Workshop 1: Geosynthetics in Transportation Geotechnics

Presentations e-Book

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Sponsored by
Organizing Committee

Fumio Tatsuoka | Tokyo University of Science, Japan
Jorge Zornberg | Texas University at Austin, USA
José Luís Machado do Vale | IGS, Portugal
José Neves | IST, University of Lisbon, Portugal

Organized by

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Contact
José Neves (Portugal)
Email: jose.manuel.neves@tecnico.ulisboa.pt

Universidade do Minho
Departamento de Engenharia Civil, Azurém, P-4800-058 Guimarães, Portugal
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Preface

Geosynthetic-reinforced soil structures, the use of geosynthetics in pavement and related engineering are now one of the indispensable 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.
• Key issues in practice.
• Perspective.

This book contains the oral presentations and was prepared from the input files supplied by the authors. The order of the oral presentations follows the definitive programme of the workshop.

Fumio Tatsuoka | Jorge Zornberg | José Luís Machado do Vale | José Neves
Venue and location ...

Workshop 1: Geosynthetics in Transportation Geotechnics
Audience ...

Sponsors exhibition ...

Workshop 1: Geosynthetics in Transportation Geotechnics
Lunch ...

Restaurant ...

Workshop 1: Geosynthetics in Transportation Geotechnics
SESSION 1.A
Chair | Fumio Tatsuoka

1 | Research and Construction of Geosynthetic-Reinforced Soil Integral Bridges
Keynote speaker | Fumio Tatsuoka

2 | The First GRS Integral Bridge with FHR Facing in Europe – Experiences from Design and Construction
Speaker | Stanislav Lenart

3 | Modelling Geogrid-reinforced Railway Ballast Using the Discrete Element Method
Speaker | Cholachat Rujikiatkamjorn
SESSION 1.B
Chair | Fumio Tatsuoka

4 | Performance Improvement of Rail Track Structure using Artificial Inclusions - Experimental and Field Studies
Speaker | Sinniah K. Navaratnarajah

5 | Basal Reinforced Piled Embankments
Speaker | Suzanne J.M. van Eekelen
SESSION 2.A
Chair | Jorge Zornberg

6 | Geosynthetics with Enhanced Lateral Drainage Capabilities in Roadway Systems
   Keynote speaker | Jorge Zornberg

7 | Effect of Geogrid on Railroad Ballast Particle Movement
   Speaker | Hai Huang

8 | Geosynthetic Subgrade Stabilization – Field Testing and Design Method Calibration
   Speaker | Eli Cuelho
SESSION 2.B
Chair | Jorge Zornberg

9 | Contact Pressure Distribution on Weak Subgrades due to Repeated Traffic on Geocell Reinforced Base Layers
Speaker | Jorge Zornberg

10 | The Use of Geosynthetics in Water Conveyance Structures - The Panama Canal Expansion Project, Third Set of Locks Water Saving Basins
Speaker | José Luís Machado do Vale

11 | The Use of Geosynthetics in the Construction and Rehabilitation of Transportation Infrastructures in Portugal
Speaker | José Neves
Workshop 1 – Geosynthetics in Transportation Geotechnics
Research and Construction of Geosynthetic-Reinforced Soil Integral Bridges

Fumio Tatsuoka\textsuperscript{1}, Masaru Tateyama\textsuperscript{2}, Masayuki Koda\textsuperscript{3}, Kenichi Kojima\textsuperscript{2}, Toyoji Yonezawa\textsuperscript{4}, Yoshinori Shindo\textsuperscript{4} and Shin-ichi Tamai\textsuperscript{4}

\textsuperscript{1} Tokyo University of Science, Chiba, Japan (presenting author)  
\textsuperscript{2} Railway Technical Research Institute, Tokyo, Japan  
\textsuperscript{3} East Japan Railway Company  
\textsuperscript{4} Japan Railway Construction, Transport and Technology Agency
GRS Integral Bridge

1. GRS wall
2. Full Height Rigid facing
3. Girder

Structural integration

Gravel bags

Firmly connected

0. Ground improvement (when necessary)
GRS integral bridge at Haipe, Sanriku Railway

Geogrid-reinforced Cement-mixed gravelly soil

F: Foundations of the collapsed bridge

6 April 2014
Contents

1. Advantages of GRS RWs with staged-constructed full-height rigid facing
   – the basic technology for GRS integral bridge

2. Recent GRS structures for railways in Japan
   - from GRS RWs toward GRS integral bridges

3. GRS integral bridge - the latest GRS technology

4. Concluding remarks
Contents

1. Advantages of GRS RWs with staged-constructed full-height rigid facing
   – the basic technology for GRS integral bridge

2. Recent GRS structures for railways in Japan
   - from GRS RWs toward GRS integral bridges

3. GRS integral bridge - the latest GRS technology

4. Concluding remarks
Conventional RW is a cantilever structure

Large forces in the facing & large overturning moment & large lateral load at the facing bottom

Needs for a massive & strong facing & a pile foundation

Relatively low stability, particularly against seismic loads
1995 Kobe Earthquake
Collapse of gravity type walls at Ishiyagawa

The wall had been seismic-designed against $k_h = 0.2$ with $F_s = 1.5$, but collapsed!
Immediately after completion, 1992

GRS RW with a FHR facing for a rapid transit at Tanata

A week after the 1995 Kobe Earthquake

The wall survived!
Two basic force equilibriums with reinforced soil walls: (A) along the potential active failure plane → always considered in design

Reinforcement

Potential active failure plane

Active zone

Facing

Tensile force in reinforcement

Active earth pressure, $P_A$
Two basic force equilibriums with reinforced soil walls:
(A) along the potential active failure plane → always considered in design
(B) at the facing → very important, but often ignored

Reinforcement

Potential active failure plane

Active zone

Facing

Tensile force in reinforcement

Active earth pressure, $P_A$

Paramount importance of connection strength
Available tensile forces when the connection strength is zero, or if the facing is very flexible

→ No earth pressure at the wall face

→ Low tensile forces in the reinforcement, in particular at the low wall level

→ In the active zone, low confining pressure, therefore, low soil strength

→ Low stability of the wall
Available tensile forces when the facing is rigid enough & the connection strength is high enough

→ High earth pressure at the wall face

→ High tensile forces in the reinforcement

→ In the active zone, high confining pressure, therefore, high soil strength

→ High stability of the wall
FHR facing versus discrete panel/block facing

- Electric pole
- Noise barrier
- FHR facing
- Backfill
- Geogrid reinforcement
- Noise barrier
- Crash barrier
- Additional fill
- Modular block facing
- Reinforcement
4th June 2015, collapse of a bridge by the dislodging of the girder from the top of the discrete panel facing of a Terre Armee wall, IS-85 in Lusk, Wyoming, USA (Chadrad. com. KCSR):

- Flood in the nearby river \(\Rightarrow\) Scouring in the subsoil supporting the facing \(\Rightarrow\) Displacement/deformation of the facing \(\Rightarrow\)
- Displacement of the support of the girder \(\Rightarrow\) Dislodging of girder-
28 July 2013
Collapse of many soil structures of railway (Yamaguchi line) by heavy rainfall and flood and their restoration to GRS structures→

Locations of GRS RWs with a stage-constructed FHR facing as of June 2014
Flood from a river attacking the embankment
Collapse of embankment by scouring at the toe

Railway (Yamaguchi line)

30 cm

80 cm

Workshop 1: Geosynthetics in Transportation Geotechnics
Restoration to GRS structure

- Geogrid layers
- GRS RW with FHR facing
- Slope 1:1.5
- 30 cm
- 8 cm
- Replaced gravel layer
- Earthwork: about 13,000 m³
Restoration to GRS structure

Designed against flood and seismic load

FHR facing has a strong resistance against the scoring by flood at the wall toe
FHR facing has a strong resistance against the scoring by flood at the wall toe.
3D effects of full-height rigid (FHR) facing!

Each unit of “FHR facing together with reinforced backfill” located between construction joints behaves as a monolith.

→ Even if local failure is going to take place somewhere in the wall, it does not develop towards the collapse of the whole wall.
3D effects of full-height rigid (FHR) facing!

Against lateral load $H$, each unit of FHR facing together with reinforced backfill behaves as a monolith.

→ A FHR facing becomes a foundation for super-structures, such as electric poles, noise barrier walls, bridge girders etc.
FHR facing increases the stability against concentrated load on the wall crest

Tatsuoka et al. (1989) 12ISMFE, Rio de Janeiro
GRS RW with a full-height rigid (FHR) facing:

The FHR facing is "a continuous beam supported by reinforcement layers at many levels and a small span"

- Very small forces in the facing ⇒ simple facing structure
- Small overturning moment & lateral force at the bottom of the facing ⇒ usually, no need for a pile foundation!
- Stable, particularly against seismic loads→
**Conventional technologies**

- Electric pole
- Noise barrier
- Ballast track
- Unreinforced backfill
- Pile foundation
- RC wall structure
- Noise barrier
- Crash barrier
- RC viaduct

**MSE technologies with discrete facing**

- Noise barrier
- Crash barrier
- Additional fill
- Modular block facing
- Backfill
- Reinforcement
- Girder
- Sill beam
- Modular block facing
- Reinforcement

- **Limited occupied space**
- **Facing supports super structures**

- **More cost-effective, but**
- **Larger occupied space**
- **Facing does not support super structures effectively**
Conventional technologies

- Electric pole
- Noise barrier
- Ballast track
- RC wall structure
- Unreinforced backfill
- Pile foundation

RC viaduct

- Noise barrier
- Crash barrier
- RC wall structure
- Unreinforced backfill

- Limited occupied space
- Facing supports supper structures

GRS technologies with staged constructed FHR facing

- Electric pole
- Noise barrier
- FHR facing
- Backfill
- Geogrid reinforcement

- Crash barrier
- RC slab roadbed
- FHR facing
- Backfill
- Geogrid reinforcement

GRS integral bridge

- Structural integration
- 3. Girder
- 2. FHR facing
- 1. GRS wall
- 0. Ground improvement (when necessary)

- And more cost-effective!
JR Kobe line, Amagasaki:

In this case, ballasted track
The functions of facing (summary)

1) The facing is an important and essential **structural component** confining the backfill and developing large tensile forces in the reinforcement.

2) The earth pressure at the facing should be **high** enough to provide sufficient confining pressure to the backfill.

3) The facing should be flexible enough to accommodate the deformation of supporting ground during construction, but **should be rigid enough during service**. This can be achieved by **staged-construction**.
Staged construction of FHR facing

- Why necessary
- How to do
- Benefits ➔
Staged construction - 1:
- Construction with a help of gravel gabions placed at the shoulder of each soil layer

1) Leveling pad for facing
2) Placing geosynthetic & gravel gabions
Staged construction - 2:
- Construction with a help of gravel gabions placed at the shoulder of each soil layer

1) Leveling pad for facing
2) Placing geosynthetic & gravel gabions
3) Backfilling & compaction
4) Second layer

Good compaction of the backfill by:
1) a small lift resulting from a small vertical spacing of reinforcement layers; and
2) no rigid facing during backfill compaction
Staged construction - 3:
- Construction with a help of gravel gabions placed at the shoulder of each soil layer

1) Leveling pad for facing
2) Placing geosynthetic & gravel gabions
3) Backfilling & compaction
4) Second layer
5) Completion of wrapped-around wall
6) Casting-in-place RC facing
Staged construction - 4:
- After sufficient compression of backfill and supporting ground has taken place, a full-height rigid facing is constructed by casting-in-place concrete directly on the wrapped-around wall.

→ The facing/ reinforcement connection is not damaged during and after construction.

→ Construction using compressive backfill on a compressive soil layer becomes possible.
Casting-in-place concrete directly on the geogrid-wrapping-around wall face:

1) Fresh concrete enters the gravel bags through the aperture of the geogrid (PVA has very high resistance against high PH).

2) A firm connection between the facing and the reinforcement is ensured (PVA has a good adhesiveness with concrete; and the bi-axial structure enhances the connection strength).

Typical polymer geogrid: bi-axial made of PVA.

5) Completion of wrapped-around wall

6) Casting-in-place RC facing

Firm connection
Staged construction - 4:
- After sufficient compression of backfill and supporting ground has taken place, a full-height rigid facing is constructed by casting-in-place concrete directly on the wrapped-around wall.

→ The facing/reinforcement connection is not damaged by differential settlement between the facing and the reinforcement during and after construction.

→ Construction using compressive backfill on a compressive soil layer becomes possible.
Nagano wall:
- for a yard for Shinkansen (bullet train)
- constructed 1993 - 1994
Nagano wall:
- for a yard for Shinkansen (bullet train)
- constructed 1993 - 1994

- GRS RW, 2 m-high & 2 km-long, supporting a yard for a new bullet train line
- Backfill: nearly saturated soft clay
- Constructed on a thick very soft clay deposit
  - no pile foundation
  - staged construction
    1) GRS wall w/o FHR facing
    2) preload fill
    3) settlement
    4) removing preload fill
    5) FHR facing

(Tatsuoka et al., 1997)
- Settlement of the embankment by preloading: about 1 m
- Casting-in-place of FHR facing after removing the preload fill.

GRS RW during preloading (height) before preloading: 3.5 m after preloading: 2.5 m
20 years after construction, 6th July, 2014
Staged construction - 5:
- Completed.

Re-construction of an existing slope to a vertical wall for a yard of high-speed train at Biwajima, Nagoya
A yard of high-speed trains at Biwajima, Nagoya, 1989 - 1990
- Average wall height= 5 m & total length= 930 m
GRS RW with a full-height rigid (FHR) facing supporting very busy urban trains in Tokyo

Near Shinjuku Station, Tokyo, constructed during 1995 – 2000
The functions of facing (summary)

1) The facing is an important and essential **structural component** confining the backfill and developing large tensile forces in the reinforcement.

2) The earth pressure at the facing should be **high** enough to provide sufficient confining pressure to the backfill.

3) The facing should be flexible enough to accommodate the deformation of supporting ground during construction, but **should be rigid enough during service**. This can be achieved by staged-construction.
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3. GRS integral bridge - the latest GRS technology

4. Concluding remarks
Three generations of elevated structures for HST lines (Shin-kansen) in Japan

**Embarkment**; 54% of the total length

- Gentle slope

**Conventional type RWs**

- Embarkment; 54% of the total length
- [Tokaido, opened 1964]
  - Often stop/speed-down of train by heavy rainfalls
  - Low stability against earthquakes
  - Larger deformation and bumps behind bridge abutments,
    → very costly long-lasting maintenance & reinforcing
  - Occupation of wide space

- [From 1972, Sanyo, Joetsu & Tohoku]
  - High cost
  - No use of excavated soil

- Where relevant

- [Since 2000]
  - High stability (rainfalls, earthquakes)
  - High cost-effectiveness in construction & maintenance

- RC viaduct

- GRS RW with FHR facing

(basically no piles)
Total wall length: 167 km
Total number of site:
- GRS RWs: 1139 (two in Vietnam)
- GRS bridge abutments: 35
- GRS integral bridges: 5
Zero problematic case during and after construction

No. of GRS RWs: 250 (4) → 216 (1) → 31 (2) → 85 (18) → 139 (8) → 155 (1) → 26
(No. of GRS bridge abutments & GRS integral bridges)

Locations of GRS RWs with a stage-constructed FHR facing as of June 2016
From 1982: research at the University of Tokyo & Railway Technical Research Institute

2004 Niigata-ken-Chuetsu Earthquake

Restart of construction of new bullet train lines

1995 Kobe Earthquake

2011 Great East Japan EQ
GRS RWs with FHR facing for railways, including high-speed trains, that had been constructed in the affected area of the 2011 Great East Japan EQ

<table>
<thead>
<tr>
<th>Location</th>
<th>Count</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aomori</td>
<td>58</td>
</tr>
<tr>
<td>Iwate</td>
<td>23</td>
</tr>
<tr>
<td>Miyagi</td>
<td>9</td>
</tr>
<tr>
<td>Akita</td>
<td>1</td>
</tr>
<tr>
<td>Yamagata</td>
<td>3</td>
</tr>
<tr>
<td>Fukushima</td>
<td>1</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>95</strong></td>
</tr>
</tbody>
</table>

- Designed against very high seismic load (level 2); and
- No damage to all the GRS RWs.

These facts validate the current seismic design code for railway RWs.

Adjacent to Natori River,
Sendai City
Completed 1994
Wall length= 400 m
Hokkaido High-Speed Train Line (Shin-kansen)

Opened in the beginning of 2016

A number of GRS structures were densely constructed in place of conventional type structures

Construction started 2013

Nearly completed
Various GRS structures at Montaro

Hokkaido Shinkansen

<table>
<thead>
<tr>
<th>GRS structures</th>
<th>Length or number</th>
<th>Max. height (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R GRS RW</td>
<td>3,528 m</td>
<td>11.0</td>
</tr>
<tr>
<td>A GRS abutment</td>
<td>29</td>
<td>13.4</td>
</tr>
<tr>
<td>I GRS integral bridge</td>
<td>1</td>
<td>6.1</td>
</tr>
<tr>
<td>B GRS box culvert</td>
<td>3</td>
<td>8.4</td>
</tr>
<tr>
<td>T GRS tunnel protection</td>
<td>11</td>
<td>12.5</td>
</tr>
</tbody>
</table>

(By the courtesy of Japan Railway Construction, Transport and Technology Agency)
Typical GRS RW for Hokkaido HST Line: immediately after RC facing was constructed by casting-in-place concrete
For Hokkaido HST line:

1) GRS retaining walls having full-height rigid facing for a length of 3.5 km, totally in place of conventional type cantilever RWs

2) 29 GRS bridge abutments, totally in place of conventional type bridge abutments

3) A GRS integral bridge

4) Three GRS box culvert structures integrated to GRS RWs

5) Eleven GRS protection structures at the tunnel entrance
Cost Ratio: GRS RW versus conventional type RW

<table>
<thead>
<tr>
<th></th>
<th>Construction</th>
<th>Maintenance</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 m-thick relatively soft ground:</td>
<td>0.32</td>
<td>0.5</td>
<td>0.33</td>
</tr>
<tr>
<td>- piles for conventional type RWs</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- no piles for GRS RWs</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(the case shown in the figure)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Relatively stiff ground:</td>
<td>0.81</td>
<td>0.51</td>
<td>0.77</td>
</tr>
<tr>
<td>- no piles for conventional type</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RWs &amp; GRS RWs</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

All units in m

Cost = 1.0

Designed by using \( (k_h)_{\text{design}} = 0.2 \)

Cast-in-place piles

Backfill: soil density: 2.0 g/cm³
\( ? = 35^\circ, \ c = 0 \)

Geogrid \( (T_1 = 37.5 \text{ kN/m}) \)

Leveling concrete
Gravel blanket

Ground (clayey soil, N=10,
20 m-thick)
soil density: 1.6 g/cm³
\( ? = 0, \ c = 62 \text{ kPa} \)
Summary:

Why GRS structures have become the standard soil structures for Japanese railways replacing conventional type embankments, RWs and bridges?

1. Higher performance
   - for a long term; and
   - against earthquakes, heavy/prolonged rainfalls, floods …

2. Lower cost for:
   - construction; and
   - long-term maintenance
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4. Concluding remarks
Technical problems with conventional type bridges

1. Piles
   - Ground settlement & lateral flow by the weight of backfill, and associated negative friction & bending of the piles

2. RC abutment
   - Displacement by earth pressure

3. Backfill
   - Earth pressure

4. Bearings (fixed or movable)
   - High cost for construction & long-term maintenance

5. Girder
Technical problems with conventional type bridge ➔

1. Piles ➔
2. RC abutment ➔
3. Backfill ➔
4. Bearings (fixed or movable) ➔ Low seismic stability ➔
5. Girder

Settlement by self-weight, traffic load & seismic load ➔

Displacement by seismic earth pressure ➔

Ground settlement & lateral flow by seismic load ➔

What is a solution?
Towards GRS Integral bridge:

- Problems with conventional type bridges

- Integral bridge; a structural engineering solution

- GRS RW bridge; a geotechnical engineering solution

- GRS Integral bridge;
  - the solution
  - Importance of strong connection between the reinforcement and the full-height rigid facing

Conventional type

To solve several problems with backfill

To solve several problems with RC structures

GRS RW

Combined

Integral

GRS Integral
Integral bridge

1. Piles

2. RC facing

3. Continuous girder Integration

4. Backfill

Low construction & maintenance cost of the structural part, due to:
- a) no use of girder bearings &
- b) the use of a continuous girder
However, several unsolved old problems!

Long-term service issue:

a. settlement by self-weight & traffic load
b. large deformation by seismic load

1. Piles

2. RC facing

3. Continuous girder integration

4. Backfill

Displacement due to earth pressure

Earth pressure (static & dynamic)

Ground settlement & lateral flow by the weight of backfill, and associated negative friction & bending of the piles.
New problems with integral bridges!

- Seasonal thermal expansion & contraction of the girder
- Lateral cyclic displacements
- Settlement*
- Increase in the earth pressure*

* Due to the dual ratcheting mechanism
Static lateral cyclic loading tests under plane strain conditions in 1g (considered model scale: 1/10)
Unreinforced backfill:

A significant increase in the passive earth pressure with cyclic loading
Unreinforced backfill:
Significant settlement in the backfill with cyclic loading

Why?
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Dual ratchet mechanism

S → A1

Formation of active wedge

No major deformation outside the active wedge

a)

A1 → P1

Active wedge deforms as part of passive wedge, but not recovering the active displacement during S → A1

Formation of passive wedge

b)
Dual ratchet mechanism

- **P1 → A2**
  - Reactivation of active displacement of the active wedge
  - No displacement & deformation of the passive wedge outside the active wedge

- **A2 → P2**
  - Reactivation of passive deformation of the passive wedge
Towards GRS Integral bridge:

- Problems with conventional type bridges

- Integral bridge; a structural engineering solution

- **GRS RW bridge; a geotechnical engineering solution**

- GRS Integral bridge;
  - the solution
  - Importance of strong connection between the reinforcement and the full-height rigid facing
A better solution: GRS bridge abutment, placing the girder on the top of the facing via bearings

1. GRS RW
2. FHR facing
3. Movable & fixed bearings
4. Girder
A New High-Speed Train Line in Kyushu Island

Existing bullet train line

Bullet train line under planning

Hakata

Takada

Nagasaki

Nishi Kagoshima

The first bridge abutment at Takada, completed March 2003.
- Vertical loading test to ensure the vertical bearing capacity at the base of the RC facing
- Lateral loading test to ensure the connection strength and the stability of the RC facing
A 13.4 m-high GRS-RW bridge abutment at Mantaro for a new high-speed train line, the south end of Hokkaido

**Why not removing the bearings?**

In total, about 60 GRS RW bridge abutments completed or designed (as of June 2012)

- Yet, still problems by using bearings (i.e., high cost for construction/maintenance & low seismic stability)
Towards GRS Integral bridge:

- Problems with conventional type bridges
- Integral bridge; a structural engineering solution
- GRS RW bridge; a geotechnical engineering solution
- GRS Integral bridge;
  ▶ the solution
  ▶ Importance of strong connection between the reinforcement and the full-height rigid facing
The current best solution: GRS Integral Bridge

- **0. Ground improvement (when necessary)**
- **1. GRS wall**
- **2. FHR facing**
- **3. Girder**

**Structural integration**

Firmly connected
NR: not reinforced

Effects of thermal cyclic deformation of the girder on the stability of backfill

Number of loading cycles, N (cycles)

Residual settlement of the backfill at 5 cm back of the facing, $S_g/H (%)$

D/H = 0.2 or 0.6 %

NR (no reinforcement): D/H = 0.2 %

NR (not reinforced)

H = 50.5 cm

$S_g/H (%)$
NR: not reinforced

R&NoC: reinforced, but no connection

This is not a solution!

Residual settlement of the backfill at 5 cm back of the facing, $S_g/H$ (%)

Number of loading cycles, $N$ (cycles)

D/H= 0.2 or 0.6%

Hinge-support

NR: not reinforced

R & NoC: reinforced, but no connection

NR: D/H= 0.6%

R & NoC: D/H= 0.6%

NR (no reinforcement): D/H= 0.2%

H= 50.5 cm
NR: not reinforced

R&NoC: reinforced, but no connection

R&C: reinforced; and connection

**Residual settlement of the backfill at 5 cm back of the facing, \( S_{g}/H \) (%)**

- **NR (no reinforcement):** D/H = 0.6%
- **R & NoC (Reinforced & No Connected):** D/H = 0.2%
- **R & C (Reinforced & Connected):** D/H = 0.6%

**Number of loading cycles, N (cycles)**

- **NR (no reinforcement):** D/H = 0.2%
- **R & NoC:** D/H = 0.6%
- **R & C:** D/H = 0.6%
Shaking table tests in 1g  
(considered model scale: 1/10)

D: displacement transducer  
M: movable (sliding) shoe  
F: fixed (hinged) shoe  
L: L shaped metal fixture

Mass of 205 kg (equivalent length of model girder= 2.0m)

Conventional type

Integral bridge

GRS RW bridge

GRS Integral bridge

Width= 60 cm

Nine local load cells.

Five local load cells

Target for photogrammetric method

Grid reinforcement

Connected

Mass of 205 kg (equivalent length of model girder= 2.0m)

Unit: cm
The stability of GRS integral bridge is usually controlled by the connection failure between the facing and the reinforcement.
Residual settlement at the backfill in shaking table tests

![Diagram showing settlement at the backfill](image)

- **Conventional**
- **GRS RW**
- **Integral**
- **Most stable**

Graph showing:
- Base acceleration, $\alpha_{\text{max}}$ (gal)
- Settlement of the backfill, $S_5$ (mm)

- **GRS RW**
- **Integral**
- **Out of measurement range**

A very high dynamic stability of GRS Integral bridge
A full-scale model of GRS integral bridge, completed Feb. 2009 at Railway Technical Research Institute, Japan

27 November 2008

April. 2009
Cyclic lateral loading tests applying
1) thermal deformation of the girder; and
2) level 2 design seismic loads (Jan, 2012)
A number of GRS structures were densely constructed in place of conventional type structures.

GRS Integral Bridge, constructed 2011 - 2012

Construction started 2013

Nearly completed
First full-scale GRS integral bridge, for a new high-speed train line, Kikonai at the south end of Hokkaido

(14th Oct. 2011)
First full-scale GRS integral bridge, for a new high-speed train line, Kikonai at the south end of Hokkaido

[Diagram of the bridge structure with dimensions and annotations]

(31 July 2012)
Great tsunami
2011 Great East Japan Earthquake
Damage to over 340 bridges by great tsunami during the 2011 Great East Japan Earthquake

A railway bridge (Tsuyano-gawa bridge) that lost multiple simple-supported girders by tsunami forces
Girder bearings and approach fill are two major weak components of bridge for seismic & tsunami forces

A solution by GRS integral bridge
Total wall length: 158 km
Total number of site:
- GRS RWs: 1017 (one in Vietnam)
- GRS bridge abutments: 33
- GRS integral bridges: 4
Zero problematic case during and after construction

Locations of GRS RWs with a stage-constructed FHR facing as of June 2014
Sanriku Railway:
- constructed 30 years ago taking into account tsunami effects.
- However, three bridges were lost by the tsunami during the 2011 Great East Japan EQ.
Sanriku Railway:
- Constructed 30 years ago taking into account tsunami effects.
- However, three bridges were lost by the tsunami during the 2011 Great East Japan EQ.

Immediately after the earthquake at Koikoreobe

30 March 2011
Comparison among different bridge types at Koikorobe

- **Two-span simple bridge (the same as the one collapsed by tsunami)**
  - Construction & maintenance cost: Relatively high
  - Unreinforced cement-mixed gravelly soil
  - Survived abutment, New abutment

- **Single continuous girder bridge with a pair of bearing**
  - Construction & maintenance cost: Relatively high
  - Unreinforced cement-mixed gravelly soil
  - Survived abutment, New abutment

- **GRS integral bridge with no bearing (adopted plan)**
  - Construction & maintenance cost: Relatively low
  - Geogrid-reinforced cement-mixed gravelly soil
  - Survived abutment, New abutment
Comparison among different bridge types at Koikorobe

Two-span simple bridge (the same as the one collapsed by tsunami)

Seismic stability
- Low

Anti-tsunami stability
- Low

Single continuous girder bridge with a pair of bearing

Seismic stability
- Intermediate

Anti-tsunami stability
- Low

GRS integral bridge with no bearing (adopted plan)

Seismic stability
- High

Anti-tsunami stability
- High

Highest performance/cost ratio
GRS integral bridge at Koikorobe for Sanriku Railway

Geogrid-reinforced Cement-mixed gravelly soil

F: Foundations of the collapsed bridge

19.93 m

P1

1.8 m

1.2 m

A1

1.2 m

To south

Ground improvement

4.7 m

Local road

5.0 m

Bed rock

6.5 m

6.5 m

5.0 m

4.7 m

6.5 m

Local road

6.5 m

A2

19.93 m

Koikorobe stream

Koikorobe

Bed rock

6.5 m

3 November 2013

Workshop 1: Geosynthetics in Transportation Geotechnics
GRS integral bridge at Koikorobe for Sanriku Railway

Geogrid-reinforced Cement-mixed gravelly soil

F: Foundations of the collapsed bridge

6 April 2014
Workshop 1: Geosynthetics in Transportation Geotechnics
Haipe, Sanriku Railway

Immediately after the earthquake

Tunnel for railway

30 March 2011
GRS integral bridge at Haipe, Sanriku Railway

22 May 2013
GRS integral bridge at Haipe, Sanriku Railway

Geogrid-reinforced Cement-mixed gravelly soil

F: Foundations of the collapsed bridge

27.8 m

2.1 m

2.2 m

4.7 m

4.7 m

32.16 m

Local road

Haipe stream

Ground improvement

Bed rock

4.5 m

8.5 m

8.5 m

To south
GRS integral bridge at Haipe, Sanriku Railway

Geogrid-reinforced Cement-mixed gravelly soil

F: Foundations of the collapsed bridge

Bed rock

Local road

4.7 m

Haipe stream

Ground improvement

27.8 m

32.16 m

2.1 m

2.2 m

4.5 m

8.5 m

6 April 2014
GRS integral bridge at Haipe, Sanriku Railway

Geogrid-reinforced Cement-mixed gravelly soil

F: Foundations of the collapsed bridge

Bed rock

Ground improvement

Local road 4.7 m

Haipe stream

27.8 m

32.16 m

2.1 m

2.2 m

20 May 2014
Two major components of wall deformation:

TG: Thermal (annually cyclic) deformation of the girder

SC: Drying shrinkage of concrete; relatively large initially, gradually decreasing with time.

From the second year, reversible cyclic displacements with a relatively small amplitude
Two major components of wall deformation:
TG: Thermal (annually cyclic) deformation of the girder
SC: Drying shrinkage of concrete; relatively large initially, gradually decreasing with time.

→ From the second year, reversible cyclic displacements with a relatively small amplitude

![Graph showing deflection of the girder with terms 1 to 6]
Shima-no-koshi at Sanriku Railway

Before the EQ

Immediately after the EQ,

RC frame structure (viaduct) collapsed by tsunami
Shima-no-koshi Station, Sanriku Railway (August 2011)

Railway track level: 14 m

Tunnel exit

Highest level of tsunami (about 22 – 23 m)

Seaside

Previous Shimano-koshi station
GRS embankment and GRS integral bridge, Shima-no-koshi, Sanriku Railway

AP1: approach fill (well compacted well-graded gravel)
AP2: approach fill (lightly compacted cement-mixed well-graded gravel reinforced grid layers connected to the facing)

GRS embankment also as a tsunami-barrier

Concrete slope crib work (65 cm-thick) connected to grid layers
Concrete facing (30 cm-thick) connected to grid layers
Geogrid (T= 30 kN/m)
Ground improvement by cement-mixing-in-place
GRS embankment and GRS integral bridge, Shima-no-koshi, Sanriku Railway

AP1: approach fill (well compacted well-graded gravel)
AP2: approach fill (lightly compacted cement-mixed well-graded gravel reinforced grid layers connected to the facing)

30 March 2011
GRS embankment and GRS integral bridge, Shima-no-koshi, Sanriku Railway

AP1: approach fill (well compacted well-graded gravel)
AP2: approach fill (lightly compacted cement-mixed well-graded gravel reinforced grid layers connected to the facing)

20 May 2014
AP1: approach fill (well compacted well-graded gravel)
AP2: approach fill (well compacted lightly cement-mixed well-graded gravel reinforced with grid layers connected to the facing)

30 March 2011
Workshop 1: Geosynthetics in Transportation Geotechnics

3rd ICTG 2016
04-07 September 2016, Guimarães, Portugal

Pier of collapsed bridge (removed)

Masonry river RWs (removed)

TP 10.0m
AP1
8.0m
AP1
10.7m
AP2
5.0m
7.5m
10.7m

Tunnel

AP1: approach fill (well compacted well-graded gravel)
AP2: approach fill (well compacted lightly cement-mixed well-graded gravel reinforced with grid layers connected to the facing)

GRS integral bridge

Box culvert

Old abutment (removed)

19 June 2013
AP1: approach fill (well compacted well-graded gravel)
AP2: approach fill (well compacted lightly cement-mixed well-graded gravel reinforced with grid layers connected to the facing)
Conclusions – 1

Geosynthetic-reinforced soil retaining walls (GRS RWs) having a stage-constructed full-height rigid (FHR) facing have been constructed as important permanent RWs for a total length of about 160 km in Japan. It is now the standard RW technology for railways.

Its current popular use is due to high cost-effectiveness, in particular high performance during severe earthquakes, heavy rainfalls etc.; and low cost for construction and maintenance.
Conclusions – 2

A great number of embankments and conventional type RWs collapsed during severe natural disasters (i.e., earthquakes, heavy rains, floods, tsunami …). 

Many of them were reconstructed to GRS RWs with a stage-constructed FHR facing.
Conclusions – 3 ➔

**GRS integral bridge** was developed by extending the technology of GRS RW with FHR facing.

Compared with the conventional type bridge, **GRS integral bridge** is much more cost-effective with much higher with negligible bumps behind the facing and a high stability during long-term service and against natural disasters.

These features can be attributed to the staged construction of FHR facing firmly connected to the geogrid layers.

For these reasons, **GRS integral bridge** is relevant to bridges for railways and roads at many places.
The first GRS integral bridge with FHR facing in Europe – experiences from design and construction

Stanislav Lenart

Slovenian National Building and Civil Engineering Institute (ZAG)
The first GRS integral bridge in the world, constructed at high-speed train line (Kikonai, Hokkaido, Japan)
Introduction

• The use of geosynthetic reinforced soil (GRS) technology has become common practice in the design of infrastructure projects, mainly due to:
  - cost savings,
  - simple and rapid construction technique,
  - reduced construction time,
  - reduced environmental effects,
  - good seismic performance,
  - etc.
Introduction

• Long tradition of (permanent) GRS bridge abutments in Europe
  - France: Terre Armee (Vidal, 1972)
  - UK: Carmarthen in 1981 (Brady, 1987)
  - Germany: River Gera in Arnstadt, in 1996 (Herold, 2002)
  - and many more.

• Major challenges in construction of bridge supporting structures (bridge piers and abutments)
  - surcharge load applied to the top of GRS structures near to the facing
  - elimination of bridge deck bearings
  - scour protection
Introduction

- Typical maintenance problems on conventional short span bridges

Differential settlements on the bridge-embankment transition

Wing degradation

Deterioration of bearings
Introduction

• Bridge across the Pavlovski potok stream in the village of Žerovinci in north-eastern Slovenia

- rehabilitation of local traffic infrastructure (investment in railway line rehabilitation)
- box-shaped culvert
- insufficient water flow capacity
- deep layer of soft foundation soil
- very short deadlines

<table>
<thead>
<tr>
<th>Depth [m]</th>
<th>Description</th>
<th>Soil properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 – 0.5</td>
<td>sandy gravel</td>
<td>(N_{60}=6)</td>
</tr>
<tr>
<td>0.5 – 3.0</td>
<td>sandy clay with inclusions of gravel and sand</td>
<td>(N_{60}=8), (c'=1.6) kPa, (\varphi'=25.7^\circ), (w=33.5%), (I_p=14.3%)</td>
</tr>
<tr>
<td>3.0 – 5.0</td>
<td>clayey and silty sand</td>
<td>(N_{60}=12), (w=29.1%), (I_p=10.4%)</td>
</tr>
<tr>
<td>5.0 – 8.0</td>
<td>silty sand</td>
<td>(N_{60}=24)</td>
</tr>
<tr>
<td>8.0 – 11.0</td>
<td>decayed stratified marl</td>
<td>(N_{60}=36)</td>
</tr>
<tr>
<td>11.0 – 17.0</td>
<td>sandy marl</td>
<td>(N_{60}=32)</td>
</tr>
<tr>
<td>17.0 – 23.3</td>
<td>sandy-silty clay</td>
<td>(N_{60}=32)</td>
</tr>
<tr>
<td>23.3 – 26.3</td>
<td>sandy marl - solid</td>
<td>(N_{60}=32)</td>
</tr>
<tr>
<td>Water level depth: 2.7 m</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Introduction

• Bridge across the Pavlovski potok stream in the village of Žerovinci in north-eastern Slovenia
  - Reinforced concrete slab, integrated onto a pair of geosynthetic reinforced soil bridge abutments was proposed, bridge span 6.0 m
  - Recently completed research project on deformation properties of GRS provided required data for the design of the GRS bridge abutments
Design

- Two possible approaches in the integration of the bridge deck onto the top of the GRS abutment without the use of bearings:
  - 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 (Japan, Tatsuoka et al., 2009)
  - A single-span simply-supported deck is placed, without structural integration, on top of the GRS, immediately behind the facings (USA FHWA, Adams et al., 2010)
Design

Bridge deck fully structurally integrated into the top of a pair of full-height rigid (FHR) facings of GRS walls (Tatsuoka et al., 2009)

The importance of the facing-reinforcement connection !!!!
Design

• High connection strength between the reinforcement layers and the FHR facing is crucial for proper performance of GRS RWs with FHR facings

• Contractors in Slovenia (Europe?) might not have sufficient experience of stage-constructed GRS RWs with FHR facings

! high risk of low quality execution !!!

B – gabions bags
C – geosynthetic layers
D – backfill material
Thus modified solution was proposed:

Single-span deck is placed, without structural integration on top of the GRS, immediately behind the full-height rigid (FHR) facings.

Conservative but safe

Smooth bridge-embankment transition (no differential settlements)

No transfer of surcharge load from the bridge superstructure to the facing. No extra load on foundation soil.
Load scheme

• Dead weight of the structure & traffic loads
• Load model LM1: a pair of tandem axles on each conventional lane, accompanied by a uniform load, EN 1991-2)
• Bridge superstructure is supported directly at the top of the abutment as a simply-supported beam → maximum design vertical pressure 305 kPa (FEM)
• A bearing width of 0.85 m was defined
• Details (eg. space between the top of the facing and bottom of the slab deck, required geogrid tensile strength, etc.) were defined based on deformation properties of lab tested GRS specimens
Design

Workshop 1: Geosynthetics in Transportation Geotechnics
Design details

- Required geogrid tensile strength (Tf = 80 kN/m)
- Backfill material properties: c' = 0 kPa, φ' = 36°
- Bridge span 6.0 m, Abutment height: 2.75 m
- Vertical distance of reinforcement layers: 30 cm / 10 cm (intermediate layers beneath the bridge bearings)
- RC facing thickness: 15 cm (only for scour protection)
- Gap between the top of the facing and bottom of the slab deck: 8 cm
Design details

Gap between the top of the facing and bottom of the bridge superstructure (i.e. slab deck)

Facing before concreting with tube of a horizontal inclinometer and barbicans installed already
Construction

Construction of the gravel foundation, before wrapping the foundation with geosynthetics

Construction of the GRS abutments by placing gravel bags on the shoulder of each layer and compaction of the backfill
Construction
Construction
Construction: effect of reinforcement pre-stressing

(a) Procedure for stage-constructing the retaining structure without the use of a temporary supporting system

(b) construction of the full height rigid (FHR) facings by means of cast-in-situ concrete (after Tatsuoka et al., 1997):
A – the initial shallow foundation (levelling pad) for the facing,
B – the gabion bags,
C – the geosynthetic reinforcement layer,
D – the backfill material, and
H – the cast-in-situ concrete facing
Construction: effect of reinforcement pre-stressing

The measured values of the horizontal strains in one of the geosynthetic layers depending on the distance of the strain gauges from the abutment facing.
Results of observations (+)

GRS integral bridge

Conventional RC bridge with deep piled foundations

In case of conventional nearby reinforced-concrete bridge abutments, which is located 50 m upstream, deep piled foundations using piles with a diameter of 100 cm and a length of 24 m have been needed. The geosynthetic reinforced soil technology significantly reduced the construction costs and time. GRS bridge abutments can be constructed within a couple of weeks without being influenced by outside weather conditions.
Results of observations (+)

Significant *decrease of concrete* needed for GRS abutments in comparison to conventional steel-reinforced concrete abutments (67.7 % decrease)

<table>
<thead>
<tr>
<th>Element</th>
<th>Amounts of concrete needed [m$^3$]</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>RC abutments</td>
<td>GRS abutments</td>
</tr>
<tr>
<td>Piles (D=100cm L=24m)</td>
<td>75</td>
<td>-</td>
</tr>
<tr>
<td>Pile caps (120/120cm)</td>
<td>23</td>
<td>-</td>
</tr>
<tr>
<td>Abutments (d=50cm)</td>
<td>21</td>
<td>9</td>
</tr>
<tr>
<td>Wing walls (d=30cm)</td>
<td>7</td>
<td>5</td>
</tr>
<tr>
<td>Approach slabs</td>
<td>12</td>
<td>-</td>
</tr>
<tr>
<td>Superstructure</td>
<td>35.5</td>
<td>42</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>173.5</strong></td>
<td><strong>56</strong></td>
</tr>
</tbody>
</table>
Results of observations (-)

• Single-sided formwork was needed to construct the facing structure of the GRS abutments. Their implementation was rather complex.
• GRS facings were considered mostly as a scour protection measure, thus a minimum thickness, equal to 15 cm, and minimum structural reinforcement were decided. Additional problems due to relatively thin RC facing structures can arise when vibrating the cast-in-situ concrete.
• Bridge deck is constructed as a simply-supported slab, thus the internal mid-span bending moment is much greater than in the case of a frame structure. Thus more reinforcement is needed. Also, a longer RC slab has to be provided due to the necessary bearing area.
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.

• Very short deadlines and a thick layer of soft foundation soil

• 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, this technology was modified to a bridge deck placed on top of the GRS, immediately behind the FHR facings.

• The presented solution is beneficial particularly for short span bridges that need to be designed and built in a very short time.
Acknowledgements

• Co-funding of research by the Slovenian Ministry of Education, Science, Culture and Sport
• Professor Fumio Tatsuoka for his valuable advice and encouraging approach during the design and construction of the bridge
Workshop 1 – Geosynthetics in Transportation Geotechnics
Modelling Geogrid-reinforced Railway Ballast Using the Discrete Element Method

Ngoc Trung Ngo, Buddhima Indraratna, and Cholachat Rujikiatkamjorn

Centre for Geomachanics and Railway Engineering
University of Wollongong, Australia
PROBLEMS IN RAIL TRACK SUBSTRUCTURE

- Ballast Crushing
- Subgrade Clay Pumping
- Coal fouling
- Void Clogging
- Differential settlement (Courtesy, Sulker)
- Poor Drainage
THE USE OF GEOSYNTHETICS IN RAIL TRACKS

- Geogrids reinforce and confine ballast, resulting in a reduced settlement and decreased lateral movement of ballast

- Lack of availability of a comprehensive computational model to study the interaction of ballast aggregates with geogrids (i.e. interlocking / confinement effects)
Role of Ballast Fouling on Track Performance

Void Contaminant Index (VCI) proposed by UOW

\[ VCI = \frac{(1+e_f)}{e_b} \times \frac{G_{s,b}}{G_{s,f}} \times \frac{M_f}{M_b} \times 100 \]

- \( e_b \) = Void ratio of clean ballast
- \( e_f \) = Void ratio of fouling material
- \( G_{s,b} \) = Specific gravity of clean ballast
- \( G_{s,f} \) = Specific gravity of fouling material
- \( M_b \) = Dry mass of clean ballast
- \( M_f \) = Dry mass of fouling material

Sub ballast
Subgrade
Coal

Slurry formation
Fine Subgrade
Clay
Impeded Track Drainage due to Ballast Contamination

Hydraulic Conductivity (k) of fouled ballast

\[
k = \frac{k_b \times k_f}{k_f + \frac{VCI}{100} \times (k_b - k_f)}
\]


Variation of hydraulic conductivity with Void Contaminant Index
Stress-strain behaviour of clean and fouled ballast during drained triaxial tests at 3 confining pressures (Indraratna et al. 2012)
LABORATORY STUDY OF GEOGRID-REINFORCED BALLAST

Large-Scale direct shear box
Dimension: 300x300x200mm

Ballast collected at Bombo Quarry, Wollongong

Coal fines

Biaxial Geogrid
Aperture size = 40mm

Workshop 1: Geosynthetics in Transportation Geotechnics
Shear stress-strain behaviour of fresh and fouled ballast with and without geogrid inclusion (Indraratna et al. 2011)
CYCLIC LOADING TESTS FOR GEOGRID-REINFORCED BALLAST

Cubical Triaxial Apparatus to Simulate a Track Section
(Specimen: 800x600x600 mm)

Sample for testing

Placement of geogrid in the ballast layer

Applying lateral confinement
Variations in the deformation of fresh and fouled ballast with and without geogrid with varying VCIs (Indraratna et al. 2013)
Discrete Element Modelling (DEM) of Geogrid in Tracks

DEM model for ballast and geogrid (Ngo et al. 2014)

\[ \Delta F_i^a = (-k_{ia}A\Delta U_{i}^a)n_i \]

\[ \Delta F_i^e = -k_{ie}A\Delta U_{i}^e \]

The elastic moment-increments are calculated by:

\[ \Delta M_i^a = (-k_{ia}I\Delta \theta_{i}^a)n_i \]

\[ \Delta M_i^e = -k_{ie}I\Delta \theta_{i}^e \]

\[ \sigma_{\text{max}} = \frac{F_i^a}{A} + \frac{|M_i^a|}{I}R \]

\[ \tau_{\text{max}} = \frac{|F_i^e|}{A} + \frac{|M_i^e|}{J}R \]
Ballast aggregates are modelled by clump logic which is connecting many spherical balls together.

- Coal fines are modelled by adding predetermined amount of 1.0mm balls.
- Large-scale direct shear box of 300mm x 300mm x 200mm is simulated in DEM and sheared up to shear strain of 14%.
- Results obtained from the DEM model agree well with laboratory measurement.
Comparison of shear stress and displacements for DEM simulation of reinforced ballast

DEM Modelling Geogrid-reinforced Ballast under Shearing Loads
CONCLUSIONS

- Role of fouling on track structure
- Use of geosynthetic to mitigate track deterioration
Acknowledgement

- Australian Research Council (ARC) for substantial funding
- Centre for Geomechanics and Railway Engineering, University of Wollongong, Australia
- Past and Present research students, Research Associates and Technical Staff
- Industry Organisations: RailCorp (NSW), ARTC, QLD Rail, ARUP, Coffey Geotechnics, Douglas Partners. Roads & Traffic Authority, Queensland Department of Main Roads, Port of Brisbane Corporation, Port Kembla Port Corporation

Thank You!
Workshop 1 – Geosynthetics in Transportation Geotechnics
Performance Improvement of Rail Track Structure using Artificial Inclusions - Experimental and Field Studies

Sinniah K. Navaratnarajah¹, Buddhima Indraratna¹, and Tim Neville²

1. Centre for Geomechanics and Railway Engineering, University of Wollongong, Wollongong City, NSW, Australia

2. Australian Rail Track Corporation Ltd., Broadmeadow, NSW, Australia
Contents

- Introduction
- Laboratory Investigations - Use of Geosynthetics in rail tracks (geogrids, geocomposite, and shock mats)
- Case Studies
  1. Instrumented track at Bulli, NSW, Australia
     Fresh and recycled ballast stabilized with geocomposite
  2. Instrumented track at Singleton, NSW, Australia
     Ballast stabilized with various geogrids, geocomposite and shock mats
- Conclusions
Demand for freight and passenger transport has increased in the last decades.

- Large repetitive loads from traffic cause rapid degradation and deformation of tracks.
- Inclusions of resilient materials (geosynthetics & shock mats) help to reduce such adverse effects of cyclic loads.
Laboratory Investigations

Related laboratory studies on use of Geosynthetics in rail tracks

- Geogrid
- Geotextile
- Geocomposite (Geogrid+Geotextile)
- Shock mats
Cyclic Process Simulation Test Facilities, Designed and Built at UoW

**Cylindrical Triaxial Equipment**
(Specimen: 300 mm dia. x 600 mm high)

Capacity:
100 kN dynamic actuator load
Loading frequency up to 40 Hz

**Prismoidal Triaxial Rig to Simulate a Track Section**
(Specimen: 800x600x600 mm)

Capacity:
100 kN dynamic actuator load
Loading frequency up to 40 Hz

Independent movable vertical walls controls confining pressure and lateral strain
Effect of Confining Pressure on Particle Degradation (Cyclic Loading)

\[ BBI = \frac{A}{A+B} \]

- PSD = particle size distribution
- \( d_{50} = d_{95} \) of largest sieve size
- \( d_{95} \) = smallest sieve size
- \( 2.36 \) = smallest sieve size

Ballast Breakage Index (BBI)

Indraratna, Lackenby and Christie (2005)
*Geotechnique, ICE, UK, Vol. 55(4), 325-328.*
Stress-Strain response of railway ballast stabilized with Geosynthetics (Large-Scale Cyclic Loading)

## Effect of High Impact Loads and Track Degradation

<table>
<thead>
<tr>
<th>Subgrade Type</th>
<th>Location of Shock Mat</th>
<th>Ballast Breakage Index (BBI)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Stiff</strong></td>
<td>Without shock mat</td>
<td>0.170</td>
</tr>
<tr>
<td></td>
<td>Shock Mat above ballast</td>
<td>0.145 (↓ 15%)</td>
</tr>
<tr>
<td></td>
<td>Shock Mat below ballast</td>
<td>0.129 (↓ 24%)</td>
</tr>
<tr>
<td></td>
<td>Shock Mat above &amp; below ballast</td>
<td>0.091 (↓ 47%)</td>
</tr>
<tr>
<td><strong>Soft</strong></td>
<td>Without shock mat</td>
<td>0.080</td>
</tr>
<tr>
<td></td>
<td>Shock Mat above ballast</td>
<td>0.055 (↓ 31%)</td>
</tr>
<tr>
<td></td>
<td>Shock Mat below ballast</td>
<td>0.056 (↓ 30%)</td>
</tr>
<tr>
<td></td>
<td>Shock Mat above &amp; below ballast</td>
<td>0.028 (↓ 65%)</td>
</tr>
</tbody>
</table>

High capacity drop weight Impact test Apparatus

Case Studies

- **Instrumented track at Singleton**
  - Ballast stabilized with various geogrids, geocomposite and shock mats

- **Instrumented track at Bulli**
  - Fresh and recycled ballast stabilized with geocomposite

Image from Google Earth
Case Study: 1. Instrumented track at Bulli

Ballast stabilized with geocomposite

Instrumented Sections
Section 1: Fresh ballast
Section 2: Fresh ballast with geocomposite
Section 3: Recycled ballast with geocomposite
Section 4: Recycled ballast

Details of instrumented track
Field Instrumentation – Bulli, NSW

Ballasted track bed with geocomposite layer

Installation lateral displacement transducers

Installation of vertical and horizontal pressure cells

Installation of vertical settlement pegs
Field Trial on Instrumented Track in Bulli, NSW

Placement of geocomposite layer (geogrid+geotextile) before ballast placement

Ballast placement over the geocomposite

Settlement pegs installed at ballast-capping interface

Settlement pegs installed at sleeper-ballast interface

Displacement transducers installed at sleeper-ballast interface

Vertical and horizontal pressure Cells
Material Specification

<table>
<thead>
<tr>
<th>Material</th>
<th>Maximum particle size (mm)</th>
<th>Minimum particle size (mm)</th>
<th>Median particle size (mm)</th>
<th>Coefficient of uniformity</th>
<th>Coefficient of curvature</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fresh Ballast</td>
<td>75</td>
<td>19</td>
<td>35</td>
<td>1.5</td>
<td>1</td>
</tr>
<tr>
<td>Recycled Ballast</td>
<td>75</td>
<td>9.5</td>
<td>38</td>
<td>1.8</td>
<td>1</td>
</tr>
<tr>
<td>capping</td>
<td>19</td>
<td>0.05</td>
<td>0.26</td>
<td>5</td>
<td>1.2</td>
</tr>
</tbody>
</table>

**Fresh Ballast**
Bombo Quarry, Wollongong

**Recycled Ballast**
from Chullora Quarry, Sydney

---

**Geocomposite (Geogrid + Geotextile)**

<table>
<thead>
<tr>
<th>Biaxial geogrid</th>
<th>Nonwoven geotextile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile strength, $T_s$ (kN/m)</td>
<td>$30 \times 30$</td>
</tr>
<tr>
<td>Strain at break, $\varepsilon_b$ (%)</td>
<td>$11 \times 10^4$</td>
</tr>
<tr>
<td>Aperture size, $A$ (mm)</td>
<td>$40 \times 27$</td>
</tr>
<tr>
<td>Thickness, $t$ (mm)</td>
<td>2</td>
</tr>
<tr>
<td>Mass per unit area, $\rho_s$ (g/m$^2$)</td>
<td>420</td>
</tr>
<tr>
<td>Thickness, $t$ (mm)</td>
<td>2</td>
</tr>
<tr>
<td>Mass per unit area, $\rho_s$ (g/m$^2$)</td>
<td>140</td>
</tr>
</tbody>
</table>
### Test Results - Bulli Track

**Maximum cyclic stresses at Test Section 1**

<table>
<thead>
<tr>
<th>Measured Location</th>
<th>Under the rail</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Axle Load</strong></td>
<td><strong>20.5 ton</strong></td>
</tr>
<tr>
<td>(Passenger Train)</td>
<td>(Coal Train)</td>
</tr>
<tr>
<td>82 class locomotive</td>
<td>100 tons wagons</td>
</tr>
<tr>
<td><strong>Stress (kPa)</strong></td>
<td><strong>Vertical (σ_v)</strong></td>
</tr>
<tr>
<td>Sleeper-ballast</td>
<td>238</td>
</tr>
<tr>
<td>Ballast-capping</td>
<td>63</td>
</tr>
</tbody>
</table>

**Potential benefits of geocomposite at the ballast-capping interface**

<table>
<thead>
<tr>
<th>Deformation reduction due to geocomposite (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fresh Ballast</td>
</tr>
<tr>
<td>Vertical</td>
</tr>
<tr>
<td>Lateral</td>
</tr>
</tbody>
</table>

**Average vertical and lateral deformation of ballast**

- Geogrid apertures offered a strong mechanical interlock with ballast → Increased frictional interlock.
- The cost of geosynthetic installation is low compared to the substantial financial benefits generated by an extended life span of the track, and reduced maintenance due to more resilient behaviour by the ballast.

Indraratna et al. (2010). JGGE, ASCE, 136(7): 907-917

---

*Workshop 1: Geosynthetics in Transportation Geotechnics*
Case Study: 2. Instrumented track at

Ballast stabilized with various geogrids, geocomposite and shock mats

Instrumented Sections
A, C: Fresh ballast
1,2,3,4,5: Fresh ballast + geosynthetics
B: Fresh ballast + shock mat

Details of instrumented track

<table>
<thead>
<tr>
<th>Section</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>50 m</td>
<td>140 m</td>
<td>50 m</td>
<td>140 m</td>
</tr>
<tr>
<td>A</td>
<td>Geogrid 1 (44 mm x 44 mm)</td>
<td>Geogrid 2 (65 mm x 65 mm)</td>
<td>Geogrid 3 (40 mm x 40 mm)</td>
<td>Geo-Composite</td>
</tr>
<tr>
<td>Section 5</td>
<td>Geogrid 3 (31 mm x 31 mm)</td>
<td>No grid</td>
<td>No grid</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>Shock Mat (UBM)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Experimental track sections are part of Third Track of Minimbah Bank Stage 1 Line

Track on soft embankment
Track on hard rock
Track on bridge
Field Instrumentation – Singleton, NSW

Locations of pressure cells & settlement pegs

Shock mat above bridge deck

Deformation frame
Field Trial on Instrumented Track in Singleton, NSW

- Geogrid layer placed above the capping
- Mudies Creek Bridge pressure cells installation
- Ballast-subballast peg
- Support base and collar
- Displacement Monitoring Frame
- Settlement pegs placement in the track
- Placing of shock mat on bridge deck, Feb. 2010
Data Acquisition

- Both electronic data acquisition and manual measurements were taken.
- A simple survey technique is used to obtain the movements of pegs.
- Data acquisition was performed at high frequency (2000 Hz) to capture real-time stress-strain behaviour.
- Data were obtained daily for three days, weekly for three weeks, monthly for three months, and quarterly thereafter.
### Material Specification

Technical specifications of different types of geosynthetics

<table>
<thead>
<tr>
<th>Material</th>
<th>Geogrid 1</th>
<th>Geogrid 2</th>
<th>Geogrid 3</th>
<th>Geogrid 4</th>
<th>Geocomposite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>Biaxial</td>
<td>Biaxial</td>
<td>Biaxial</td>
<td>Biaxial*</td>
<td>Non-woven Geotextile</td>
</tr>
<tr>
<td>Tensile stiffness, $E_s$ (MN/m)</td>
<td>$1.8 \times 1.8$</td>
<td>$1.5 \times 1.5$</td>
<td>$1.5 \times 1.5$</td>
<td>$2.0 \times 2.0$</td>
<td>$0.3 \times 0.5^*$</td>
</tr>
<tr>
<td>Tensile Strength, $T_u$ (kN/m)</td>
<td>$36 \times 36$</td>
<td>$30 \times 30$</td>
<td>$30 \times 30$</td>
<td>$40 \times 40$</td>
<td>$6 \times 10$</td>
</tr>
<tr>
<td>Strain at Break, $\varepsilon_b$ (%)</td>
<td>$15 \times 15$</td>
<td>$15 \times 15$</td>
<td>$15 \times 15$</td>
<td>$15 \times 15$</td>
<td>$60 \times 40$</td>
</tr>
<tr>
<td>Aperture Size, A (mm)</td>
<td>$44 \times 44$</td>
<td>$65 \times 65$</td>
<td>$40 \times 40$</td>
<td>$31 \times 31$</td>
<td>-</td>
</tr>
<tr>
<td>Thickness, t (mm)</td>
<td>3</td>
<td>3</td>
<td>4</td>
<td>3</td>
<td>2.9</td>
</tr>
<tr>
<td>Specific mass, $\rho_s$ (g/m²)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>150</td>
</tr>
</tbody>
</table>

*The values are indicated as $0.3 \times 0.5$; where $0.3$ is machine direction (longitudinal to the roll) and $0.5$ is transverse direction (across the roll width).

---

**Geogrid**

MD – Machine Direction
TD – Transverse direction

For eg.,

$1.5 \times 1.5$ means MD $\times$ TD
Maximum cyclic vertical stresses

<table>
<thead>
<tr>
<th>Vertical stress, $\sigma_v$ (kPa) measured at</th>
<th>Sections A and 1 (soft embankment)</th>
<th>Sections C and 5 (hard rock)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sleeper-ballast interface</td>
<td>170 - 180</td>
<td>215 - 230</td>
</tr>
<tr>
<td>Ballast-capping interface</td>
<td>30 - 35</td>
<td>90 - 110</td>
</tr>
</tbody>
</table>

- Vertical deformation curtailed by 10-32% by using geosynthetics. (additional interlocking provided by the geogrid aperture).
- Geogrid was more affective for a soft embankment than for the hard rock area.
- Geogrid 3 with 40 mm $\times$ 40 mm size apertures performed better (optimum aperture size $1.15D_{50}$ of ballast)

Vertical deformation of ballast at soft and hard embankment

- Rubber mats reduce ballast degradation at the concrete bridge track.

Indraratna et al. (2013), ICE-GI, 167(1): 24-34
Conclusions

1. Laboratory and Field studies of geosynthetics to improve overall stability of rail tracks was studied.

2. Geogrids increase confining pressure and reduce deformation in rail tracks, while energy absorbing Shock mats reduce particle breakage.

3. Recycled ballast can be stabilize with geosyntetic for improved track performances.

4. The field trials demonstrate the implications of track deterioration, and the advantages of track modernization using synthetic inclusions.
Acknowledgements

- Australian Research Council (ARC)
- Centre for Geomechanics and Railway Engineering, University of Wollongong, Australia
- Cooperative Research Centre (CRC) for Rail Innovation
- Industry Partners: Sydney Trains (NSW), Aurizon, ARTC
- Technical Staff: Alan Grant, Cameron Neilson, Ian Bridge

Thank You
Basal Reinforced Piled Embankments

Suzanne J.M. van Eekelen

Deltares, Netherlands
suzanne.vaneekelen@deltares.nl
Basal Reinforced Piled Embankments

Experiments, field studies and the development and validation of a new analytical design model
What is a (Basal Reinforced) Piled Embankment?

GR = geosynthetic reinforcement
Workshop 1: Geosynthetics

measured = calculated

measured GR strain vs. calculated GR strain
calculated GR strain 2010 method

measured GR strain

2010 method
Measurements:
  Zaeske 2001, Germany
  Van Duijnen et al 2010, Netherlands
  Huang et al 2009, Finland
  Oh and Shin 2007, Korea
  Haring et al, 2008, N210, Netherlands
  Weihrauch 2013, Hamburg, Germany
  Vollmert et al 2007, Bremerhaven, Germany.
  Almeida et al 2007, Rio de Janeiro, Brazil
  Briancon and Simon 2012, France
  Van Eekelen et al 2012a, Netherlands
  Van Eekelen et al 2012b, Woerden, Netherlands
piles

“soft soil”
"soft soil"
Geosynthetic reinforcement
embankment (fill)
Load → step 1 arching

load part A

“residual” → step 2 → strain
Workshop 1: Geosynthetics

- Triangular: EBGEO
- Uniform: BS8006
- Inverse Triangular: FRENCH ASIRI

3rd ICTG 2016
04-07 September 2016, Guimarães, Portugal
Load distribution ↔ deformation GR
GR deflection (vertical deformation)
Observed load distribution:
More precise (van Eekelen et al., 2015):
Load → step 1 arching → load part A → “rest” → step 2 → strain
Excel sheet with equations: www.piledembankments.com
Observed:

Calculated with the Concentric Arches model:
measured strain

strain calculated with 2010 method

2.5 x

calculated = measured

strain calculated with the Concentric Arches model

1.1 x
Conclusions

2010 method (EBGEO/CUR 226): calculates 2.5 times the measured strain

Experiments: load distribution inversed triangular

Explanation: new Concentric Arches model

Result: 1.1 times the measured strain

“perfect” match

Therefore: Adopted in new Dutch Design Guideline
Core members of committee ‘Dutch Guideline Piled Embankments’

- Chair: Suzanne van Eekelen, Deltares
- Reporter: Marijn Brugman, Arthe Civil & Structure
- Chair: Marco Peters, Grontmij
- Chair: Piet van Duijnen, Geotec Solutions
- Chair: Martin de Kant, RHDHV
- Chair: Maarten Profittlich, Fugro
- Chair: Jeroen Dijkstra, Cofra
- Chair: Lars Vollmert, Naue/BBG
- Chair: Eelco Oskam, Movares
- Chair: Maarten ter Linde, Strukton
- Chair: Daan Vink, CRUX Engineering
- Chair: Marco Peters, Grontmij
Design Guideline
CRCpress.com or amazon.com

Free excel with the equations:
www.piledemembankments.com

International course:
15/16 November in Delft, Netherlands
https://paotm.nl search for “basal”
Most important publications about this research:


This PhD thesis include:


Workshop 1 – Geosynthetics in Transportation Geotechnics
Geosynthetics with Enhanced Lateral Drainage Capabilities in Roadway Systems

Jorge G. Zornberg¹, Marcelo Azevedo¹, Mark Sikkema², and Brett Odgers²

1. The University of Texas at Austin, United States of America
2. TenCate Geosynthetics
Source: Zornberg et al. (2016)
Workshop 1: Geosynthetics in Transportation Geotechnics

Source: Zornberg et al. (2016)
Geosynthetics in Roadway Systems

Pavement applications involving geosynthetics:

1. Mitigation of Reflective Cracking in Asphalt Overlays
2. Separation
3. Stabilization of Road Subgrades
4. Stabilization of Road Bases
5. Improved Drainage

5 pavement applications, involving 1 or more geosynthetic functions each
Mitigation of Reflective Cracking

Source: Zornberg et al. (2016)
Workshop 1: Geosynthetics in Transportation Geotechnics

Separation Application

Source: Zornberg et al. (2016)
Stabilization of Road Subgrades

Source: Zornberg et al. (2016)
Stabilization of Road Bases

Source: Zornberg et al. (2016)
Geosynthetics for Improved Drainage

Rainfall

Asphalt layer

Base

Moisture infiltration

Geosynthetic

Subgrade

Water capillary rise

Source: Zornberg et al. (2016)
Geosynthetics for Improved Drainage
Geosynthetics for Improved Drainage

Typical GS products include:

• NW Geotextile separation/filter for free draining base and/or subbase layers
• Geocomposite horizontal drainage layers (to replace or augment free draining base)
• Woven geotextiles with enhanced lateral drainage capabilities ("wicking" geotextiles)
Impact of Drainage on Pavement Design

$m_i$: Affects structural layer coefficients
(for untreated base and subbase materials)

<table>
<thead>
<tr>
<th>Quality</th>
<th>% Time Saturated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&lt; 1%</td>
</tr>
<tr>
<td>Excellent</td>
<td>1.40 - 1.35</td>
</tr>
<tr>
<td>Good</td>
<td>1.35 - 1.25</td>
</tr>
<tr>
<td>Fair</td>
<td>1.25 - 1.15</td>
</tr>
<tr>
<td>Poor</td>
<td>1.15 - 1.05</td>
</tr>
<tr>
<td>Very Poor</td>
<td>1.05 - 0.95</td>
</tr>
</tbody>
</table>
Some Bad News:
Drains not Always Drain
Unsaturated Geomaterials Behavior

Hydraulic Conductivity, K

Clay
Silt
Sand or Geotextile

θ_{res.}
θ_{sat}

Volumetric Moisture Content, θ
Water Retention Curve (WRC)

- Nonwoven geotextile
- Nonwoven geotextile WRC
- Sand, 50% RD
- Sand WRC
- Low PI Clay, 72.8% RC
- Low PI Clay WRC

(McCartney, Zornberg, and Kuhn 2005)
Column Test Studies

(McCartney, Zornberg, and Kuhn 2005)
Source: Zornberg et al. (2016)
Now the Good News: Geosynthetics can be engineered to provide Enhanced Drainage
Enhanced Lateral Drainage

- Conventional geotextiles provide in-plane drainage after saturation of the soil-geotextile interface:
  - In Non-Woven Geotextiles: Through the large void spaces in its open structure
  - In Woven Geotextiles: Through void spaces of crossed-over yarns

- Enhanced Lateral Drainage involves providing additional in-plane drainage capacity that is mobilized due to suction gradients (or “wicking”) within the geotextile yarns.

Conventional geotextile fiber  “Wicking” fiber with engineered cross-section to increase specific surface
Workshop 1: Geosynthetics in Transportation Geotechnics
Workshop 1: Geosynthetics in Transportation Geotechnics
1. Enhanced Lateral Drainage of Moisture Migrating Upward from a High Water Table

Source: Zornberg et al. (2016)
Daniel Boone Bridge, Missouri, USA

Source: Zornberg et al. (2016)
2. Enhanced Lateral Drainage of Moisture Migrating Downward from the Surface

Source: Zornberg et al. (2016)
Garwood Railroad Sliding, Idaho, USA

Source: Zornberg et al. (2016)
Garwood Railroad Sliding, Idaho, USA

Source: Zornberg et al. (2016)
3. Control of Pavement Damage Caused by Frost Heave

Source: Zornberg et al. (2016)
Pioneer Mountains Scenic Byway, Montana, USA
Asphalt Cement – 75 mm

Base – 200 mm

Subbase – 500 mm

Subgrade

Pioneer Mountains Scenic Byway, Montana, USA

Source: Zornberg et al. (2016)
Asphalt Cement – 100 mm
Base – 200 mm
Subbase – 250 mm
Subgrade
Geotextile Reinforcement
ELD Geotextile

Pioneer Mountains Scenic Byway, Montana, USA

Source: Zornberg et al. (2016)
Pioneer Mountains Scenic Byway, Montana, USA

Source: Zornberg et al. (2016)
4. Control of Pavement Damage caused by Expansive Clays

Source: Zornberg et al. (2016)
Objective: Control of Differential Settlements over Expansive Clays, SH21, Texas

- A stretch of almost 10 miles of SH21 Highway, Texas, USA, is founded on highly expansive clays
- This portion of SH21 has shown poor performance, resulting in costly maintenance operations
- The Texas Department of Transportation (TxDOT) designed a rehabilitation plan for SH21 as part of State Highway Improvement Plan

Source: Zornberg et al. (2016)
State Highway 21, Texas, USA

Source: Zornberg et al. (2016)
Control of Differential Settlements over Expansive Clays, SH21, Texas, USA

• The main distresses observed included major longitudinal and edge cracking, vertical deformation, rutting, and faulting.

• An evaluation involving eight test sections constructed with four different types of separator geotextiles (GT) was incorporated into the improvement plan.

• The selected geotextiles included:
  1) a generic nonwoven GT that was originally used by TxDOT in that area
  2) a high strength wicking fabric woven GT
  3 & 4) two high strength woven GT manufactured with non-wicking fabric

• Geotextiles were used on top of the subgrade soil separating the clay subgrade from granular pavement layers.

Source: Zornberg et al. (2016)
Workshop 1: Geosynthetics in Transportation Geotechnics

Source: Zornberg et al. (2016)
Control of Differential Settlements over Expansive Clays, SH21, Texas, USA

- A series of moisture and temperature sensors were installed beneath the geotextile within the subgrades soil.

- Monitoring the moisture sensor readings along with the observation of the performance of the road will provide valuable insights into the potential benefits of the wicking fabrics in enhancement of the hydraulic and/or mechanical performance of the road.

Source: Zornberg et al. (2016)
Section with ELD Geotextile

Control Section
Source: Zornberg et al. (2016)
5. Enhanced Lateral Drainage in Soil Improvement Projects

- **SR 12**: Steep Slope
- **Prefabricated Vertical Drains**: (60 ft depth, 5 ft triangular spacing)
- **Geosynthetic-reinforced Steep Slope**
- **Surcharge**
- **New Embankment**
- **Existing Embankment**
- **Clayey Peat**
- **Clay**
- **Silty Sand**

Source: Zornberg et al. (2016)
State Route 12, California, USA

Source: Zornberg et al. (2016)
Conclusions

- The incorporation of wicking yarns into woven geotextiles has led to the development of ELD geosynthetics, which are capable of conveying moisture stored in unsaturated pavement layers.
- Specific applications of ELD geosynthetics have been identified to be beneficial to pavement performance. They include:
  a) enhanced lateral drainage of moisture migrating upward from a high water table;
  b) enhanced lateral drainage of moisture infiltrating downward from the surface;
  c) control of frost heave-induced pavement damage;
  d) control of pavement damage caused by expansive clay subgrades; and
  e) enhanced lateral drainage in projects involving soil improvement.
Conclusions (Cont.)

• The use of ELD geosynthetics has shown pavement benefits that complement those strictly related to enhanced lateral drainage. This includes multiple additional applications of geosynthetics in pavements, including separation, subgrade stabilization, and base stabilization.
• The use of ELD geosynthetics has shown cost savings associated with a decrease in thickness of the base.
• 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, condition surveys to compare performance of pavement sections with and without ELD geosynthetics, or in-situ monitoring of moisture content. case history).
Final Remarks

Overall, data on roadway performance from a number of case histories indicates that enhanced lateral drainage in roadways offers often significant opportunities to improve the performance of a wide range of transportation projects.
Obrigado!
Thank You!
Effect of Geogrid on Railroad Ballast Particle Movement

Shushu Liu¹, Hai Huang¹, Tong Qiu¹, Jayhyun Kwon²

1. The Pennsylvania State University
2. Tensar International Incorporation
Introduction

• Railroad Ballast
  • Large sized angular aggregates;
  • Horizontal and rotational movement.

• Geogrid
  • Interlocking with particles;
  • Application in railroad ballast.
Previous Research Studies

Results of cyclic load tests at Queens University.
Previous Research Studies

Maximum vertical stresses at interface between base and subgrade.

Vertical stress distributions at 120th load cycle.

Qian, Han et al. (2011)
Geogrid-Aggregate Interlock Mechanism Investigation Using DEM Approach

Ngo et al. 2014
Geogrid-Aggregate Interlock Mechanism Investigation Using DEM Approach
Ballast modeling shows particle horizontal movement and rotation are important modes of particle movement.
“SmartRock”

- Shape;
- Wireless device;
- Data storage;
- Sleep mode;
- Translation, rotation and orientation.
Real Time Rotation
Rotation + Translation
Laboratory Test – Ballast
# Laboratory Test – Geogrid

## Physical Properties of Geogrids Used in Track Stabilization

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Method</th>
<th>Units</th>
<th>Geogrid Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aperture shape</td>
<td>Observation</td>
<td></td>
<td>Equilateral Triangular</td>
</tr>
<tr>
<td><strong>Aperture size</strong> (machine x cross machine direction)</td>
<td>Direct measurement</td>
<td>mm</td>
<td>60 x 60</td>
</tr>
<tr>
<td>Flexural rigidity (Machine direction)</td>
<td>ASTM D7748-12</td>
<td>mg-cm</td>
<td>2,000,000</td>
</tr>
<tr>
<td>Radial stiffness @ 0.5% strain</td>
<td>ASTM D6637-10</td>
<td>kN/m</td>
<td>350</td>
</tr>
<tr>
<td>Junction efficiency</td>
<td>ASTM D7737-11</td>
<td>%</td>
<td>93</td>
</tr>
</tbody>
</table>
Laboratory Testing – WITHOUT Geogrid

Beneath Rail Seat

Beneath Edge of Tie
Laboratory Testing – WITHOUT Geogrid

SmartRocks
Displacement and Stiffness

Vertical Displacement (cm)

-3.5  -3  -2.5  -2  -1.5  -1  -0.5  0  0.5

Load Cycles

With Geogrid
Without Geogrid

Stiffness (kN/cm)

80  100  120  140  160  180  200  220

Load Cycles

With Geogrid
Without Geogrid
Particle Rotation – beneath Rail Seat

Without Geogrid

With Geogrid

Beneath Rail Seat: Without Geogrid

Beneath Rail Seat: With Geogrid

Beneath Edge of Tie: Without Geogrid

Beneath Edge of Tie: With Geogrid

Beneath Rail Seat: Without Geogrid

Beneath Rail Seat: With Geogrid
Particle Rotation – beneath Edge of Tie

**Without Geogrid**

Beneath Edge of Tie: Without Geogrid (c)

- Rotation_x
- Rotation_y
- Rotation_z

**With Geogrid**

Beneath Edge of Tie: With Geogrid (d)

- Rotation_x
- Rotation_y
- Rotation_z

Beneath Rail Seat: Without Geogrid

Beneath Rail Seat: With Geogrid
Particle Acceleration – beneath Rail Seat

(a) X direction;  
(b) Y direction;  
(c) Z direction
Particle Acceleration – beneath Edge of Tie

(a) X direction;
(b) Y direction;
(c) Z direction
Visualization: Without/With Geogrid

WITHOUT Geogrid  

WITH Geogrid
Conclusions and Future Work

- The measured ballast surface displacement and particle movement inside the ballast without geogrid illustrates the significant ballast settlement and dramatic particle translation and rotation during the “compaction” settlement phase.

- SmartRock is capable of recording and visualizing real-time particle movement including both translation and rotation.

- SmartRock can be possibly serving as a quantitatively monitoring tool as it investigates ballast performance at individual aggregate level.

- Particle translational movement and rotation were higher beneath the edge of the tie than beneath the rail seat due to lack of confinement at the slope.

- The movement of particles adjacent to the geogrid is effectively confined at both locations; especially beneath the edge of tie, the inclusion of geogrid was most beneficial to confine particle lateral movement at this location.

- More SmartRocks at different locations.

- Attempt to characterize ballast performance based on particle movement pattern.
Acknowledgements

• Financial support for development of SmartRock was provided by the Federal Railroad Administration, U.S. Department of Transportation.

• Geogrid supply: Tensar International Corporation.
Thank you for your attention!
Workshop 1 – Geosynthetics in Transportation Geotechnics
Geosynthetic Subgrade Stabilization – Field Testing and Design Method Calibration

Eli Cuelho & Steven Perkins
Montana State University
Western Transportation Institute
Bozeman, MT
USA

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Workshop 1: Geosynthetics in Transportation Geotechnics
Background

• Broad road types
  • Temporary roads and working platforms
    • Detours, haul and access roads, construction platforms, stabilized working platforms for permanent roads, embankments over soft ground
  • Permanent roads
    • Paved or unpaved
    • Millions of load applications over many years
• Potentially poor subgrade conditions
  • Low undrained shear strength
  • Low CBR
  • High water table
  • High sensitivity
Geosynthetic Benefit on Soft Subgrades

- Reducing stress in subgrade
- Preventing aggregate loss
- Preventing pumping and migration of fines
- Reducing excavation of unsuitable materials
- Reducing thickness of aggregates
- Reducing disturbance of soft ground
- Allowing for consolidation of subgrade
- Providing more uniform support
- Reducing maintenance
Stabilization: Separation Function
Stabilization: Reinforcement Function

- Lateral Restraint
- Bearing Capacity Increase
- Membrane Tension Support
Study Objective

• Address concerns raised by Departments of Transportation regarding geosynthetic used as subgrade stabilization?
  • Deficiencies in the standard design techniques
  • Lack of agreement as to which geosynthetic properties are most relevant for this application
  • Update design methodology to incorporate these material properties
  • Promote healthy competition between manufacturers
  • Potentially revise geosynthetic specifications by DOTs

• Follow-on to Phase I study completed in 2009 (Cuelho & Perkins, 2009)
Experimental Design

• Full-scale test sections
  • 17 test sections
  • TRANSCEND research laboratory in Montana

• Geosynthetic characterization
  • Wide-width tensile strength
  • Cyclic tensile modulus
  • Resilient interface shear stiffness
  • Junction strength and stiffness
  • Aperture stability modulus
General Layout of Test Sections

Direction of Traffic

Not to scale

4.9 m

15 m

IFG-1  IFG-2  IFG-3  WeG-4  WeG-5  WeG-6  WoG-7  WoG-8  KnG-9  ExG-10  IFG-11  IFG-12  WoT-13  NWoT-14  Control 1  Control 2  Control 3

Regular base thickness (avg = 27.7 cm)

Thicker base (avg = 41.4 cm)

Thickest base (avg = 63.2 cm)

Stronger subgrade (avg = 2.17 CBR)

Weaker subgrade (avg = 1.64 CBR)

Regular subgrade strength (avg = 1.79 CBR)
Geosynthetics

IFG-1, IFG-2, IFG-3
WeG-4
WeG-5
WoG-6
WoG-7
WoG-8
KnG-9
ExG-10
IFG-12
WoT-13
NWoT-14
Idealized Cross-Section (to scale)
Preparing Subgrade
Placing Subgrade
Compacted Subgrade
Screeding Subgrade
Installing Geosynthetics
Constructing Base Layer
Test Vehicle

- 20.6 metric tons
- 8 kph
Rut Measurement

*Measurements were made at 0, 3, 10, 20, 40, 70, 80, 102, 125, 175, 250, 300, 325, 351, 395, 440, 540, 640, and 740 truck passes
Linear Regression Analysis

• Determine material properties most related to performance
• Evaluated at 1.0, 2.0 and 2.5 in. of rut
• Material properties evaluated
  • Wide-width strength at 2%
  • Wide-width strength at 5%
  • Ultimate wide-width strength
  • Cyclic tensile stiffness at 0.5, 1.0, 1.5, 2.0, 3.0 and 4.0% strain
  • Resilient interface shear stiffness
  • Junction strength
  • Junction stiffness (secant stiffness at 0.05 in. displacement)
  • Aperture stability modulus
Regression Analysis Results in XMD

Results using all data

Results using select data
Regression Results from Phase I Study

Cross-Machine Direction Results

Machine Direction Results
Summary of Regression Results

• Greatest correlation is with junction stiffness/strength
• Followed by tensile strength in cross-machine direction
  • 2%, 5% and cyclic modulus
• Considering results from Phase I
  • Junction stiffness/strength correlations peak at 75 mm rut
  • Wide-width tensile strength takes over
Giroud-Han Design Equation

\[
h = \frac{1 + k \log N}{\tan \alpha_0 [1 + 0.204(R_E - 1) + \left(\frac{P}{\pi r^2} \left(\frac{s}{f_s}\right) [1 - \xi \exp\left(-\omega \left(\frac{r}{h}\right)^n\right)] N_c c_u - 1\right) r}
\]

- \(h\) = compacted base course thickness {m}
- \(N\) = number of axle passes
- \(k\) = constant dependent on base thickness and reinforcement
- \(\alpha_0\) = initial stress distribution angle = 38.5°
- \(R_E = \min\left(\frac{E_{bc}}{E_{sg}}, 5.0\right) = \min\left(\frac{3.48 CBR_{bc}^{0.3}}{CBR_{sg}}, 5.0\right)\)
- \(P\) = tire load {kN}
- \(r\) = radius of equivalent tire contact area {m}
- \(s\) = allowable rut depth {m}
- \(f_s\) = reference rut depth {m}
- \(c_u\) = subgrade undrained shear strength {kPa}
- \(N_c\) = bearing capacity factor (5.71 for geogrid-reinforced roads)
- \(\xi, \omega, and n\) are constants calibrated by Giroud and Han (2004b) using data from unpaved, unreinforced roads (\(\xi = 0.9\), \(\omega = 1.0\), and \(n = 2.0\))
- \(k = (0.96 - 1.46j^2) \left(\frac{r}{h}\right)^{1.5}\)
Back-Calculate $k'$

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

$h = 0.276$ m; average thickness of base course layer
$R_E = 4.8$; average $CBR_{bc\_field} = 20$, average $CBR_{sg} = 1.79$
$P = 37.63$ kN
$r = 0.139$ m
$N_c = 5.71$
$c_u = 62.7$ kPa
$f_s = 75$ mm

- Use $N$ for different levels of rut:
  - $s = 38.1$ mm
  - $s = 50.8$ mm
  - $s = 63.5$ mm
  - $s = 76.2$ mm
- Calculate $k'$
Linear Regression of $k'$ to Material Properties

- 38.1 mm rut
- 50.8 mm rut
- 63.5 mm rut
- 76.2 mm rut

R-Squared

- 0.38
- 0.50
- 0.63
- 0.76
- 0.89
- 1.0

Materials:
- WWT-2%
- WWT-5%
- WWT-Ult.
- CTS-0.5%
- CTS-1.0%
- CTS-2.0%
- CTS-4.0%
- RSM
- Junc. Str.
- Junc. Stiff.
- ASM
Junction Strength/Stiffness in XMD

- Ultimate strength of junction in shear
- Junction stiffness = secant stiffness at 1.3 mm displacement \( \text{MN/m/m} \)
Junction Stiffness versus $k'$
Validation

Final form of design equation:

\[
h = \frac{1.26 \left[ 1 + k' \left( \frac{T}{R} \right)^{1.5} \log N \right]}{1 + 0.204 \left( R_E - 1 \right)} \left[ \frac{P}{\pi r^2} \right] \left[ 1 - 0.9 \exp \left( - \left( \frac{T}{R} \right)^2 \right) \right] N_c c_u - 1 \]

\[ R^2 = 0.855 \]
Summary and Conclusions

• Deficiencies in current design method hindering widespread adoption
• Disagreement on material properties associated with good performance
• Full-scale research with multiple test sections
• Regression analysis showed junction stiffness and tensile strength in cross-machine direction as directly linked to performance
• Giroud-Han design equation calibrated based on results of test sections to include junction stiffness
Questions are Welcome

Thank you for your interest!

Eli Cuelho & Steven Perkins

Montana State University
Western Transportation Institute
Bozeman, MT
USA
Workshop 1 – Geosynthetics in Transportation Geotechnics
Contact Pressure Distribution on Weak Subgrades due to Repeated Traffic on Geocell Reinforced Base Layers

Sireesh Saride¹, Jorge Zornberg²

1. Indian Institute of Technology Hyderabad, India
2. The University of Texas at Austin, United States of America
OUTLINE

• Introduction
• Research Objectives
• Test Setup
• Materials Used
• Experimental Program
• Results and Discussion
• Summary and Conclusions
Typical Cross-Section of a Road
Low Volume Roads
Rural Road Problems

- Rutting
- Fatigue cracking replication
Issues with Flexible Pavements

Rutting

Fatigue cracking
Factors affecting Pavement Performance

- Weak Subgrades
- Excessive Loading
- Material Failure
- Regional Issues
- Design Philosophy

- Hence, higher contact stresses would transfer to the weak subgrades
- Leads to high rutting...
Improvement Techniques

• Stabilization Techniques
  • Subgrade level
  • Base/subbase level

• Geosynthetic Reinforcements
  • Geogrids
  • Geocells
Classification of Geosynthetics

Geogrids

Geotextiles (Woven)

Geotextiles (Non Woven)

Geocells

Geomembranes
Possible Reinforcement Functions Provided by Geosynthetics

(After Haliburton et al., (1981))
Why Geocell?

- Geocell has **lateral confinement** due to its honeycomb structure

Image Source: www.esi.info
Research Objectives

• To Study the behavior of geocell reinforced base layers overlying weak subgrades under repetitive traffic loading.

• To quantify the improvement of geocell reinforcement over weak subgrades.

• To understand and quantify the contact stress reduction due to geocells
Experimental Study
Computer Controlled Servo Hydraulic Actuator Test Setup
Materials used

- Sand
- Aggregate
- Clay
- HDPE Geocell
- Surface Layer
### Properties of Dry Sand & Aggregates

<table>
<thead>
<tr>
<th>Properties</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{10}$, mm</td>
<td>0.20</td>
</tr>
<tr>
<td>$D_{30}$, mm</td>
<td>0.32</td>
</tr>
<tr>
<td>$D_{60}$, mm</td>
<td>0.48</td>
</tr>
<tr>
<td>Sand Classification (USCS)</td>
<td>SP</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.63</td>
</tr>
<tr>
<td>$e_{\text{max}}$</td>
<td>0.74</td>
</tr>
<tr>
<td>$e_{\text{min}}$</td>
<td>0.51</td>
</tr>
<tr>
<td>$\Phi$ at 75, 70, 30 % $R_D$</td>
<td>41°, 37°, 34°</td>
</tr>
</tbody>
</table>

#### Particle Size Distribution

- **Sand**
- **Aggregate**

<table>
<thead>
<tr>
<th>Material</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggregate</td>
<td>MoRTH’s Base Grade III</td>
</tr>
</tbody>
</table>
## Properties of Clayey Soil

<table>
<thead>
<tr>
<th>Properties</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay Classification (USCS)</td>
<td>SC</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>21</td>
</tr>
<tr>
<td>Liquid Limit</td>
<td>46</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.69</td>
</tr>
<tr>
<td>MDD (gm/cm³)</td>
<td>1.72</td>
</tr>
<tr>
<td>OMC (%)</td>
<td>15</td>
</tr>
</tbody>
</table>
# Engineering Properties of Geocells

<table>
<thead>
<tr>
<th>Properties</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material Composition</td>
<td>Polymer – High Density Polyethylene (HDPE) with density of 0.935-0.965 g/cm³</td>
</tr>
<tr>
<td>Weld Spacing (mm)</td>
<td>356</td>
</tr>
<tr>
<td>Cell Depth (mm)</td>
<td>75, 100, 150, 200</td>
</tr>
<tr>
<td>Cell Size (±10%) (mm)</td>
<td>259 x 224</td>
</tr>
<tr>
<td>Cell Area (± 4%)</td>
<td>290</td>
</tr>
<tr>
<td>Min. Cell Seam Strength (N)</td>
<td>2100</td>
</tr>
</tbody>
</table>
Preparation of Test Section

1. A 5 kg static compactor – Clay subgrade
2. Pluviation / raining technique – Sand bases
3. A plate vibrator - Aggregate bases
Typical Test Setup With Instrumentation
Schematic of Test Setup

- LVDT’s
- Surface layer
- Geocell
- Strain Gauges
- Pressure Cells
- Test Tank (1m x 1m x 1m)
Instrumentation

Geocell with Strain Gauges

Earth pressure Cells
Cyclic Load Tests

- Cyclic load: Loading rate = 1 kN for 20 sec. (0.05 Hz)
Repetitive Load Tests

• Haversine load pulse at 1 Hz frequency
<table>
<thead>
<tr>
<th>Description</th>
<th>Test Nomenclature</th>
<th>Constant Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Unreinforced Granular aggregate base over Clayey Soil Subgrade</strong></td>
<td>U G C</td>
<td>$\gamma_d = 23.1 \text{kN/m}^3$, $C_u = 10 \text{kPa}$, $H/D = 1.67$</td>
</tr>
<tr>
<td><strong>Unreinforced Granular aggregate base over Clayey Soil Subgrade and Surface Layer</strong></td>
<td>U G C SL</td>
<td>Surface Layer, $\gamma_d = 23.1 \text{kN/m}^3$, $C_u = 10 \text{kPa}$, $H/D = 1.67$</td>
</tr>
<tr>
<td><strong>Geocell Reinforced Granular aggregate base over Clayey Soil Subgrade</strong></td>
<td>G G C</td>
<td>$\gamma_d = 23.1 \text{kN/m}^3$, $C_u = 10 \text{kPa}$, $H/D = 1.67$, $b/D = 4$, $h/D = 1.33$</td>
</tr>
<tr>
<td><strong>Geocell and Basal Geogrid Reinforced Granular aggregate base over Clayey Soil Subgrade</strong></td>
<td>G BG G C</td>
<td>$\gamma_d = 23.1 \text{kN/m}^3$, $C_u = 10 \text{kPa}$, $H/D = 1.67$, $b/D = 4$, $h/D = 1.33$, $B/D = 4.33$</td>
</tr>
<tr>
<td><strong>Geocell Reinforced Granular aggregate base over Clayey Soil Subgrade and Surface Layer</strong></td>
<td>G G C SL</td>
<td>Surface Layer, $\gamma_d = 23.1 \text{kN/m}^3$, $C_u = 10 \text{kPa}$, $H/D = 1.67$, $b/D = 4$, $h/D = 1.33$</td>
</tr>
<tr>
<td><strong>Geocell and Basal Geogrid Reinforced Granular aggregate base over Clayey Soil Subgrade and Surface Layer</strong></td>
<td>G BG G C SL</td>
<td>Surface Layer, $\gamma_d = 23.1 \text{kN/m}^3$, $C_u = 10 \text{kPa}$, $H/D = 1.67$, $b/D = 4$, $h/D = 1.33$, $B/D = 4.33$</td>
</tr>
</tbody>
</table>
Repeatability of Tests

Repeated load

Cyclic load
Performance Indicator

CPR: Contact pressure reduction

\[
(CPR) = \left(1 - \frac{CP_{interface}}{AP}\right) \times 100
\]

CP: Contact Pressure at the base-subgrade interface (kPa)
AP: Applied Pressure (kPa)
Contact pressure distribution in unreinforced beds **without** surface layer

**Sand Base**

**Granular Base**

- **Unreinforced Sand**
  - $X/D = 1.5$ (left)
  - $X/D = 1.0$ (left)
  - $X/D = 0$ (Center)
  - $X/D = 1.0$ (right)
  - $X/D = 2.0$ (right)
  - Applied Pressure

- **Unreinforced Granular Base**
  - $X/D = 1.5$ (left)
  - $X/D = 1.0$ (left)
  - $X/D = 0$ (Center)
  - $X/D = 1.0$ (right)
  - $X/D = 2.0$ (right)
  - Applied Pressure
Contact pressure distribution in unreinforced beds with surface layer

Sand Base

Granular Base
Contact pressure distribution in unreinforced beds without Surface Layer

**Sand Base**
- Applied Pressure = 55
- Applied Pressure = 110
- Applied Pressure = 165

**Soft Clay**
- Pressure Sensor

**Granular Base**
- Applied Pressure = 55
- Applied Pressure = 110
- Applied Pressure = 165
- Applied Pressure = 220
- Applied Pressure = 275
- Applied Pressure = 330

**Pressure Sensor**
Contact pressure distribution in unreinforced beds with Surface Layer
Contact pressure distribution in geocell reinforced beds **without** Surface Layer

![Graph showing contact pressure distribution](image)
Contact pressure distribution in geocell reinforced beds with Surface Layer

**Sand Base**

**Granular Base**
## Test Results

<table>
<thead>
<tr>
<th>Test Case</th>
<th>CPR (%)</th>
<th>( M_r )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>CLT</td>
<td>RLT</td>
</tr>
<tr>
<td>USC</td>
<td>33.3</td>
<td>35</td>
</tr>
<tr>
<td>GSC</td>
<td>90.2</td>
<td>92</td>
</tr>
<tr>
<td>UGC</td>
<td>55</td>
<td>76</td>
</tr>
<tr>
<td>GGC</td>
<td>89</td>
<td>90</td>
</tr>
</tbody>
</table>
Conclusions

• Geocell can improve the structural stiffness of the pavement bases.
• Performance of the pavement bases can be increased by paving with surface layer
• Contact Pressure on the weak subgrade is reduced by about 90%
• Contact Pressure is constant with number of load repetitions
• Granular bases performed better than Sand bases
Thank you
Workshop 1 – Geosynthetics in Transportation Geotechnics
The use of geosynthetics in water conveyance structures
The Panama Canal Expansion Project, Third Set of Locks
Water Saving Basins

José Luís Machado do Vale
President of IGS Portugal
Carpi Tech, Switzerland
Jose.Vale@carpitech.com
Water Saving Basins Panama

• EMPLOYER: ACP
• MAIN CONTRACTOR: GUPC
• LINING WATER SAVING BASINS
• PROJECT COMPANY: CARPI TECH BV / CARPI PANAMA
• DESIGNER: CICP
• 570,000 m$^2$ SIBELON CNT 3750–CNT 4400
• 2014-2016
Localization of the Locks
The existing Canal has 3 blocks of locks: Miraflores (a difference of height of 9 meters each between locks) and Pedro Miguel (9 meters height) on the Pacific side and the locks of Gatun (9 meters each between locks) on the Atlantic side.

New set of locks: Each block of locks provides 3 hops of 9 meters each and Water Saving Basins.
Workshop 01: “Geosynthetics in Transportation Geotechnics”
Workshop 01: “Geosynthetics in Transportation Geotechnics”
Locks Description

- Each Lock chamber is connected with two culverts to three Water Saving Basins.
- The scope of the WSBs is to save the 60% of the fresh water needed to operate the lock chamber.
Original design – geomembrane totally COVERED by concrete
Carpi alternative design – geomembrane EXPOSED
All concrete cover layer deleted, except for the access roads to the intakes

CARPI DESIGN
FULLY EXPOSED SOLUTION
100 YEARS DURABILITY
Carpi Design Guidelines

- Avoiding concrete cover for ballast
- Realization of a network of access roads to the intakes for cleaning and maintenance
- Anchoring on vertical walls by tensioning SS profiles (CARPI PATENT)
- Anchoring on bottom and slopes by tensioning trenches (CARPI PATENT)
- Slopes and verticals → SIBELON CNT 4400
  (3,0 mm PVC + 500 gr/m² geotextile)
- Bottom area → SIBELON CNT 3750
  (2,5 mm PVC + 500 gr/m² geotextile)
Carpi Solution Advantages

• Trackable successful previous experience
• 40 years of experience in exposed solutions
• Tailor-made materials (100 years expected durability)
• No risks of damages during cover construction
• Easy and inexpensive maintenance and possibility of easy inspection
• Good behavior in case of seismic event
• Faster installation
• LESS OVERALL CONSTRUCTION COST
View of the installation test, Start of waterproofing works
Mock Up - September 2015
Tensioning Trenches Installation Sequence (Carpi patent)

• Stage 1: Preparation of subgrade
• Stage 2: Excavation of trenches
• Stage 3: Laying of geocomposite

8 m (slopes) /12 m (bottom)
Tensioning Trenches Installation Sequence (Carpi patent)

- Stage 4: Tensioning of geocomposite by filling remaining alternate trenches
Tensioning Trenches Installation Sequence (Carpi patent)

• Stage 5: Tensioning of geocomposite by filling remaining alternate trenches
Tensioning Trenches Installation Sequence (Carpi patent)

- Stage 6: Installation of geocomposite over ballasted trenches
Tensioning Trenches Installation Sequence
Tensioning Trenches Installation Sequence
Tensioning Trenches Installation Sequence
Tensioning Trenches Installation Sequence
Tensioning Trenches Installation Sequence
Tensioning Trenches Installation Sequence
Tensioning Trenches Installation Sequence
Tensioning Trenches

The bottom is perfectly flat avoiding formation of wrinkles and waves
Carpi Anchoring Solutions on slopes

- Punctual Rock anchors for vertical anchoring profiles
- Punctual Soil Nailing Anchors on Slopes
- Mechanical Perimeter Seal around concrete Structures
- Anchor trenches in the rock fill embankments
Carpi Anchoring Solutions

• Mechanical Perimeter Seal around concrete Structures, Joint treatment at the Dividing walls.
Carpi Anchoring Solutions

- Punctual Anchors
Carpi Anchoring Solutions

- Punctual Rock anchors for vertical anchoring profiles.
Carpi Anchoring Solutions

• Punctual Rock anchors for vertical anchoring profiles
Carpi Anchoring Solutions

• Punctual Rock anchors for vertical anchoring profiles
Carpi Anchoring Solutions

- Punctual Soil Nailing Anchors on Slopes.
Carpi Anchoring Solutions

• Punctual Soil Nailing Anchors on Slopes.
Carpi Anchoring Solutions

• Punctual Soil Nailing Anchors on Slopes.
Anchor trenches at the rock fill embankments (Carpi patent)
Anchor trenches at the rock fill embankments (Carpi patent)
Carpi Anchoring Solutions

- Anchor trenches at the rock fill embankments
Carpi Anchoring Solutions

• Anchor trenches at the rock fill embankments
Carpi Anchoring Solutions

• Anchor trenches at the rock fill embankments
Carpi Anchoring Solutions

- Anchor trenches at the rock fill embankments
Works concluded
Panama Canal Expansion 18 Water Saving Basins - 570,000 m²
The use of geosynthetics in water conveyance structures

The Panama Canal Expansion Project, Third Set of Locks Water Saving Basins

Thanks for your attention
Workshop 1 – Geosynthetics in Transportation Geotechnics
The Use of Geosynthetics in the Construction and Rehabilitation of Transportation Infrastructures in Portugal

José Neves¹, Helena Lima², Fernanda Rodrigues²

1. Instituto Superior Técnico, Universidade de Lisboa, Portugal
2. Infraestruturas de Portugal, Lisboa, Portugal
Main goals

1. To present the Work Group WG2 of the Portuguese Committee on Transportation Geotechnics

2. To summarize the Portuguese experience on the use of geosynthetics in road pavements and rail tracks
Topics

1. Introduction
2. Road pavements
3. Rail tracks
4. Conclusions
1. Introduction

Road network in Portugal: motorways

<table>
<thead>
<tr>
<th>Year</th>
<th>Length (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1990</td>
<td>316</td>
</tr>
<tr>
<td>2012</td>
<td>2,988</td>
</tr>
</tbody>
</table>

(Total length of road network - 14,284 km)

Source: Portuguese Network Directory, 2016
1. Introduction 2/4

Rail network in Portugal: railway lines in operation

<table>
<thead>
<tr>
<th>Year</th>
<th>Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>1990</td>
<td>3,582 km</td>
</tr>
<tr>
<td>2015</td>
<td>2,546 km</td>
</tr>
</tbody>
</table>

1,935 km (single track)
611 km (multiple track)

Source: Portuguese Network Directory, 2016
1. Introduction

Road: LENGTH OF ROAD NETWORK

<table>
<thead>
<tr>
<th>Motorways</th>
<th>Main or national roads</th>
<th>Secondary or regional roads</th>
<th>Other roads (*)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BE 1763</td>
<td>13 229</td>
<td>13 491</td>
<td>138 869</td>
</tr>
<tr>
<td>BG 541</td>
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Railways: LENGTH OF LINES IN USE

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Source: EU Transport in figures, Statistical Pocketbook, 2015

6th

19th

Workshop 1: Geosynthetics in Transportation Geotechnics
1. Introduction

Portuguese Committee on Transportation Geotechnics (Portuguese Geotechnical Society)

Working Group WG2 (created in 2012)

Reinforcement of geomaterials and its implications in pavement and rail track design

GEOreinforce

www.georeinforce.pt

12 members:

- Universities
- Laboratories
- Companies
- Road and Rail Agency
2. Road pavements 1/2

Subgrade
Distribution of the use of geosynthetics (2001-2012): 500,000 m²; 45 road works

Type of use
- Road construction: 58%
- Slope stabilization: 9%
- Viaducts and bridges rehabilitation: 9%
- Pavement rehabilitation: 24%

Type of geosynthetics
- Geotextiles: 94%
- Geogrids: 6%
2. Road pavements 2/2

Subgrade - Quantities and costs

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<th>Geogrids</th>
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<td><img src="geogrids_chart.png" alt="" /></td>
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**45,000 m²/year:**
- 44,300 m² geosynthetics
- 700 m² geogrids

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<tr>
<td>• 1.50 €/m² geosynthetics</td>
<td>• 7.80 €/m² geogrids</td>
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3. Rail tracks 1/3

Use of geosynthetics

- Slope stabilization
- Drainage/filtration
- Reinforcement

Example of geogrid-reinforced ballast layer (2016)
North Line Railway
Alfarelos/Pampilhosa - km 194,500 to km 218,000
- Geogrid under the layer of ballast
- Quantities:
  - Geogrid: 8,740 m²
  - Composite of geogrid and nonwoven geotextile: 34,580 m²
3. Rail tracks 2/3

Cross-section of rehabilitated rail track
North Line Railway - Alfarelos/Pampilhosa - km 194,500 to km 218,000

Passengers/Cargo Loads: 22.5 ton/axle
Ballast: granite
Geogrid: biaxial
3. Rail tracks 3/3

Placement of the geogrid under the layer of ballast
North Line Railway - Alfarelos/Pampilhosa - km 194,500 to km 218,000

Track-mounted undercutting machine that rolls out the geogrid prior to new ballast being dropped in place over the geogrid
4. Conclusions

Road pavements

✓ In the case of soft subgrade and in order to improve the pavement bearing capacity, the use of geosynthetics was often a suitable solution.

Rail tracks

✓ In general, the geotextiles have been applied in various functions (separation, reinforcement, drainage, filtration) in the rehabilitation of the existing railways. However, the geogrids are only being applied as reinforcement with more significance since 2016.
The Use of Geosynthetics in the Construction and Rehabilitation of Transportation Infrastructures in Portugal

Obrigado !

Thank you !