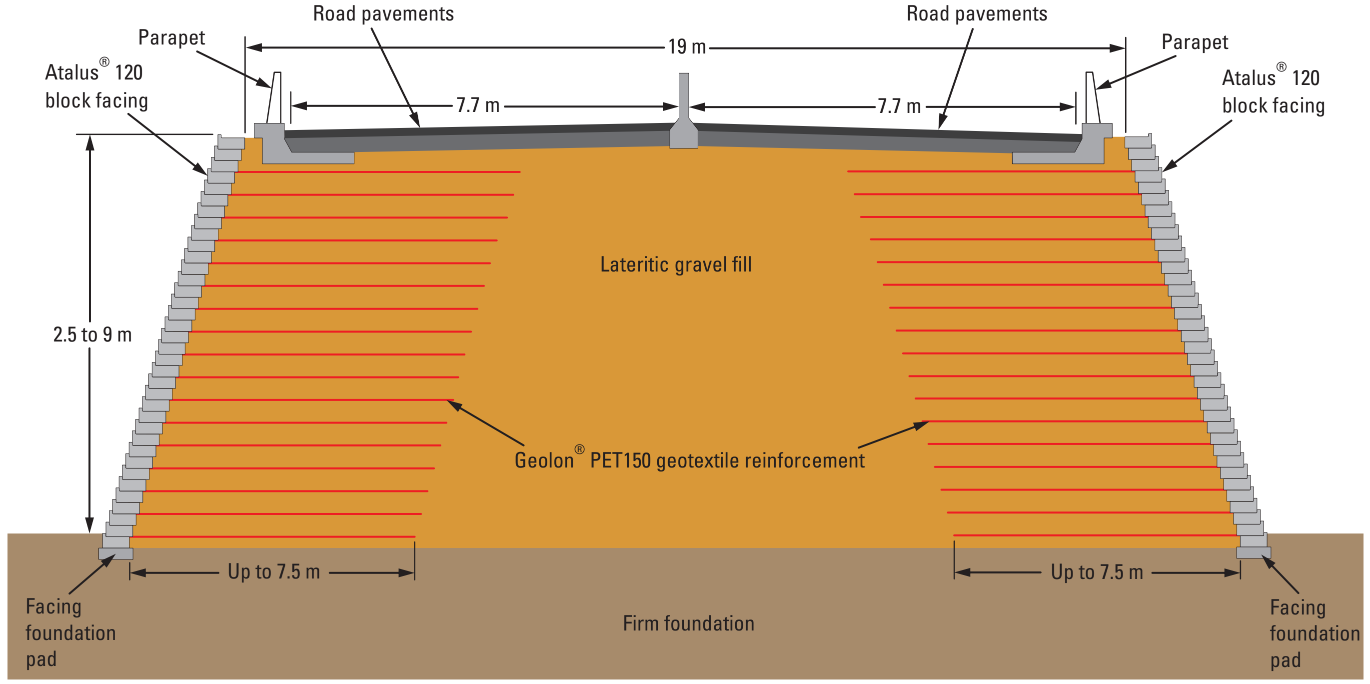
Saving considerable travel time for users, it will increase road safety and contribute to the improvement, well-being and productivity of the local inhabitants of the capital and the neighbouring communes of Signoghin, Baskuy, Nongremassom and Kamboisin that have a population of more than 800,000 inhabitants. The Northern interchange consists of a network of highway access roads that required the construction of embankments to meet road alignment conditions. Consequently, embankment heights ranged from 2.5 m to 9 m, and had to be constructed in an urban area of limited available space.

The site investigation and laboratory testing were carried out by the National Laboratory of Public Works (LNBTP) under the supervision of the Design Consultant. This involved all aspects of materials evaluation and the development of suitable design parameters. The fill material approved for the embankments was lateritic gravel commonly available in the area of Ouagadougou. Another point of importance was that the location of the interchange was over a river which resulted in further complexity regarding access of suitable aggregate deposits for construction of revetments and hydraulic concrete.

To steepen the embankment slopes and therefore limit the area of fill to reduce the cost of the earthworks the client chose to use retaining walls made of concrete modular blocks as the facing. The retaining wall system consisted of wet-cast Atalus® modular block units as the facing with layers of Geolon® PET150 geotextile reinforcement laid into the backfill. The vertical spacing of the Geolon® PET150 geotextile reinforcement was two block heights, being 0.5 m, and this was constant for the full height of the walls. The reinforced fill for the retaining walls was the same lateritic gravel material as was used for the main embankment fills, where a conservative friction angle of 35° was assumed for design.

The long term design strength of the geotextile reinforcement was calculated following the French NF G 38 064 Standard, accounting for the lifetime of the structure and the conditions on site. An average ambient temperature of 30°C was considered for the creep and durability reduction factors over the 100 year design life. The adoption of an ambient temperature of 30°C was considered appropriate for the ground conditions of Burkina Faso. Accounting for the various reinforcement partial reduction factors resulted in an overall reinforcement reduction factor of 3.3 for the 100 year design life. When this overall reduction factor was applied to the calculated reinforcement design loads Geolon® PET150 geotextile reinforcement satisfied all performance requirements at all levels in the retaining walls.

Placing Atalus® modular block facing units



Typical cross section through the interchange approach embankments

The retaining wall alignments at the interchange follow a succession of curves and reverse curves, and is connected vertically with various bridge structures. Within some curves it was necessary to change the wall slope angle from 70° to 76°. A solution was found to this complication by locally varying the setback distances of the block facing units. A range of wall geometries and alignments were required for this site. The longest wall length was 500 m and maximum height 9 m, with lowest radius of curvature being only 8 m. All these variations were handled efficiently by using the modular block and geotextile reinforcement system.

Once the retaining walls were completed select plant species were planted in the cavities of the modular block facing units.

The use of a modular block facing in combination with geotextile reinforcement solved the problems of embankment edge stability, complex horizontal alignments and aesthetics. The flexibility of the system allowed its adaptation to different horizontal curves and complex wall geometries. This technique is fully adaptable to the constraints of the environment and also makes it possible to use different soil types as the reinforced fill component for the retaining wall system.

Client: MIDT - DGOA, Burkina Faso.

Consultant: AGEIM, Ouagadougou, Burkina Faso.

Contractor: Sogea Satom, Ouagadougou, Birkina Faso.

Block facing with Geolon® PET150 geotextile reinforcement

Curved retaining wall system at interchange

One of the completed retaining walls

Reinforced soil walls: Flyover abutments, Dakar, Senegal

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.

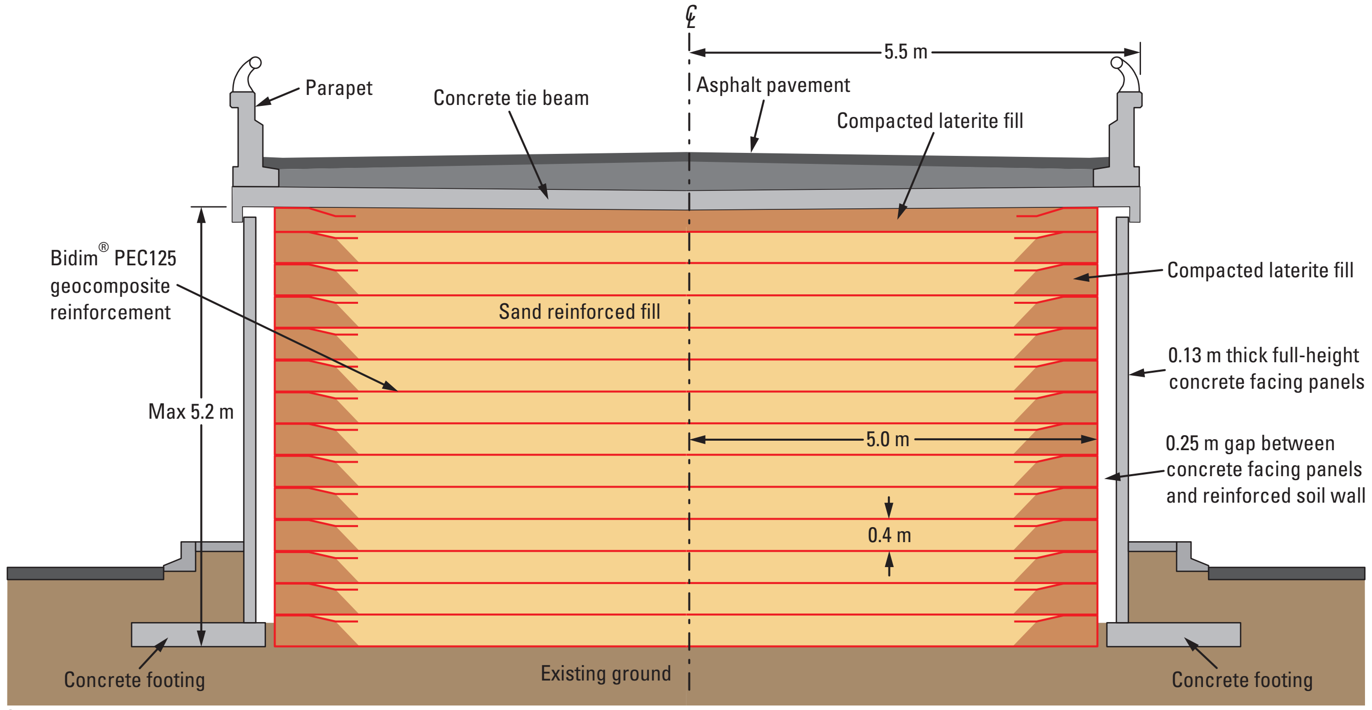
The geosynthetic reinforcement used in the reinforced soil walls was Bidim® PEC125 geocomposite reinforcement. Bidim® PEC125 geocomposite reinforcement consists of high modulus polyester fibres manufactured into a composite structure, and has an initial tensile strength of 125 kN/m. The geocomposite reinforcement was placed at 0.4 m vertical spacings throughout the height of the abutments.

To construct the reinforced soil walls to a vertical face, external steel shutters were used to confine the placement and compaction of the laterite fill. When the reinforced fill lift was completed 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 the consolidation period, 0.13 m thick, full-height, prefabricated, concrete panels were installed. These sat on a concrete footing, and were positioned at a 0.25 m spacing away from the reinforced soil wall face to ensure no horizontal stresses were exerted on the panels. The panels were restrained at the top inside a concrete tie beam.

Details of the removable steel shutter used to construct the vertical abutment face



Cross section through the reinforced soil abutment at maximum height

To monitor the performance of the abutment walls the Geodetect® fibre-optic strain monitoring system was incorporated with the geocomposite reinforcement. The monitoring results showed that at the end of abutment construction horizontal strains varied from 0.3% to 1.0% which are very low. 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.

Placing Geodetect® fibre-optic strain measurement strips in the retaining wall

Compacting laterite fill against the removable steel shutter to construct the vertical reinforced soil abutment face

Placing full-height prefabricated concrete panels to create the exterior face of the abutment walls

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 m^2 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 downwards 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 m^3 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.

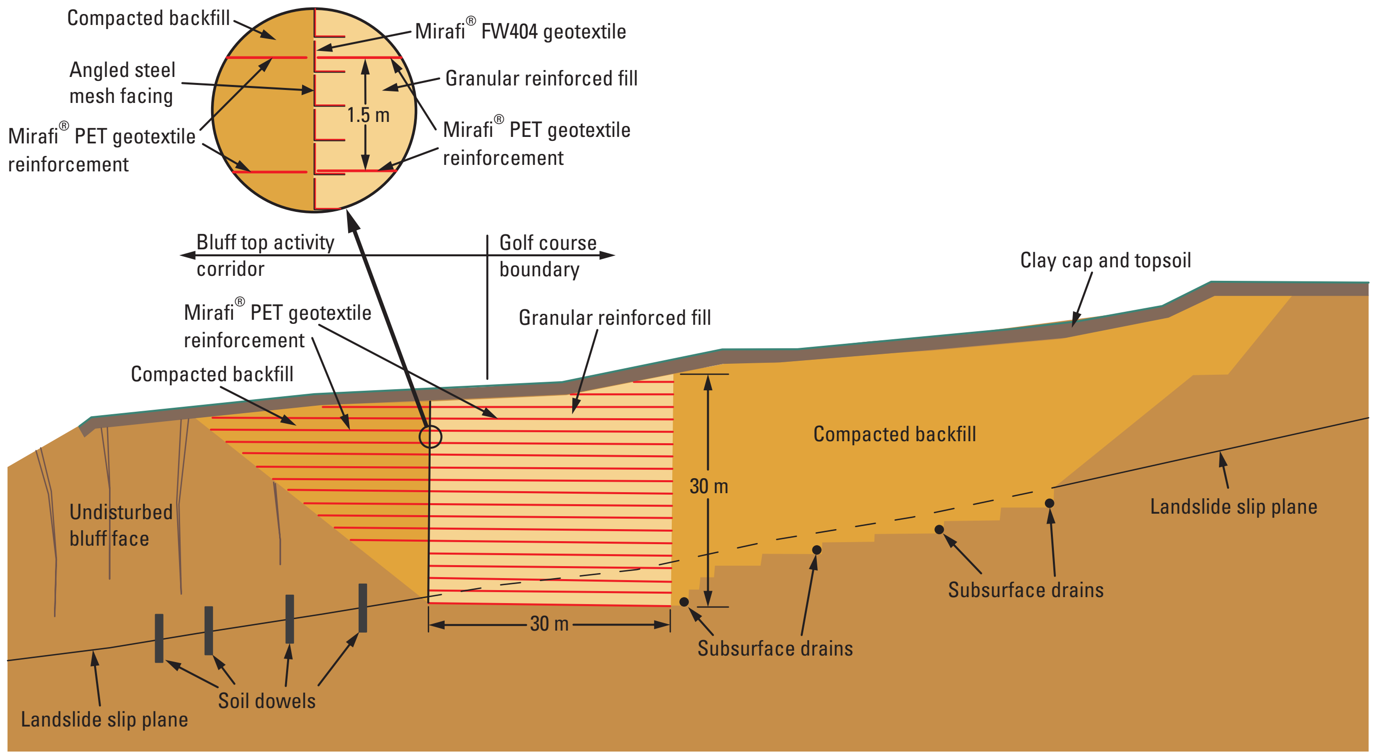
A number of design options were investigated to restore the landslide. The best of these involved partial removal and rebuilding of the landward portion of the landslide as this would achieve the intended purposes, have the least alteration to the natural landform, and be the most feasible from a geotechnical 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 6 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 slide area (over 1.3 million m^3 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.

Landslide through the 18th fairway and green



Section through the landslide restoration showing the details of the shear-key wall

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 over 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.

Excavation to the base of the shear-key wall

Placing and compacting the granular reinforced fill for the shear-key wall

Completed 18th fairway and green across the top of the restored landslide

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