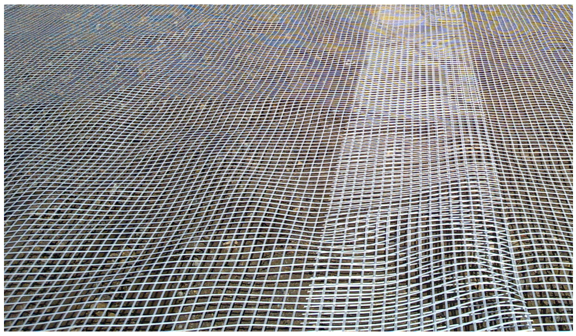
3rd International Conference on
Transportation Geotechnics

4-7 September 2016 | Guimarães | Portugal

Workshop 1: Geosynthetics in Transportation Geotechnics

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Proceedings

University of Minho
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Workshop 1: Geosynthetics in Transportation Geotechnics

EXTENDED ABSTRACTS BOOK

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Preface

Geosynthetic-reinforced soil structures, the use of geosynthetics in pavement and related engineering are now one of the indispensible components in transportation geotechnics for roads and railways. Now it is the time to collect and summarize its state-of-the-art and discuss on the perspectives of the use of geosynthetics for transportation infrastructures (roads, airfields and railways).

The main goals of the workshop are:

- State-of-the-art of the use of geosynthetics in transportation geotechnics.
- Theory and research of geosynthetics engineering for transportation engineering.
- Issues in practice.
- Perspective.

This book contains the extended abstracts of the oral presentations and was prepared from the input files supplied by the authors. The order of the extended abstracts follows the definitive programme of the workshop. The author index is given at the end.

Guimarães, Portugal

Fumio Tatsuoka
Jorge Zornberg
José Machado do Vale
José Neves

Table of contents

Research and Construction of Geosynthetic-Reinforced Soil Integral Bridges

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1 Overview

Geosynthetic-reinforced soil (GRS) integral bridge was developed to overcome several inherent serious problems with conventional type bridges comprising a simple-supported girder (or multiple girders) supported via bearings typically by RC abutments retaining unreinforced backfill (and a pier or piers for multiple girders). The problems include: 1) relatively high construction and maintenance costs with relatively long construction time resulting from the use of bearings; massive abutment structures usually supported by piles; 2) bumps immediately behind the abutments; and 3) a relatively low stability of the girders supported by roller bearings and the approach embankment against seismic, floods and tsunami loads.

GRS integral bridge was developed by extending the technology of GRS retaining wall having staged-constructed full-height rigid (FHR) facing, developed in the mid-1980s (Figure 1). This type of GRS retaining walls have been constructed at more than 1,100 sites (Figure 2a) and for a total wall length of more than 160 km (Figure 2b), mainly for railways including high-speed train lines (Shinkansen in Japanese).

For a GRS integral bridge, a pair of GRS walls (and an intermediate pier or piers if necessary for a long span) are first constructed (Figure 3). After the deformation of the supporting ground and the backfill of the GRS walls has taken place sufficiently, steel-reinforced full-height-rigid (FHR) facings are constructed by casting-in-place concrete on the wall face wrapped-around with the geogrid reinforcement. Finally a continuous girder is constructed with both ends integrated to the top of the FHR facings. The girder is also connected to the top of an intermediate pier, or piers, if constructed. The background and history of the development of GRS integral bridge is described. The first four case histories are the one completed in 2012 for a new high-speed train line and the other three completed in 2014 to restore a railway damaged by a great tsunami of the 2011 Great East Japan Earthquake, are reported. Figure 4 shows the one constructed at Haipe.

2 Main Findings

1. GRS integral bridge was developed by extending the technology of GRS retaining wall having staged-constructed full-height rigid (FHR) facing.
2. Compared with the conventional type bridge comprising a simple-supported girder (or girders), GRS integral bridge is much more cost-effective with a reduced period of construction and its performance is much higher with negligible bumps behind the facing and a high stability during long-term service and against severe earthquakes, floods and tsunamis.
3. These characteristic features can be attributed to the staged construction of FHR facing that is firmly connected to the geogrid layers and the structural integration of a continuous girder to the top of the FHR facings.
4. For these reasons 2 and 3, GRS integral bridge is relevant for railways and roads at many places.

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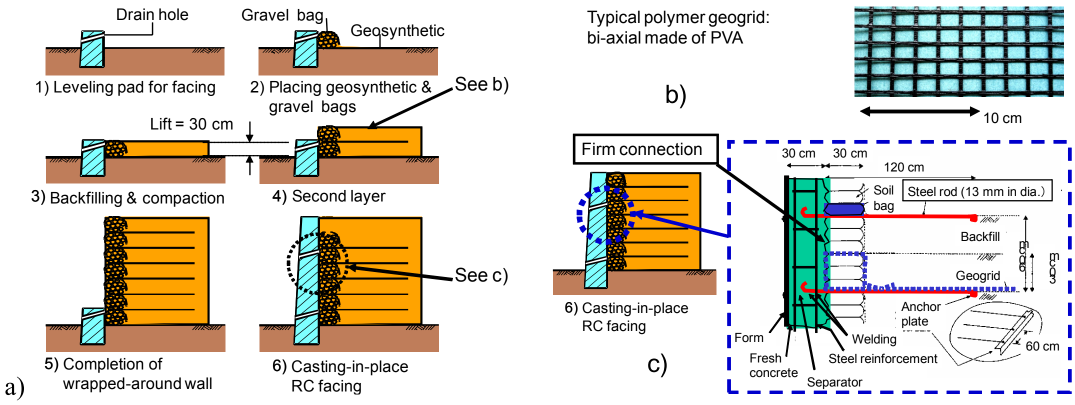

Figure 1: GRS RW with FHR facing: a) staged construction; b) a typical geogrid type; and c) facing construction at stage 6

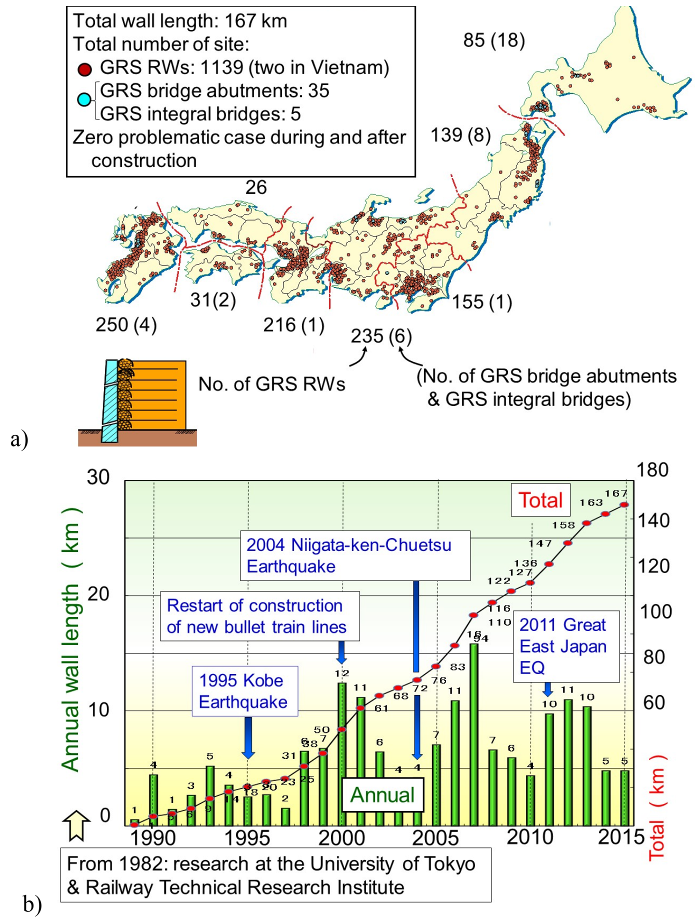

Figure 2: a) Construction sites; and b) history of GRS structures including RWs with a staged-constructed FHR facing, GRS abutments and GRS integral bridges (as of June 2016)

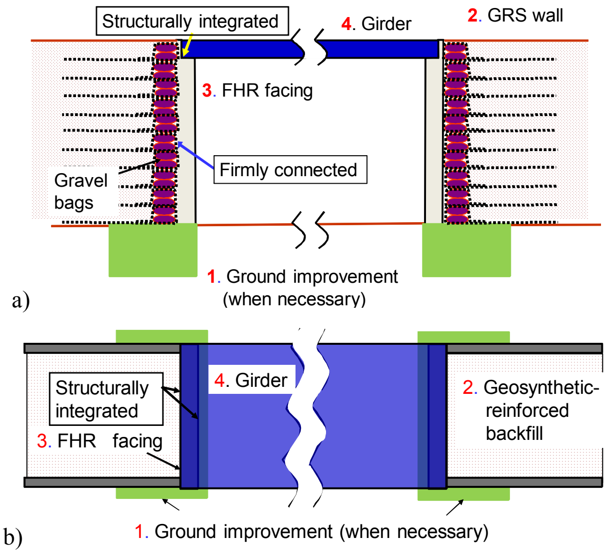

Figure 3: Structure of GRS integral bridge (the numbers denote the construction steps): a) elevation; and b) plan

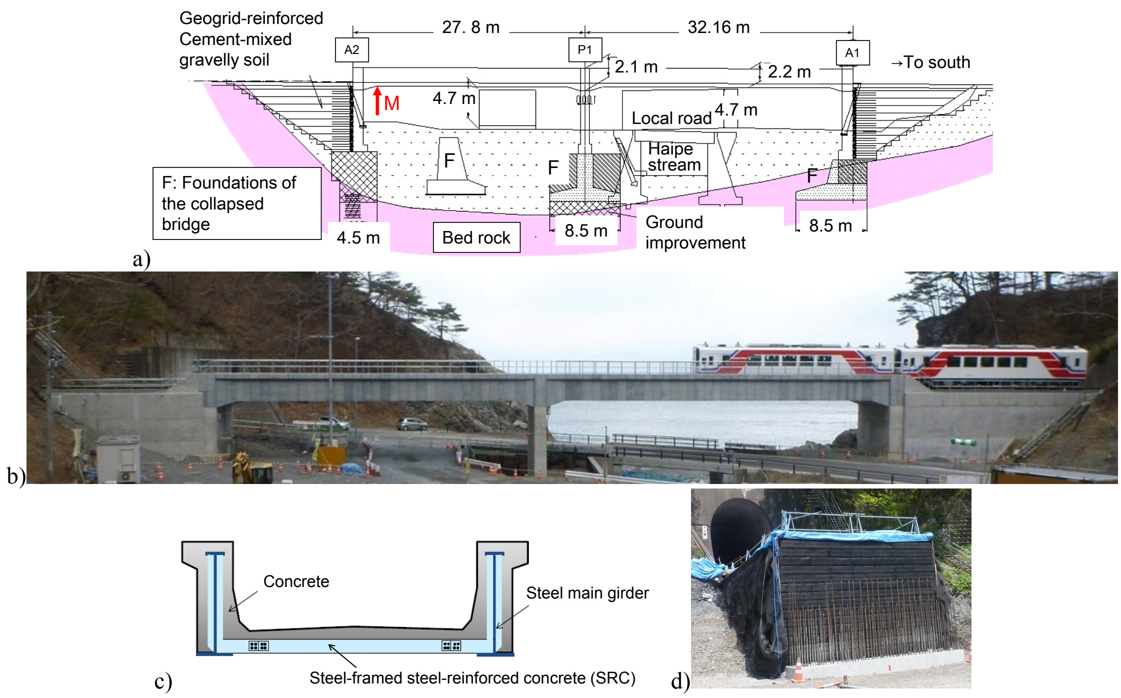

Figure 4: Haipe-sawa bridge, Sanriku Railway: a) structure of GRS integral bridge seen from the inland: b) completed GRS integral bridge (6th April 2014); c) typical cross-section of the through girder; and d) the north abutment immediately before constructing FHR facing (22 May 2013)

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The first GRS integral bridge with FHR facing in Europe – experiences from design and construction

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1 Introduction

The use of geosynthetic reinforced soil (GRS) technology has become common practice in the design of retaining structures and embankments for infrastructure projects all over the world. This is due to cost savings, the simple and rapid construction technique, the reduced construction time, the reduced environmental effects, good seismic performance, and some other factors. Also, a number of studies have been conducted over the last two decades with the aim of investigating the size of the surcharge load which can be applied to the top of GRS structures, and the applicability of GRS technology to the construction of bridge supporting structures such as bridge piers and abutments (Tatsuoka et al., 1997, 2009; Adams et al., 2002; Wu et al., 2006). Particularly in the case of soft foundation ground, there are two main reasons to use GRS abutments instead of traditional piles: (1) to reduce the overall costs of the project, and (2) to reduce or potentially eliminate the "bridge bumps" which arises from differential settlement between approach embankments and traditionally reinforced-concrete (RC) bridge abutments supported by piles (Helwany et al., 2003). The same reasons were relevant to the design of the new bridge over the Pavlovski potok stream in the village of Žerovinci (in north-eastern Slovenia), which is presented in this paper. The other two important reasons are: (1) the very short deadlines defined by the investor for the opening of the bridge for traffic within a period of two months at the end of 2014 (this included both the design and the construction of the bridge), and (2) a deep layer of soft foundation soil, which encouraged the authors to propose the use of GRS integrated bridge abutments.

Pioneering work in the use of integrated GRS bridge abutments, although with some basic differences, was performed in Japan by Tatsuoka and his colleagues (Tatsuoka et al., 2009), and in the USA by the FHWA (Adams et al., 2010). Tatsuoka et al. (2009) suggested the use of a continuous deck with both of its ends fully structurally integrated into the top of a pair of full-height rigid (FHR) facings of GRS walls (described later in this paper). On the other hand, the FHWA (Adams et al., 2010) developed a bridge system in which a single-span simply-supported deck is placed, without structural integration, on top of the GRS, immediately behind the facings. The latter typically consist of modular blocks. The advantages of the GRS integrated bridge when the deck is fully structurally integrated onto the top of a pair of FHR facings, as presented by Tatsuoka et al. (2009), are as follows: (1) the construction and maintenance of bearings becomes unnecessary, (2) the reinforced-concrete deck becomes more slender due to a significant reduction in the bending moment resulting from the flexural resistance at the connection between the deck and the facing, (3) seismic stability is increased significantly due to the increased structural integrity and reduced weight of the deck, and (4) due to the greater structural integrity and smaller cross-section of the deck, the bridge's resistance to tsunami or similar water flows during floods is significantly increased. It should be noted that the GRS integrated bridge, as presented by the FHWA, does not benefit from any reduction in the mid-span bending moment, which is a feature of frame-type bridge structures having deck-facing structural integrity, as proposed by Tatsuoka et al. The FHWA type of GRS integrated bridge is constructed as a single simply-supported deck supported by a pair of abutments, and is therefore more appropriate for short span bridges. In any case both systems have been successfully implemented into practice over the last few years.

2 Bridge across the Pavlovski potok stream in Žerovinci

The bridge across the Pavlovski potok stream in the village of Žerovinci in north-eastern Slovenia is a part of the investment into the modernisation of the Pragersko – Hodoš railway-line, which is the biggest investment in the infrastructure in Slovenia at the moment. In order to maintain roads and the local traffic infrastructure, the Municipality of Ormož decided to invest in the construction of a new bridge on the roadway designated 802501

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across the Pavlovski potok stream. In a case of high water levels the existing bridge was constantly flooded due to the insufficient water flow capacity of its box-shaped culvert.

No preliminary geological-geotechnical investigations of the ground had been performed at the exact location of the bridge. Instead, all the available data and results from investigations performed for the design of another bridge located 50 m upstream and supported by deep foundations using piles were taken into account. Table 1 summarizes the geological-geotechnical data, including the results of standard penetration tests, which were obtained at the site of the upstream bridge.

Due to the low shear strength and compressibility of the sub-soil as presented by these geotechnical conditions, a deep piled foundation would have been used if a traditional bridge were to have been constructed. Due to time and cost limitations, however, it was decided to employ GRS bridge abutments with FHR facings that are stage-constructed in order to ensure good external stability while simultaneously minimizing the potential negative effects of significant ground settlement after completion of the bridge.

Table 1: Geological-geotechnical data obtained from investigations that were performed for a bridge that was located 50 m upstream

Depth [m]	Description	Soil properties
0.0 – 0.5	sandy gravel	
0.5 – 3.0	sandy clay with inclusions of gravel and sand	$(N_1)_{60}$=6
3.0 – 5.0	clayey and silty sand	$(N_1)_{60}$=8, c'= 1.6 kPa, φ'=25.7°, w=33.5%, I_p=14.3
5.0 – 8.0	silty sand	$(N_1)_{60}$=12, w=29.1%, I_p=10.4%
8.0 – 11.0	decayed stratified marl	$(N_1)_{60}$=24
11.0 – 17.0	sandy marl	$(N_1)_{60}$=36
17.0 – 23.3	sandy-silty clay	$(N_1)_{60}$=32
23.3 – 26.3	sandy marl - solid	

Water level depth: 2.7 m

The existing bridge was demolished and replaced by a new one. It was proposed that a reinforced concrete slab, integrated onto a pair of geosynthetic reinforced soil bridge abutments, should be built. The distance between the facings of the abutments is 5.50 m. Thus, the new bridge consists of an RC simply-supported slab, supported by a pair of geosynthetic reinforced soil abutments, as shown in Figure 1. This type of construction is effective for the reduction of construction time and costs, while simultaneously eliminating the need for the use of heavy construction machinery as well as alleviating the bumps in front of and behind the bridge, which usually occur due to differential settlement. A detailed view showing how the bridge deck rests on the GRS soil abutments is shown in Figure 2.

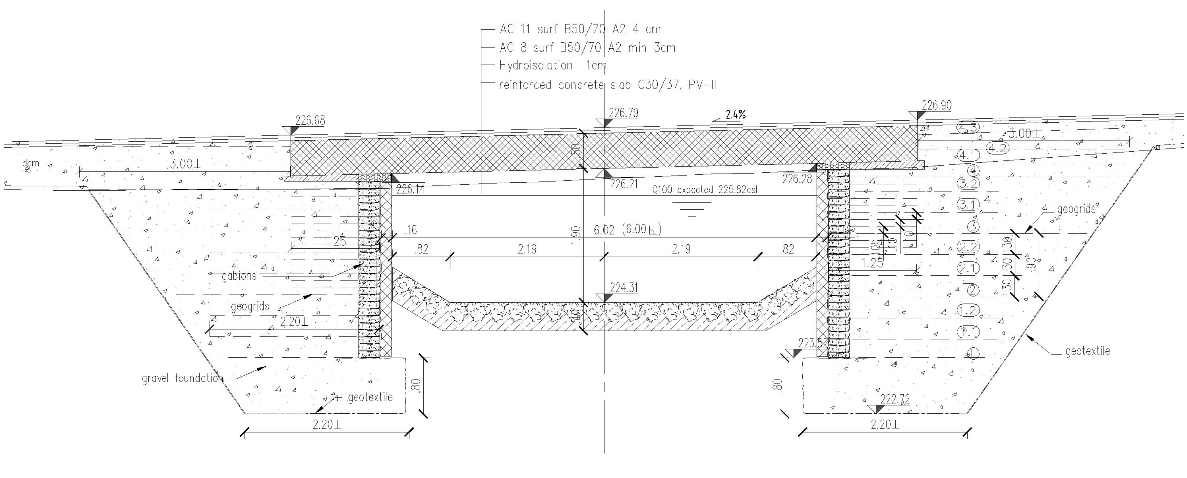

Figure 1: Longitudinal cross-section of newly designed bridge supported by reinforced soil abutments

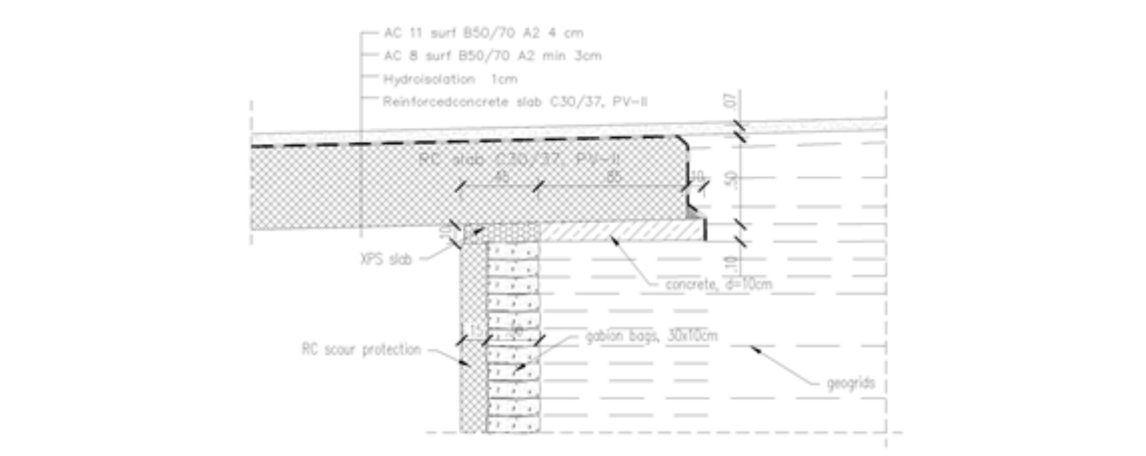

Figure 2: Detail showing how the bridge deck rests on one of the geosynthetic reinforced soil abutments

3 Design and construction

The foundations for the abutments were built first, using well-compacted gravelly soil, which was wrapped-around with a layer of geosynthetic. Afterwards, the staged construction of GRS retaining walls (RW) with full height rigid (FHR) facings has proceeded. This type of construction has a long tradition in Japan (Tatsuoka et al., 1997), but not elsewhere in the world. As shown in Figure 1, after site preparation (temporary excavation works), and formation of foundations, soil bags (gabions) were placed at the shoulder of each soil layer during the construction process (Figure 3). The soil bags function as a temporary but stable facing structure during the construction works, since they resist the earth pressure generated by the compaction works and by further backfilling at higher levels. After sufficient deformation of the subsoil and the backfill has occurred during the construction of the geosynthetic-reinforced backfill, the full height rigid (FHR) facings were constructed by means of cast-in-situ concrete (Figure 4) in the space between the formwork and the wall face which is wrapped-around with geosynthetic (geogrid) reinforcement.

As a high connection strength between the reinforcement layers and the FHR facing is crucial for proper performance of GRS RWs with FHR facings (Tatsuoka et al., 1997), authors of the bridge across the Pavlovski potok decided not to take a risk with the first structurally fully GRS integrated bridge in this part of the world. They therefore decided to construct the bridge system with the bridge deck placed on top of the GRS, immediately behind the FHR facings, i.e. to construct the bridge as a single-span simply-supported beam, without using a pair of bearings, i.e. it is supported by the two GRS abutments. Thus the bridge across the Pavlovski potok stream, presented in this paper, combines the two approaches, the one used in Japan and the other proposed by the FHWA, for GRS integrated bridge design.

Figure 3: Construction of the GRS abutments by placing gravel bags on the shoulder of each layer and compaction of the backfill

Figure 4: Formwork for cast-in-situ concrete of full height rigid (FHR) facings at the wall face

Both GRS bridge abutments were built within a period of less than 10 days due to the simplicity of the construction processes involved (Figure 3). Since the bridge crossed a water channel, the foundations of the reinforced concrete facings were located at a depth of 150 cm in order to prevent scouring beneath the GRS mass. The ends of the horizontal layers of the geogrid reinforcement were rigidly connected to the vertical steel reinforcement inside the facing structure by means of additional strips (anchors). In order to provide scour protection, four wing walls were extended into a rip-rap structure at the bottom of the water channel. This structure consisted of rocks placed in concrete, and provided a control channel for the stream, extending for another 5.0 m from each side of the abutments. The bridge deck was placed directly on the top of the geosynthetic-reinforced backfill of the bridge abutments using a thin layer of embedded concrete (Figure 5).

Figure 5: Formwork for cast-in-situ concrete of full height rigid (FHR) facings at the wall face

4 Results of observations and field monitoring

4.1 Foundations

Considering the geological structure of the ground at the construction site, in case of conventional reinforced-concrete bridge abutments, deep piled foundations using piles with a diameter of 100 cm and a length of 24 m would have been needed, as in the case of the nearby railway bridge, which is located 50 m upstream. The geosynthetic reinforced soil technology significantly reduced the construction costs and time.

4.2 Concreting works

Piles, pile caps, steel-reinforced concrete abutments, wing walls of RC retaining structures, and approach slabs become unnecessary when the described GRS technology is used, which significantly reduces the amount of concrete needed. Based on data obtained from the nearby newly-built railway bridge, which had steel-reinforced concrete abutments founded on deep piled foundations, the quantity of concrete works needed when constructing such conventional bridge abutments was compared to the case of the integrated GRS bridge across the Pavlovski potok stream. The results showed that, in the case of steel-reinforced concrete abutments, nearly 120 m3 of extra concrete would have been needed in comparison with the geosynthetic reinforced soil abutments (Table 2).

Since less concrete was needed, less formwork, too, was needed. However, despite the fact that only single-sided formwork was needed to construct the facing structure of the GRS abutments, their implementation was rather complex since ground with good load bearing capacity was not available to support the formwork.

In the design, the GRS facings were considered mostly as a scour protection measure. As the design was based on the characteristics of a simply-supported beam, structural integrity between the bridge deck and the facings was not necessary. The aim was to not increase the vertical load acting on the abutments due to the self-weight of the facing structure. For this reason a minimum thickness of the facing structure, equal to 15 cm, was decided upon. Also, minimum structural reinforcement was installed in the facings in order to ensure their rigidity and to prevent the occurrence of any kind of cracks in the concrete (Figure 4).

In order to ensure uniform thickness of the facing structure (15 cm) throughout the whole height of the abutment, the gabion bags have to be placed in their outer vertical position very precisely, with only small deviations. This is because, when vibrating the cast-in-situ concrete, additional problems can arise, in the case of relatively thin RC facing structures, if the front face of the gabion bags is not precisely achieved. In order to avoid such problems, in the presented case self-compacting concrete was used. It should be noted that this problem becomes less serious when the thickness of the FHR facings is increased, e.g. to 30 cm, which corresponds to the practice in Japan.

Table 2: Comparison of the amounts of concrete needed for bridge abutments at Žerovinci in case of GRS bridge abutments and reinforced concrete abutments

Element	Amounts of concrete needed [m^3]		Difference	
	RC abutments	GRS abutments	[m^3]	[%]
Piles (D=100cm L=24m)	75	-	75	-100
Pile caps (120/120cm)	23	-	23	-100
Abutments (d=50cm)	21	9	12	-57.1
Wing walls (d=30cm)	7	5	2	-28.5
Approach slabs	12	-	12	-100
Superstructure	35.5	42	-6.5	18.3
Total	**173.5**	**56**	**117.5**	**-67.7**

4.3 Static load design

Since the bridge deck is constructed as a slab which is simply-supported by a pair of GRS abutments, the internal mid-span bending moment line is much greater than in the case of a frame structure. Thus more reinforcement is needed in the lower zone, and less in the upper zone. Also, a longer RC slab has to be provided in order to provide the necessary bearing area.

4.4 Construction time

The time needed to construct the GRS abutments was significantly reduced due to the simple construction processes and techniques in comparison with steel-reinforced concrete abutments. GRS bridge abutments can be constructed within a couple of weeks without being influenced by outside weather conditions. Taking into account the provisions of the Slovenian legislation, concrete bridge decks have to be constructed in the conventional way using formwork, steel-reinforcement, and cast-in-situ concrete. In the case of high water levels, pumping from excavated areas may also be needed when following this conventional construction method. The use of precast concrete (PC) bridge decks is not allowed in Slovenia. However, the use of such PC bridge decks would significantly reduce the construction time in this kind of project, with a short span bridge.

5 Conclusions

The first GRS integrated bridge with FHR facings in Europe was constructed across the Pavlovski potok stream in the village of Žerovinci at the end of 2014. Its design was accompanied by very short deadlines and a thick layer of soft foundation soil, where deep pile foundations would become necessary in the case of the conventional type of abutments, using steel-reinforced concrete. Due to the lack of previous experience with the staged construction of GRS RW with FHR facings, the authors decided to combine this technology, which is widely used in Japan, with the GRS integrated bridge system construction approach presented by the FHWA (USA). Thus the bridge system consists of a bridge deck which is placed on top of the GRS, immediately behind the FHR facings, i.e. it acts as a simply-supported beam with its ends supported on a pair of GRS abutments. The experience gained from the design and construction of the bridge presented in this paper indicates the many significant advantages of GRS bridge abutments compared to conventional steel-reinforced concrete cantilevered abutments. The presented solution is beneficial particularly for short span bridges that need to be designed and built in a very short time.

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Modelling geogrid-reinforced railway ballast using the discrete element method

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1 Introduction

Geogrids have been increasingly used in ballasted track substructure. The primary benefit of the geogrid is providing additional confinement and strengthening the ballast due to interlocking of surrounding ballast aggregates with the apertures of the geogrid. As a result, this significantly decreases lateral spreading, and leads to extension of the track maintenance cycles. Subjected to repeated train loading, ballast becomes fouled, which adversely affects the strength and deformation of ballasted tracks. If ballast is badly fouled, the clogging will reduce its ability to drain properly, which causes track instability associated with substantial deformation.

Current literature on the interface behaviour of geogrid-ballast subjected to monotonic and cyclic loadings is still limited both in experimental study and numerical simulation, particularly when ballast becomes fouled (Tutumluer et al. 2011; McDowell et al. 2012; Ngo et al. 2016). The interface behaviour between the geogrid and the ballast has not been examined in detail or incorporated into Australian ballasted rail track designs. This paper presents major results of tests carried out at the University of Wollongong, where static and cyclic testing of ballast were conducted using large-scale apparatus. In this study, large-scale direct shear tests were carried out for fresh and coal-fouled ballast with and without the inclusion of geogrids to study the interface behaviour of ballast and geogrid. A novel Track Process Simulation Apparatus (TPSA) was used for fouled ballast to examine the cyclic response of geogrid-reinforced fouled ballast. Numerical simulations using the discrete element method (DEM) were also implemented to model the interface behaviour of geogrid reinforced ballast subjected to shear loading.

2 Laboratory Studies

A series of large-scale direct shear tests for fresh and coal-fouled ballast reinforced by the geogrid were carried out for fresh and fouled ballast (Indraratna et al. 2011). The results indicated that geogrid increases the shear strength and apparent angle of shearing resistance. However, when ballast was fouled, the benefits of geogrid reinforcement decreased in proportion to the increasing level of fouling. Cyclic tests for coal-fouled ballast were conducted using the TPSA (800 mm long, 600 mm wide and 600 mm high), fabricated to simulate realistic track conditions (Indraratna et al. 2013). The geogrid was placed on the top of the sub-ballast, followed by 300 mm thick ballast layer compacted to field unit weight of 15.5 kN/m^3. The cyclic load was then applied through a servo hydraulic actuator with a maximum pressure of 420 kPa at a frequency of 15 Hz, representing 20 tonnes/axle train loading travelling at an approximate speed of 80 km/h. All tests were conducted at a frequency of 15 Hz, and tested up to 500,000 load cycles. Laboratory results showed that the inclusion of geogrid significantly decreases the vertical and lateral deformations of fresh and fouled ballast. Indeed, when ballast grains are compacted over the geogrid, they partially penetrate and project through the apertures creating a strong mechanical interlock between the geogrid and restrained ballast grains. This interlocking effect enables the geogrid to act as a non-horizontal displacement boundary that confines and restrains ballast grains from free movement, which in turn decreases ballast deformation.

3 Modelling geogrid-reinforced ballast

DEM simulations were also conducted to study the stress-strain behaviour of fresh and fouled ballast (VCI=40%). Irregularly-shaped ballast grains were modelled by clumping of many spherical balls together (Fig. 1a). A large-scale direct shear box was simulated by rigid walls with free loading plate placed on the top boundary. A biaxial geogrid with an aperture of 40 mm x 40 mm, similar to the geogrids tested in the laboratory

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was modelled by clumping a number of small spheres together (Fig. 1b). Fouled ballast was modelled by injecting of a predetermined number of 1.5 mm balls into the voids of fresh ballast (Figs. 1c, d). For a given normal stress and level of fouling, results obtained from the DEM simulation agree well with those measured experimentally, showing that the DEM model in this study can be used to predict the mobilized shear strength and deformation of a geogrid-reinforced ballast assembly (Ngo et al. 2014).

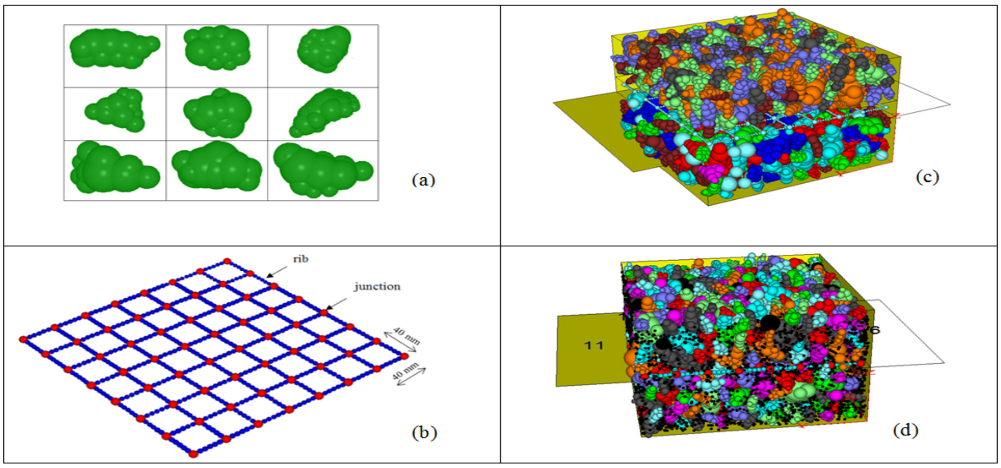

Figure 1: DEM simulation for direct shear test: (a) simulated particles; (b) simulated geogrid; (c) direct shear test for fresh ballast; (d) direct shear test for 40%VCI-fouled ballast (modified after Ngo et al. 2014)

4 Conclusions

The reinforcement effect of geogrids on improved shear strength and decreased deformation of ballast were studied using large scale laboratory tests. At a given level of ballast fouling, the results showed that geogrid increases the shear strength and reduces ballast deformation. DEM simulations were conducted to study the stress-strain behaviour and corresponding volumetric change of fresh fouled ballast. For a given normal stress and level of fouling the DEM simulation captured the shear stress-strain behaviour of ballast.

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Performance Improvement of Rail Track Structure using Artificial Inclusions - Experimental and Field Studies

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1 Introduction

The rail transportation system in many countries including Australia plays a significant role in the conveyance of bulk freight and passengers. However, increased train speed and higher freight load contribute to increased deterioration of track geometry and thereby resulting in high maintenance costs. In order to improve track performance and reduce the cost of maintenance, artificial inclusions such as geosynthetic grids, geotextiles and shock mats can be beneficial (Brown et al., 2007; Indraratna et al., 2014; Raymond, 2002). If the waste ballast is cleaned and re-used in track reinforced with geosynthetics, it is also an economically feasible alternative.

Large-scale cyclic triaxial testing conducted at the University of Wollongong indicated that recycled ballast stabilised with a layer of geocomposite (geogrid bonded with nonwoven geotextiles) perform more effectively than geogrids or geotextile alone, and also the geotextile layer prevented ballast fouling due to the upward migration of fines from layers of subballast and subgrade. The results of large-scale impact tests clearly illustrated how shock mats (shock absorbing rubber mats) improved the performance of ballast subjected to impact blows. Field trials were conducted on instrumented railway tracks at City of Bulli and Singleton, New South Wales (NSW), Australia to study (1) the effectiveness of a geocomposite installed at the ballast-capping interface; (2) the relative advantages of different types of geosynthetics (geogrids, geocomposite) on track performance; (3) the relative performance of recycled ballast (moderately graded) compared to the fresh ballast (uniformly graded); and (4) the effectiveness of shock mats in minimising ballast degradation at the stiff concrete bridge deck. The experimental and field study indicated that geosynthetic inclusions played a vital role in improving the performance of the track.

2 Experimental Studies

To understand the effectiveness of geogrid, geotextile and geocomposite, a series of cyclic loading tests was conducted simulating a 25-ton axle train running at a speed of 110 km/h (test frequency 15 Hz) using a large-scale process simulation prismoidal triaxial apparatus. Tests were conducted on fresh railroad ballast stabilised with geosynthetic inclusions placed at the ballast-subballast interface. The geogrid was more effective than the geotextile owing to a strong mechanical interlock between the grid apertures and particles of ballast (Indraratna and Nimbalkar, 2013). When geogrid bonded with geotextile (geocomposite) shown to be very effective in controlling both deformation and particle degradation. Figure 1(a) shows the vertical plastic deformation and the particle breakage evaluated from the ballast breakage index (BBI) proposed by Indraratna et al. (2005).

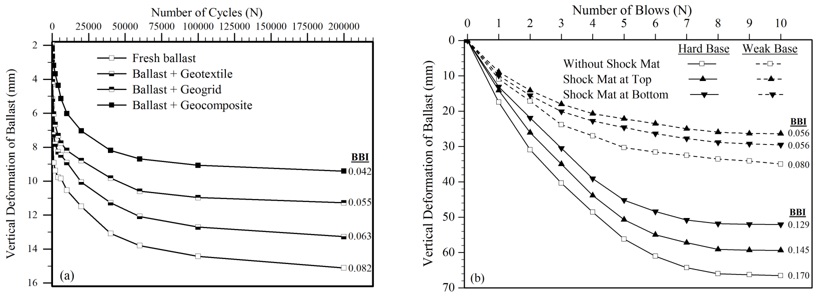

Figure 1: Vertical Deformation and Ballast Breakage Index (BBI) (a) Cyclic; and (b) Impact load test (data sourced from Indraratna and Nimbalkar, 2013; Nimbalkar et al., 2012)

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Impact load occur due to wheel-rail irregularities were simulated using a high-capacity drop-weight impact testing equipment with a cylindrical triaxial sample consisted of ballast layer on top of either a hard or a weak subgrade layer. Figure 1(b) shows the vertical plastic deformation and BBI for unreinforced and ballast stabilised with shock mats (placed either at the top or bottom of the ballast layer). The inclusion of shock mats warranted that the ballast deformation and particle breakage were significantly attenuated. The shock mats were more beneficial when placed on a hard foundation such as a concrete bridge deck compared to a relatively soft foundation such as a natural soil.

3 Field Studies

The performance of fresh and recycled ballast stabilised with geocomposite was studied with the aid of instrumented track sections located at the town of Bulli near Wollongong City (Indraratna et al., 2010). As of the results shown in Figure 2(a), the recycled ballast showed improved performance compared to fresh ballast in terms of deformation because the recycled ballast consisted of moderately graded particles with less angularity (corners already broken from previous use) and very minimal corner breakage compared to the fresh ballast with uniformly graded particles with very sharp angular corners. Recycled ballast reinforced with geocomposite performed as well as fresh ballast without any reinforcement, which indicated the potential reuse of large volumes of used ballast stabilised with geocomposite.

The performance of ballasted track with different types of geosynthetic and varying in situ conditions was investigated on track sections constructed near Singleton, NSW (Indraratna et al., 2014). From the results shown in Figure 2(b), the vertical deformation of the ballast layer was curtailed significantly by using geosynthetics in the ballast-subballast interface for similar subgrade sections. Geogrid with 40 × 40 mm size apertures performed better than the other types of geogrids used in this study. The stiffer track condition at the bridge deck clearly implied that the inclusion of the shock mat above the bridge deck significantly reduced ballast breakage as shown in Figure 2(c).

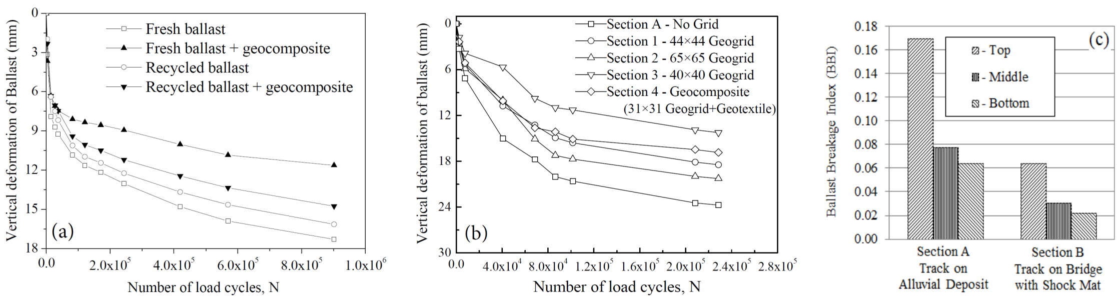

Figure 2: Vertical deformation (a) Bulli; (b) Singleton tracks and (c) BBI from Singleton track (data sourced from Indraratna et al., 2010; 2014)

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Basal Reinforced Piled Embankments

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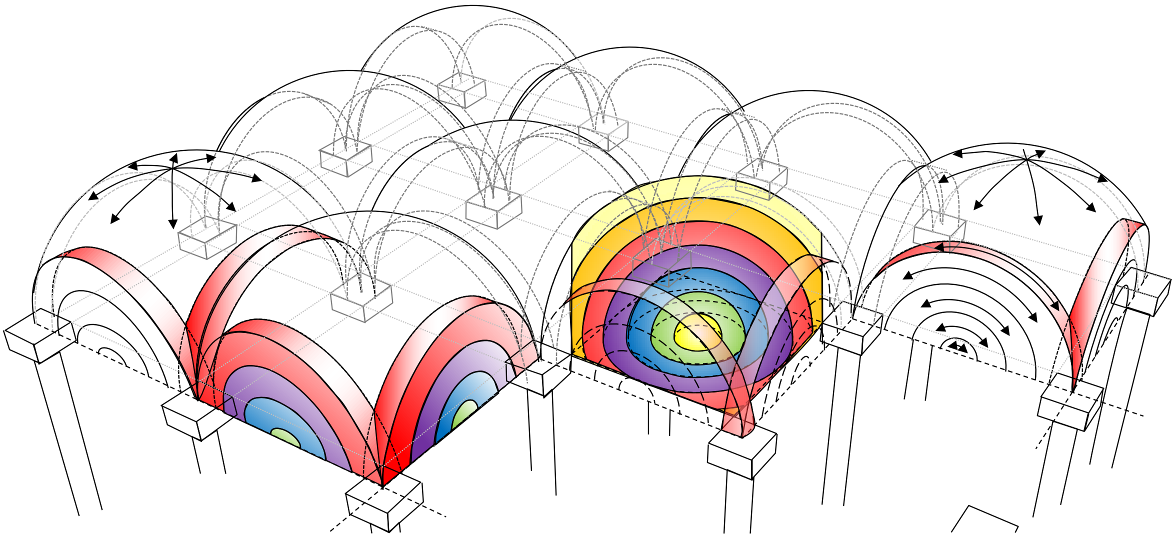
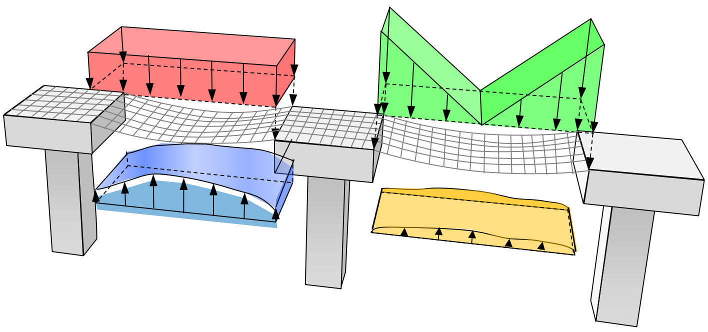
1 Design method for the basal reinforcement of a piled embankment

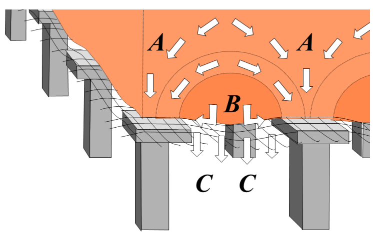
A series of nineteen scaled 3D experiments were conducted on basal reinforced piled embankments. Based on this, the new Concentric Arches was developed, which is a design model for the basal geosynthetic reinforcement (GR) of a piled embankment (Figure 1).



(a) Calculation step 1 Arching. The load is transferred along the concentric 3D hemispheres towards the GR strips and pile caps and then via the concentric 2D arches towards the pile caps.



(b) Calculation step 2: When there is no subsoil support, or almost no subsoil support, the inverse triangular load distribution on the GR strips gives the best match with measurements. When there is significant subsoil support, the load distribution is approximately uniform



(c) Resulting load distribution

Figure 1: The new Concentric Arches model (Van Eekelen, 2015) consists of two calculation steps: (a) step 1 calculates the load distribution in A and B+C and (b) step 2 calculates the GR strain that occurs in the GR strip between adjacent pile caps (c) resulting load distribution (*A, B, C*)

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2 Validation of the new and an old analytical model

The new model and the design model of Zaeske (2001), which was adopted in the 2010 versions of the German and Dutch design guidelines (EBGEO, 2010 and CUR226, 2010) were validated with measurements from seven full-scale projects and four series of scaled model experiments (Figure 2).

The GR strain calculated with Zaeske, 2001 is on average 2.5 times the measured strain. The GR strain calculated with the new model is on average 1.1 times the measured GR strain. This is almost a perfect match. The new Dutch design guideline CUR226 (2016) has therefore adopted the Concentric Arches model.

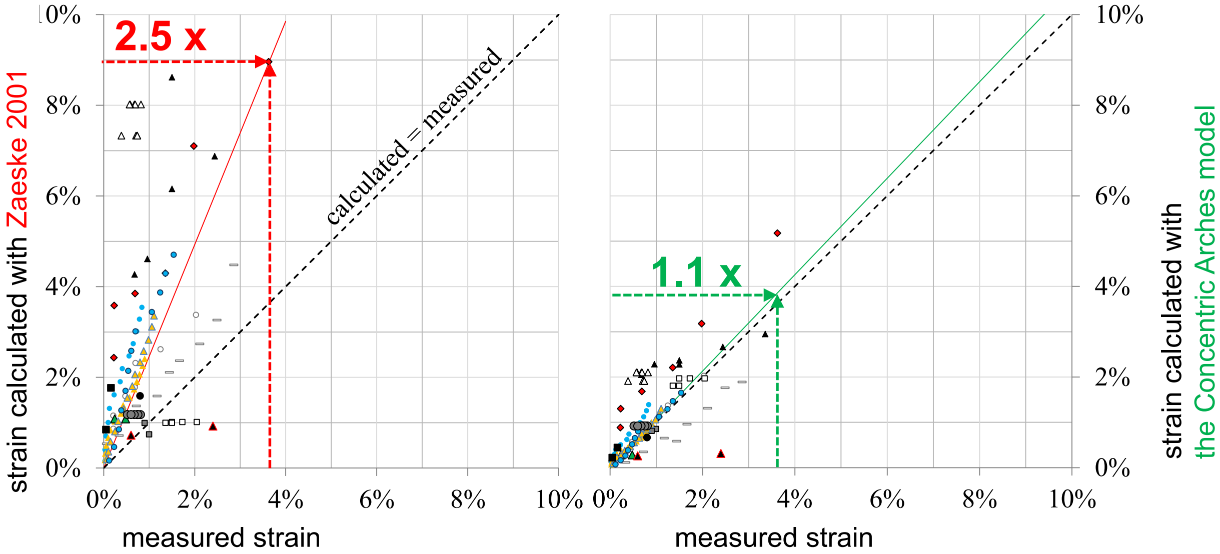

Figure 2: Comparison calculations and measurements in seven field projects and four series of experiments. Van Eekelen, 2015 gives the sources of the references given in this picture, which are not given in the references of this paper due to space limitations. Calculations without safety factors

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Geosynthetics with Enhanced Lateral Drainage Capabilities in Roadway Systems

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1 Overview

While moisture-induced distress is one of the major causes of premature pavement failure, geosynthetics are only seldom used to provide internal drainage within the structure of roadway systems. This is likely because conventional geosynthetic drains are only suitable to manage flow under saturated soil conditions, whereas unsaturated conditions prevail in pavement systems. Recent insight into the interaction between geosynthetics and unsaturated soils has led to new advances in geosynthetic manufacturing, including the development of geotextiles with enhanced lateral drainage (ELD), which allow drainage even under unsaturated conditions (see Figure 1).

Zornberg et al. (2016) highlight the benefits of ELD in a number of roadway situations, including: (1) enhanced lateral drainage of moisture migrating upward from a high water table, (2) enhanced lateral drainage of moisture infiltrating downward from the surface, (3) control of frost heave-induced pavement damage, (4) control of pavement damage caused by expansive clay subgrades, and (5) enhanced lateral drainage in projects involving soil improvement. The mechanisms of moisture migration, as well as the impact of ELD are evaluated in each of these situations.

Additionally, case histories involving recently constructed pavements designed using ELD geosynthetics are presented each specific drainage application. The selected case histories involve post-construction evaluation of the ELD geosynthetic's performance either through assessment of lateral drainage, condition survey of pavement sections with and without ELD geosynthetics, or in-situ monitoring of moisture content. Assessment of the data collected illustrates the beneficial impact of ELD used in the various pavement design scenarios.

2 Main Findings

Even though a prime cause for pavement distress is the presence of moisture in its structural layers, the use of internal drainage in pavement systems has not been typically adopted in design. While the volume of moisture within pavement sections may not be particularly significant, its removal cannot typically be accomplished with conventional drainage systems because soil moisture is stored under unsaturated conditions. Recent advances in the manufacturing of woven geotextiles have led to the incorporation of specially designed fibers with a uniquely designed cross-section characterized by the presence of deep grooves. The characteristics of this fiber cross-section promote capillary flow within the fiber itself, which provides enhanced lateral drainage by mobilizing moisture under unsaturated soil conditions.

A number of roadway situations are highlighted that would benefit from the use of enhanced lateral drainage. Case histories in Missouri, Idaho, Montana, Texas and California are presented to illustrate various scenarios where enhanced lateral drainage should be considered in pavement design. More importantly, the case histories also include post-construction evaluation of the ELD geosynthetic's performance, either through assessment of lateral drainage, condition survey of pavement sections with and without ELD geosynthetics, or in-situ monitoring of moisture content. The following conclusions can be drawn from the information presented in this work:

- Conventional geosynthetic and granular drainage systems are unable to convey flow from base and subgrade materials under unsaturated conditions. Yet, the incorporation of wicking yarns into woven geotextiles has led to the development of ELD geosynthetics, which are capable of conveying moisture stored in these unsaturated pavement layers.

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- Specific applications of ELD geosynthetics, with appropriate termination details, in roadway design have been identified to be beneficial to pavement performance. They include: (1) enhanced lateral drainage of moisture migrating upward from a high water table; (2) enhanced lateral drainage of moisture infiltrating downward from the surface; (3) control of frost heave-induced pavement damage; (4) control of pavement damage caused by expansive clay subgrades; and (5) enhanced lateral drainage in projects involving soil improvement.
- The use of ELD geosynthetics has shown pavement benefits complementing those strictly related to enhanced lateral drainage. This includes multiple additional applications of geosynthetics in pavements, including separation (see all five case histories), subgrade stabilization (see Missouri, Montana, Texas and California case histories) and base stabilization (see Idaho case history).
- The use of ELD geosynthetics, often because of additional applications they offer, has shown cost savings associated with a decrease in thickness of the base layer (see Missouri, Idaho and California case histories).
- Evaluation of post-construction performance indicates that use of ELD geosynthetics provides enhanced drainage, as intended in design. This is based on an evaluation of field observations of effective lateral drainage (see Missouri, Idaho, Montana and California case histories), condition surveys to compare performance of pavement sections with and without ELD geosynthetics (see Montana and Texas case histories) or in-situ monitoring of moisture content (see Texas case history).

Overall, data on roadway performance from a number of case histories reported in this work indicates that enhanced lateral drainage in roadways offers often significant opportunities to improve the performance of a wide range of transportation projects.

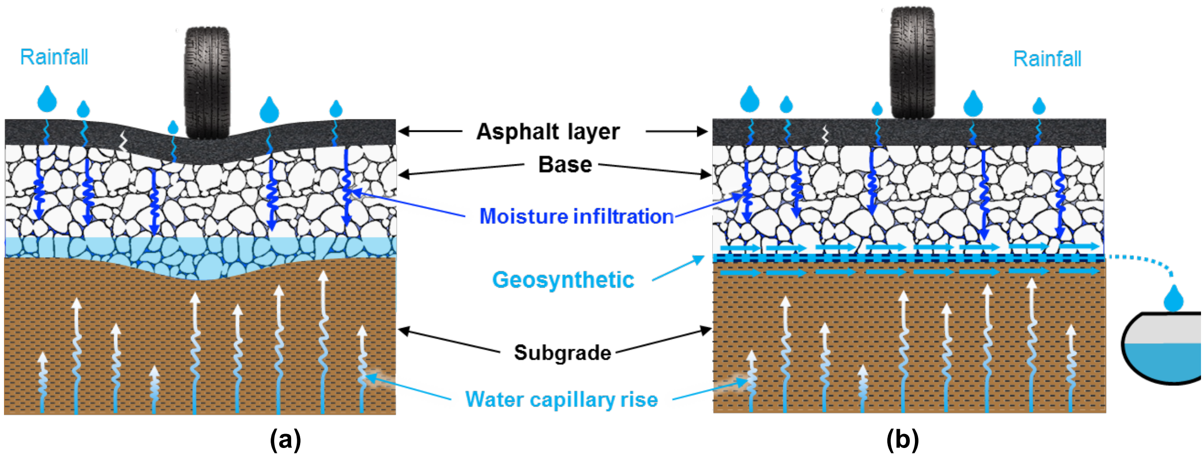

Figure 1: Use of Geosynthetics in Improved Internal Drainage: (a) Case not including geosynthetics, (b) Case including geosynthetics

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Effect of Geogrid on Railroad Ballast Particle Movement

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1 Introduction

Individual ballast particle movement significantly affects ballast performance. In order to improve ballast performance, geogrids are widely used to create interlocking of ballast particles to reduce particle movement. To better quantify the effect of geogrid on particle movement, two types of ballast box tests were conducted in this study: one without geogrid as a control and the other one with a layer of multiaxial geogrid placed at 25 cm below the top of the ballast. The ballast box represents a half section of a railroad track structure consisting of a ballast layer, two crossties, and a rail (I-beam). A wireless device – “SmartRock” was embedded underneath the rail seat and the edge of a tie in the ballast layer to monitor particle translation and rotation under cyclic loading.

2 Main results and conclusions

The results indicate that 1) horizontal translation and rotation are important modes of movement for ballast particles under cyclic loading; 2) particle translational movement and rotation were higher beneath the edge of the tie than those beneath the rail seat; 3) the particle movement, such as translation and rotation, were significantly reduced with an inclusion of the geogrid under 500 load cycles. This investigation also demonstrates that the SmartRock is capable of recording and visualizing real-time particle movement including translation and rotation and, hence, can be used as a fundamental research and monitoring tool in railroad.

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Geosynthetic Subgrade Stabilization – Field Testing and Design Method Calibration

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1 Introduction

For low-volume roadways and temporary construction platforms where excavation and replacement of inferior subsoils may not be cost effective, soil stabilization using geosynthetics may provide a working platform so that the base course aggregate layer can be properly constructed and overall rutting reduced. Geosynthetics are planar polymeric materials that have been extensively used in these situations (i.e., subgrade stabilization) to reinforce and/or separate poorer naturally deposited soils from the crushed aggregate layer. The separation function is primarily attributed to geotextiles, while the reinforcement function may be derived from both geotextiles and geogrids. Subgrade stabilization is typically applicable for unpaved temporary roads such as haul roads, or construction platforms to support permanent roads. These roads are generally characterized by low volumes of heavy vehicles that can tolerate deeper ruts. According to the National Highway Institute, geosynthetic stabilization techniques used for these types of roads are "one of the more important uses of geosynthetics" (Holtz et al., 2008). Historically, geotextiles were first used in these applications; however, geogrids are more commonly used in recent years. The first design for geotextile stabilization of unpaved roads was created in the late 1970s by Steward et al. (1977) based on soil mechanics theory and experimental data generated in the laboratory and field. Since then, several alternative designs for geogrid stabilization have also been created (Tingle and Webster, 2003; Giroud and Han, 2004a; USCOE, 2003). Limitations within each of these methods, lack of calibration for a wide variety of products, and a growing variety of the types, strengths, and composition of geosynthetic reinforcement products has introduced uncertainty in the design of geosynthetic-reinforced unpaved roads.

Geosynthetics can improve the performance of weak subgrades under temporary unpaved roads by the following mechanisms: 1) reduction of plastic shear stresses that cause bearing capacity failure in the subgrade, 2) reduction of maximum normal stresses on the subgrade surface by improved load distribution, 3) increase in the bearing capacity of the subgrade by confining lateral movement at the subgrade-base interface and a reorientation of the induced shear stresses, 4) increase in the bearing capacity and stress reduction attributable to the "tensioned membrane effect" in rutted areas, 5) lateral restraint and reinforcement of base course aggregates, and 6) reduction of mixing between subgrade and base soils (Hufenus et al., 2006; Maxwell et al., 2005; Giroud and Han, 2004a; Leng, 2002; Perkins et al., 2005; Watn et al., 2005). These improvements in subgrade performance can facilitate compaction, reduce the gravel surface thickness, delay rut formation, and extend the service life of unpaved roads, particularly in cases of very soft subgrades with a California Bearing Ratio (CBR) less than three (Benson et al., 2005; Hufenus et al., 2006).

The current practice of using geosynthetics for subgrade stabilization is primarily based on empirical evidence from constructed test sections. Field tests constructed strictly for research purposes, instead of during scheduled rehabilitation or reconstruction activities, offer better control over study variables such as careful preparation of soil and reduced incidental trafficking. Despite this, it is still difficult to achieve uniform conditions throughout a project site utilizing the natural subgrade (e.g., Fannin and Sigurdsson, 1996; Edil et al., 2002; and Hufenus et al., 2006). Conversely, research studies in which a subgrade soil was artificially placed demonstrate better consistency (e.g., Santoni et al., 2001; Perkins, 2002; Tingle and Webster, 2003, Cuelho and Perkins, 2009; Cuelho et al., 2014).

While laboratory studies can be conducted more quickly and usually include more alternatives, they are only able to simulate field conditions and, as Hufenus et al. (2006) point out, there "are no incontrovertible indications from laboratory tests of the influence that the geosynthetic will have on the performance of the pavement under trafficking" (p. 23). Thus, the need still exists for field tests that provide uniform conditions and incorporate a variety of geosynthetics in order to develop a sufficient database of performance results. The need for such a database of information is resoundingly clear in light of the fact that there is still not a

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universally accepted and calibrated design method for unpaved roads (or construction platforms) that incorporate both soil and geosynthetic material properties.

A simple existing design published by the Federal Highway Administration (FHWA, 1995) is based on the U.S. Forest Service method developed by Steward et al. (1977). A more recent design method for unpaved roads that attempts to incorporate geosynthetic properties was theoretically derived based on the stresses that develop at the base-subgrade interface. The impact of these stresses and the subgrade bearing capacity were related to rut depth based on empirical data (Giroud and Han, 2004a). However, only limited data were used to calibrate the model: 1) field data from Hammitt (1970) for unreinforced unpaved sections, and 2) lab data from Gabr (2001) that involved two versions of one type of geosynthetic (integrally formed geogrid). One parameter (the bearing capacity, N_c) in the model can take on three different values depending on whether the roadway is 1) unreinforced, 2) geotextile reinforced, or 3) geogrid reinforced design. If a geogrid is under consideration, the aperture stability modulus is used, but only if the material property is within the approximate range of the types of geogrids tested by Gabr (2001) for which the model was calibrated. Even though this design method was intended to be used to design reinforced and unreinforced unpaved roads, there are inherent limitations in how it models the contribution of various geosynthetics that should be considered. While this model is an improvement over less sophisticated designs from the 1980s (Giroud and Noiray, 1981 and Giroud et al., 1985), there is still a need to investigate the performance of geosynthetics in controlled field tests. Calibrations with additional data sets may be sufficient, although geosynthetic material properties other than aperture stability modulus should be considered.

Based on these limitations, two ambitious projects were sponsored by the Montana Department of Transportation (MDT) to evaluate the performance and behavior of a wide variety of geogrids and geotextiles when used as subgrade stabilization (Cuelho and Perkins, 2009; Cuelho et al., 2014). The primary objective of these research efforts was developed based on deficiencies in the standard design techniques and lack of agreement as to which geosynthetic properties are most relevant for this application. The results of this research were used to understand which properties are most relevant to this application, and consequently to update the design methodology to incorporate these material properties. The result of which should be a more accurate design method that more broadly encompasses materials with which good experience exists.

2 Experimental Program

This research project was specifically planned to quantify differences in performance of various geosynthetic products under the same conditions (i.e., same subgrade strength and base course thickness). In addition, supplemental test sections were constructed to study the effect that variations in subgrade strength and base course thickness had on the performance. Specifically, three control sections (i.e., no geosynthetic) were constructed, each having different thickness of base course aggregate, and three test sections were built using the same integrally-formed geogrid (test sections IFG-1, IFG-2, and IFG-3), each having different subgrade strengths. The final arrangement of the test sections is shown in Figure 1, which includes the average subgrade strengths and base thicknesses. Each test section was 4.9 m wide and 15 m long. The Transcend research test facility managed by the Western Transportation Institute at Montana State University was used for this study.

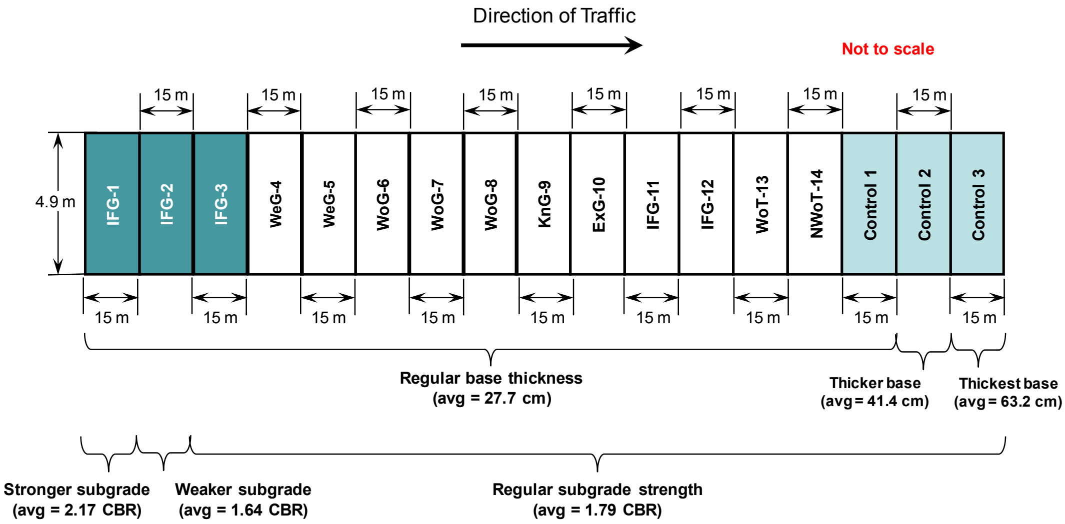

Figure 1: General layout of test sections

Twelve geosynthetic products (ten geogrids and two geotextiles) were used in this research project to evaluate their relative performance under the conditions presented herein. Five laboratory tests were used to characterize the geosynthetics used in this research, and include wide-width tensile strength (ASTM D 4595 and ASTM D 6637), cyclic tensile modulus (ASTM D 7556), resilient interface shear stiffness (ASTM D 7499), junction strength (ASTM D 7737), and aperture stability modulus (Kinney, 2000).

Construction of the test sections began with preparing and placing the subgrade, followed by installing the geosynthetics and instrumentation, and finally preparing and placing the base course aggregate. Preparation and construction of the subgrade and base course was monitored extensively to ensure that these materials were placed in a consistent and uniform manner.

The subgrade was built in six lifts that were approximately 15 cm deep for a total depth of about 0.9 m. The final thickness of the base course was about 28 cm of gravel, on average. Two of the three control test sections contained thicker base material. The Control 2 test section was constructed to be about 41 cm thick, and the Control 3 test section was constructed to be about 63 cm thick.

Trafficking was accomplished using a three-axle dump truck that weighed 20.6 metric tons and had 620 kPa tire pressure. Trafficking was always in one direction, and the speed was approximately 8 kph to ensure that dynamic loads were not induced in the test sections from any unevenness in the gravel surface. Trafficking was applied until rut levels reached 75 mm. Rut measurements were made at 1-meter intervals along two longitudinal lines, corresponding to the outside rear wheels of the test vehicle. These measurements were used to determine rut as a function of the difference in the elevation of the measurement points over time.

3 Data Analysis

Longitudinal rut measurements were the primary means used to determine the relative performance of each test section. An empirical correction procedure was implemented to adjust the rut response for these two properties so that direct performance comparisons between test sections were more accurate. After adjustments for subgrade strength and base course thickness were applied to the rut data, the remaining behavioral differences between the reinforced test sections could more confidently be attributed to the geosynthetic reinforcement.

An analysis of the longitudinal rut responses was conducted to determine which geosynthetic material properties were most related to the performance of a particular test section. This analysis was conducted at various rut depths to determine whether different material properties affected performance at various levels of rut. Predicted values were used in the regression analysis for test sections that did not reach failure. The following material properties were considered in this analysis:

- Wide-width tensile strength at 2%
- Wide-width tensile strength at 5%
- Ultimate wide-width tensile strength
- Cyclic tensile stiffness at 0.5, 1.0, 1.5, 2.0, 3.0 and 4.0 percent
- Resilient interface shear stiffness in the cross-machine direction
- Junction strength in the cross-machine direction
- Junction stiffness in the cross-machine direction, determined by taking the secant stiffness of the junction strength response at 1.3 mm of displacement
- Aperture stability modulus

Linear regression was used in this analysis because there were too few points to clearly indicate a more sophisticated regression equation, and it provided sufficient information to be able to compare data fit between individual factors or to observe changes or trends in data fit for multiple variables. The geosynthetic material property that best related to the performance of the test sections was the strength and stiffness of the junctions in the cross-machine direction, and this property correlated better with performance as rut increased.

A linear regression analysis was also conducted using rut data from Phase I of this project (Cuelho and Perkins, 2009). Six of the test sections from Phase I used the same geosynthetics as this project (IFG-3, WeG-4, WeG-5, WoG-6, WoG-8, and NWoT-14). These test sections had very similar subgrade strengths but 75 mm less base aggregate thickness, creating a more severe condition than Phase II. Considering a similar approach as before, the regression analysis using performance data from Phase I indicates, overall, that tensile strength in both material directions relate to performance at higher levels of rut, while junction strength relates to performance at lower levels of rut. The relationship with junction stiffness peaks at 76.2 mm of rut. Aperture stability modulus is also related to early performance of the Phase I test sections.

Considering both phases of this effort, junction stiffness and tensile strength are the two most important properties associated with good performance of geogrids used in this application and under these conditions. These two properties work together to ensure proper reinforcement of the gravel layer and increased longevity of the road (evident as an increased number of load passes). Geogrids with weak junctions were unable to fully utilize the strength in the cross-machine direction because the load transfer through the individual members and into the junctions was weak. Overall, this analytical process helped establish geosynthetic material properties associated with good performance. The geotextile materials used during these studies showed good performance as subgrade stabilization, but material properties associated with their performance was difficult to establish due to the limited number of test sections and lack of relevant tests to properly characterize these types of materials for this application.

4 Calibration of Design Equation

Information from the testing program described above was used to calibrate the design equation associated with the methodology developed by Giroud and Han (2004a, 2004b). The generic design approach developed by Giroud and Han (also referred to as the G-H method) takes into consideration the geometry of the unpaved structure, the level of truck traffic, the truck axle configuration and loading, rut depth and serviceability, the properties of the base course and subgrade materials, the ratio of base course and subgrade strength, and the properties of the geogrid.

The single property that varied within the test sections was the number of axle passes of the truck (N) for a particular rut level. The value of k' was determined at various levels of rut (s = 38.1 mm, 50.8 mm, 63.5 mm and 76.2 mm) and axle passes. Because k' represents the material property of the geogrid associated with design, it was plotted with respect to the various properties of the geosynthetic, as in the regression analysis previously described above. Similar to that analysis, linear regression was used to fit the data from these comparisons.

Not surprisingly, a similar pattern of interdependence between k′ and junction stiffness emerged from these analyses. Therefore, junction stiffness emerged from this analysis as the primary material property related to rutting performance. This does not mean that tensile strength in the cross-machine direction is unimportant, as indicated by the previous regression analyses. Development of the tensile capacity of the geogrid was critically dependent on the ability of the junction to transmit stresses into members oriented in the cross-machine direction. Therefore, materials that have sufficient tensile strength but weak junctions will not perform well. The converse is also true – strong junctions with weak members will also not perform well. Reasons why the aperture stability modulus did not correlate to performance is unknown. The individual slopes of k′ versus junction stiffness varied as a function of rut, as shown in Figure 2. The information contained in Figure 2 can be used to determine the value of k′ based on junction stiffness (in units of MN/m/m).

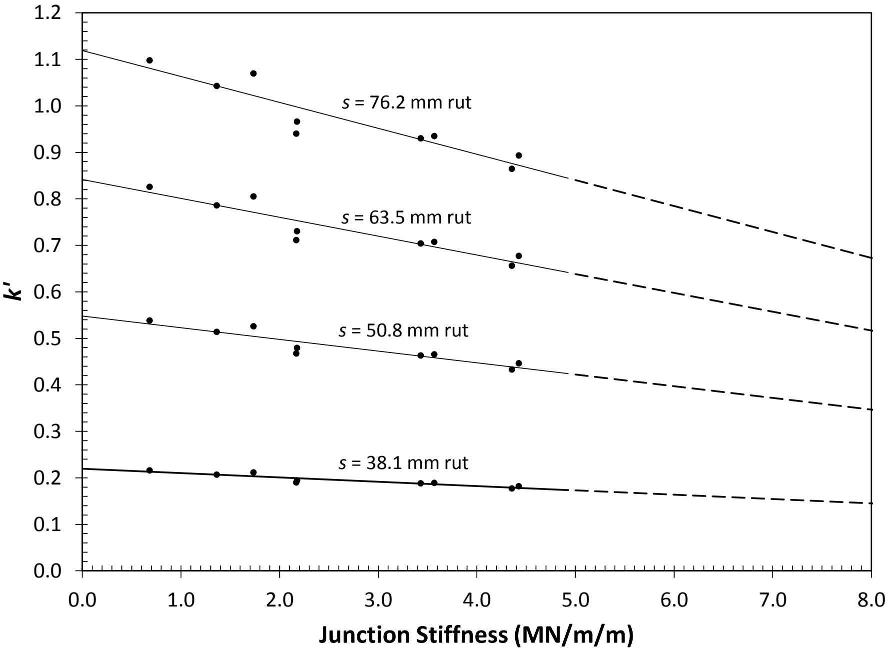

Figure 2: Junction stiffness versus k' for various levels of rut

To validate the accuracy of this method, the design equation was rearranged in terms of the number of traffic passes, and the predicted number of passes was determined based on junction stiffnesses of the geogrids used in this project. Values for k' were determined using the relationships presented in Figure 2. The predicted values were compared to the actual number of passes to reach various levels of rut, the results of which are shown in Figure 3. These results indicate that there is good correlation between the predicted and actual values (R^2 = 0.857).

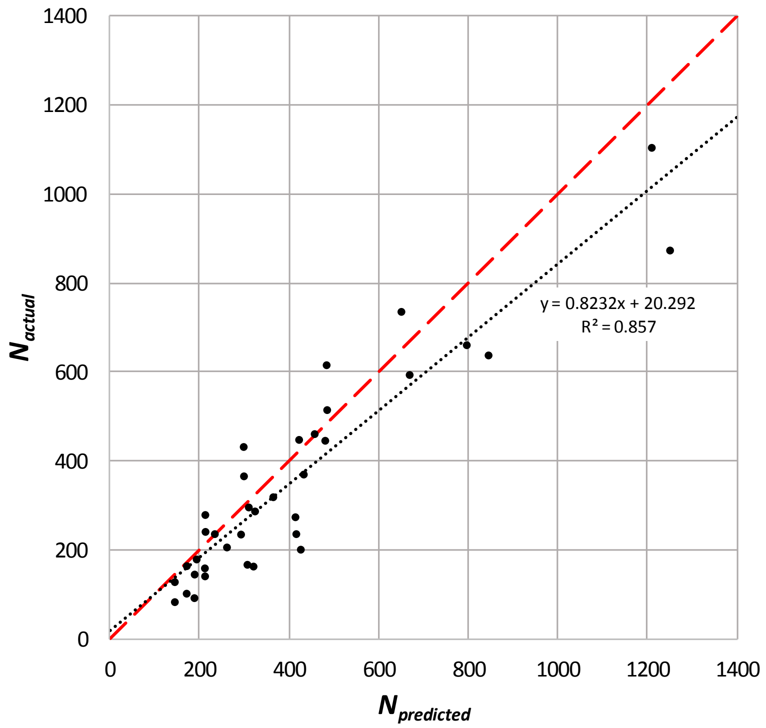

Figure 3: Comparison of predicted axle passes to actual axle passes

The final form of the design equation for geosynthetic reinforced unpaved roadways, based on the development work by Giroud and Han (2004a) and the calibration based on field test sections constructed by Cuelho et al. (2014), is shown in Equation 1. It should be noted that the base course layer thickness (h) is present on both sides of the equation; therefore, an iterative process is necessary to determine a single value of h for a given set of conditions. The variable k' is based solely on junction stiffness (ASTM D7737), and can be determined using the relationships presented in Figure 2. The information contained in Figure 2 is shown for various levels of expected rut, as indicated by the multiple lines. The regression lines are extended to include products with greater junction strengths than the materials used in this study.

$$h = \frac{1.26\left[1+k'\left(\frac{r}{h}\right)^{1.5}\log N\right]}{1+0.204(R_E-1)}\left[\sqrt{\frac{\frac{P}{\pi r^2}}{\left(\frac{s}{f_S}\right)\left[1-0.9\exp\left(-\left(\frac{r}{h}\right)^2\right)\right]N_c c_u}}-1\right]r \qquad \text{Equation 1}$$

5 Summary and Conclusions

Geosynthetics are routinely used to stabilize weak soils in transportation applications; however, deficiencies in the standard design techniques have made widespread adoption of the existing methods slow. In addition, agreement as to which geosynthetic properties are most relevant to subgrade stabilization applications is lacking. A rigorous program was undertaken by researchers at the Western Transportation Institute at Montana State University, and sponsored by the Montana Department of Transportation, to build and traffic multiple test sections containing various types of geosynthetic reinforcement products. The results of this effort were used to better understand which properties are most relevant to subgrade stabilization of unpaved roads, and consequently to update the design methodology to incorporate these material properties.

Test sections were built and trafficked in a large-scale controlled laboratory environment to study the performance of various geosynthetics to stabilize weak subgrades. A comprehensive set of material tests were conducted to more thoroughly evaluate the potential relationship between geosynthetic material properties and the relative performance of the test sections. The results of a regression analysis showed that junction stiffness was the best indicator of performance under these conditions.

The Giroud-Han method is currently the most sophisticated method to design geosynthetic stabilized unpaved roads; however, calibration of this method to date is based on very limited data. Performance data from the full-scale field tests built as part of this research effort were used to calibrate the Giroud-Han design equation. Geogrid junction stiffness was the primary indicator of performance in these test sections and was therefore used as the material property to calibrate the design equation. Correlations between predicted axle passes and actual axle passes indicated good predictions using the calibrated equation. The newly calibrated design equation presented herein replacing aperture stability modulus with junction stiffness to describe the contribution of the geogrid.

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Contact Pressure Distribution on Weak Subgrades due to Repeated Traffic on Geocell Reinforced Base Layers

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1 Introduction

Design of pavements over weak subgrades is often a challenging task for design engineers. Stabilizing the weak subgrades with either cementitious compounds or geosynthetics is a common practice to improve the engineering behavior. Geocells is a recent introduction in geosynthetics as a reinforcement material, which offers all round confinement to the infill material against planar products. In addition, the geocell reinforced bases exhibit bending resistance, tensile strength, and shear strength, and intercept the failure planes reaching the subgrades. Understanding of these mechanisms originated mostly from the static plate load tests, however, the loading is repetitive in nature under moving traffic conditions. Hence, it is important to understand the behavior of weak subgrades under cyclic and repetitive traffic conditions when geocell reinforcement is adopted for overlying base layers.

2 Experimental Program and Results

This paper presents the results from a series of large scale repeated load tests performed on an unreinforced and geocell reinforced dense granular base layers overlying weak clayey soil subgrades. A dense sand base layer was prepared at 75% relative density over a weak clayey subgrade of undrained strength of 10 kPa. A tire pressure of 550 kPa equivalent to a single axle wheel load (ESAL) of 40 kN was applied through a circular steel plate using a dynamic hydraulic actuator of 100 kN capacity. Several earth pressure cells were placed at the interface of the dense base and weak subgrade layers to measure the contact pressure distribution on the latter layer. Both cyclic and repeated load tests were conducted to visualize the contact pressure distribution on the weak subgrades. The results have illustrated the variation of contact pressure distribution patterns over weak subgrades.

It is observed that the dense sand and granular base layers over weak subgrades improved the load carrying capacity of the subgrade. The surface layer on the unreinforced test bed has further improved the performance in terms of bearing pressure. Additional improvement is noticed with the geocell mattress in the dense base layers. A more uniform pressure distribution is depicted with the three dimensional mattress which offers all round confinement to the infill base material. It is further noticed that the granular base layer has performed superior to the dense sand layer in terms of pressure distribution and load carrying capacity due to high frictional resistance of the base material (Figure 1).

It is also observed that the contact pressure has reduced with the number of load repetitions against the incremental cyclic loading. It is interesting to note that the stiffer base layer acts like a rigid deep slab on the soft or weak subgrade and shows a reduction in the contact pressure distribution. With the inclusion of geocell mattress, the flexural rigidity of the granular base layer is expected to enhance further and hence, the base layer behaves as a rigid layer over weak subgrades. As high as 90% reduction in contact pressure distributed over on the weak subgrades with geocell reinforcement. Overall, there is no stress concentration and accumulation noticed with an increase in the repetitive load cycles due to resilient behavior of the geocell reinforcement (Figure 1).

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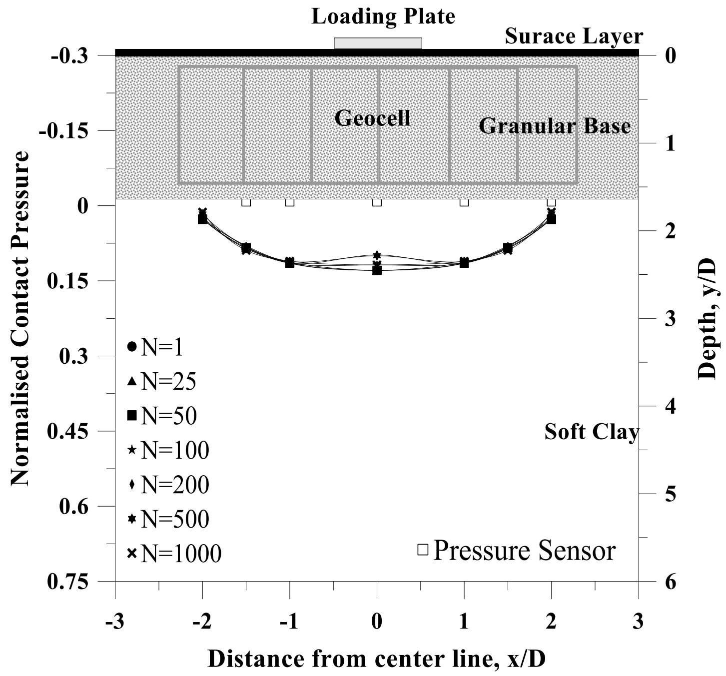

Figure 1: Contact pressure distribution on weak subgrade due to repeated traffic load (550 kPa) on geocell reinforced granular base – with surface layer

The Use of Geosynthetics in Water Conveyance Structures - The Panama Canal Expansion Project, Third Set of Locks Water Saving Basins

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1 Introduction

The Panama Canal expansion project, opened to traffic in June 26, 2016, involves the addition of 6 locks to the 77 km long navigation canal connecting the Atlantic with the Pacific Ocean. The new locks increased the canal width and depth necessary to support modern container ships up to 12.600 TEU.

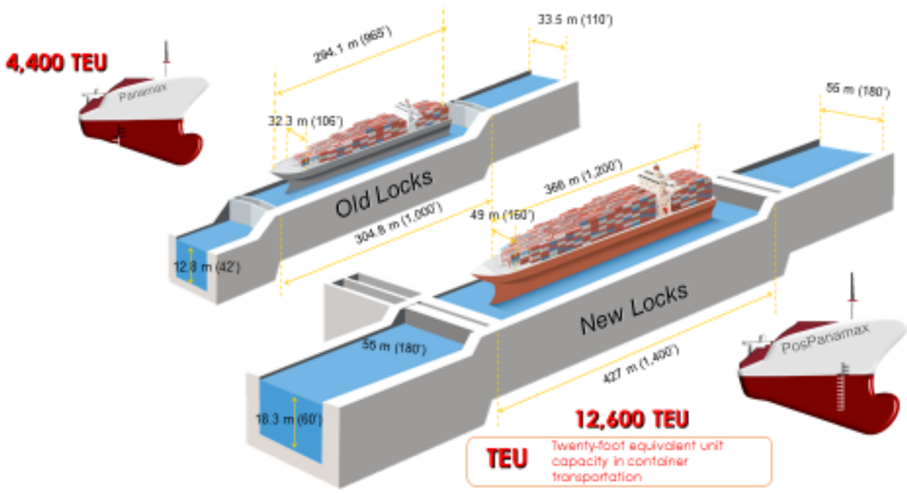

Figure 1: Comparison between the locks

In order to save water of Lake Gatun for the operation of the locks, 18 Water Saving Basins (WSB), 9 on the Atlantic side locks and 9 on the Pacific side locks, were built and waterproofed with PVC geomembranes and geocomposites.

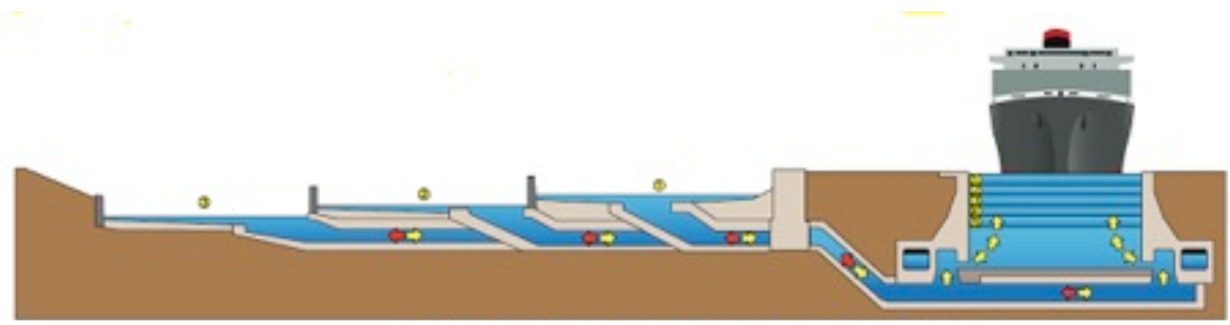

Figure 2: Operating scheme of a lock

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Figure 3: General View of the Atlantic locks

The construction works consortium, GUPC – Grupo Unidos Por el Canal, includes the Spanish contractor Sacyr, Salini-Impregilo of Italy, Jan de Nul of Belgium, and the Panamanian company CUSA. The waterproofing subcontractor was the company Carpi Tech.

2 Scope of the executed works

The Waterproofing Works consisted in the supply and installation of a flexible geomembrane liner (FGL) system in each WSB, together with other geosynthetic materials for drainage and protection. The waterproofing works included anchorage systems in soil, rock and concrete; tensioning, ballasting and expansion systems; fixtures for accommodating lining penetrations and connections; gaskets, seams and sealing systems; appurtenant fixtures and materials as necessary to produce a complete and fully functioning FGL system, meeting the design and performance requirements.

The waterproofing of the WSB at the Atlantic and Pacific site covered the whole surface, bottom and slopes, joints at the dividing concrete walls of each basin, reaching the intake of the conduits, two at each basins.

All concrete structures in contact with water, and not covered by the geomembrane, have a perimeter watertight seal designed to guarantee the watertightness.

3 The design

The design of the Water Saving Basins at the Panama Canal Expansion Project was developed based on more than 50 years of experiences in similar structures. The contractors' consortium, with an international consultants' consortium, in close collaboration with the specialized sub-contractor, developed the construction design adapting and optimizing the design baselines to the conditions encountered during the construction of the basins. The forces considered for the design of the anchoring system were uplift by wind speed and hydrodynamic water pressure, erosion due to fluctuations on the operation level of the basins, sliding of slopes and ground geotechnical conditions.

Figure 4: General View of Lower Chamber in the Pacific Site

4 Materials used

The whole surface lined by the waterproofing system, 600.000,00 m^2, is on direct contact with the excavated soil or hearthfill. The waterproofing system is composed by a geocomposite consisting of a Polyvinylchloride (PVC) geomembrane, 3.0 mm thick on the slopes and 2.5 mm thick on the bottom, heat-bonded to a Polypropylene (PP) geotextile with 500 g/m^2 mass per unit area, covering an 8 mm thick geodrain, composed by a geonet coupled with geotextile filters at both faces to avoid the intrusion of fines. The geodrain layer is double in the perimeter of the intakes under the maintenance road to increase the capacity of the drainage. The higher thickness chosen for the PVC geomembrane on the slopes was due to its exposure to the UV radiation during normal operation of the basins (14 cycles per day).

5 Anchoring system

The waterproofing system anchorage is done by different mechanical fixing, like deep soil anchors, ballast trenches filled with concrete, ballast concrete blocks, tensioning profiles and trenches, each one with the common purpose of holding the geomembrane against the terrain to reach a smooth lining in case of all the mentioned forces. Each type of anchors was design depending on the anchoring subgrade, which changed without regular fixed pattern from Gatun rock, to clay, to excavated basalt or rock/soil filling, to concrete, to shotcrete.

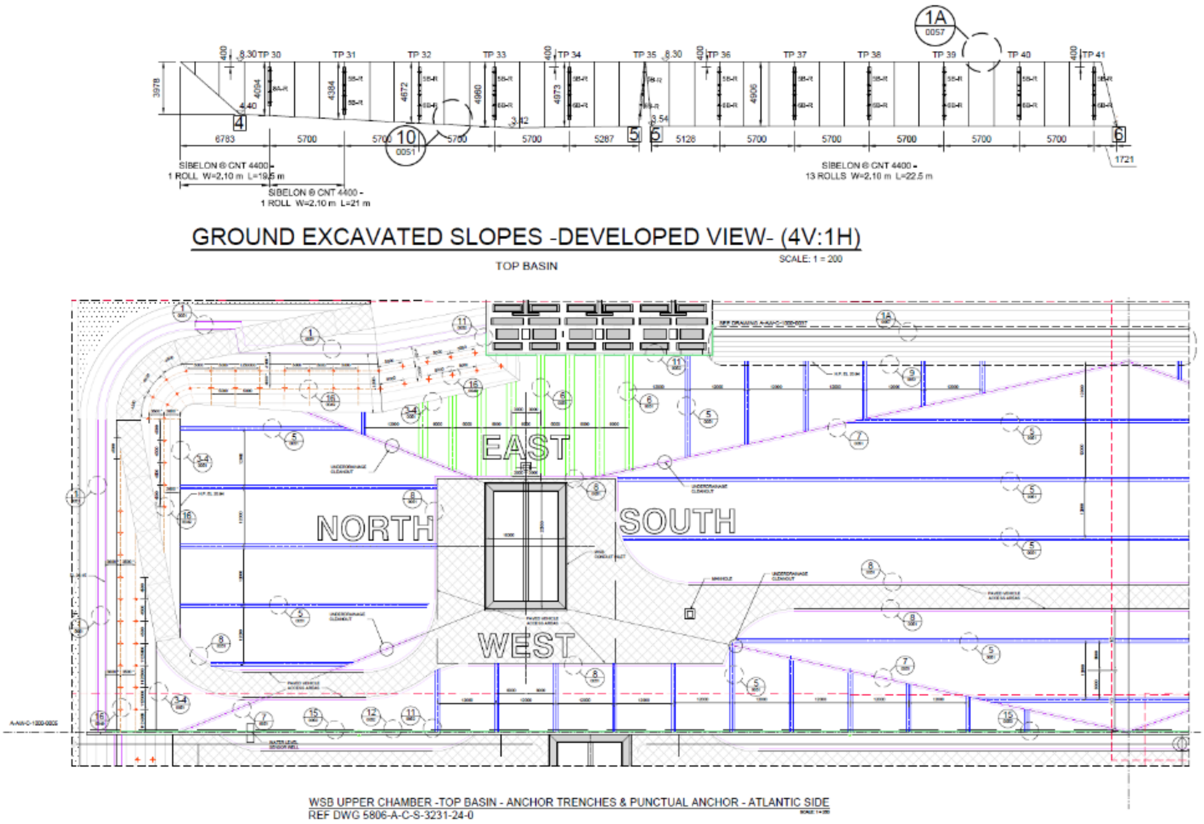

Figure 5: General View plan of a basin with location of trenches, deep anchors, and typical distribution of tensioning profiles

5.1 Trenches anchoring system

The trenches concept used mainly in the bottom of the basins and in the top anchorage, consist in a ballast of concrete placed on top of the impervious layer with the function of holding the system on place and tensioning it at the same time.

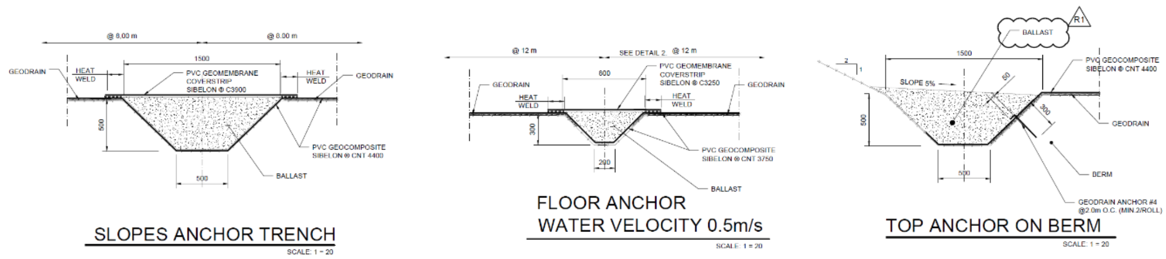

Figure 6: Anchor trenches Atlantic site

At the pacific site, the slopes were constructed with rockfill and the anchoring trenches were design differently, with concrete beams with a strip of geomembrane where the liner was welded.

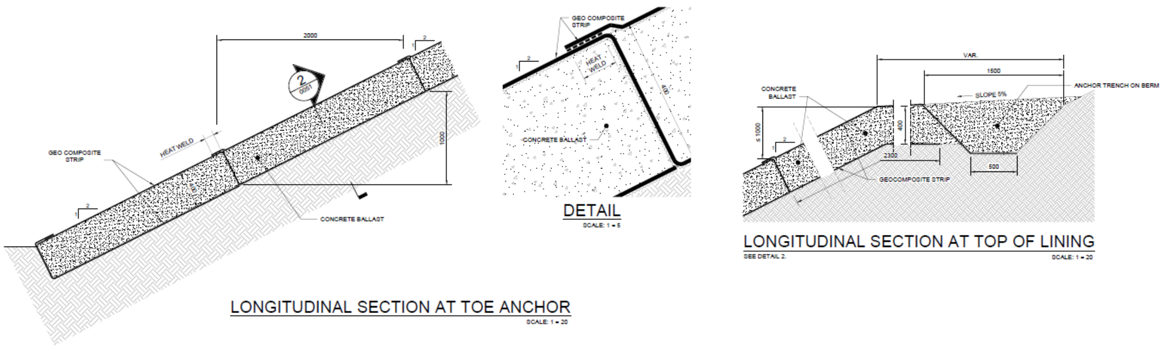

Figure 7: Anchor trench Pacific site

5.2 Deep anchors and soil nailing anchors

The deep anchors were designed for the fastening and tensioning of the geocomposite liner on vertical excavations protected by shotcrete, and consist in 1.0 m long deep anchors grouted on the rock. On concrete structures, the tensioning profiles were fixed by 15.0 cm long chemical anchors. At the Atlantic site, in fragmented rock and weathered rock, the anchoring of the geocomposite liner on the slopes was performed with soil nailing, by grouted anchors, 2.0 m long. These deep anchors hold in place the geocomposite liner by means of a stainless steel plate at the contact with the soil.

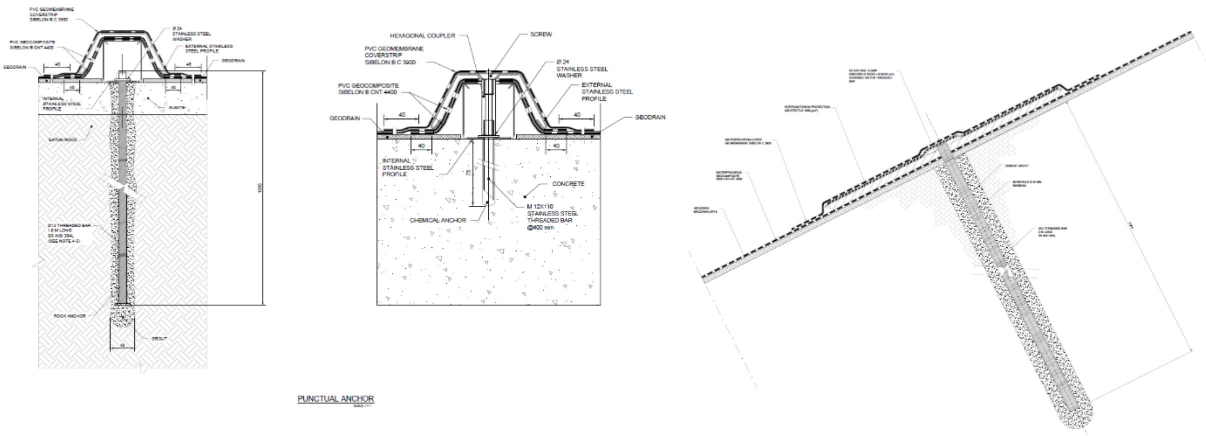

Figure 8: Deep anchors

6 Conclusion

The installation of the waterproofing system of the Water Saving Basins was executed in simultaneous with the civil works activities, in ~~a~~ permanent coordination with the main contractor, to avoid major interferences and damages to the waterproofing liner. The design underwent some optimizations and changes adapting the baseline design to the real conditions of the site; this was possible only due to the flexibility and adaptability of the waterproofing system and the expertise of the specialized contractors and designers. The installation works were completed in less than a year, in difficult weather and site conditions, and they fulfilled all the contract requirements and passed all tests on completion.

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The Use of Geosynthetics in the Construction and Rehabilitation of Transportation Infrastructures in Portugal

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1 Introduction

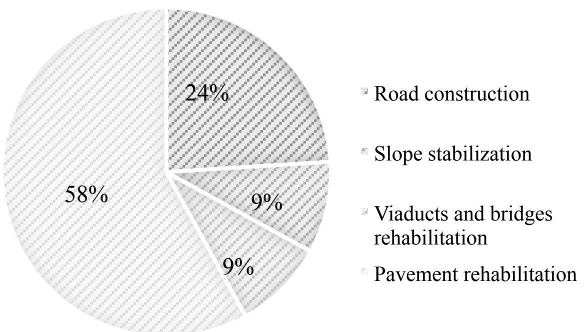
The main purpose is to summarize de use of geosynthetics in the construction and rehabilitation of road pavements and rail tracks in Portugal. The summary of the Portuguese experience was performed by the Work Group WG2 – Reinforcement of Geomaterials and its Implications in Pavement and Rail Track Design – of the Portuguese Committee on Transportation Geotechnics (GEOreinforce@, 2016).

2 Portuguese experience in the use of geosynthetics

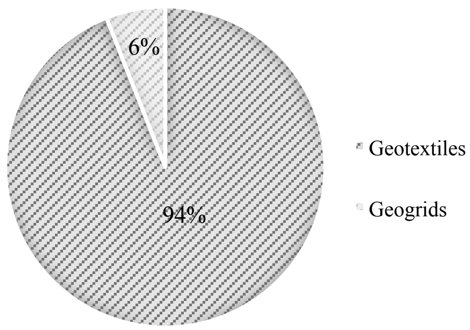
2.1 Road pavements

The Portuguese government has promoted an important activity of increase and benefit of the main road network in last three decades, since EU adhesion. In the end of 2012, this road network was a total length of about 14,284 km of which 2,988 km were motorways, 6,505 km were main or national roads, and 4,791 km were secondary or regional roads. In 1990, the total length of motorways was only 316 km (EU, 2015).

According the Portuguese Transportation Infrastructures Administration (Lima, 2013), a total of 500,000 m^2 of geosynthetics were applied in the subgrade layer in 45 cases of road construction since 2001 to 2012. The Figure 1a shows the use of geosynthetics in different types of works: 24% in road construction, 9% in slope stabilization works, 9% in viaducts and bridges rehabilitation, and 58% in pavement rehabilitation. The increased use of geosynthetics in the rehabilitation of pavements may have been associated to the existence of soft soils in the local, where the geosynthetics can be an easier solution to implement in order to increase the global pavement stiffness. In the case of new road construction, the geosynthetics were mainly used in excavation sections. In the slope stabilization and viaducts and bridges rehabilitation works, the geosynthetics were sometimes useful in the reconstruction of the pavement in the bridge or viaduct approach embankment. The Figure 1b presents the type of geosynthetics used in the pavement subgrade: in 94% were used geotextiles and in 6% were used geogrids.



(a)



(b)

Figure 1: Distribution of the use of geosynthetics: by type of use (a) and by type of geosynthetics (b) (Lima, 2013)

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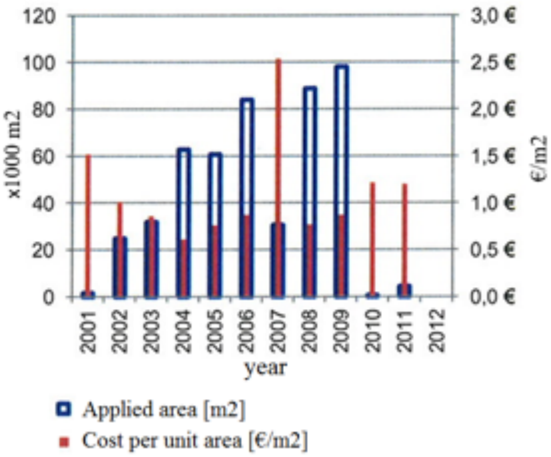
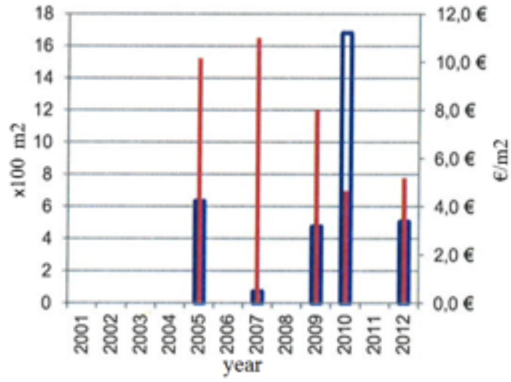
The Figure 2 shows the quantities (area) and the costs per unit area related to the use of geotextiles and geogrids in the pavement subgrade (Lima, 2013). A significant fluctuation is observed for both materials since 2001 to 2012. During this period, 45,000 m^2/year of geosynthetics have been applied in various types of works, of which 44,300 m^2 were geotextiles and 700 m^2 were geogrids. This confirms the predominate use of geotextile. The average annual cost per unit area, in the same period, for the geotextile was 1.50 €/m^2 and for the geogrid was 7.80 €/m^2, including all the costs concerning the supply and application processes.



(a)



(b)

Figure 2: Quantities and costs of geotextiles (a) and geogrids (b) (Lima, 2013)

2.2 Rail tracks

In 2015, the rail network in Portugal was composed by 2,546 km of railway lines in operation: 1,935 km of single track and 611 km of multiple track. Since 1990, when the railways had a total length of 3,582 km, a decrease of the lines in operation was observed.

In general, the geotextiles have been applied in the rehabilitation of the existing rail tracks covering various functions: separation, reinforcement, drainage, and filtration.

In 2007, an experience of reinforced railroad ballast with geogrid was performed in the North Line, between the km 73,160 and km 73,500. The North Line is the main railway line in Portugal, connecting the two main cities: Lisbon and Oporto. In this case, a quantity of 3,390.33 m^2 of geogrid was used. The total cost of application was 5.00 €/m^2.

In 2016, an important section of the North Line is being rehabilitated using biaxial geogrid-reinforced ballast, between the km 194,500 and km 218,000 (Alfarelos-Pampilhosa). In this case, single geogrid and composite of geogrid and nonwoven geotextile are being used: 8,740 m^2 of geogrid; 34,580 m^2 of geogrid and nonwoven geotextile. The Figure 3 presents the cross-section of the rehabilitated rail track through the placement of the geogrid under the ballast layer. This section has a mixed of rail traffic (passengers and cargo), corresponding to a load design of 22.5 ton/axle applied over twin-block concrete sleepers.

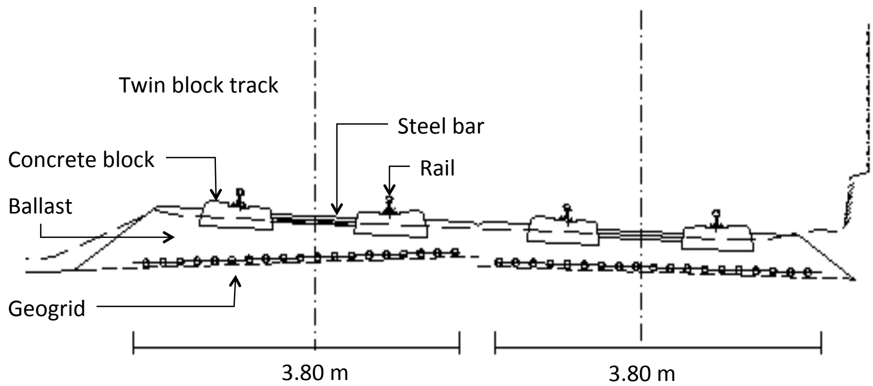


Figure 3: Cross-section of the geogrid-reinforced rail track

The Figure 4 presents the in situ reinforcement technique of the existing rail track with geogrid, performed during the night period. This figure shows a track-mounted undercutting machine that rolls out the geogrid prior to new ballast being dropped in place over the geogrid.

Figure 4: Reinforcement technique of the railroad ballast with geogrid

3 Conclusions

The paper has presented a description of the main type applications of geosynthetics in transportation infrastructures in Portugal: road pavements and rail tracks.

During the last three decades an important increase and benefit of the main road network was implemented. In the case of soft subgrade and in order to improve the pavement bearing capacity, the use of geosynthetics was often a suitable solution. Since 2001 to 2012, a total of 500,000 m^2 of geosynthetics – geotextiles and geogrids – were applied in the subgrade layer: road construction, slope stabilization, viaducts and bridge rehabilitation, and pavement rehabilitation.

Concerning the rail network and in the case of the rehabilitation of the existing railway lines in operation, the geotextiles have been used with various functions: separation, reinforcement, drainage, and filtration. However, the geogrids are only being applied as reinforcement with more significance since 2016.

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