3rd ICTG 2016

04-07 September 2016, Guimarães, Portugal

University of Minho
School of Engineering

Workshop 1: Geosynthetics in Transportation Geotechnics

Presentations e-Book

4th September 2016 | School of Engineering | University of Minho | Guimarães | Portugal

Sponsored by

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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

University of Minho (UM)
Portuguese Geotechnical Society (SPG)
Portuguese Chapter of the International Geosynthetic Society (IGS)
International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE)

Venue

University of Minho, School of Engineering, Guimarães, Portugal

Website

http://civil.uminho.pt/3rd-ICTG2016/WorkshopsThemes.php

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Preface

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

The main goals of the workshop are:

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

Universidade do Minho
Escola de Engenharia

Audience …

Sponsors exhibition …

Lunch …

Restaurant …

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

1

2

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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

4

5

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

6

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8

10

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

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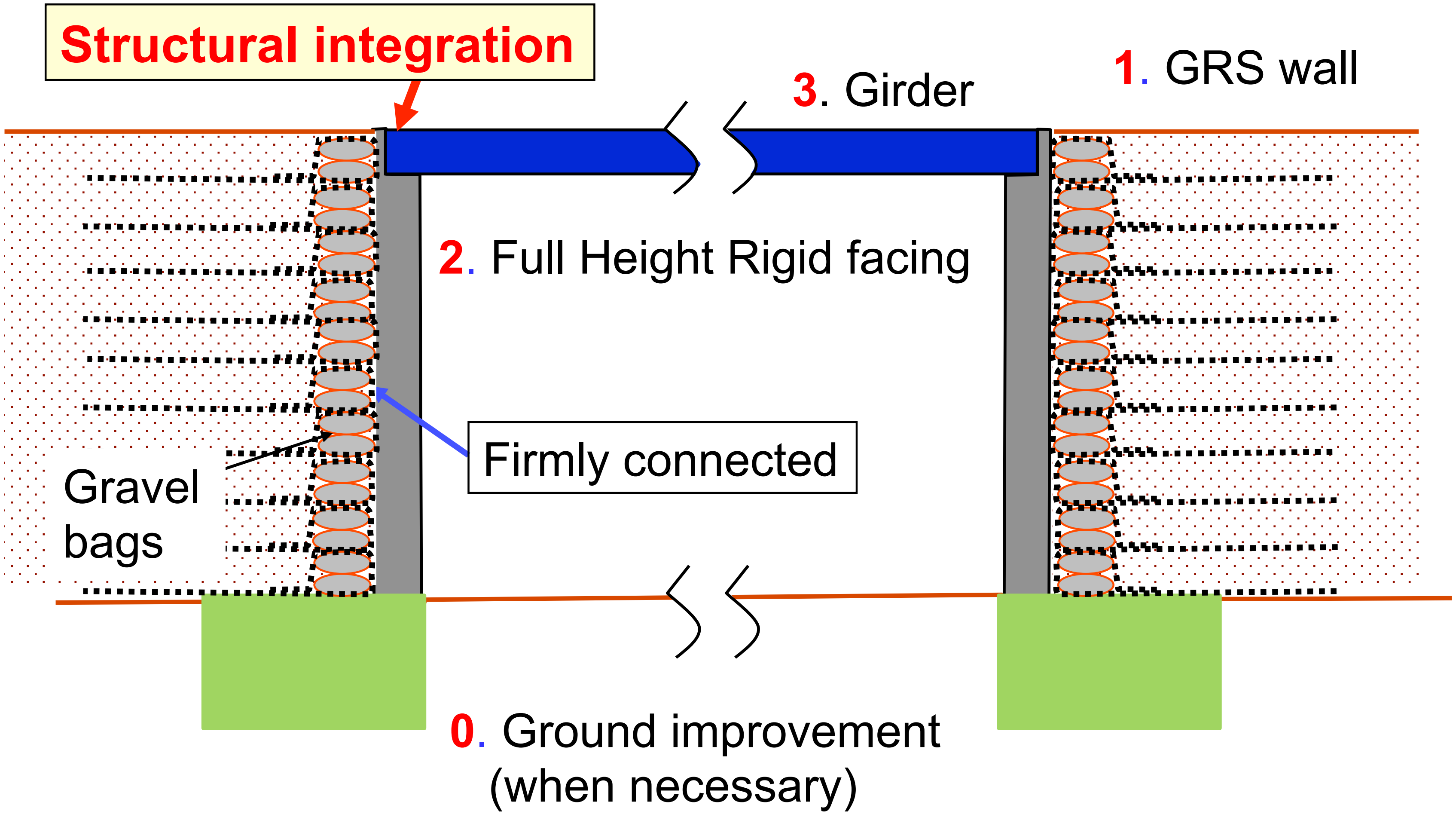
Research and Construction of Geosynthetic-Reinforced Soil Integral Bridges

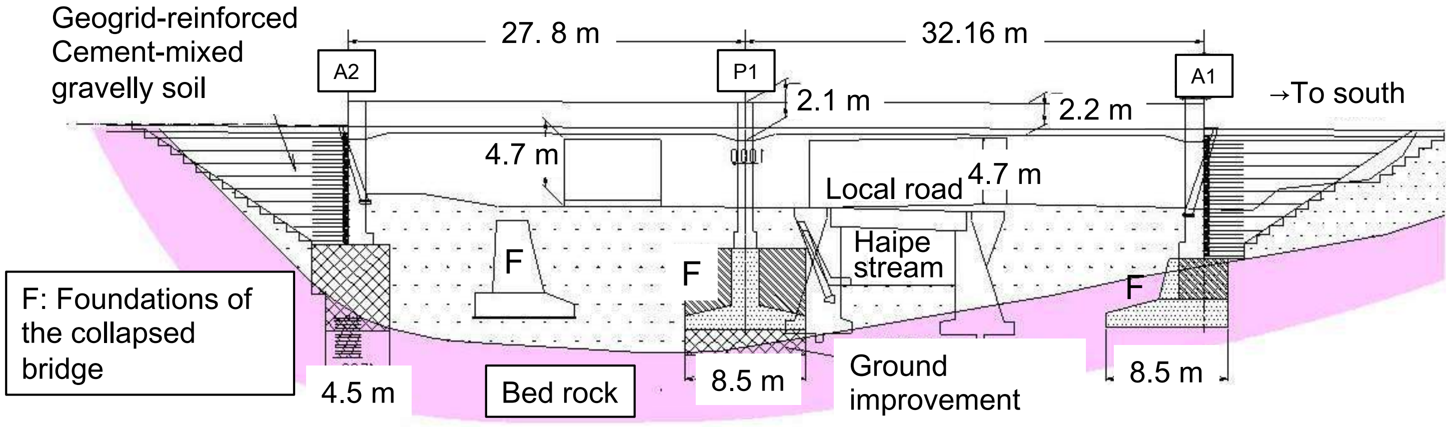
Fumio Tatsuoka[1], **Masaru Tateyama**[2], **Masayuki Koda**[3], **Kenichi Kojima**[2],
Toyoji Yonezawa[4], **Yoshinori Shindo**[4] and **Shin-ichi Tamai**[4]

[1] Tokyo University of Science, Chiba, Japan (presenting author)
[2] Railway Technical Research Institute, Tokyo, Japan
[3] East Japan Railway Company
[4] Japan Railway Construction, Transport and Technology Agency

GRS Integral Bridge



GRS integral bridge at Haipe, Sanriku Railway



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

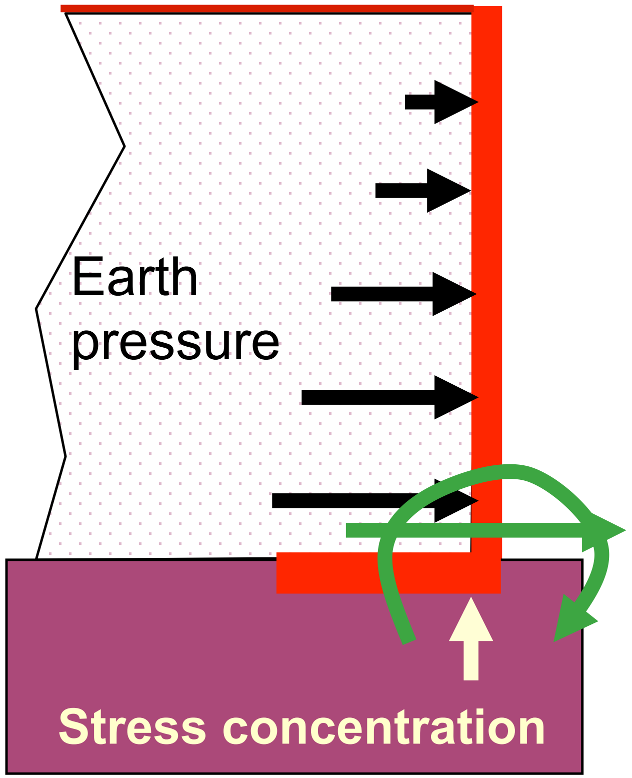
1. **Advantages of GRS RWs with staged-constructed full-height rigid facing**
   – the basic technology for GRS integral bridge

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

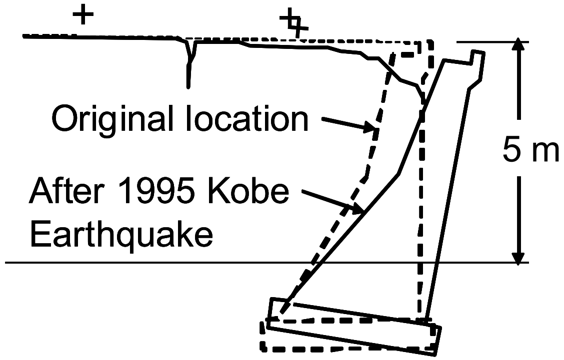


Large forces in the facing & large overturning moment & large lateral load at the facing bottom

Needs for a massive & strong facing & a pile foundation

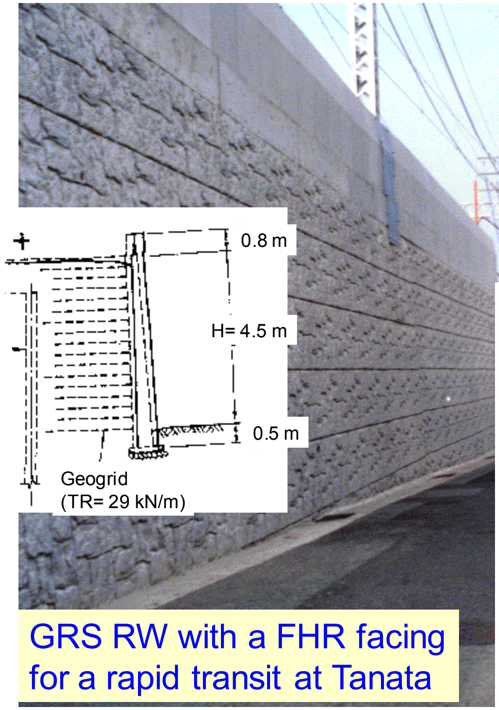
Relatively low stability, particularly against seismic loads→

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

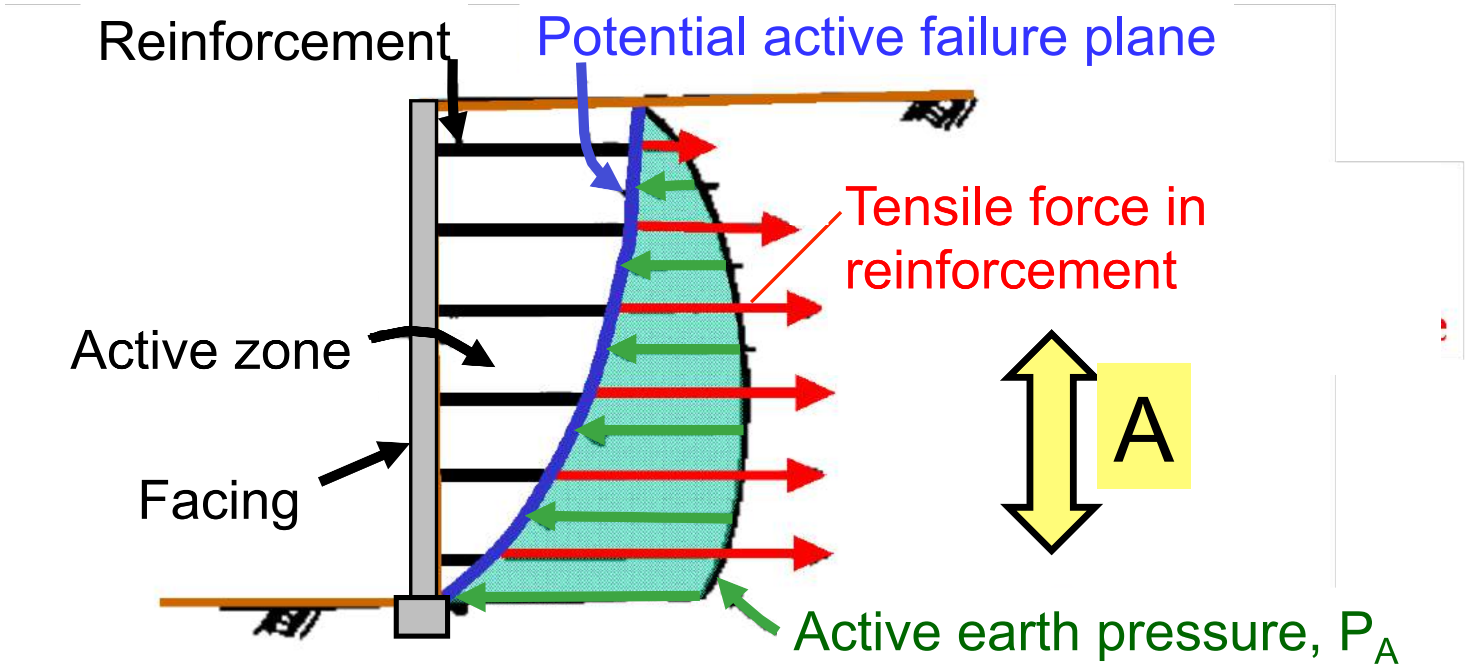


GRS RW with a FHR facing for a rapid transit at Tanata

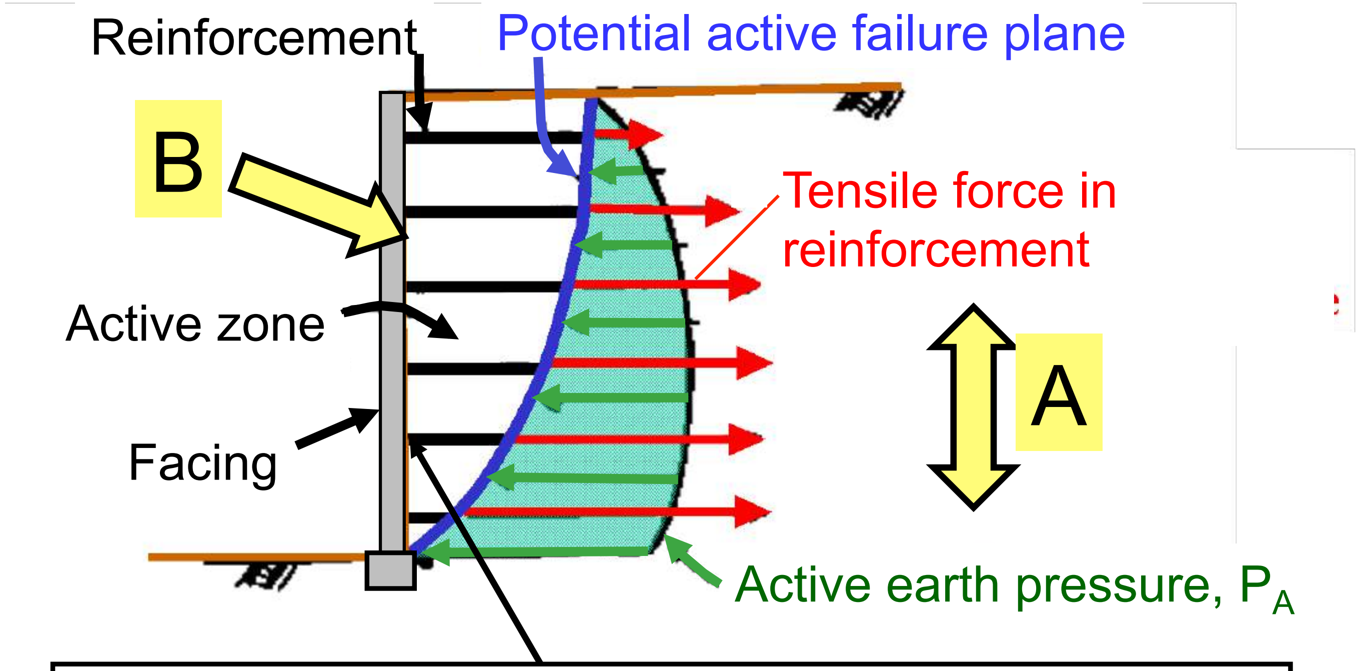
A week after the 1995 Kobe Earthquake

The wall survived!

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

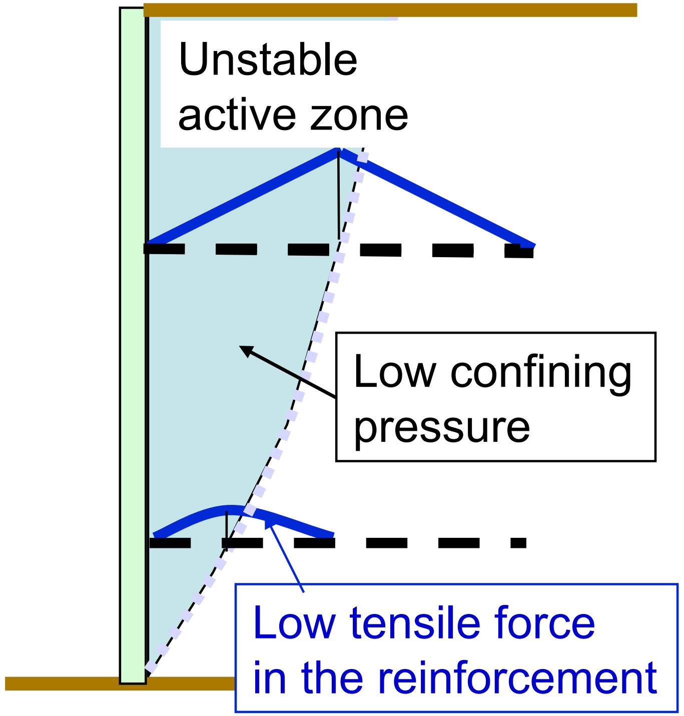


Two basic force equilibriums with reinforced soil walls:
(A) along the potential active failure plane
→→always considered in design
(B) at the facing → very important, but often ignored



Paramount importance of connection strength

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



Unstable active zone

Low confining pressure

Low tensile force in the reinforcement

→ No earth pressure at the wall face

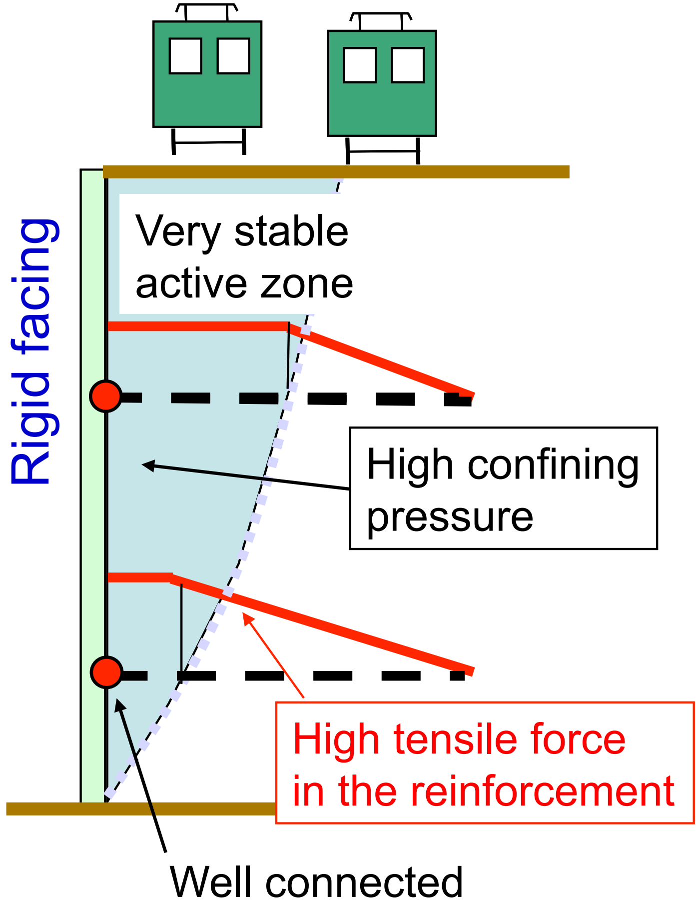
→ Low tensile forces in the reinforcement, in particular at the low wall level

→ 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



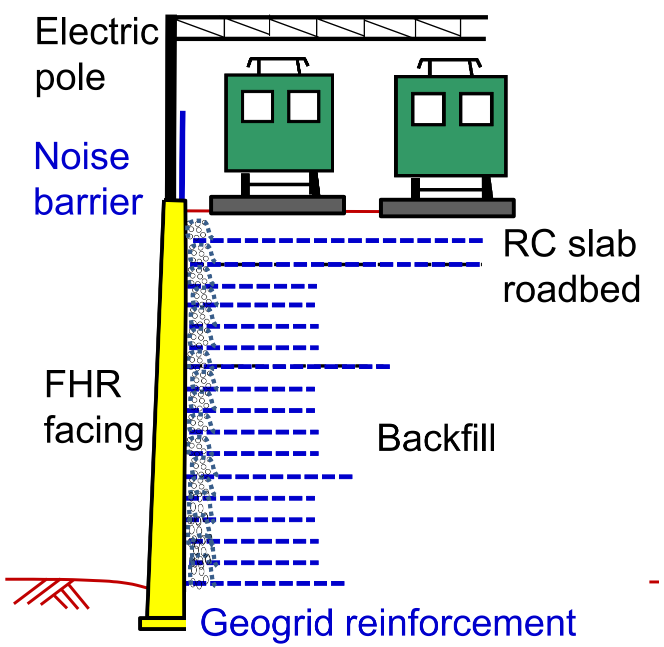
→ High earth pressure at the wall face

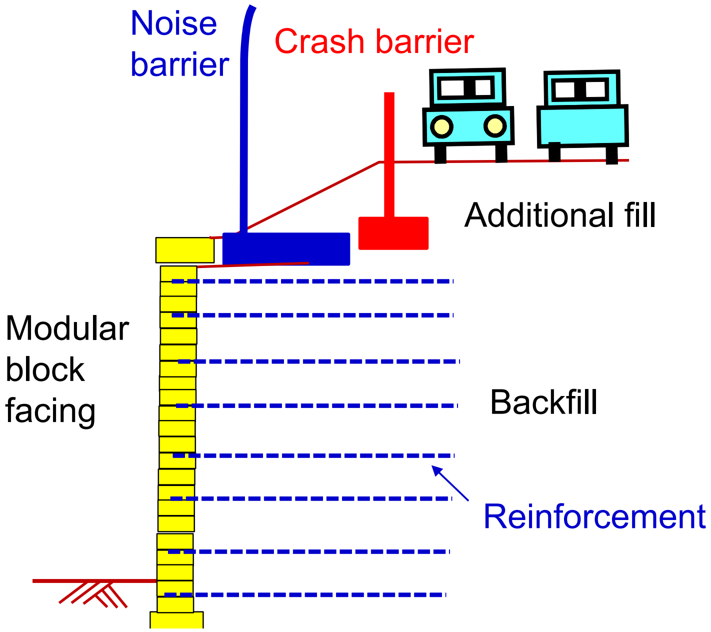
→ High tensile forces in the reinforcement

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

→ High stability of the wall

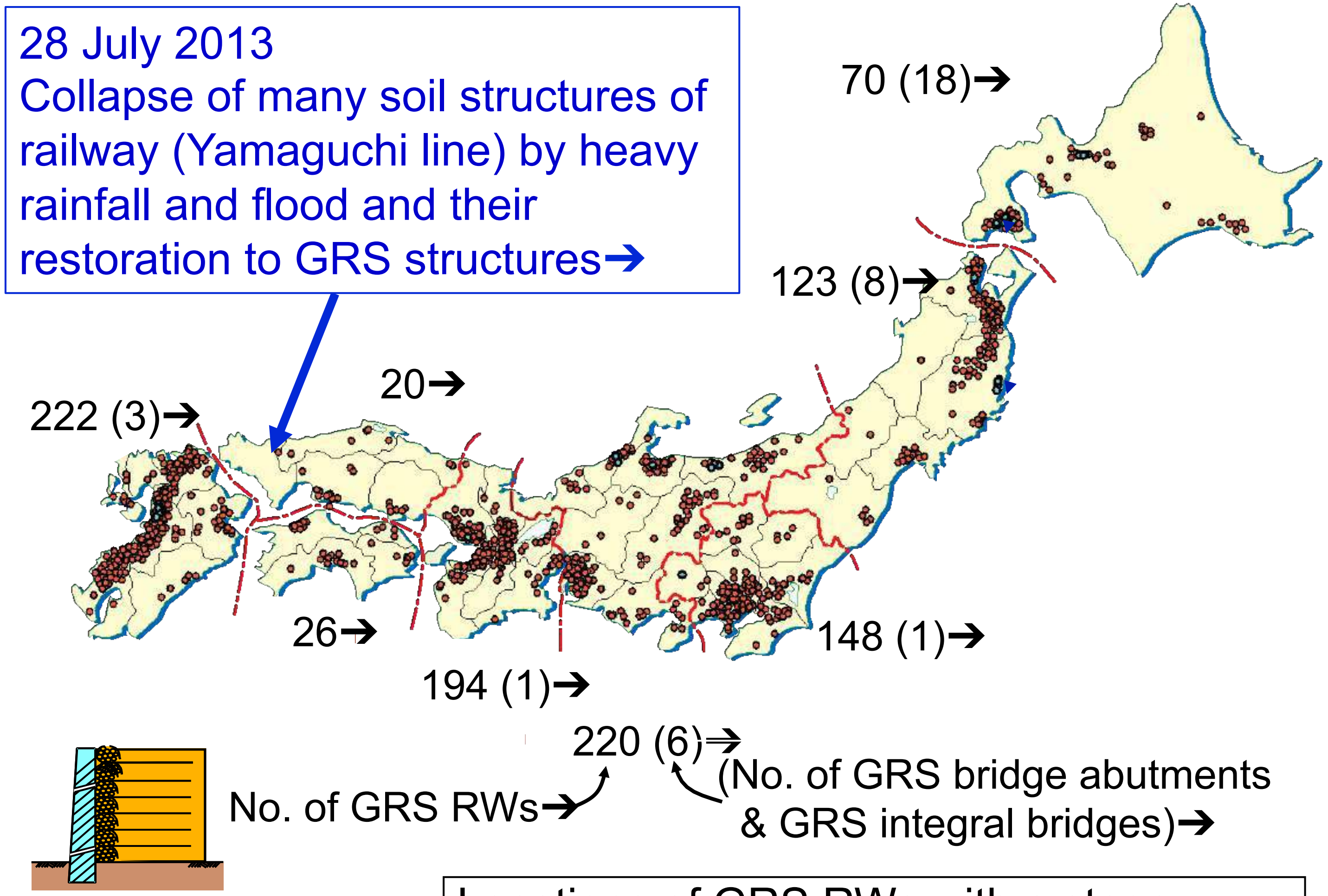
FHR facing
versus
discrete panel/block facing





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)：

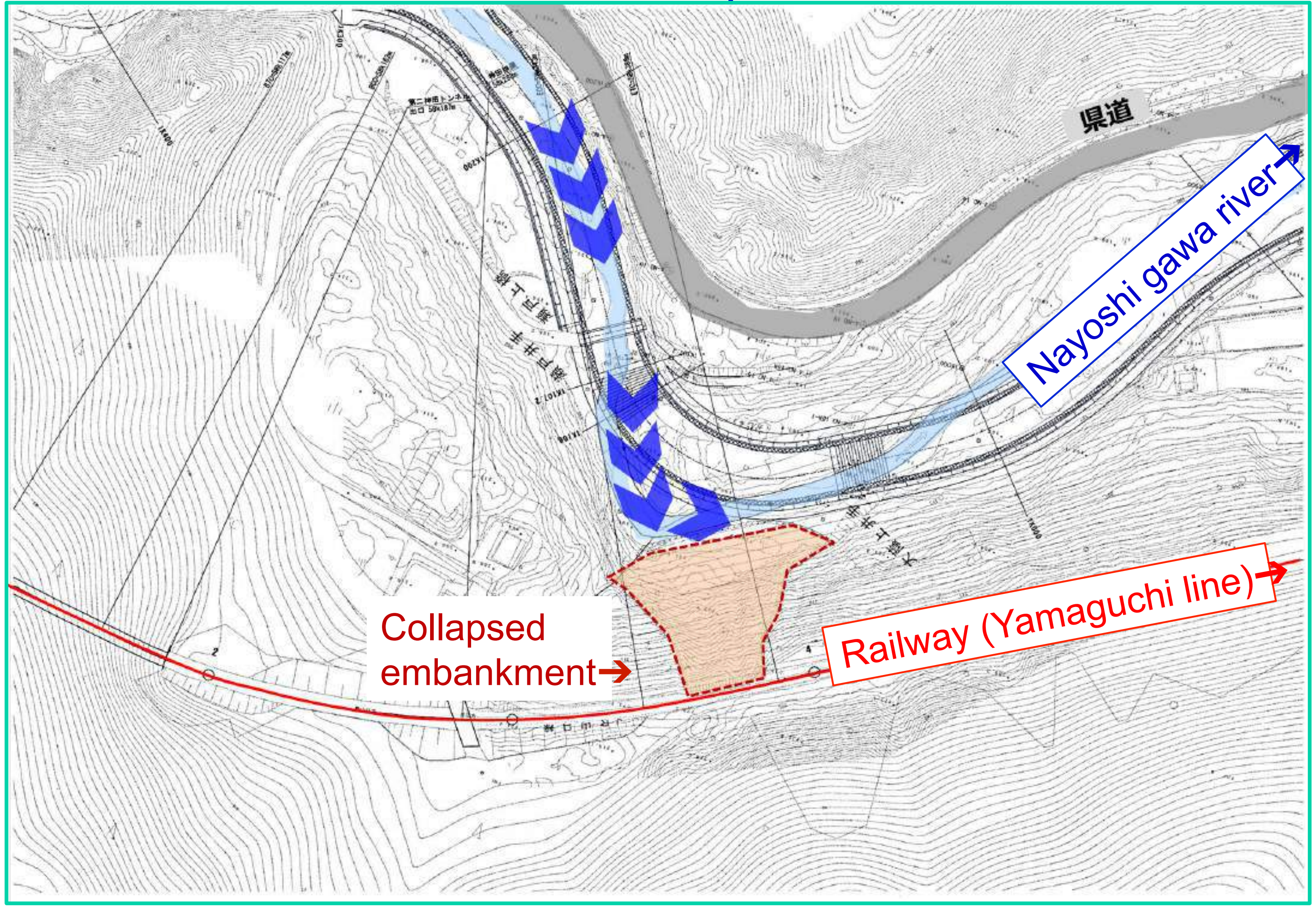
■ Flood in the nearby river ⇒➔Scouring in the subsoil supporting the facing ⇒➔Displacement/deformation of the facing ⇒➔ Displacement of the support of the girder ⇒➔Dislodging of girder-



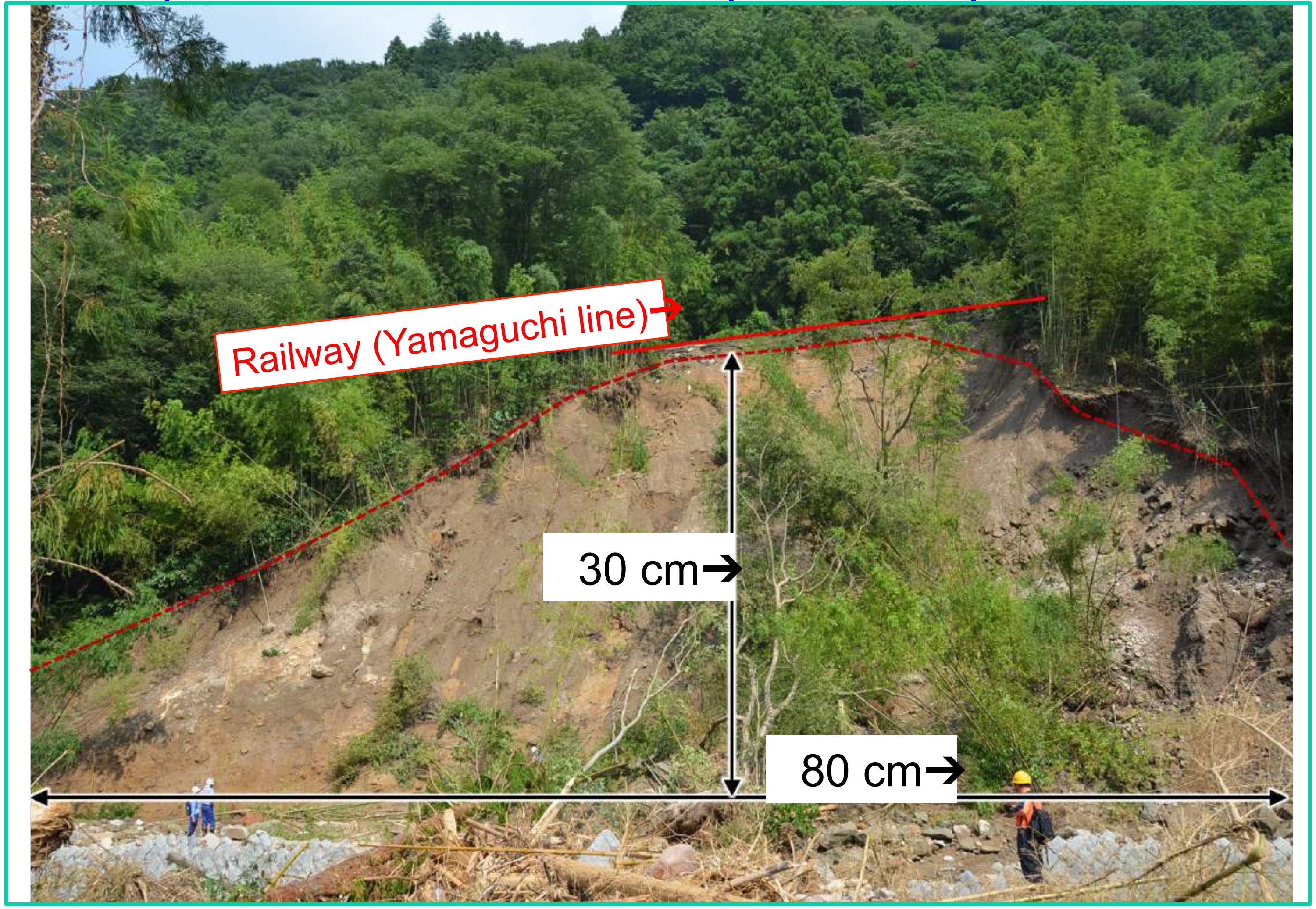
28 July 2013
Collapse of many soil structures of railway (Yamaguchi line) by heavy rainfall and flood and their restoration to GRS structures➔

Locations of GRS RWs with a stage-constructed FHR facing as of June 2014

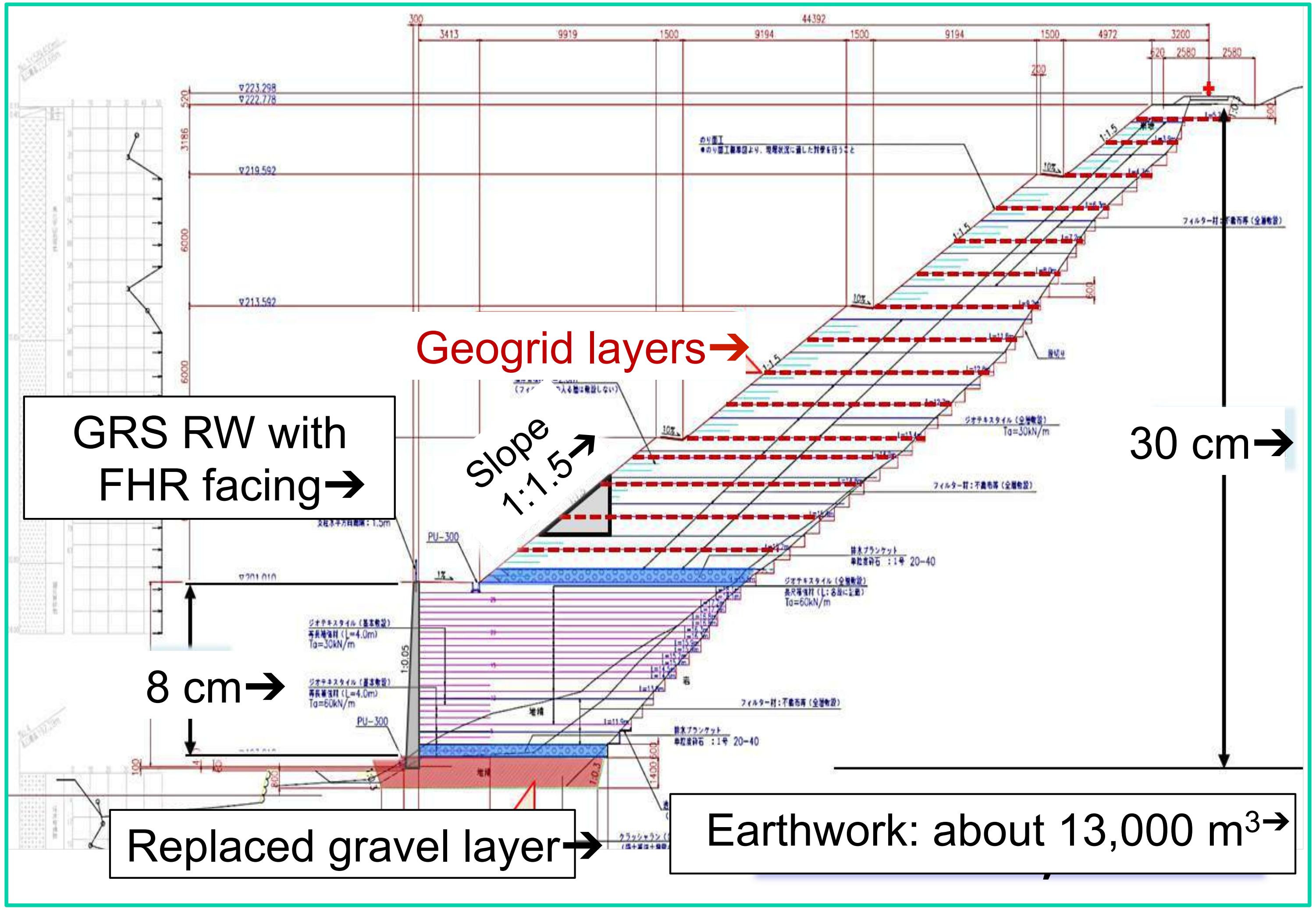
Flood from a river attacking the embankment



Collapse of embankment by scouring at the toe



Restoration to GRS structure



Geogrid layers→

GRS RW with FHR facing→

Slope 1:1.5→

30 cm→

8 cm→

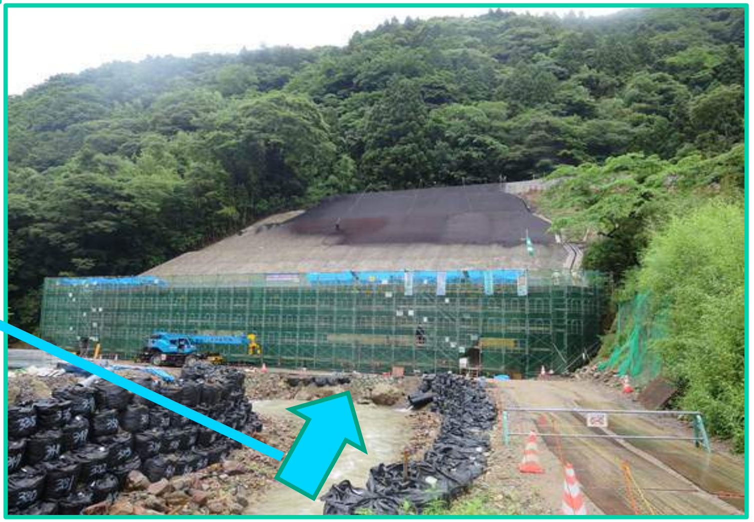
Replaced gravel layer→

Earthwork: about 13,000 m^3→

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➔



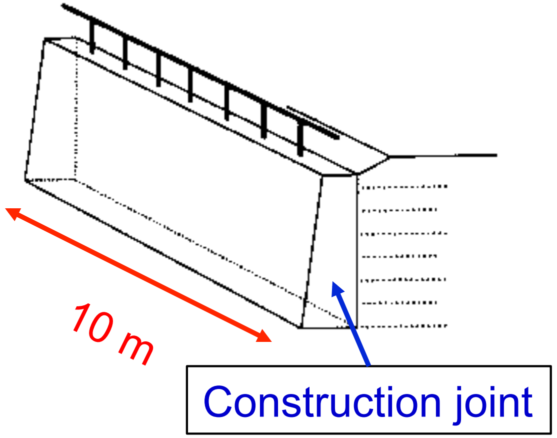
Restoration to GRS structure

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

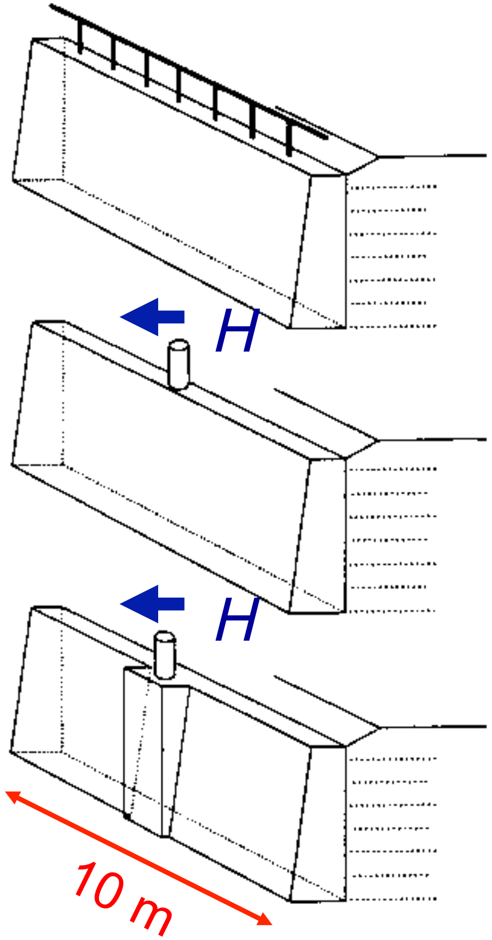
→ Even if local failure is going to take place somewhere in the wall, it does not develop towards the collapse of the whole wall.



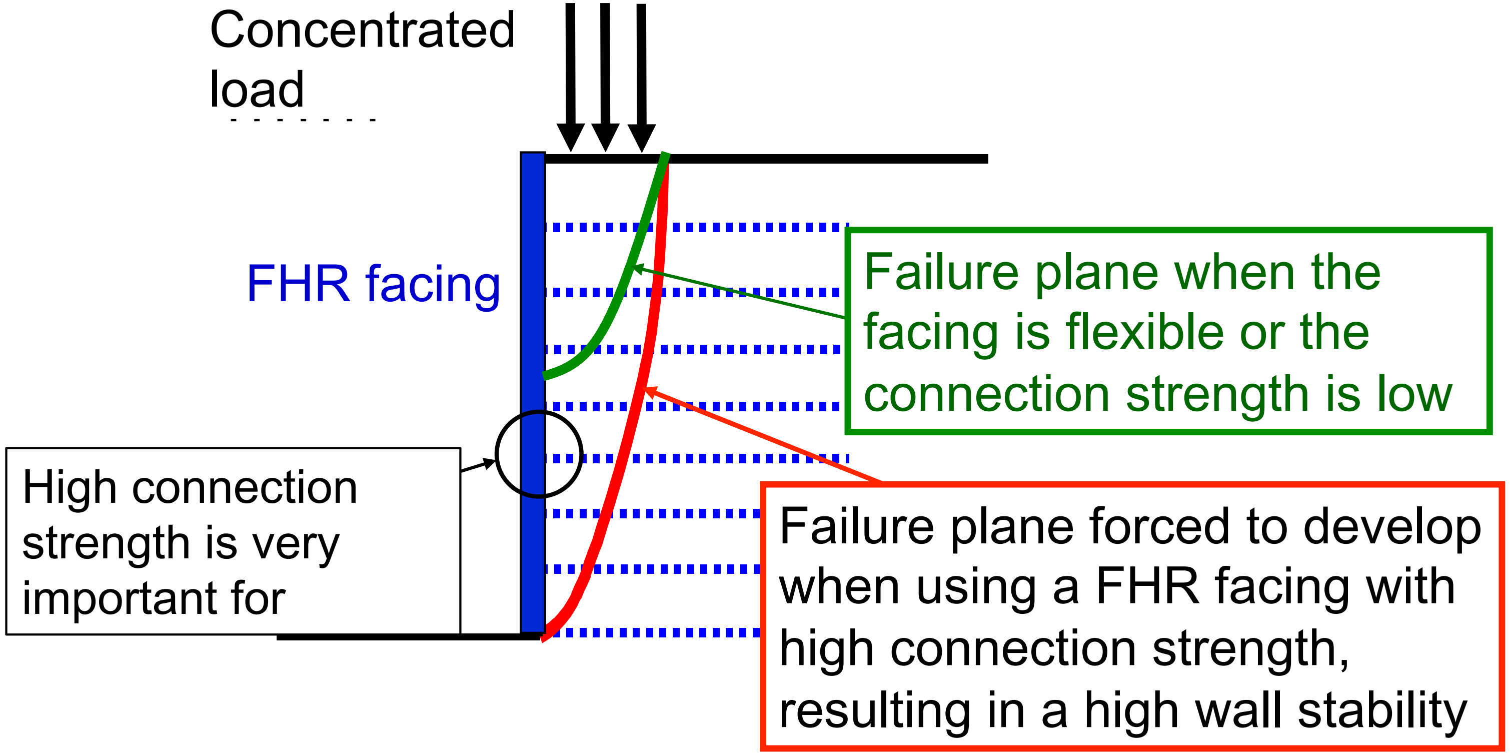
3D effects of full-height rigid (FHR) facing!

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



Concentrated load

FHR facing

Failure plane when the facing is flexible or the connection strength is low

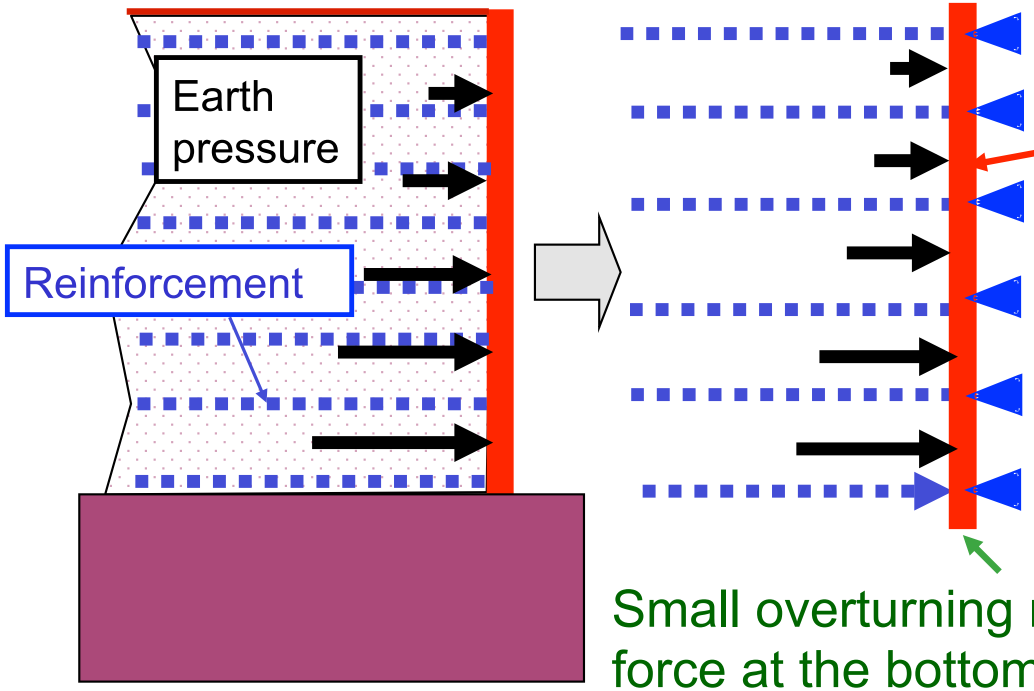
High connection strength is very important for

Failure plane forced to develop when using a FHR facing with high connection strength, resulting in a high wall stability

Tatsuoka et al. (1989) *12ISMFE, Rio de Janeio*

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"*

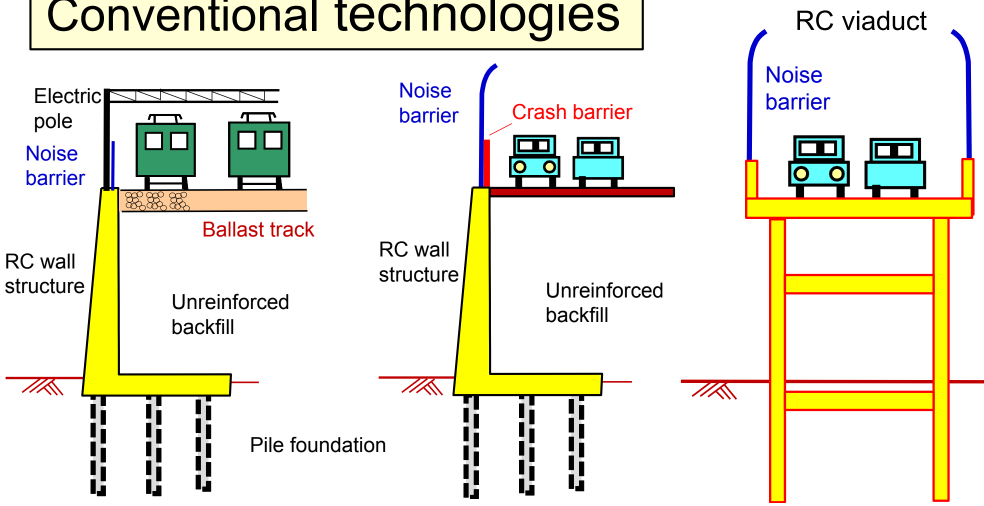


Very small forces in the facing ⇒➔ simple facing structure

Small overturning moment & lateral force at the bottom of the facing ⇒➔ usually, no need for a pile foundation!

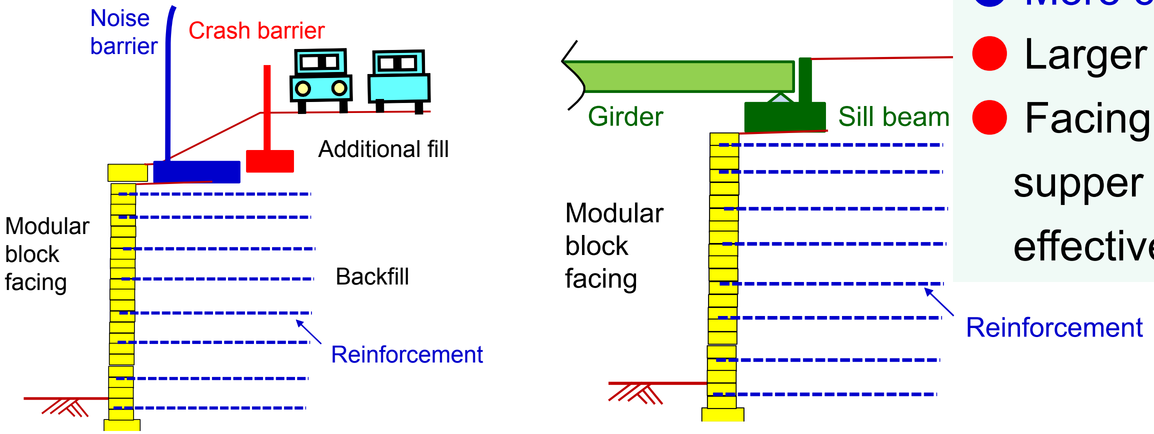
Stable, particularly against seismic loads➔

Conventional technologies



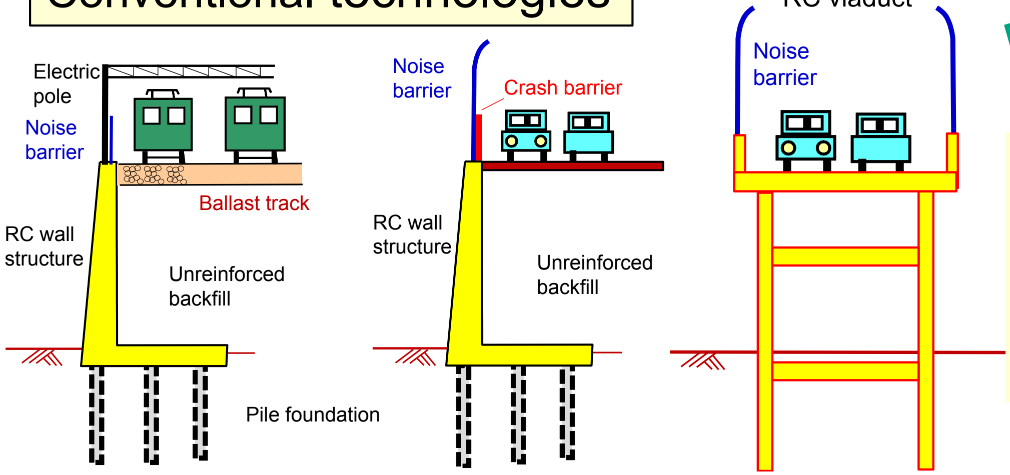
- Limited occupied space
- Facing supports supper structures

MSE technologies with discrete facing



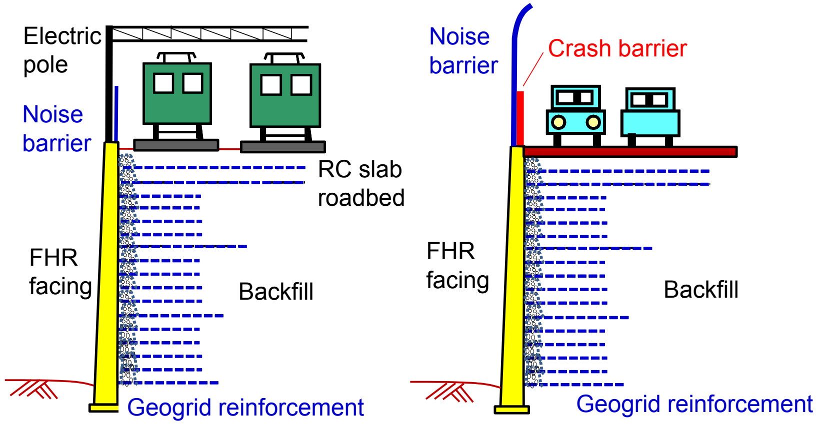
- More cost-effective, but
- Larger occupied space
- Facing does not support supper structures effectively

Conventional technologies



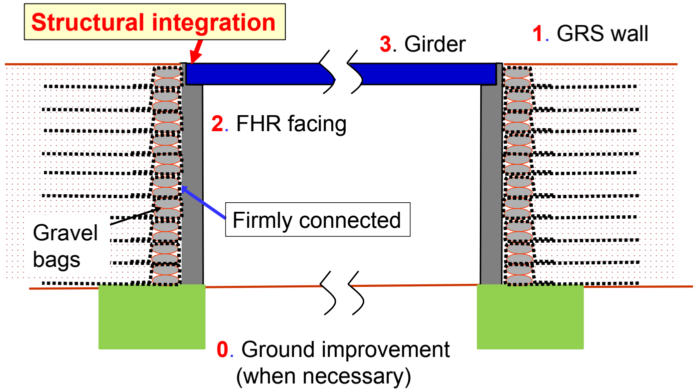
- ● Limited occupied space
- ● Facing supports supper structures

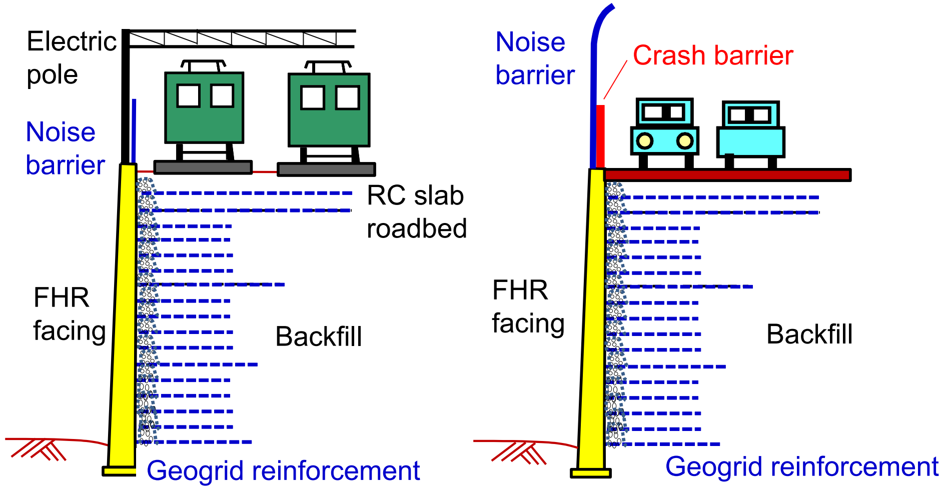
GRS technologies with staged constructed FHR facing



- ● And more cost-effective !

GRS integral bridge





JR Kobe line, Amagasaki:

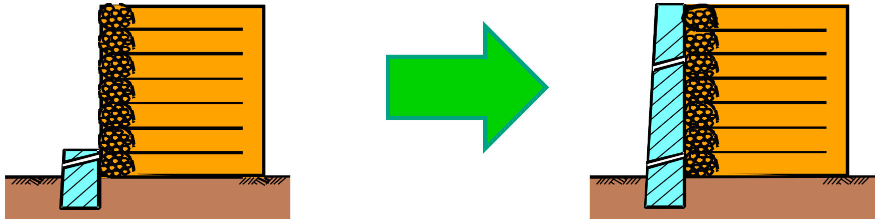
In this case, ballasted track

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.

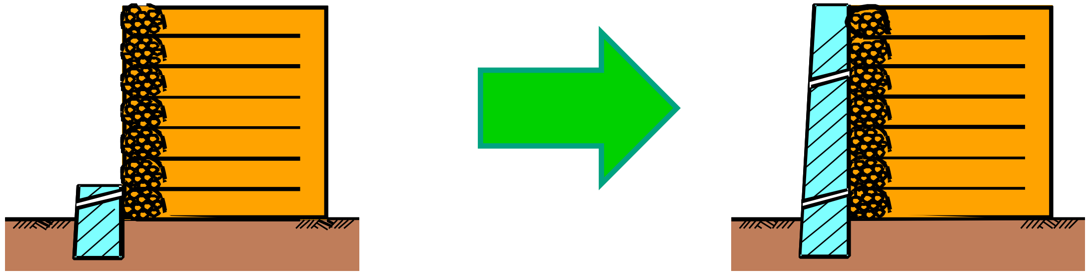
2) The earth pressure at the facing should be **high** enough to provide sufficient confining pressure to the backfill.

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**.



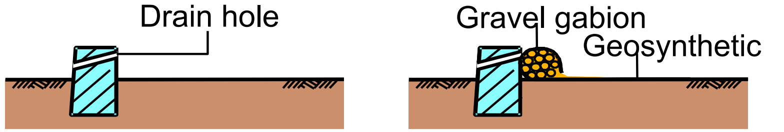
Staged construction of FHR facing

- Why necessary
- How to do
- Benefits➔



Staged construction - 1:

- Construction with a help of gravel gabions placed at the shoulder of each soil layer

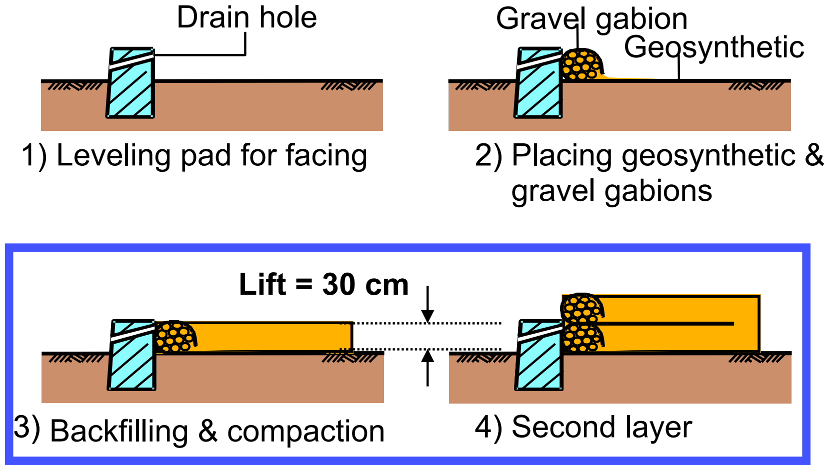


1) Leveling pad for facing

2) Placing geosynthetic & gravel gabions

Staged construction - 2:

- Construction with a help of gravel gabions placed at the shoulder of each soil layer

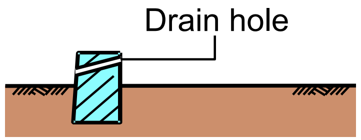


1) Leveling pad for facing
2) Placing geosynthetic & gravel gabions
3) Backfilling & compaction
4) Second layer

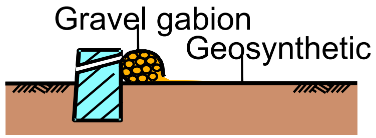
Good compaction of the backfill by:
1) a small lift resulting from a small vertical spacing of reinforcement layers; and
2) no rigid facing during backfill compaction

Staged construction - 3:

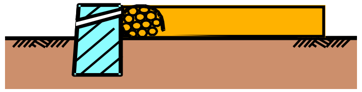
- Construction with a help of gravel gabions placed at the shoulder of each soil layer



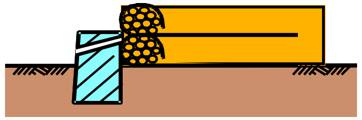
1) Leveling pad for facing



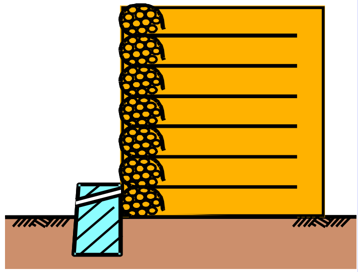
2) Placing geosynthetic & gravel gabions



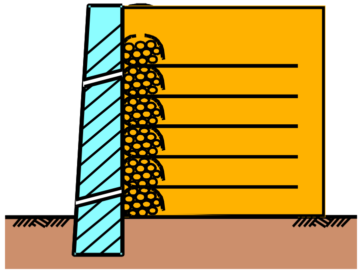
3) Backfilling & compaction



4) Second layer



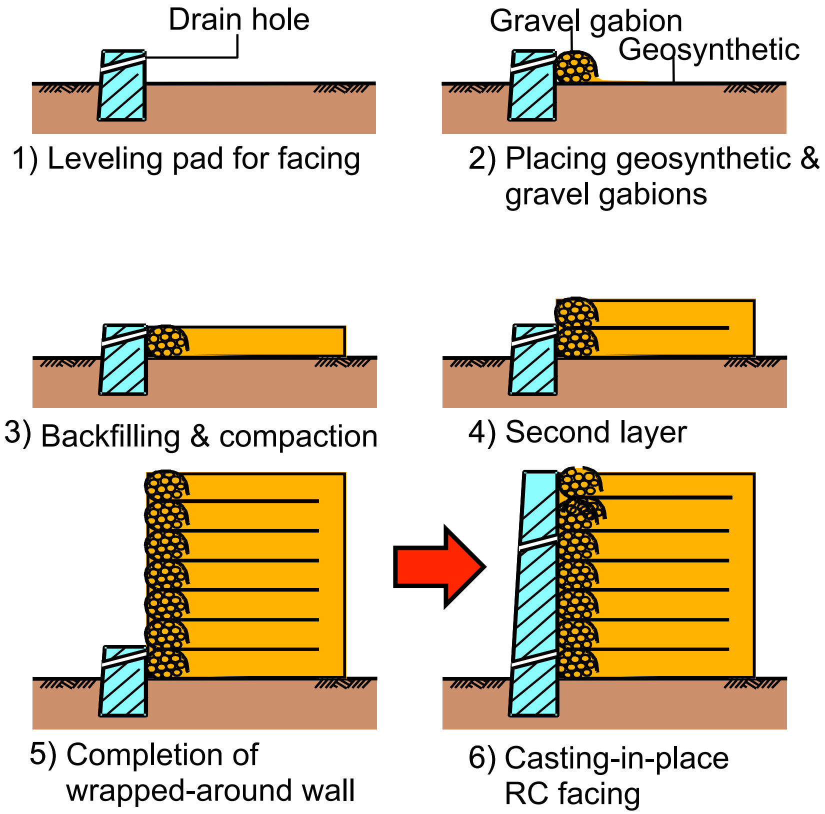
5) Completion of wrapped-around wall



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.



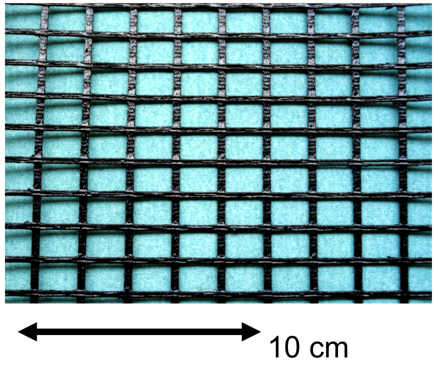
→ The facing/ reinforcement connection is not damaged during and after construction.

→ Construction using compressive backfill on a compressive soil layer becomes possible.

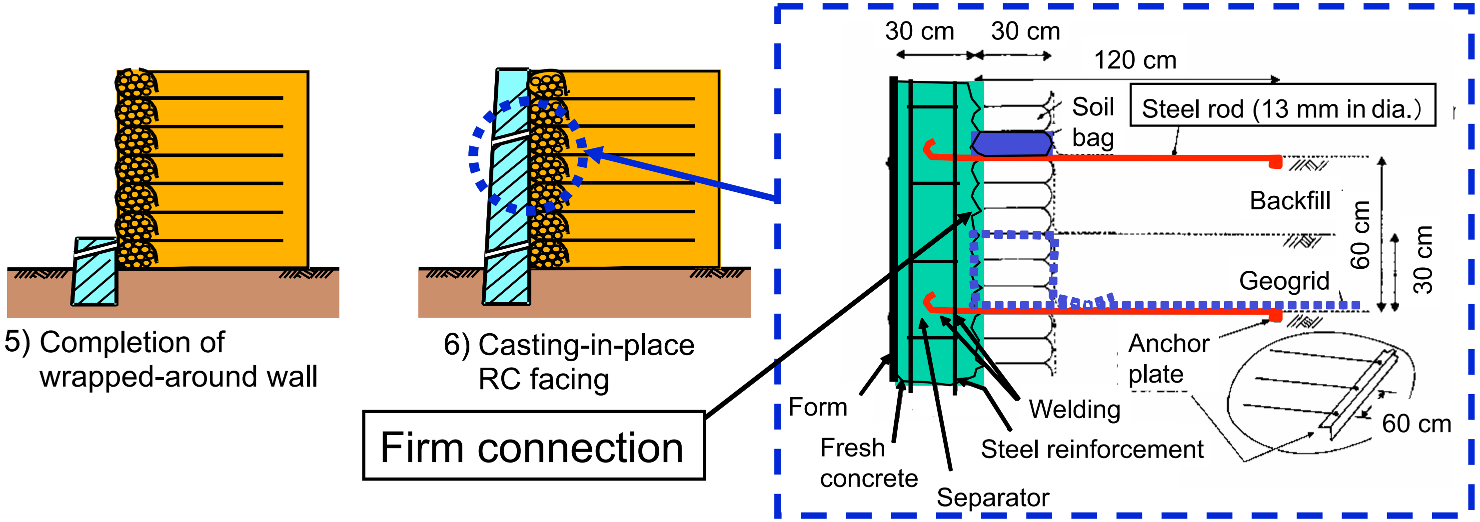
Casting-in-place concrete directly on the geogrid-wrapping-around wall face:

1) Fresh concrete enters the gravel bags through the aperture of the geogrid (PVA has vey 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

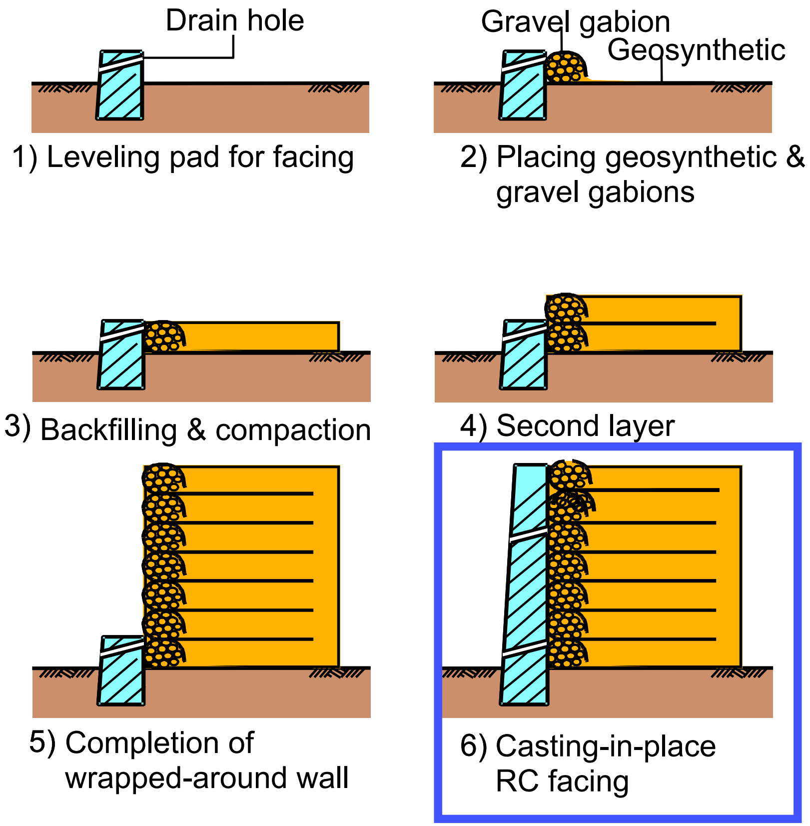


5) Completion of wrapped-around wall

6) Casting-in-place RC facing

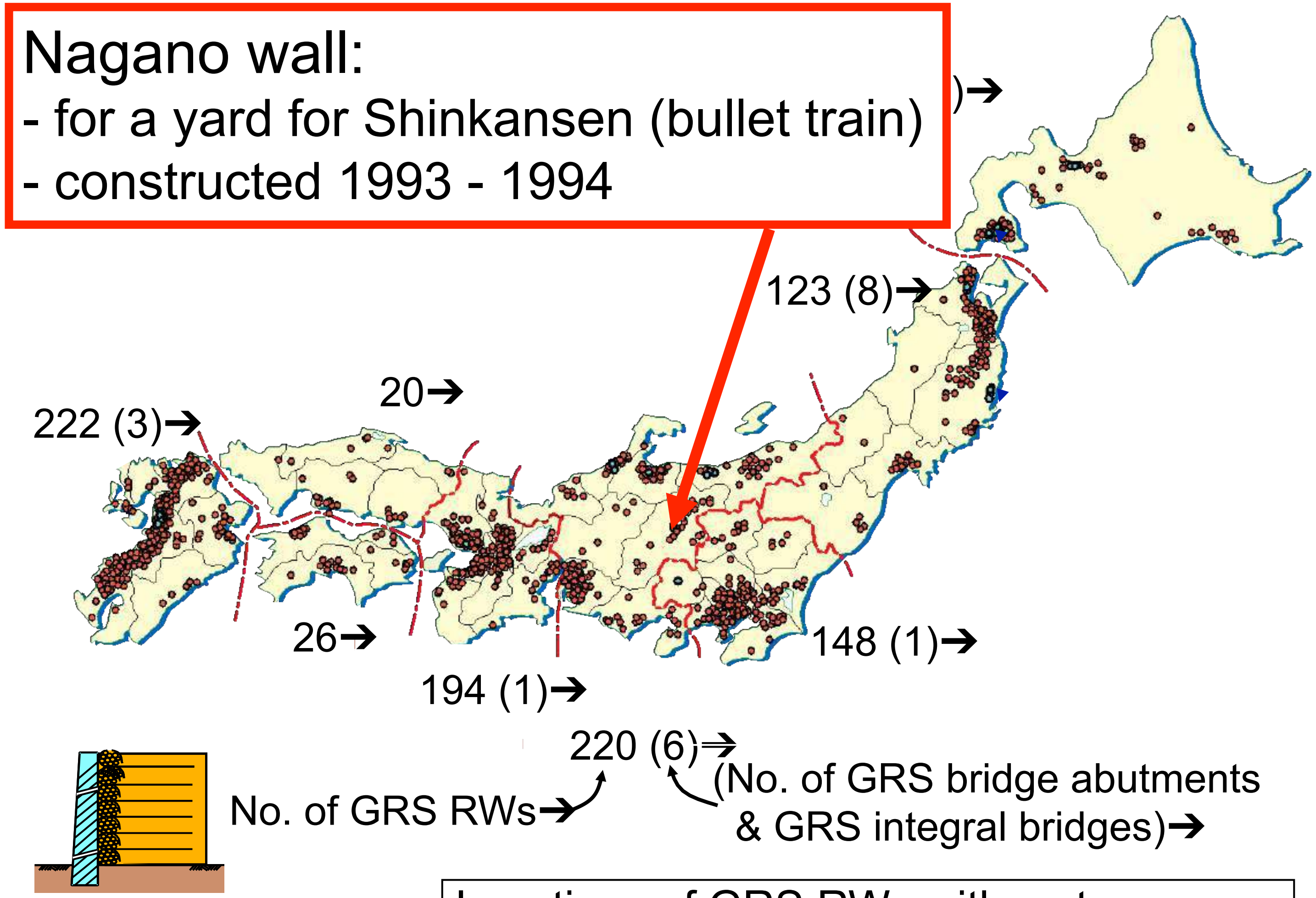
Staged construction - 4:

- 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.



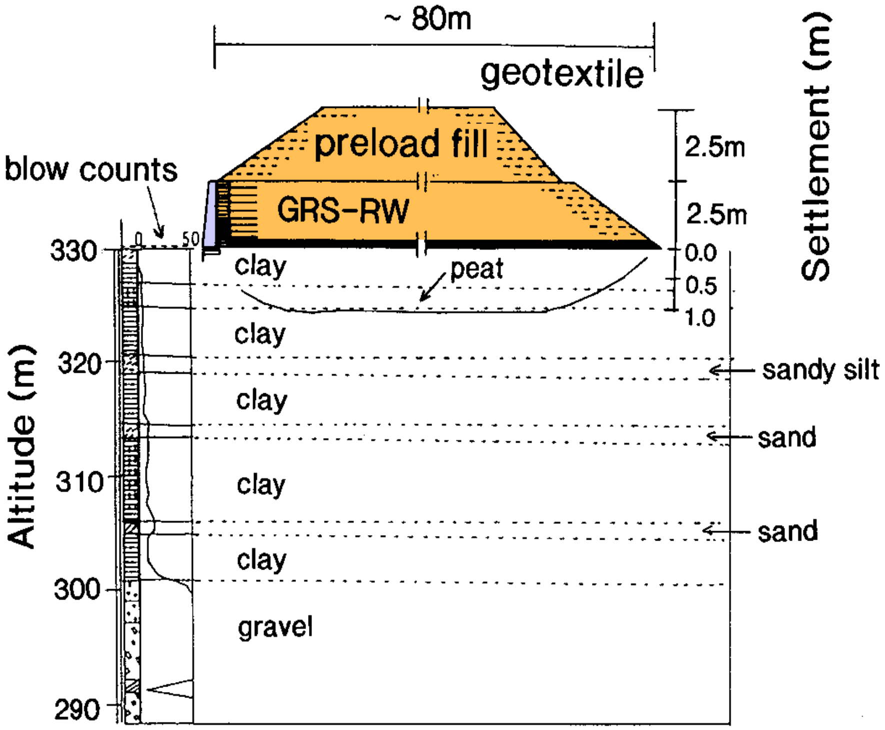
Nagano wall:
- for a yard for Shinkansen (bullet train)
- constructed 1993 - 1994

Locations of GRS RWs with a stage-constructed FHR facing as of June 2016

Nagano wall:

-for a yard for Shinkansen (bullet train)

- constructed 1993 - 1994



(Tatsuoka et al., 1997)

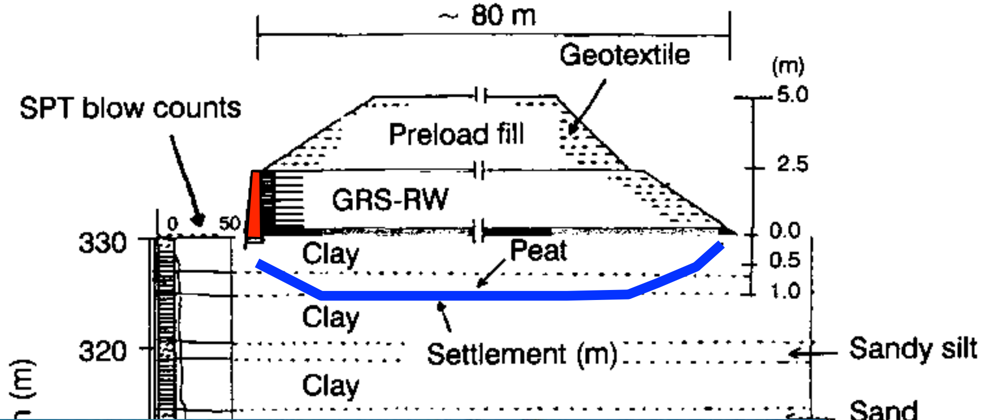
a) GRS RW, 2 m-high & 2 km-long, supporting a yard for a new bullet train line

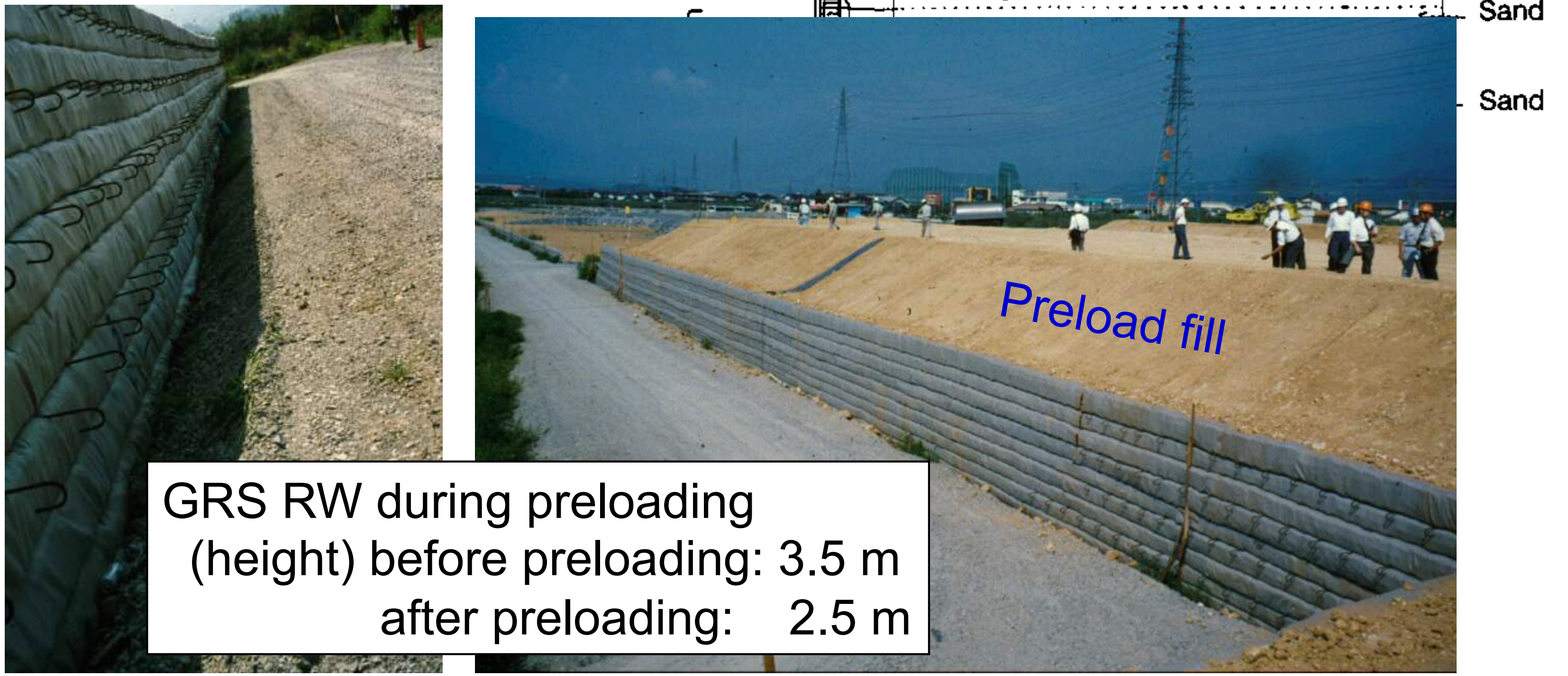
b) Backfill: nearly saturated soft clay

c) 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

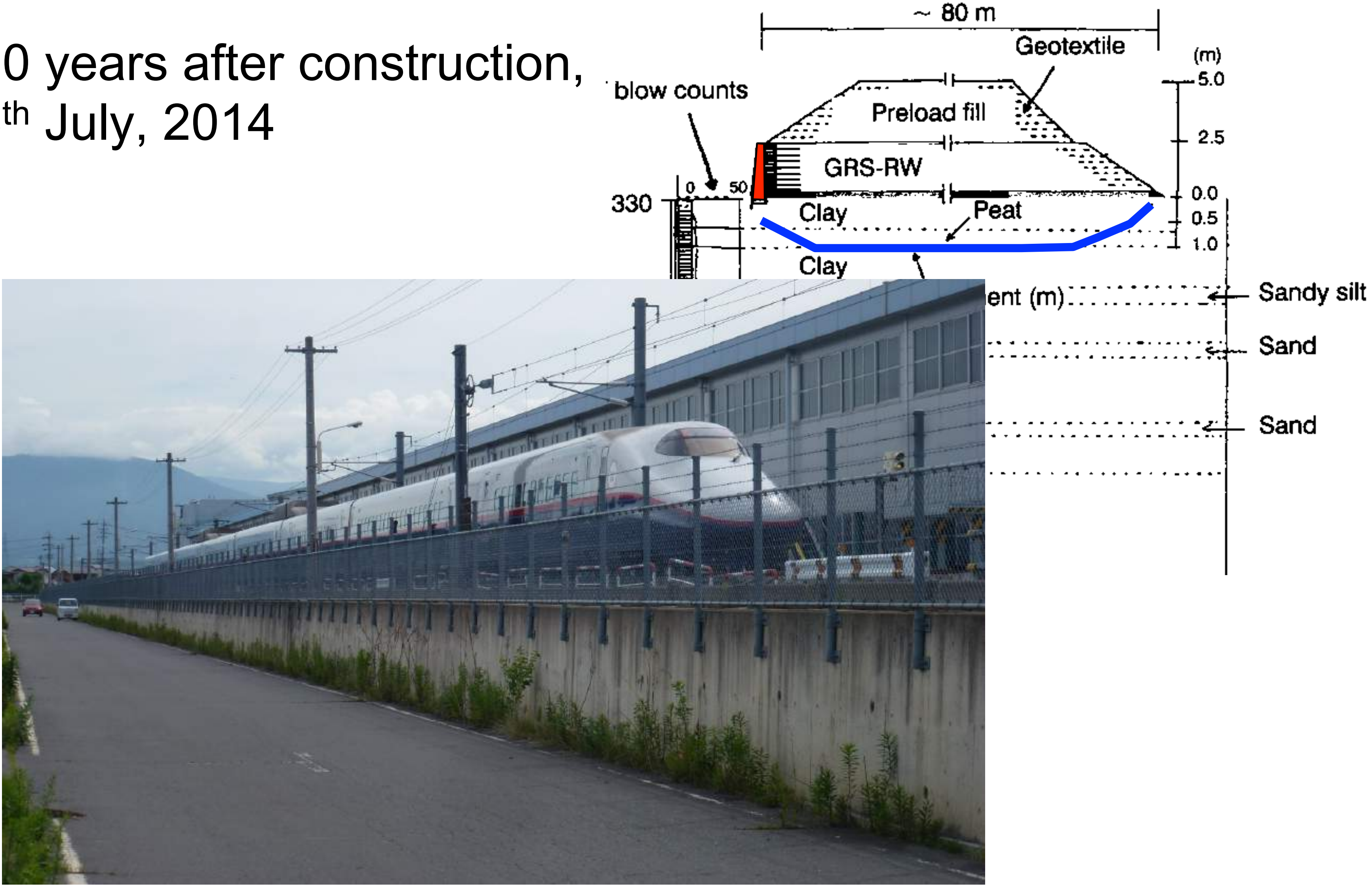
- Settlement of the embankment by preloading: about 1 m
- Casting-in-place of FHR facing after removing the preload fill.





GRS RW during preloading
(height) before preloading: 3.5 m
after preloading: 2.5 m

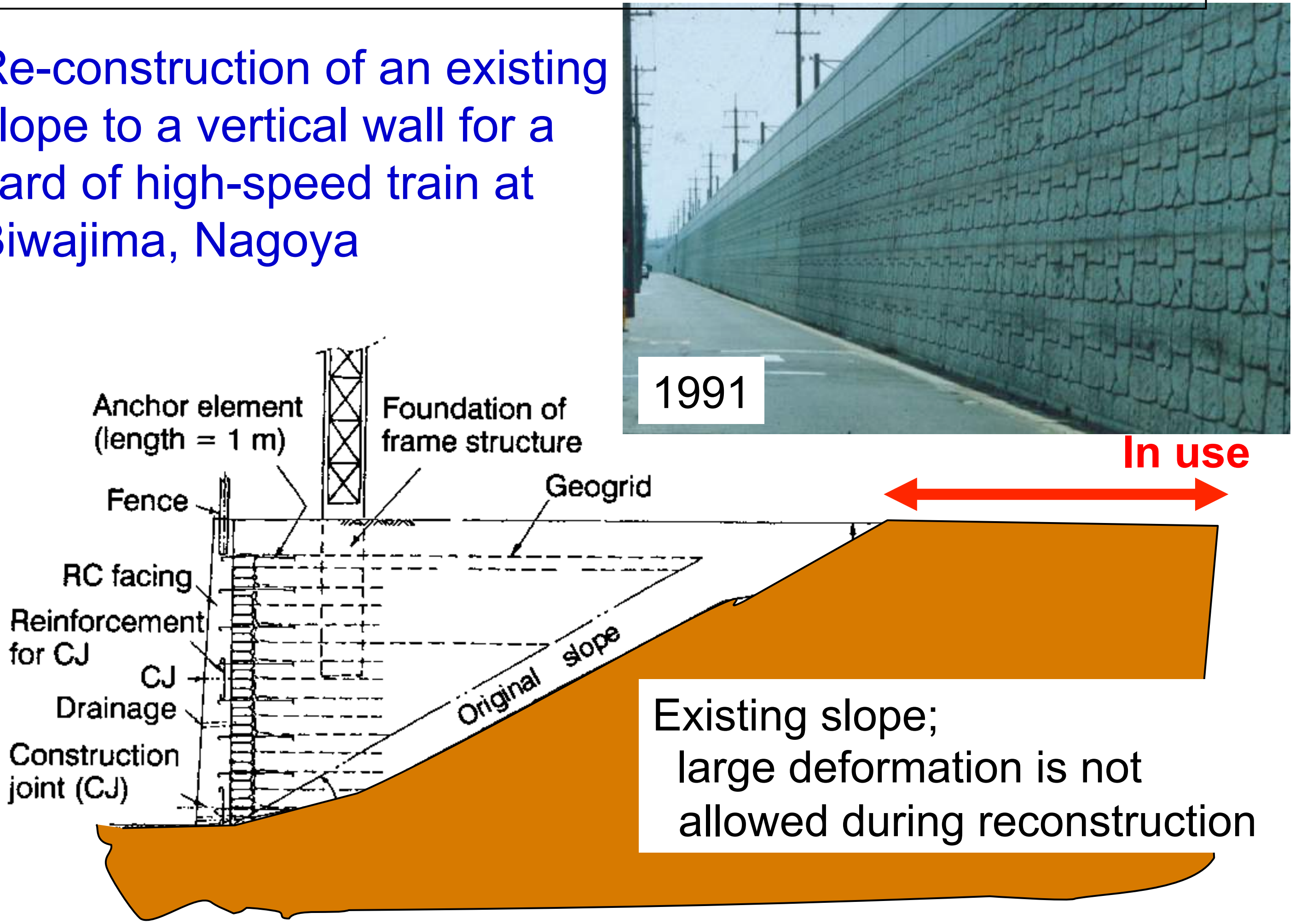
20 years after construction,
6th July, 2014



Staged construction - 5:

- Completed.

Re-construction of an existing slope to a vertical wall for a yard of high-speed train at Biwajima, Nagoya

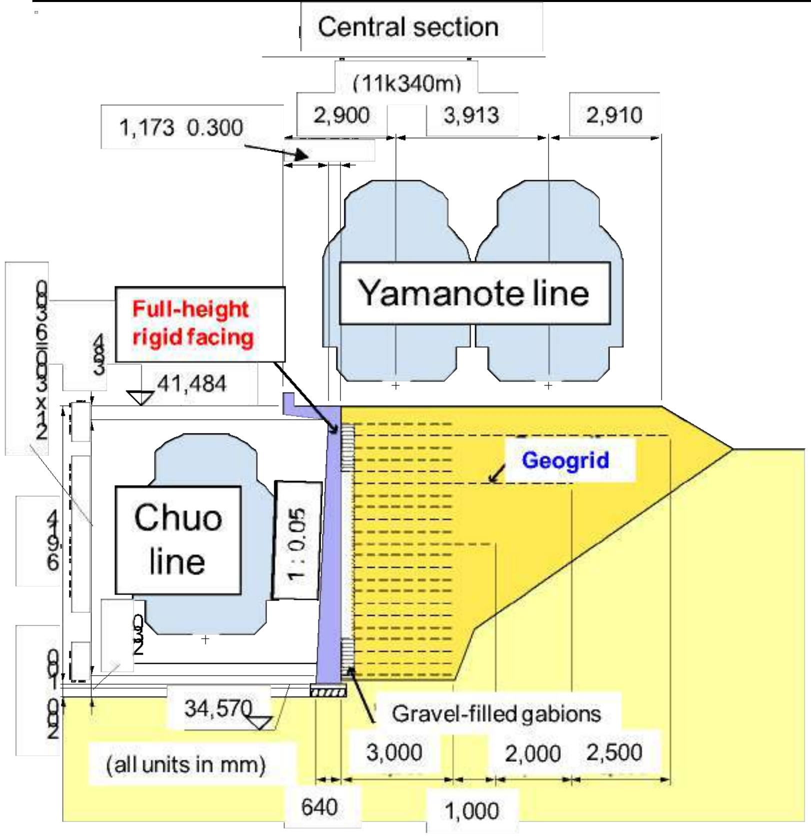


Existing slope;
large deformation is not allowed during reconstruction

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



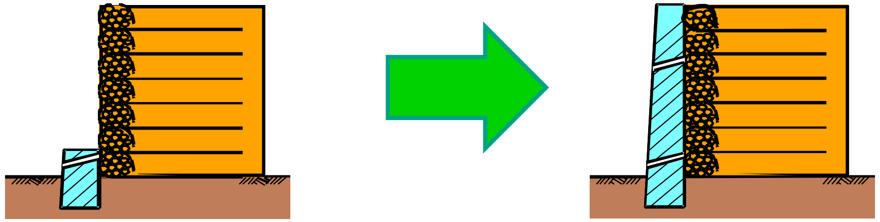
Near Shinjuku Station, Tokyo, constructed during 1995 – 2000

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

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4. Concluding remarks

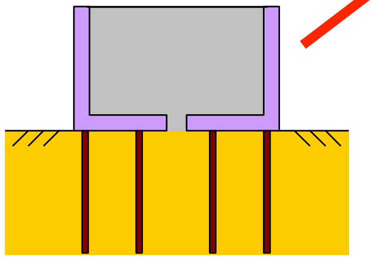
Three generations of elevated structures for HST lines (Shin-kansen) in Japan

Embankment;
54 % of the total length

Gentle slope

and

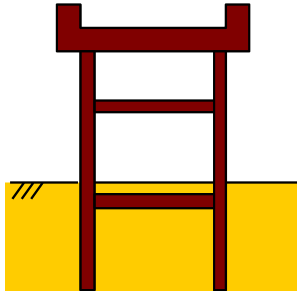
Conventional type RWs



[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

RC viaduct

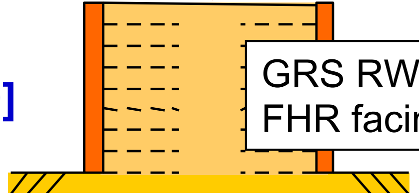


[From 1972, Sanyo. Joetsu & Tohoku]

- High cost
- No use of excavated soil➔

[Since 2000]

Where relevant



GRS RW with FHR facing

(basically no piles)

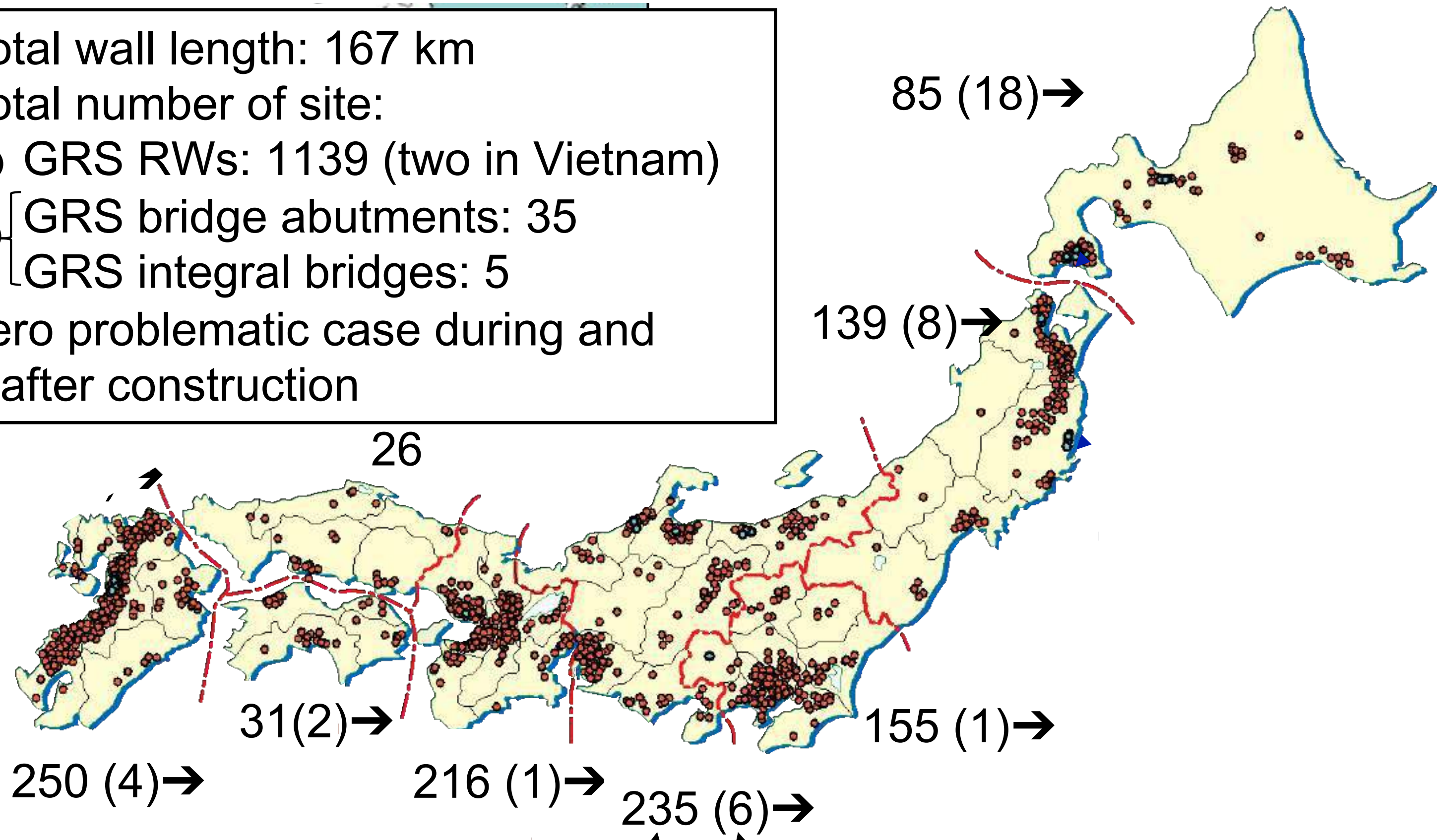
・High stability (rainfalls, earthquakes)
・High cost-effectiveness
in construction & maintenance

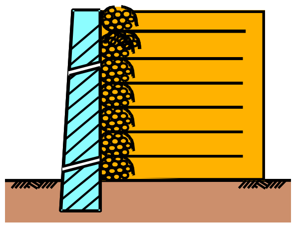
Total wall length: 167 km

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

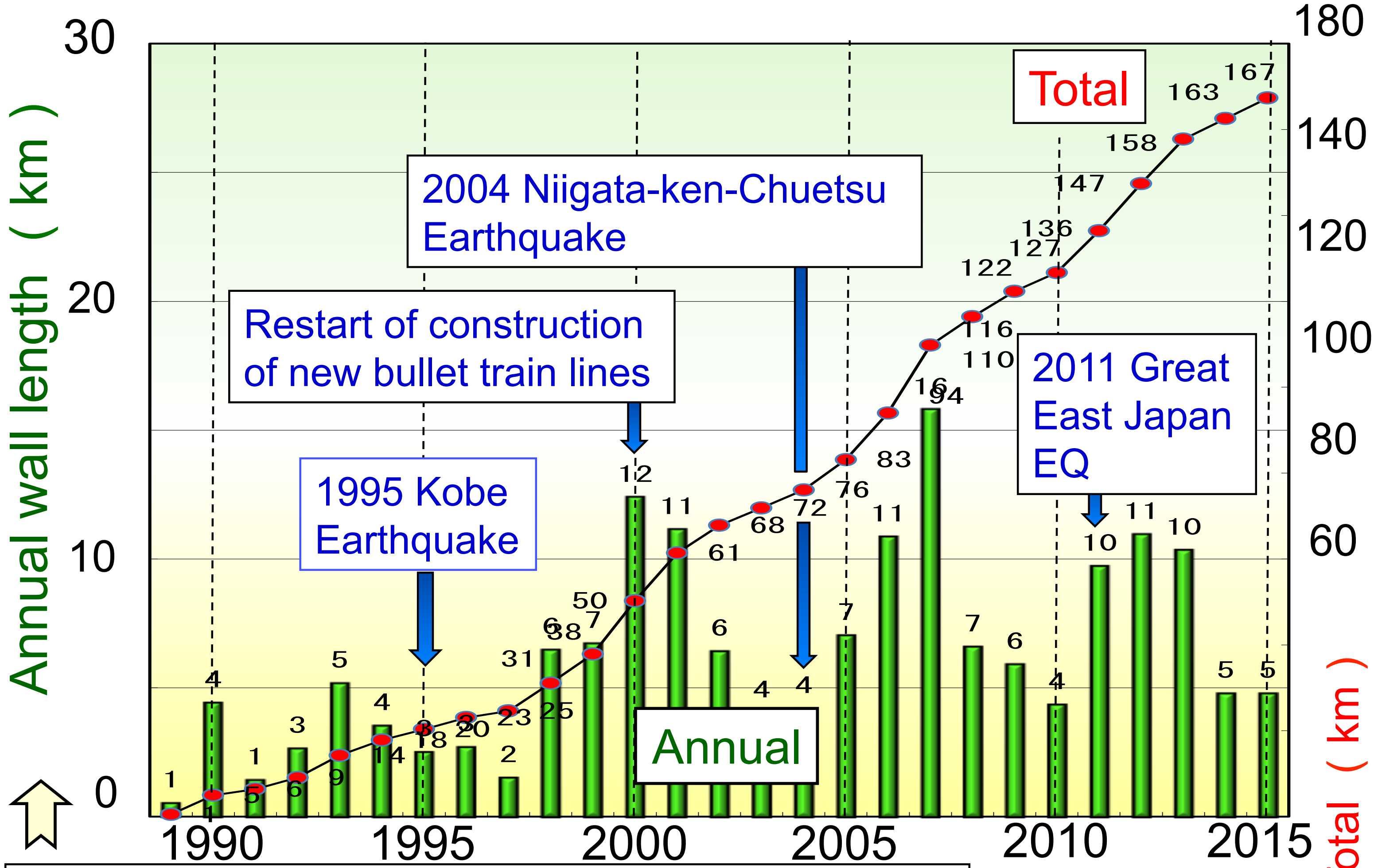




No. of GRS RWs→

(No. of GRS bridge abutments & GRS integral bridges)→

Locations of GRS RWs with a stage-constructed FHR facing as of June 2016



1995 Kobe Earthquake

Restart of construction of new bullet train lines

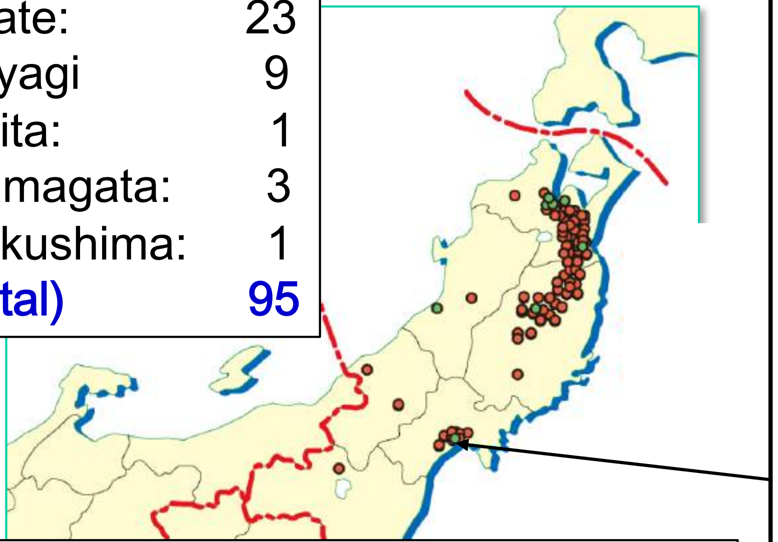
2004 Niigata-ken-Chuetsu Earthquake

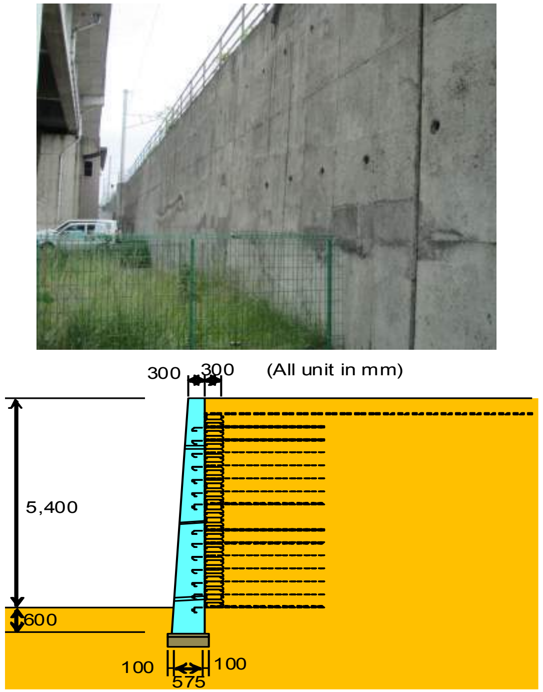
2011 Great East Japan EQ

From 1982: research at the University of Tokyo & Railway Technical Research Institute

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

Aomori:	58
Iwate:	23
Miyagi	9
Akita:	1
Yamagata:	3
Fukushima:	1
(total)	95



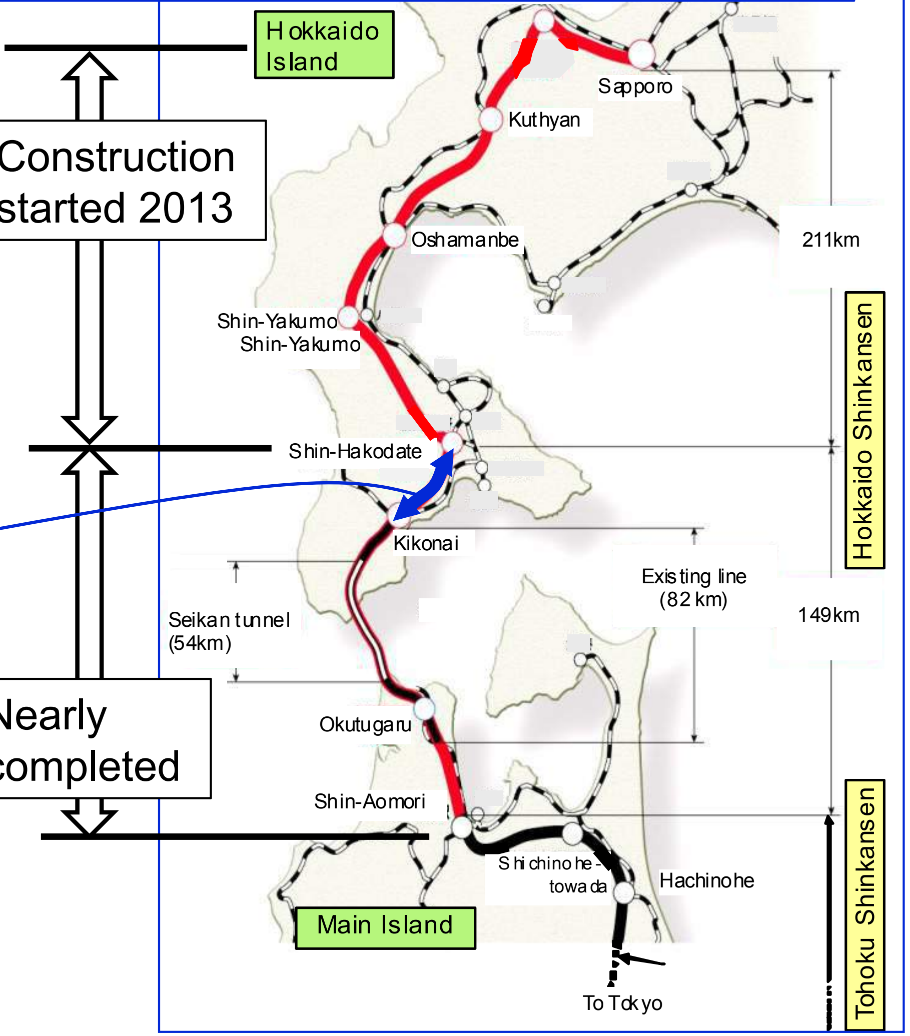


Adjacent to Natori River,
Sendai City
Completed 1994
Wall length= 400 m

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

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

Hokkaido High-Speed Train Line (Shin-kansen)



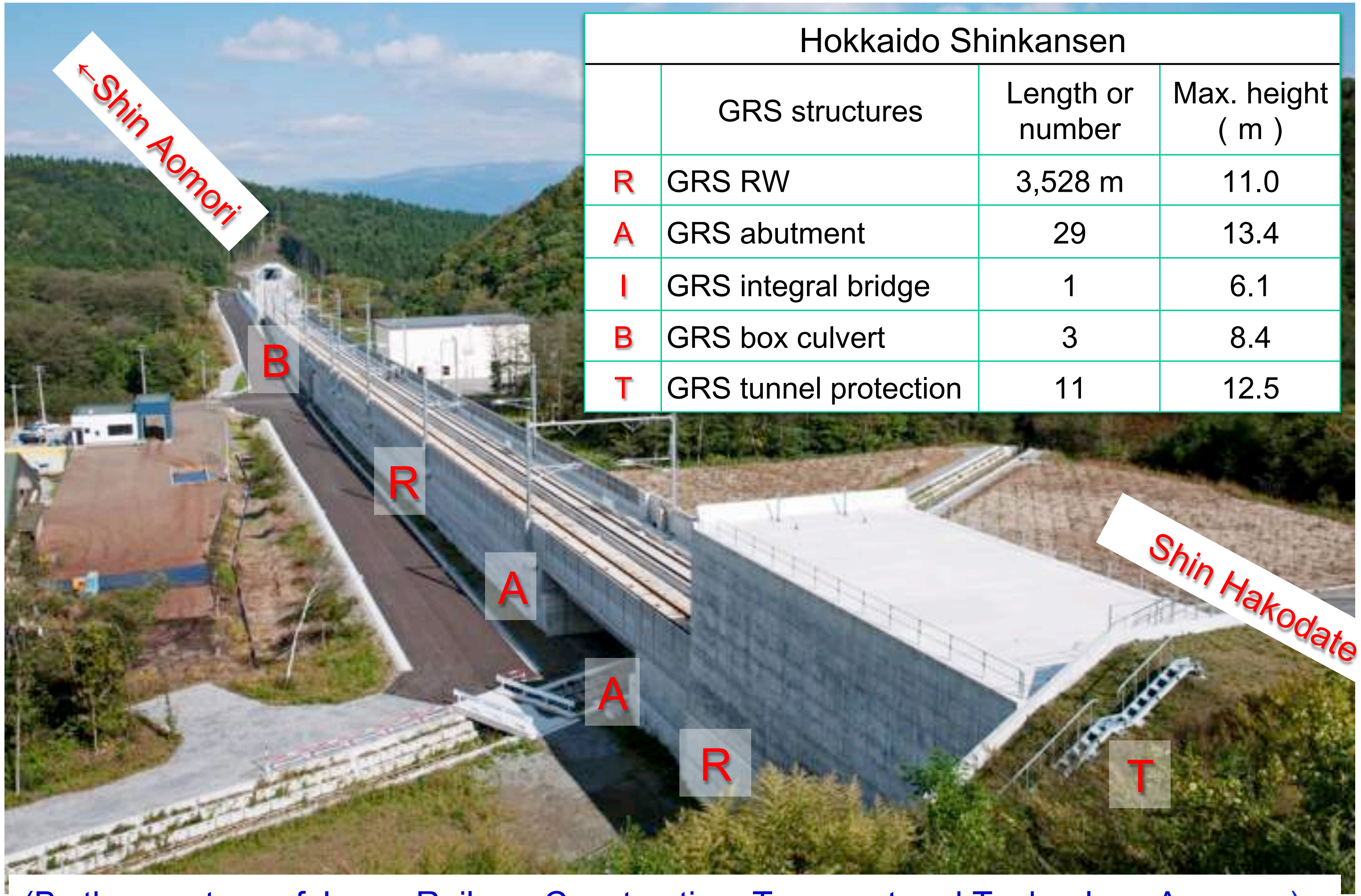
Construction started 2013

Nearly completed

Opened in the beginning of 2016

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

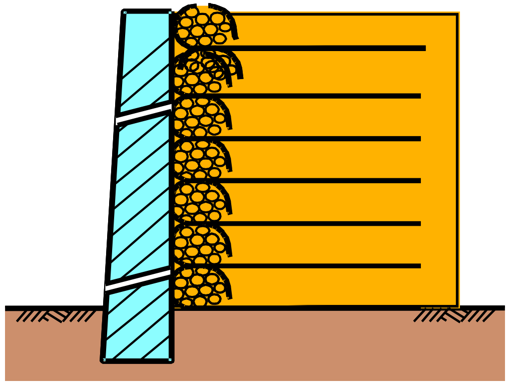
Various GRS structures at Montaro

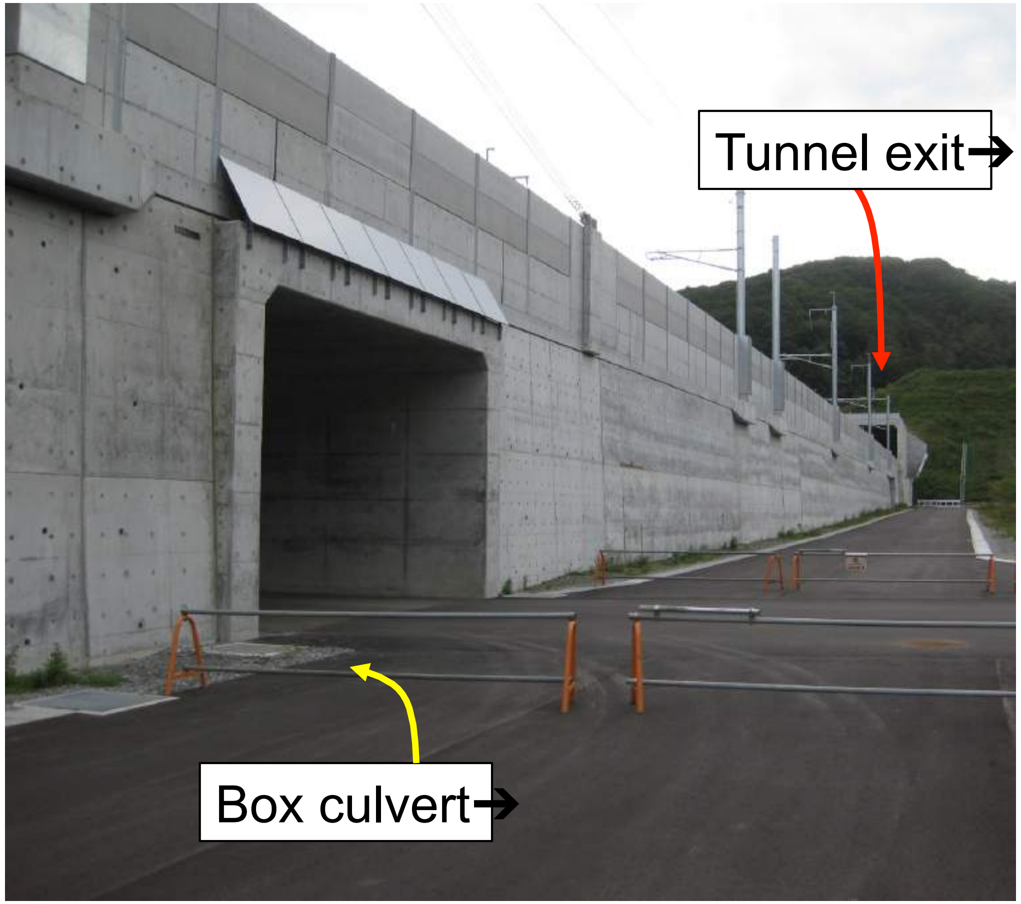


Hokkaido Shinkansen			
	GRS structures	Length or number	Max. height (m)
R	GRS RW	3,528 m	11.0
A	GRS abutment	29	13.4
I	GRS integral bridge	1	6.1
B	GRS box culvert	3	8.4
T	GRS tunnel protection	11	12.5

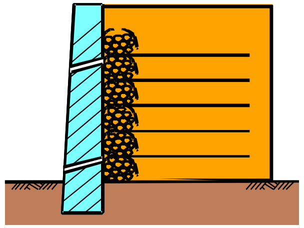
(By the courtesy of Japan Railway Construction, Transport and Technology Agency)

Typical GRS RW for Hokkaido HST Line: immediately after RC facing was constructed by casting-in-place concrete

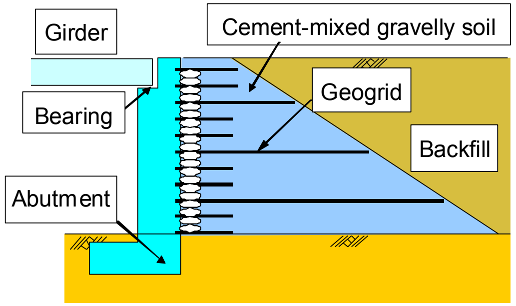




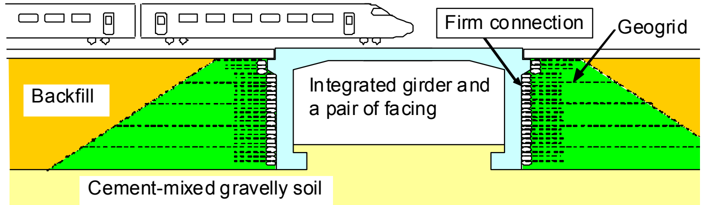
For Hokkaido HST line;



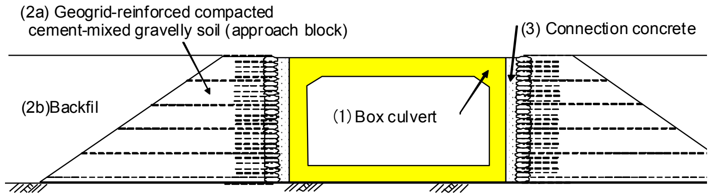
1) GRS retaining walls having full-height rigid facing for a length of 3.5 km, totally in place of conventional type cantilever RWs



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



3) A GRS integral bridge

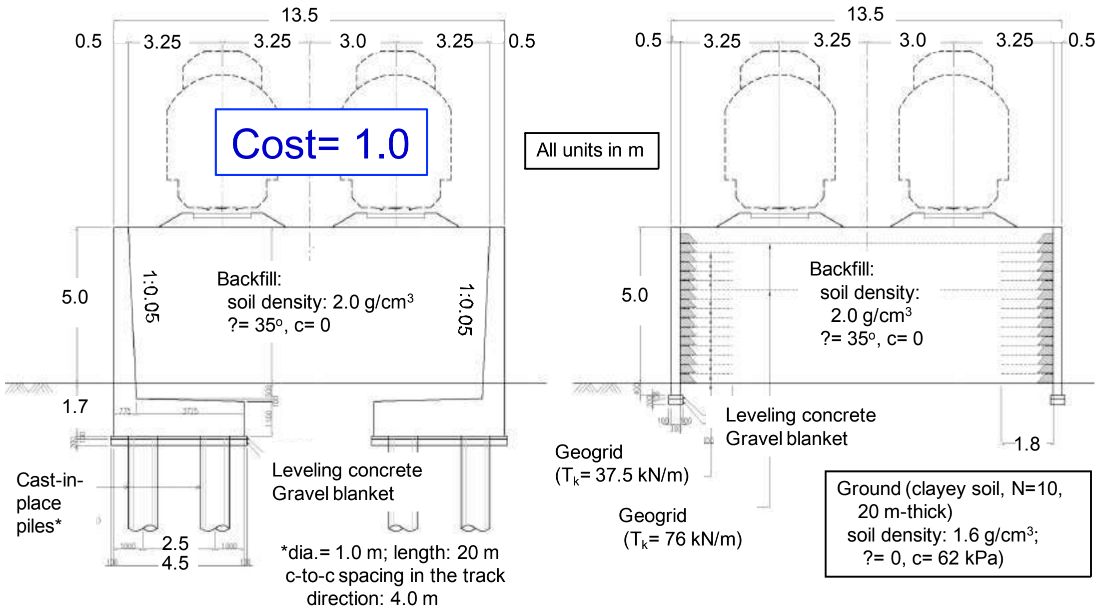


4) Three GRS box culvert structures integrated to GRS RWs

5) Eleven GRS protection structures at the tunnel entrance

Cost Ratio: GRS RW versus conventional type RW

	Construction	Maintenance	Total
20 m-thick relatively soft ground: - piles for conventional type RWs - no piles for GRS RWs (the case shown in the figure)	0.32	0.5	0.33
Relatively stiff ground: - no piles for conventional type RWs & GRS RWs	0.81	0.51	0.77



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

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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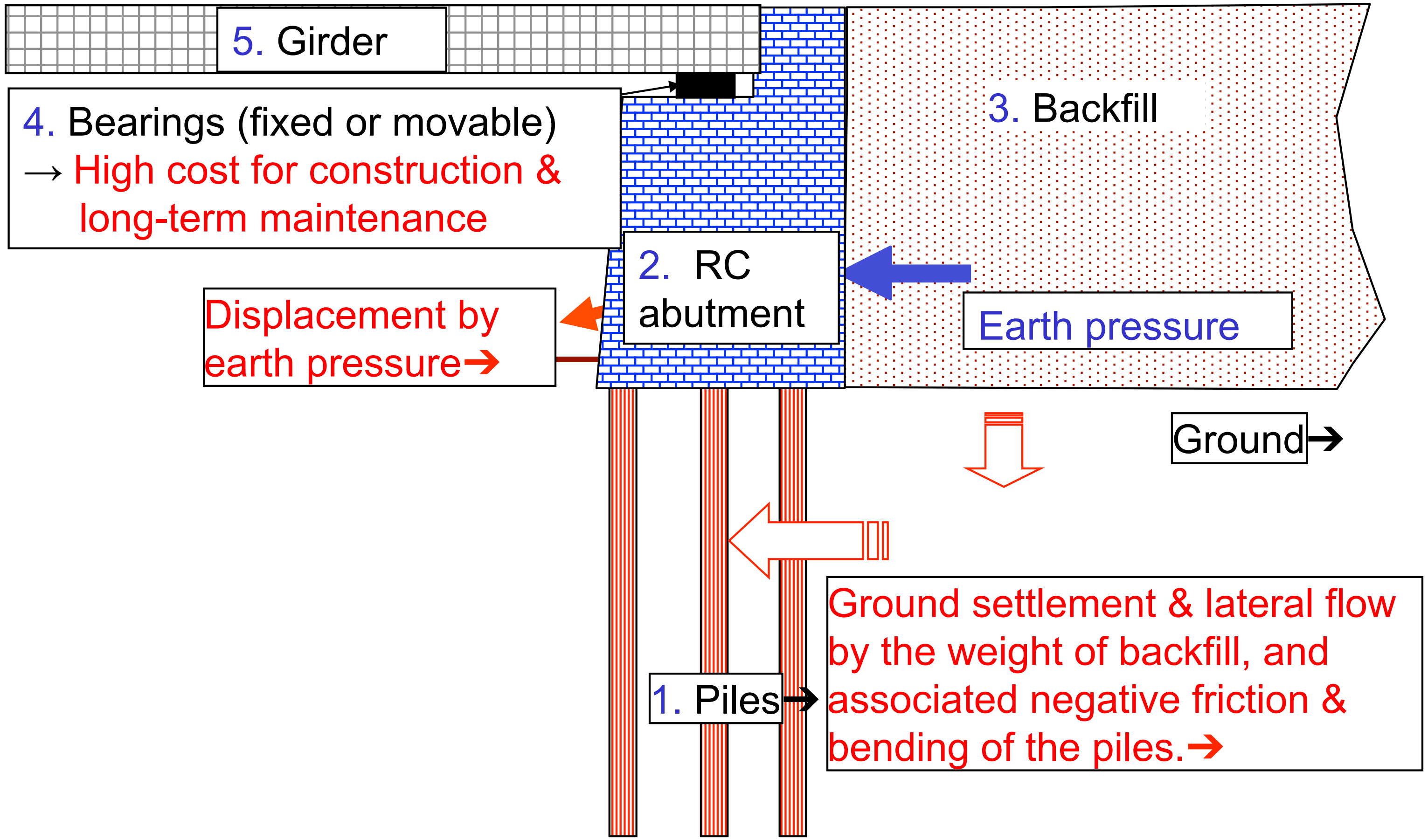
Technical problems with conventional type bridges➔



5. Girder

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

3. Backfill

2. RC abutment

Earth pressure

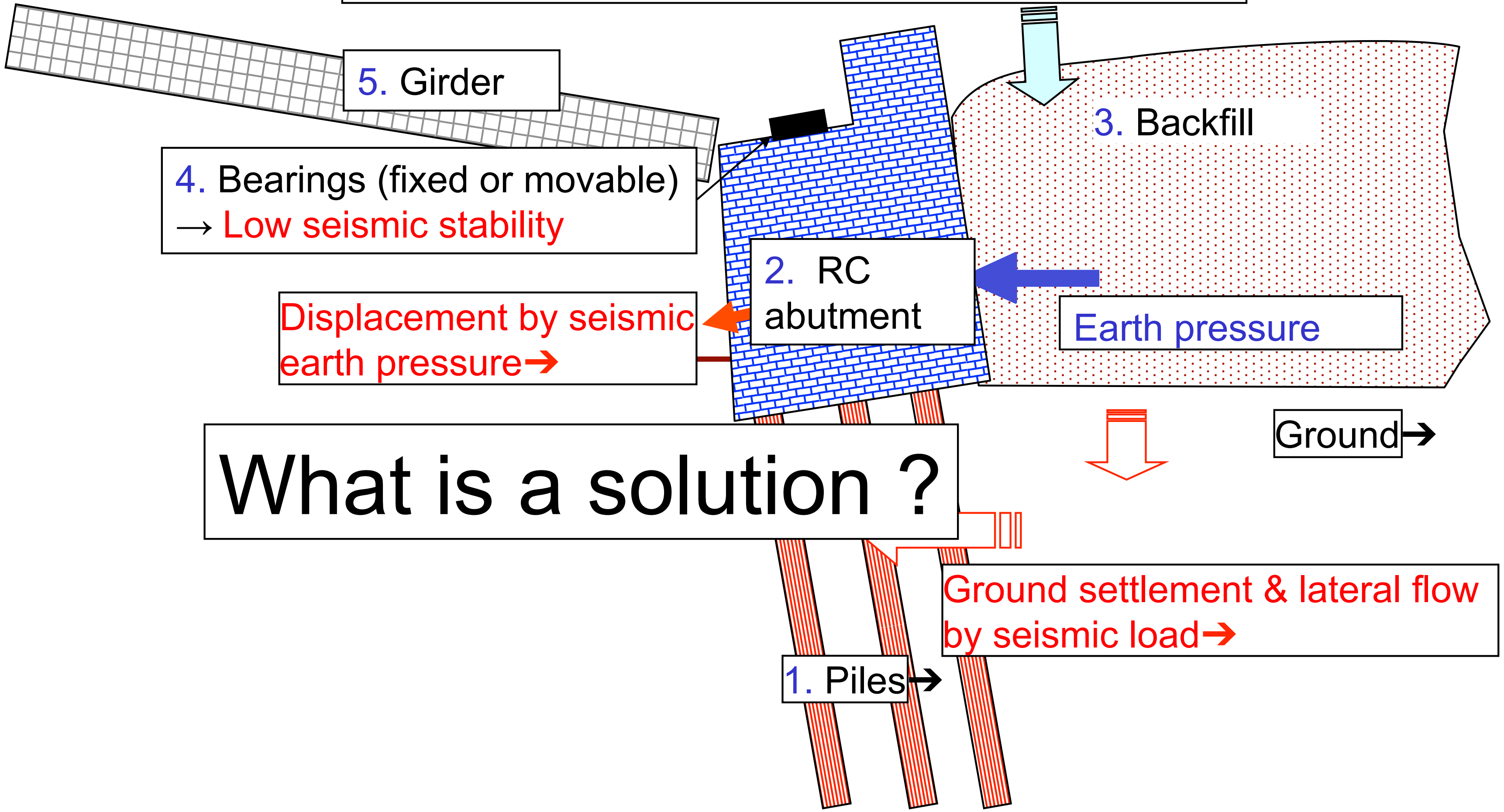
Displacement by earth pressure➔

Ground➔

1. Piles➔

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

Technical problems with conventional type bridge➔



Settlement by self-weight, traffic load & seismic load

5. Girder

3. Backfill

4. Bearings (fixed or movable)
→ Low seismic stability

2. RC abutment

Earth pressure

Displacement by seismic earth pressure➔

Ground➔

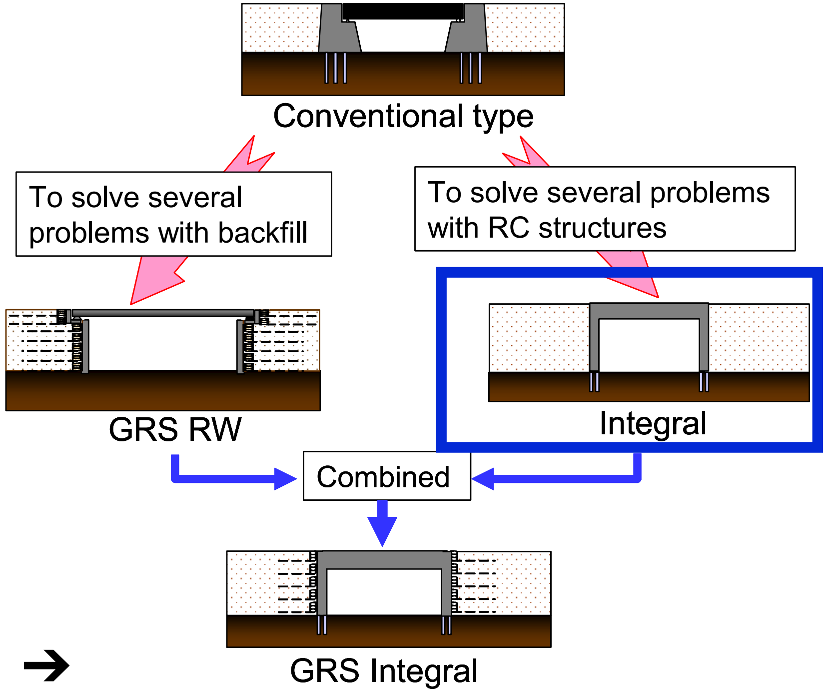
What is a solution ?

Ground settlement & lateral flow by seismic load➔

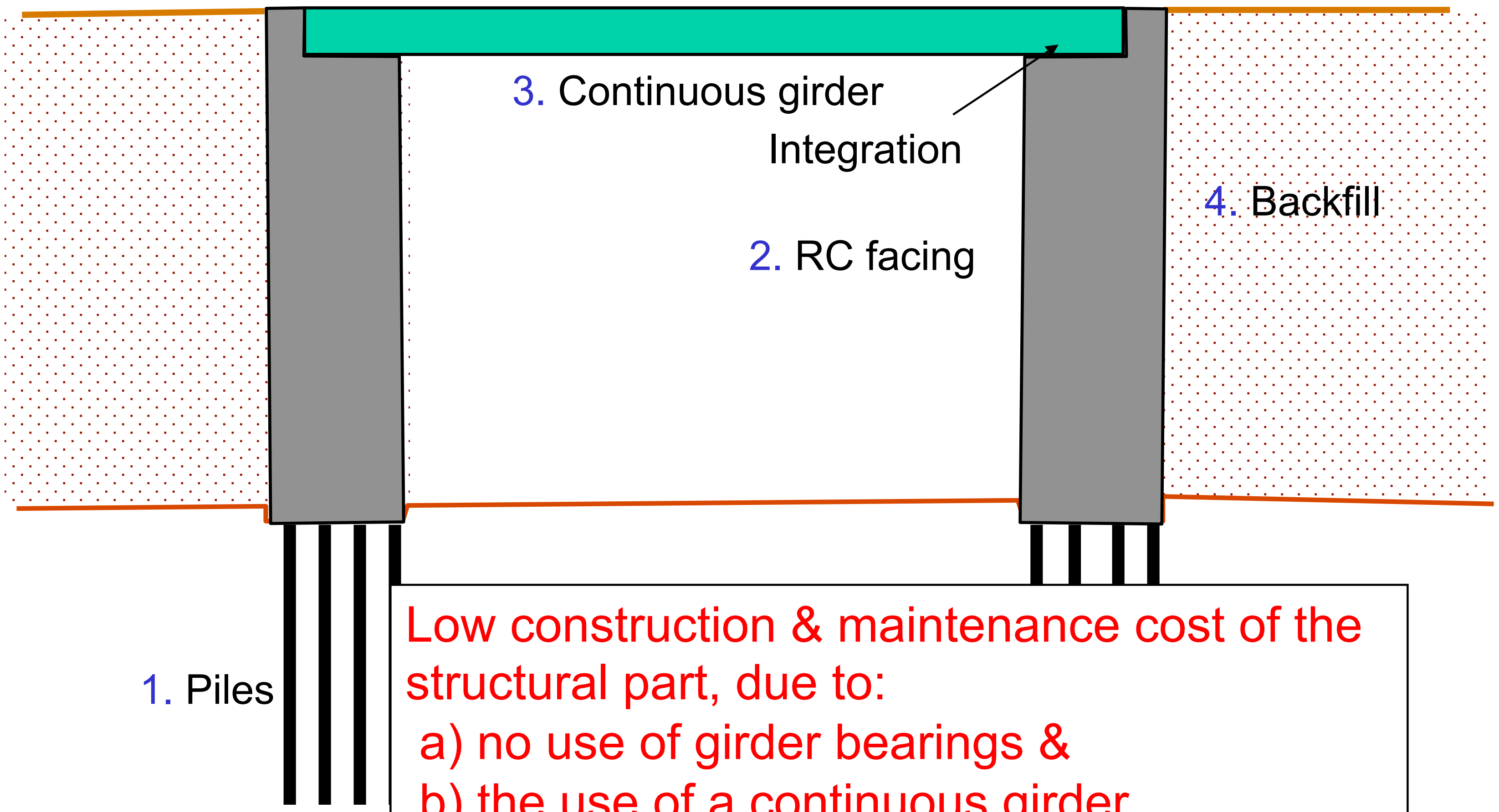
1. Piles➔

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 ➔



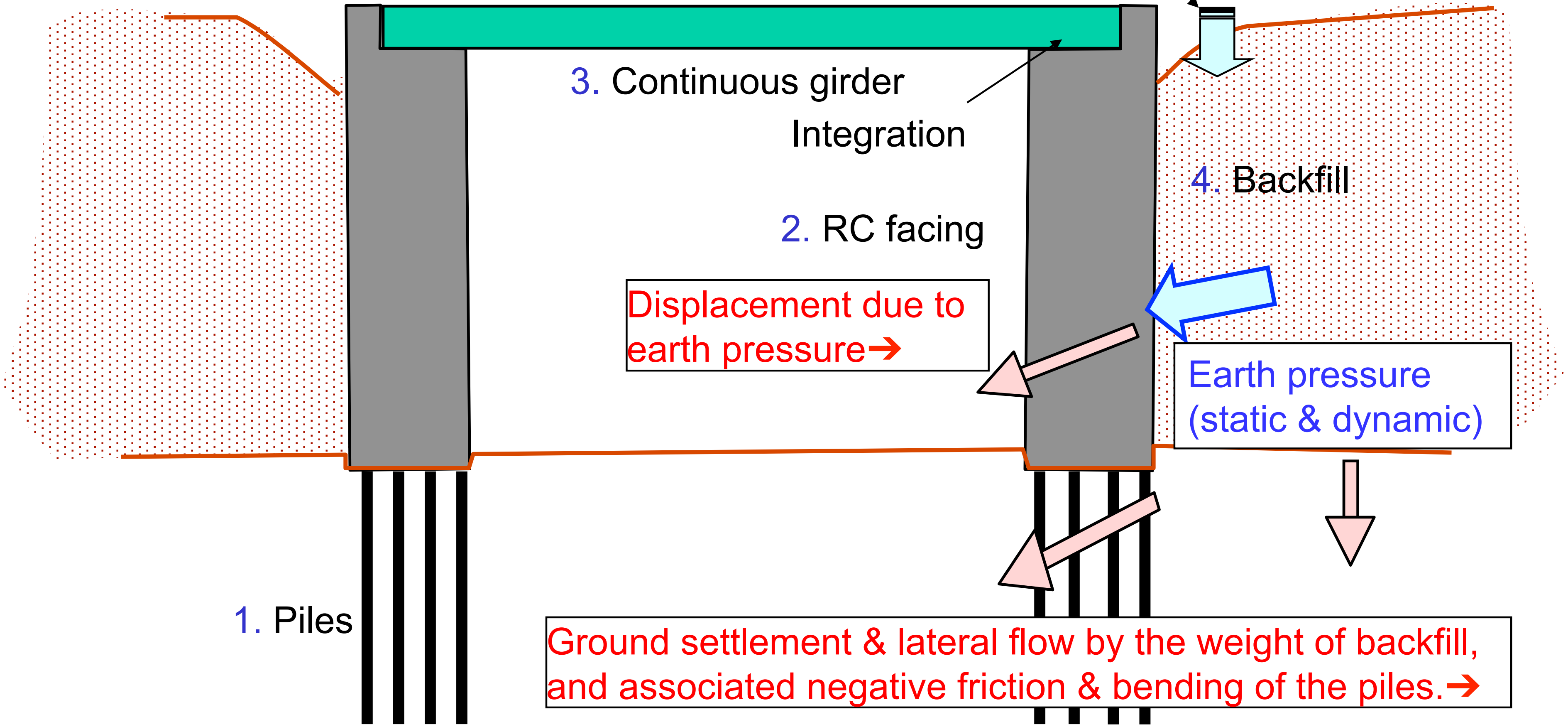
Integral bridge



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

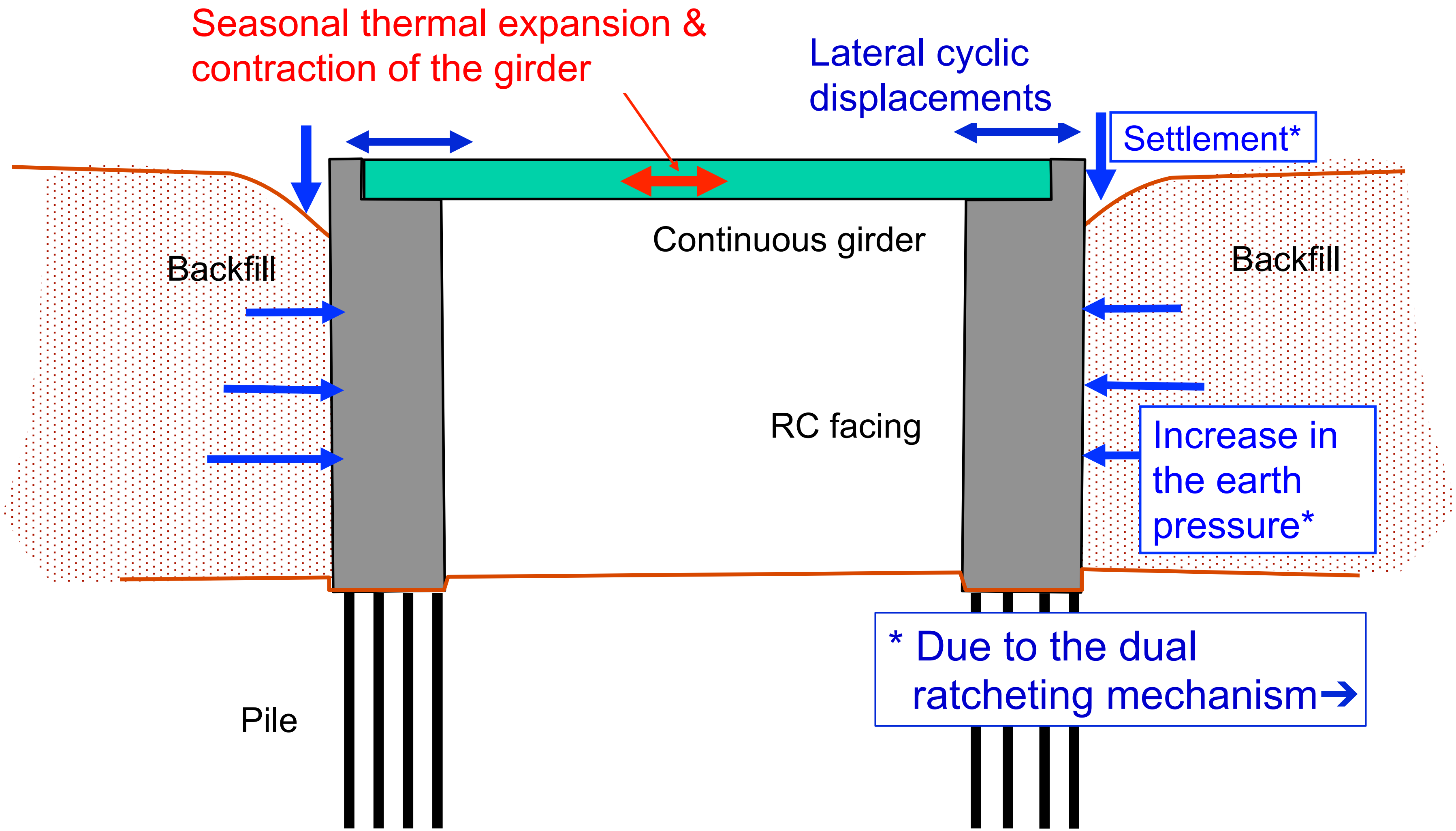


Displacement due to earth pressure→

Earth pressure (static & dynamic)

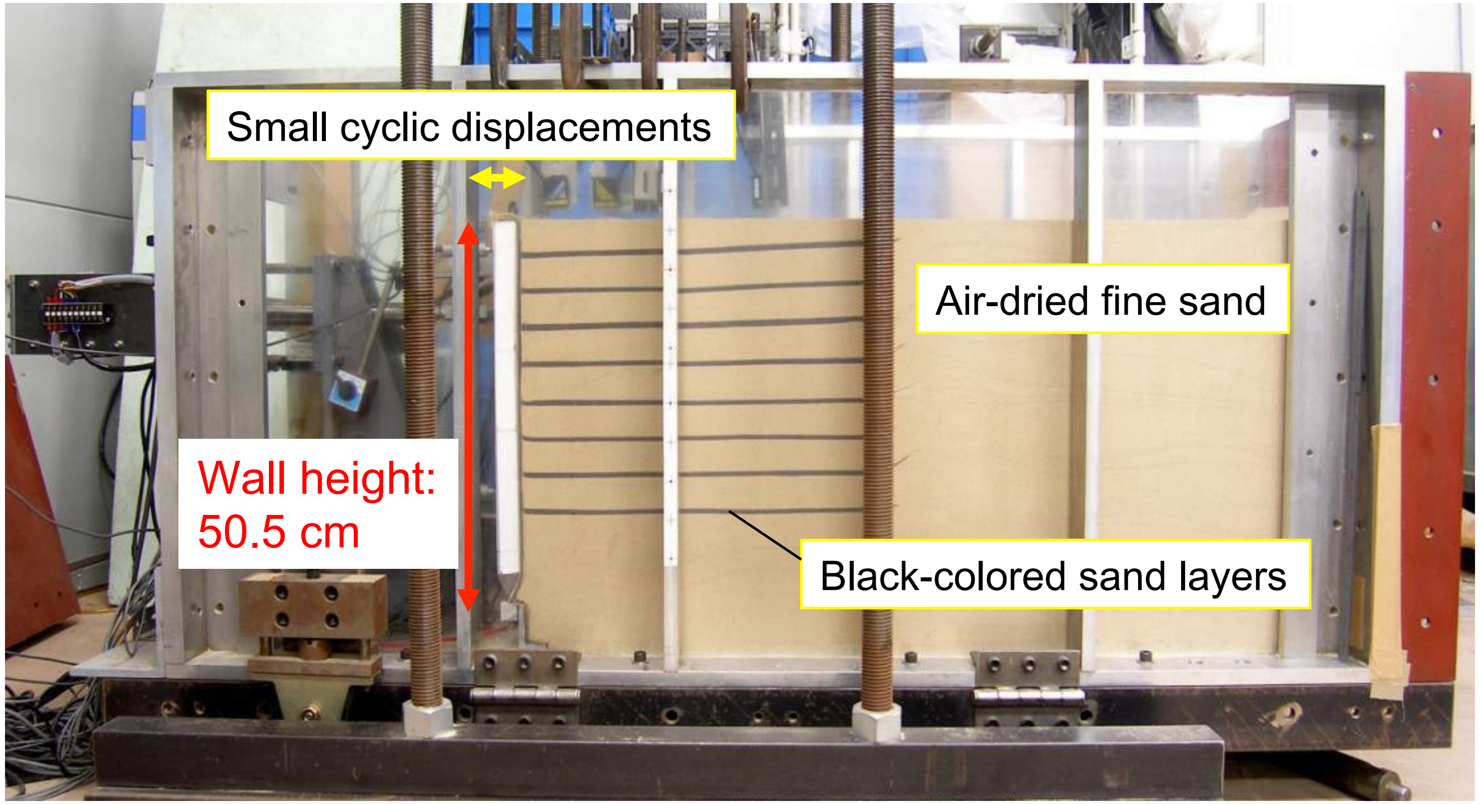
Ground settlement & lateral flow by the weight of backfill, and associated negative friction & bending of the piles.→

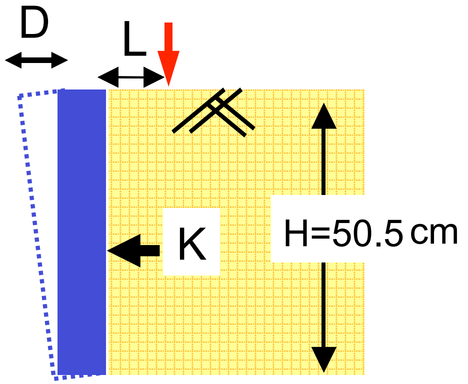
New problems with integral bridges !

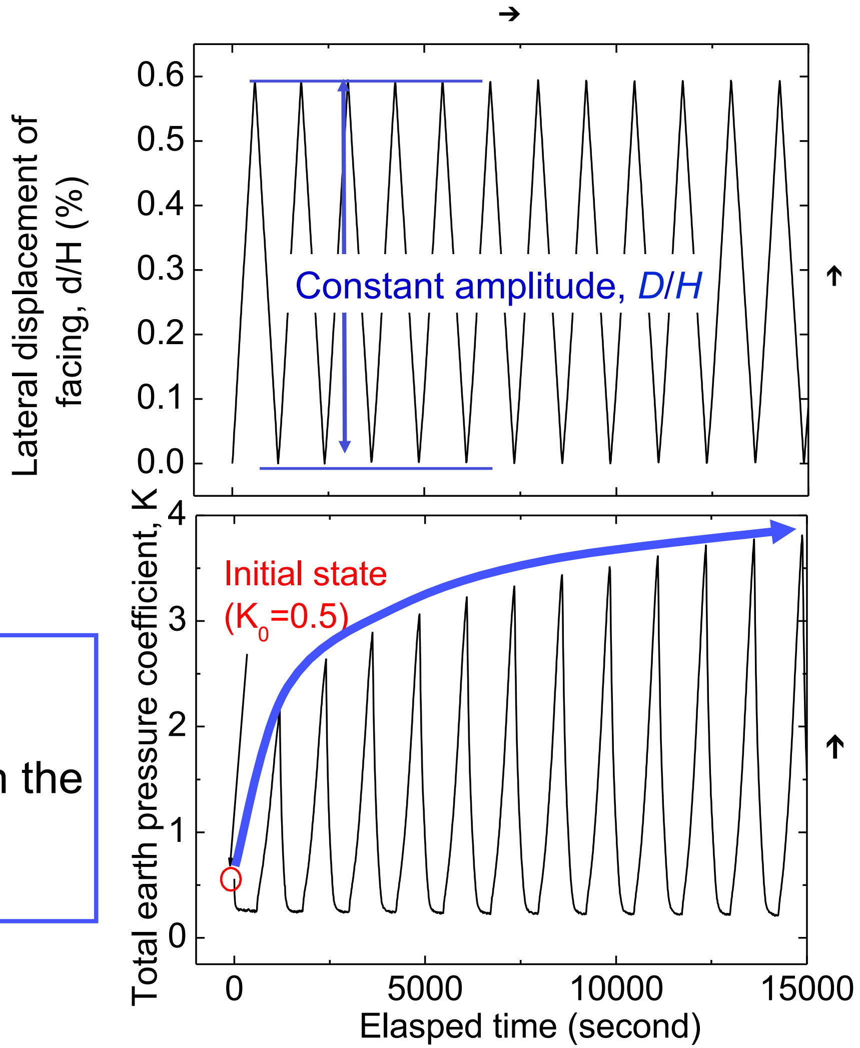


* Due to the dual ratcheting mechanism→

Static lateral cyclic loading tests under plane strain conditions in 1g (considered model scale: 1/10)

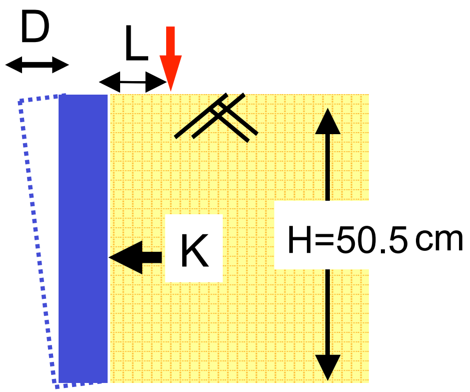


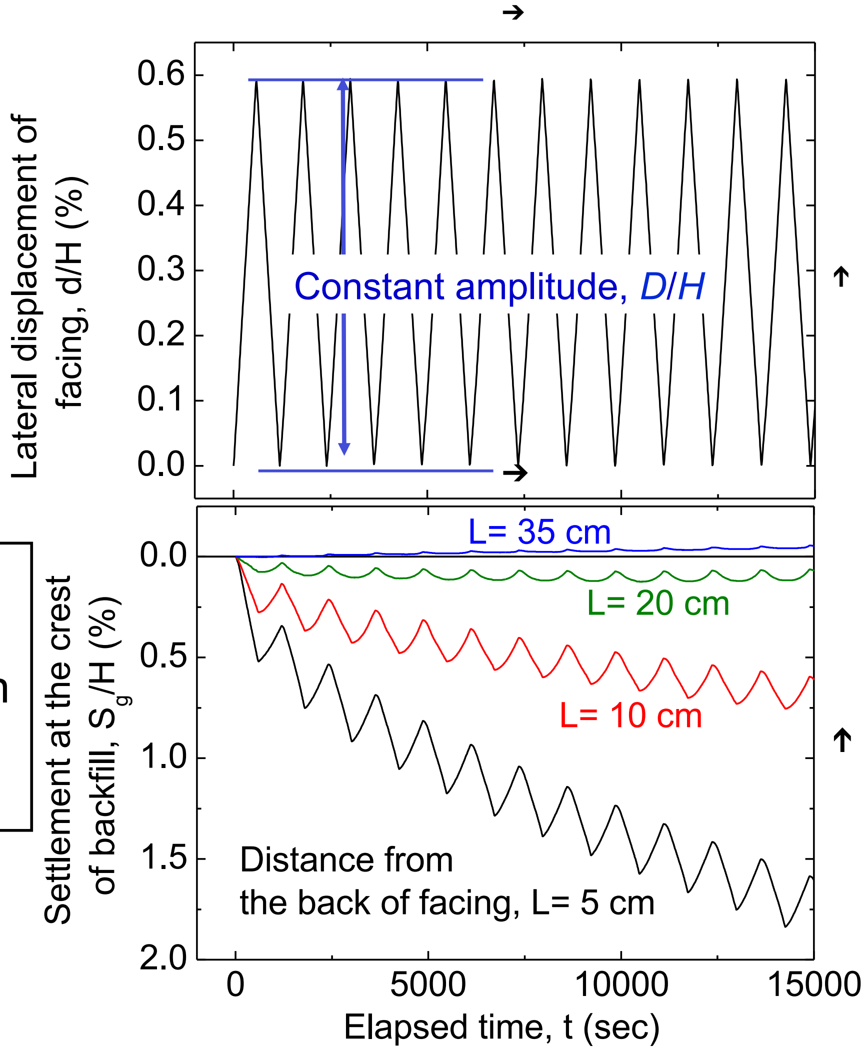




Unreinforced backfill:

A significant increase in the passive earth pressure with cyclic loading



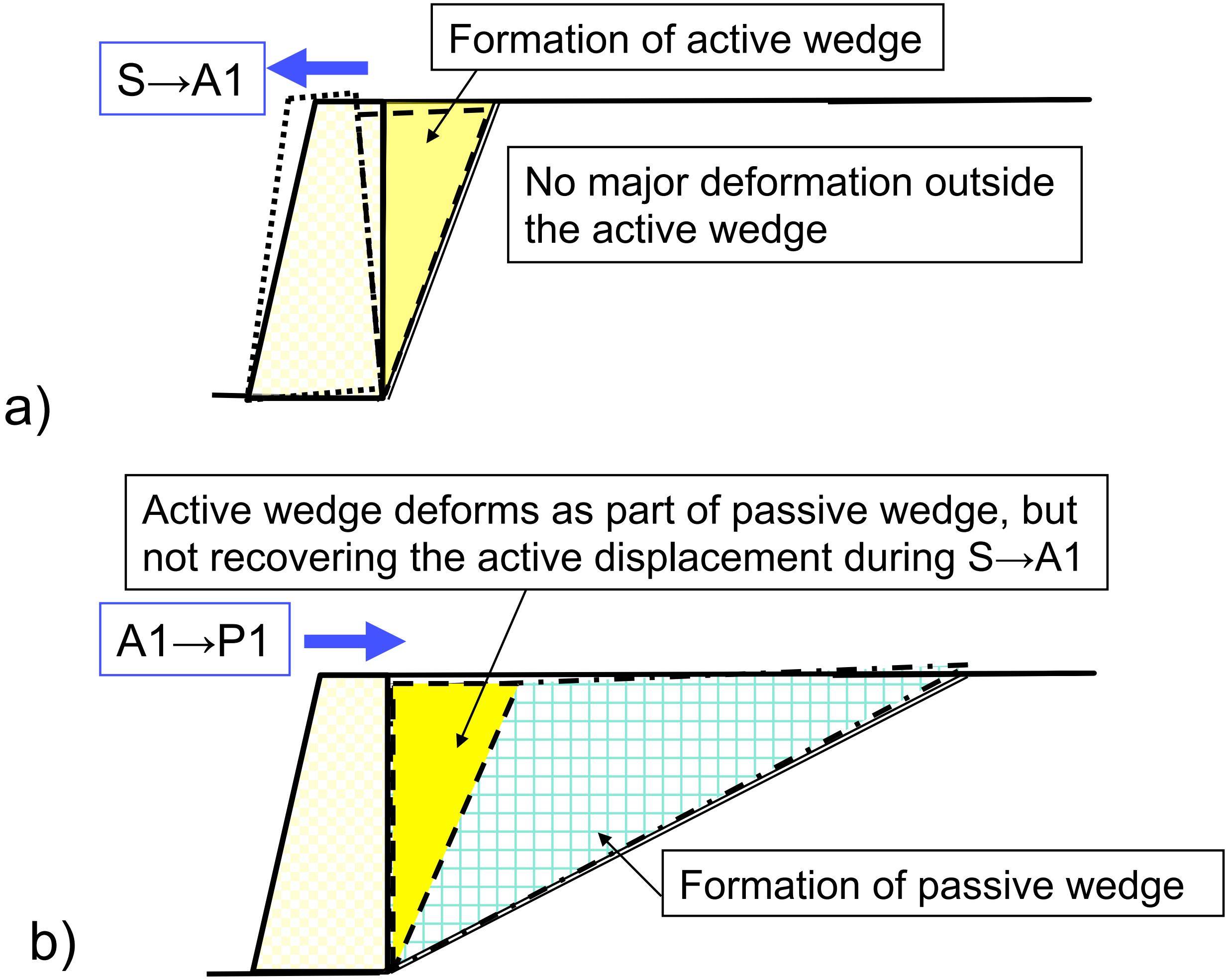


Unreinforced backfill:

Significant settlement in the backfill with cyclic loading

Why ?

Dual ratchet mechanism



S→A1

Formation of active wedge

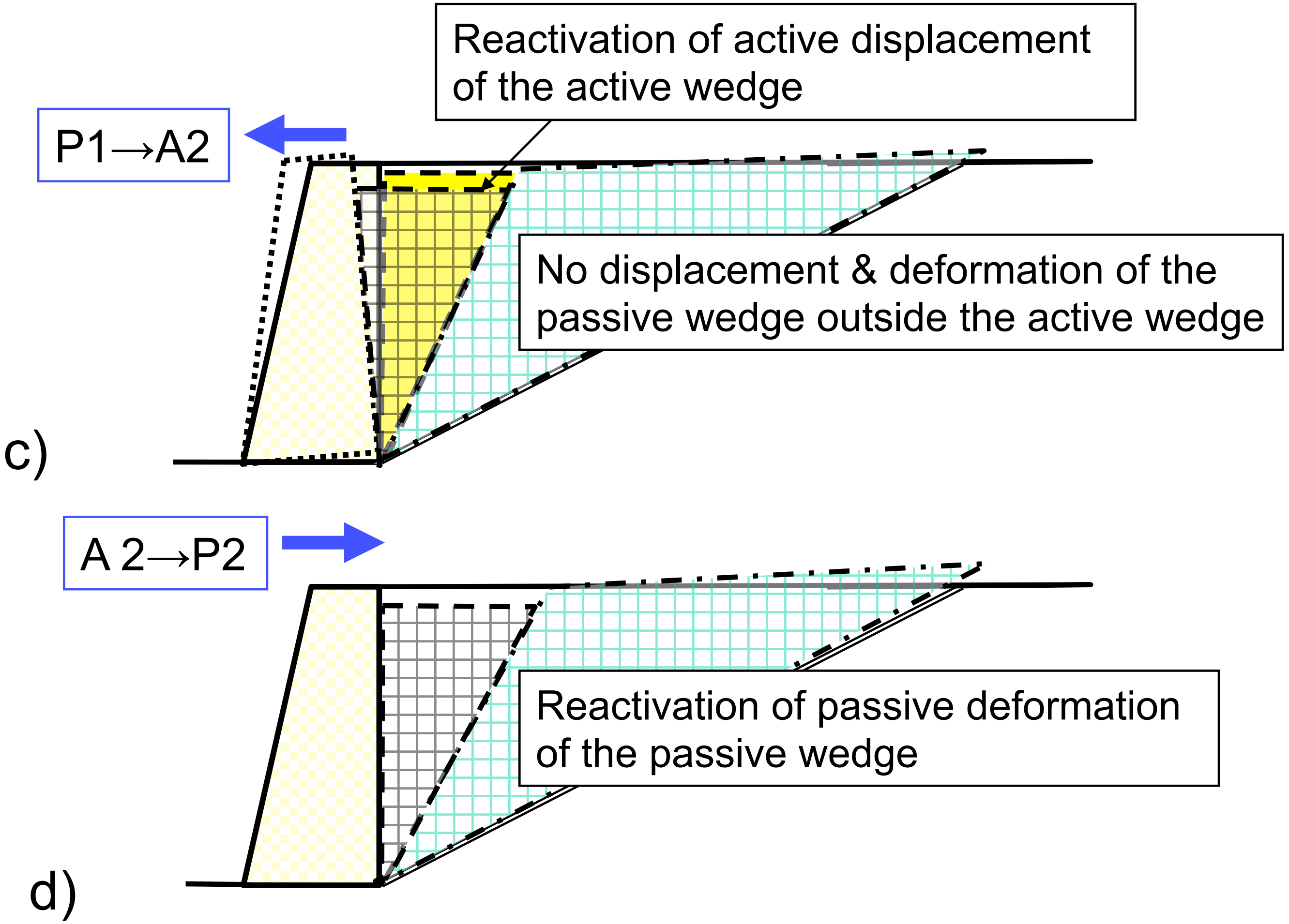
No major deformation outside the active wedge

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

A1→P1

Formation of passive wedge

Dual ratchet mechanism



Reactivation of active displacement of the active wedge

P1→A2

No displacement & deformation of the passive wedge outside the active wedge

c)

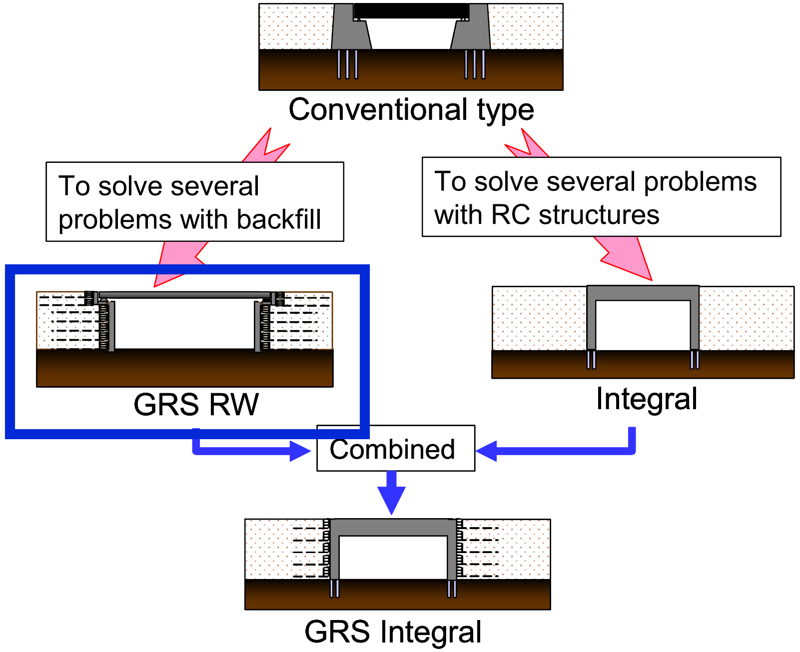
A 2→P2

Reactivation of passive deformation of the passive wedge

d)

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

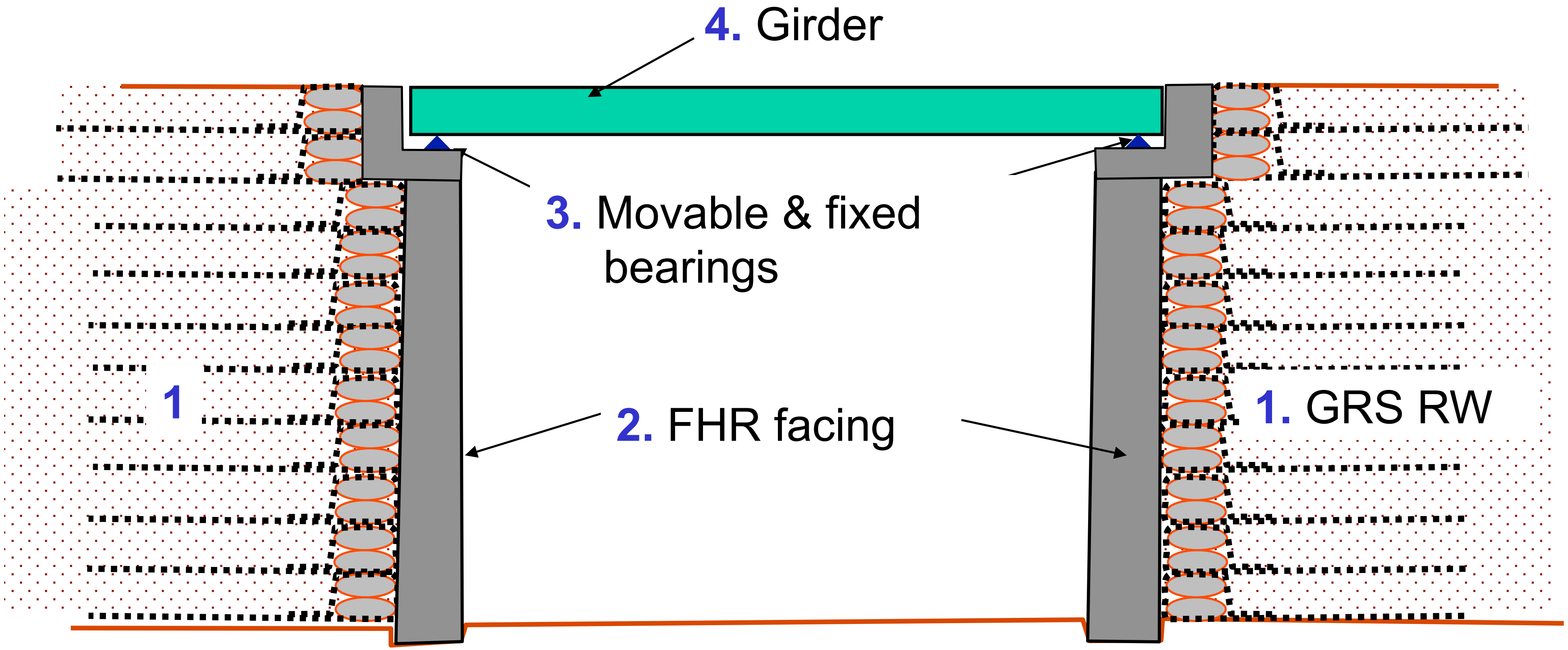
GRS RW

Integral

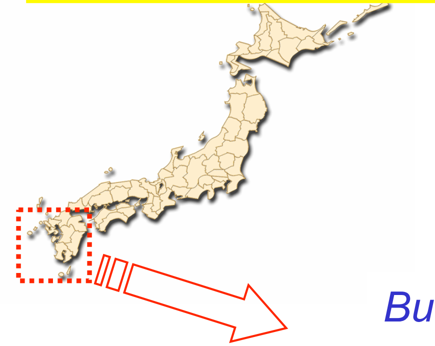
Combined

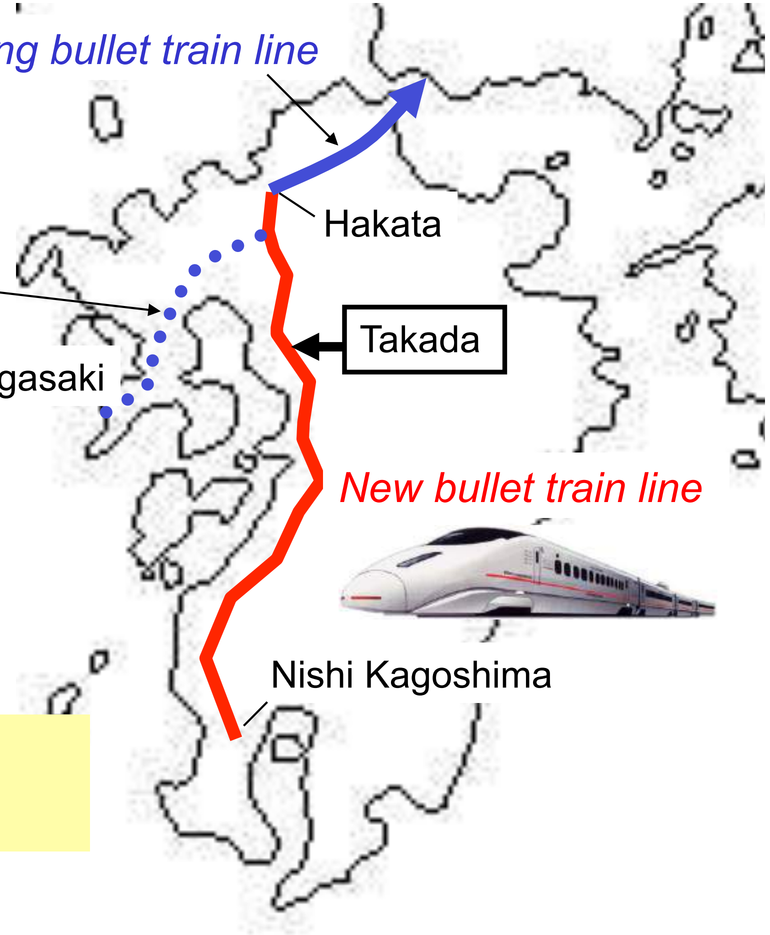
GRS Integral

A better solution: GRS bridge abutment, placing the girder on the top of the facing via bearings



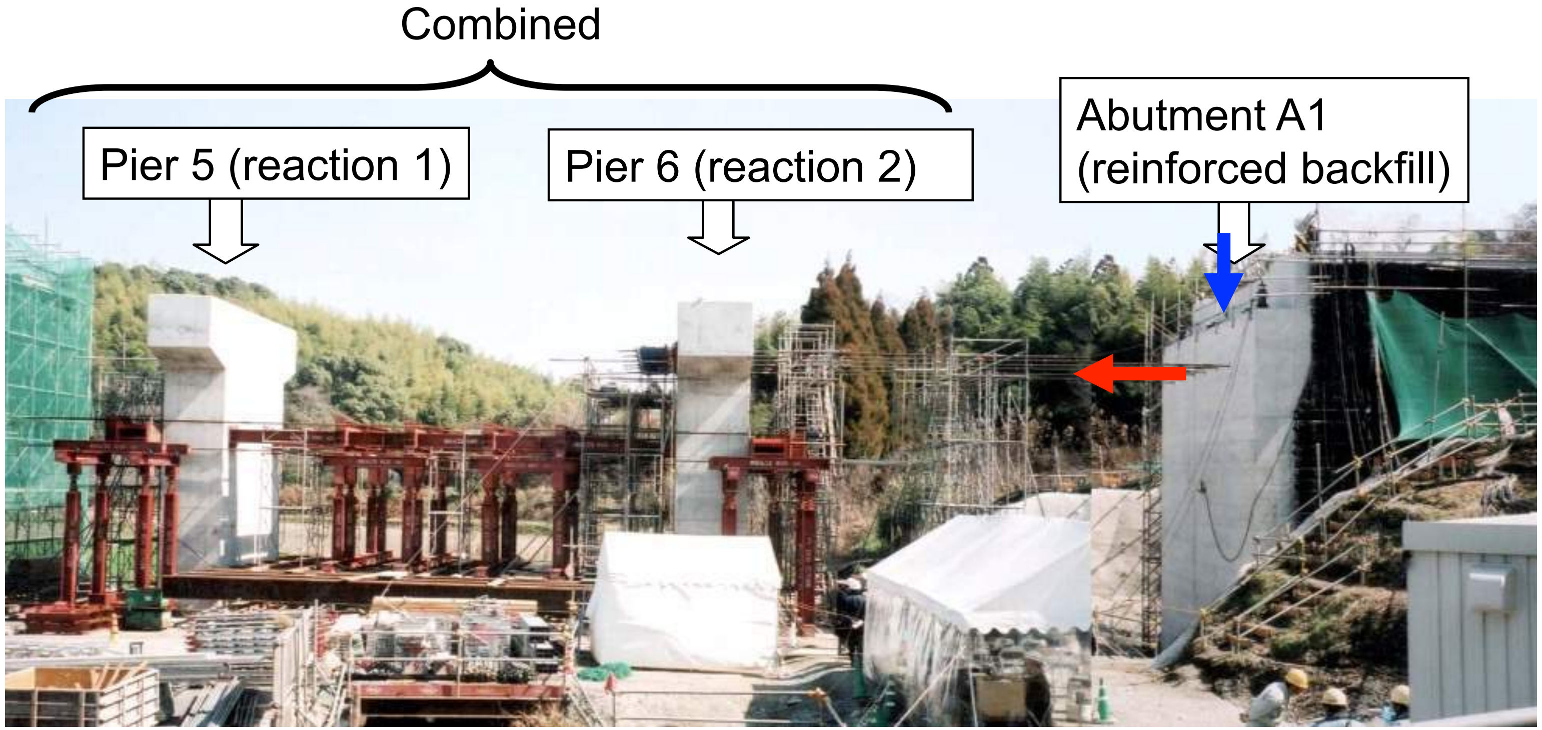
A New High-Speed Train Line in Kyushu Island





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

Oct. 2011

Why not removing the bearings ?

Bearing

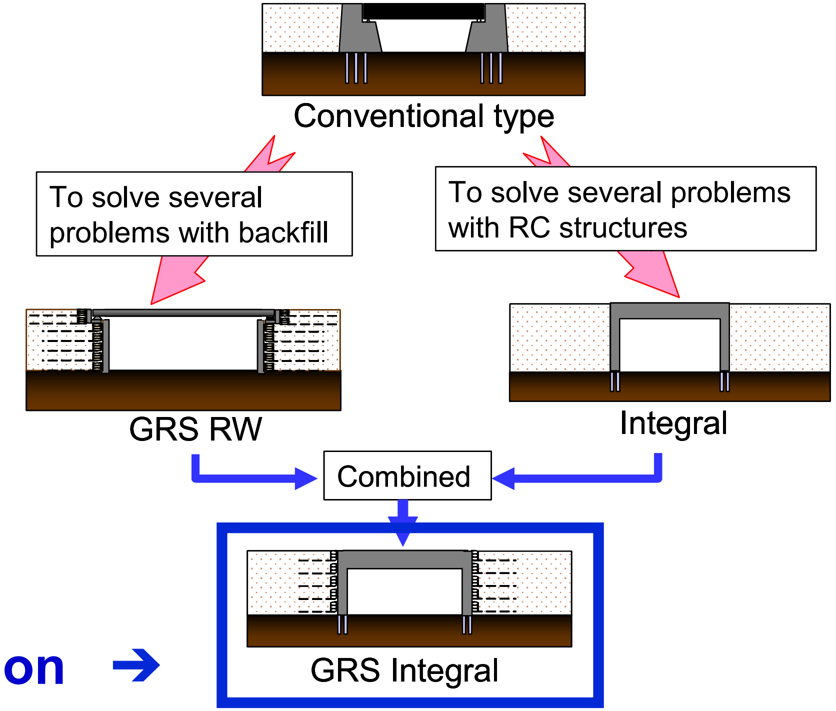
Aug. 2012

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

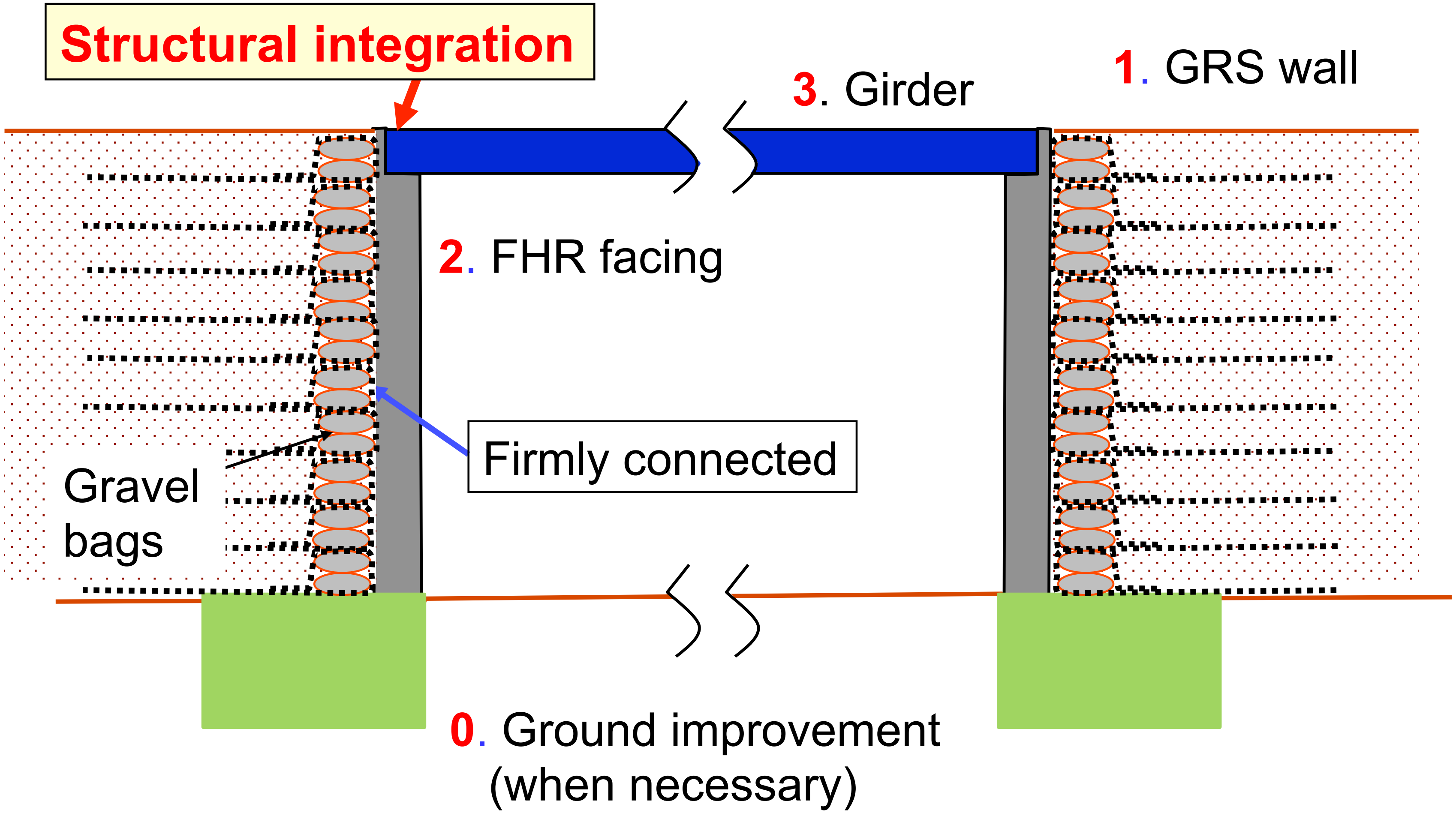
- Yet, still problems by using bearings (i.e., high cost for construction/maintenance & low seismic stability)

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



Structural integration

3. Girder

1. GRS wall

2. FHR facing

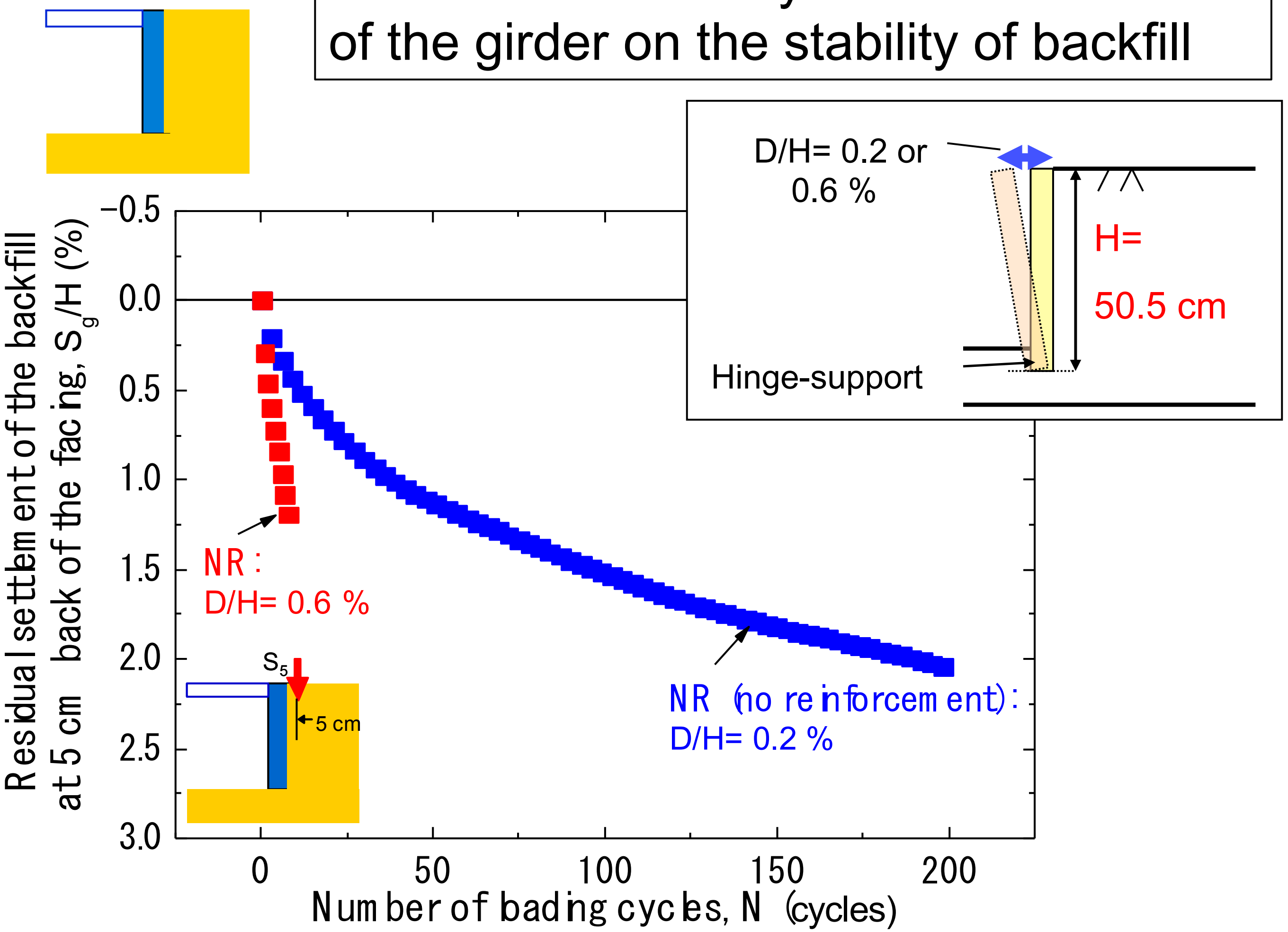
Firmly connected

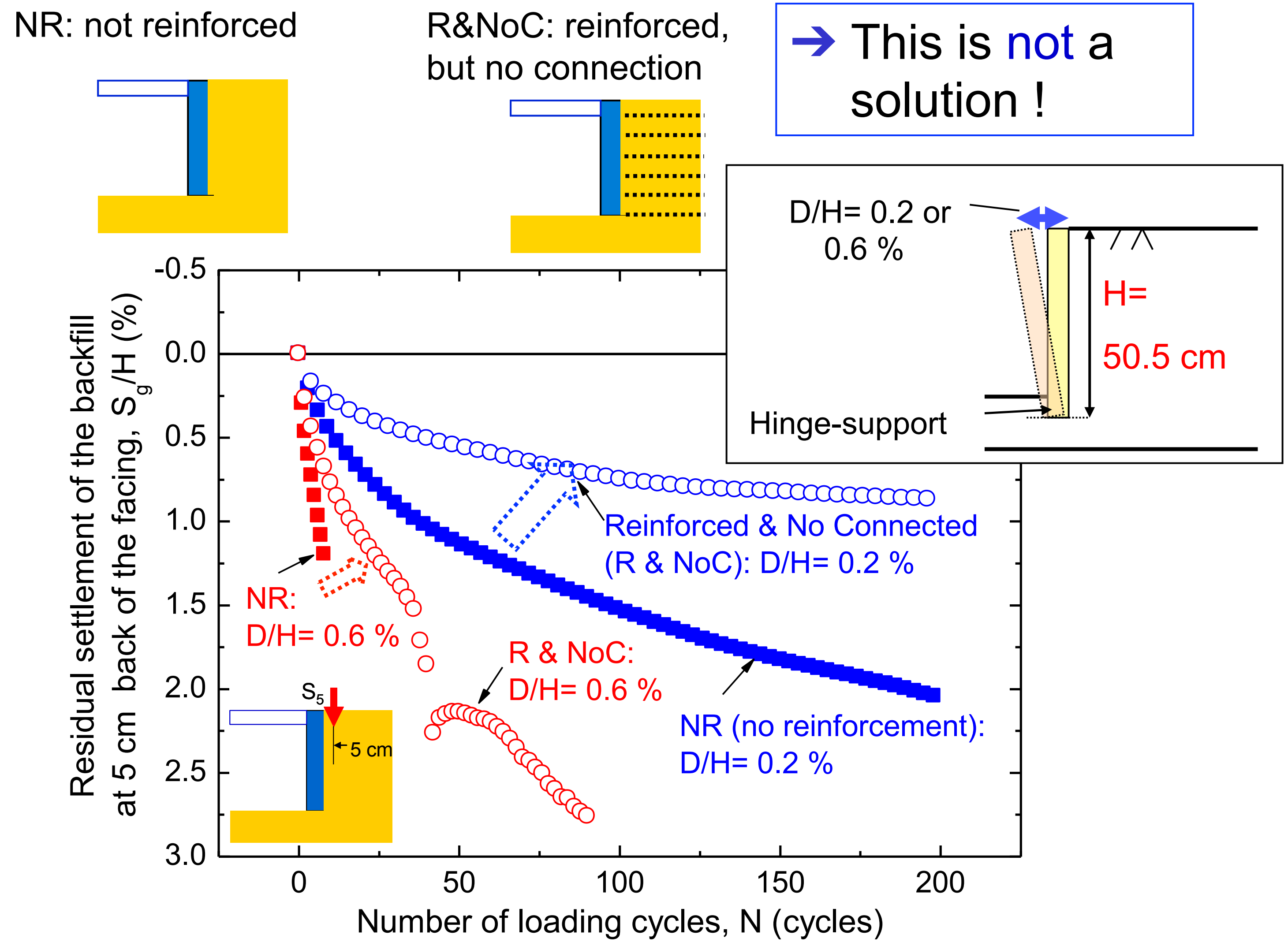
Gravel bags

0. Ground improvement (when necessary)

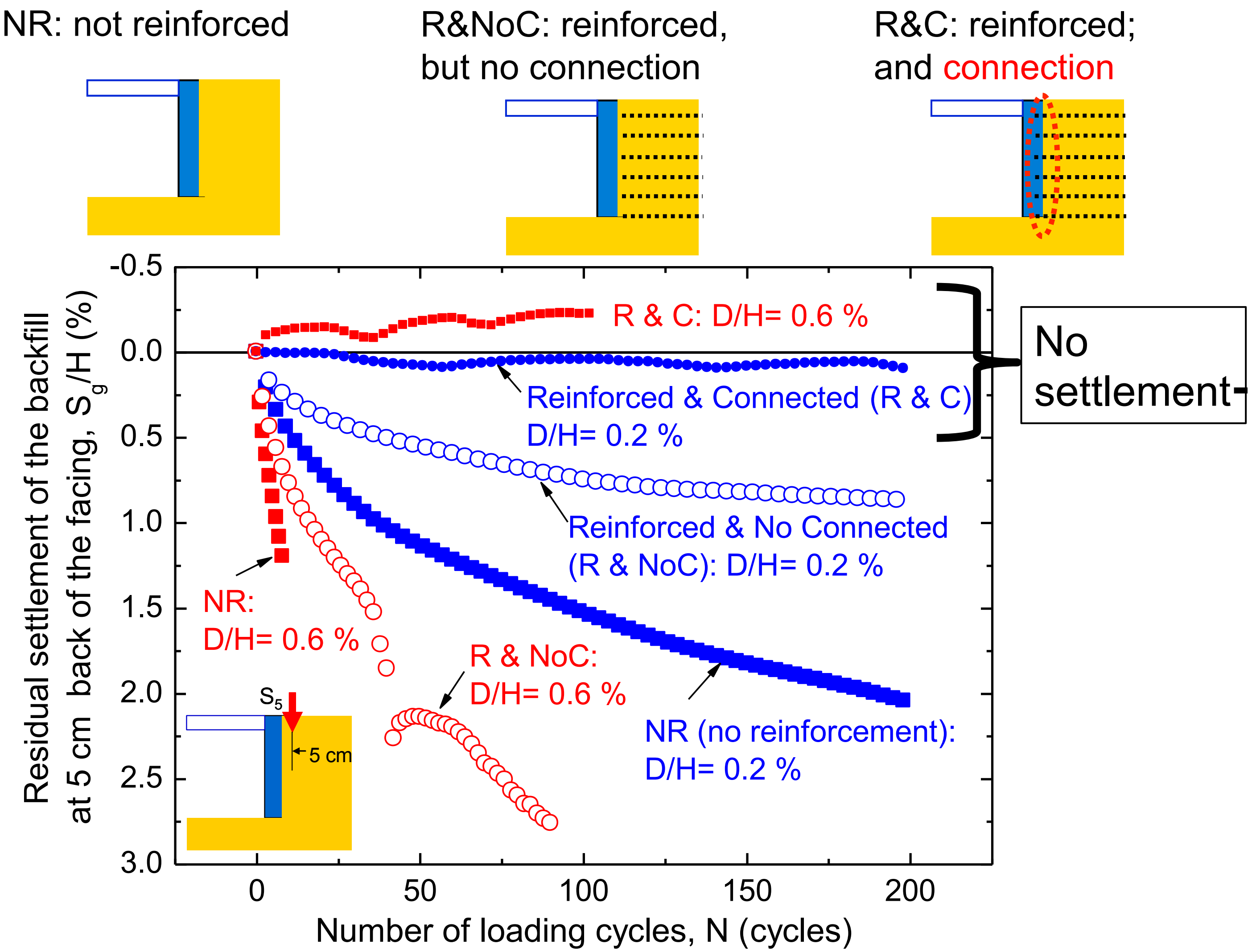
Effects of thermal cyclic deformation of the girder on the stability of backfill

NR: not reinforced





→ This is not a solution !



NR: not reinforced

R&NoC: reinforced, but no connection

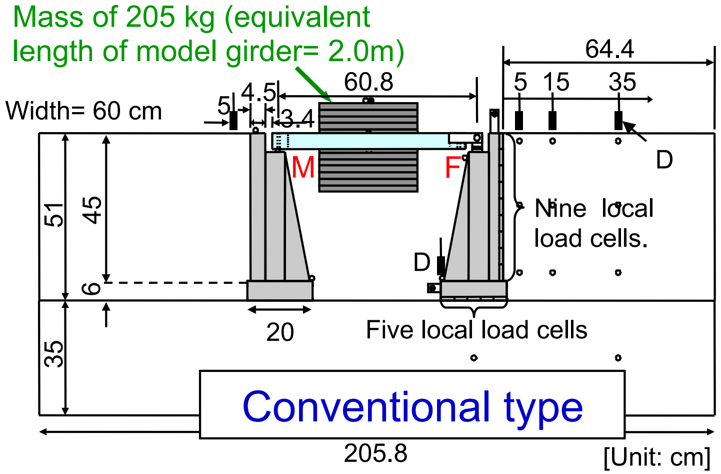
R&C: reinforced; and connection

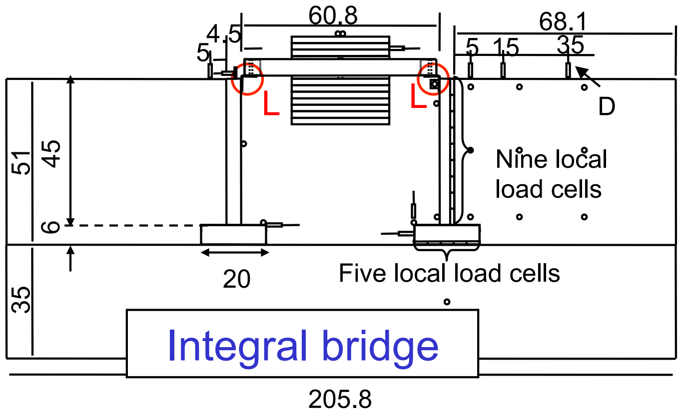
No settlement

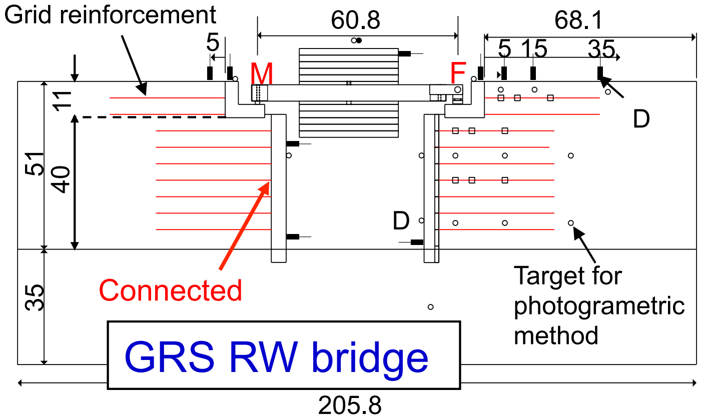
Shaking table tests in 1g

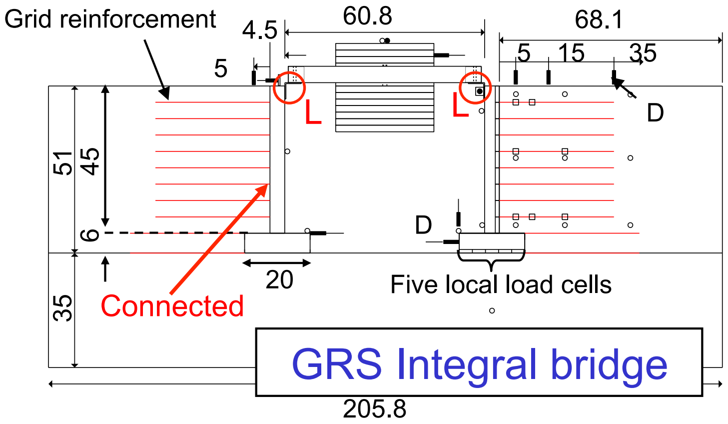
(considered model scale: 1/10)

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

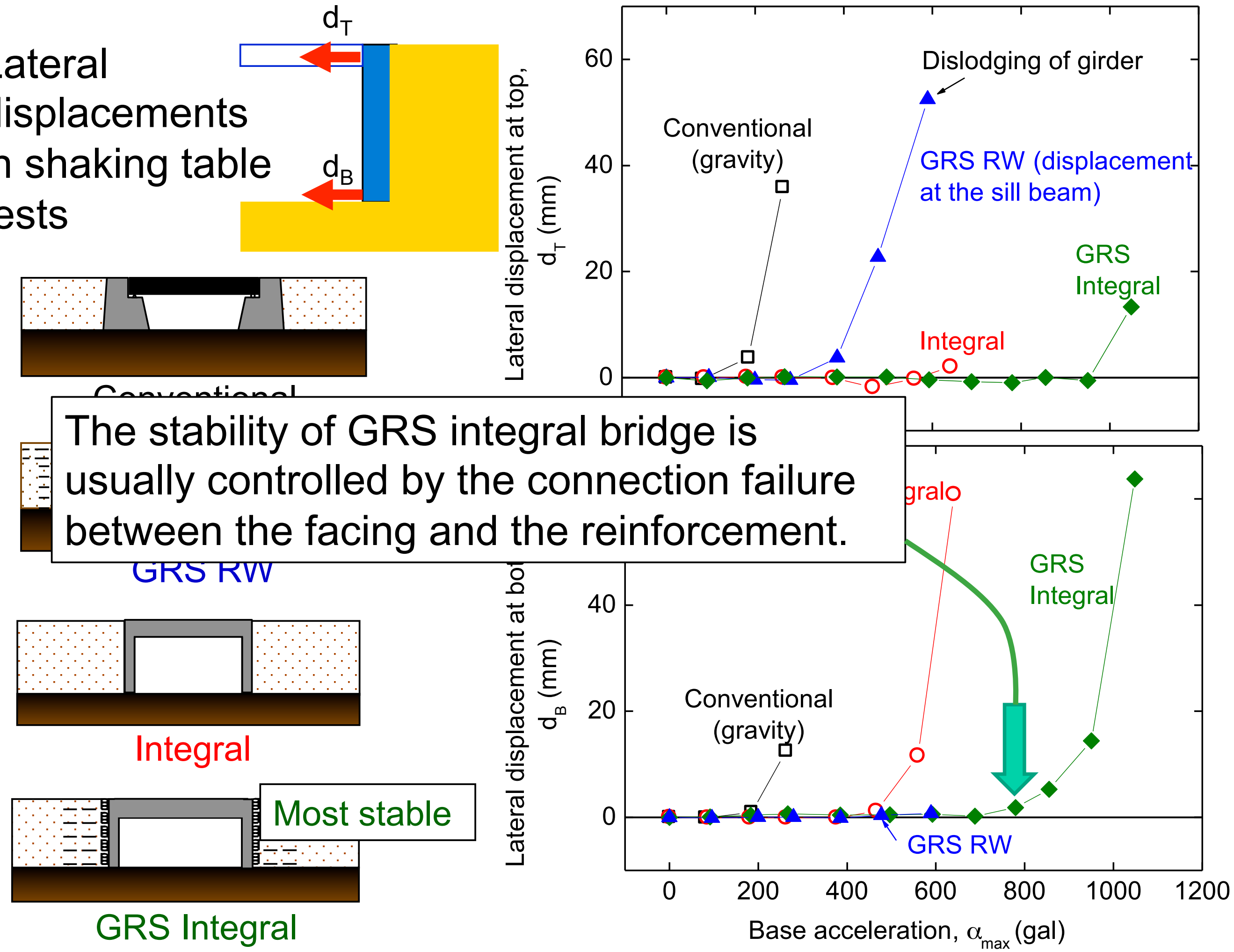








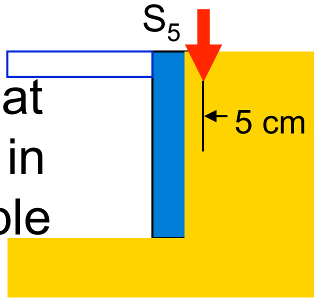
Lateral displacements in shaking table tests

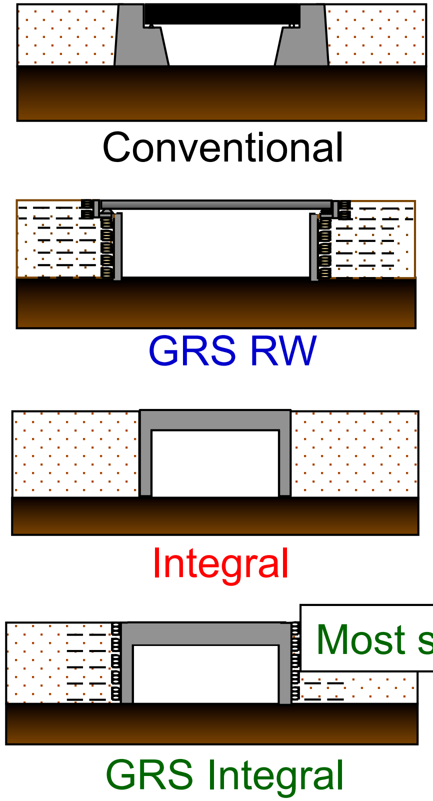


The stability of GRS integral bridge is usually controlled by the connection failure between the facing and the reinforcement.

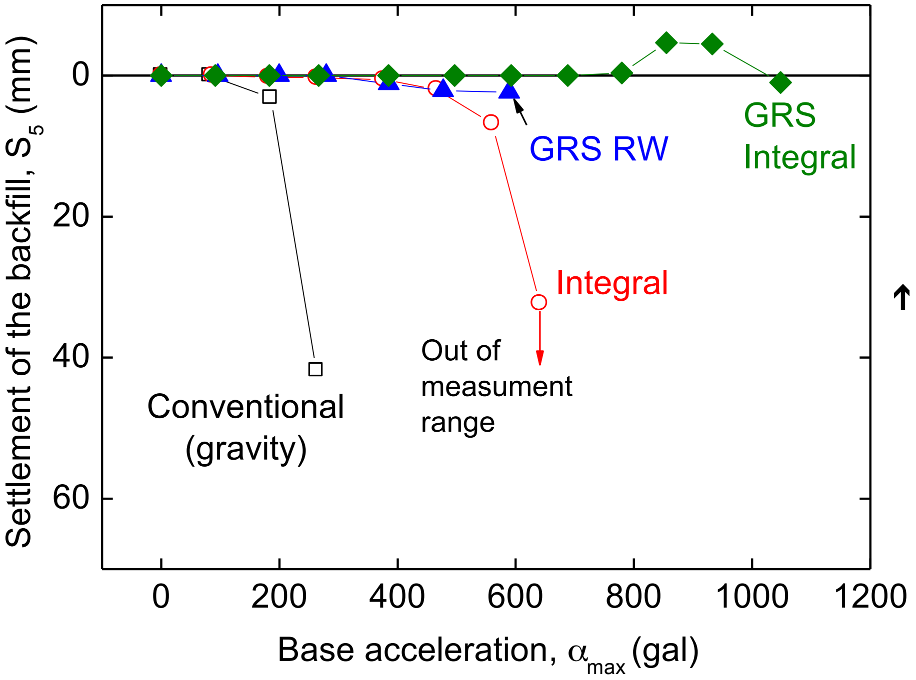
Most stable

Residual settlement at the backfill in shaking table tests



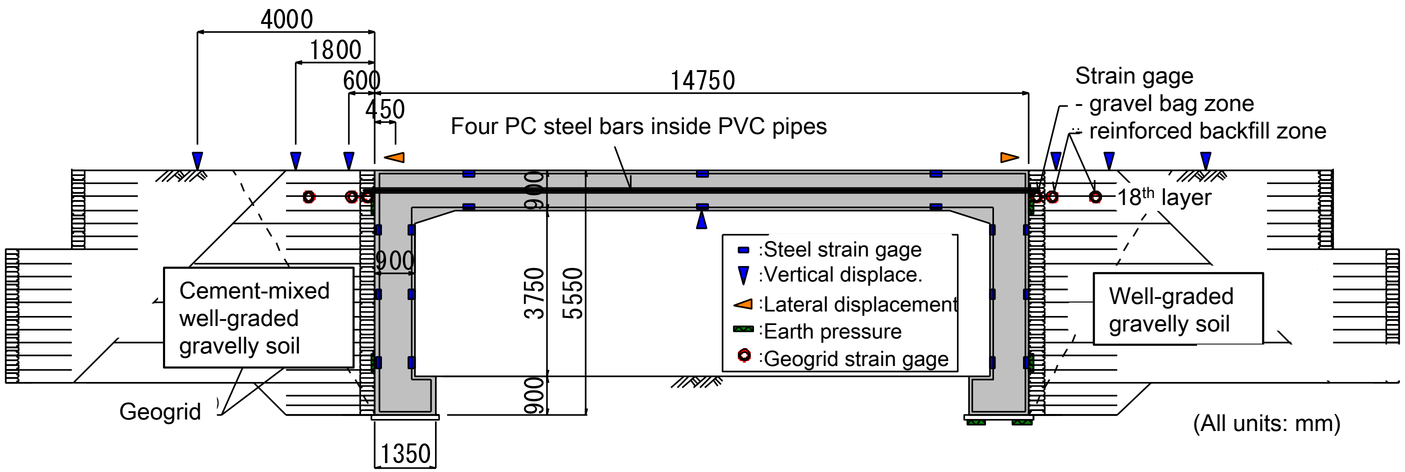


Most stable

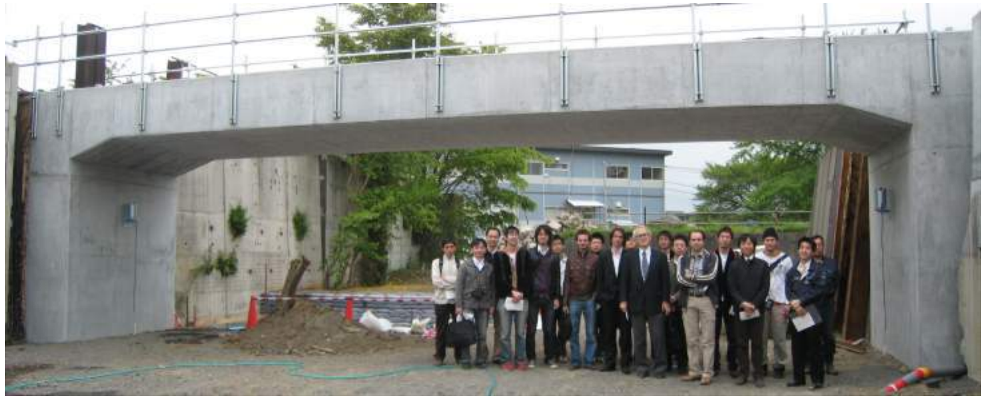


A very high dynamic stability of GRS Integral bridge

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

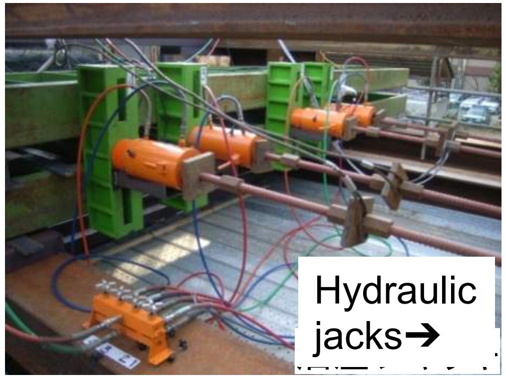


27 November 2008

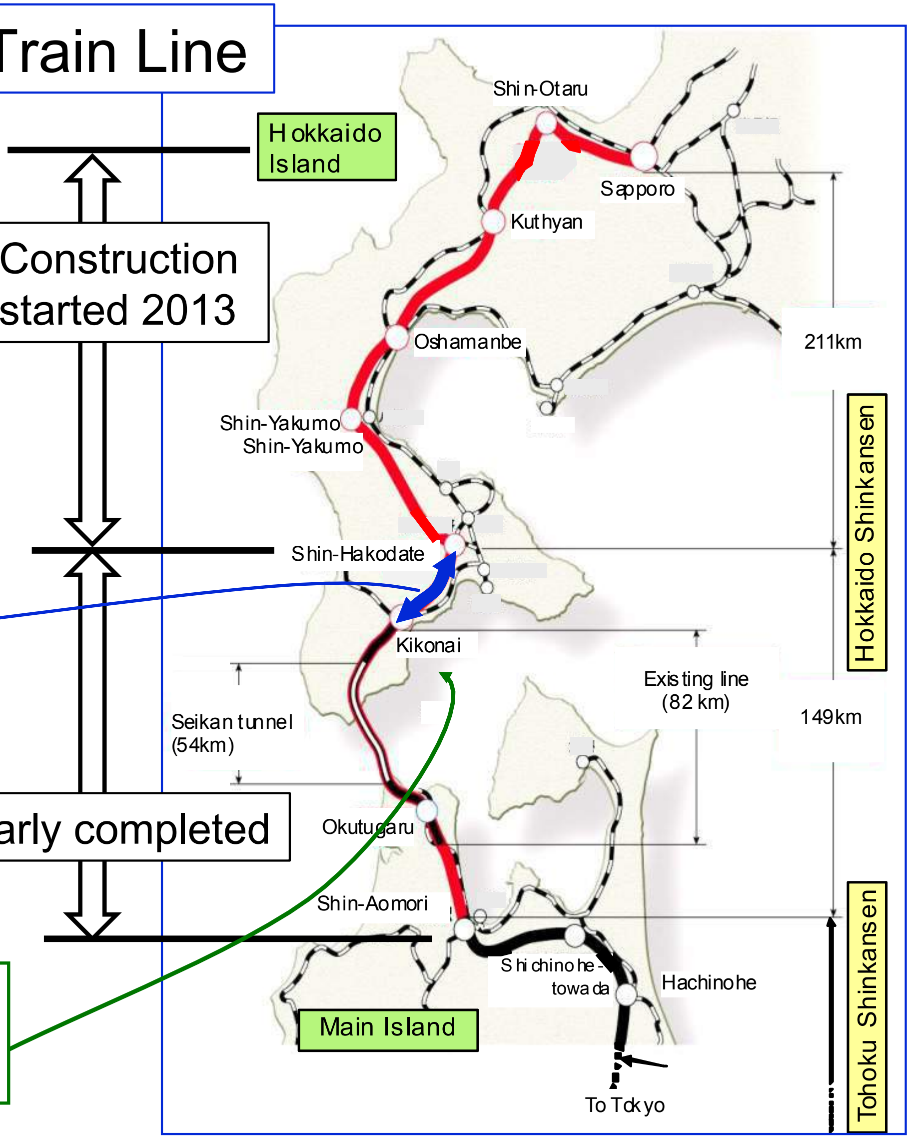


April. 2009

Cyclic lateral loading tests applying
1) thermal deformation of the girder; and
2) level 2 design seismic loads (Jan, 2012)➔



Hokkaido High-Speed Train Line



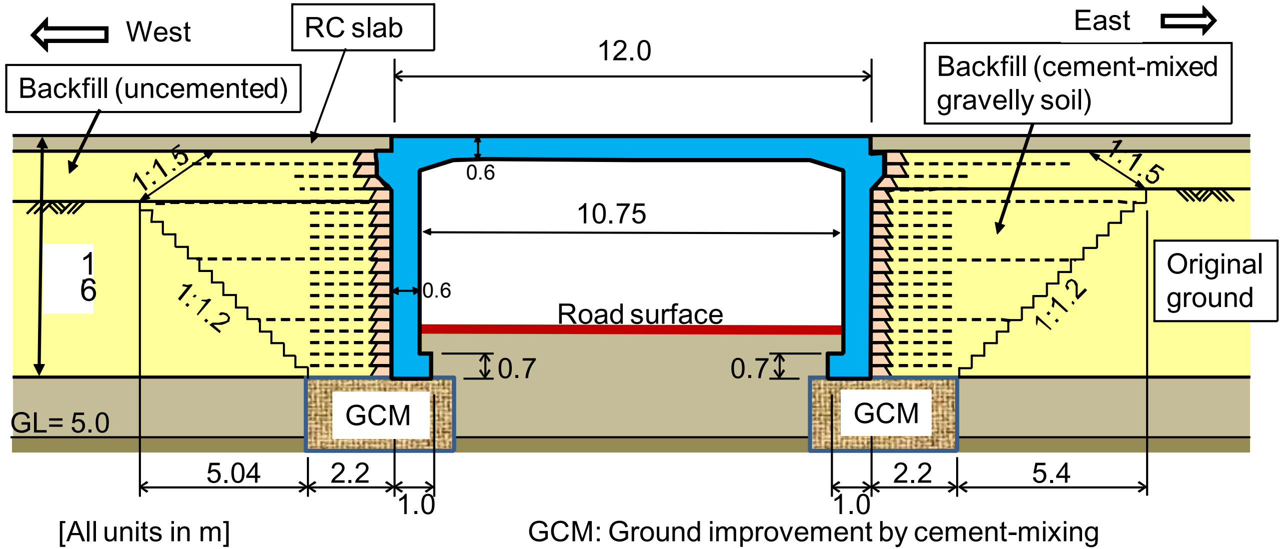
Construction started 2013

Nearly completed

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

GRS Integral Bridge, constructed 2011 - 2012

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

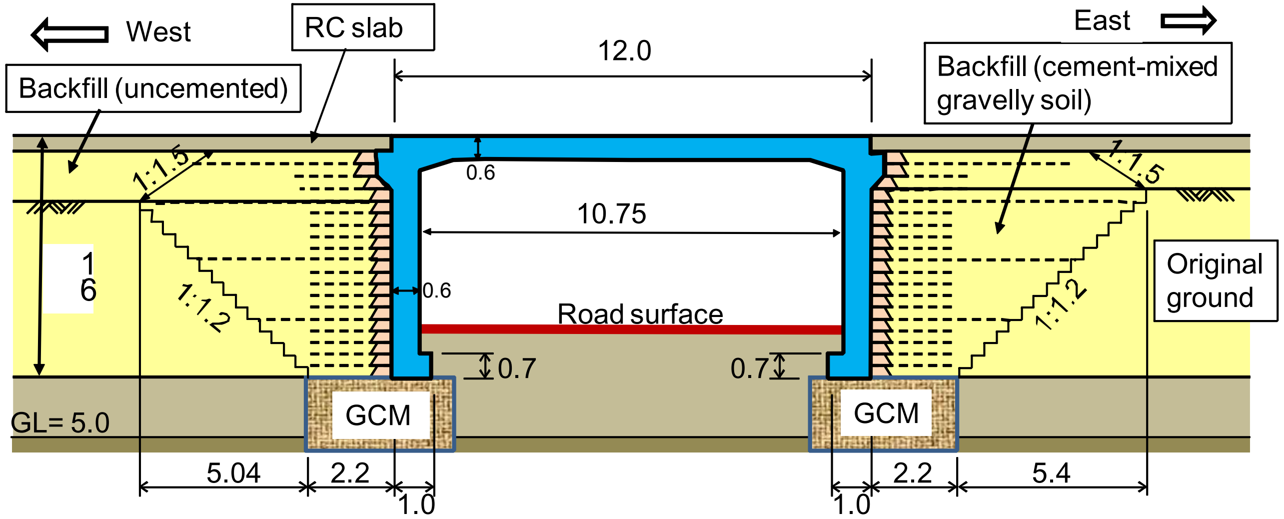


[All units in m]

GCM: Ground improvement by cement-mixing

(14th Oct. 2011).→

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



[All units in m]

GCM: Ground improvement by cement-mixing

(31 July 2012).➔

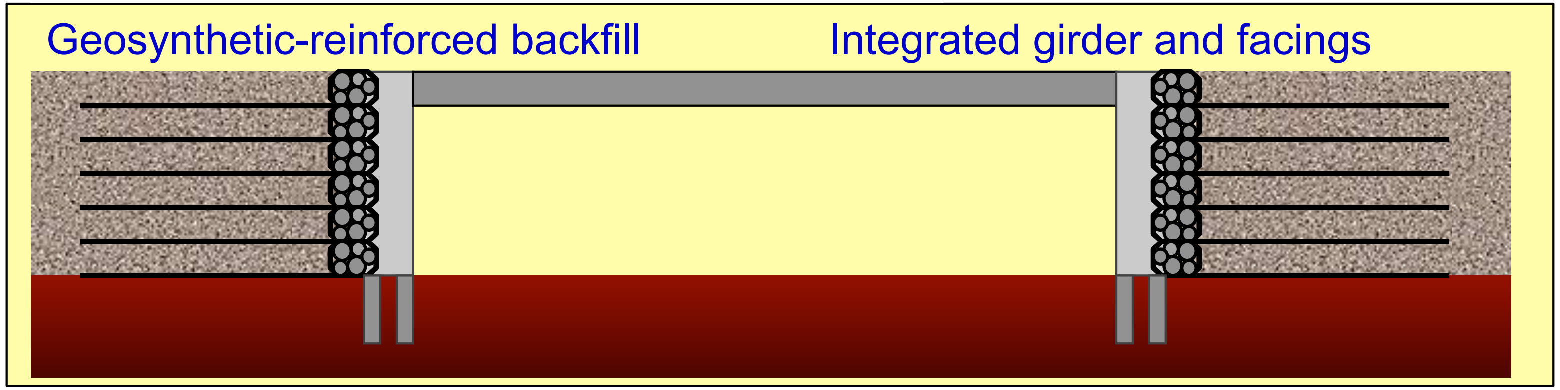
Great tsunami
2011 Great East Japan Earthquake

Damage to over 340 bridges by great tsunami during the 2011 Great East Japan Earthquake

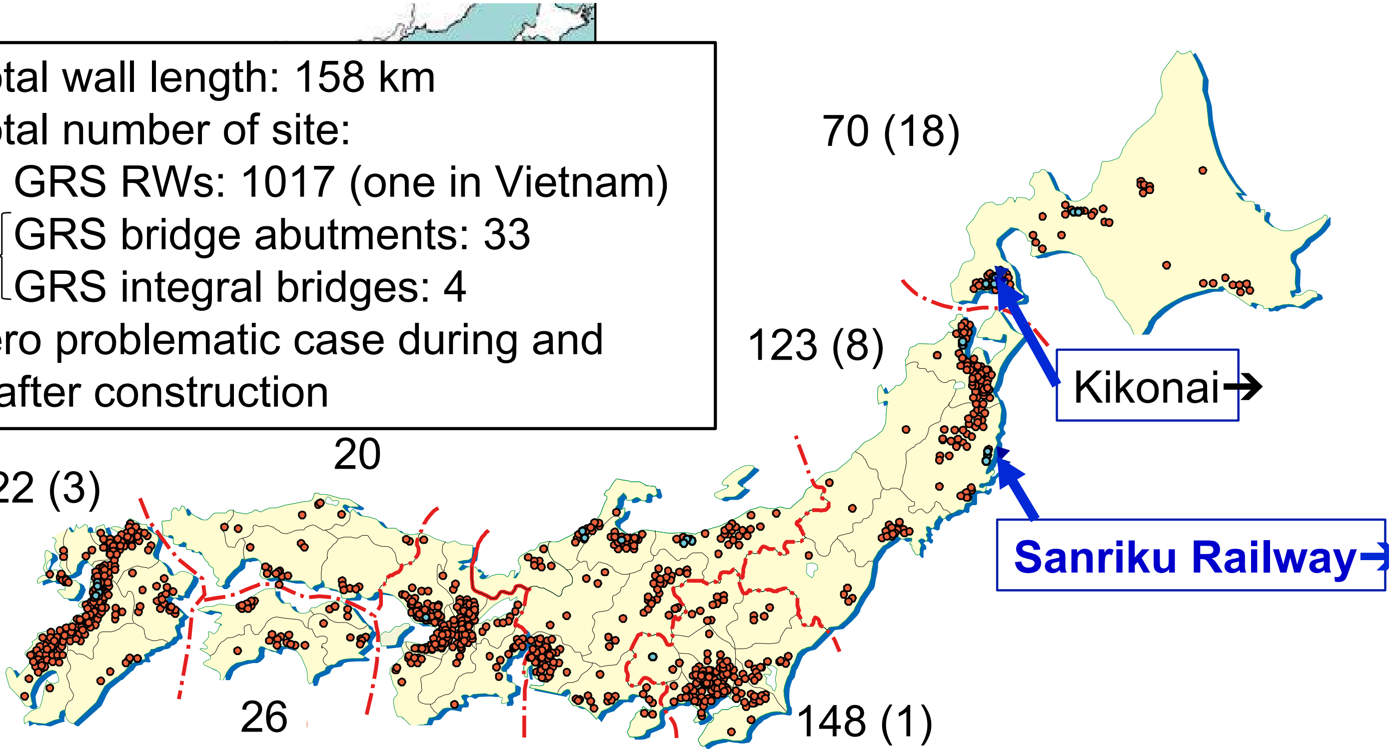
A railway bridge (Tsuyano-gawa bridge) that lost multiple simple-supported girders by tsunami forces

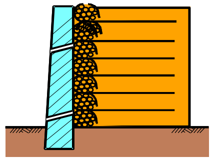
Girder bearings and approach fill are two major weak components of bridge for seismic & tsunami forces

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

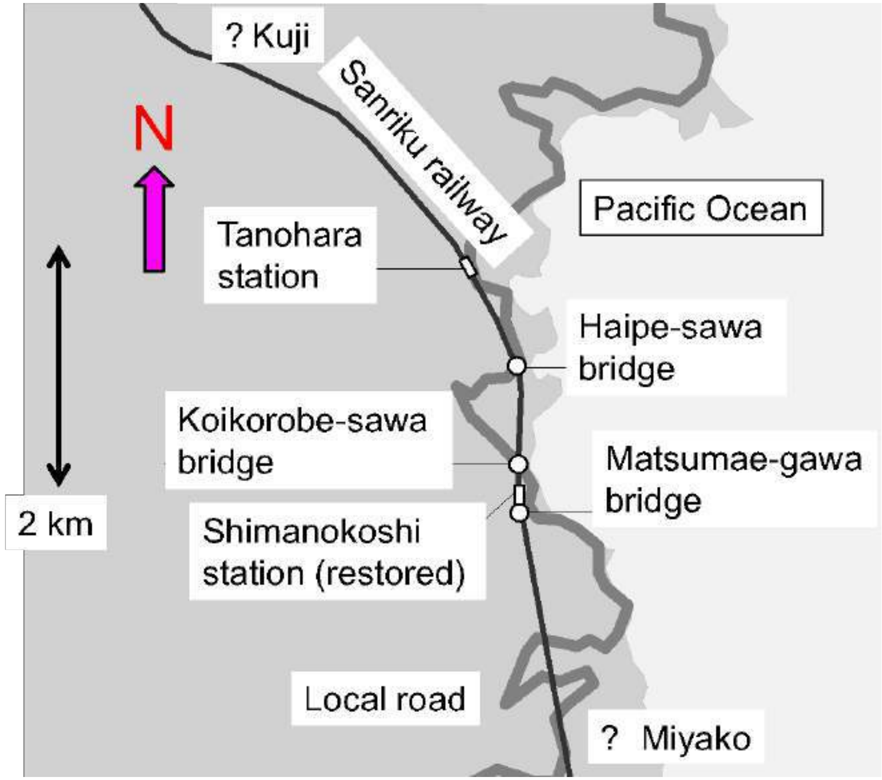




Locations of GRS RWs with a stage-constructed FHR facing as of June 2014

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:

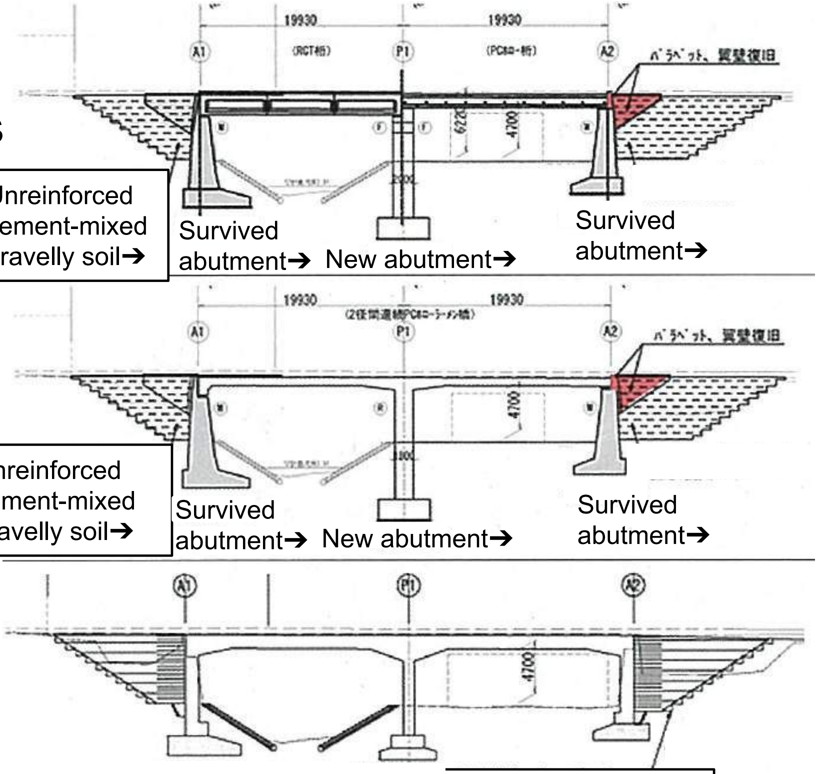
- Constructed 30 years ago taking into account tsunami effects.
- However, three bridges were lost by the tsunami during the 2011 Great East Japan EQ.

Immediately after the earthquake at Koikoreobe

30 March 2011

Comparison among different bridge types at Koikorobe➔

Construction & maintenance cost



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

Unreinforced cement-mixed gravelly soil➔

Survived abutment➔ New abutment➔ Survived abutment➔

Relatively high

Single continuous girder bridge with a pair of bearing

Unreinforced cement-mixed gravelly soil➔

Survived abutment➔ New abutment➔ Survived abutment➔

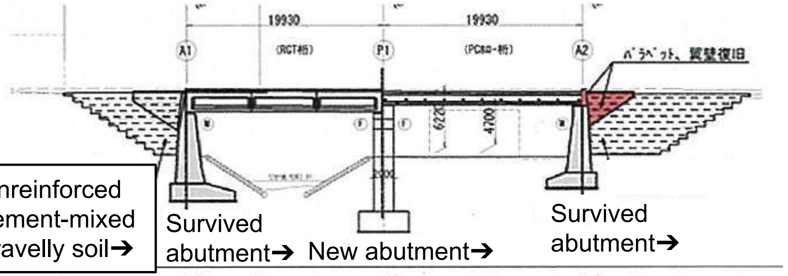
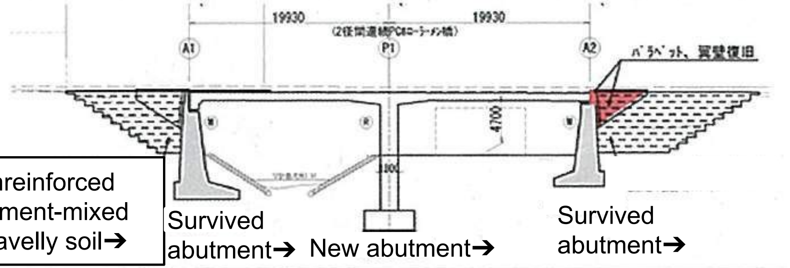
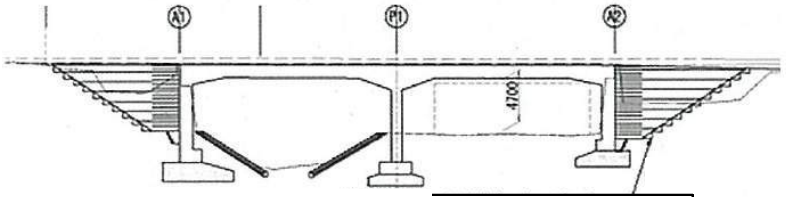
Relatively high

GRS integral bridge with no bearing (adopted plan)➔

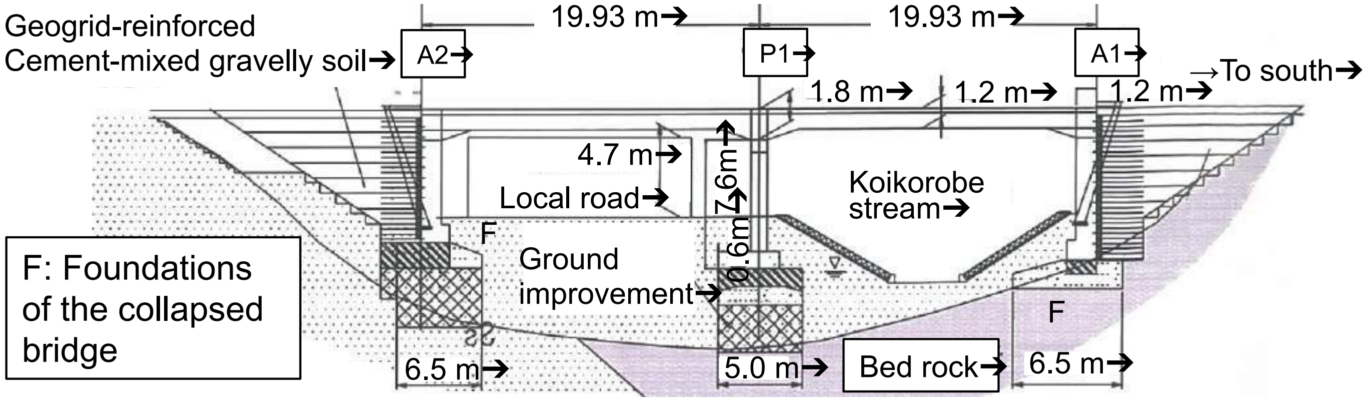
Geogrid- reinforced cement-mixed gravelly soil➔

Relatively low➔

Comparison among different bridge types at Koikorobe➔

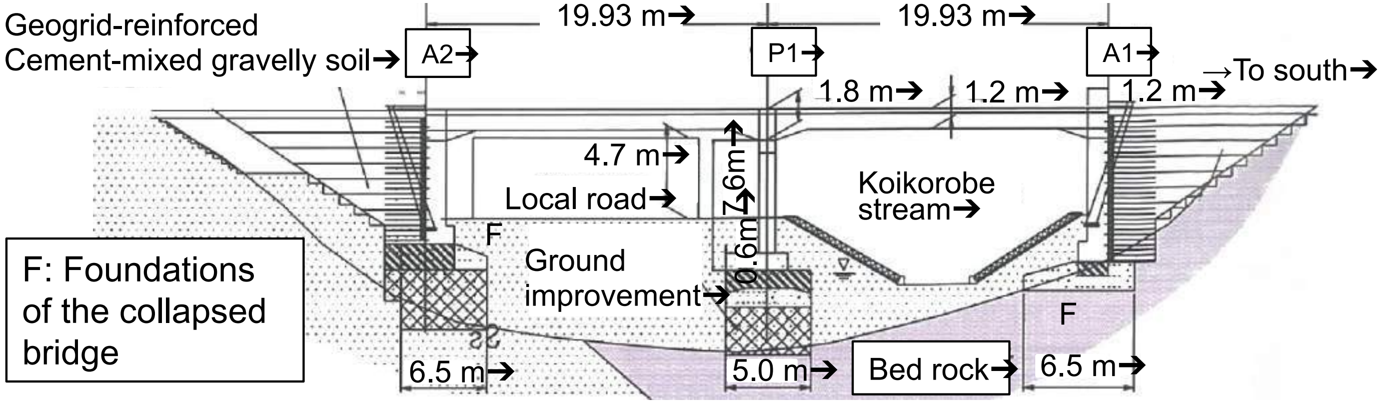
		Seismic stability	Anti-tsunami stability
Two-span simple bridge (the same as the one collapsed by tsunami)➔	 Unreinforced cement-mixed gravelly soil➔ Survived abutment➔ New abutment➔ Survived abutment➔	Low	Low
Single continuous girder bridge with a pair of bearing	 Unreinforced cement-mixed gravelly soil➔ Survived abutment➔ New abutment➔ Survived abutment➔	Interme-diate	Low
GRS integral bridge with no bearing (adopted plan)➔ Highest performance/cost ratio➔	 Geogrid- reinforced cement-mixed gravelly soil➔	High➔	High➔

GRS integral bridge at Koikorobe for Sanriku Railway

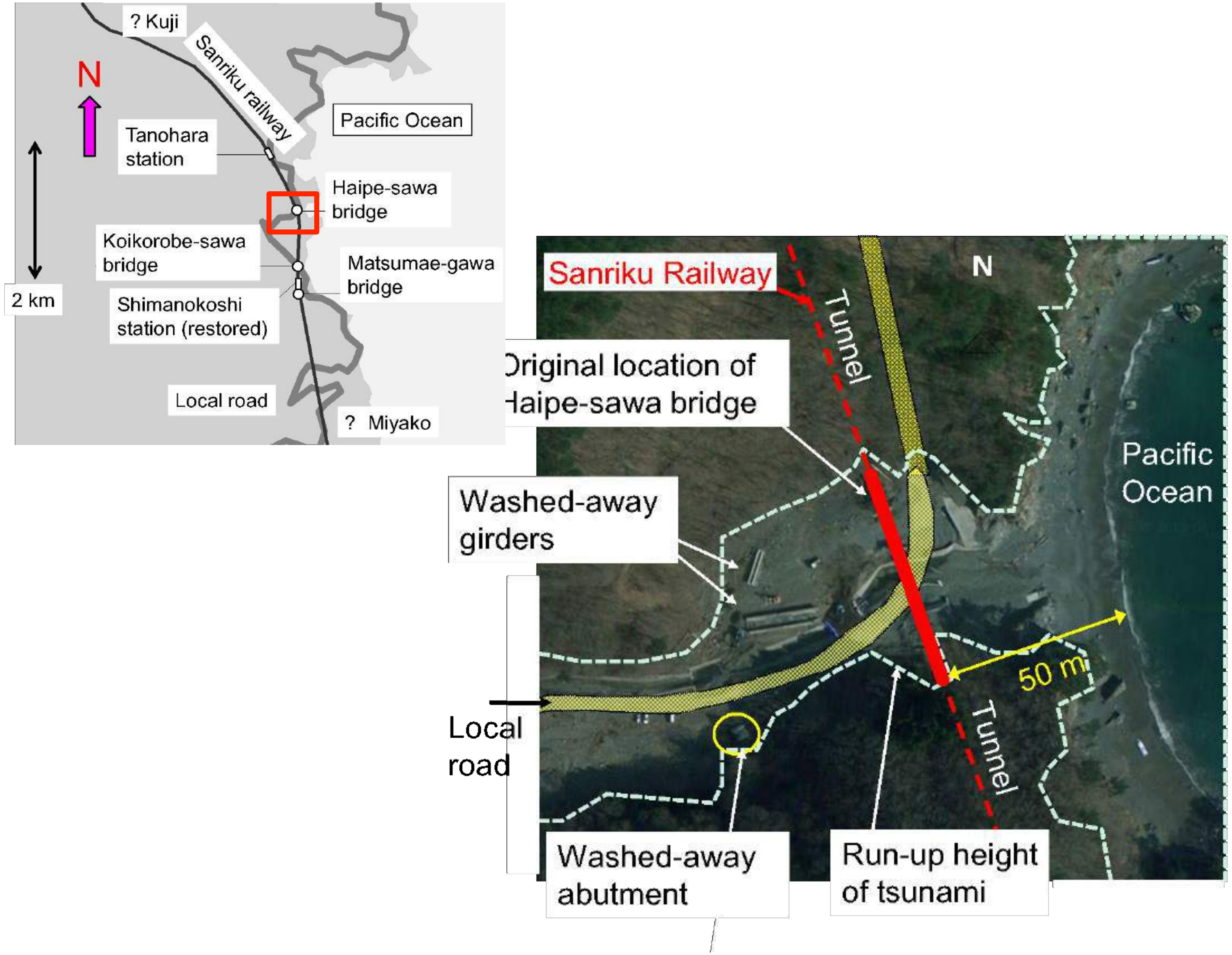


3 November 2013

GRS integral bridge at Koikorobe for Sanriku Railway



6 April 2014

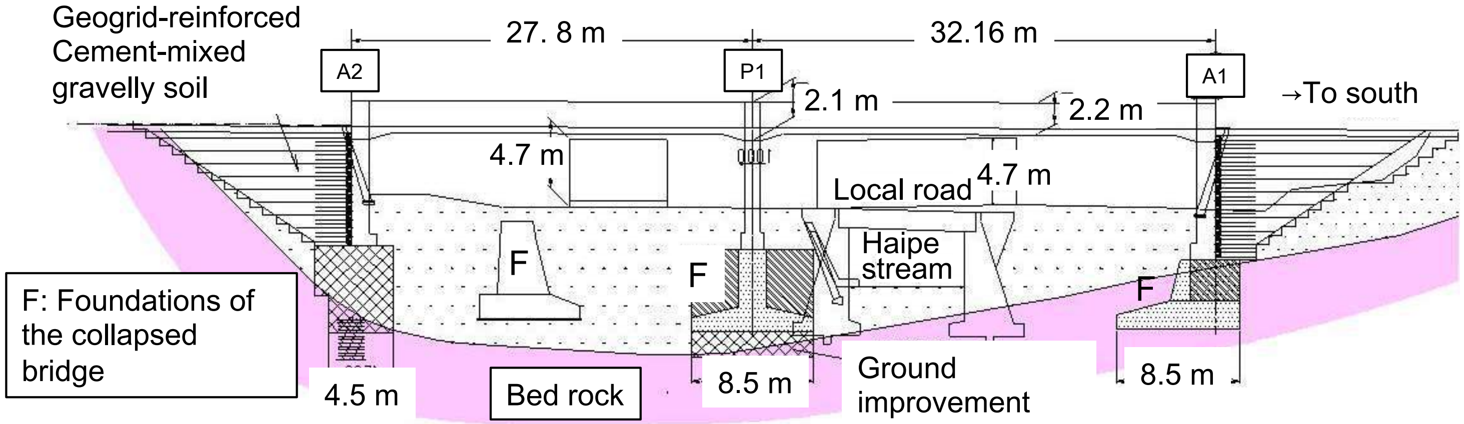

? Kuji
Sanriku railway
N
Pacific Ocean
Tanohara station
Haipe-sawa bridge
Koikorobe-sawa bridge
Matsumae-gawa bridge
2 km
Shimanokoshi station (restored)
Local road
? Miyako
Sanriku Railway
Tunnel
N
Original location of Haipe-sawa bridge
Pacific Ocean
Washed-away girders
50 m
Local road
Tunnel
Washed-away abutment
Run-up height of tsunami

Haipe, Sanriku Railway

Immediately after the earthquake

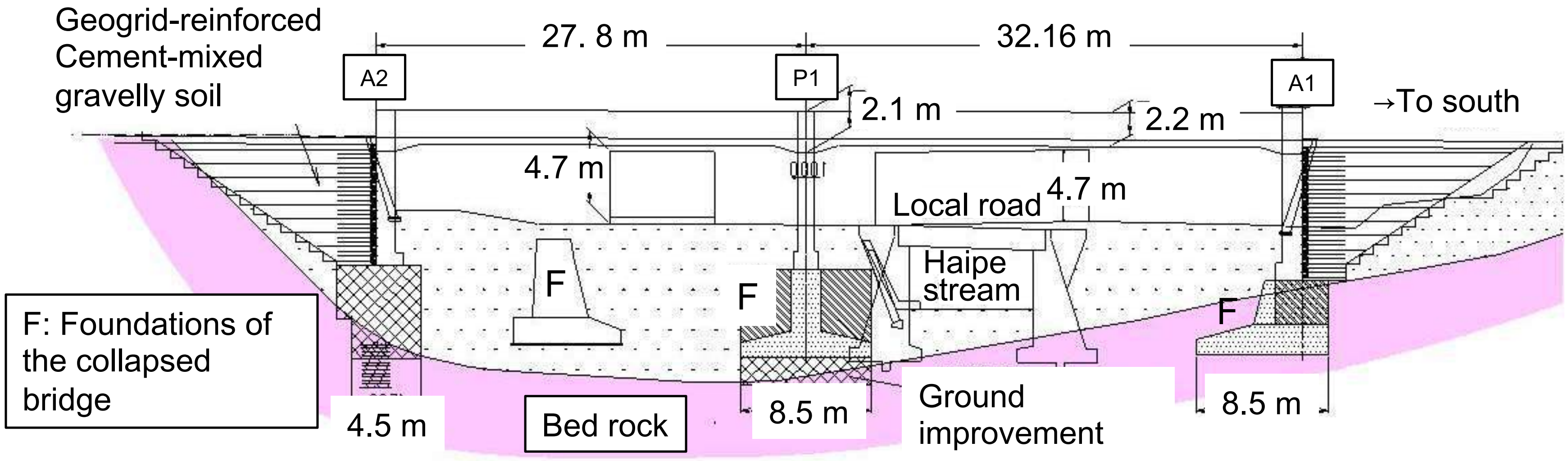
30 March 2011

GRS integral bridge at Haipe, Sanriku Railway

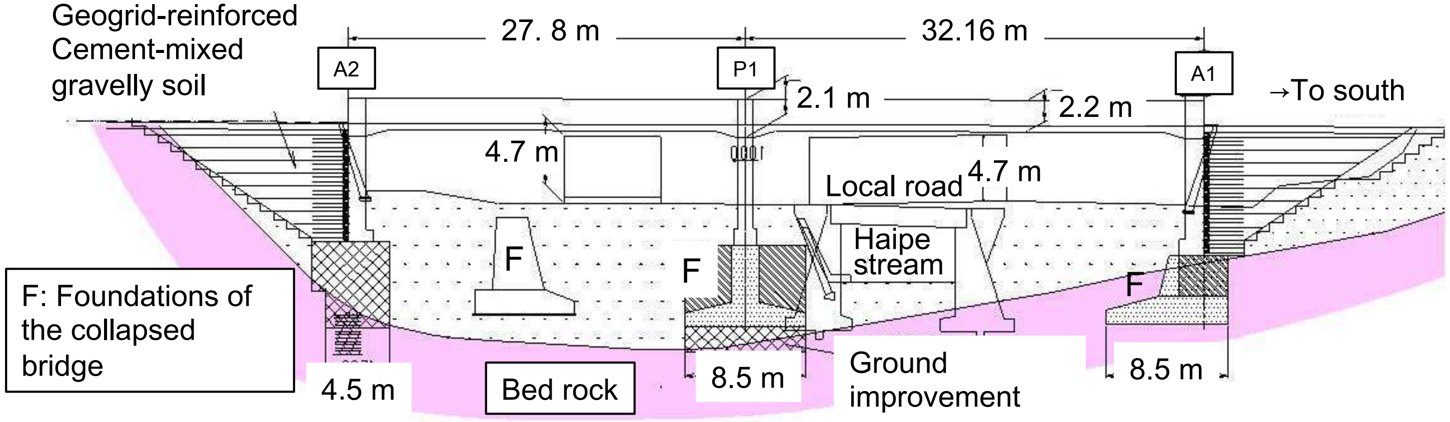


22 May 2013-

GRS integral bridge at Haipe, Sanriku Railway

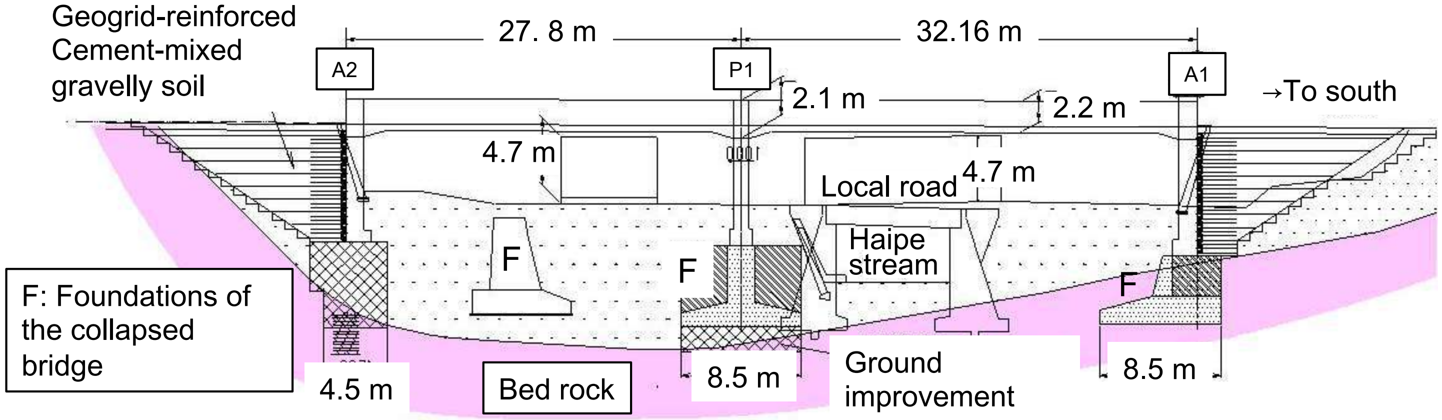


GRS integral bridge at Haipe, Sanriku Railway



6 April 2014

GRS integral bridge at Haipe, Sanriku Railway



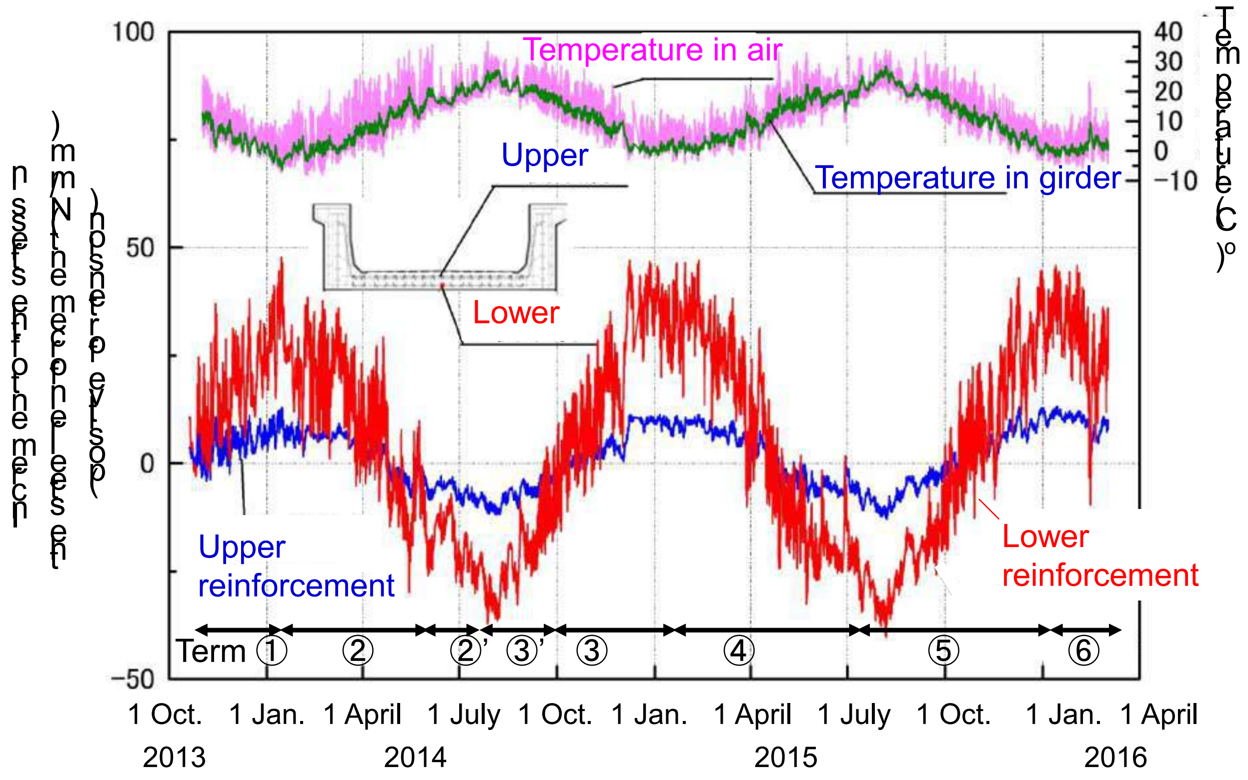
20 May 2014-

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



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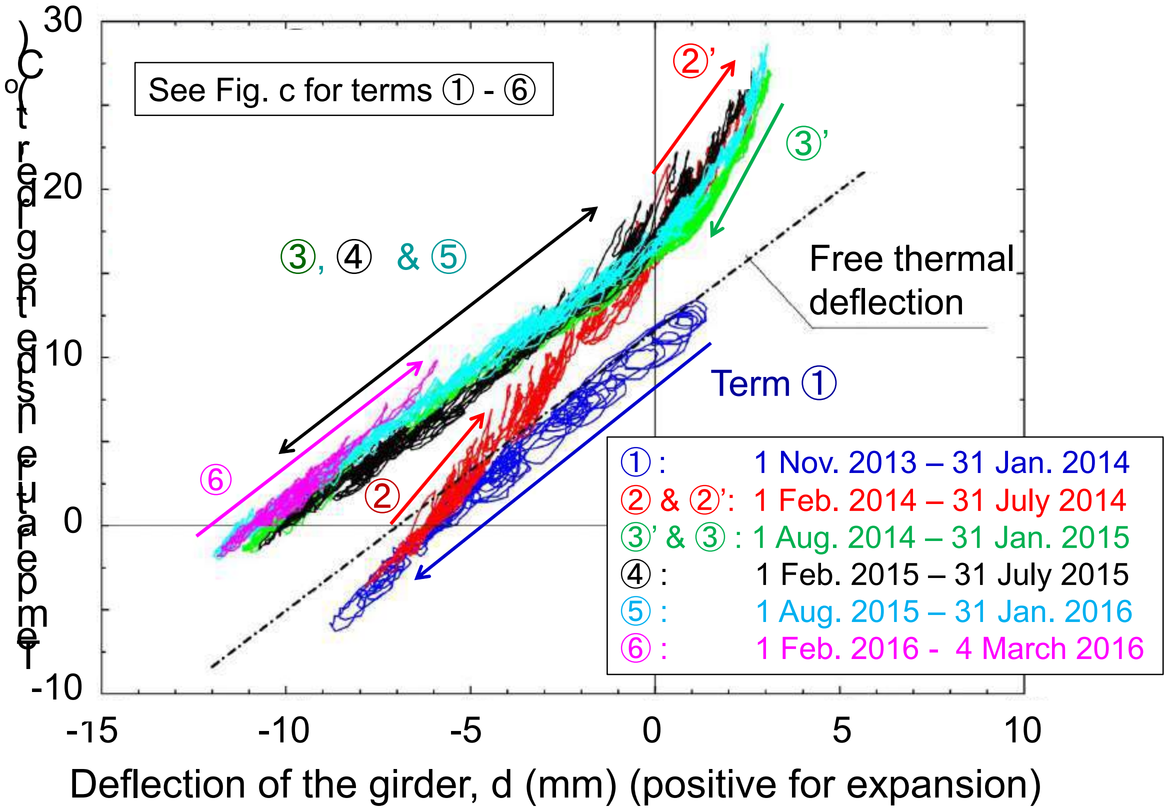


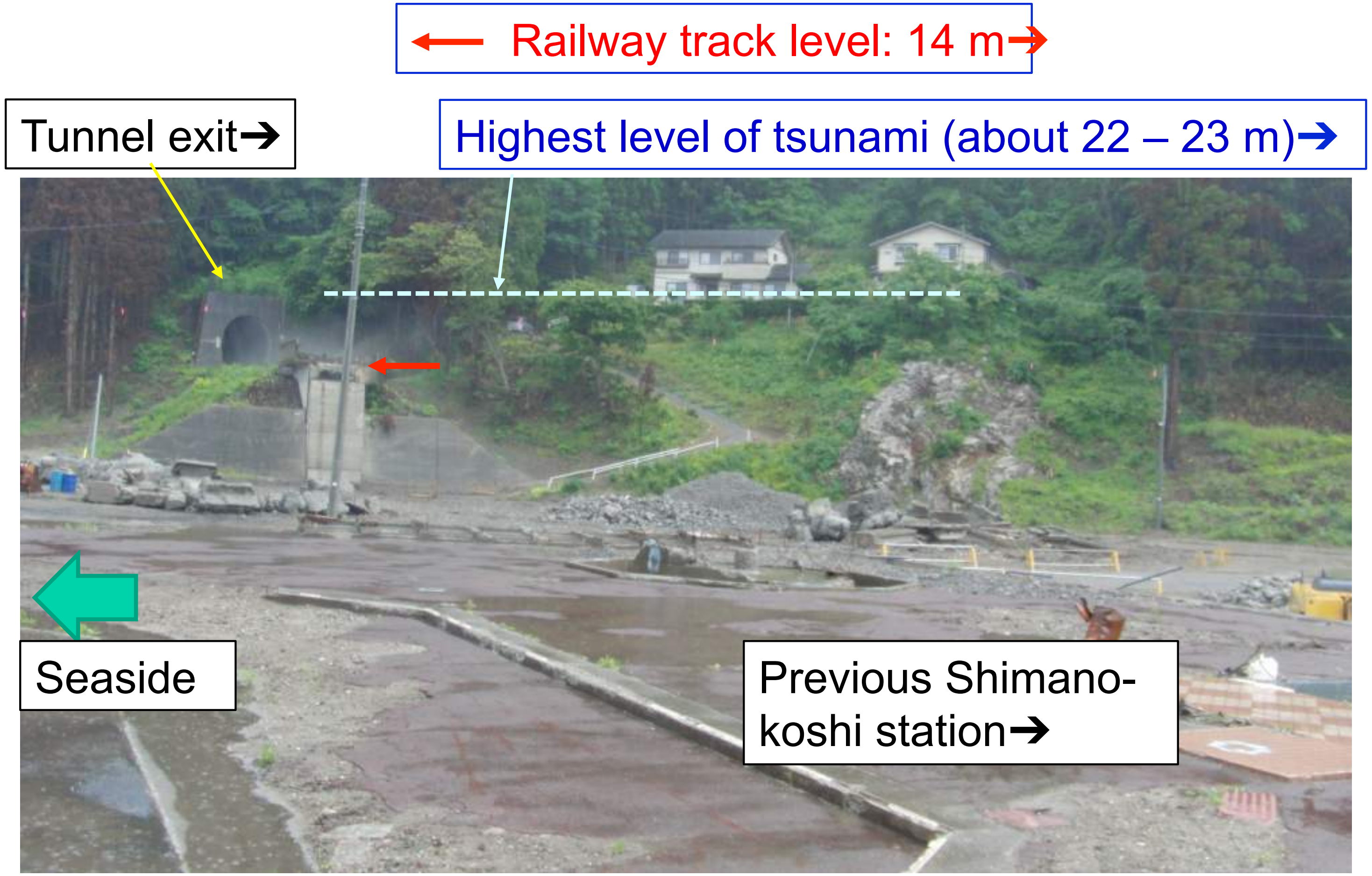
Shima-no-koshi at Sanriku Railway

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→

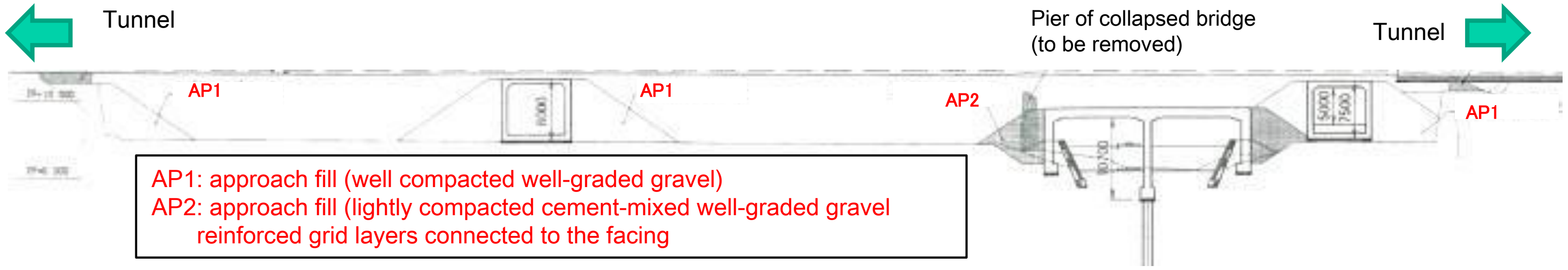
Tunnel exit→

Highest level of tsunami (about 22 – 23 m)→

Seaside

Previous Shimano-koshi station→

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



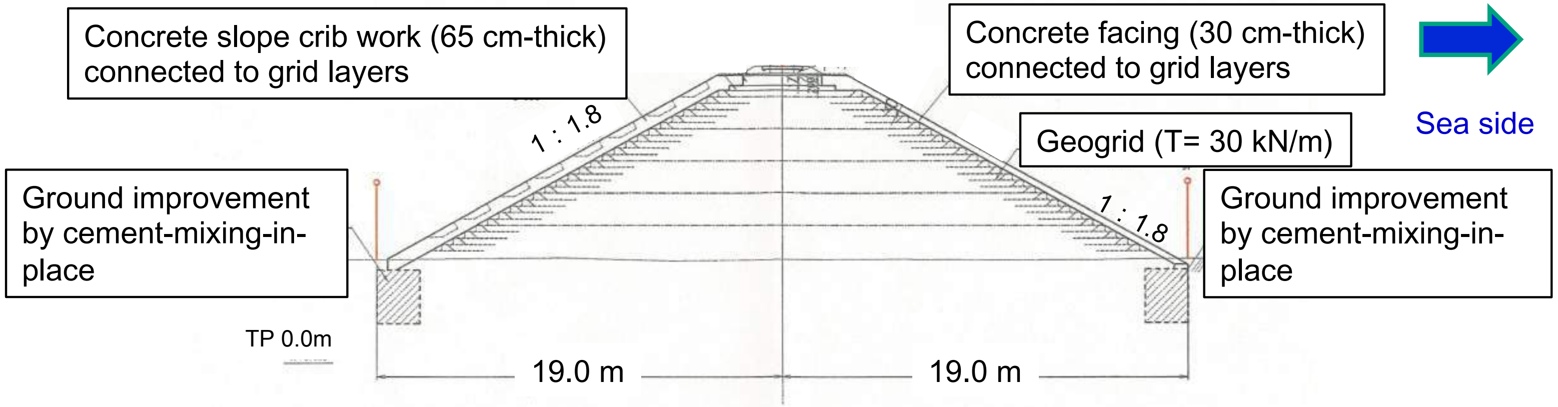
Tunnel

Pier of collapsed bridge
(to be removed)

Tunnel

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

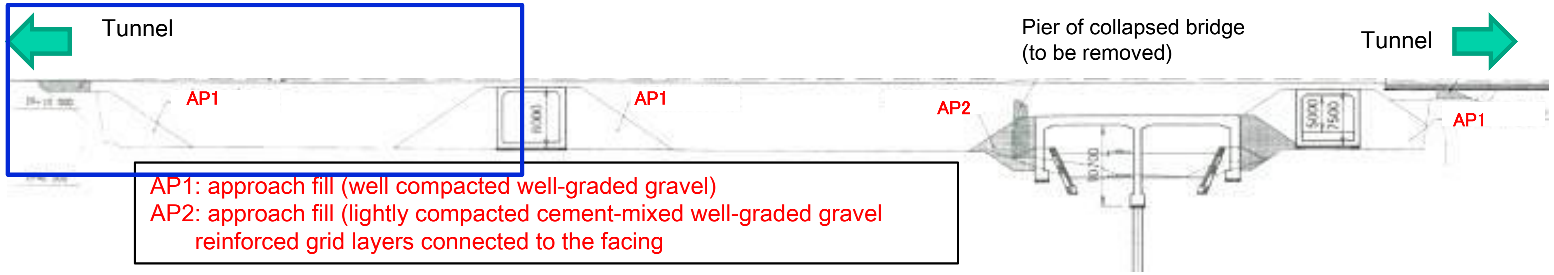
Sea side

Geogrid (T= 30 kN/m)

Ground improvement by cement-mixing-in-place

Ground improvement by cement-mixing-in-place

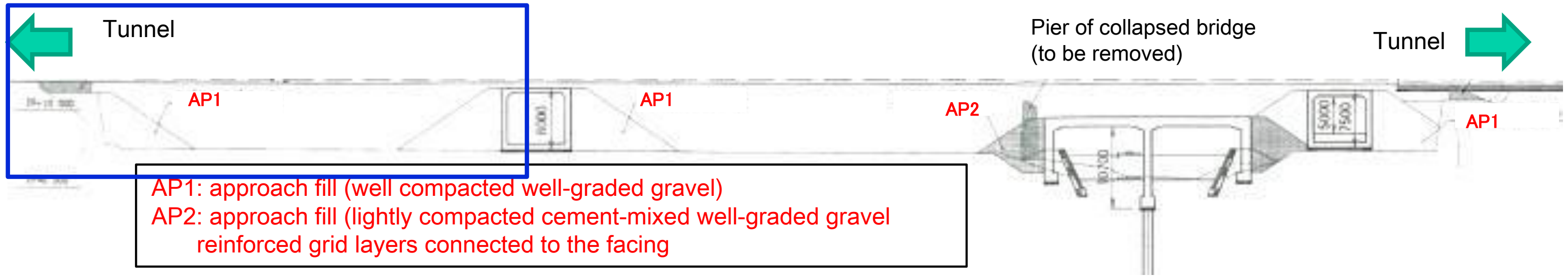
GRS embankment and GRS integral bridge, Shima-no-koshi, Sanriku Railway



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)
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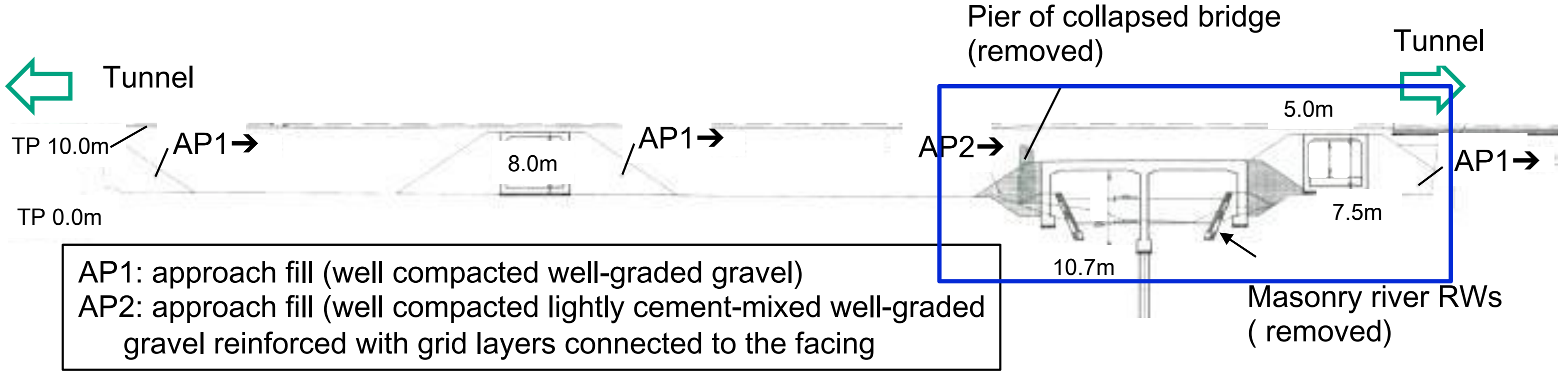
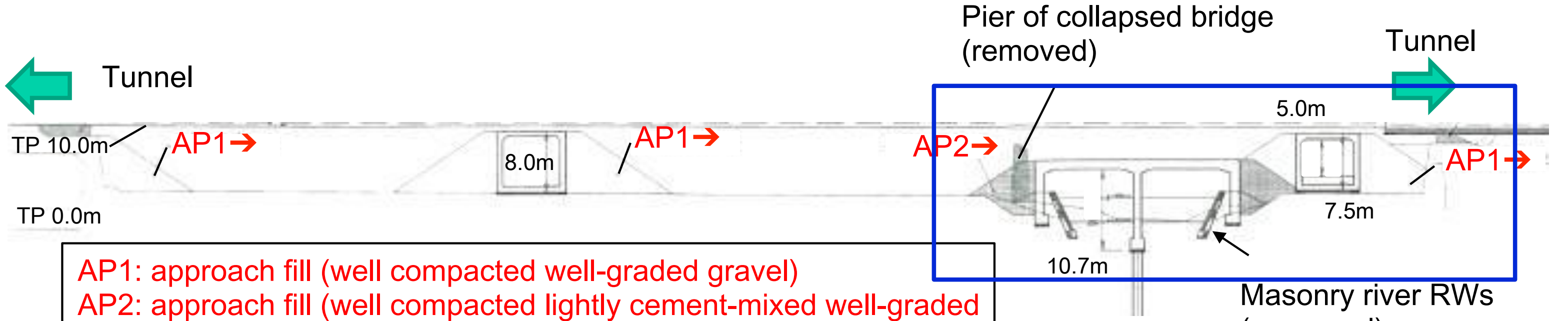
reinforced grid layers connected to the facing


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



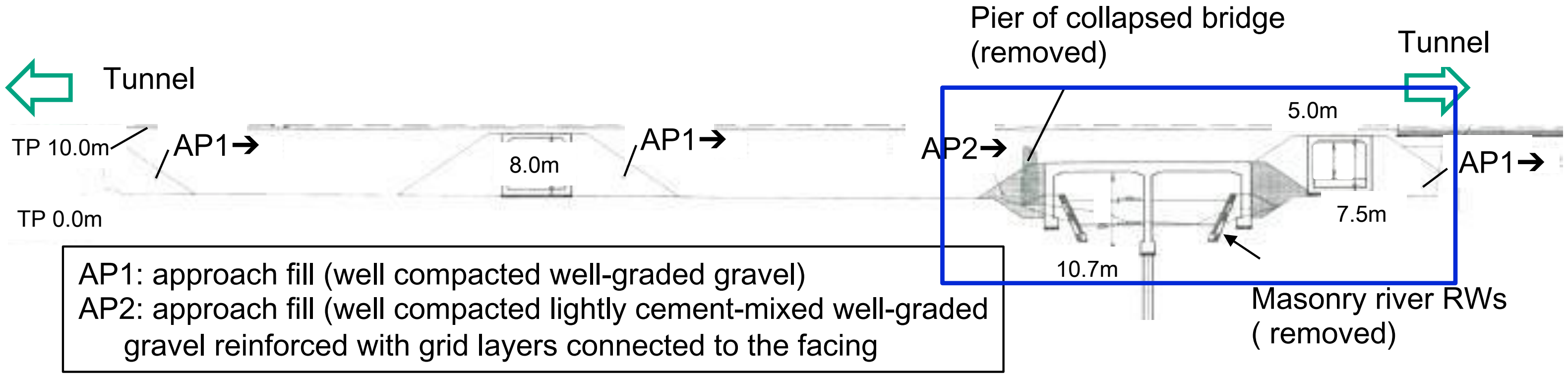
AP1: approach fill (well compacted well-graded gravel)
AP2: approach fill (well compacted lightly cement-mixed well-graded gravel reinforced with grid layers connected to the facing

GRS integral bridge

Box culvert

Old abutment
(removed)

19 June 2013



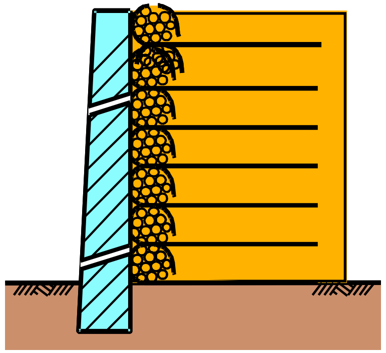
AP1: approach fill (well compacted well-graded gravel)
AP2: approach fill (well compacted lightly cement-mixed well-graded gravel reinforced with grid layers connected to the facing

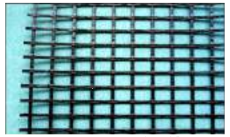
20 May 2014→

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.





Typical polymer geogrid

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.

2

3rd ICTG 2016

4-7 September 2016, Guimarães, Portugal

Universidade do Minho
Escola de Engenharia

SIMSG ISSMGE

Workshop 1 – Geosynthetics in Transportation Geotechnics

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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)

ZAG

The first GRS integral bridge in the world, constructed at high-speed train line (Kikonai, Hokkaido, Japan)

September, 2011

August, 2012

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

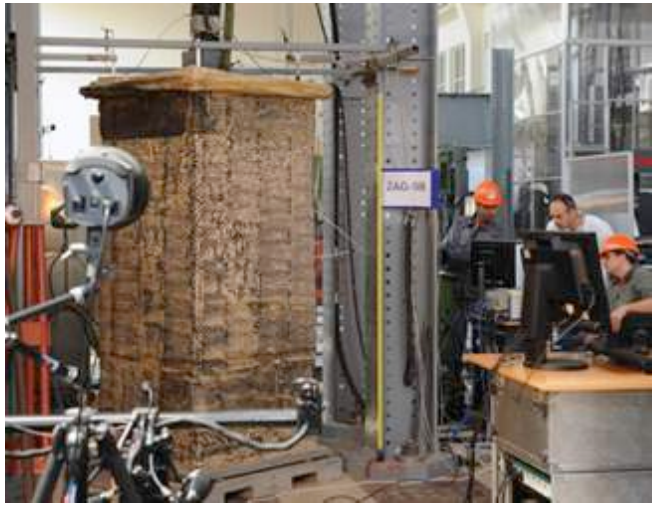
- rehabilitation of local traffic infrastructure (investment in railway line rehabilitation)
- box-shaped culvert
- insufficient water flow capacity
- deep layer of soft foundation soil
- very short deadlines

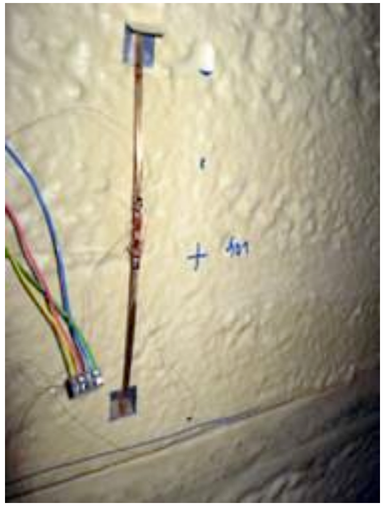
Depth [m]	Description	Soil properties
0.0 – 0.5	sandy gravel	
0.5 – 3.0	sandy clay with inclusions of gravel and sand	$(N_1)_{60}$=6
3.0 – 5.0	clayey and silty sand	$(N_1)_{60}$=8, c'= 1.6 kPa, φ'=25.7°, w=33.5%,
5.0 – 8.0	silty sand	$(N_1)_{60}$=12, w=29.1%, I_p=10.4%
8.0 – 11.0	decayed stratified marl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

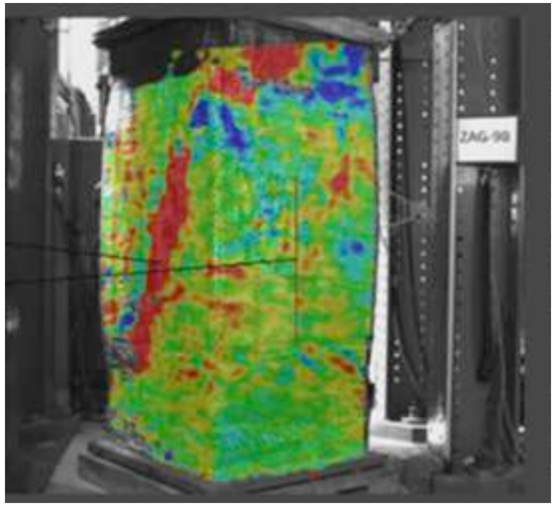
Water level depth: 2.7 m

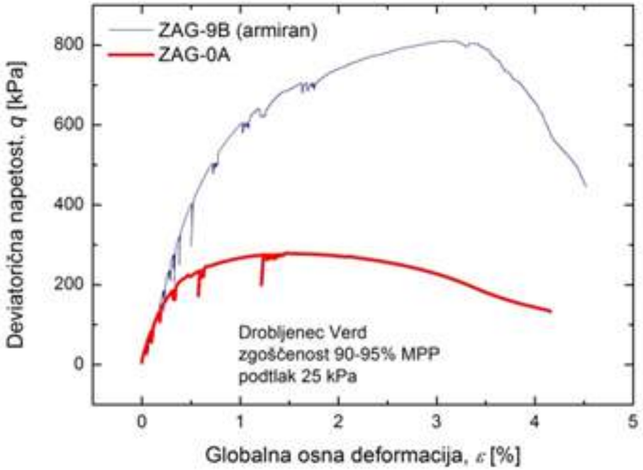
Introduction

- 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



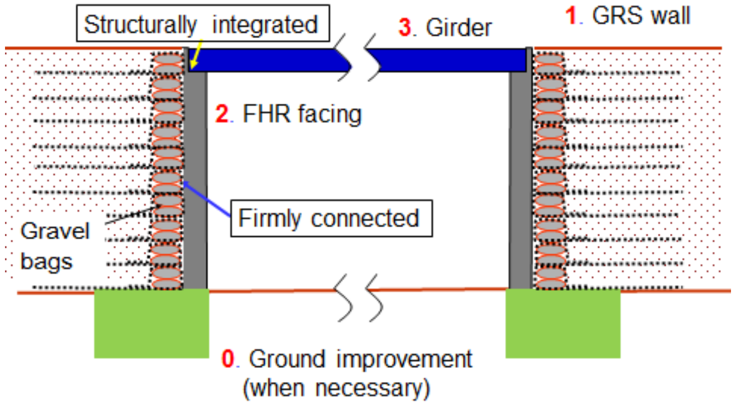


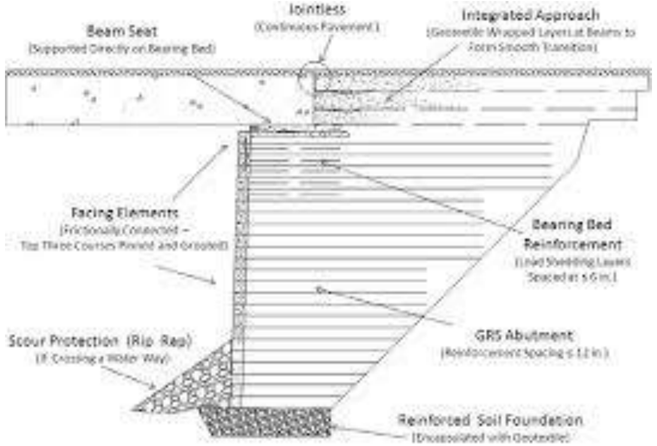




Design

- 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)

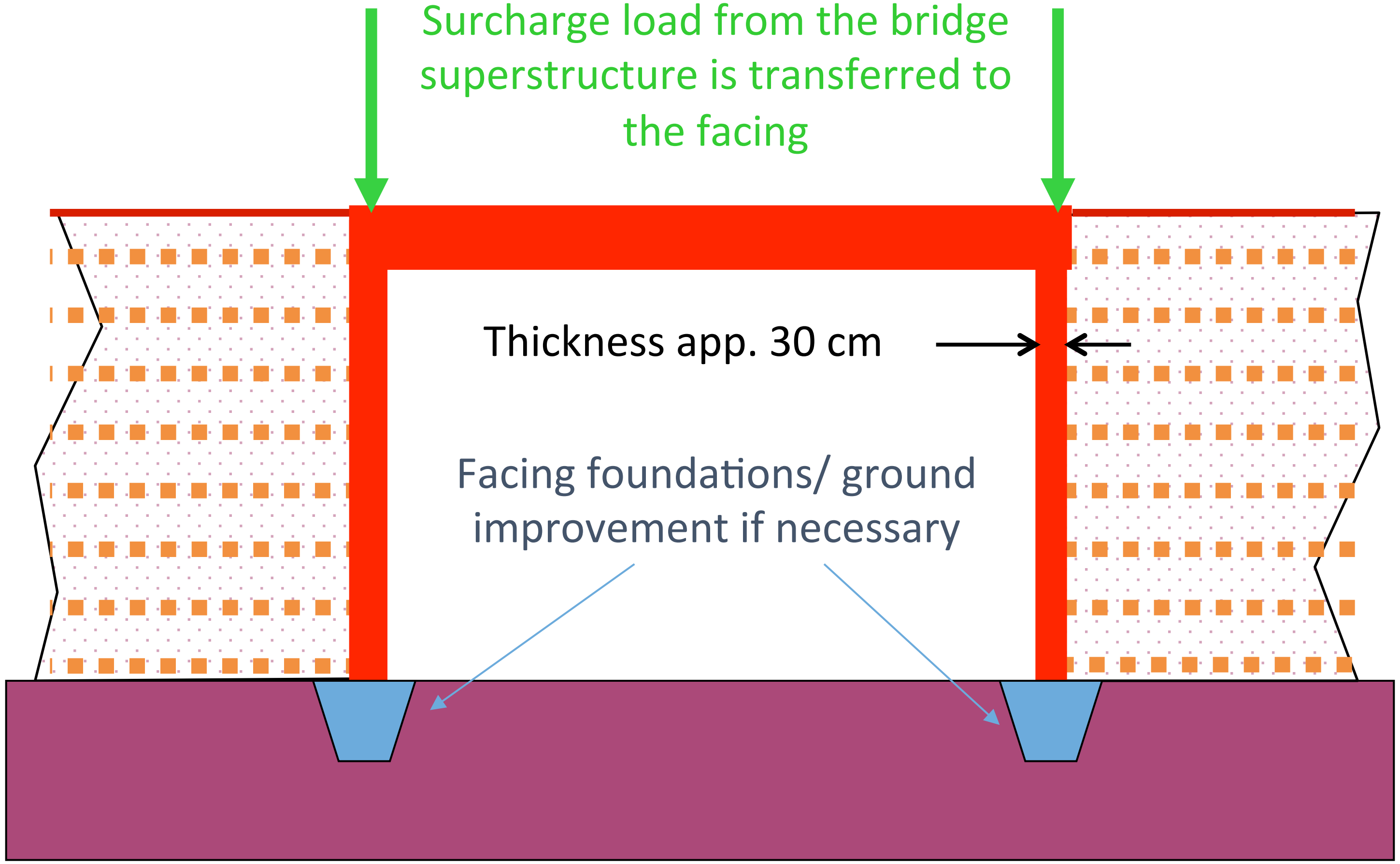




Design

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 !!!!

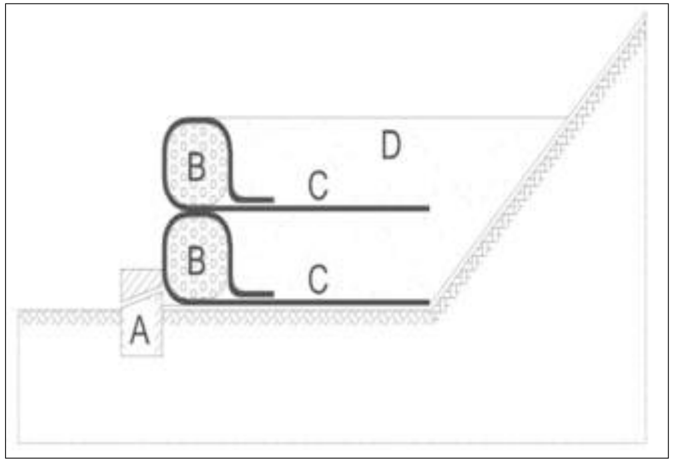


Design

- 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

↓

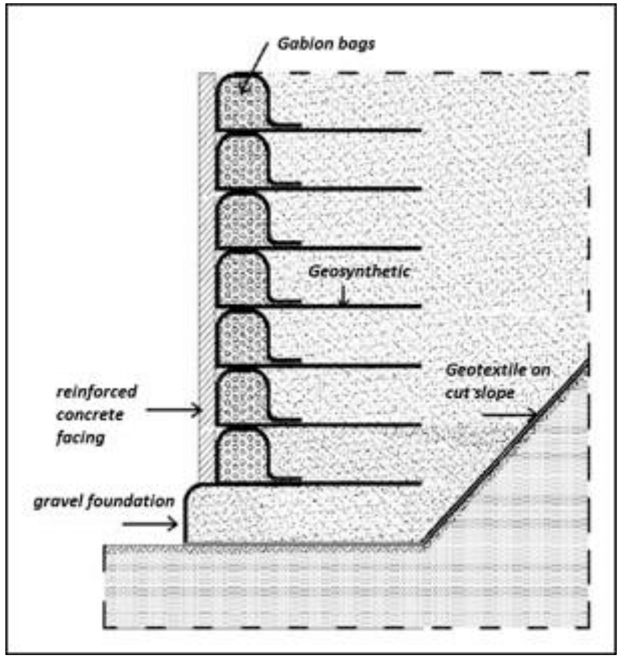
high risk of low quality execution !!!



B – gabions bags

C – geosynthetic layers

D – backfill material

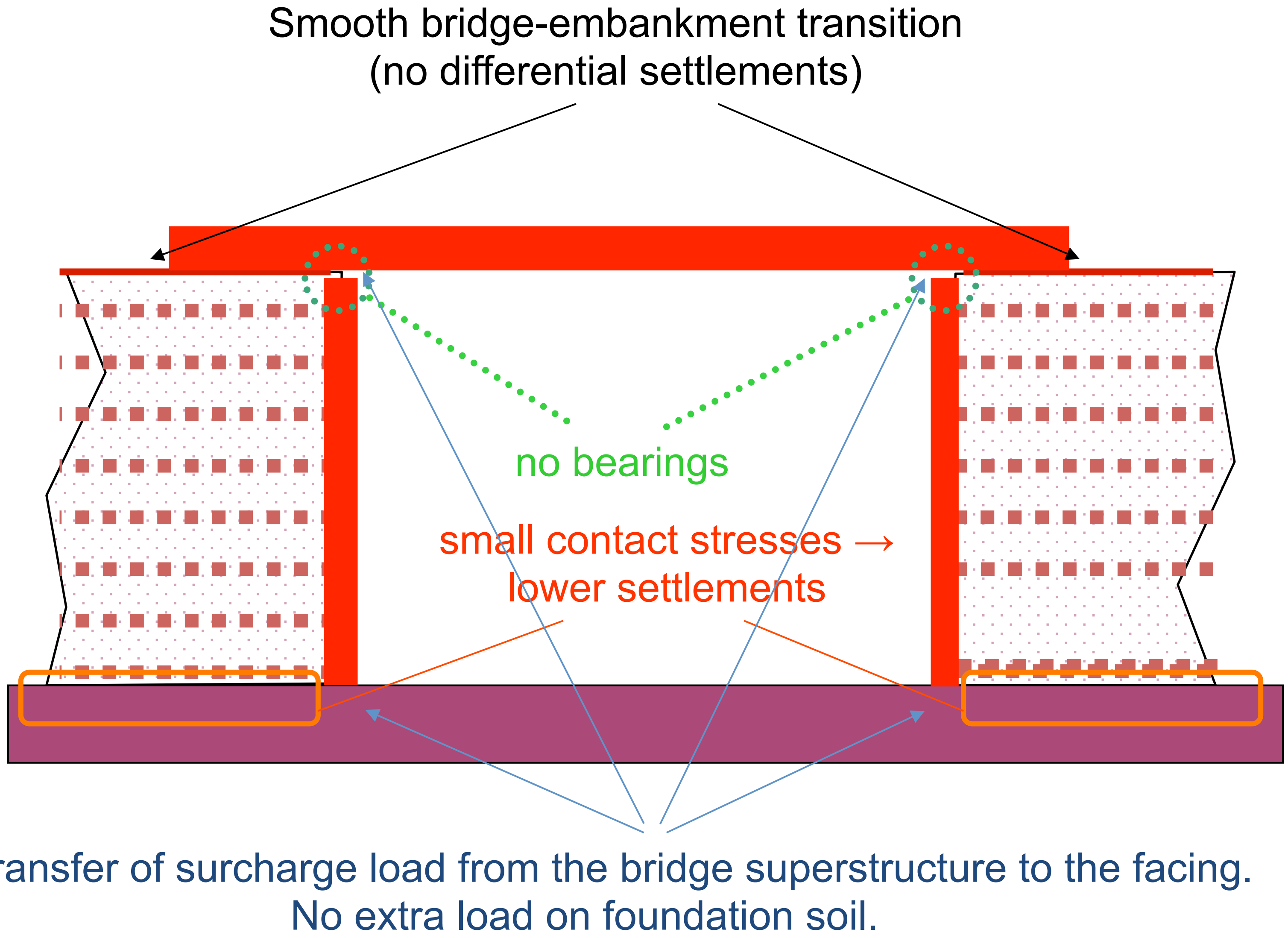


Design

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)

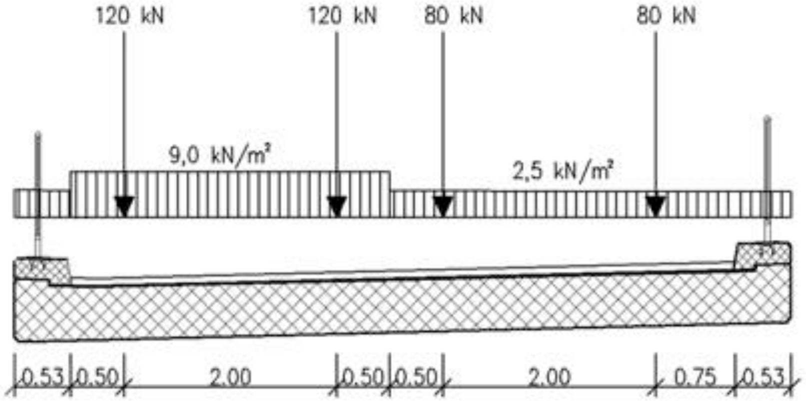
no bearings

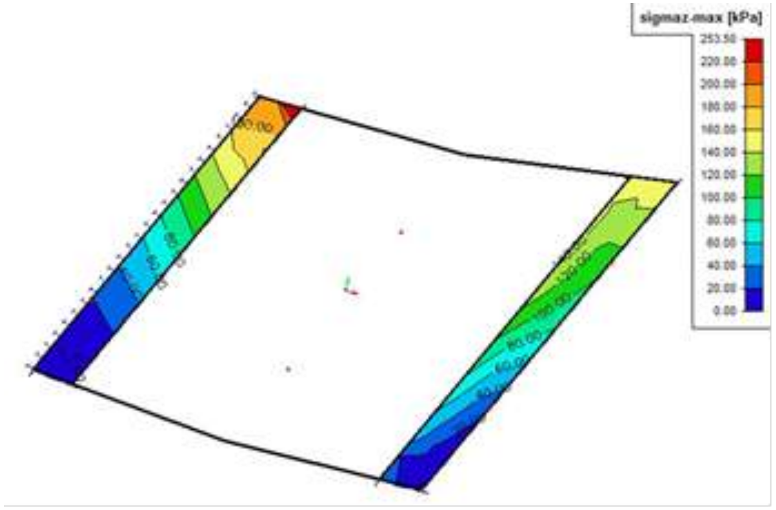
small contact stresses → lower settlements

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

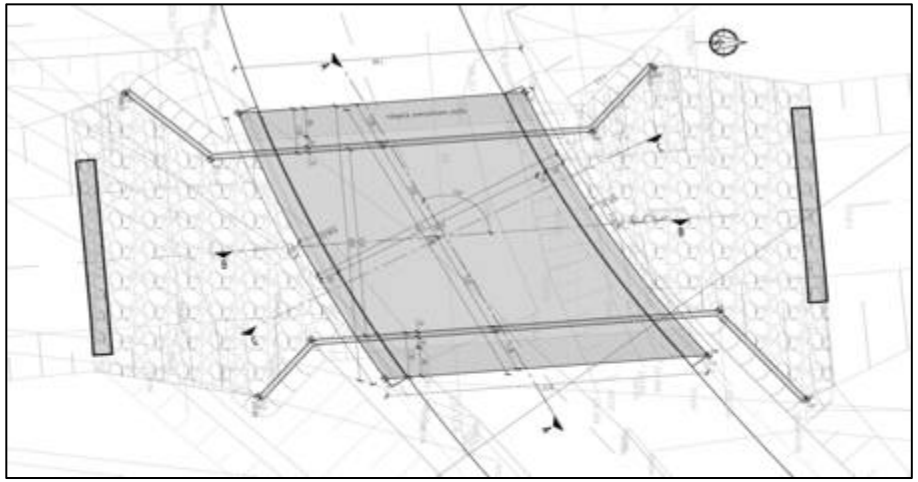
Load scheme

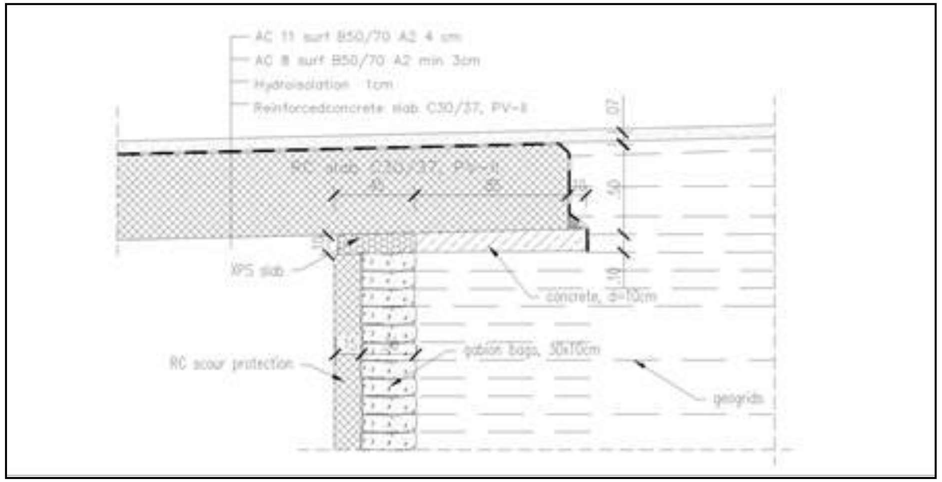
- Dead weight of the structure & traffic loads
- Load model LM1: a pair of tandem axles on each conventional lane, accompanied by a uniform load, EN 1991-2)
- Bridge superstructure is supported directly at the top of the abutment as a simply-supported beam → maximum design vertical pressure 305 kPa (FEM)
- 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

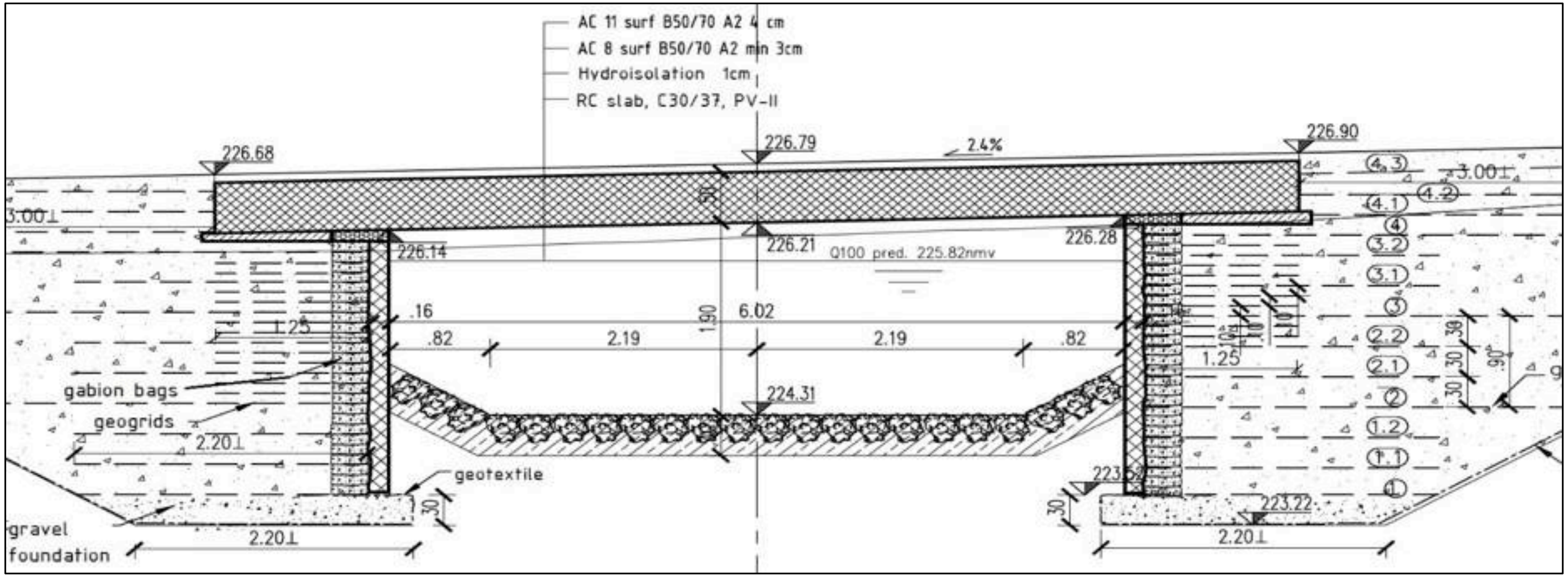




Design




AC 11 surf B50/70 A2 4 cm
AC 8 surf B50/70 A2 min 3cm
Hydroisolation 1cm
Reinforcedconcrete slab C30/37, PV-II
RC slab C30/37, PV-II
07
30
10
XPS slab
concrete, d=10cm
gabion bags, 30x10cm
RC scour protection
geogrids


AC 11 surf B50/70 A2 4 cm
AC 8 surf B50/70 A2 min 3cm
Hydroisolation 1cm
RC slab, C30/37, PV-II
226.68
226.79
2.4%
226.90
3.00⊥
30
226.14
226.21
Q100 pred. 225.82mnv
226.28
.16
6.02
1.90
1.25
.82
2.19
2.19
.82
1.25
224.31
gabion bags
geogrids
2.20⊥
geotextile
223.52
223.22
30
gravel
foundation
2.20⊥
2.20⊥
(4.3)
(4.2)
(4.1)
(4)
(3.2)
(3.1)
(3)
(2.2)
(2.1)
(2)
(1.2)
(1.1)
(1)
30
30
39
90°

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

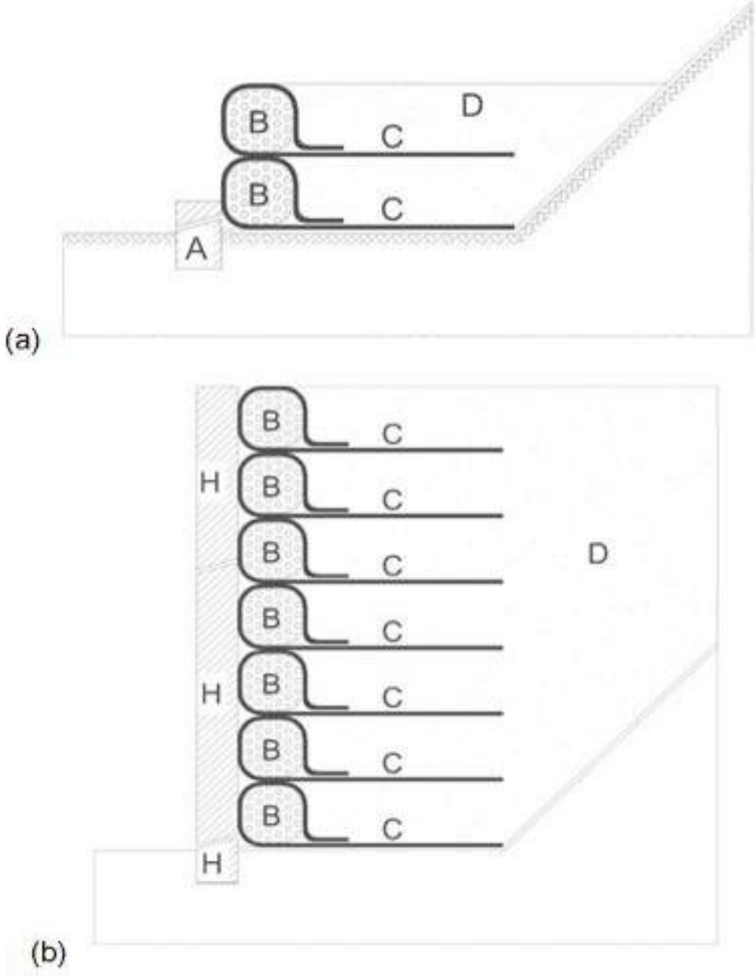
Construction of the gravel foundation, before wrapping the foundation with geosynthetics

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,

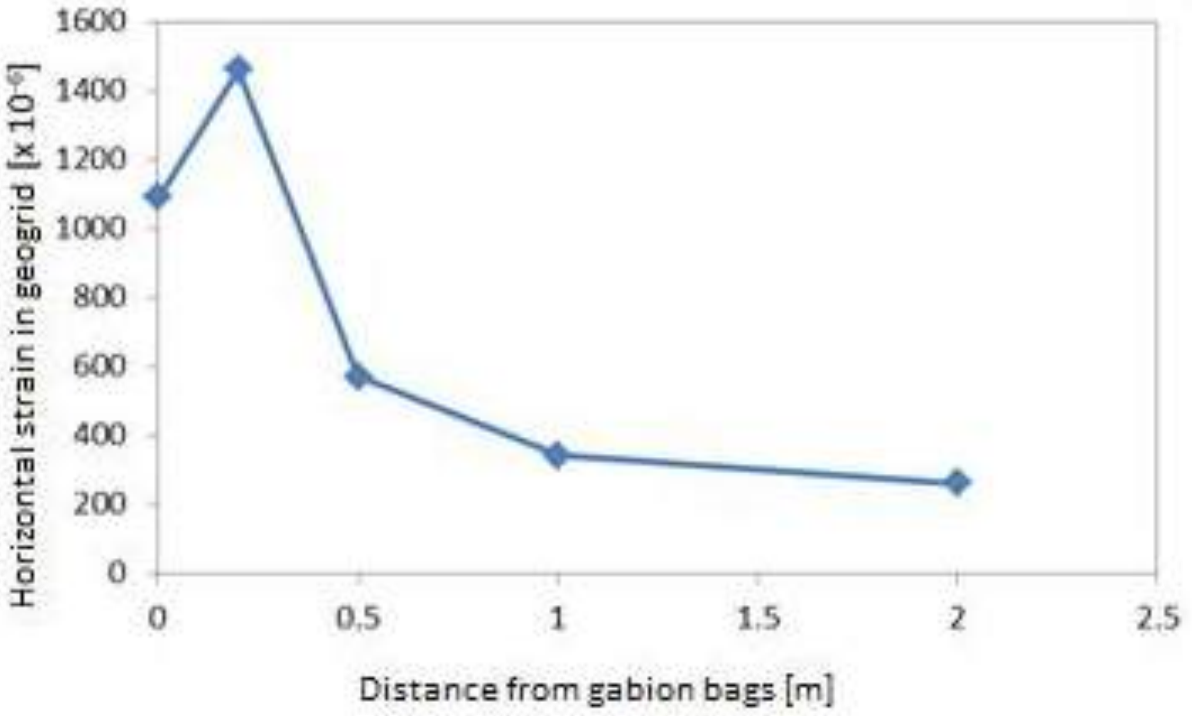
B – the gabion bags,

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)

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

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

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Workshop 1 – Geosynthetics in Transportation Geotechnics

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Modelling Geogrid-reinforced Railway Ballast Using the Discrete Element Method

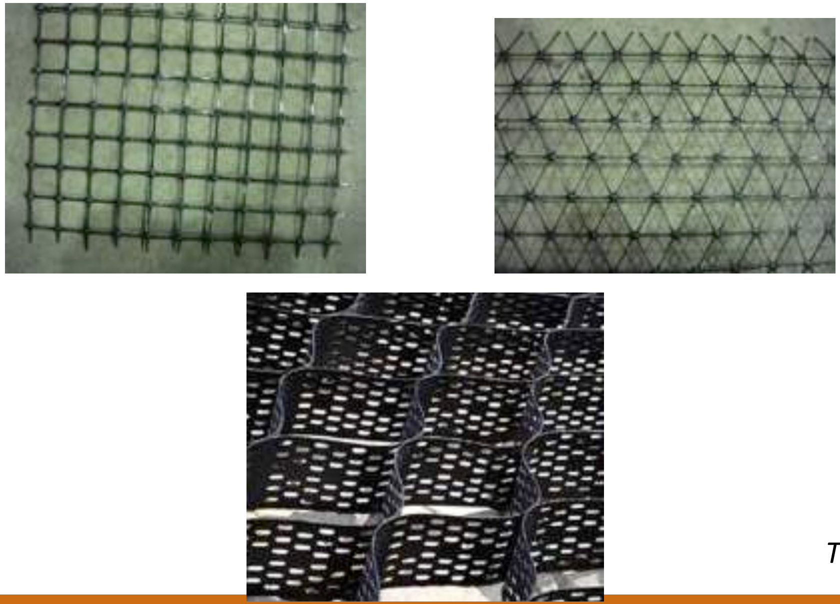
Ngoc Trung Ngo, Buddhima Indraratna, and Cholachat Rujikiatkamjorn

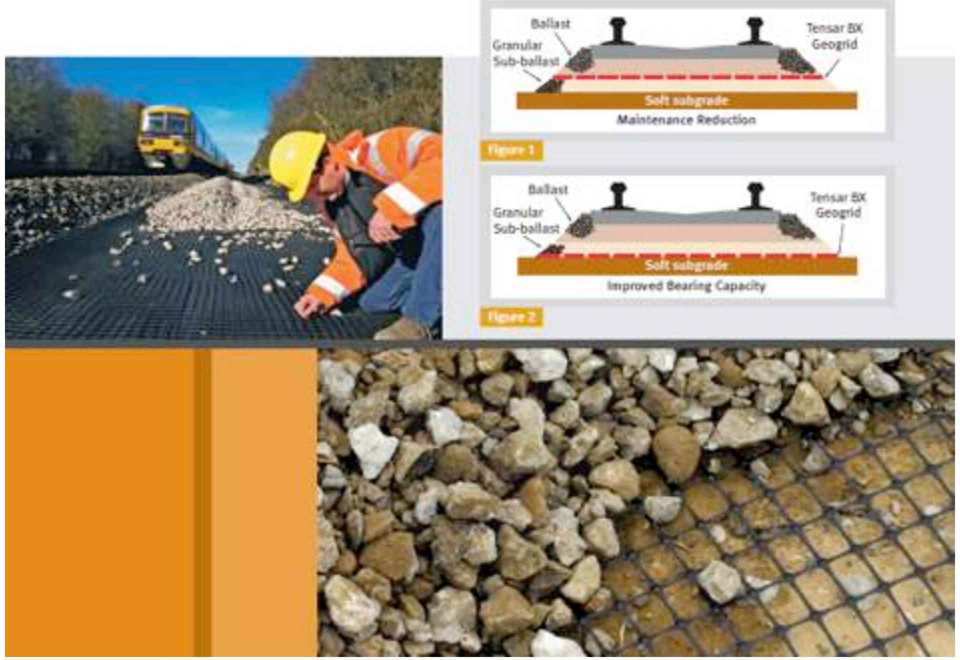
Centre for Geomachanics and Railway Engineering
University of Wollongong, Australia

PROBLEMS IN RAIL TRACK SUBSTRUCTURE

THE USE OF GEOSYNTHETICS IN RAIL TRACKS

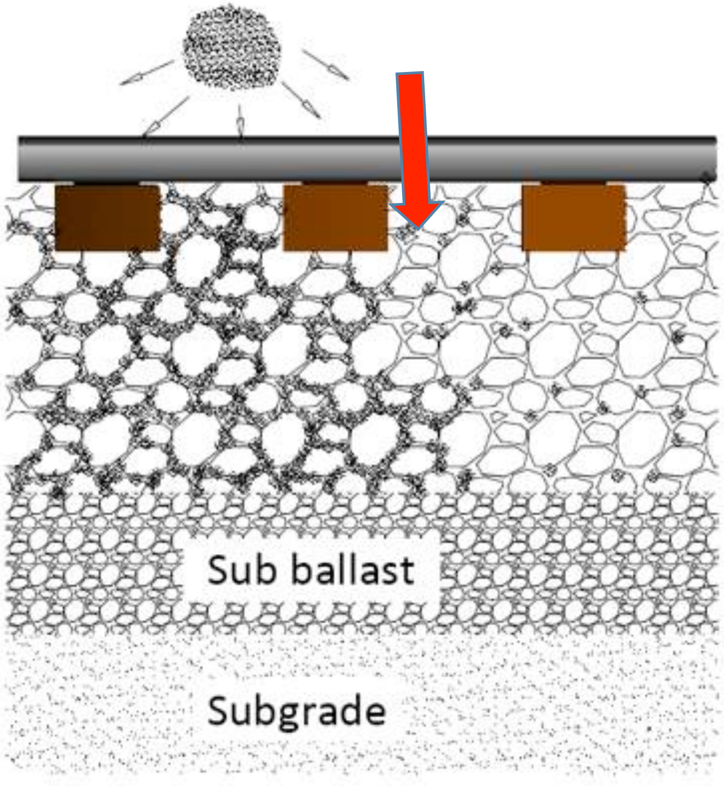
- Geogrids reinforce and confine ballast, resulting in a reduced settlement and decreased lateral movement of ballast
- Lack of availability of a comprehensive computational model to study the interaction of ballast aggregates with geogrids (i.e. interlocking / confinement effects)



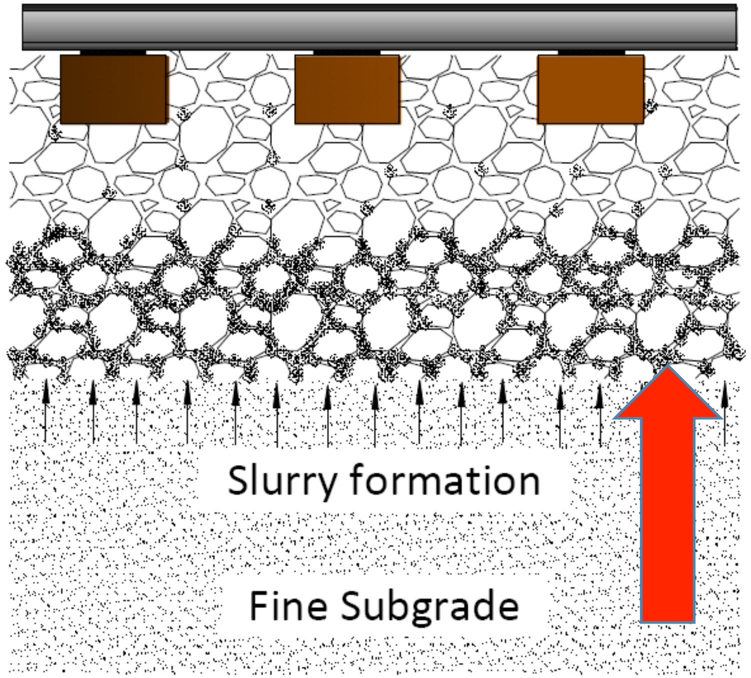


Tensar, 2012

Role of Ballast Fouling on Track Performance



Coal



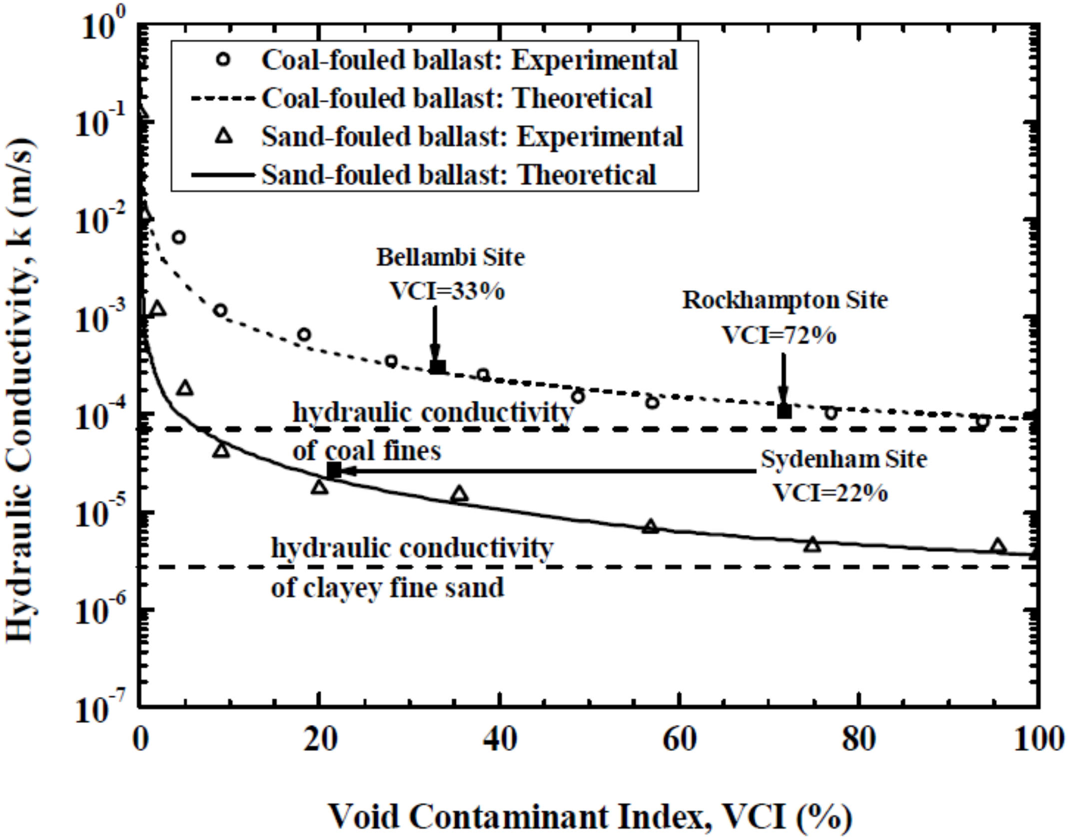
Clay

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
$G_{s\text{-}b}$ = Specific gravity of clean ballast
$G_{s\text{-}f}$ = Specific gravity of fouling material
M_b = Dry mass of clean ballast
M_f = Dry mass of fouling material

Impeded Track Drainage due to Ballast Contamination

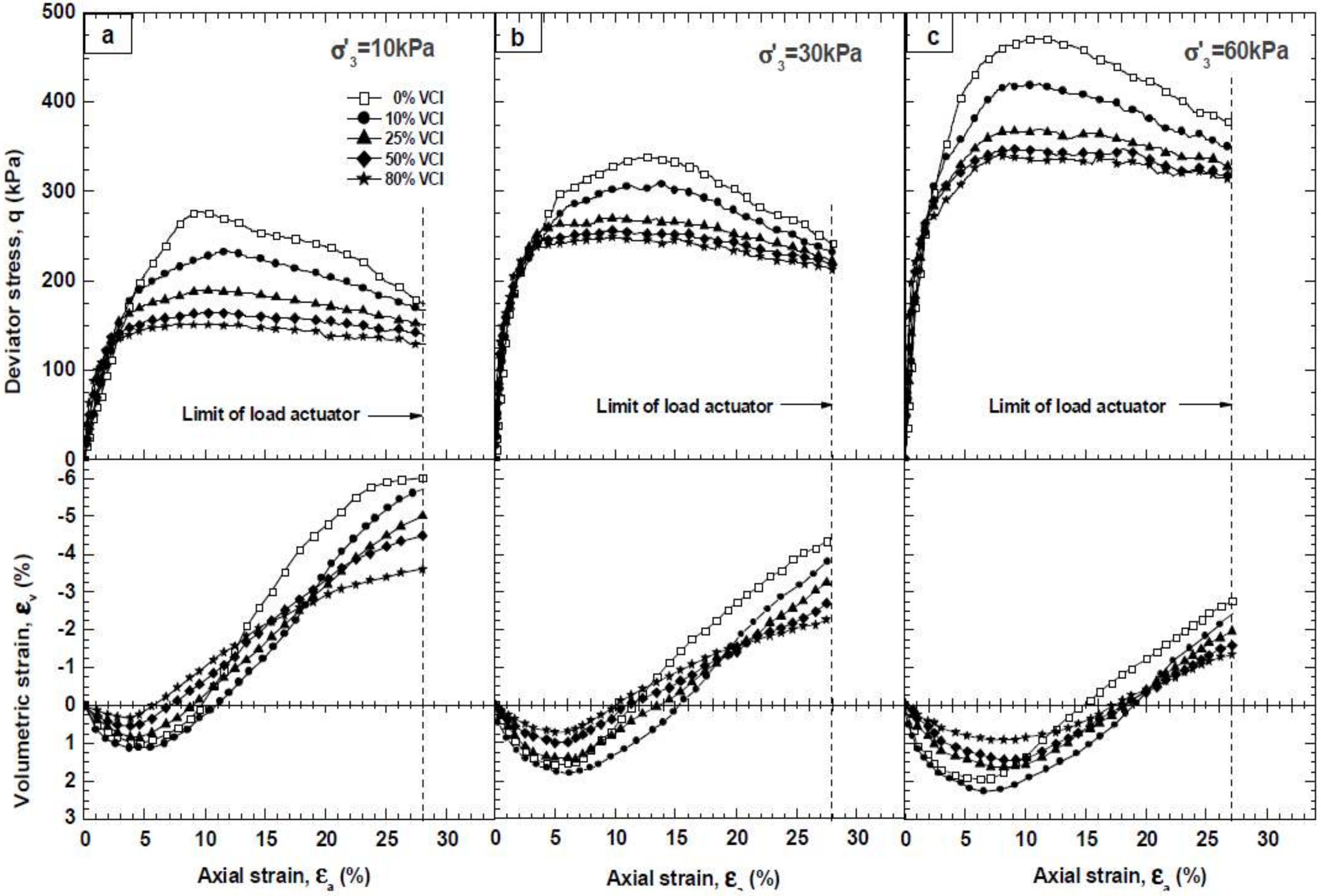


Variation of hydraulic conductivity with Void Contaminant Index

Hydraulic Conductivity (k) of fouled ballast

$$k = \frac{k_b \times k_f}{k_f + {}^{VCI}\!/_{100} \times (k_b - k_f)}$$

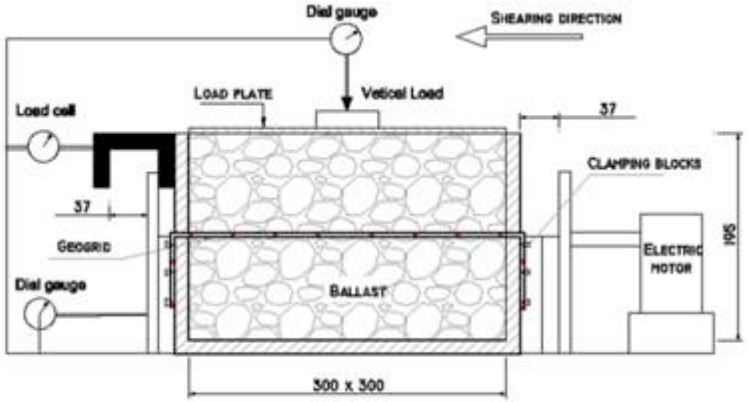
Tennakoon, Indraratna, Cholachat, Nimbalkar and Neville (2012) ASTM Geotechnical Testing Journal, 35(4): 1-12



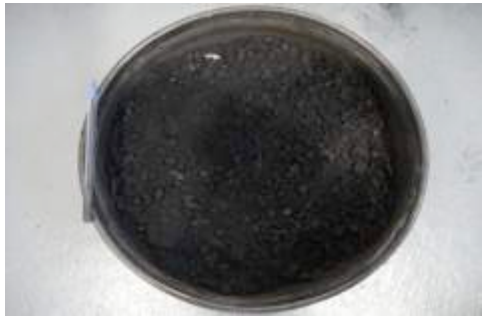
Stress-strain behaviour of clean and fouled ballast during drained triaxial tests at 3 confining pressures (Indraratna et al. 2012)

LABORATORY STUDY OF GEOGRID-REINFORCED BALLAST

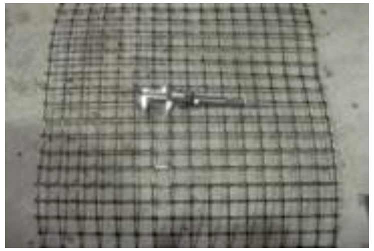
Large-Scale direct shear box
Dimension: 300x300x200mm



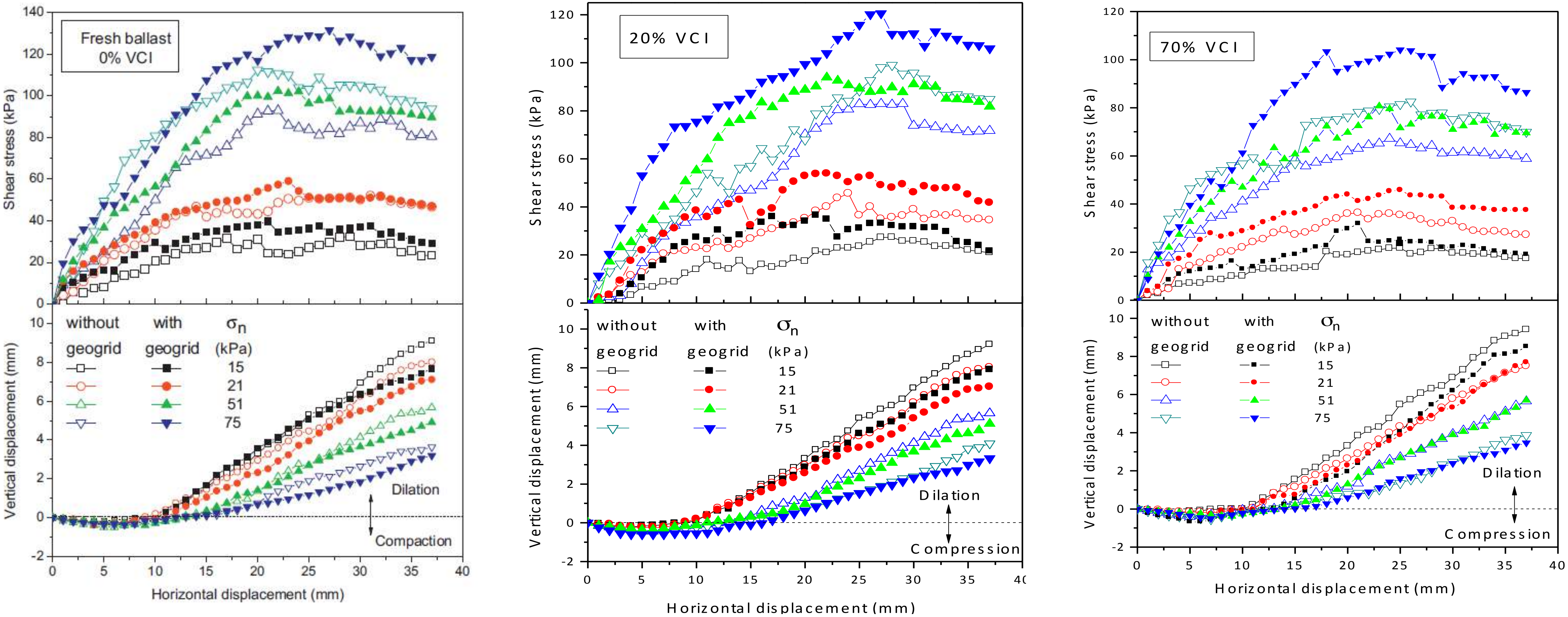
Ballast collected at Bombo Quarry, Wollongong



Coal fines



Biaxial Geogrid
Aperture size = 40mm



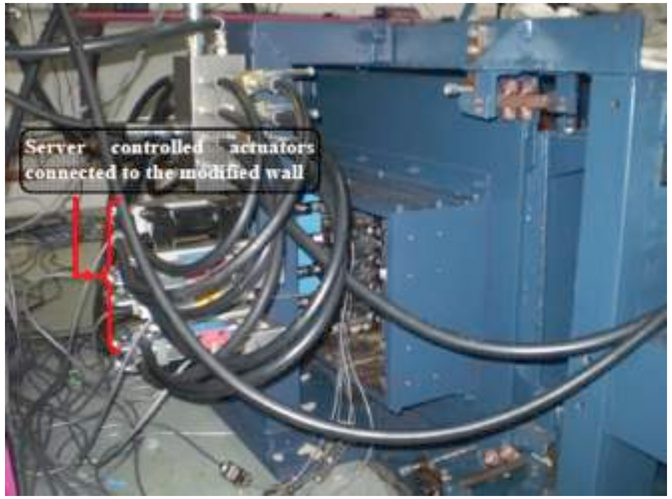
Shear stress-strain behaviour of fresh and fouled ballast with and without geogrid inclusion (Indraratna et al. 2011)

CYCLIC LOADING TESTS FOR GEOGRID-REINFORCED BALLAST

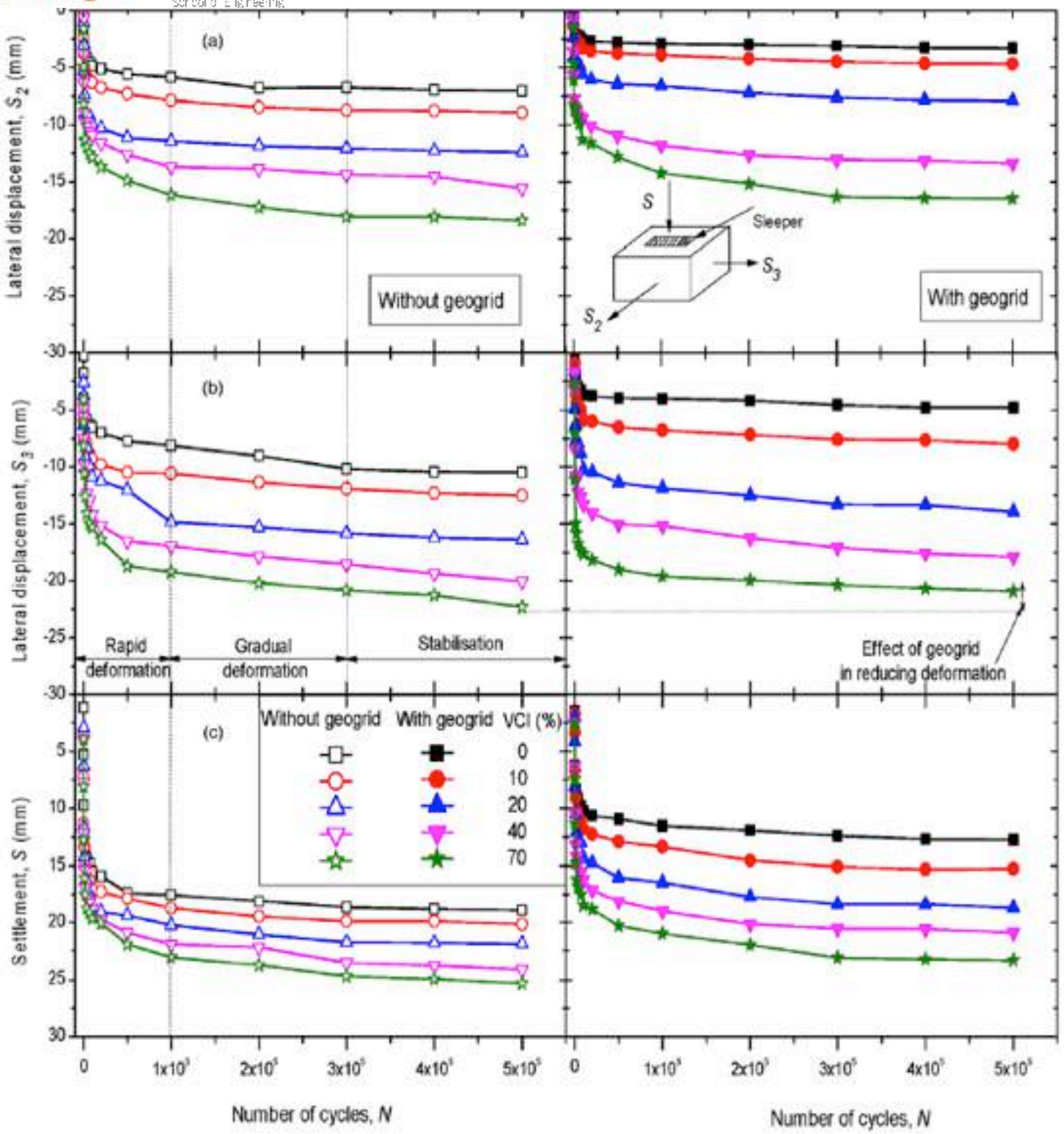
Cubical Triaxial Apparatus to Simulate a Track Section (Specimen: 800x600x600 mm)

Sample for testing

Placement of geogrid in the ballast layer

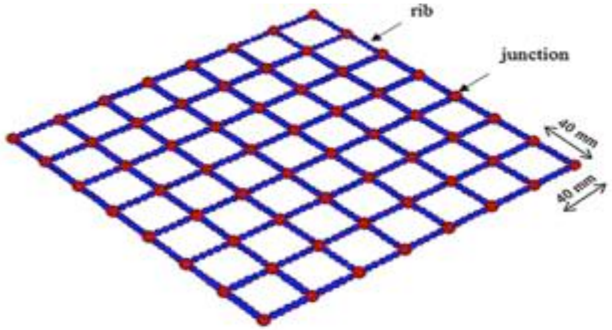


Applying lateral confinement

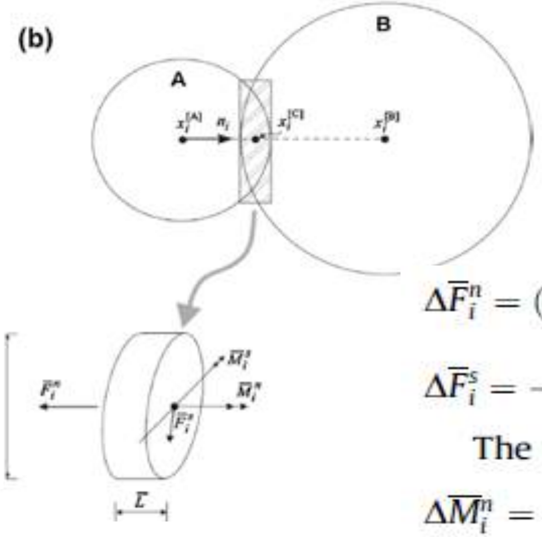


Variations in the deformation of fresh and fouled ballast with and without geogrid with varying VCIs *(Indraratna et al. 2013)*

Discrete Element Modelling (DEM) of Geogrid in Tracks



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



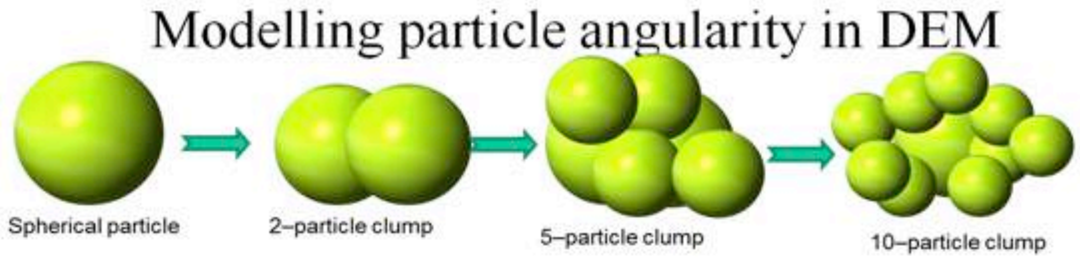
$$\Delta \bar{F}_i^n = (-\bar{k}_n A \Delta U_i^n) n_i$$

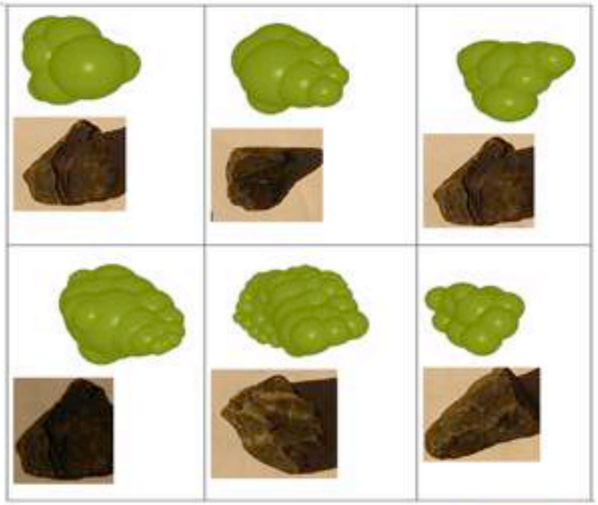
$$\Delta \bar{F}_i^s = -\bar{k}_s A \Delta U_i^s$$

The elastic moment-increments are calculated by:

$$\Delta \bar{M}_i^n = (-\bar{k}_s J \Delta \theta_i^n) n_i$$

$$\Delta \bar{M}_i^s = -\bar{k}_n I \Delta \theta_i^s$$

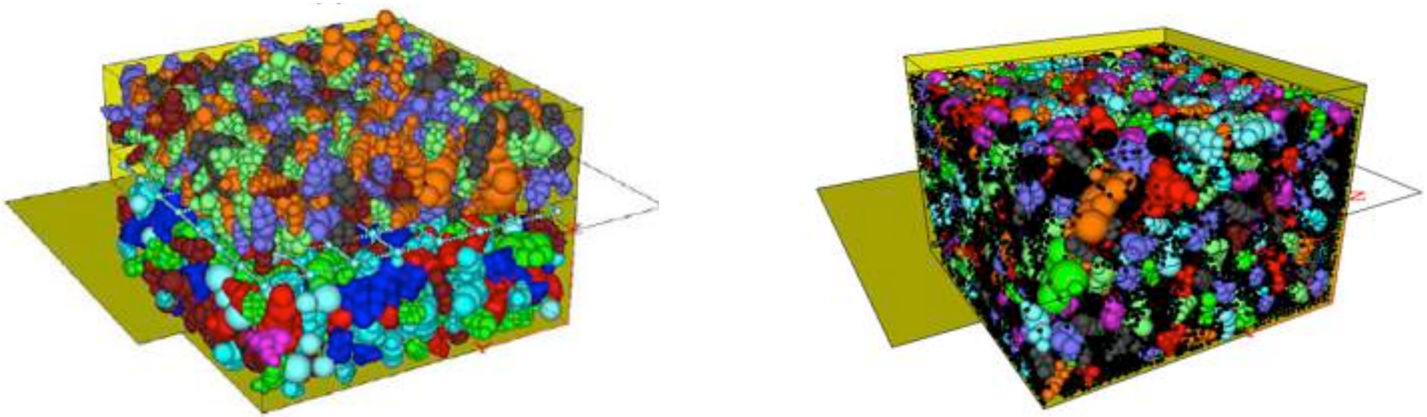




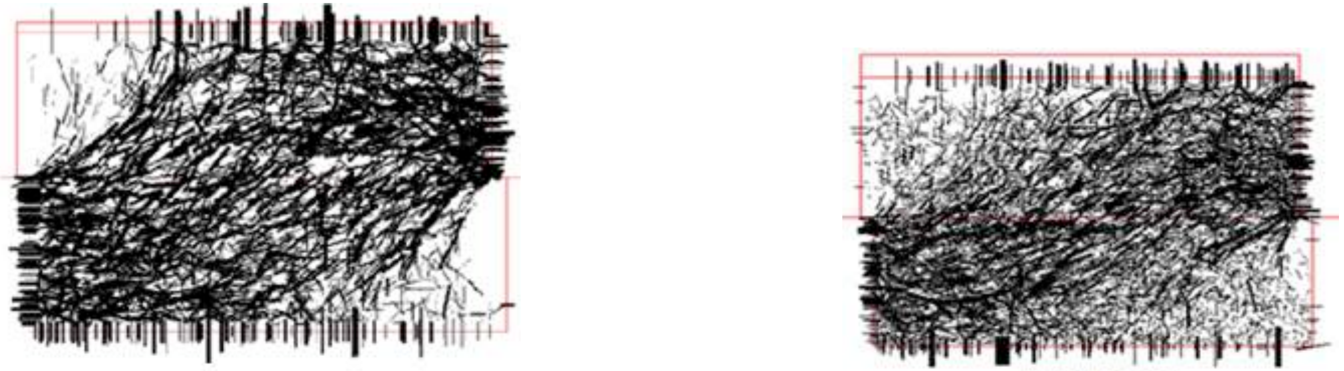
(Indraratna et al. 2014 – ASCE)

$$\sigma_{max} = \frac{-\bar{F}_i^n}{A} + \frac{|\bar{M}_i^s|}{I}\bar{R}$$

$$\tau_{max} = \frac{|\bar{F}_i^s|}{A} + \frac{|\bar{M}_i^n|}{J}\bar{R}$$



DEM model for fresh and fouled ballast (VCI=40%)



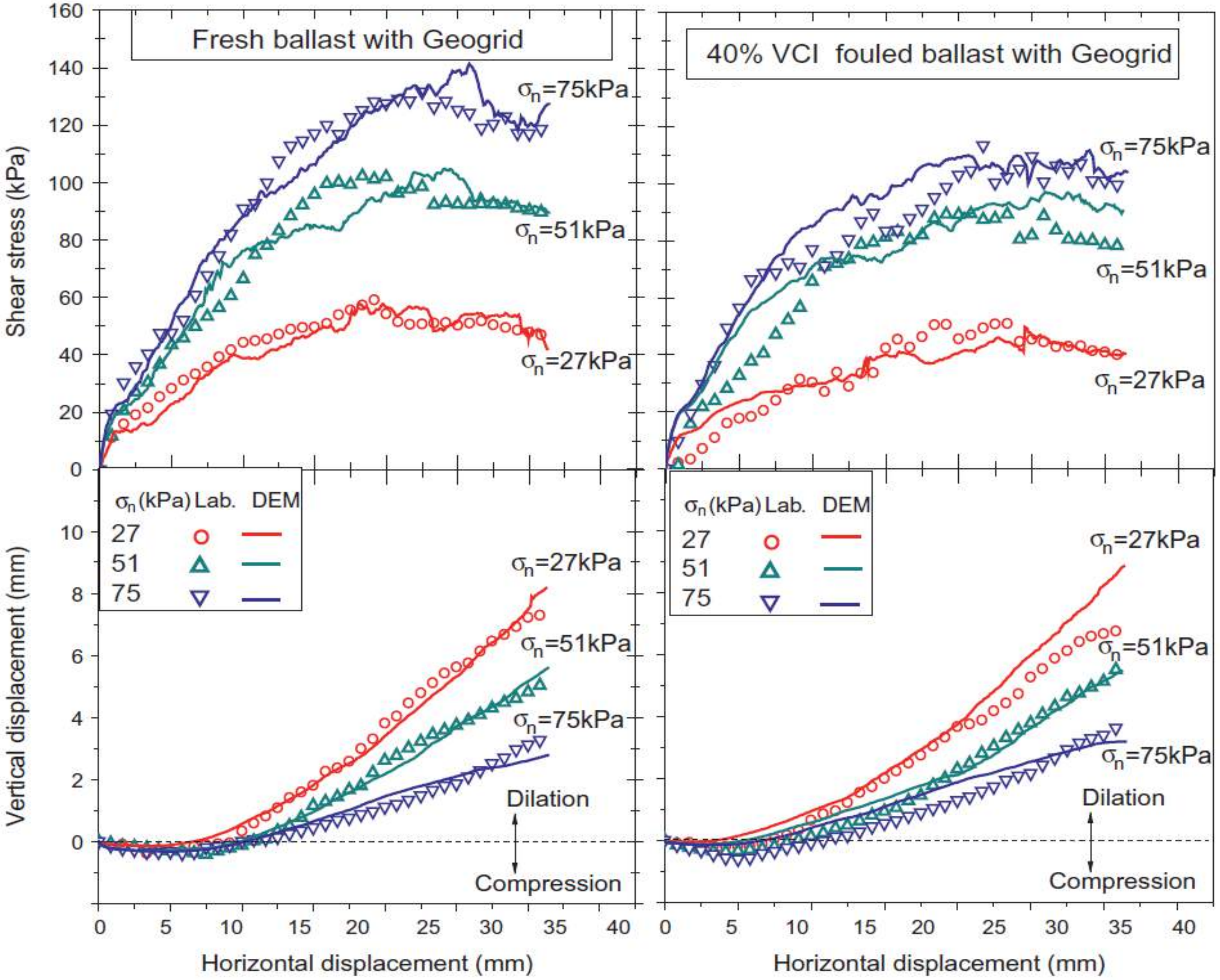
(a) VCI=0%
No. of contacts: 95,585
Maximum contact force: 1150(N)

(b) VCI=40%
No. of contacts: 519,818
Maximum contact force: 560(N)

Contact force distributions of fresh and 40%VCI-fouled (*modified after Ngo et al. 2014*)

- 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

DEM Modelling Geogrid-reinforced Ballast under Shearing Loads



Comparison of shear stress and displacements for DEM simulation of reinforced ballast

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
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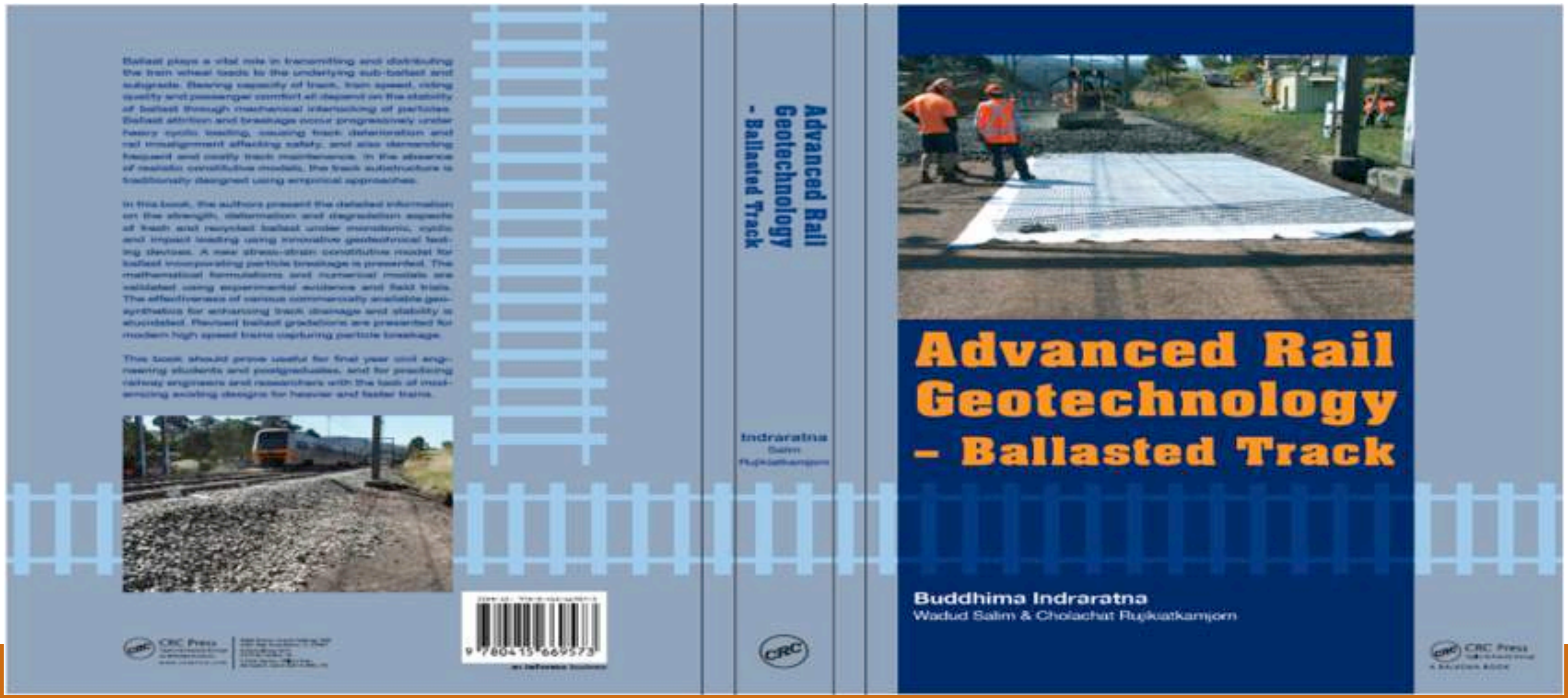
- Industry Organisations: RailCorp (NSW), ARTC, QLD Rail, ARUP, Coffey Geotechnics, Douglas Partners. Roads & Traffic Authority, Queensland Department of Main Roads, Port of Brisbane Corporation, Port Kembla Port Corporation


GROUND IMPROVEMENT CASE HISTORIES
EMBANKMENTS WITH SPECIAL REFERENCE TO CONSOLIDATION AND OTHER PHYSICAL METHODS
Buddhima Indraratna, Jian Chu, Cholachat Rujikiatkamjorn


GROUND IMPROVEMENT CASE HISTORIES
COMPACTION, GROUTING, AND GEOSYNTHETICS
Buddhima Indraratna, Jian Chu, Cholachat Rujikiatkamjorn


Advanced Rail Geotechnology – Ballasted Track
Buddhima Indraratna
Wadud Salim & Cholachat Rujikiatkamjorn


GROUND IMPROVEMENT CASE HISTORIES
CHEMICAL, ELECTROKINETIC, THERMAL AND BIOENGINEERING
Buddhima Indraratna, Jian Chu, Cholachat Rujikiatkamjorn

Thank You!

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Workshop 1 – Geosynthetics in Transportation Geotechnics

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

Sinniah K. Navaratnarajah[1], Buddhima Indraratna[1], and Tim Neville[2]

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

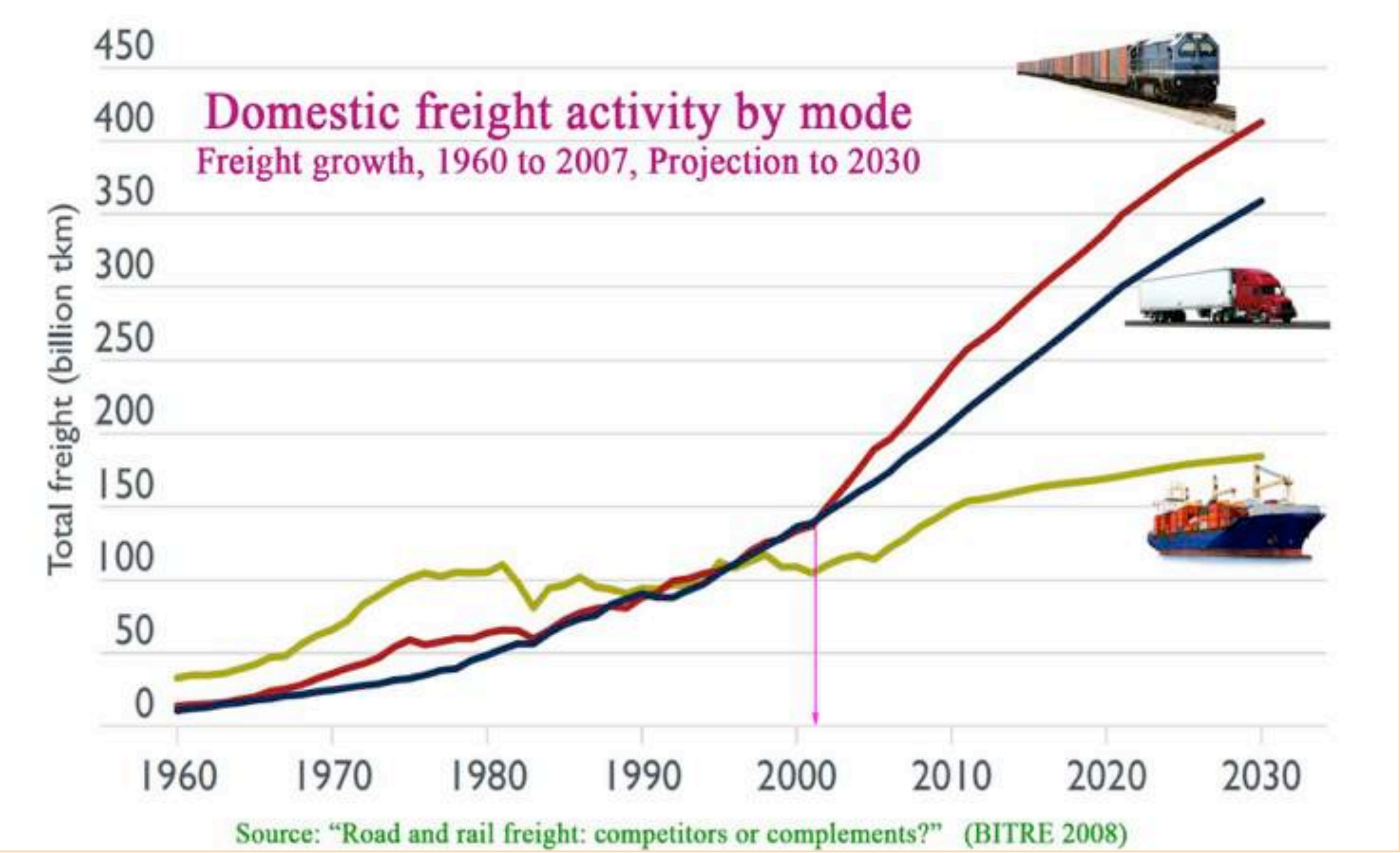
1. Instrumented track at Bulli, NSW, Australia
   Fresh and recycled ballast stabilized with geocomposite

2. Instrumented track at Singleton, NSW, Australia
   Ballast stabilized with various geogrids, geocomposite and shock mats

- Conclusions

Introduction

Demand for freight and passenger transport has increased in the last decades.


Domestic freight activity by mode
Freight growth, 1960 to 2007, Projection to 2030

Source: "Road and rail freight: competitors or complements?" (BITRE 2008)

- ➢ **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.x600 mm high)

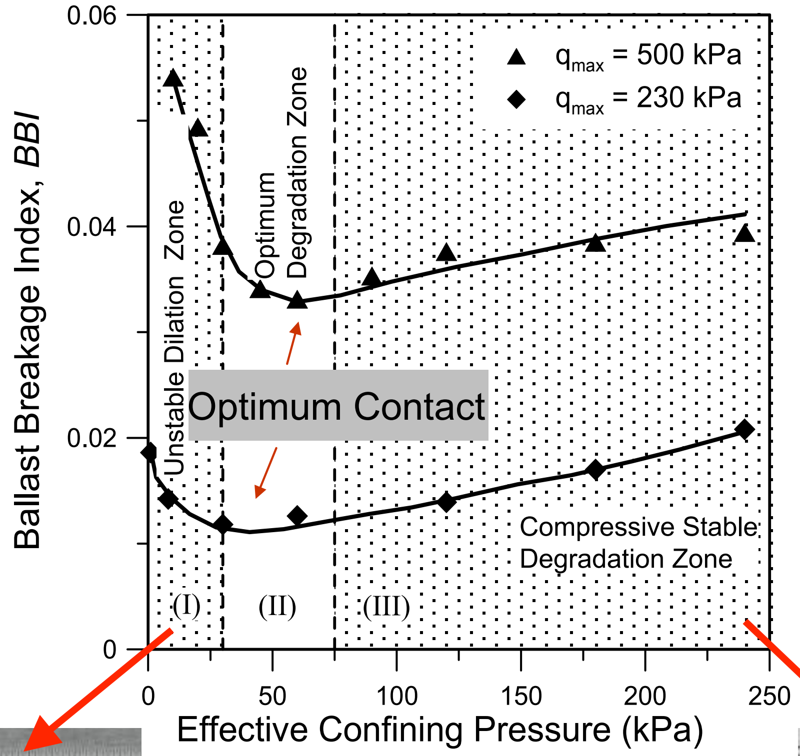
Capacity:
100 kN dynamic actuator load
Loading frequency up to 60 Hz

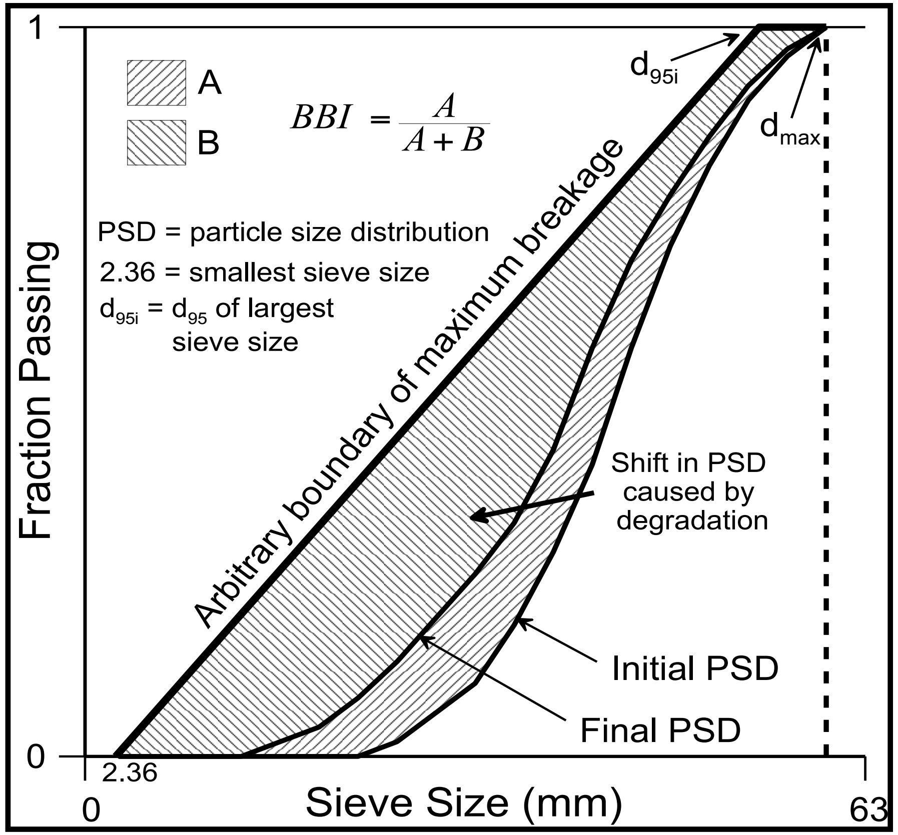
Prismoidal Triaxial Rig to Simulate a Track Section
(Specimen: 800x600x600 mm)

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)



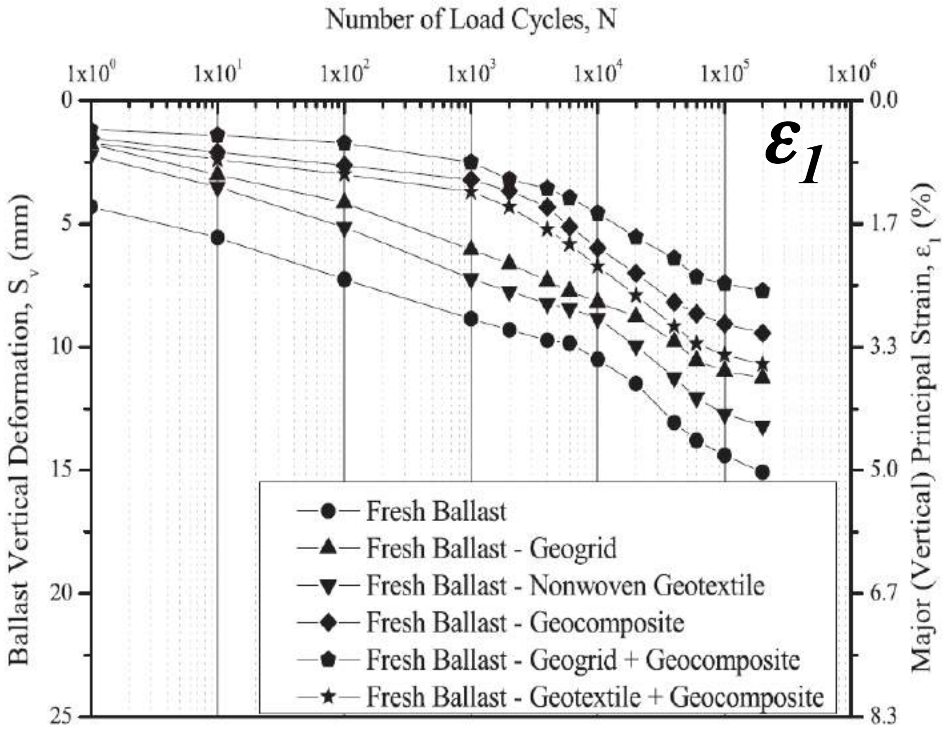


Ballast Breakage Index (BBI)

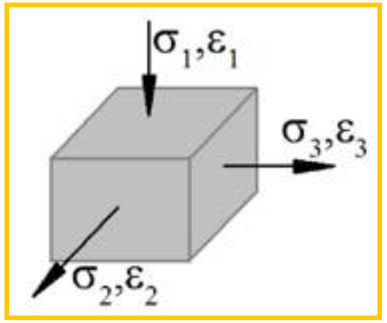
Indraratna, Lackenby and Christie (2005)

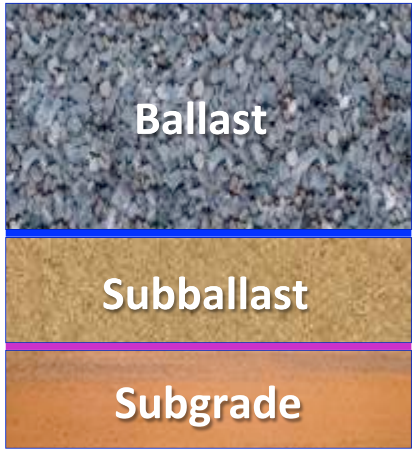
***Geotechnique, ICE, UK*, Vol. 55(4), 325-328.**

Stress-Strain response of railway ballast stabilized with Geosynthetics (Large-Scale Cyclic Loading)



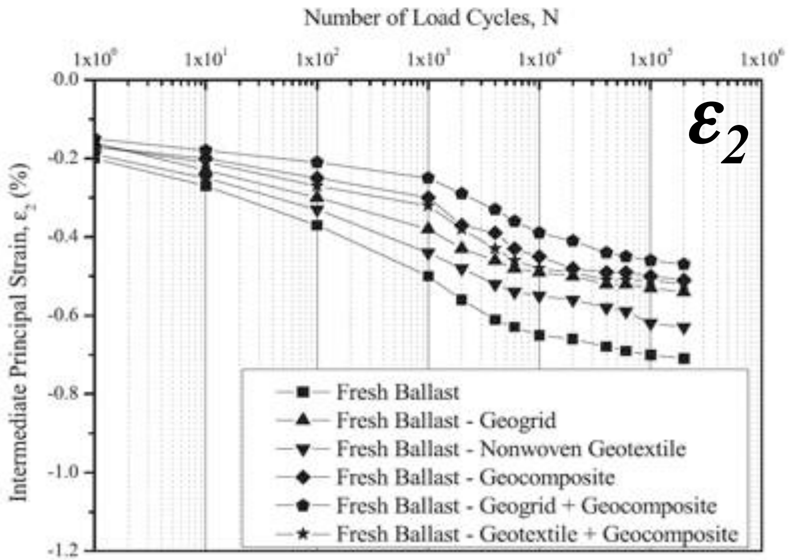
Indraratna and Nimbalkar (2013), *J. of Geotech & Geoenv Eng., ASCE*, Vol. 139(5), 684-700.

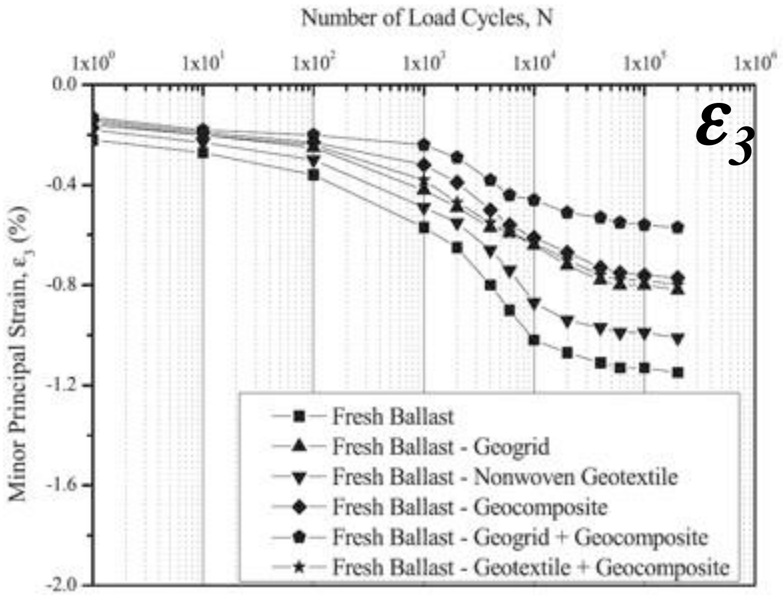




Geogrid/
Geotextile/
Geocomposite

Geocomposite





Effect of High Impact Loads and Track Degradation

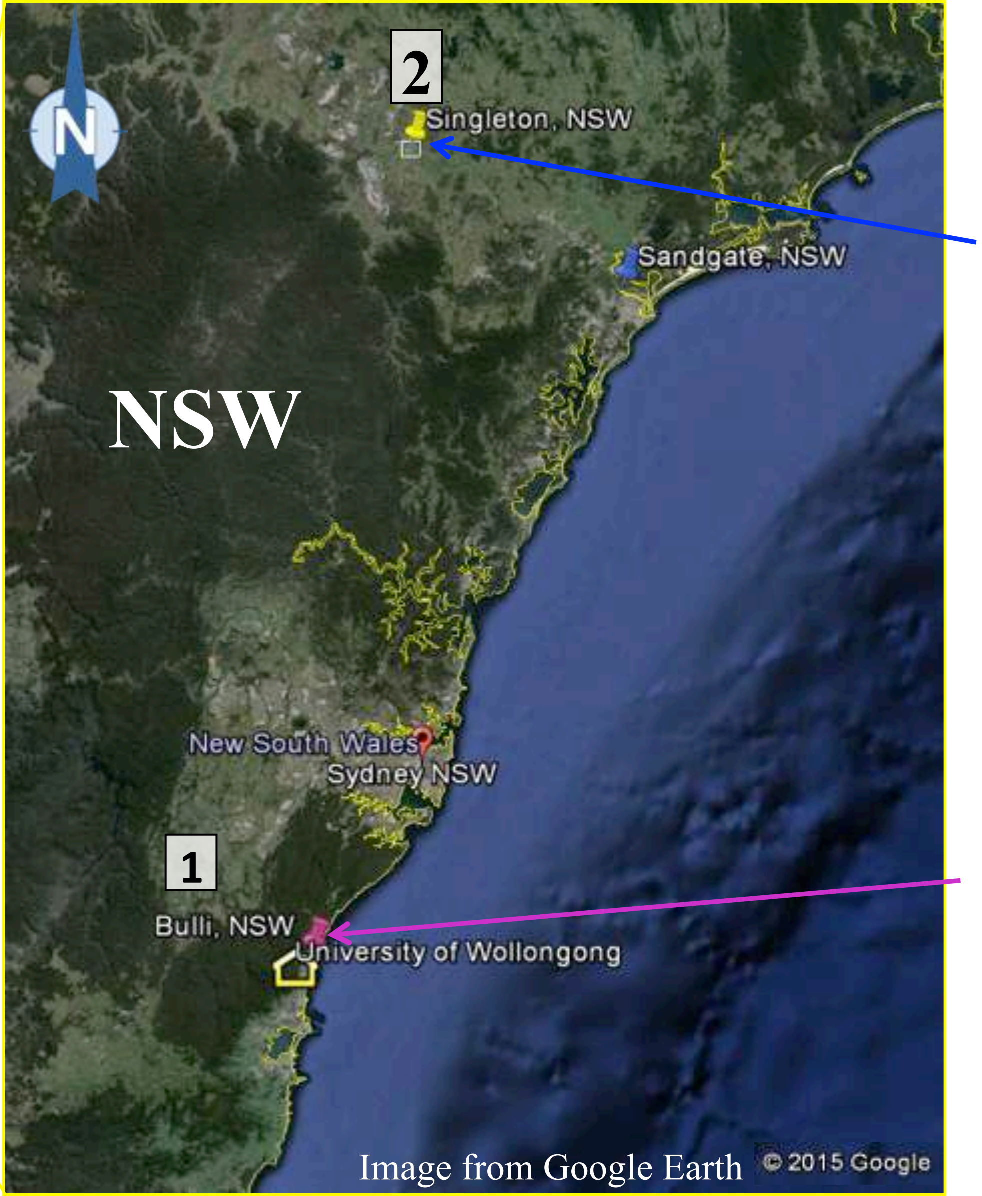
High capacity drop weight Impact test Apparatus

Subgrade type	Location of shock mat	Ballast Breakage Index (BBI)
Stiff	**Without shock mat**	**0.170**
	Shock Mat above ballast	0.145 (↓ 15%)
	Shock Mat below ballast	0.129 (↓ 24%)
	Shock Mat above & below ballast	0.091 (↓ 47%)
Soft	**Without shock mat**	**0.080**
	Shock Mat above ballast	0.055 (↓ 31%)
	Shock Mat below ballast	0.056 (↓ 30%)
	Shock Mat above & below ballast	0.028 (↓ 65%)

Shock Mat

Nimbalkar, Indraratna, Dash & Christie (2012). *JGGE, ASCE*, 138(3): 281-294

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

Case Study: 1. Instrumented track at Bulli

Ballast stabilized with geocomposite

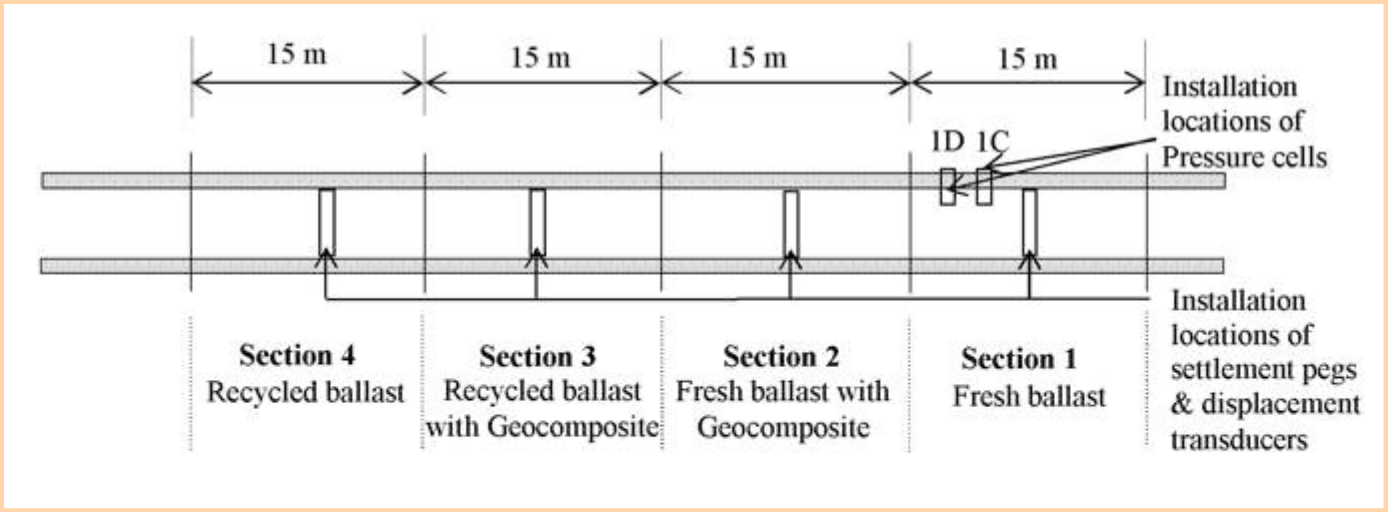
Instrumented Sections

Section 1: Fresh ballast

Section 2: Fresh ballast with geocomposite

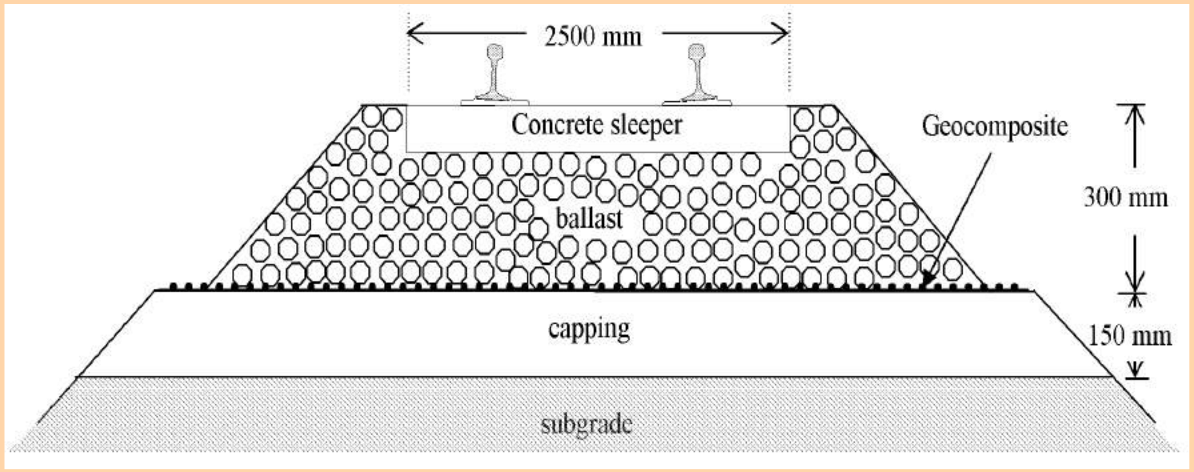
Section 3: Recycled ballast with geocomposite

Section 4: Recycled ballast

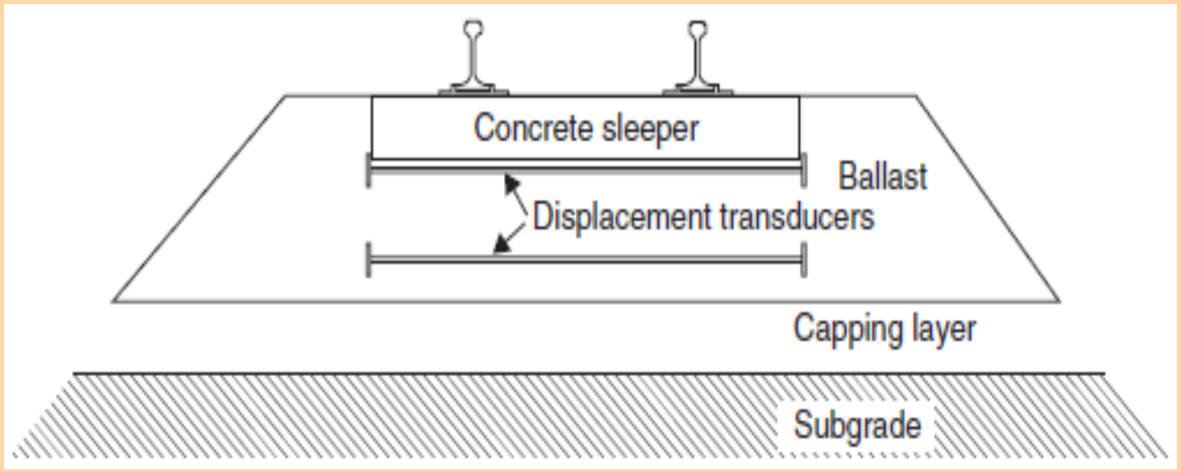


Details of instrumented track

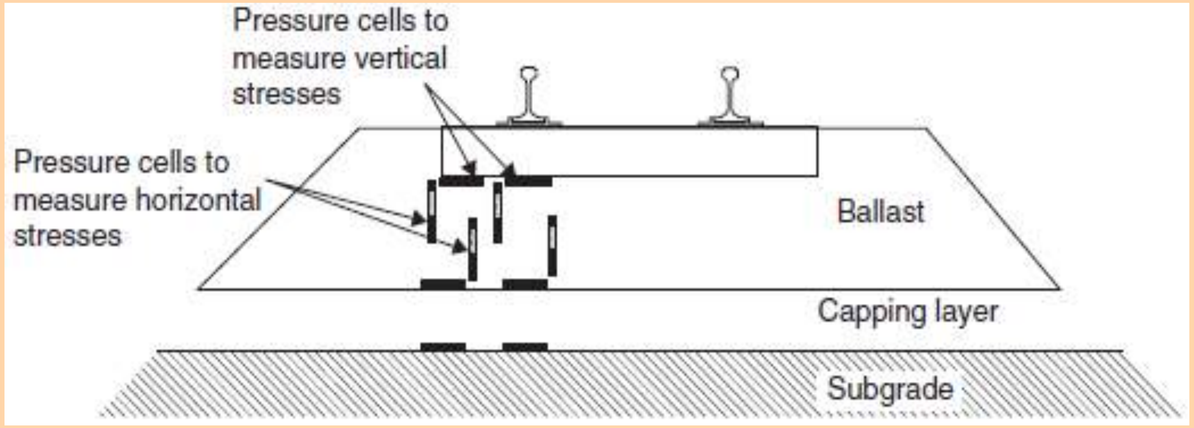
Field Instrumentation – Bulli, NSW



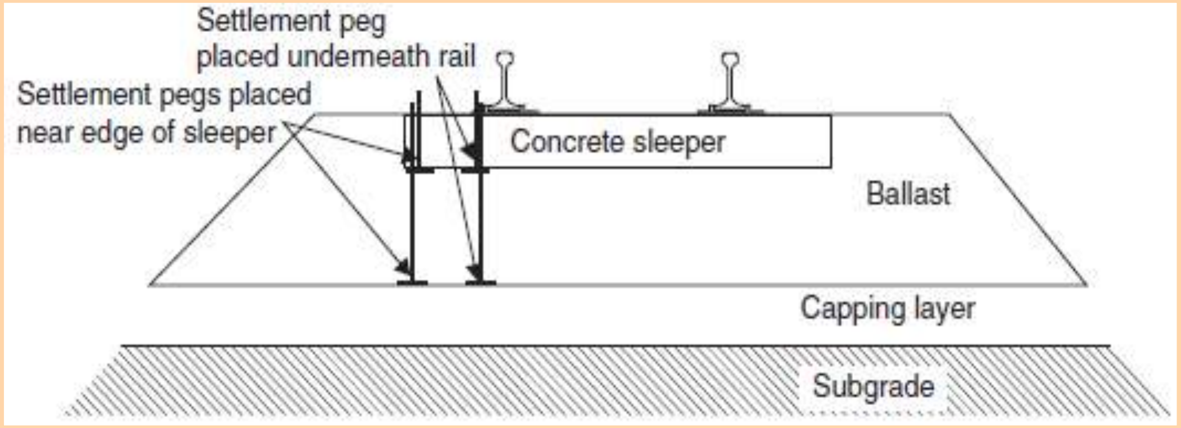
Ballasted track bed with geocomposite layer



Installation lateral displacement transducers

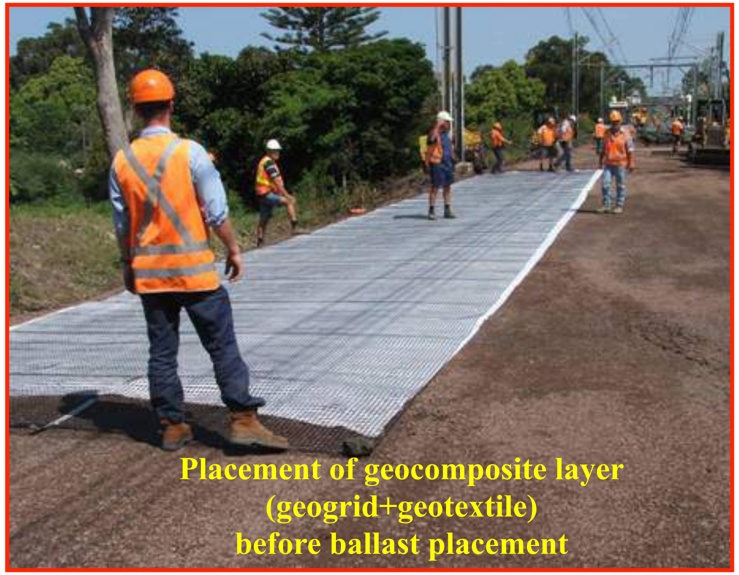


Installation of vertical and horizontal pressure cells



Installation of vertical settlement pegs

Field Trial on Instrumented Track in Bulli, NSW



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

Vertical and horizontal pressure Cells

Ballast placement over the geocomposite

Settlement pegs installed at ballast-capping interface

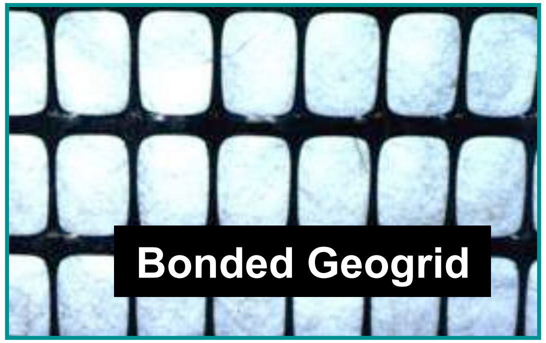
Settlement pegs installed at sleeper-ballast interface

Displacement transducers installed at sleeper-ballast interface

Material Specification

Material	Maximum particle size (mm) d_{max}	Minimum particle size (mm) d_{min}	Median particle size (mm) d_{50}	*Coefficient of uniformity* C_u	*Coefficient of curvature* C_c
Fresh Ballast	75	19	35	1.5	1
Recycled Ballast	75	9.5	38	1.8	1
capping	19	0.05	0.26	5	1.2


Geotextile


Bonded Geogrid

Fresh Ballast

Bombo Quarry, Wollongong

Recycled Ballast

from Chullora Quarry, Sydney

Geocomposite (Geogrid + Geotextile)

Biaxial geogrid		Nonwoven geotextile	
Tensile strength, T_u (kN/m)	30 × 30	Thickness, t (mm)	2
Strain at break, ε_b (%)	11 × 10*	Mass per unit area, ρ_a (g/m^2)	140
Aperture size, A (mm)	40 × 27		
Thickness, t (mm)	2		
Mass per unit area, ρ_a (g/m^2)	420		

Test Results - Bulli Track

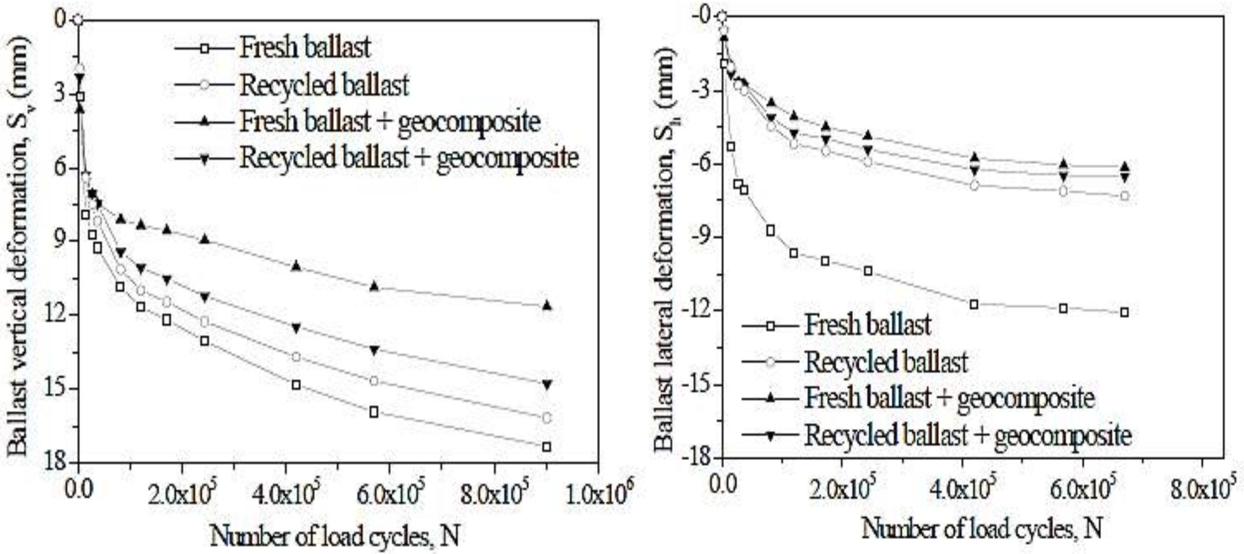
Maximum cyclic stresses at Test Section 1

Measured Location	Under the rail			
Axle Load	20.5 ton (Passenger Train 82 class locomotive)		25 ton (Coal Train 100 tons wagons)	
Stress (kPa)	Vertical (σ_v)	Horizontal (σ_h)	Vertical (σ_v)	Horizontal (σ_h)
Sleeper-ballast	238	25	293	46
Ballast-capping	63	18	86	26

Potential benefits of geocomposite at the ballast-capping interface

Deformation reduction due to geocomposite (%)		
	Fresh Ballast	Recycled Ballast
Vertical	33	9
Lateral	49	11

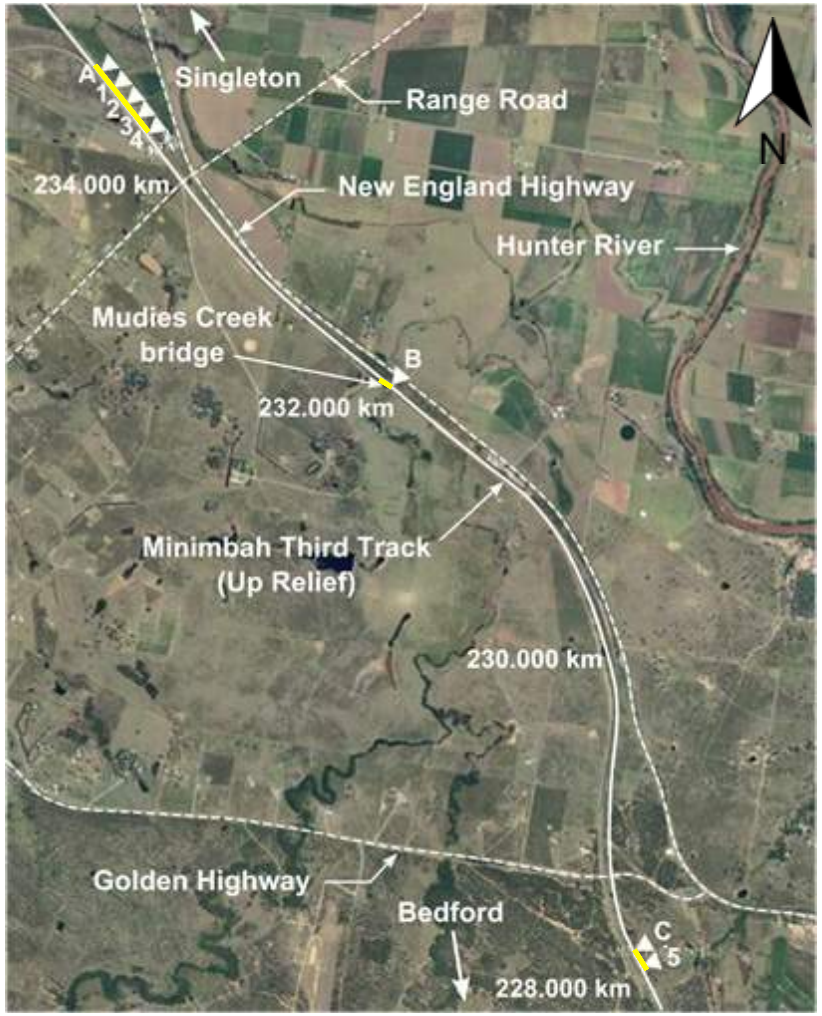
Average vertical and lateral deformation of ballast



- 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

Case Study: 2. Instrumented track at



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

Ballast stabilized with various geogrids, geocomposite and shock mats

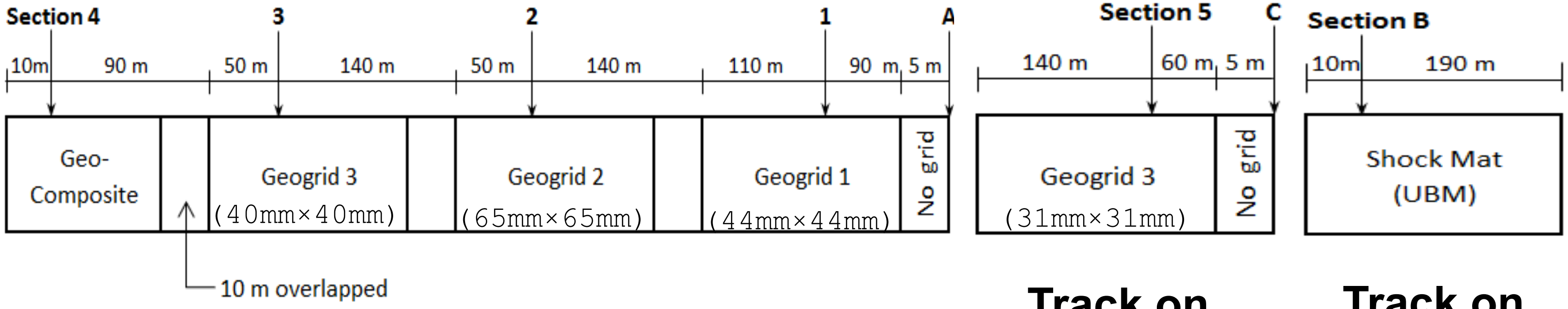
Instrumented Sections

A, C: Fresh ballast

1,2,3,4,5: Fresh ballast + geosynthetics

B: Fresh ballast + shock mat

Details of instrumented track

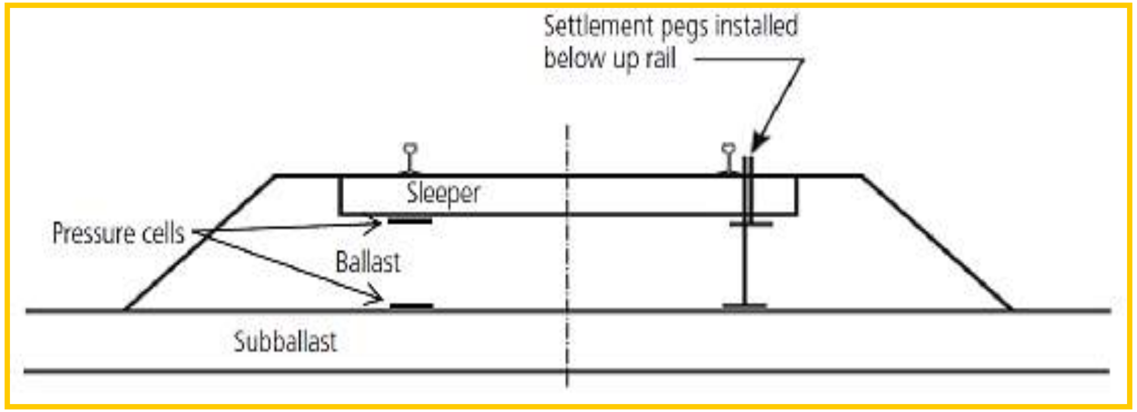


Track on soft embankment

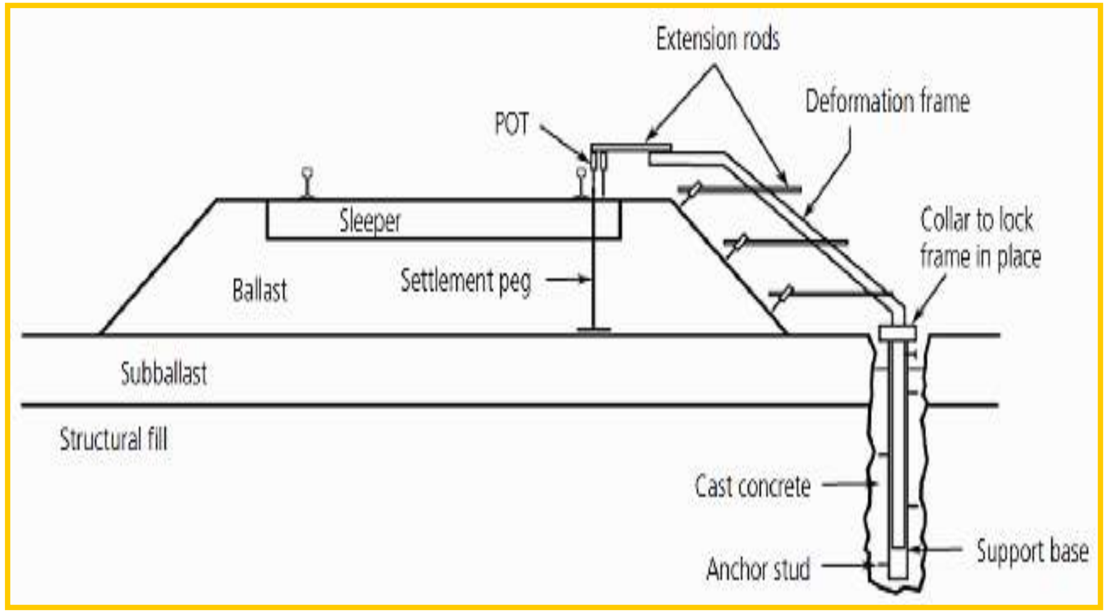
Track on hard rock

Track on bridge

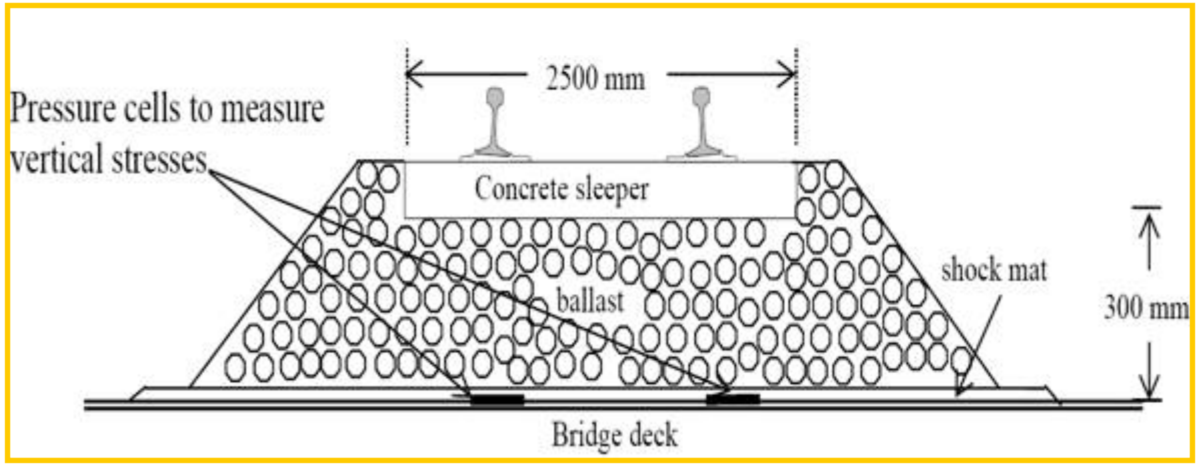
Field Instrumentation – Singleton, NSW



Locations of pressure cells & settlement pegs



Deformation frame

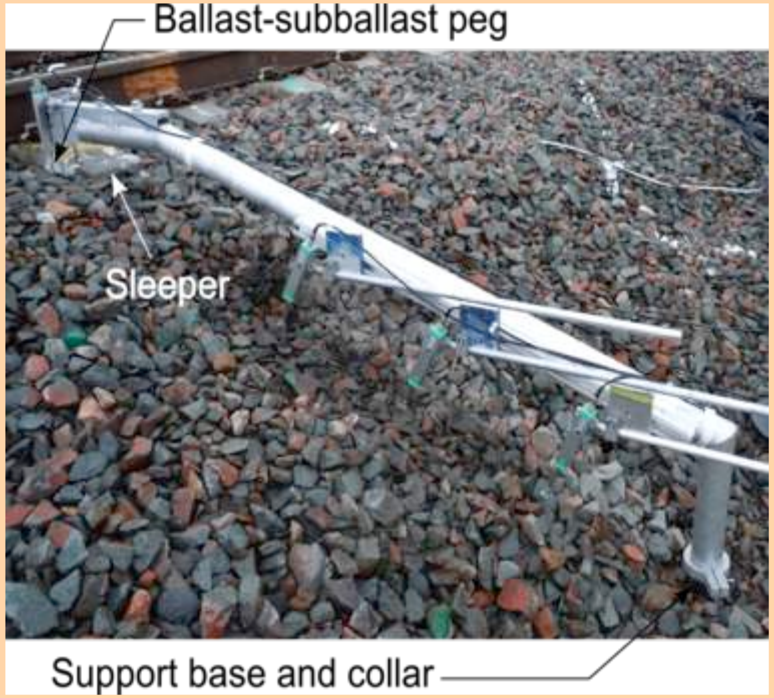


Shock mat above bridge deck

Field Trial on Instrumented Track in Singleton, NSW

Geogrid layer placed above the capping

Mudies Creek Bridge pressure cells installation

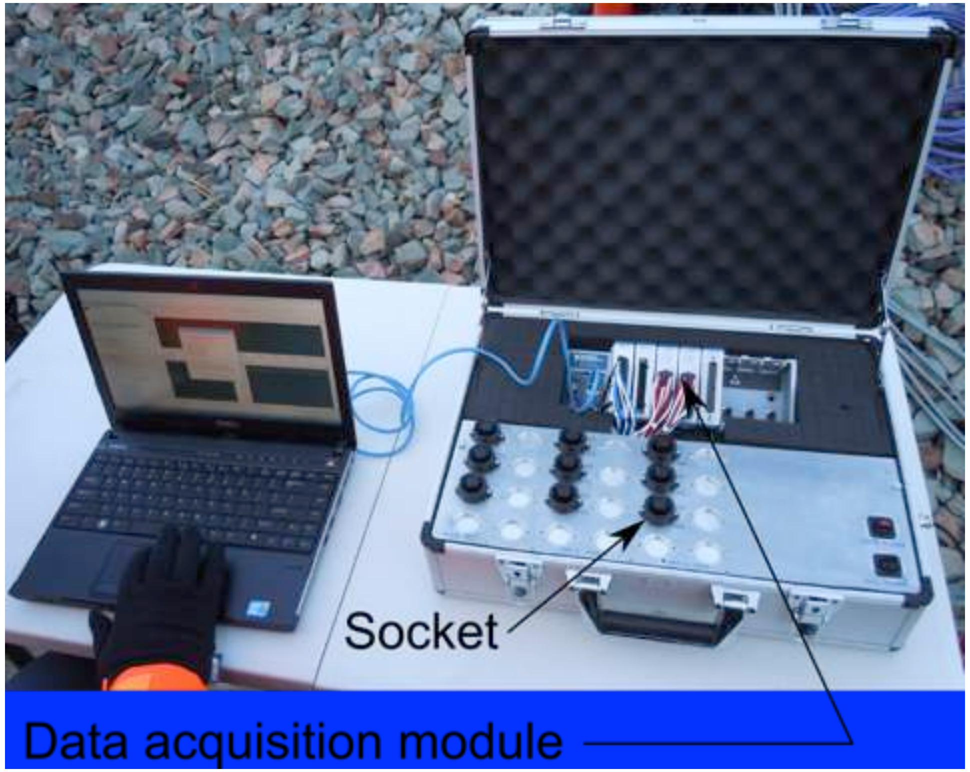


Displacement Monitoring Frame

Settlement pegs placement in the track

Placing of shock mat on bridge deck, Feb. 2010

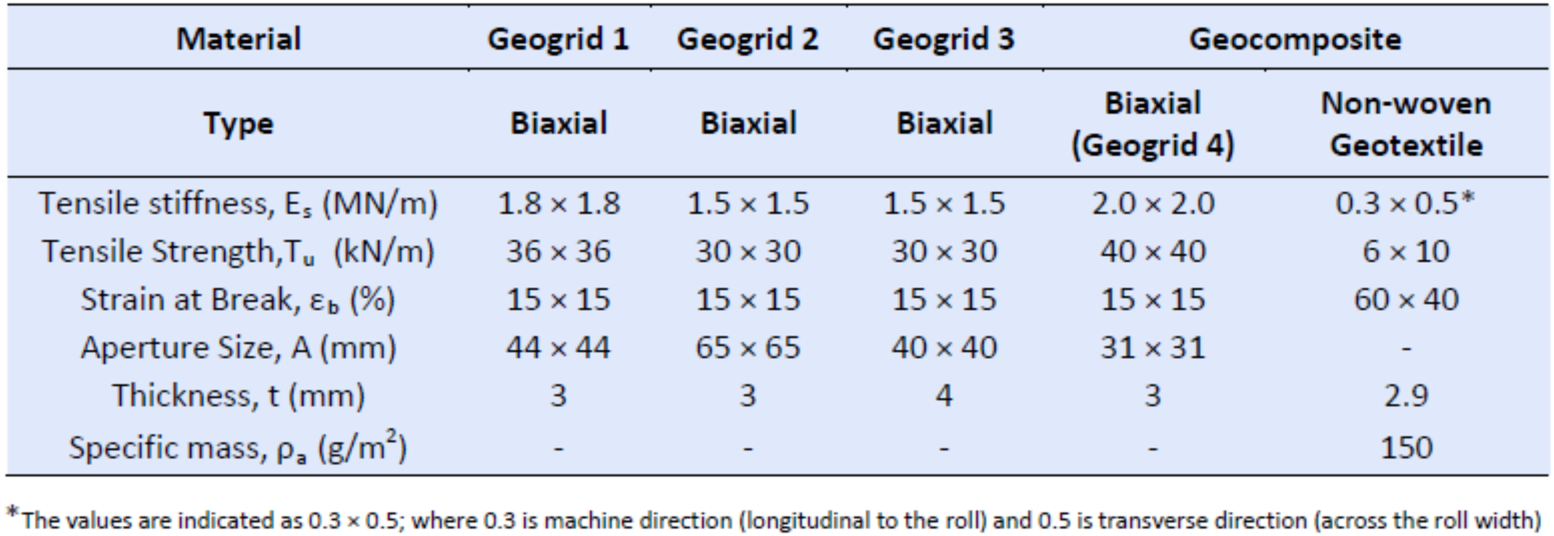
Data Acquisition



- 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

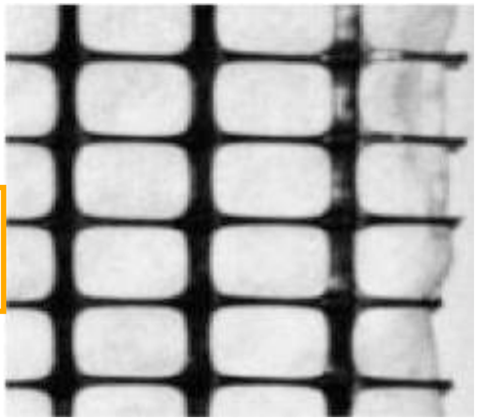
Technical specifications of different types of geosynthetics



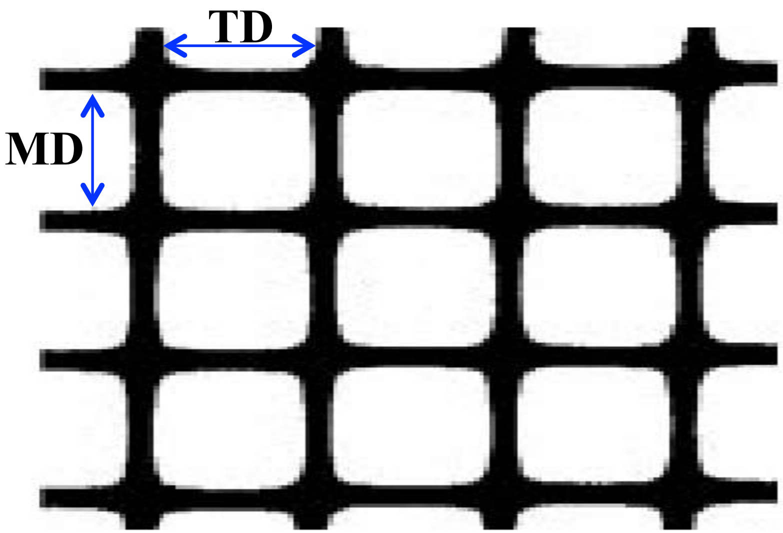
Material	Geogrid 1	Geogrid 2	Geogrid 3	Geocomposite	
Type	Biaxial	Biaxial	Biaxial	Biaxial (Geogrid 4)	Non-woven Geotextile
Tensile stiffness, E_s (MN/m)	1.8 × 1.8	1.5 × 1.5	1.5 × 1.5	2.0 × 2.0	0.3 × 0.5*
Tensile Strength, T_u (kN/m)	36 × 36	30 × 30	30 × 30	40 × 40	6 × 10
Strain at Break, ε_b (%)	15 × 15	15 × 15	15 × 15	15 × 15	60 × 40
Aperture Size, A (mm)	44 × 44	65 × 65	40 × 40	31 × 31	-
Thickness, t (mm)	3	3	4	3	2.9
Specific mass, ρ_a (g/m^2)	-	-	-	-	150

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

Geocomposite (Geogrid + Geotextile)



Geogrid



MD – Machine Direction
TD – Transverse direction

For eg.,
1.5 × 1.5 means MD × TD

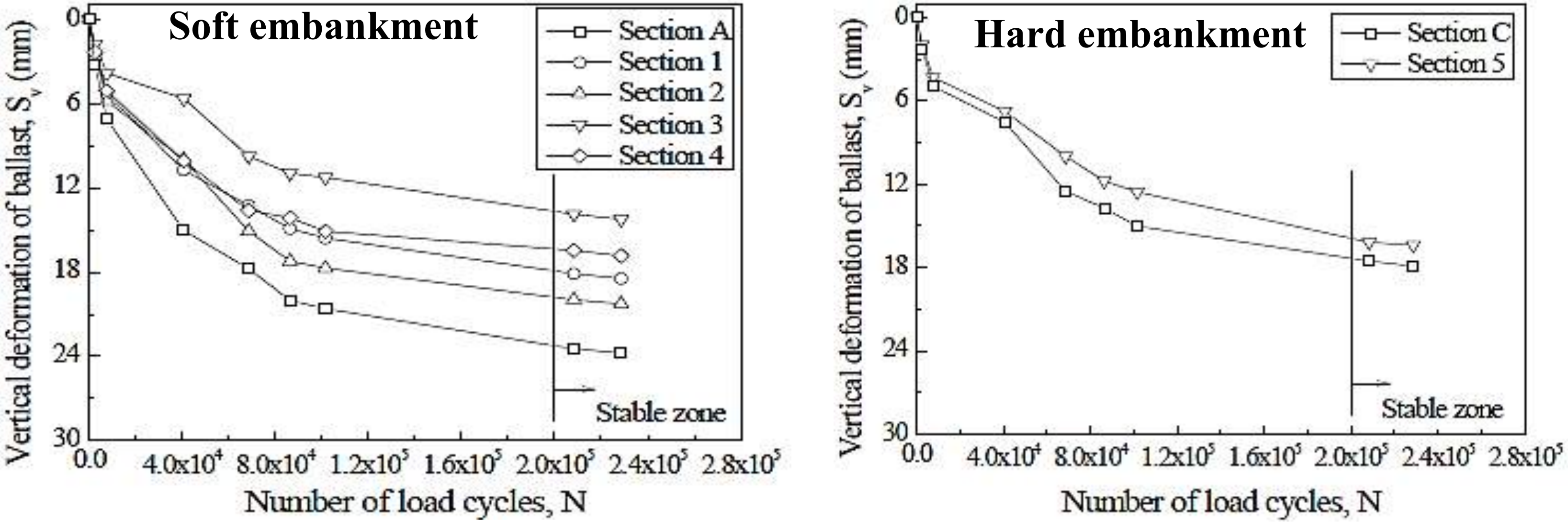
Test Results - Singleton Track

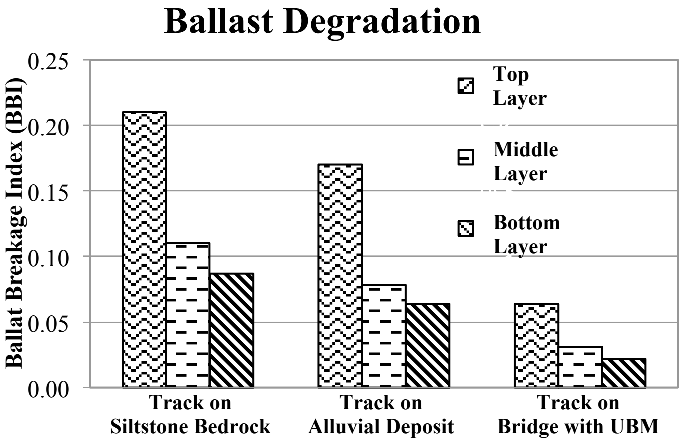
Maximum cyclic vertical stresses

Vertical stress, σ_v (kPa) measured at	Sections A and 1 (soft embankment)	Sections C and 5 (hard rock)
Sleeper-ballast interface	170 - 180	215 - 230
Ballast-capping interface	30 - 35	90 - 110

- 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 × 40 mm size apertures performed better (optimum aperture size $1.15D_{50}$ of ballast)

Vertical deformation of ballast at soft and hard embankment




Ballast Degradation

- 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

5

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4-7 September 2016, Guimarães, Portugal

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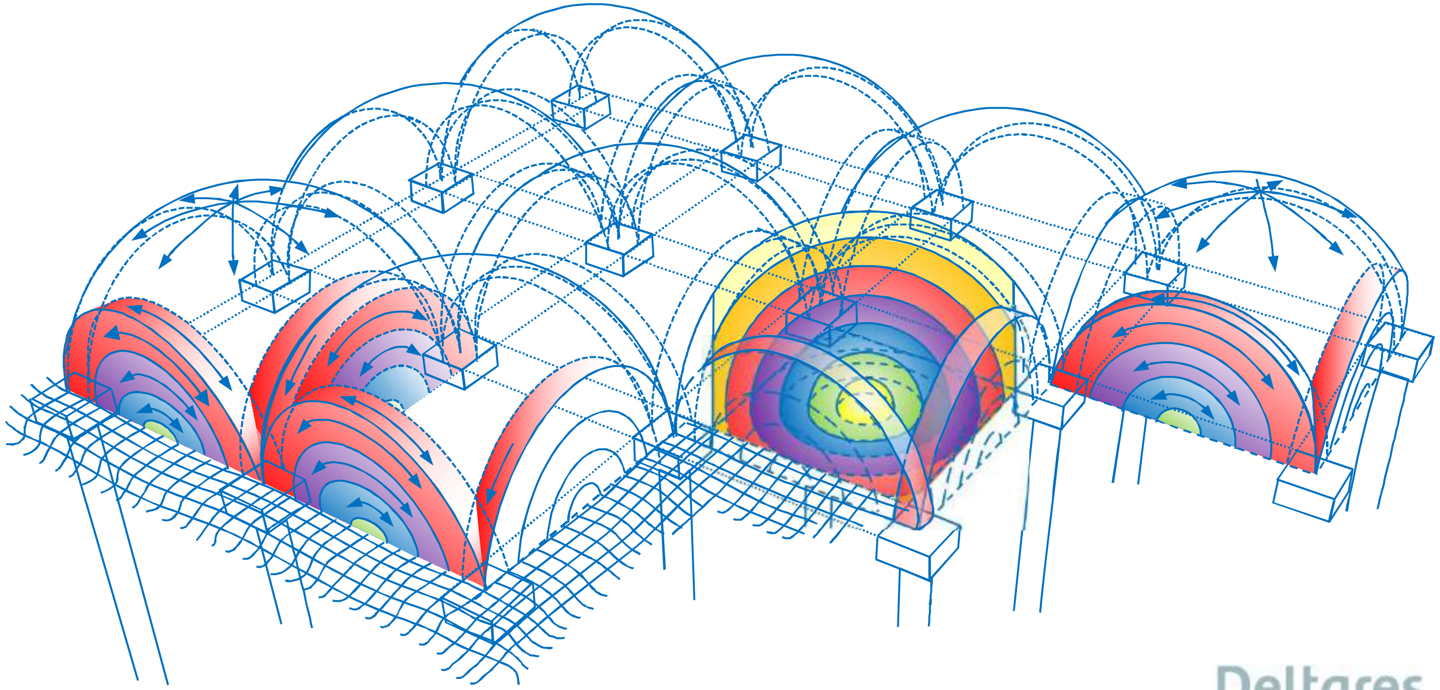
Basal Reinforced Piled Embankments



Suzanne J.M. van Eekelen

Deltares, Netherlands
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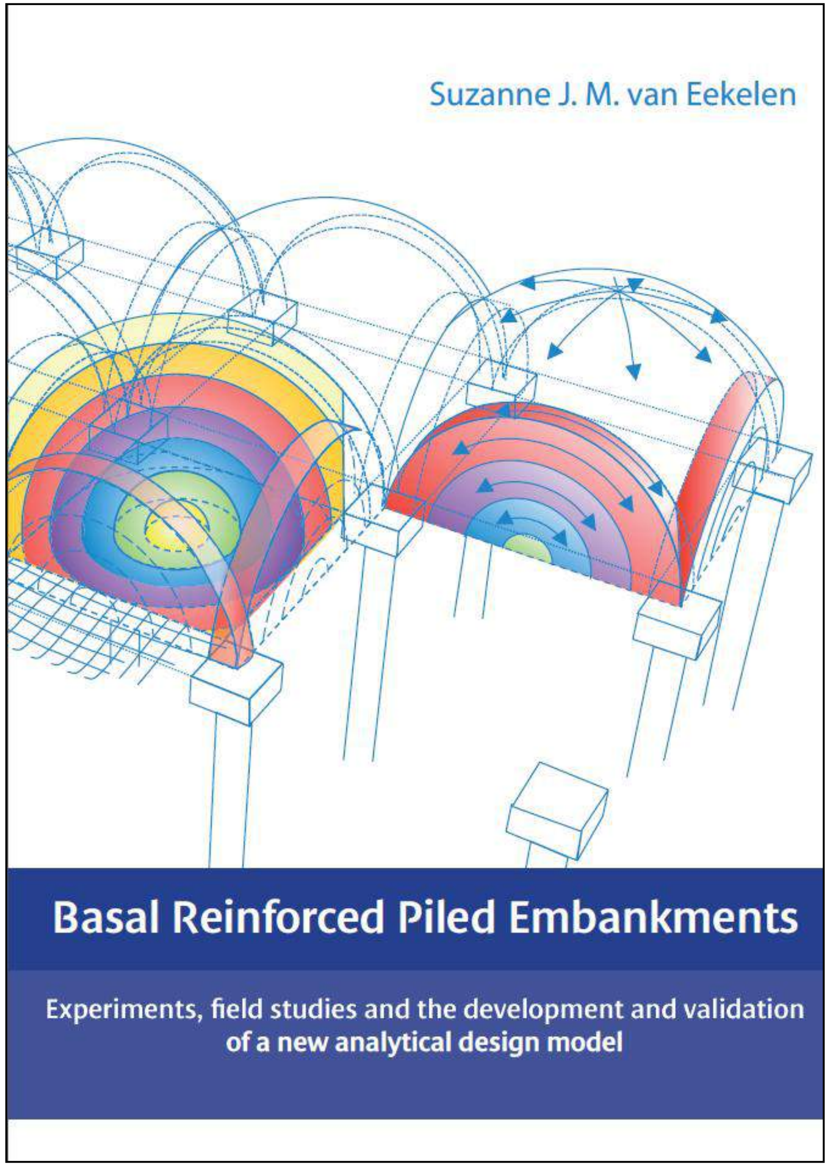
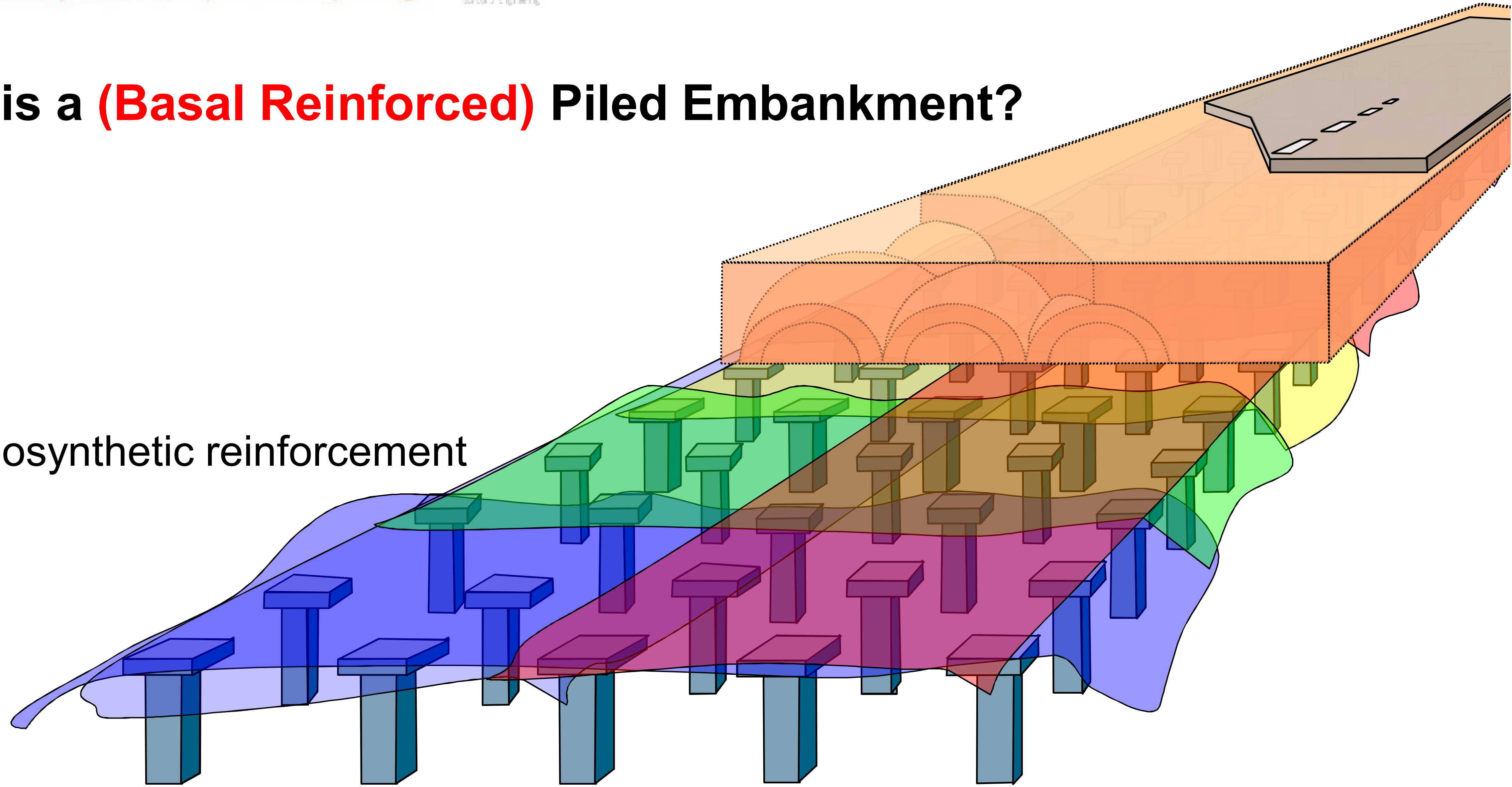


Suzanne J. M. van Eekelen

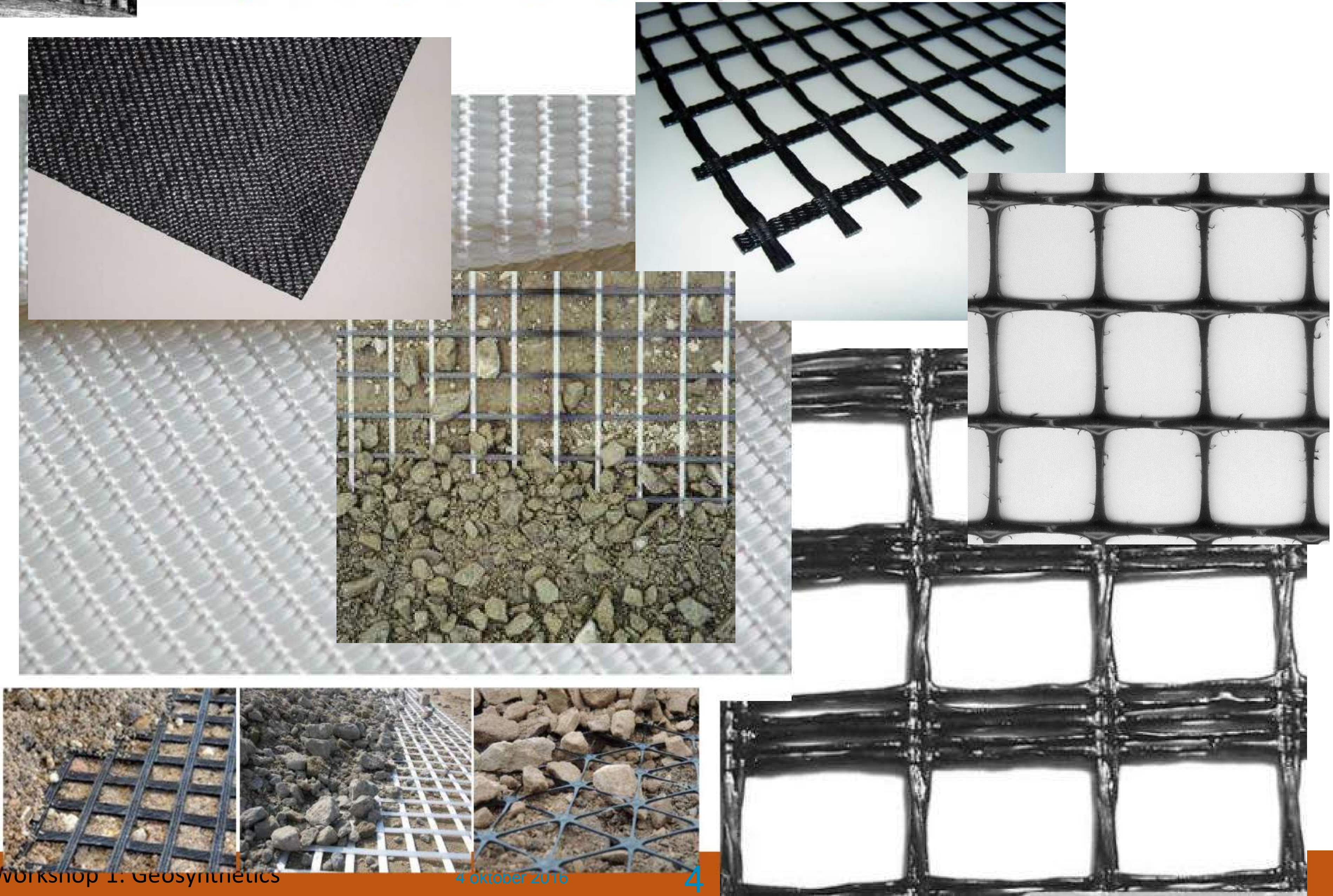
Basal Reinforced Piled Embankments

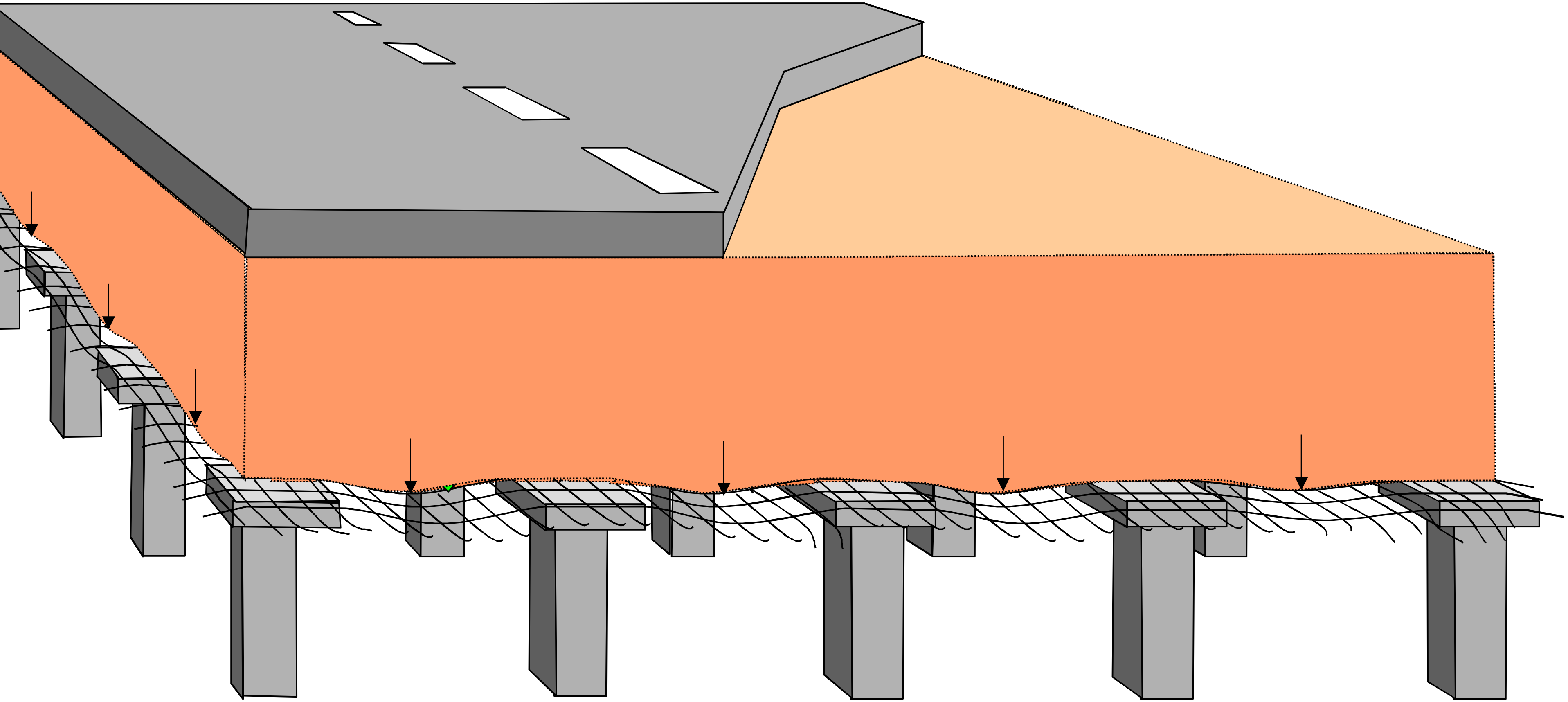
Experiments, field studies and the development and validation of a new analytical design model

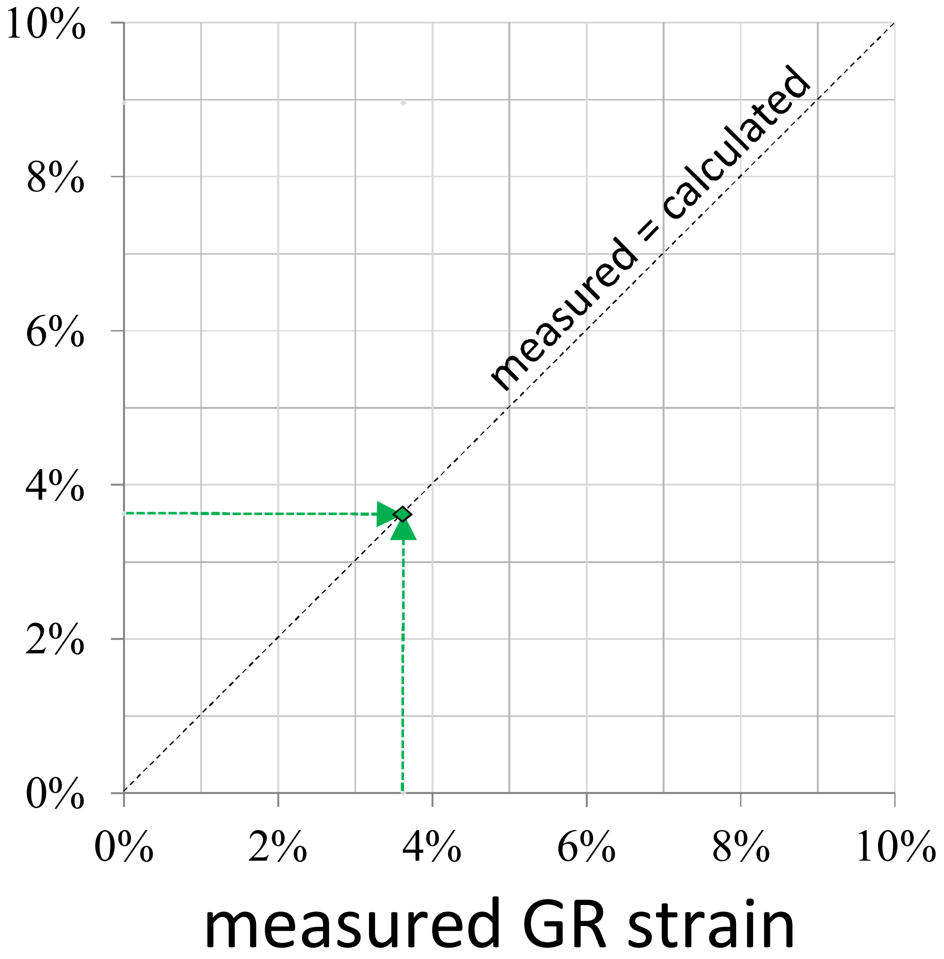
What is a (Basal Reinforced) Piled Embankment?

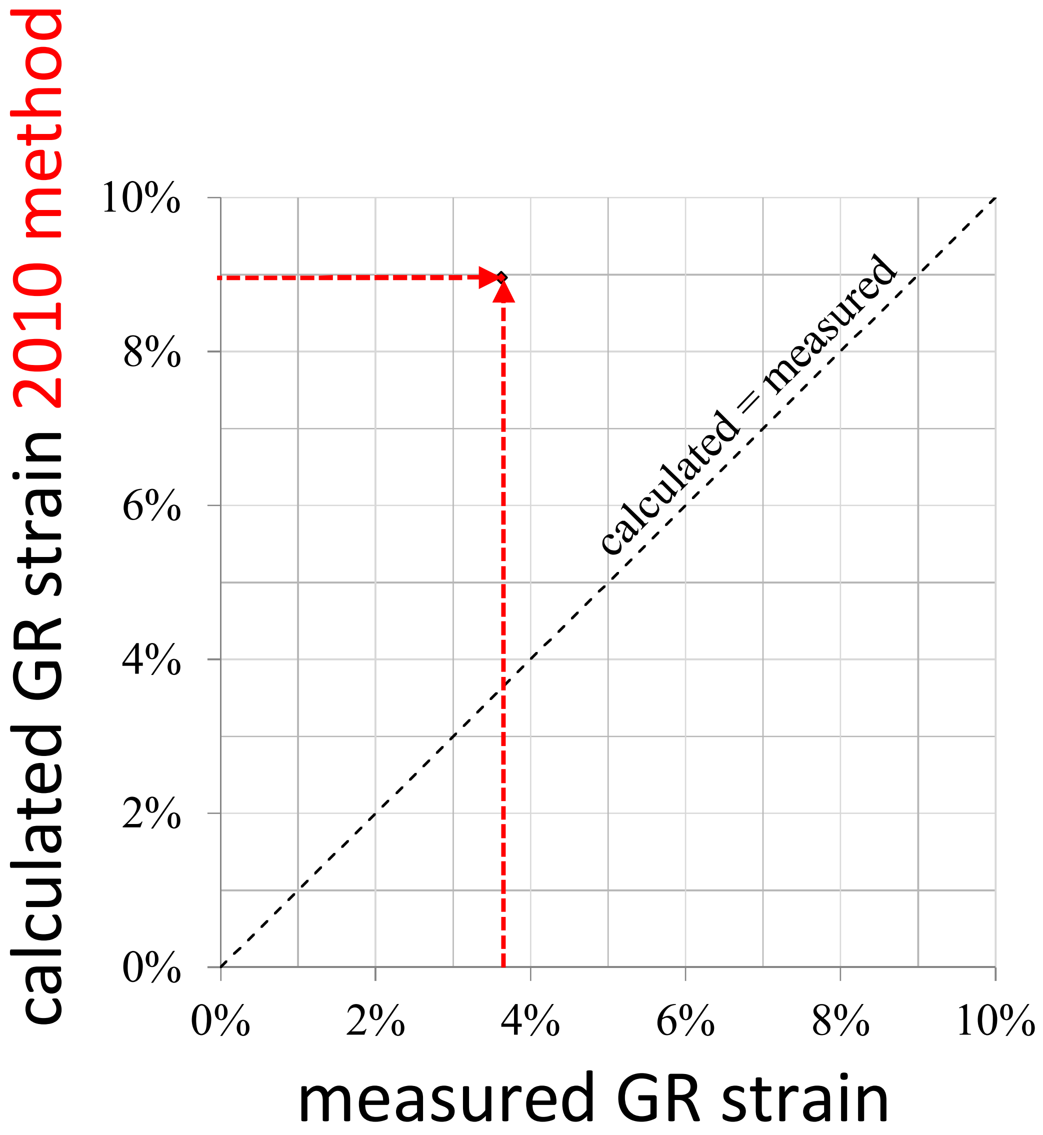


GR = geosynthetic reinforcement






10%
8%
6%
4%
2%
0%
0%
2%
4%
6%
8%
10%
measured = calculated
measured GR strain


calculated GR strain 2010 method
10%
8%
6%
4%
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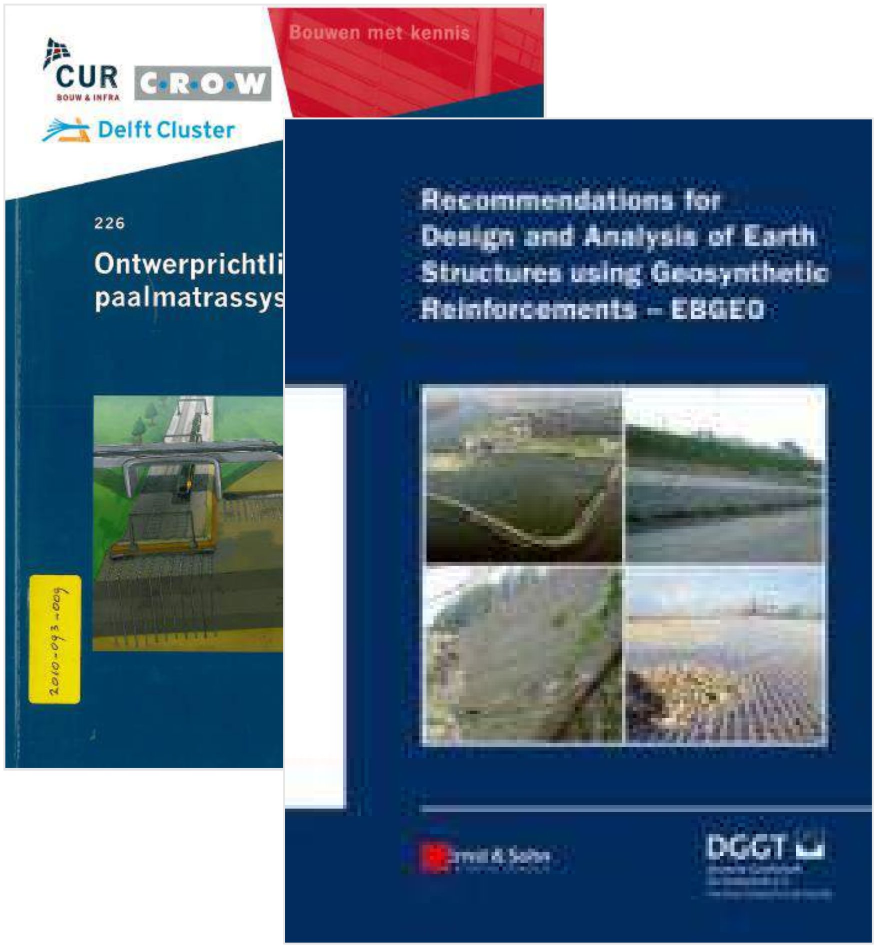
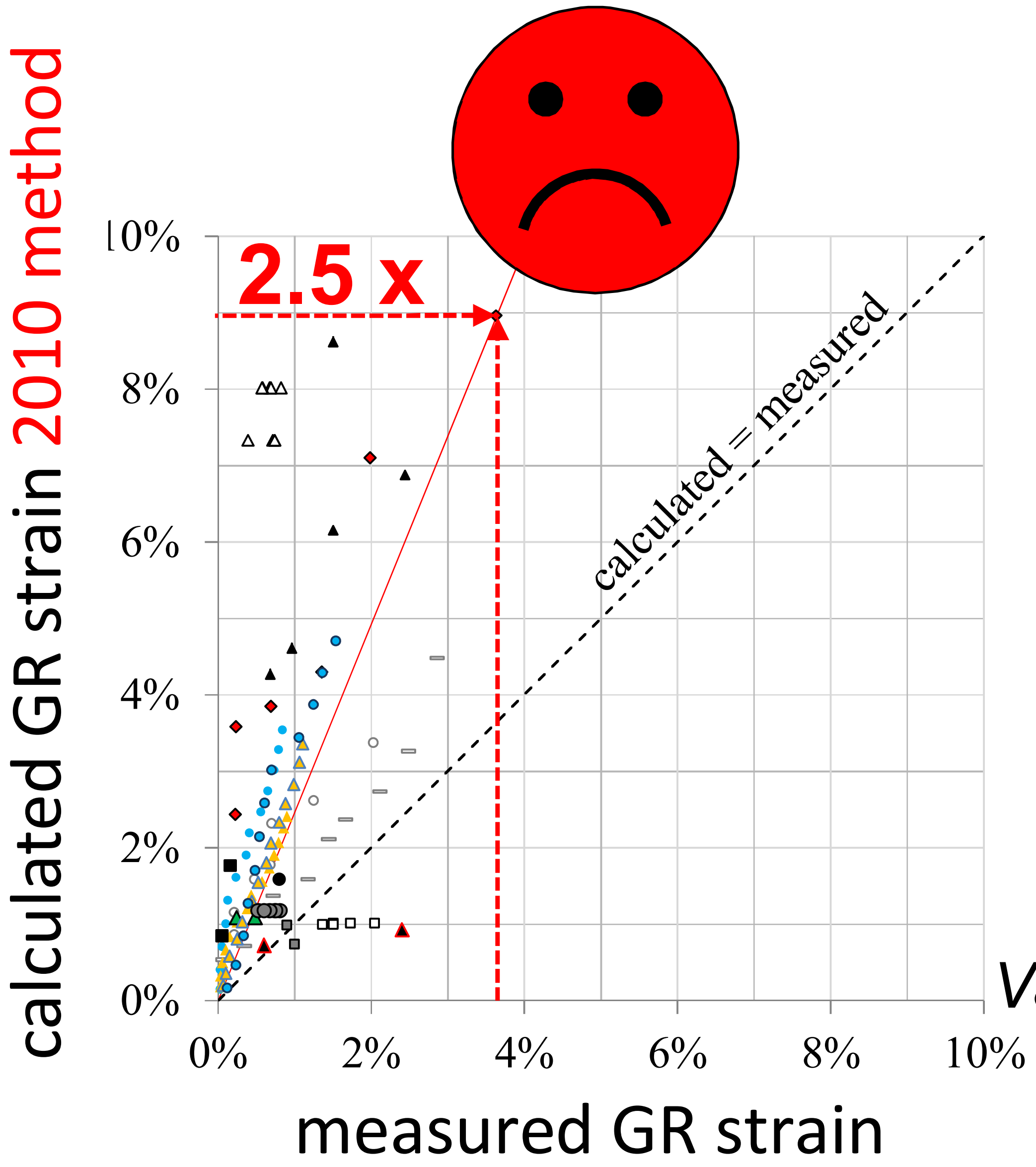
0%
0%
2%
4%
6%
8%
10%
calculated = measured
measured GR strain

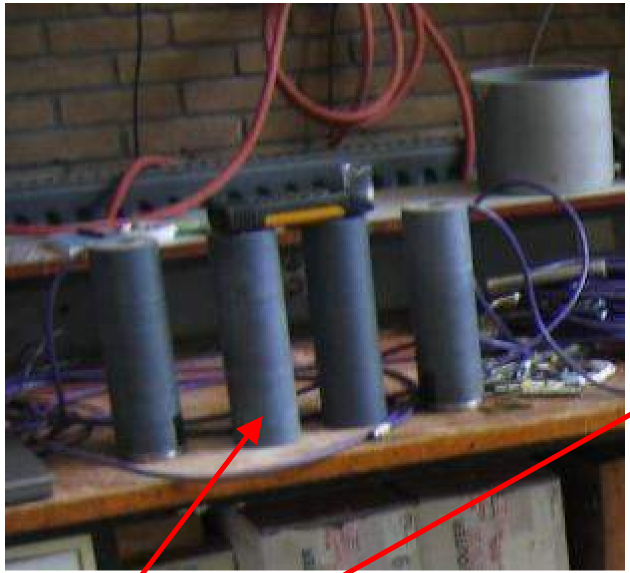

Bouwen met kennis
CUR
BOUW & INFRA
C.R.O.W
Delft Cluster
226
Ontwerprichtli
paalmatrassys
Recommendations for Design and Analysis of Earth Structures using Geosynthetic Reinforcements – EBGEO
Ernst & Sohn
DGGT

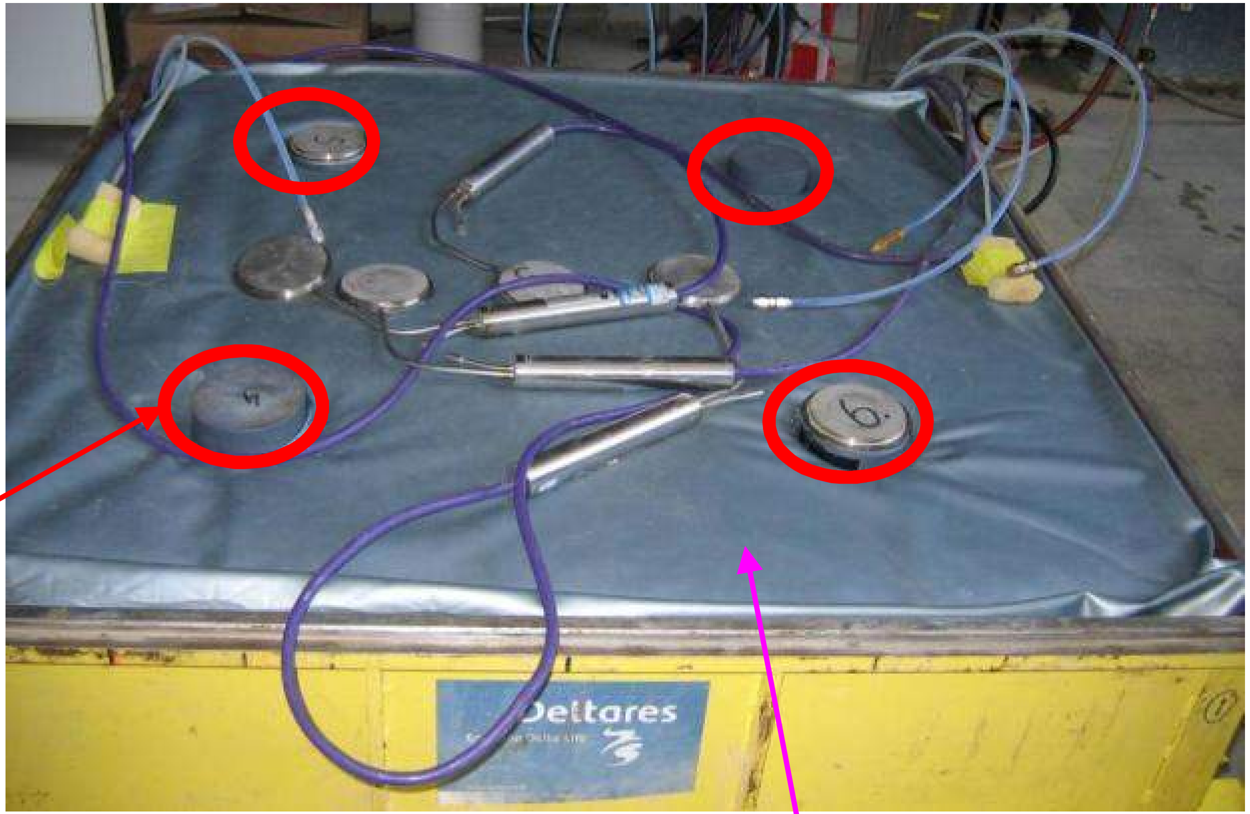
2010 method



Measurements:

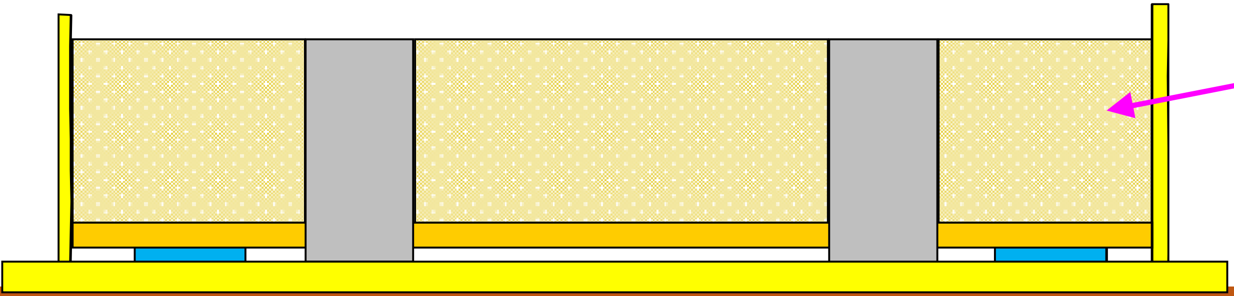
Zaeske 2001, Germany
Van Duijnen et al 2010, Netherlands
Huang et al 2009, Finland
Oh and Shin 2007, Korea
Haring et al, 2008, N210, Netherlands
Weihrauch 2013, Hamburg, Germany
Vollmert et al 2007, Bremerhafen, Germany.
Almeida et al 2007, Rio de Janeiro, Brazil
Briancon and Simon 2012, France
Van Eekelen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

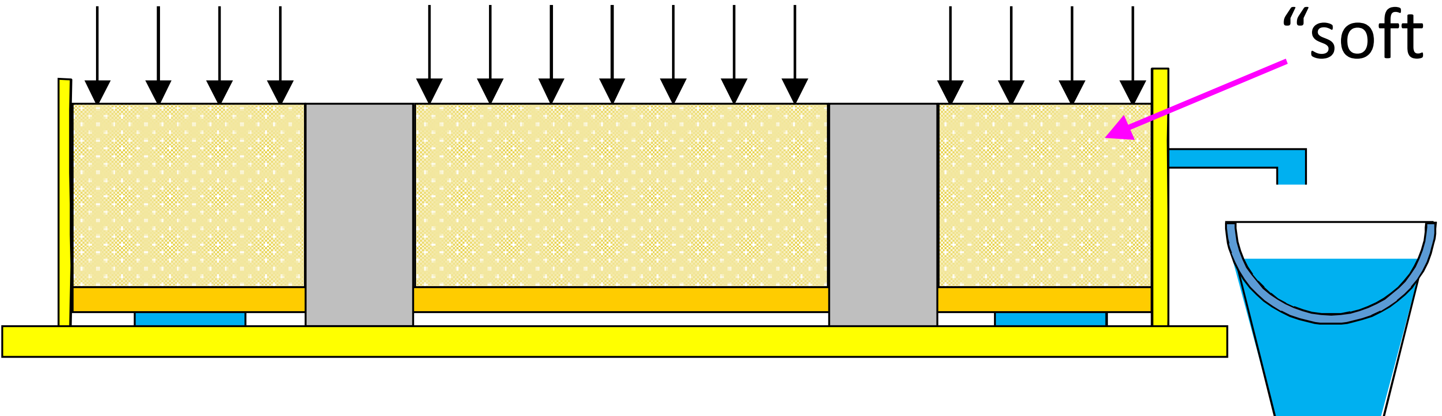




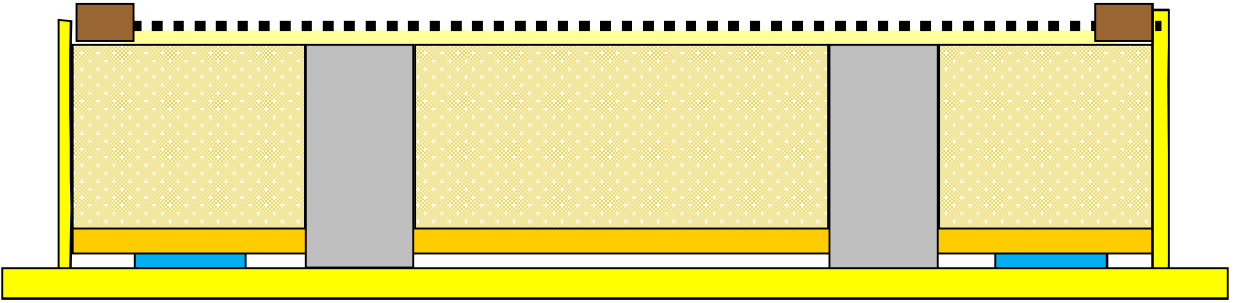
piles

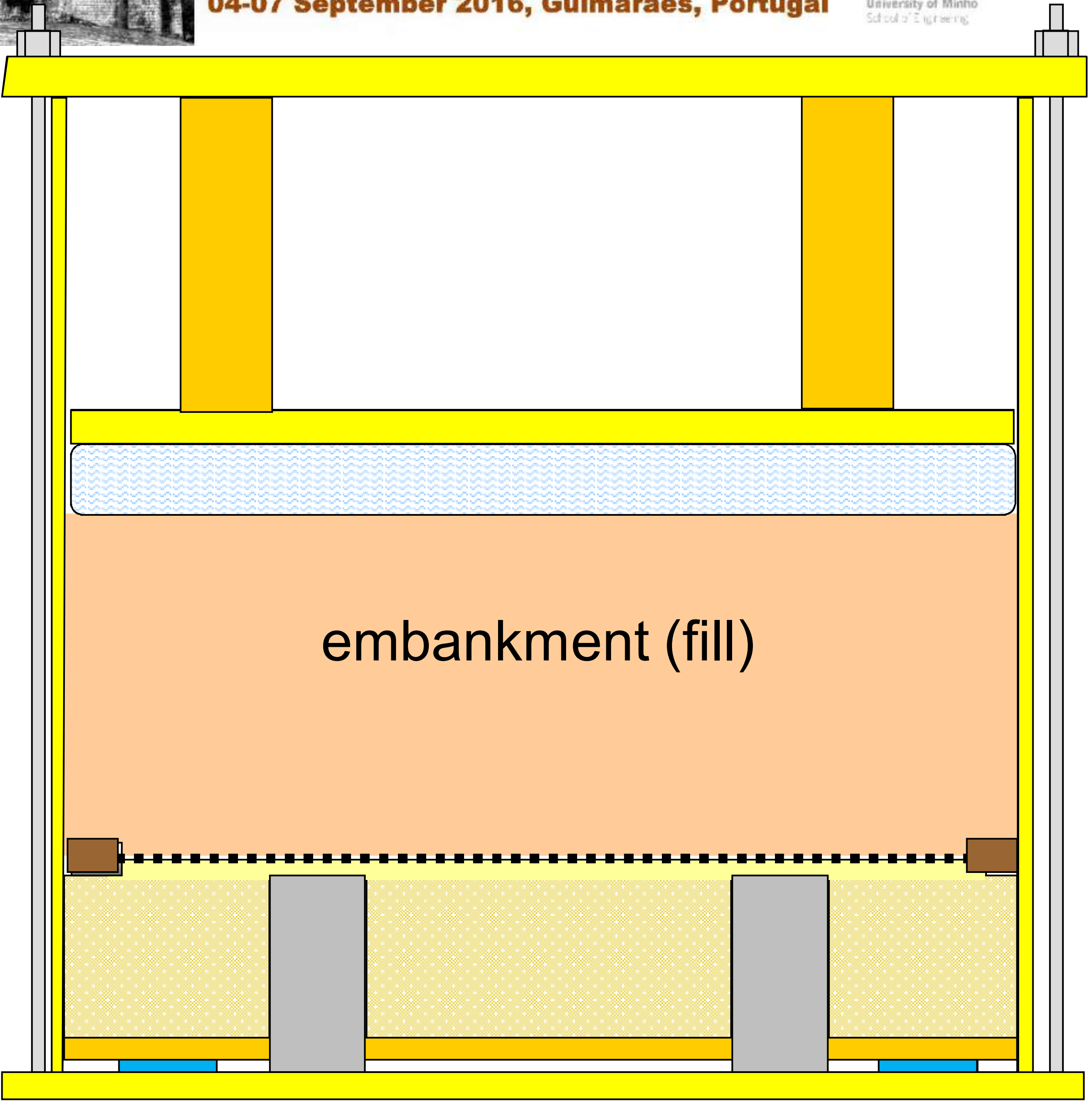
“soft soil”

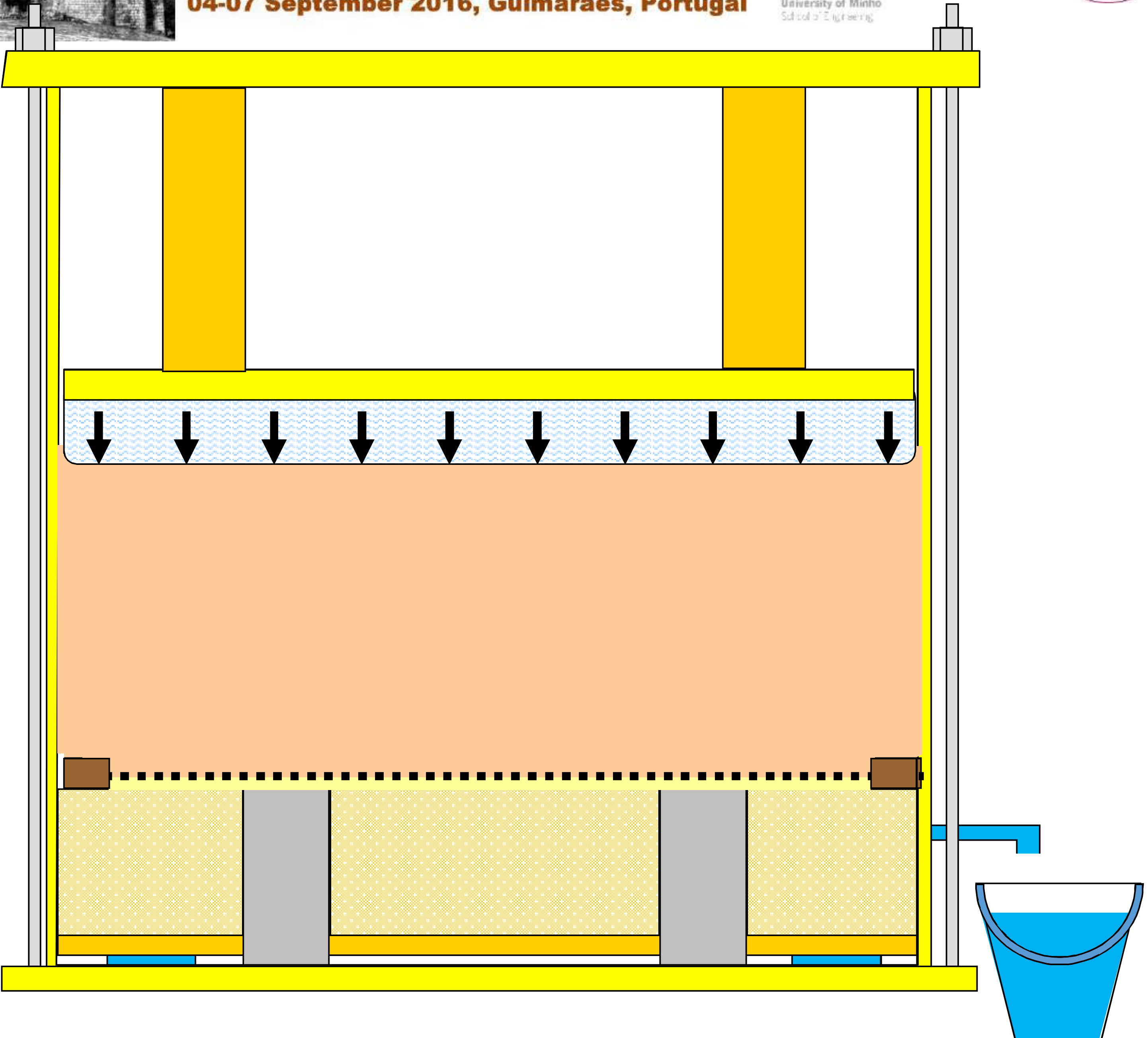


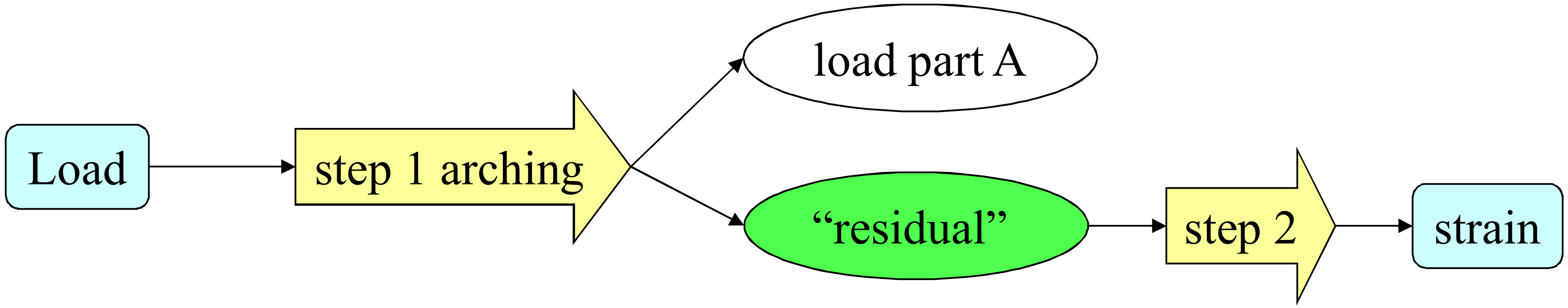

“soft soil”

Geosynthetic reinforcement



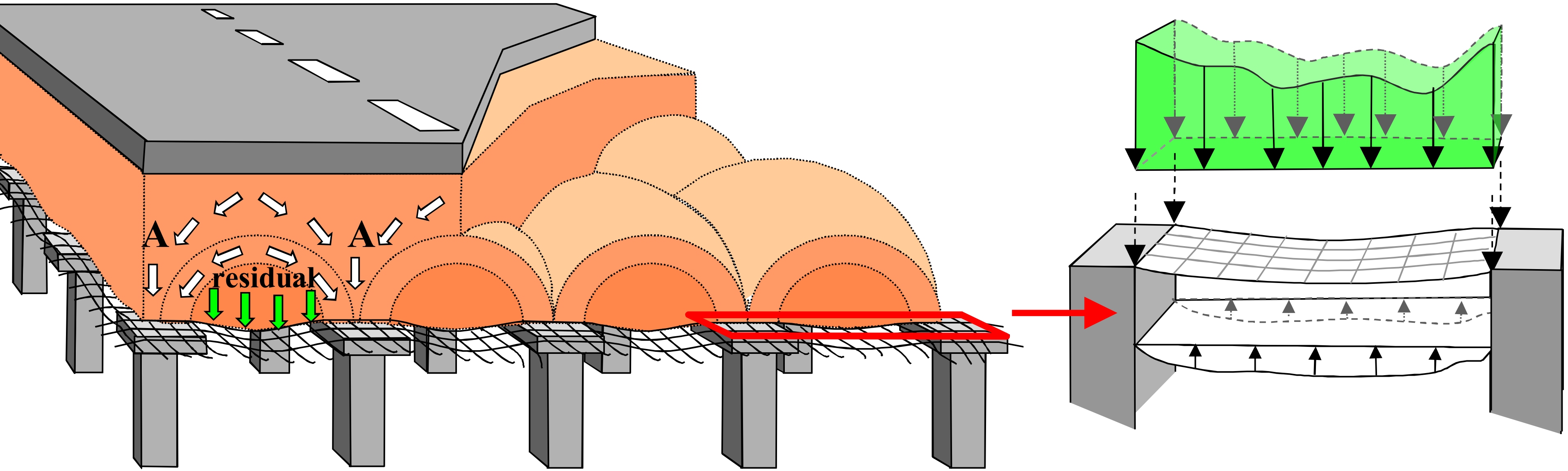

embankment (fill)

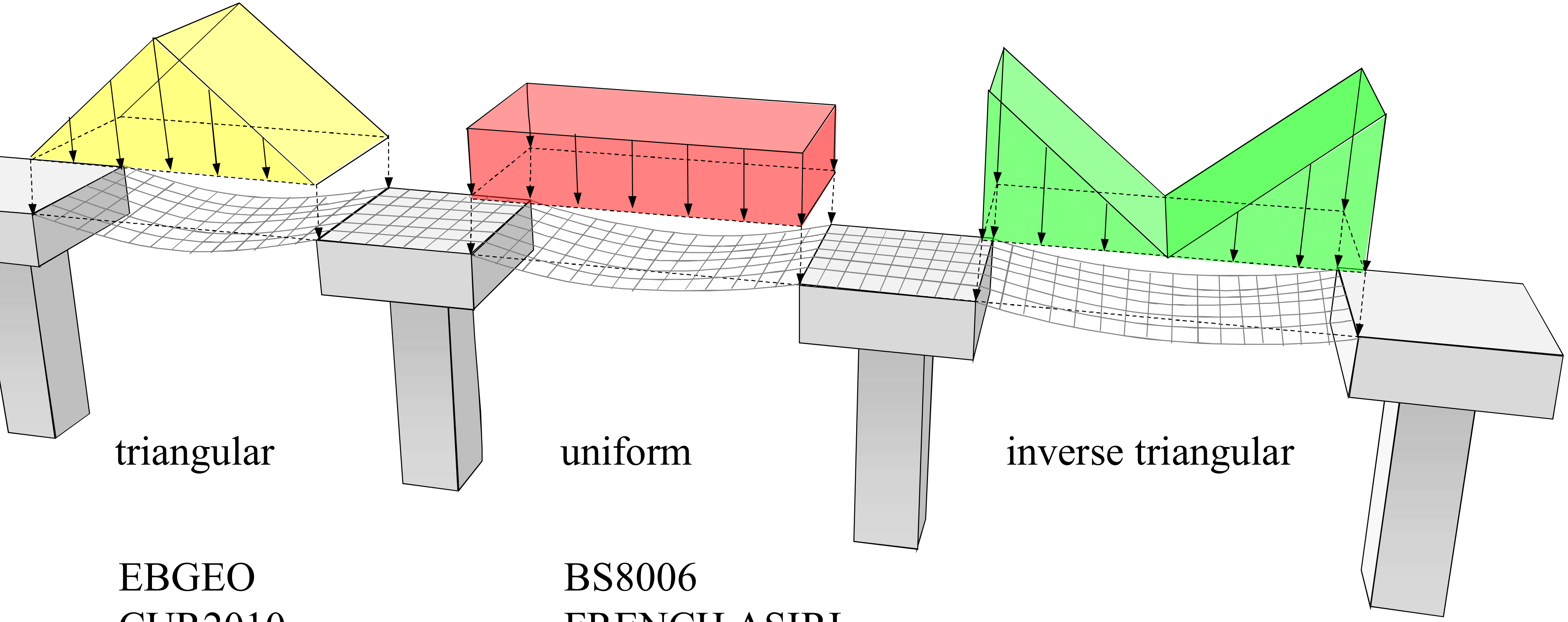




Load → step 1 arching → load part A

step 1 arching → "residual" → step 2 → strain





triangular

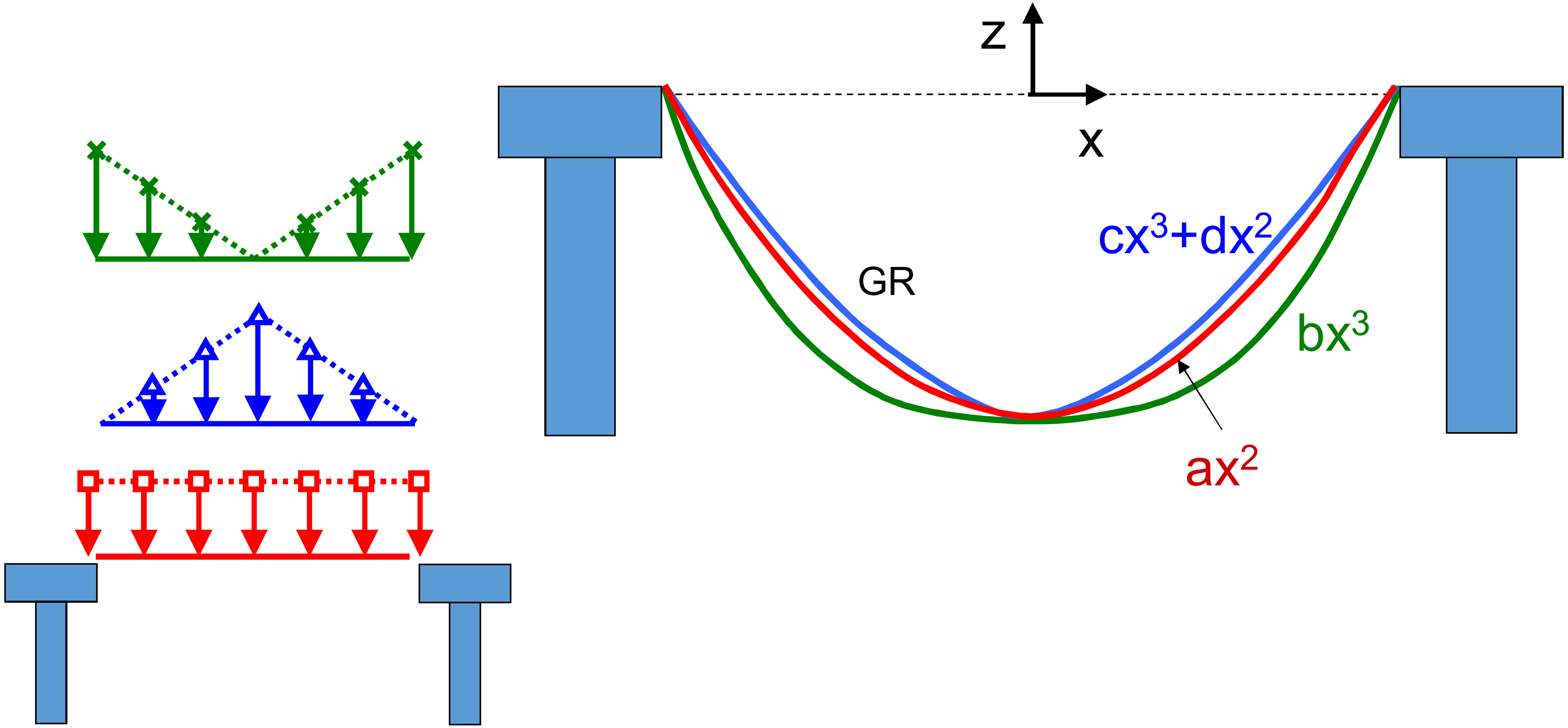
uniform

inverse triangular

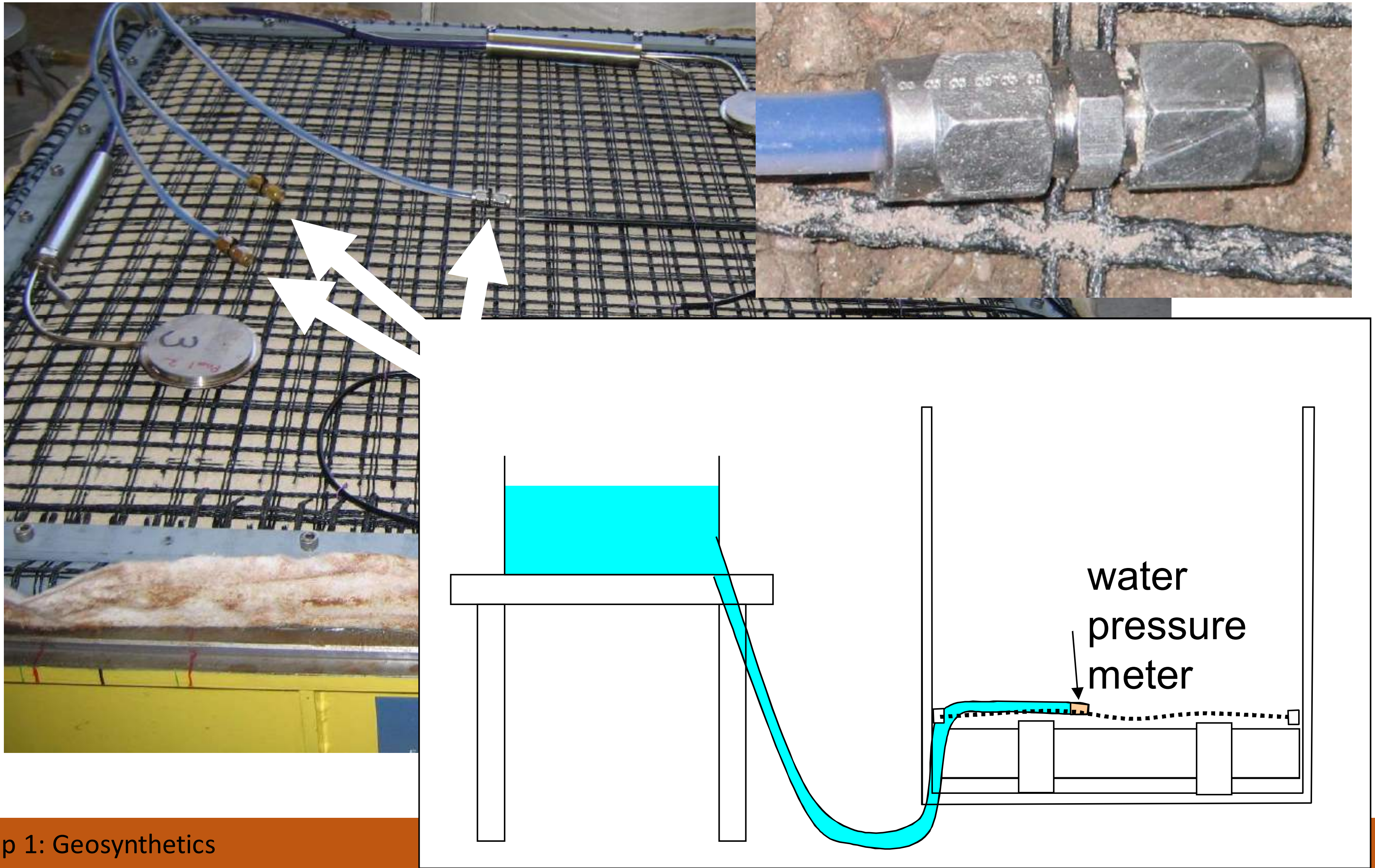
EBGEO
CUR2010

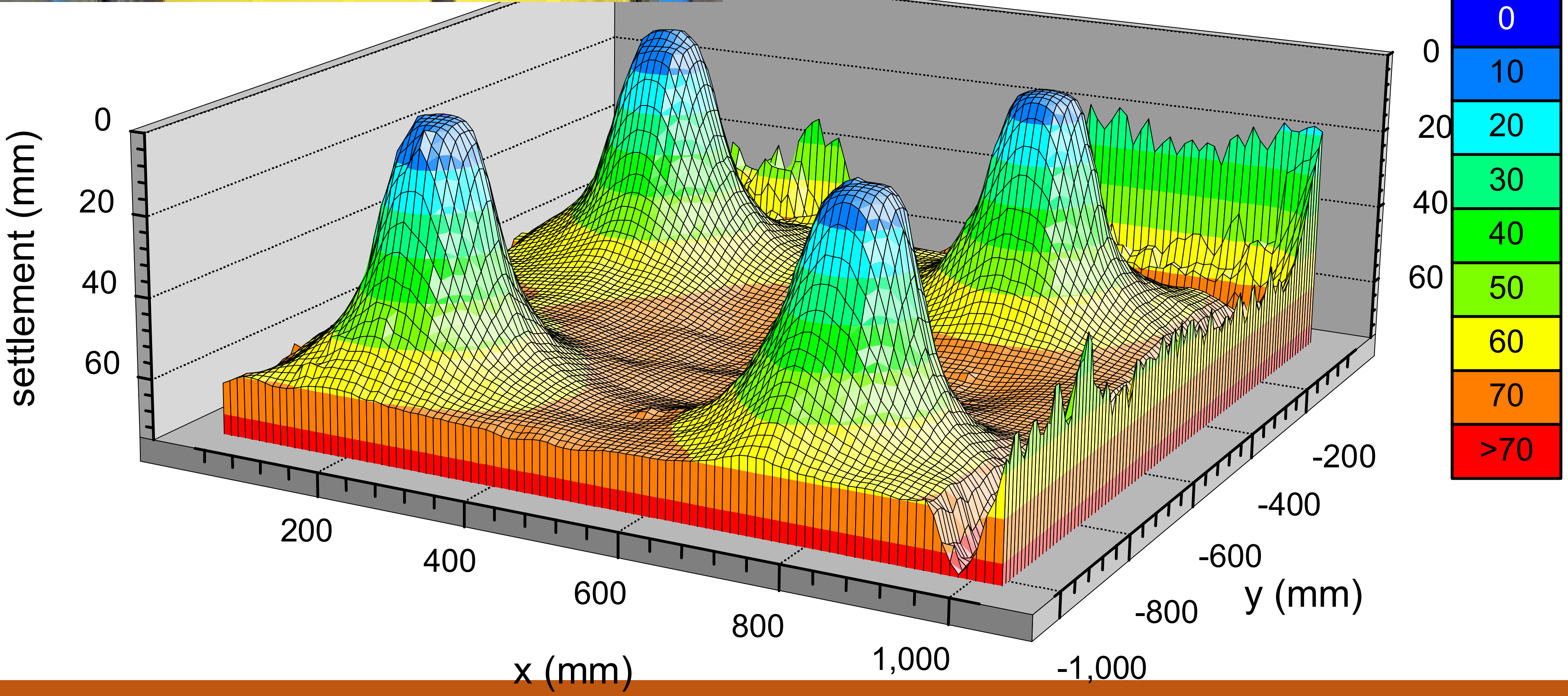
BS8006
FRENCH ASIRI

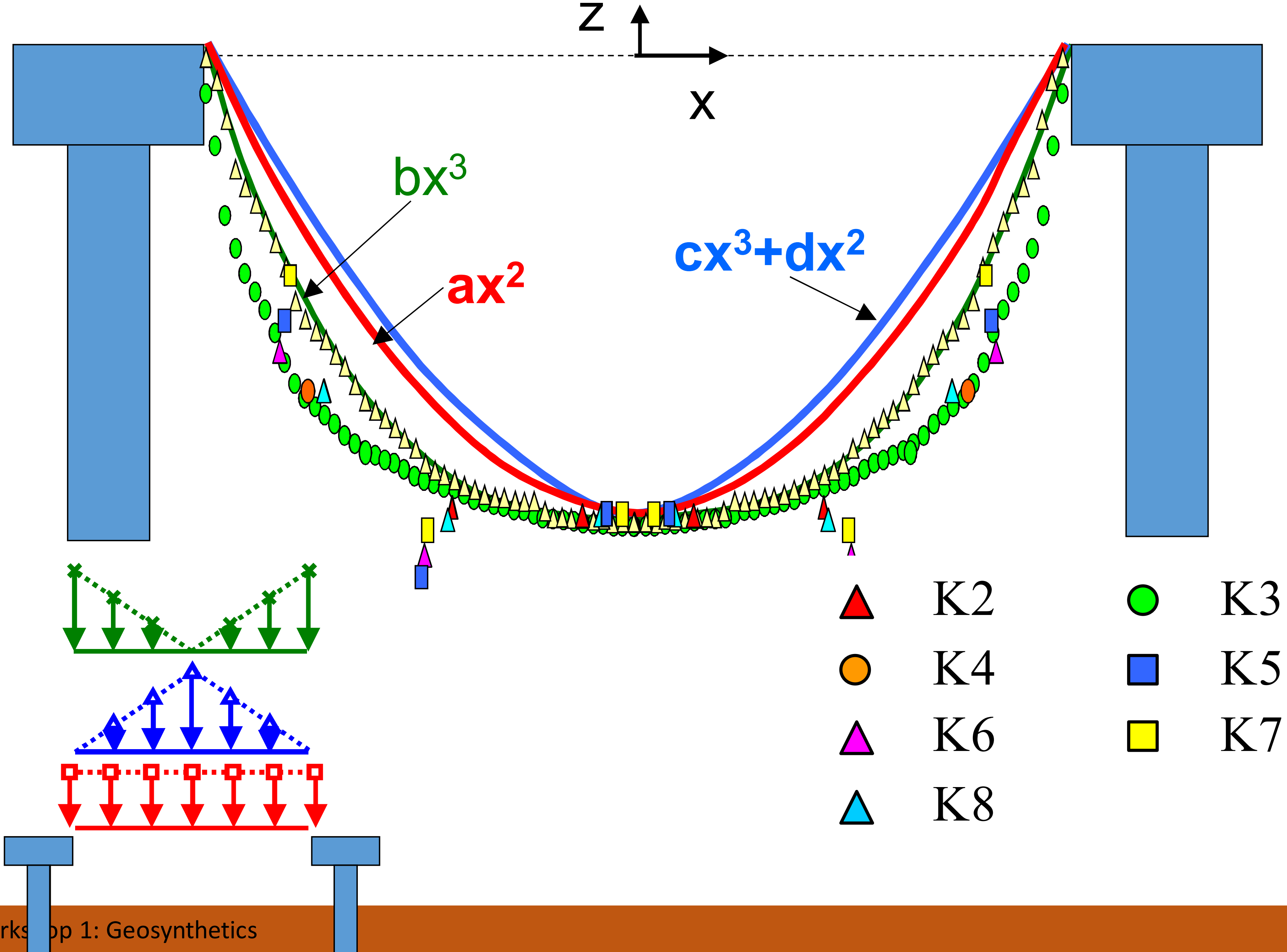
Load distribution ↔ deformation GR



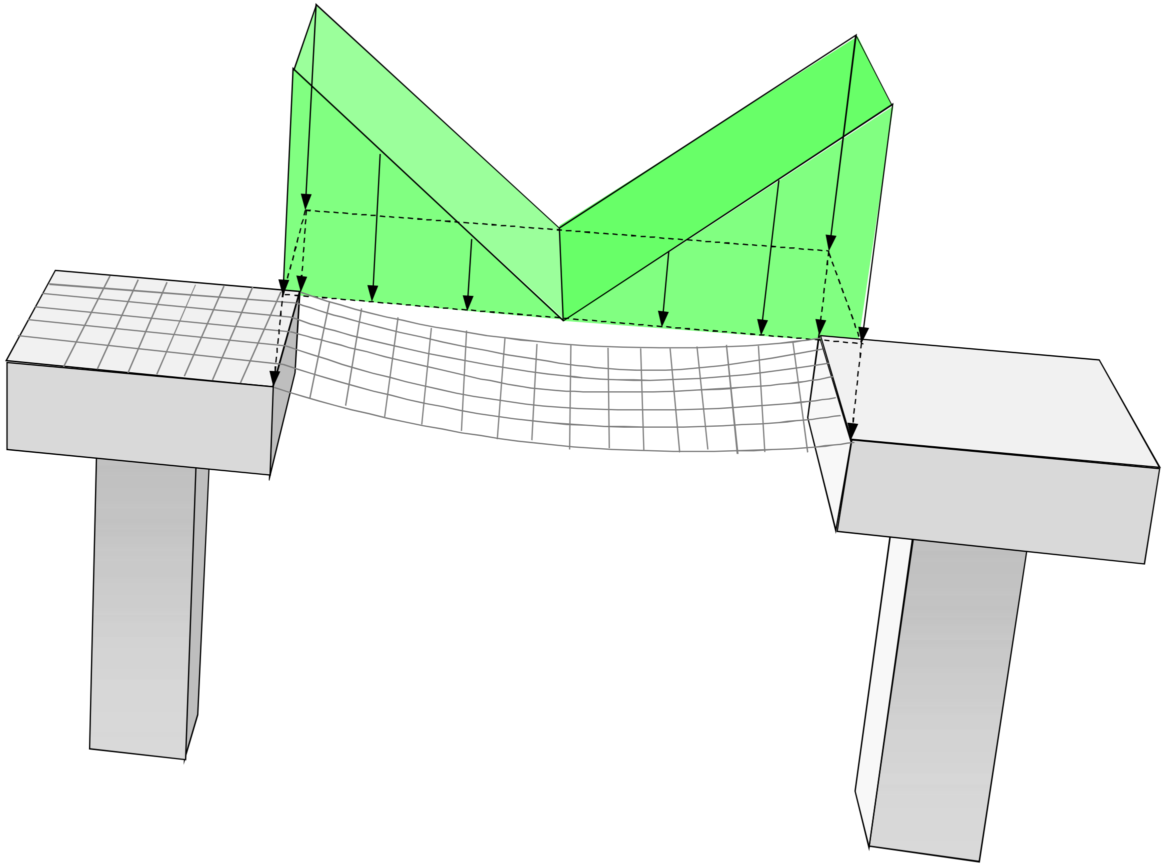
GR deflection (vertical deformation)



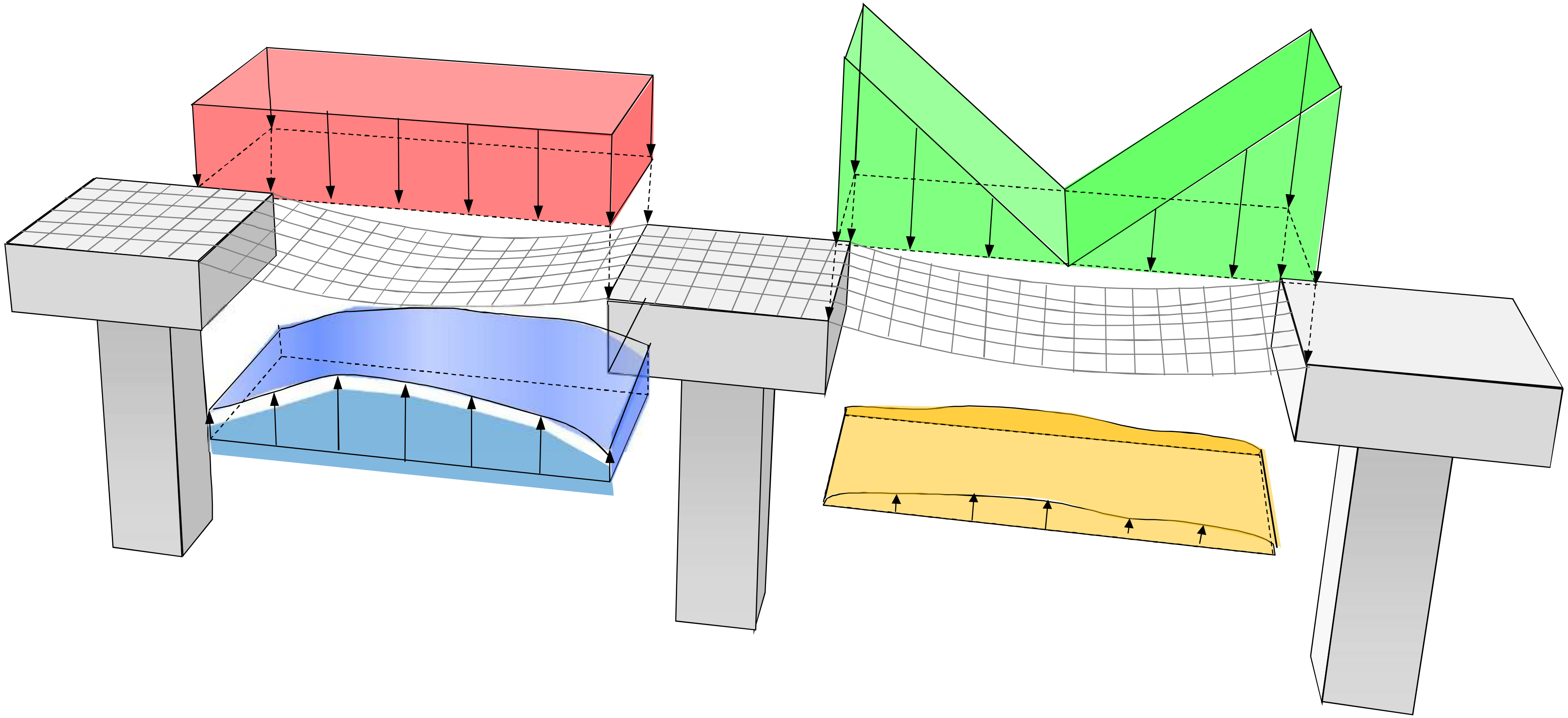

settlement (mm)
0
20
40
60
0
20
40
60
200
400
600
800
1,000
x (mm)
-200
-400
-600
-800
-1,000
y (mm)
0
10
20
30
40
50
60
70
>70

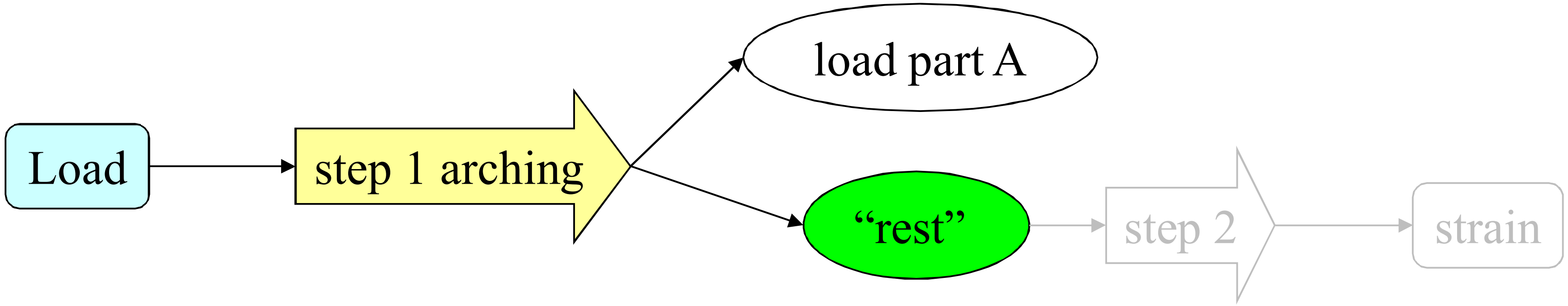

z
x
bx³
ax²
cx³+dx²
▲ K2
● K3
● K4
■ K5
▲ K6
■ K7
▲ K8

Observed load distribution:



More precise (van Eekelen et al., 2015):





Load

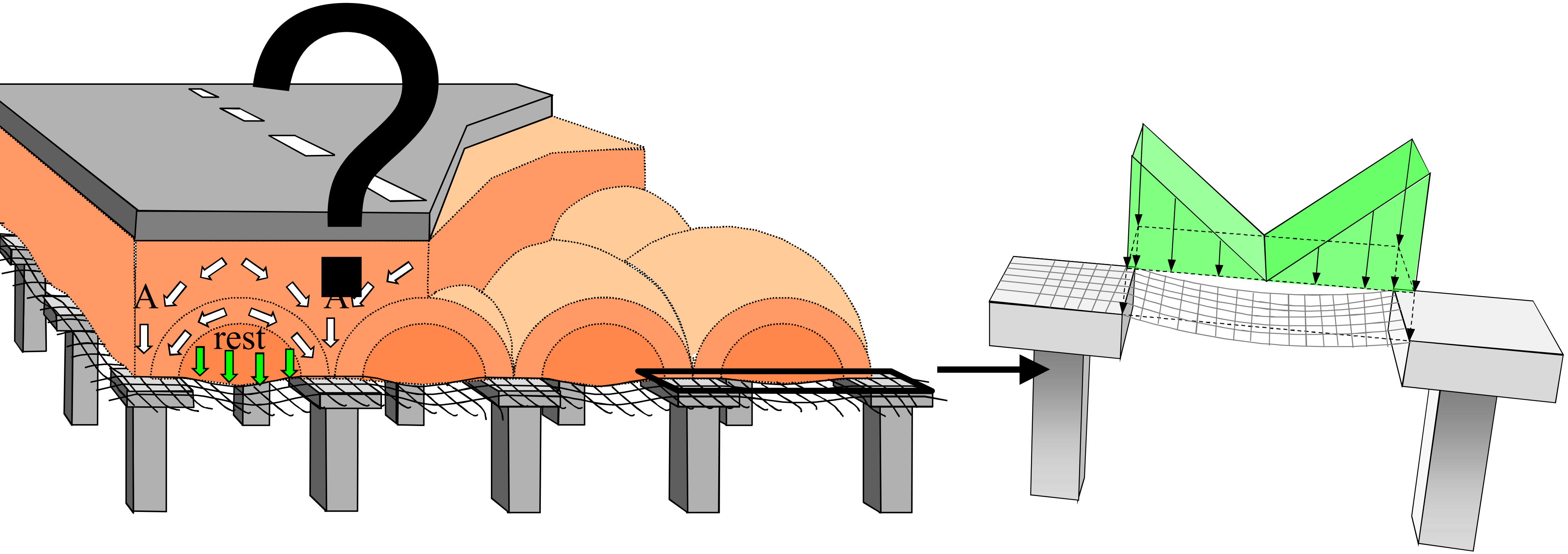
step 1 arching

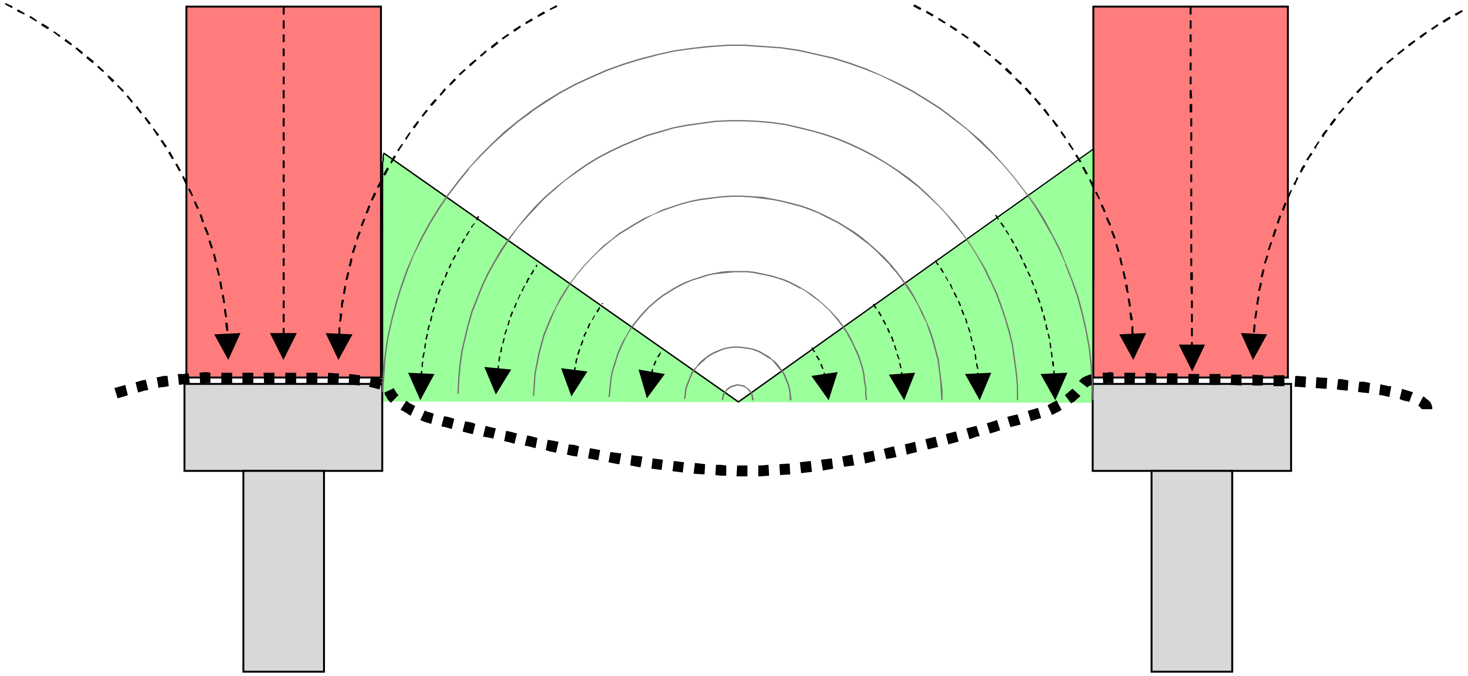
load part A

“rest”

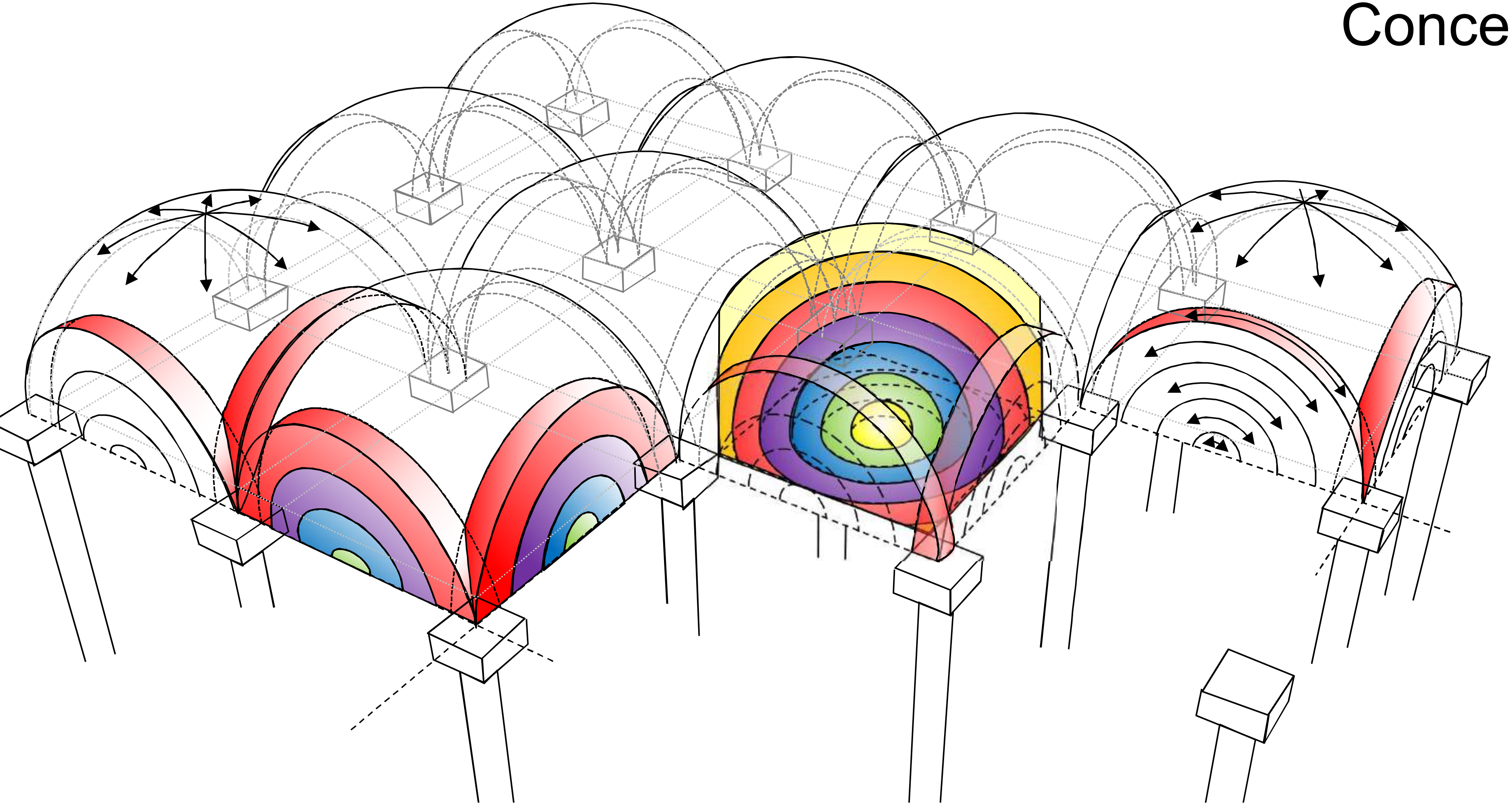
step 2

strain



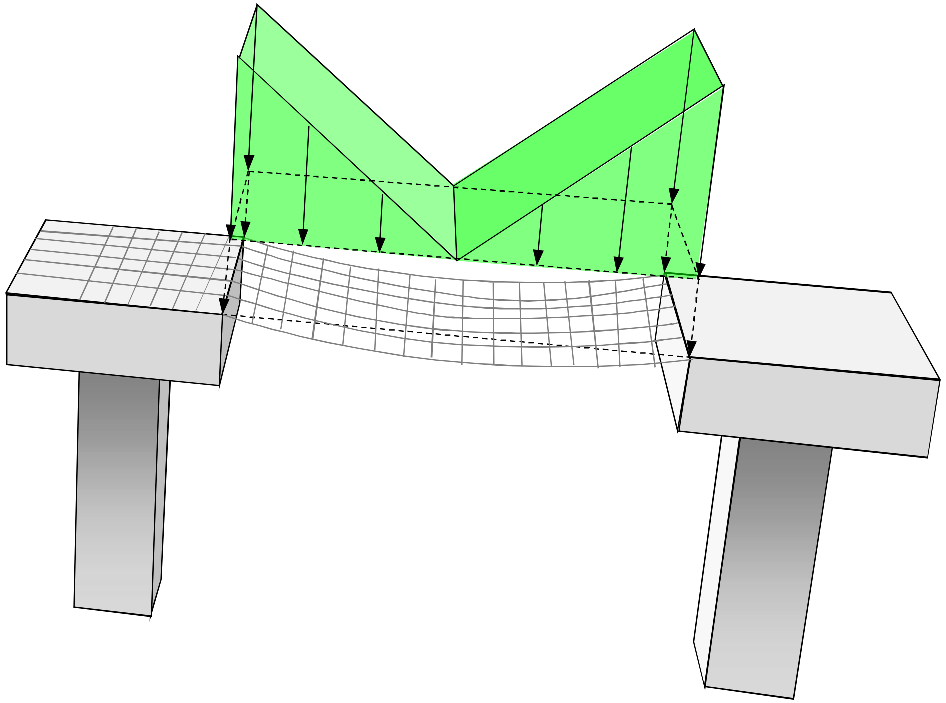


Concentric Arches Model

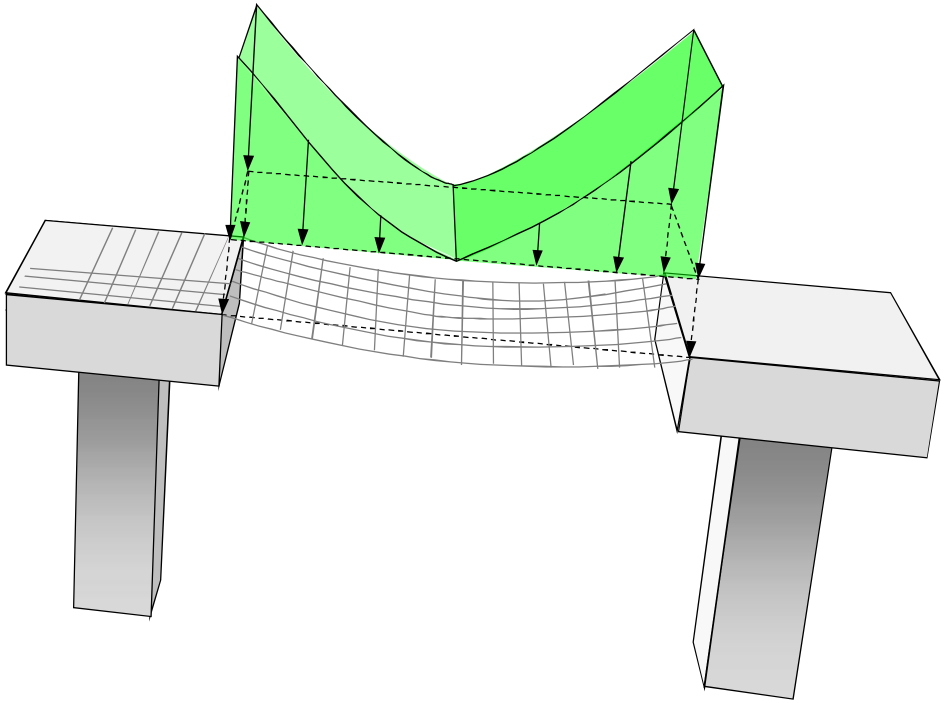


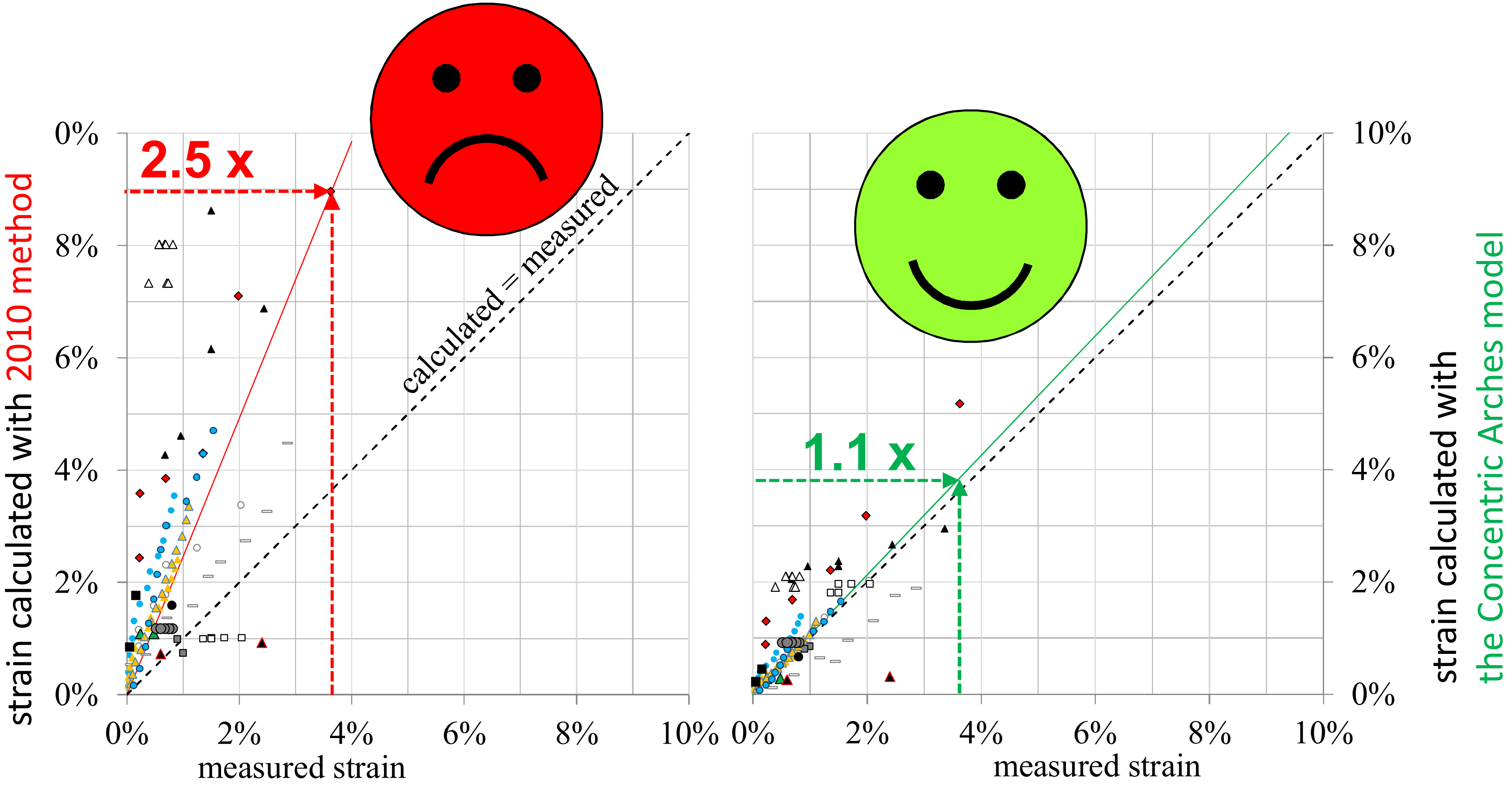
Excel sheet with equations: **www.piledembankments.com**

Observed:



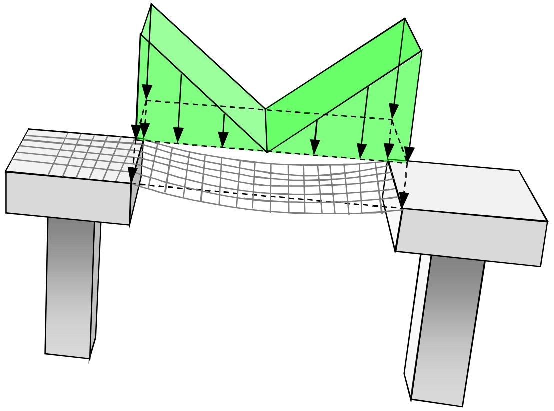
Calculated with the Concentric Arches model:




2.5 x
calculated = measured
strain calculated with 2010 method
0%
8%
6%
4%
2%
0%
0%
2%
4%
6%
8%
10%
measured strain
1.1 x
10%
8%
6%
4%
2%
0%
0%
2%
4%
6%
8%
10%
measured strain
strain calculated with
the Concentric Arches model

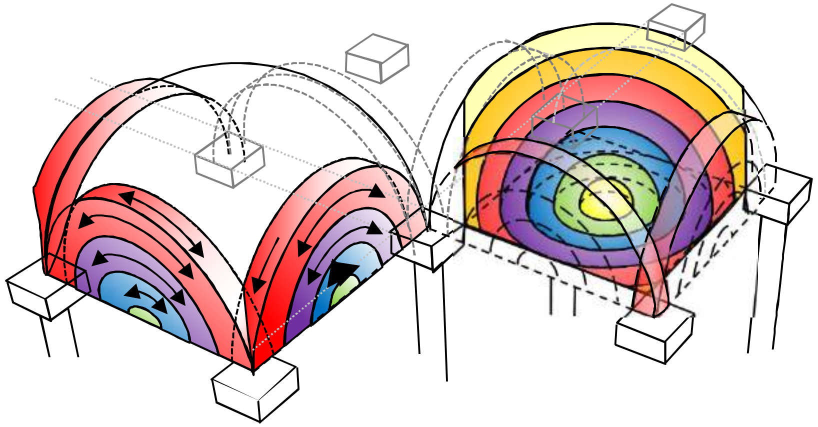
Conclusions

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 Embankents’

Jeroen Dijkstra
Cofra

Jacques Geel
Heijmans

Marco Peters
Grontmij

Chair
Suzanne
van Eekelen
Deltares

Reporter
Marijn Brugman
Arthe Civil &
Structure

Piet
van Duijnen
Geotec Solutions

Martin de Kant
RHDHV

Maarten Profittlich
Fugro

Daan Vink
CRUX
Engineering

Maarten
ter Linde
Strukton

Eelco Oskam
Movares

Lars Vollmert
Naue/BBG

SBR CUR net

Design Guideline
Basal Reinforced Piled Embankments

Editors: Suzanne J.M. van Eekelen & Marijn H.A. Brugman

CRC Press / Balkema

OBRIGADA!

Design Guideline
CRCpress.com or amazon.com

Free excel with the equations:
www.piledembankments.com

International course:
15/16 November in Delft, Netherlands
https://paotm.nl search for “basal”

Most important publications about this research:

CUR 226 (2016). S.J.M van Eekelen and M.H.A. Brugman, Eds. Design Guideline Basal Reinforced Piled Embankments.
SBRCURnet & CRC Press, ISBN 9789053676240,
https://www.crcpress.com/Design-Guideline-Basal-Reinforced-Piled-Embankments/Eekelen-Brugman/9789053676240

Van Eekelen, S.J.M. (2015). Basal Reinforced Piled Embankments. *PhD thesis Technical University of Delft, Netherlands. ISBN 978-94-6203-825-7 (print), ISBN 978-94-6203-826-4 (electronic version). Downloadable at: www.piledembankments.com, incl. an excel calculation file.*
This PhD thesis include:

- Van Eekelen, S.J.M., Bezuijen, A., Lodder, H.J., van Tol, A.F. (2012a). Model experiments on piled embankments Part I. *Geotextiles and Geomembranes 32: 69 - 81.*
- Van Eekelen, S.J.M., Bezuijen, A., Lodder, H.J., van Tol, A.F. (2012b). Model experiments on piled embankments. Part II. *Geotextiles and Geomembranes 32: 82 – 94.*
- Van Eekelen, S.J.M., Bezuijen, A., Van Tol, A.F. (2013). An analytical model for arching in piled embankments. *Geotextiles and Geomembranes 39: 78 – 102.*
- Van Eekelen, S.J.M., Bezuijen, A. van Tol, A.F. (2015). Validation of analytical models for the design of basal reinforced piled embankments. *Geotextiles and Geomembranes. 43:1, 56 - 81.*

Van Eekelen, S.J.M. (2016). The 2016-update of the Dutch Design Guideline for Basal Reinforced Piled Embankments.
In: Proc. of ICTG3, Portugal.

Van Eekelen, S.J.M. and Venmans, A.A.M. (2016). Piled embankment or a traditional sand construction: how to decide? A case study.
In: Proc. of ICTG3, Portugal.

6

3rd ICTG 2016

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Workshop 1 – Geosynthetics in Transportation Geotechnics

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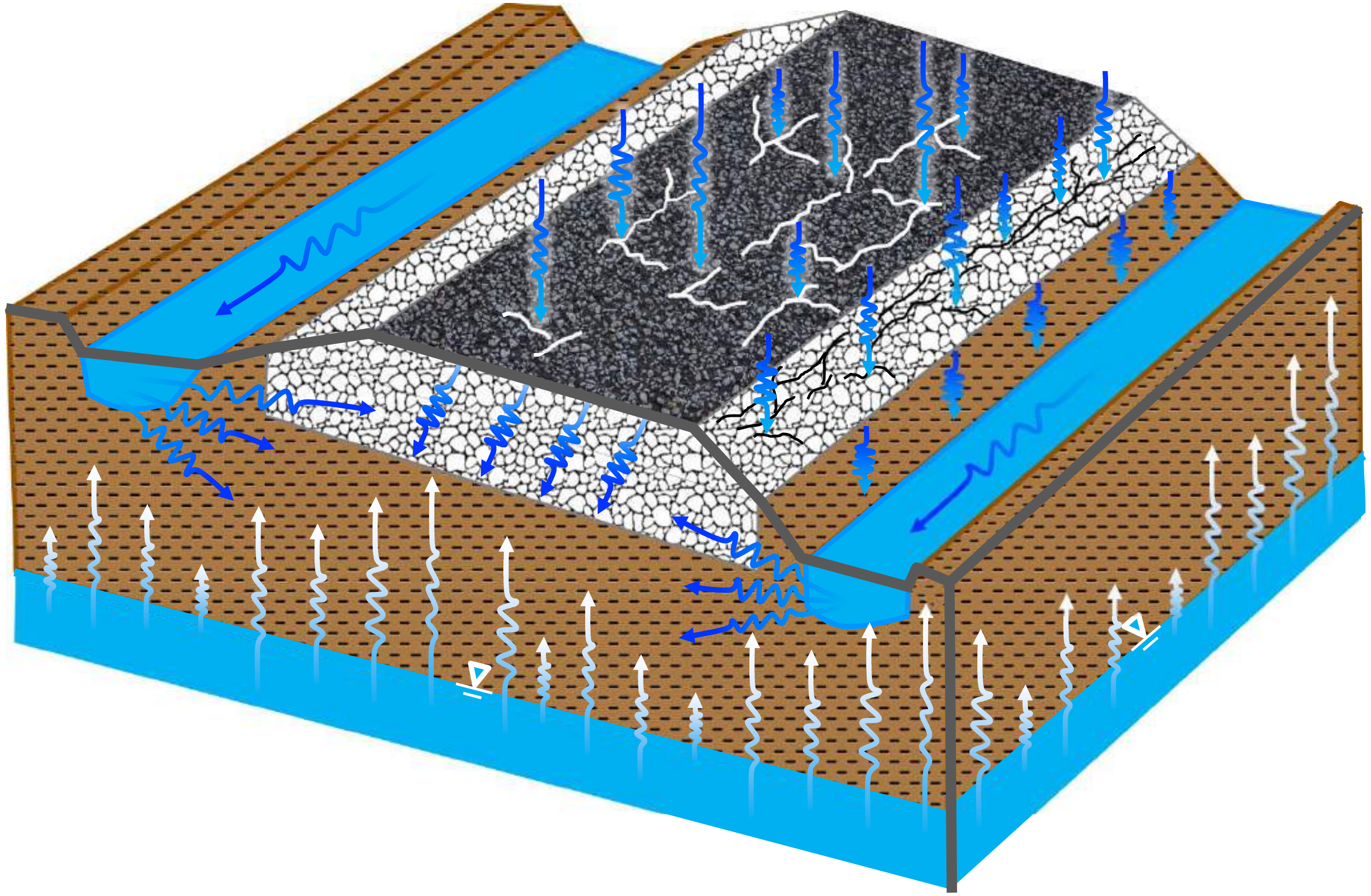
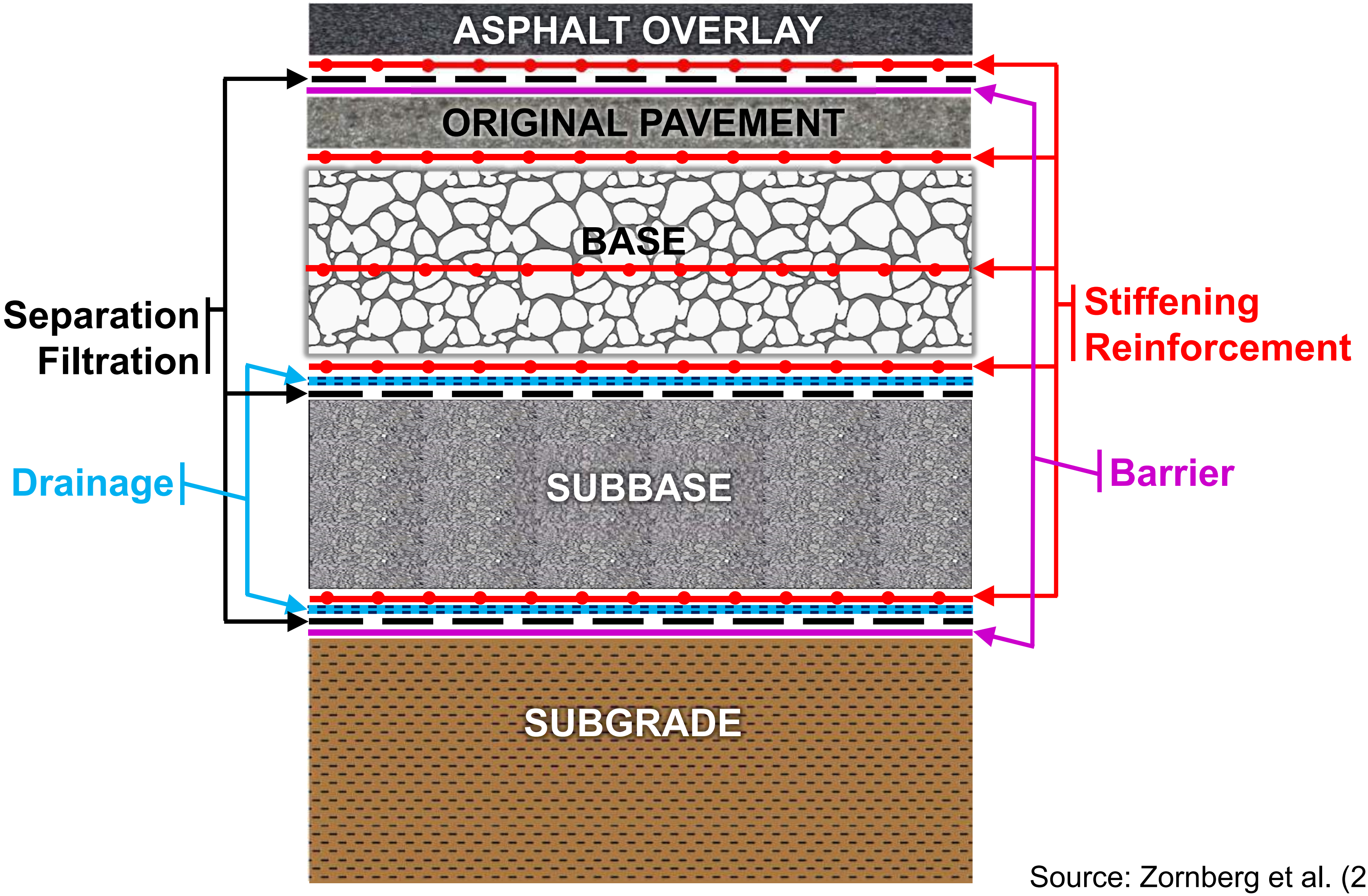
Geosynthetics with Enhanced Lateral Drainage Capabilities in Roadway Systems

Jorge G. Zornberg[1], Marcelo Azevedo[1],
Mark Sikkema[2], and Brett Odgers[2]

1. *The University of Texas at Austin, United States of America*
2. *TenCate Geosynthetics*



Source: Zornberg et al. (2016)



Source: Zornberg et al. (2016)

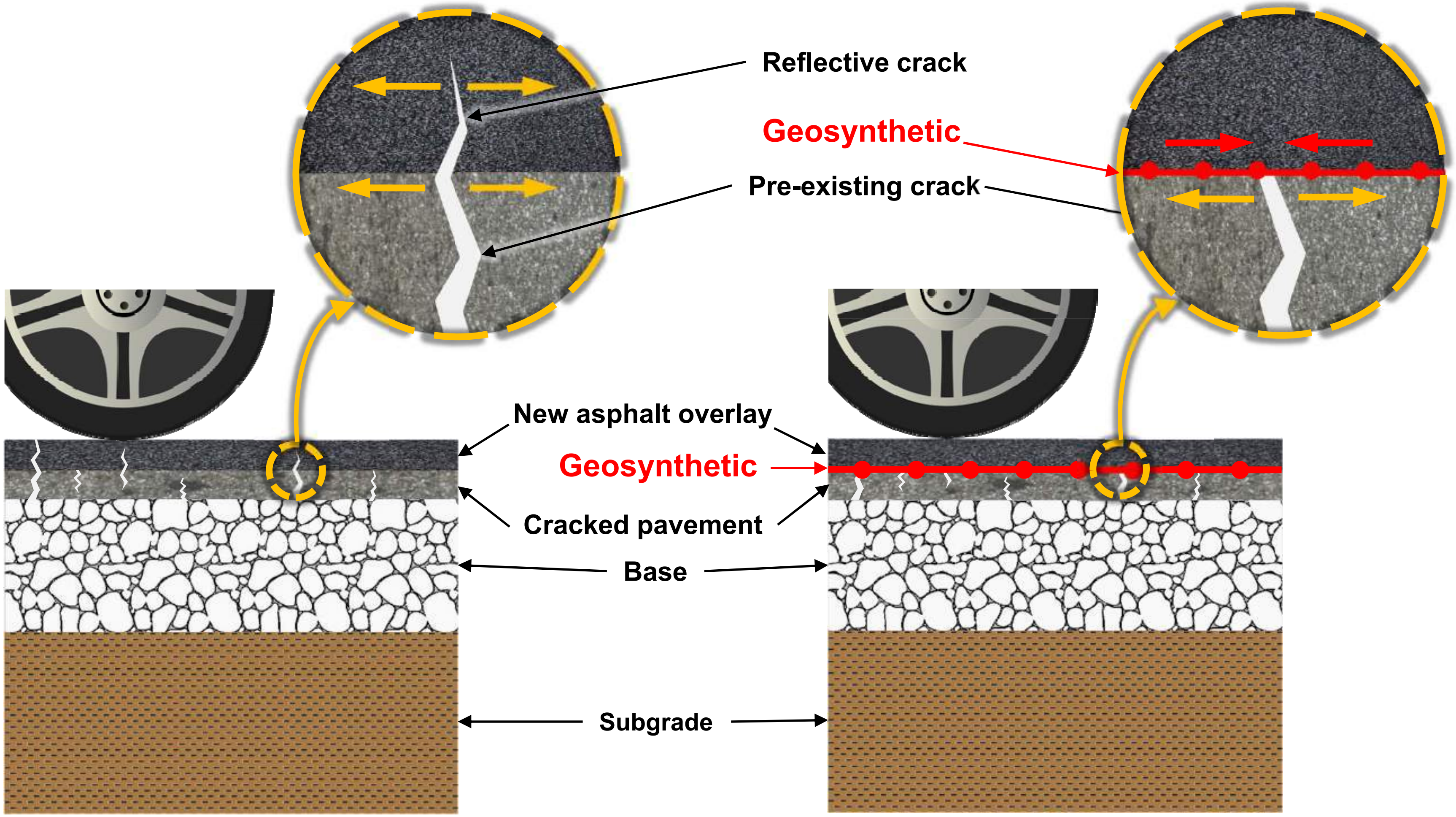
Geosynthetics in Roadway Systems

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

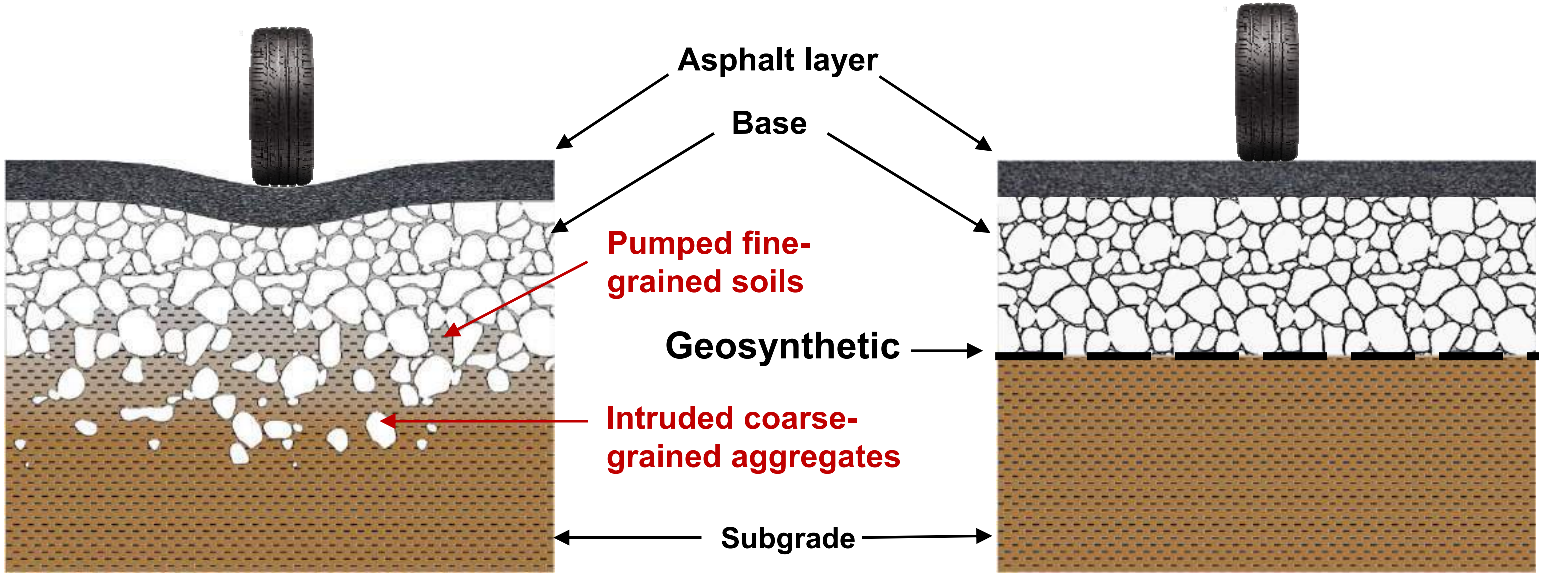
5 pavement applications, involving 1 or more geosynthetic functions each

Mitigation of Reflective Cracking



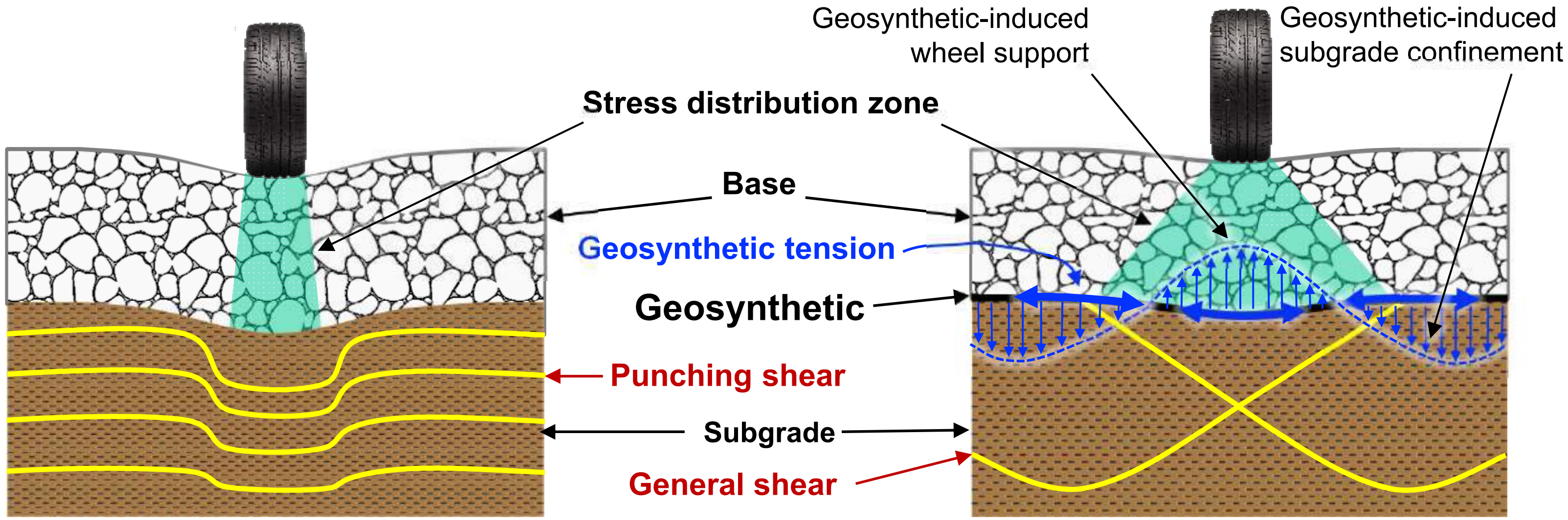
Source: Zornberg et al. (2016)

Separation Application



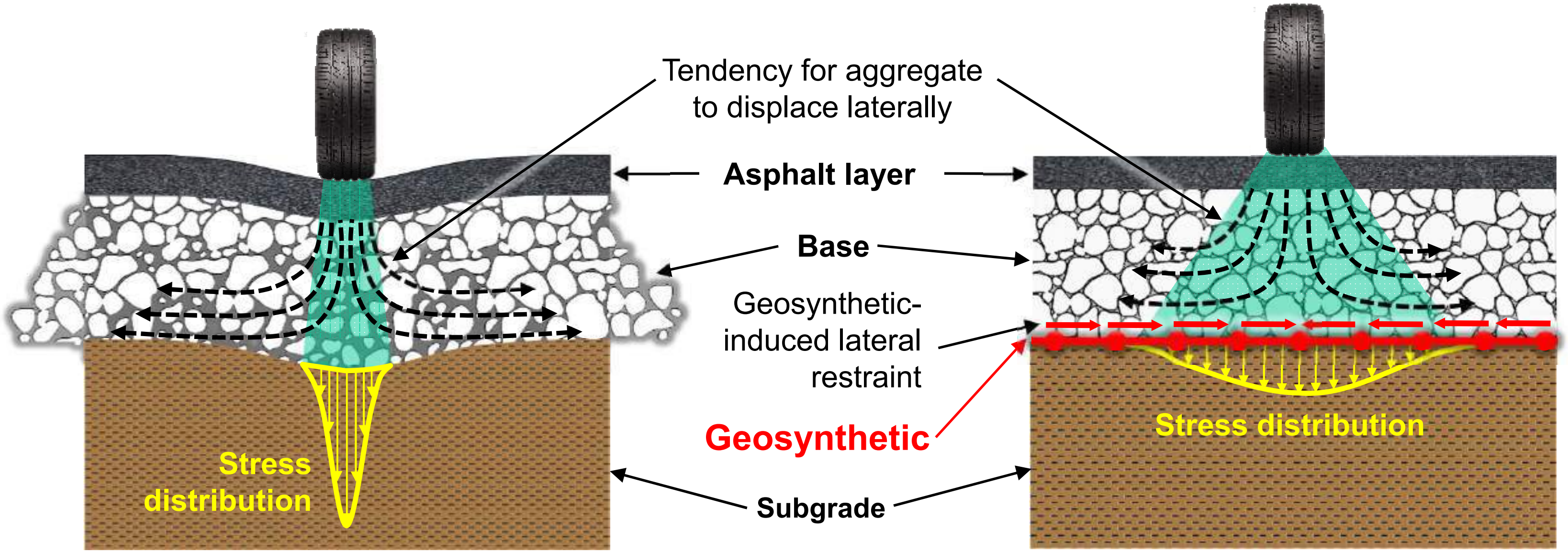
Source: Zornberg et al. (2016)

Stabilization of Road Subgrades



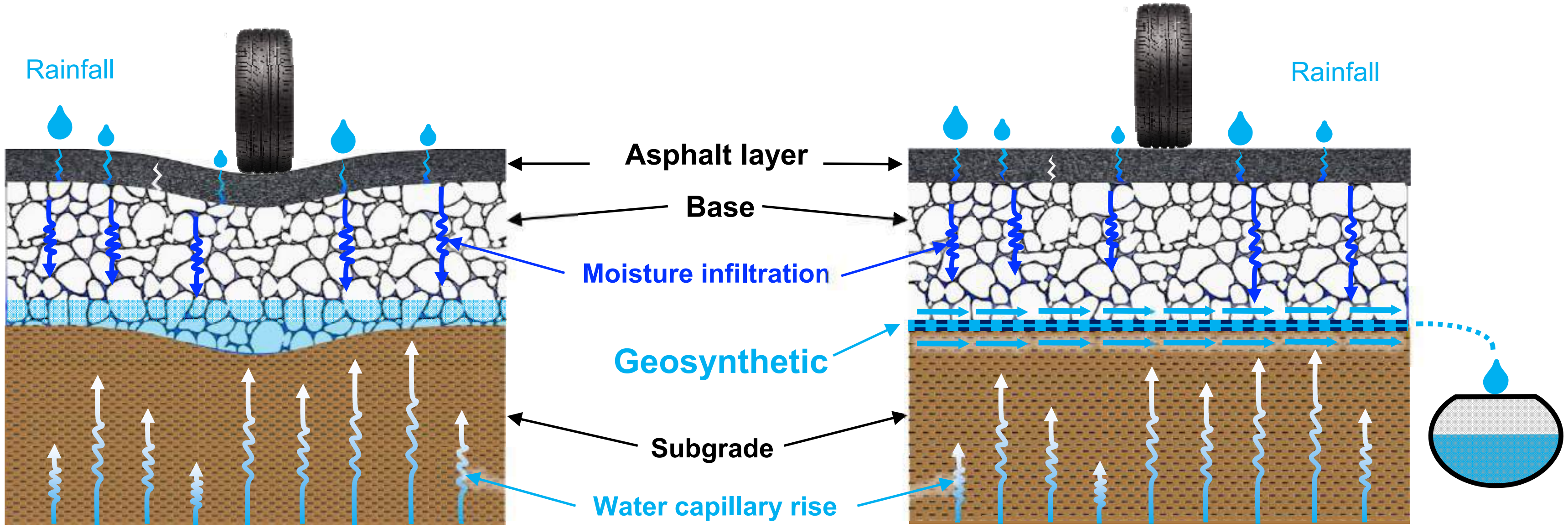
Source: Zornberg et al. (2016)

Stabilization of Road Bases



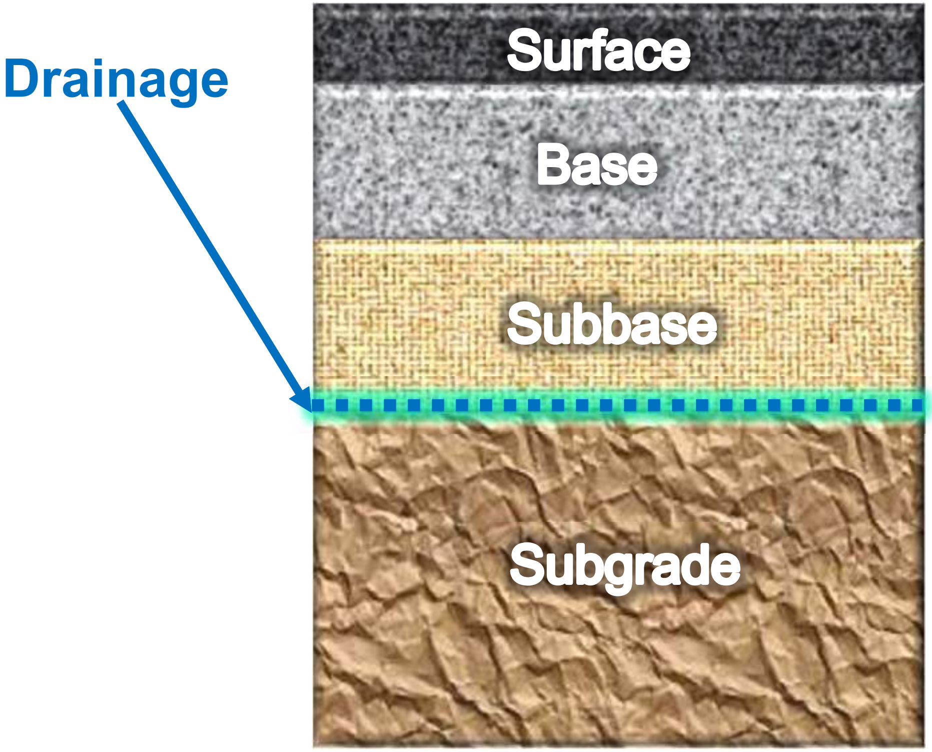
Source: Zornberg et al. (2016)

Geosynthetics for Improved Drainage



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)

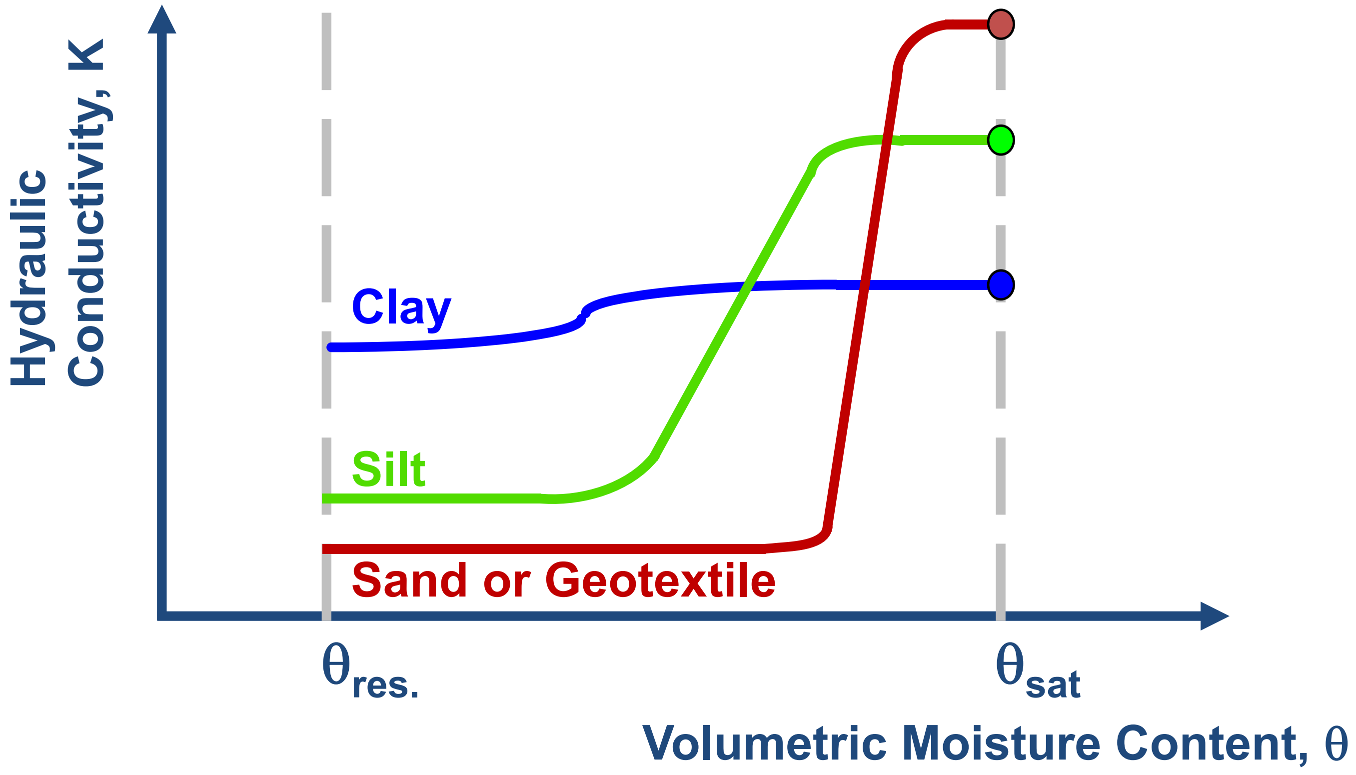
Impact of Drainage on Pavement Design

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

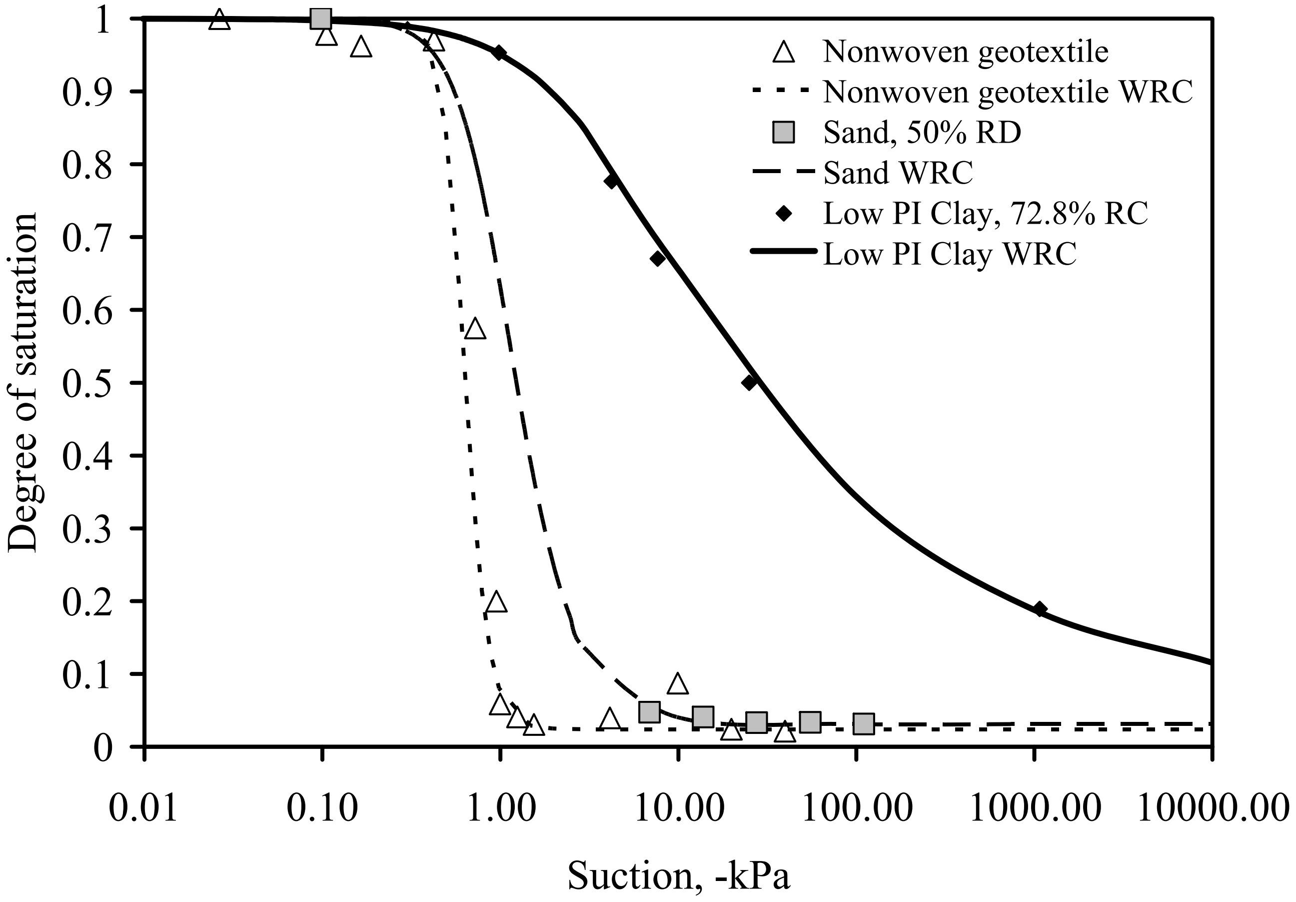
	% Time Saturated			
Quality	< 1%	1 -5 %	5 - 25%	> 25%
Excellent	1.40 - 1.35	1.35 - 1.30	1.30 - 1.20	1.20
Good	1.35 - 1.25	1.25 - 1.15	1.15 - 1.00	1.00
Fair	1.25 - 1.15	1.15 - 1.05	1.05 - 0.80	0.80
Poor	1.15 - 1.05	1.05 - 0.80	0.80 - 0.60	0.60
Very Poor	1.05 - 0.95	0.95 - 0.75	0.75 - 0.40	0.40

Some Bad News:
Drains not Always Drain

Unsaturated Geomaterials Behavior

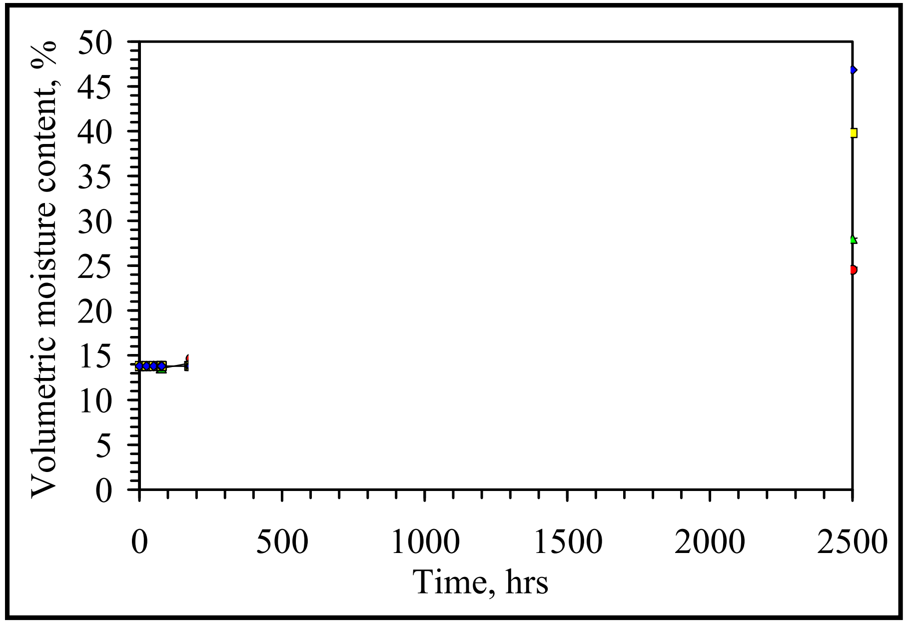


Water Retention Curve (WRC)

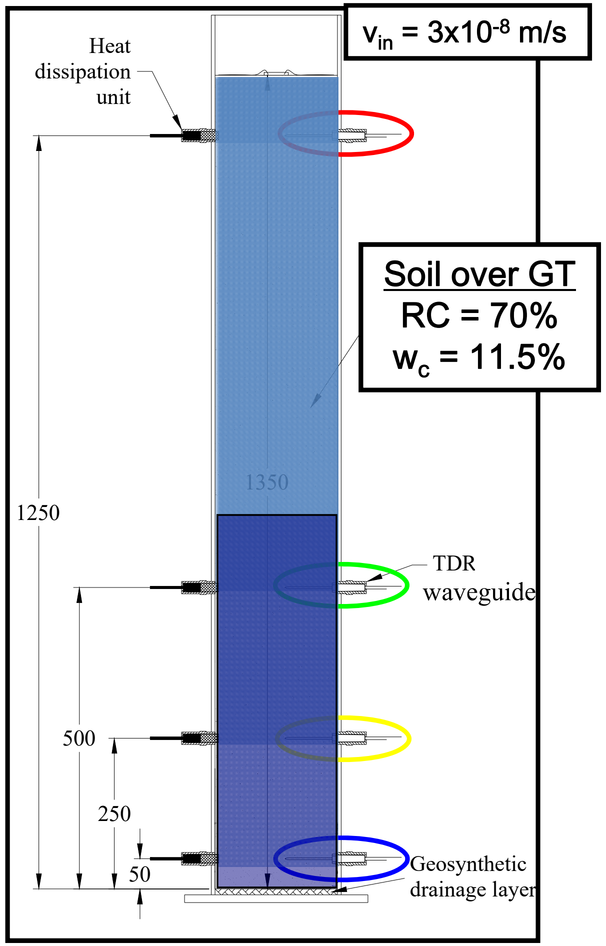


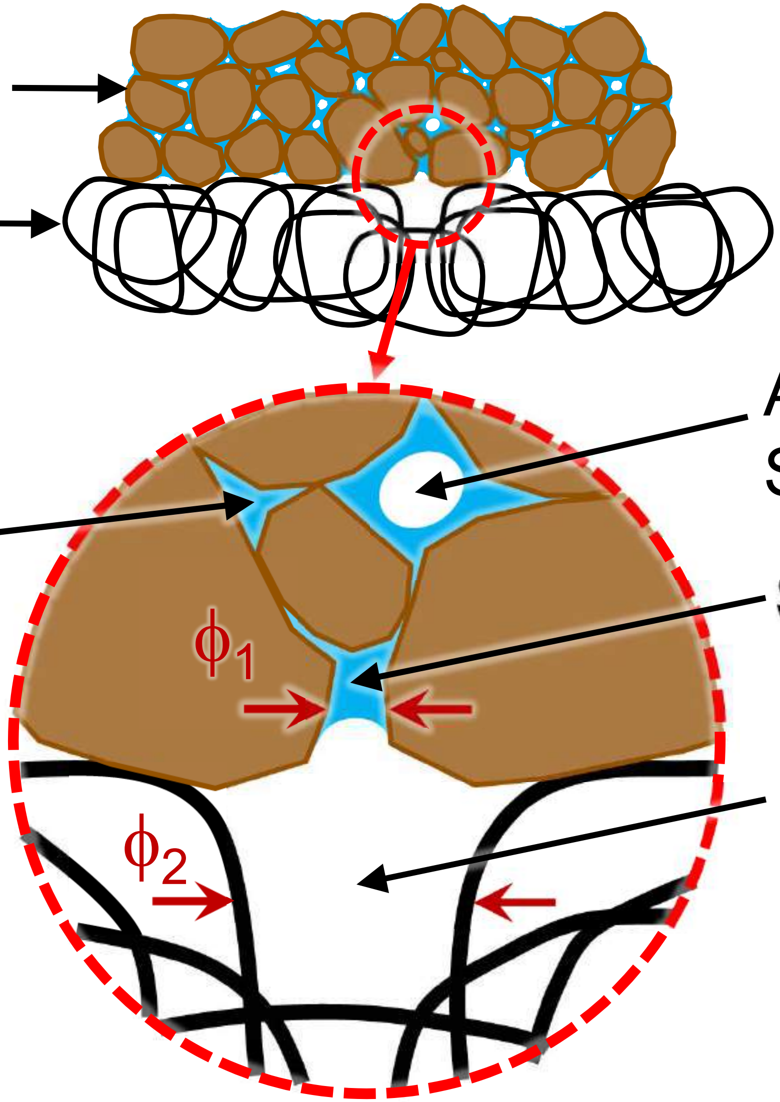
(McCartney, Zornberg, and Kuhn 2005)

Column Test Studies



(McCartney, Zornberg, and Kuhn 2005)





Soil Particle

Geotextile Fiber

Water in Soil Pore

Air in Soil Pore

Soil Pore

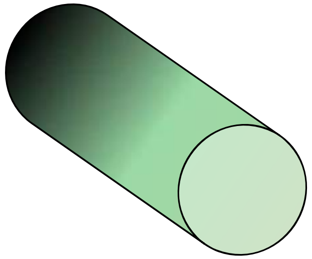
Geotextile Pore

Source: Zornberg et al. (2016)

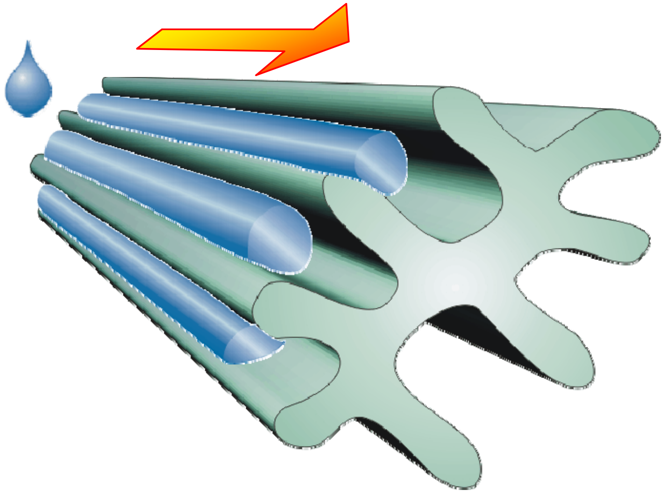
Now the Good News: Geosynthetics can be engineered to provide Enhanced Drainage

Enhanced Lateral Drainage

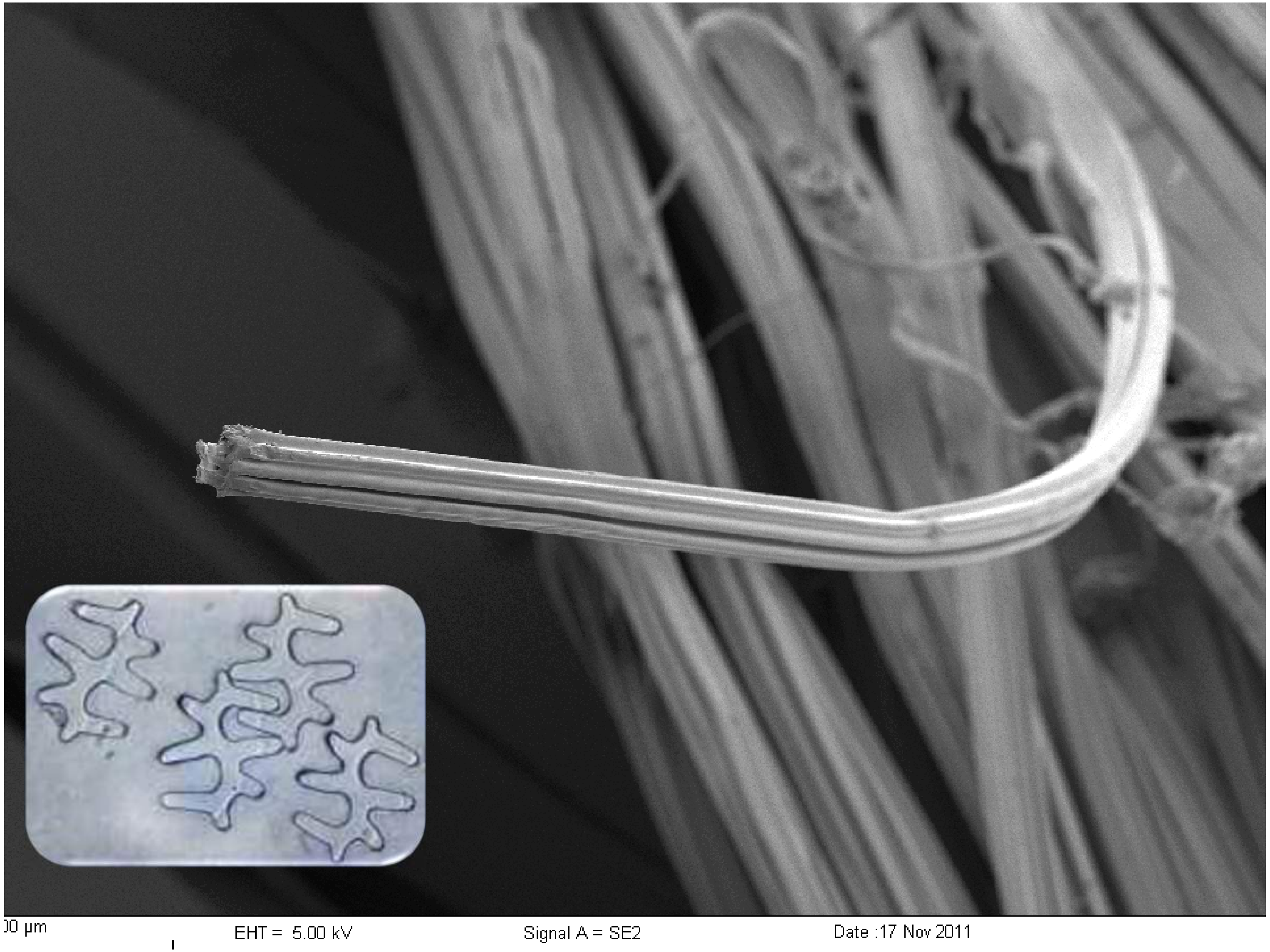
- Conventional geotextiles provide in-plane drainage **after saturation of the soil-geotextile interface:**
  - In Non-Woven Geotextiles: Through the large void spaces in its open structure
  - In Woven Geotextiles: Through void spaces of crossed-over yarns
- **Enhanced Lateral Drainage** involves providing additional in-plane drainage capacity that is mobilized due to suction gradients (or “wicking”) within the geotextile yarns.

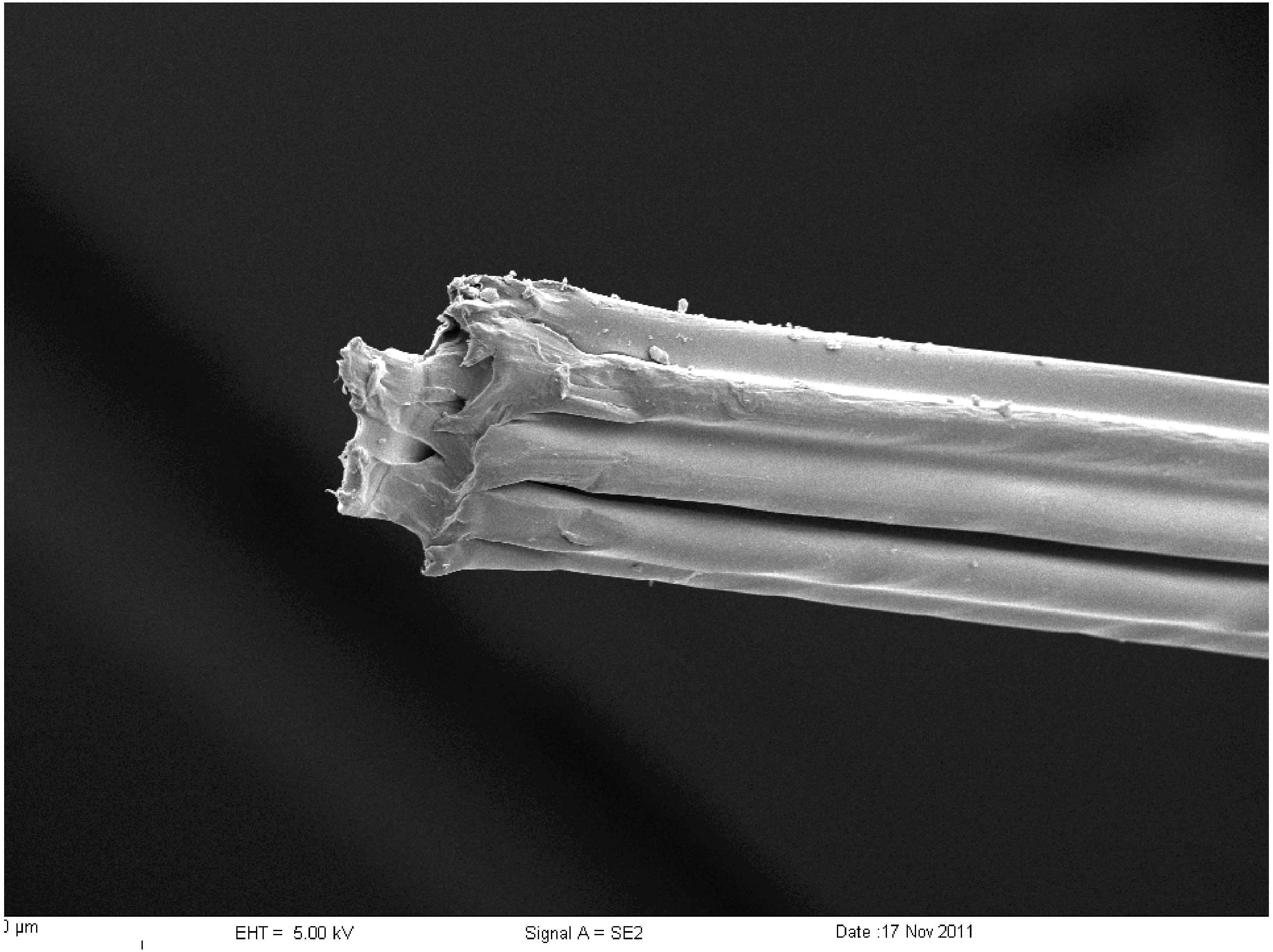


Conventional geotextile fiber



“Wicking” fiber with engineered cross-section to increase specific surface


0 µm
EHT = 5.00 kV
Signal A = SE2
Date :17 Nov 2011


µm
EHT = 5.00 kV
Signal A = SE2
Date :17 Nov 2011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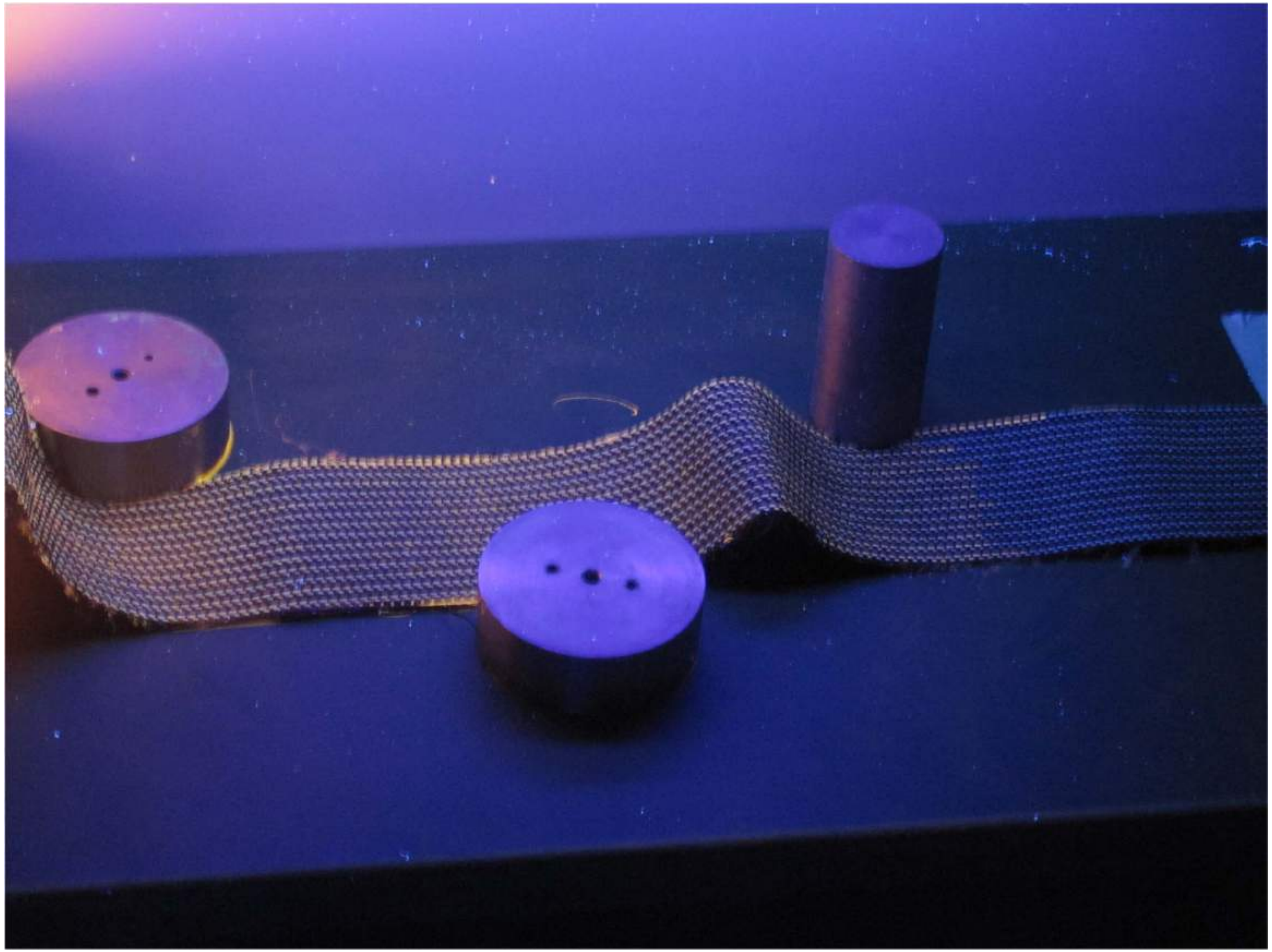
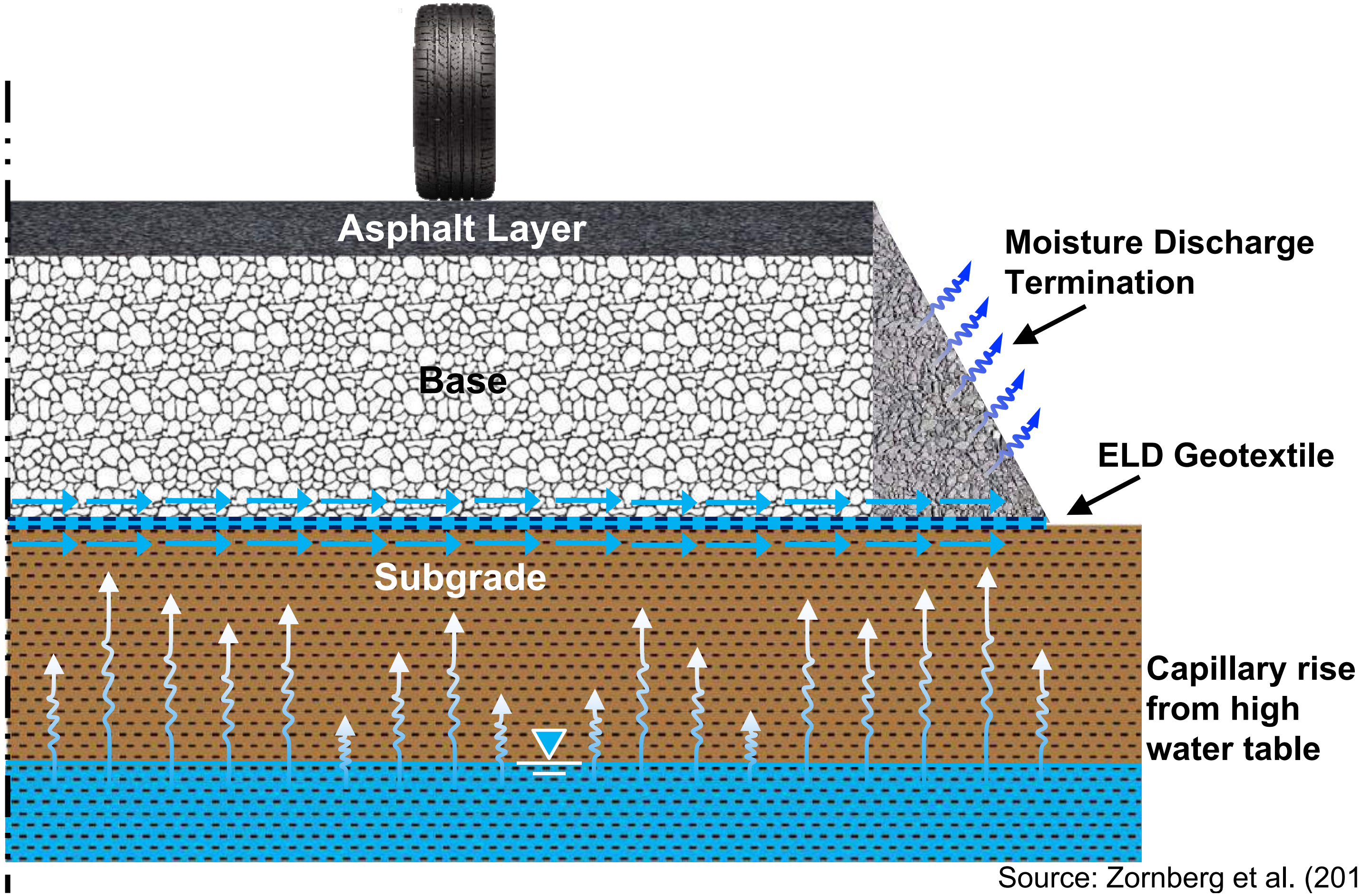


1. Enhanced Lateral Drainage of Moisture Migrating Upward from a High Water Table

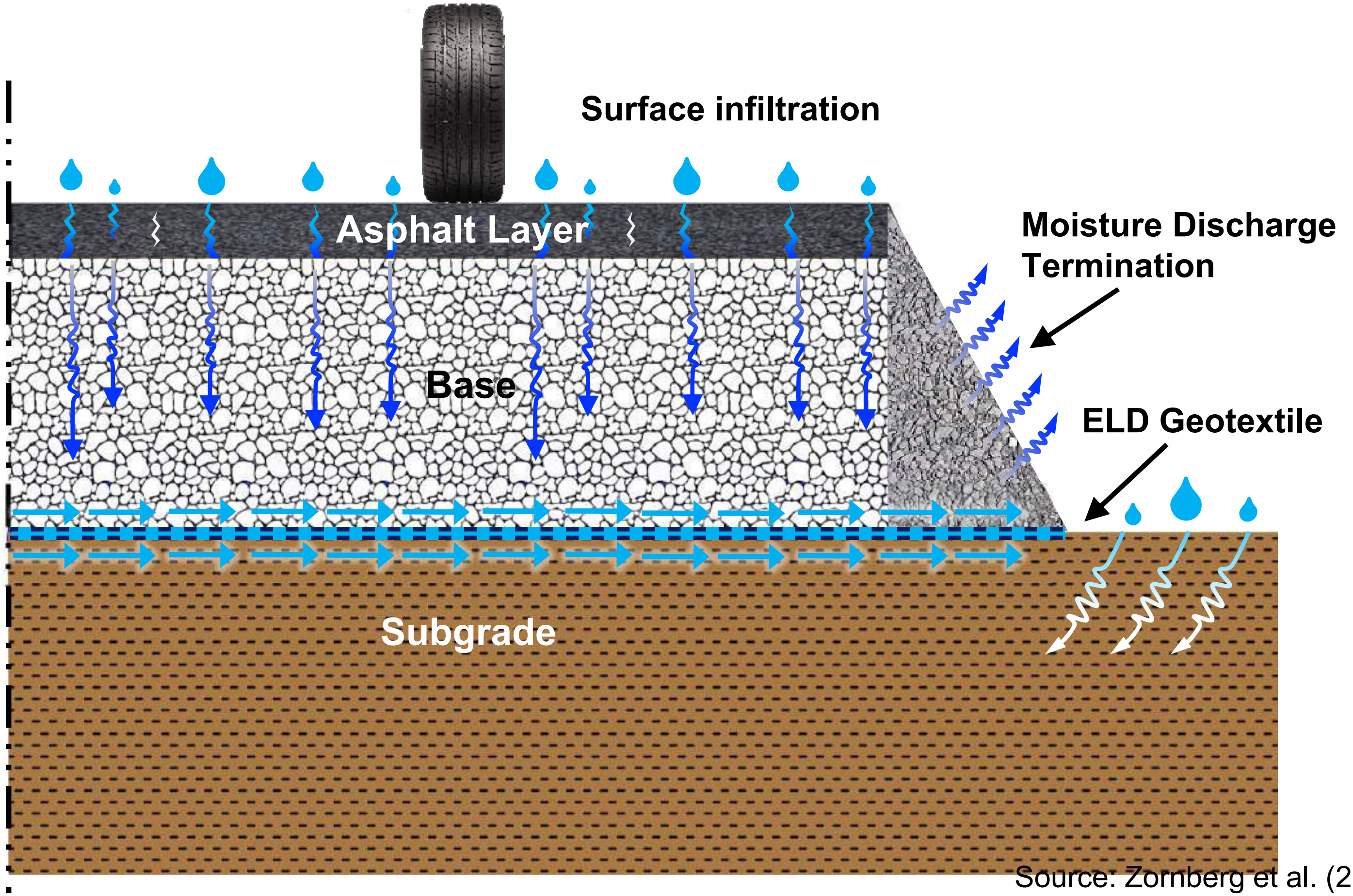


Source: Zornberg et al. (2016)

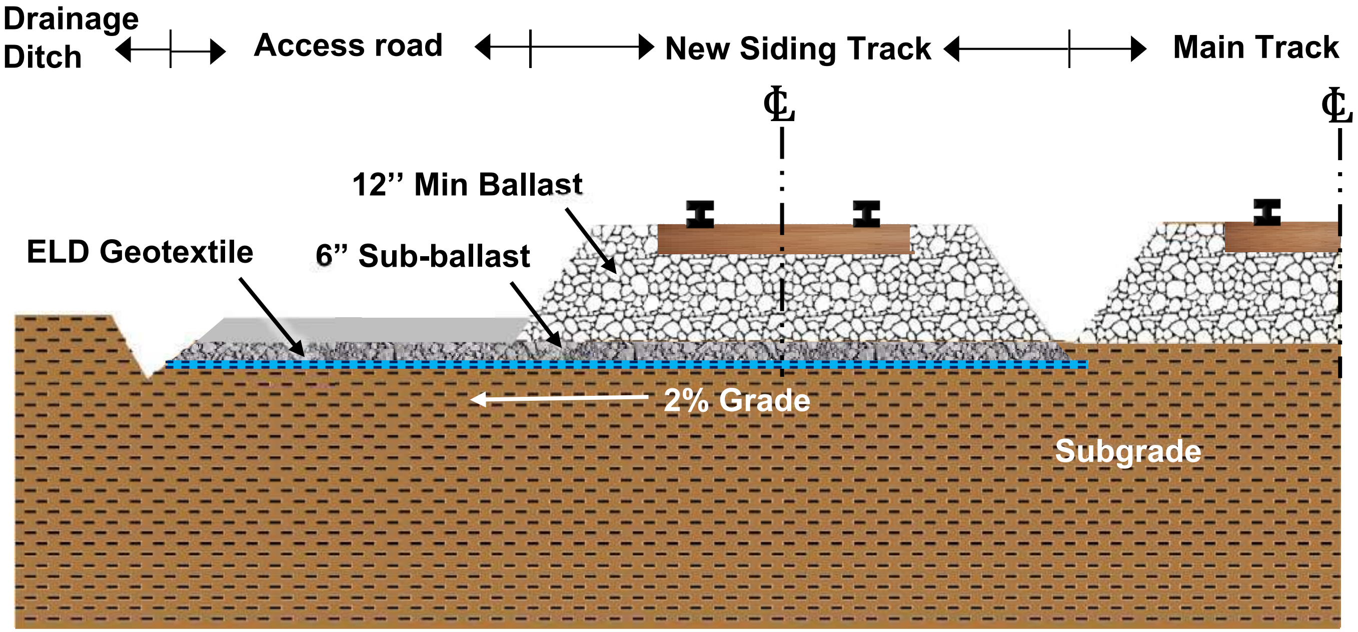
Daniel Boone Bridge, Missouri, USA

Source: Zornberg et al. (2016)

2. Enhanced Lateral Drainage of Moisture Migrating Downward from the Surface



Source: Zornberg et al. (2016)



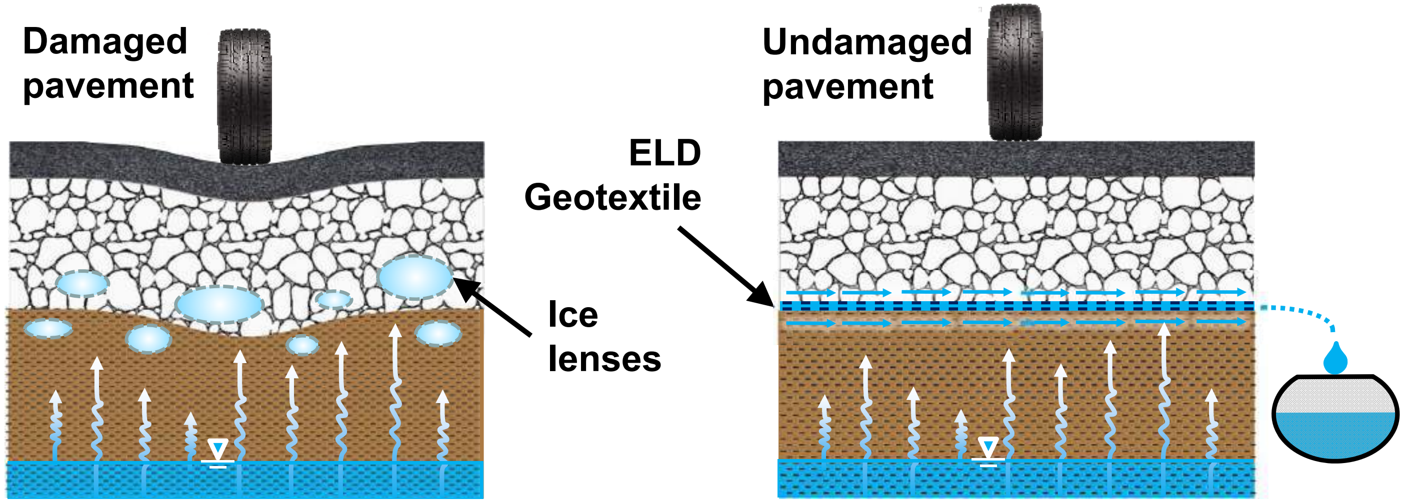
Garwood Railroad Sliding, Idaho, USA

Source: Zornberg et al. (2016)

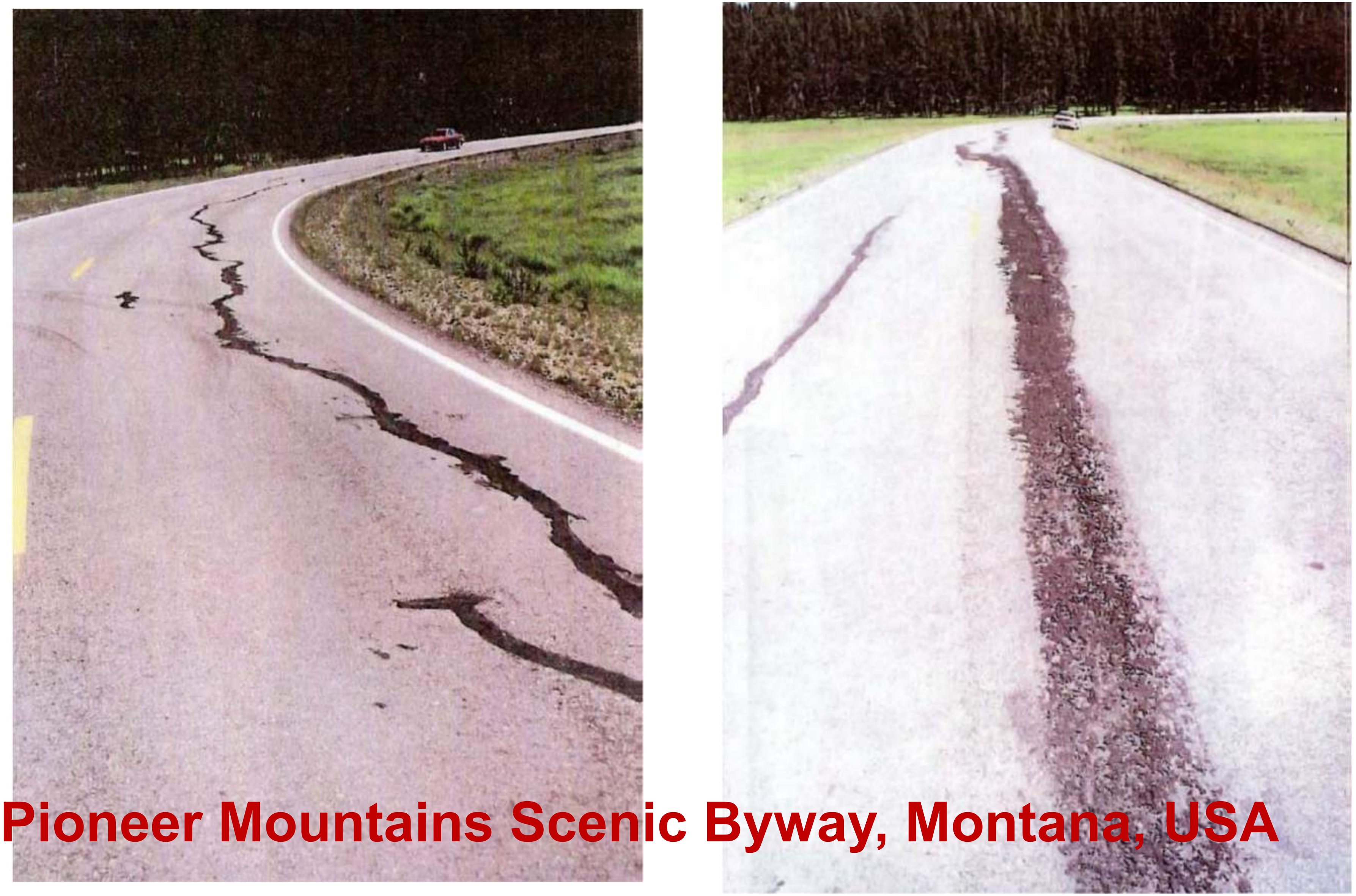
Garwood Railroad Sliding, Idaho, USA

Source: Zornberg et al. (2016)

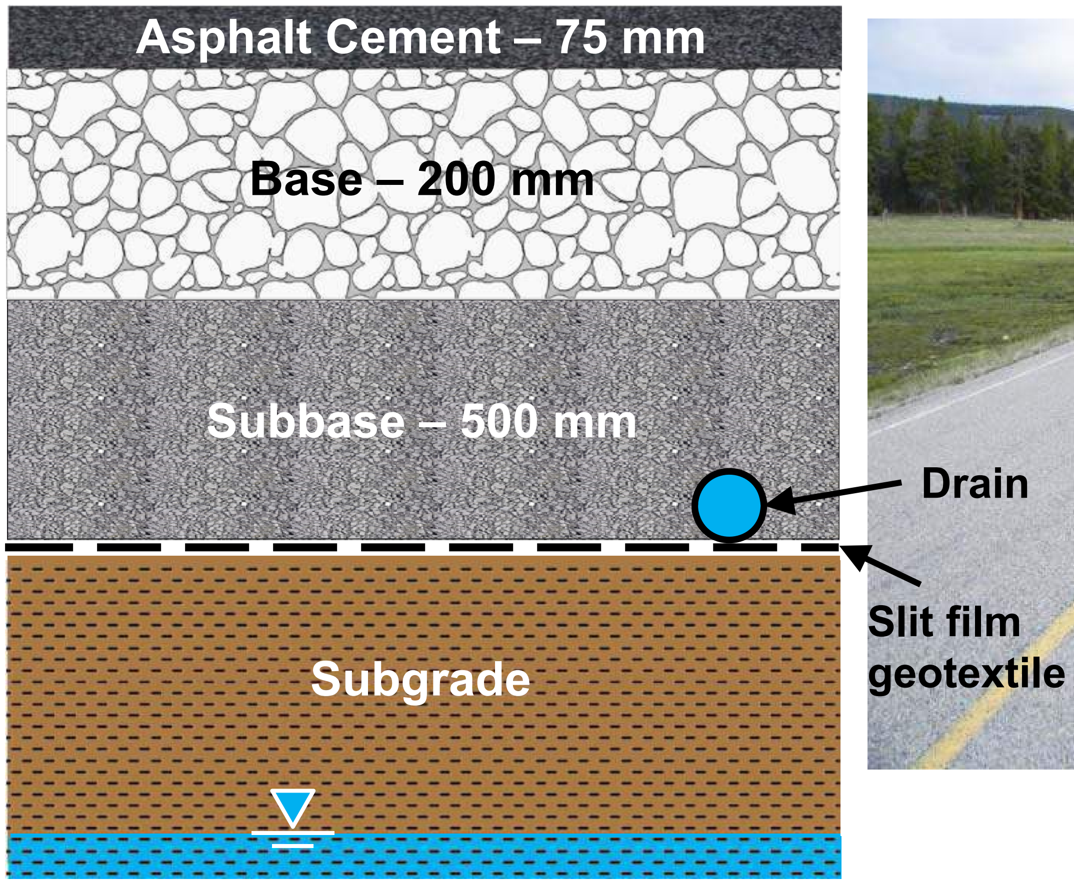
3. Control of Pavement Damage Caused by Frost Heave



Source: Zornberg et al. (2016)

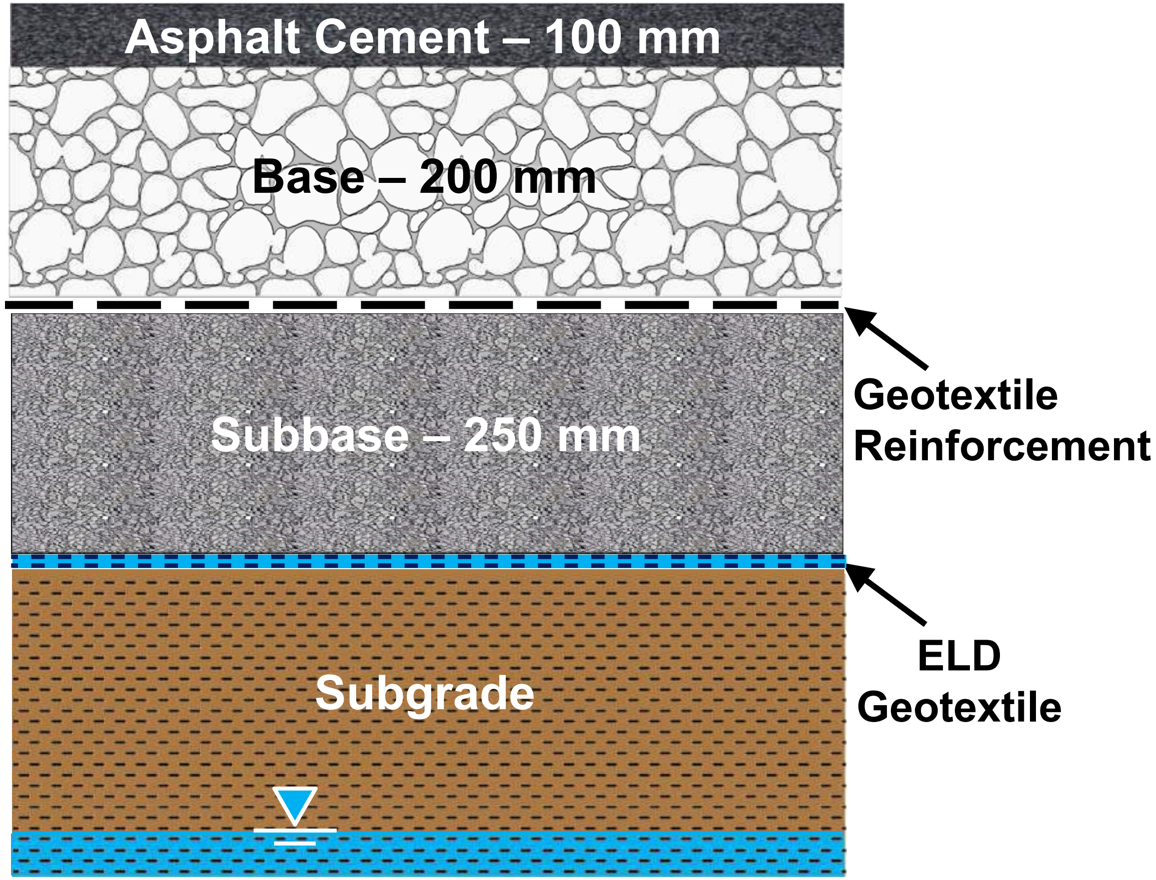


Pioneer Mountains Scenic Byway, Montana, USA



Pioneer Mountains Scenic Byway, Montana, USA

Source: Zornberg et al. (2016)



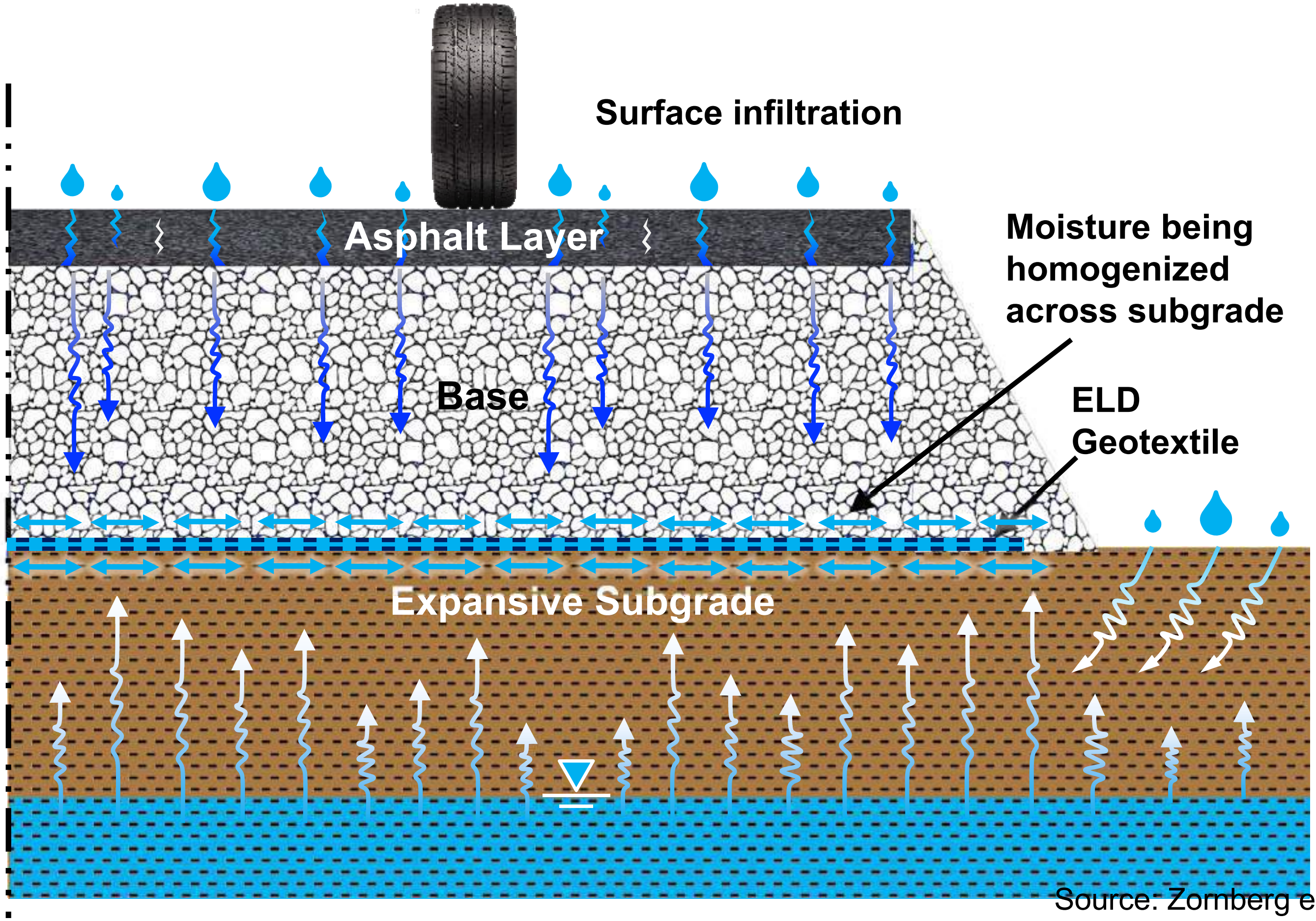
Pioneer Mountains Scenic Byway, Montana, USA

Source: Zornberg et al. (2016)

Pioneer Mountains Scenic Byway, Montana, USA

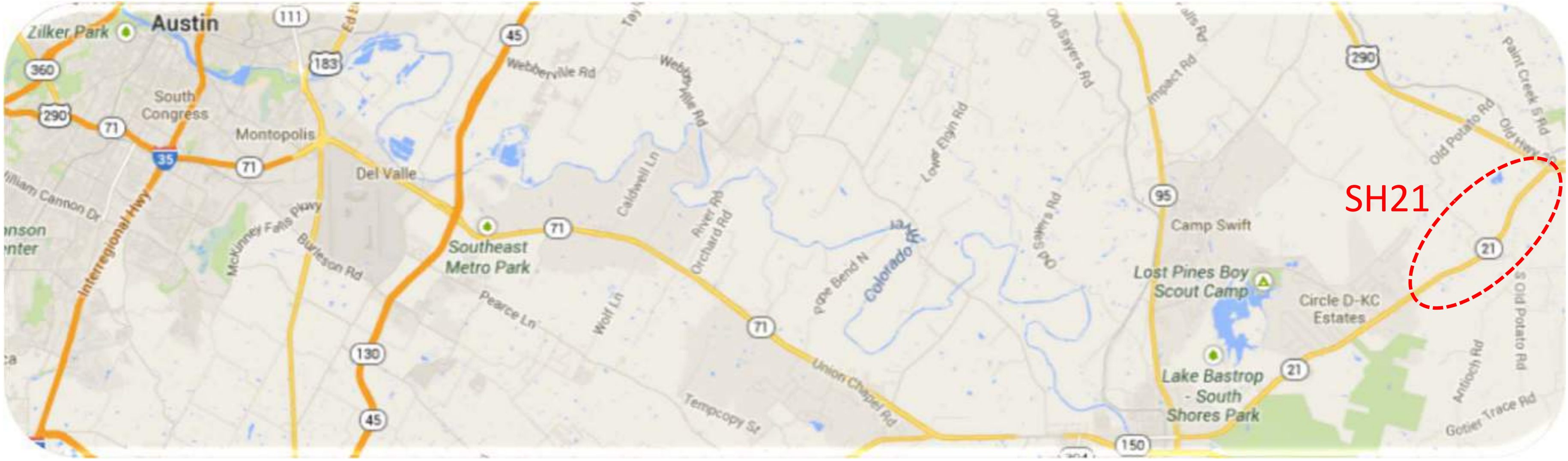
Source: Zornberg et al. (2016)

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

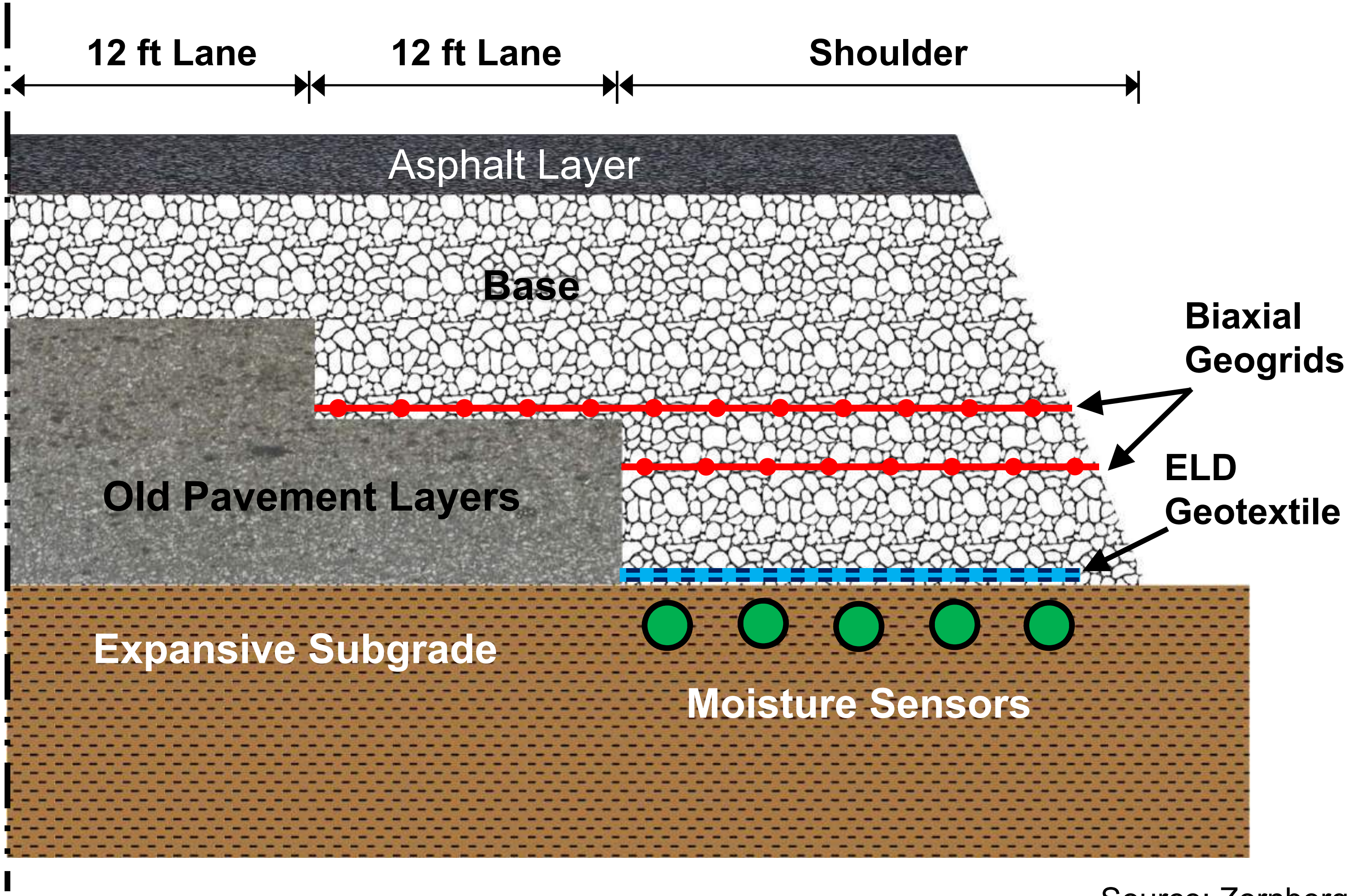
- The main distresses observed included major longitudinal and edge cracking, vertical deformation, rutting, and faulting.

- 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)



Source: Zornberg et al. (2016)

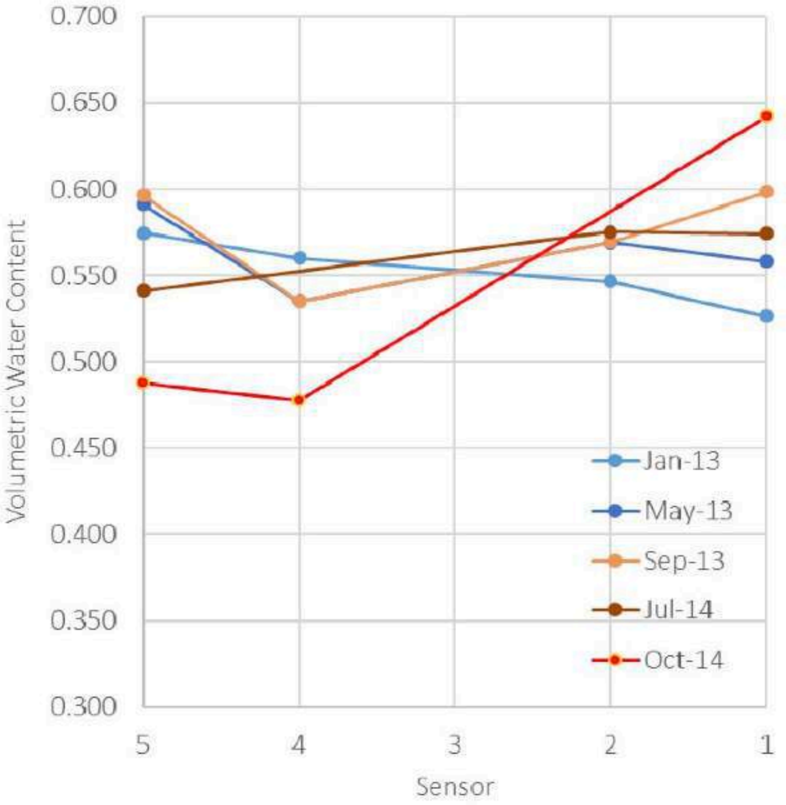
Control of Differential Settlements over Expansive Clays, SH21, Texas, USA

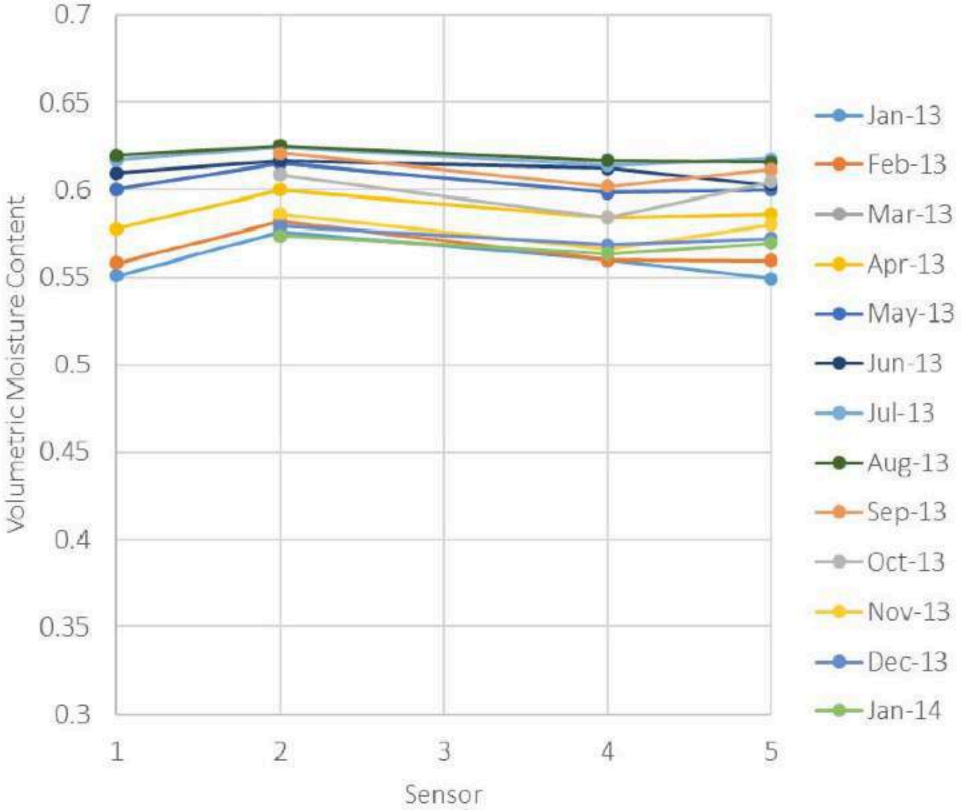
- A series of moisture and temperature sensors were installed beneath the geotextile within the subgrades soil.

- 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)

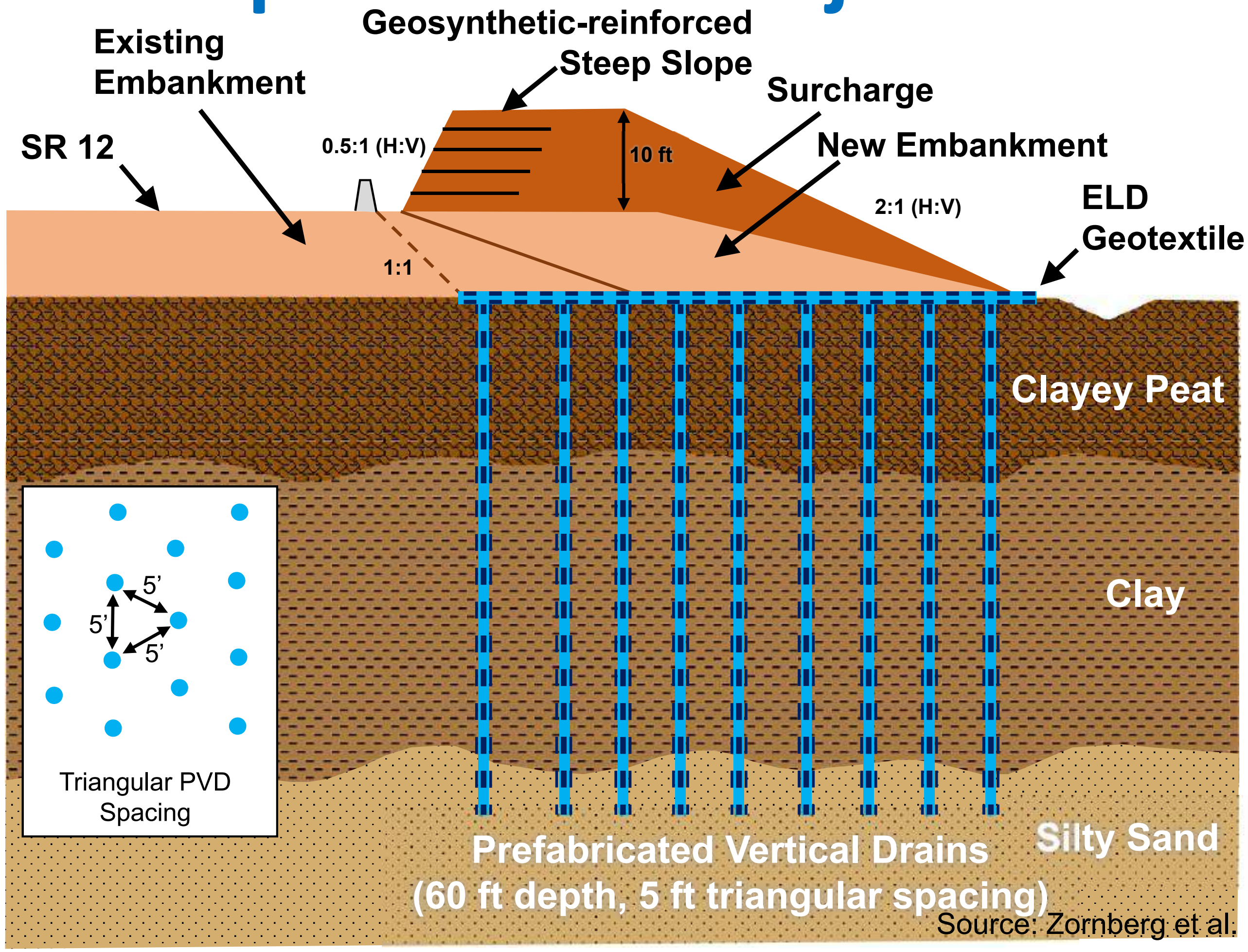

Section with
ELD Geotextile
Control Section





Source: Zornberg et al. (2016)

5. Enhanced Lateral Drainage in Soil Improvement Projects



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!

7

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4-7 September 2016, Guimarães, Portugal

Universidade do Minho
Escola de Engenharia

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Workshop 1 – Geosynthetics in Transportation Geotechnics

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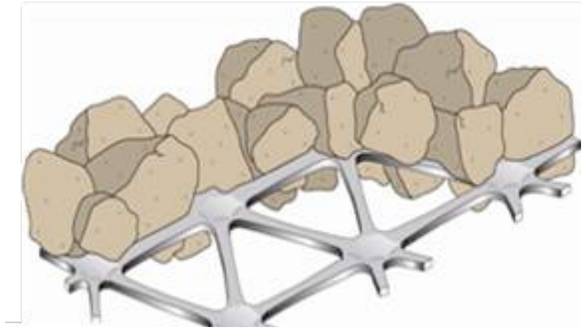
Effect of Geogrid on Railroad Ballast Particle Movement

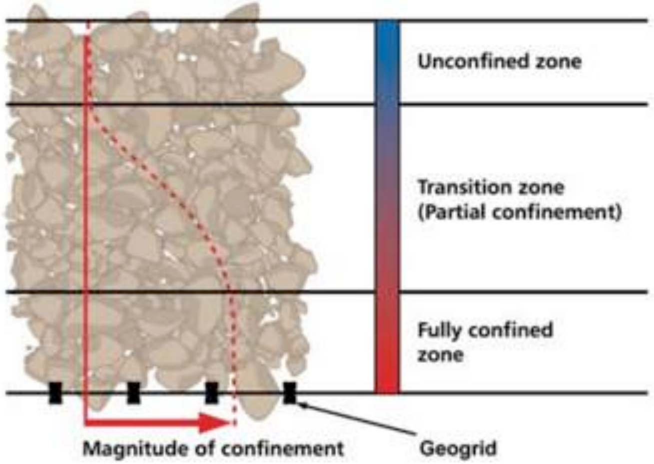
Shushu Liu[1], **Hai Huang**[1], **Tong Qiu**[1], **Jayhyun Kwon**[2]

1. The Pennsylvania State University
2. Tensar International Incorporation

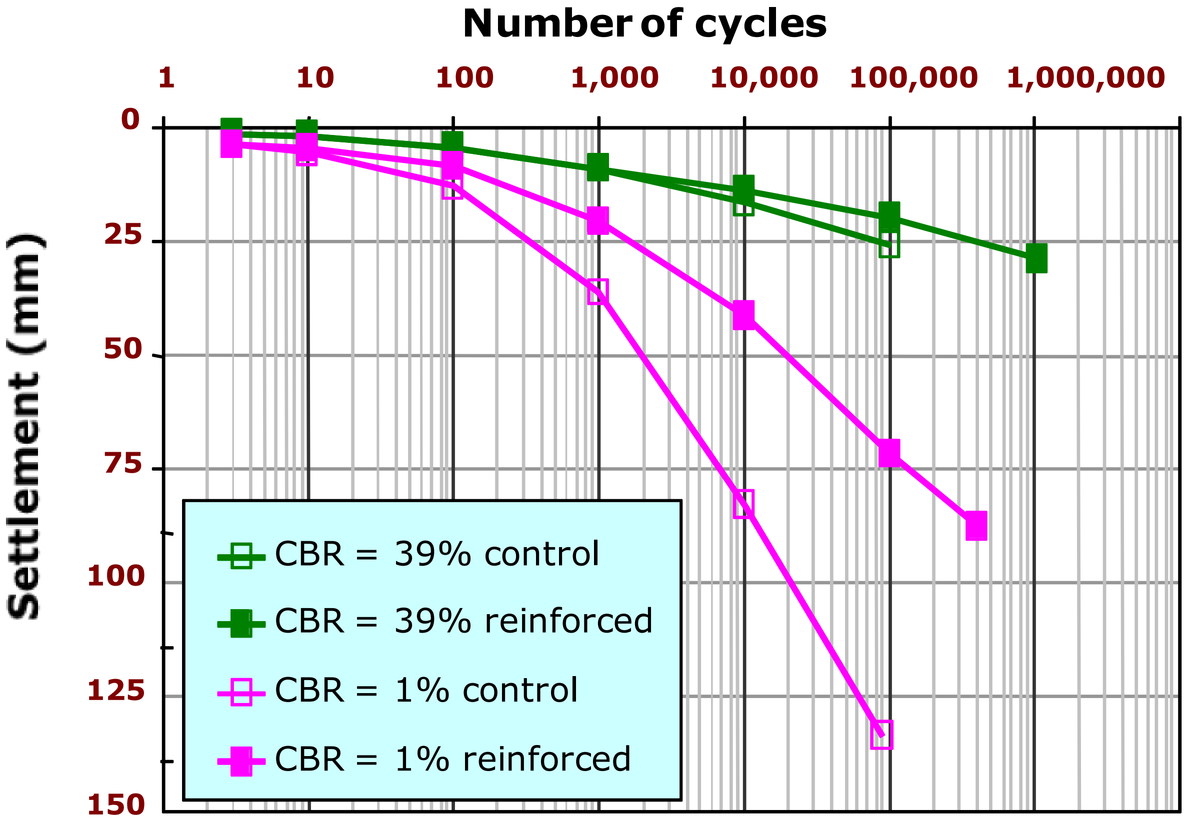
Introduction

- Railroad Ballast
  - Large sized angular aggregates;
  - Horizontal and rotational movement.
- Geogrid
  - Interlocking with particles;
  - Application in railroad ballast.



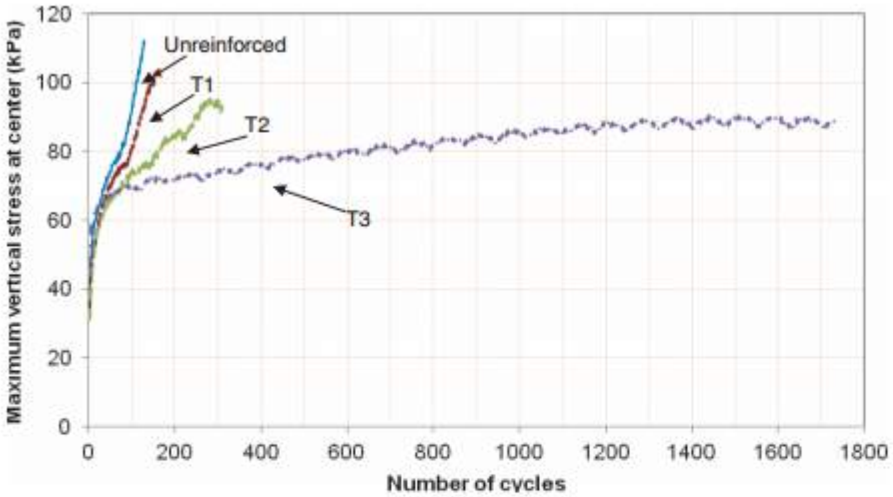


Previous Research Studies

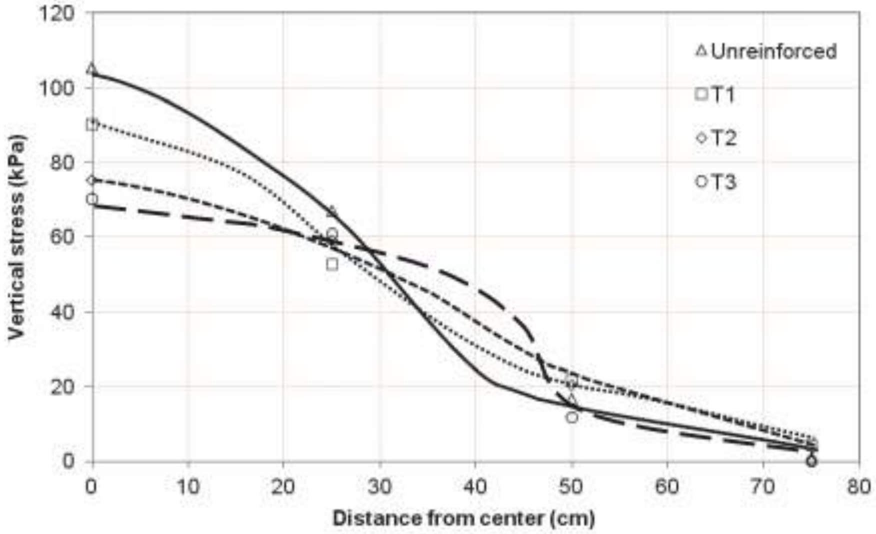


Results of cyclic load tests at Queens University.

Previous Research Studies



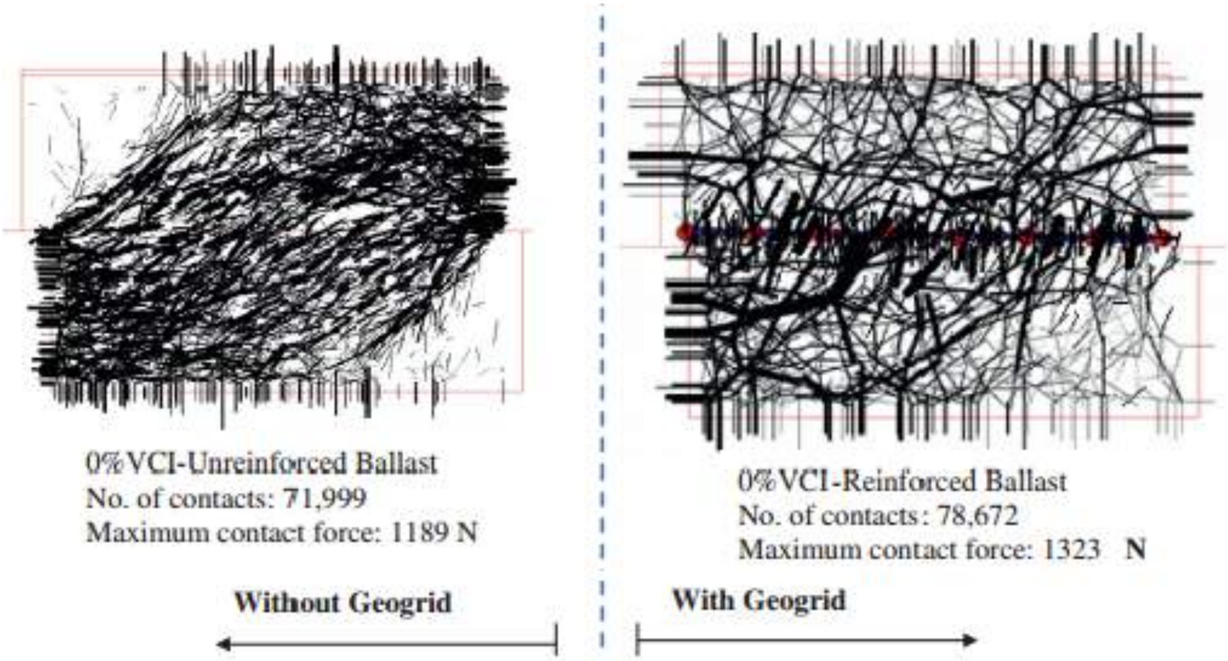
Maximum vertical stresses at interface between base and subgrade.



Vertical stress distributions at 120th load cycle.

Qian, Han et al. (2011)

Geogrid-Aggregate Interlock Mechanism Investigation Using DEM Approach

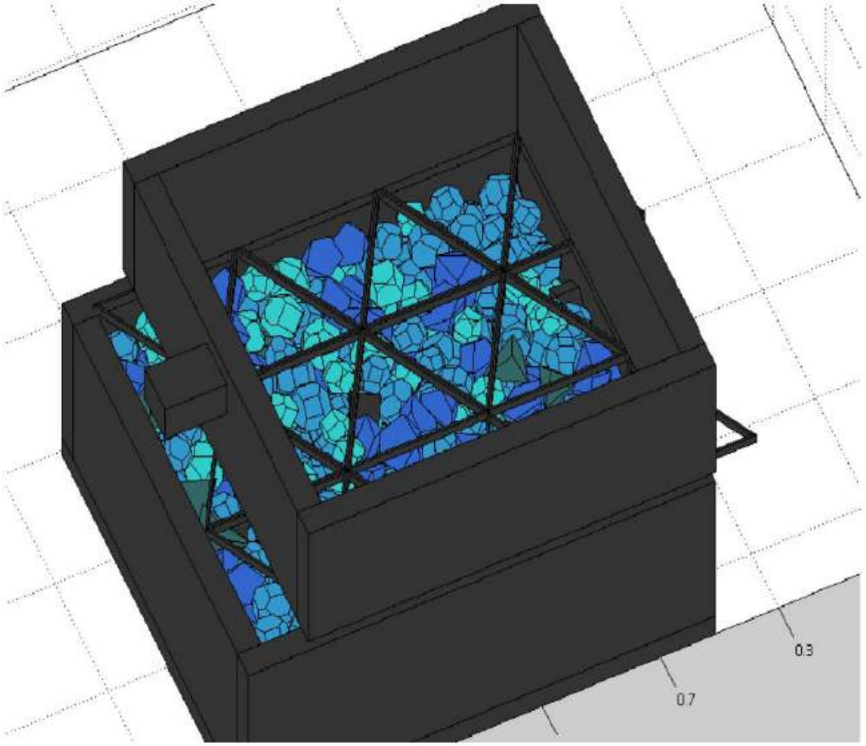


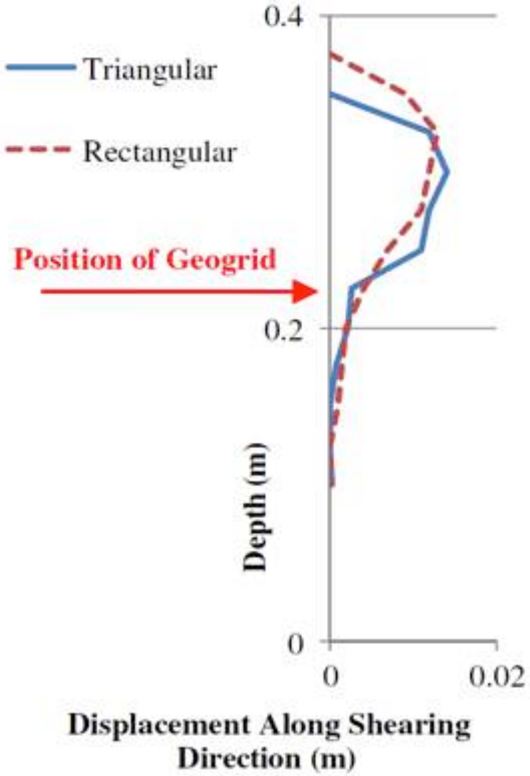
0%VCI-Unreinforced Ballast
No. of contacts: 71,999
Maximum contact force: 1189 N

0%VCI-Reinforced Ballast
No. of contacts: 78,672
Maximum contact force: 1323 N

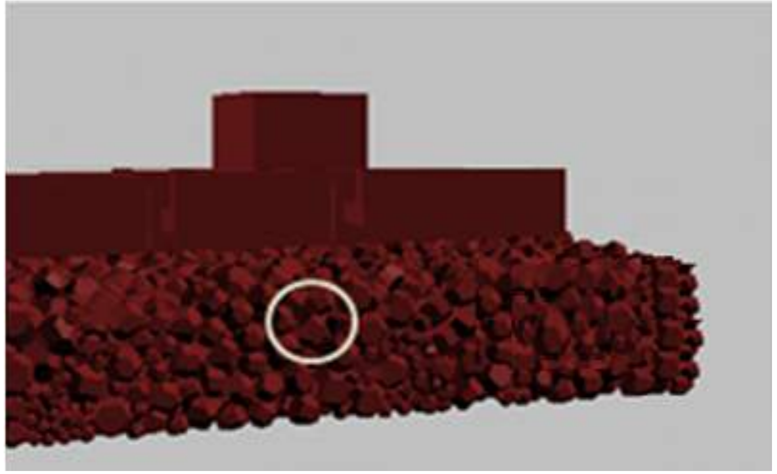
Ngo et al. 2014

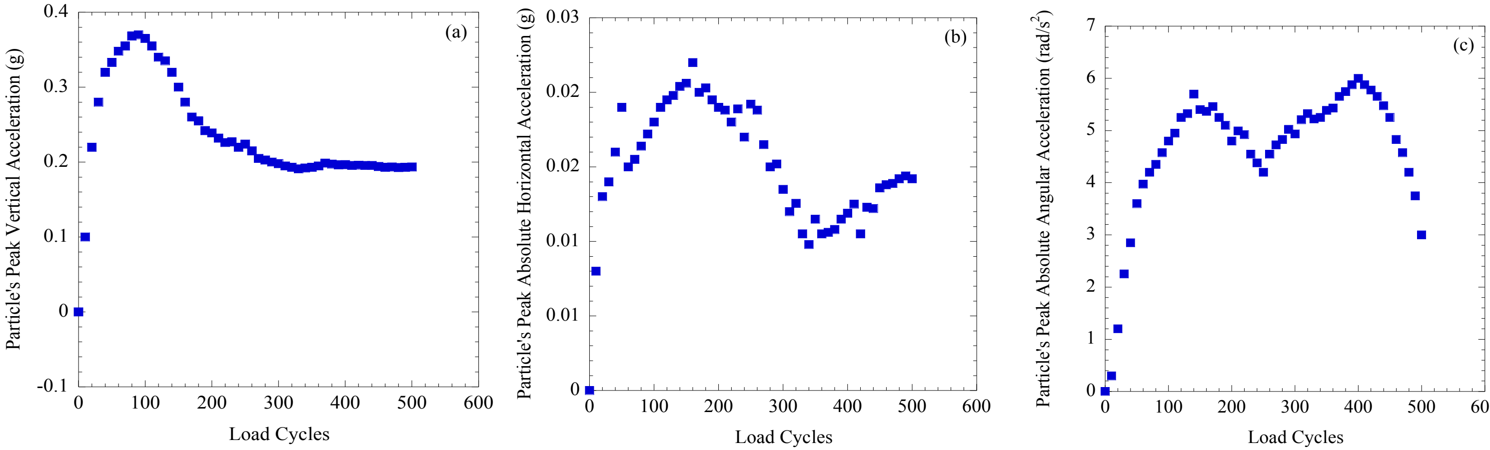
Geogrid-Aggregate Interlock Mechanism Investigation Using DEM Approach



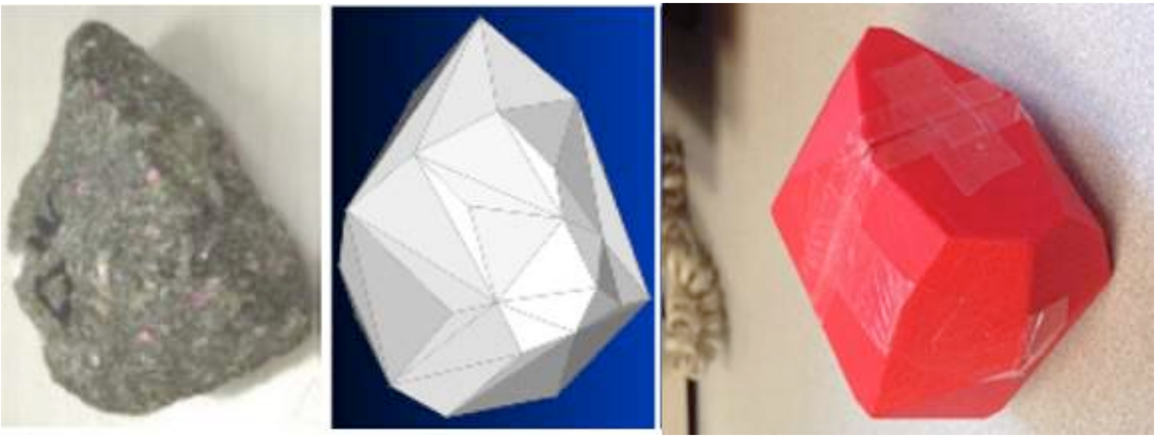


Ballast modeling shows particle horizontal movement and rotation are important modes of particle movement.



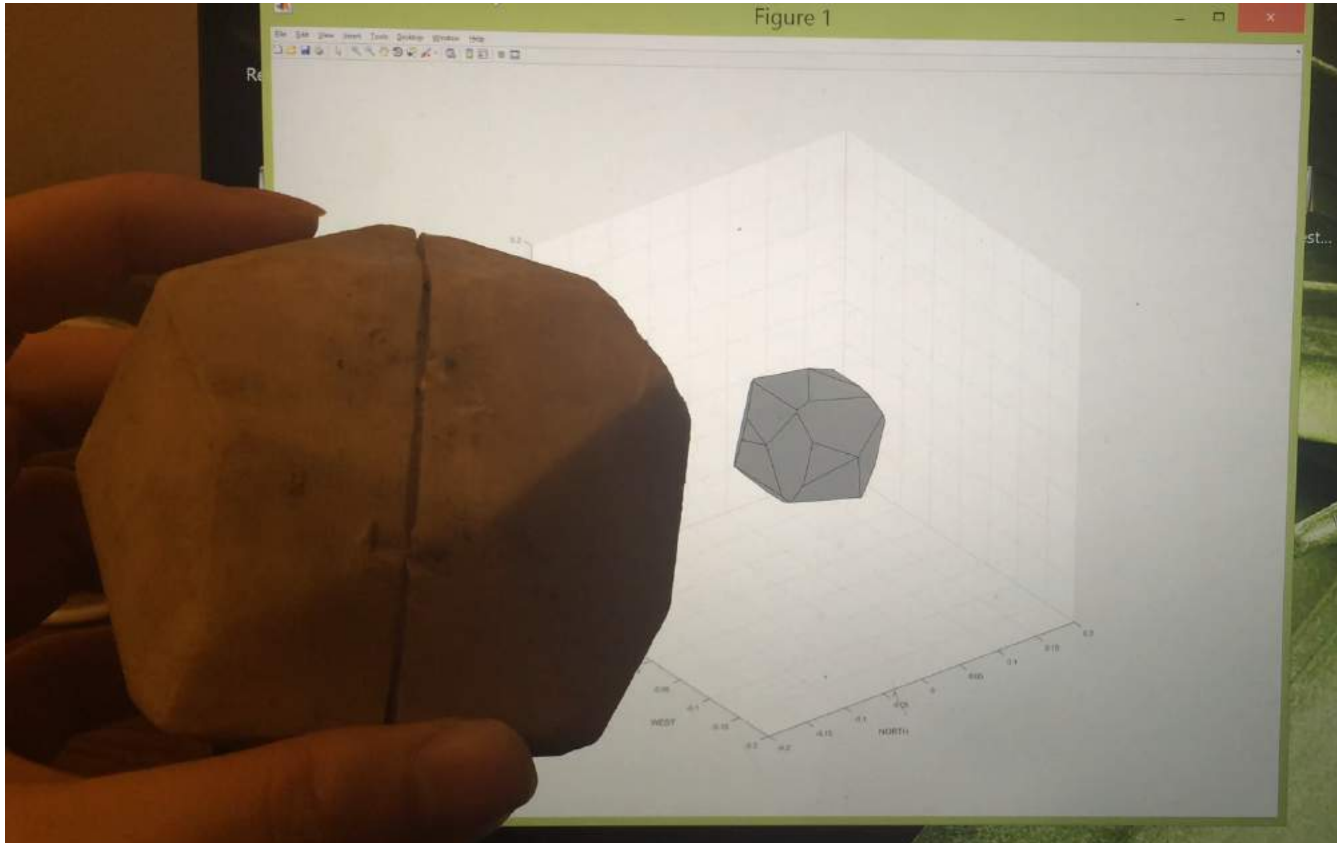


"SmartRock"

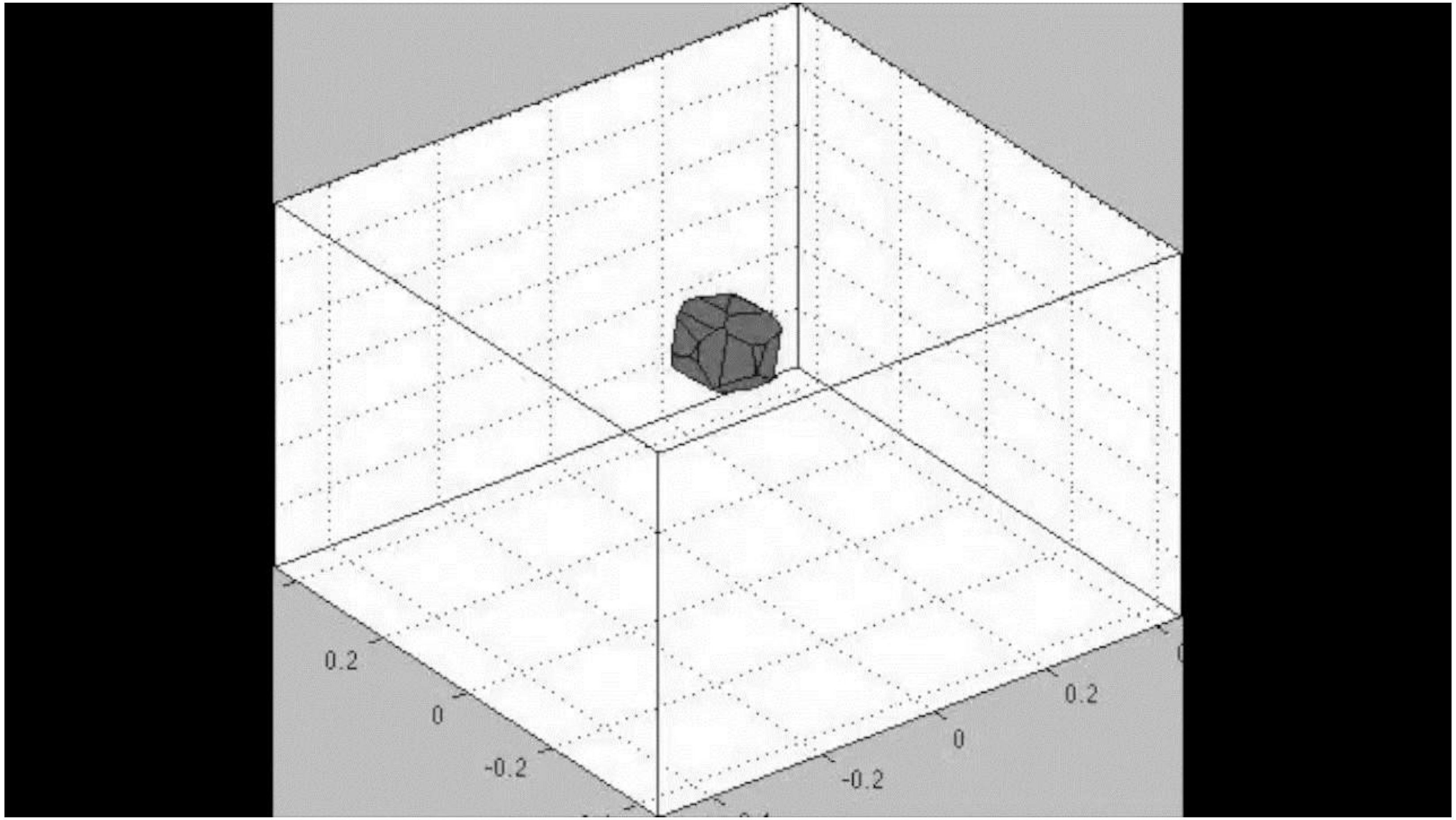


- Shape;
- Wireless device;
- Data storage;
- Sleep mode;
- Translation, rotation and orientation.

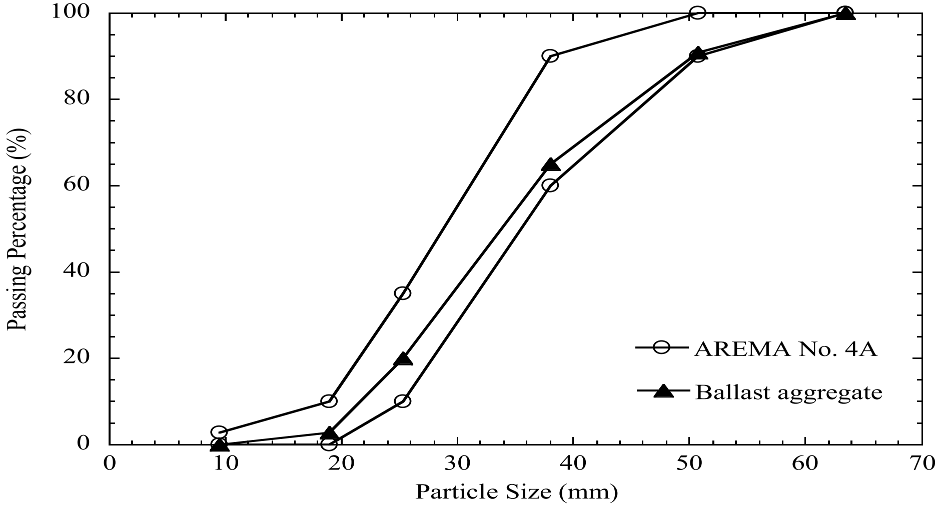
Real Time Rotation

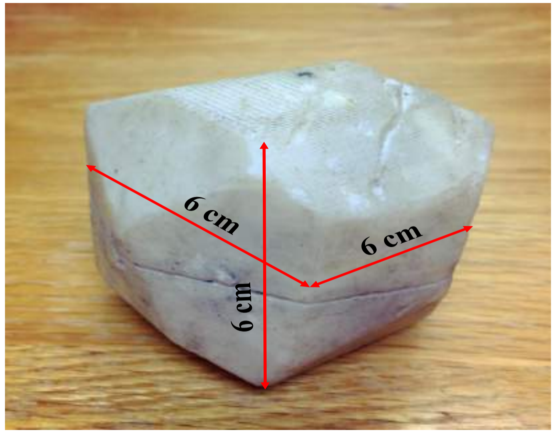


Rotation + Translation



Laboratory Test – Ballast

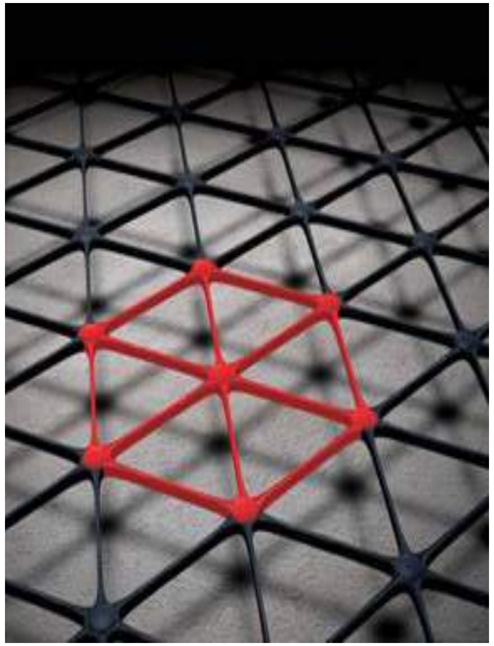




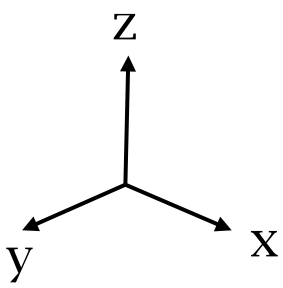
Laboratory Test – Geogrid

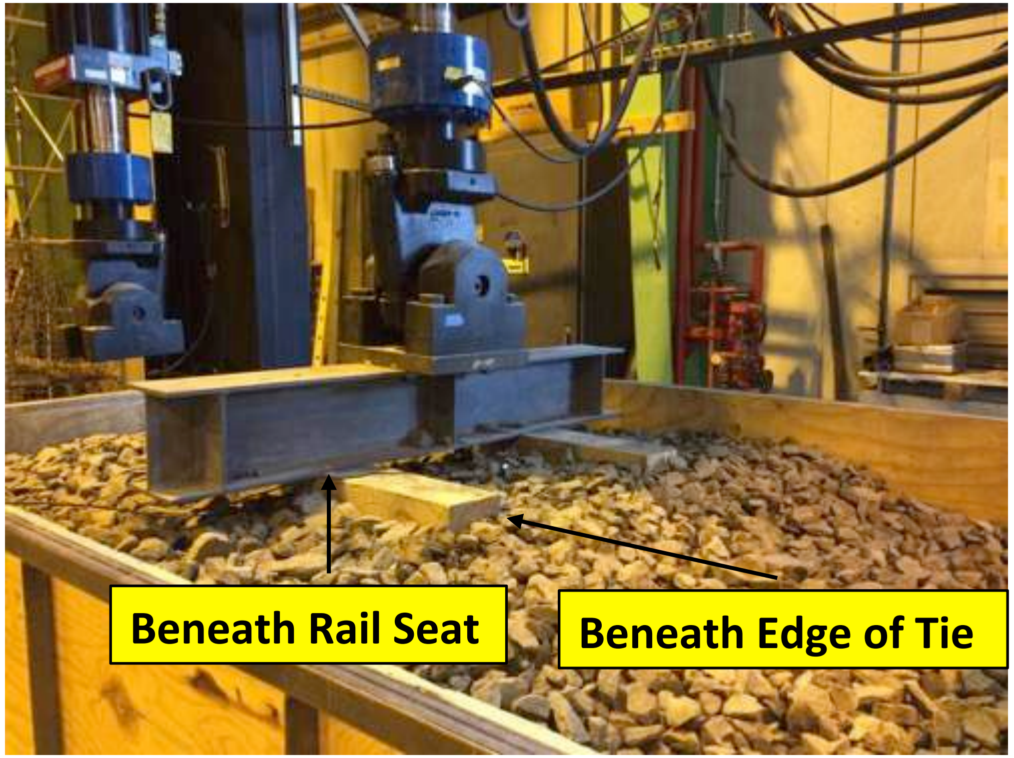
Physical Properties of Geogrids Used in Track Stabilization

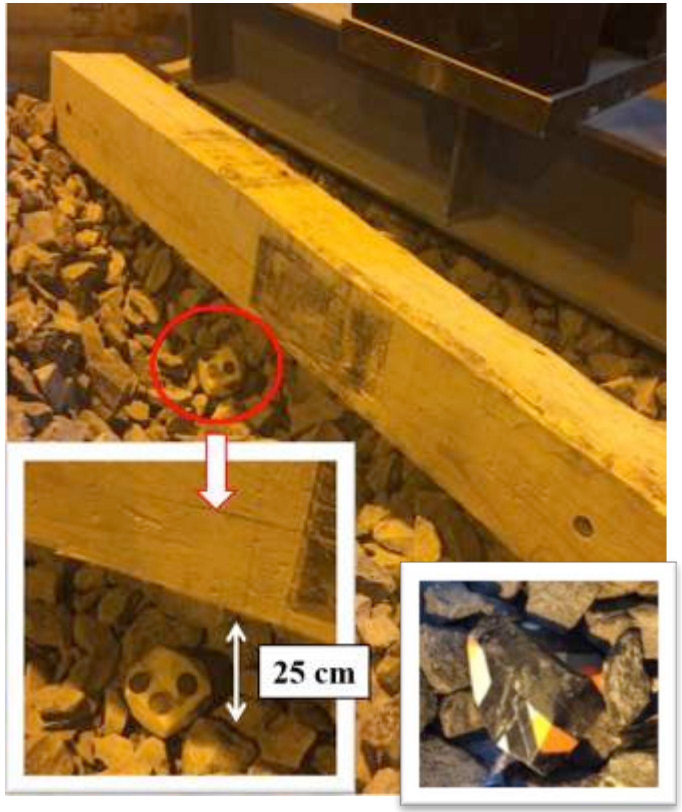
Property	Test Method	Units	Geogrid Properties
Aperture shape	Observation		Equilateral Triangular
Aperture size (machine x cross machine direction)	Direct measurement	mm	60 x 60
Flexural rigidity (Machine direction)	ASTM D7748-12	mg-cm	2,000,000
Radial stiffness @ 0.5% strain	ASTM D6637-10	kN/m	350
Junction efficiency	ASTM D7737-11	%	93



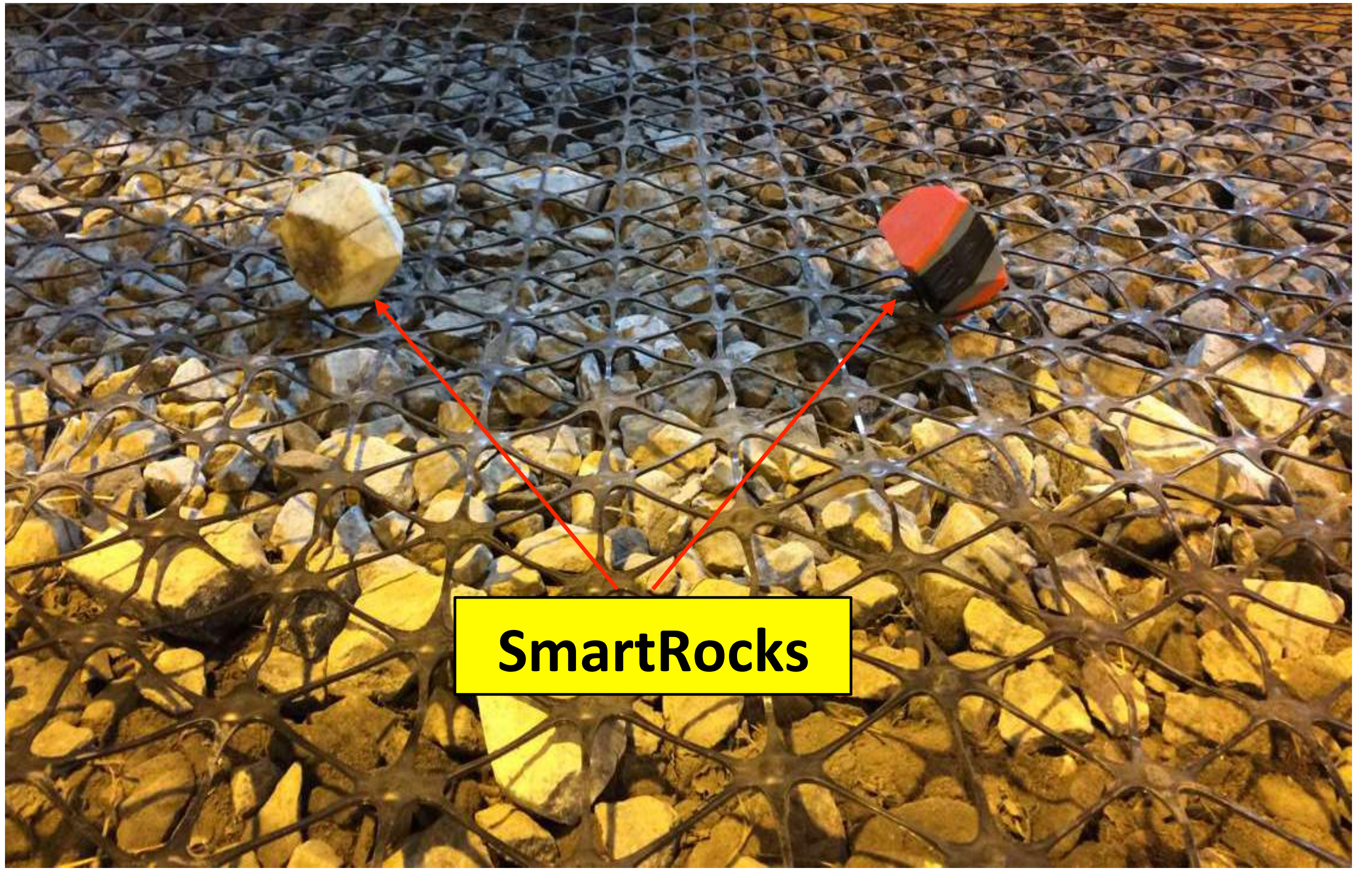
Laboratory Testing – WITHOUT Geogrid



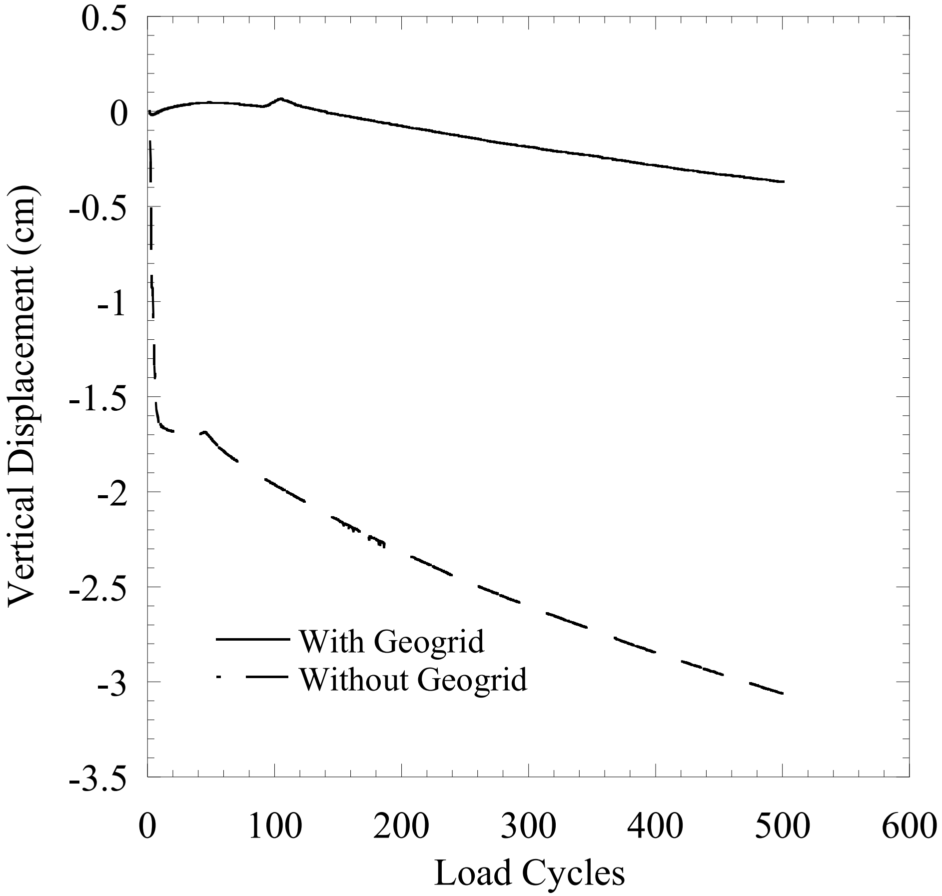


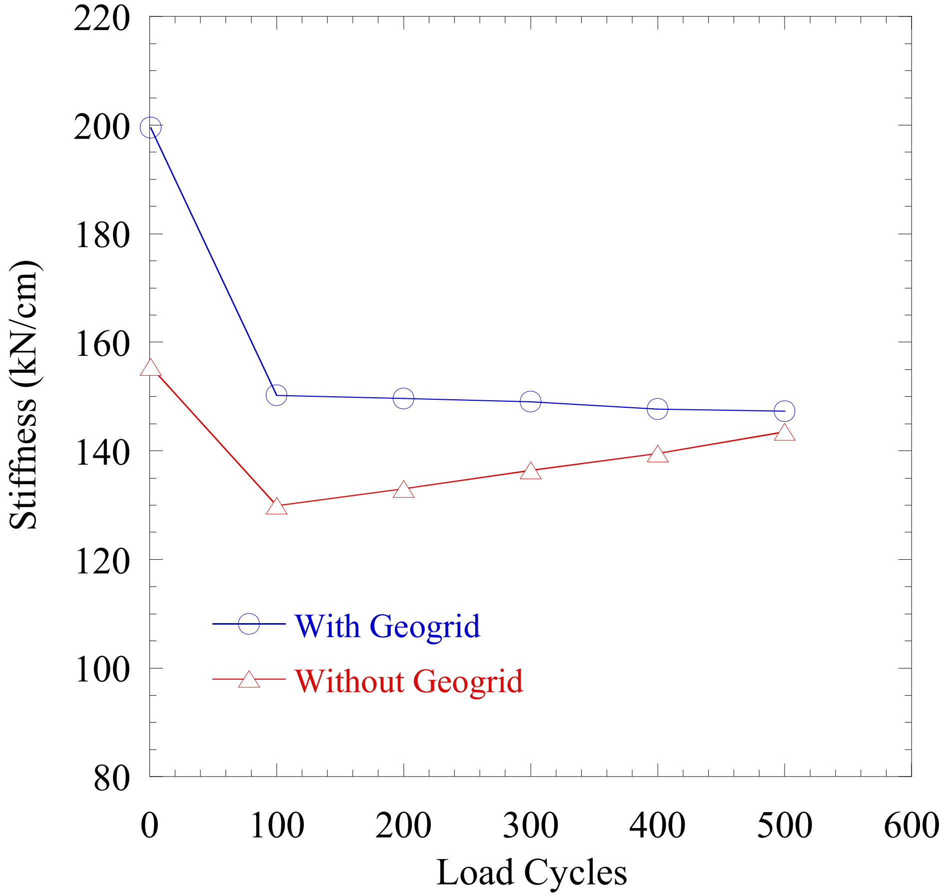


Laboratory Testing – WITHOUT Geogrid



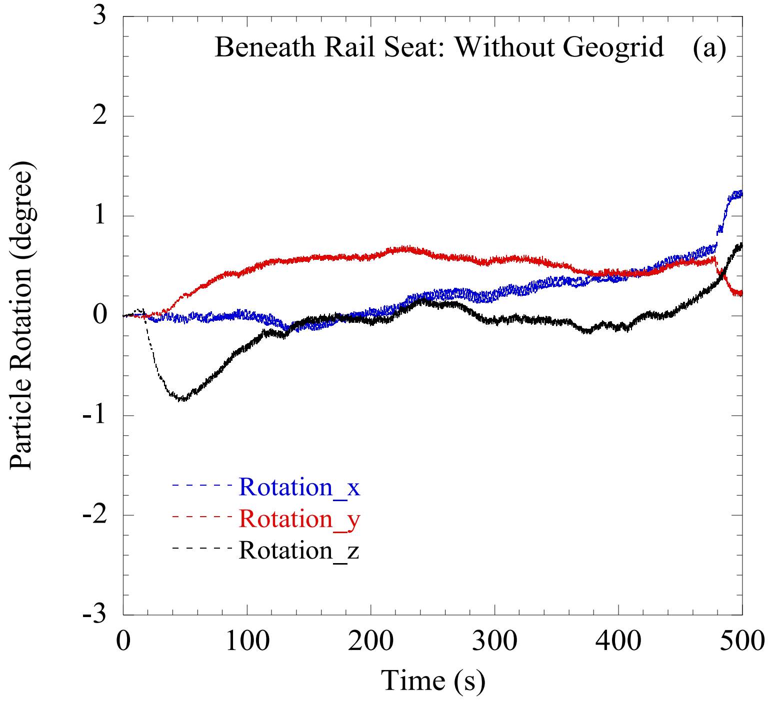
Displacement and Stiffness



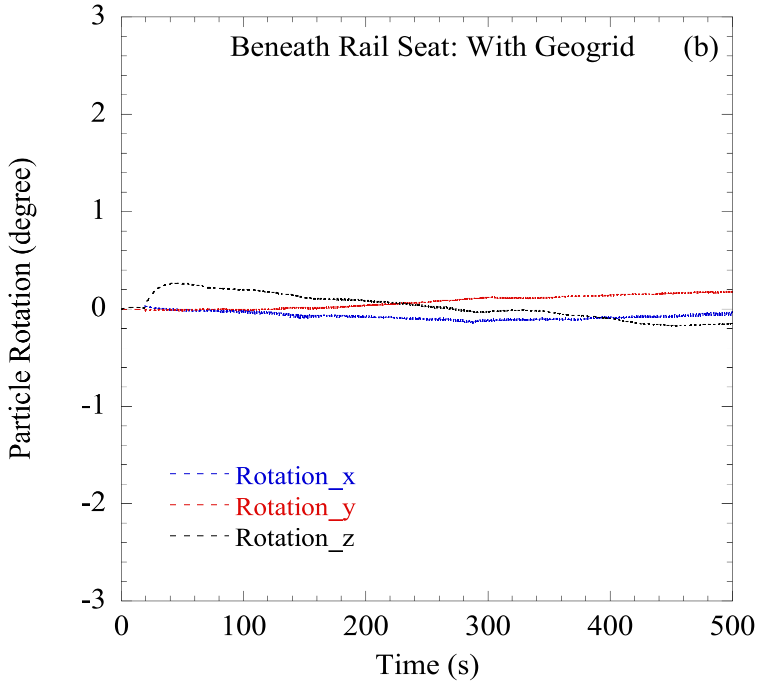


Particle Rotation – beneath Rail Seat

Without Geogrid

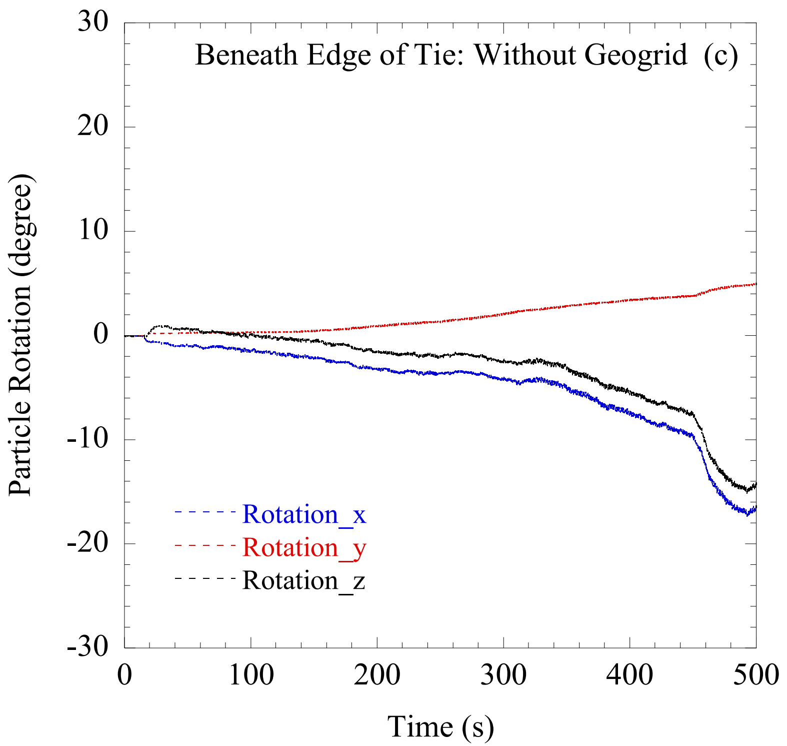


With Geogrid

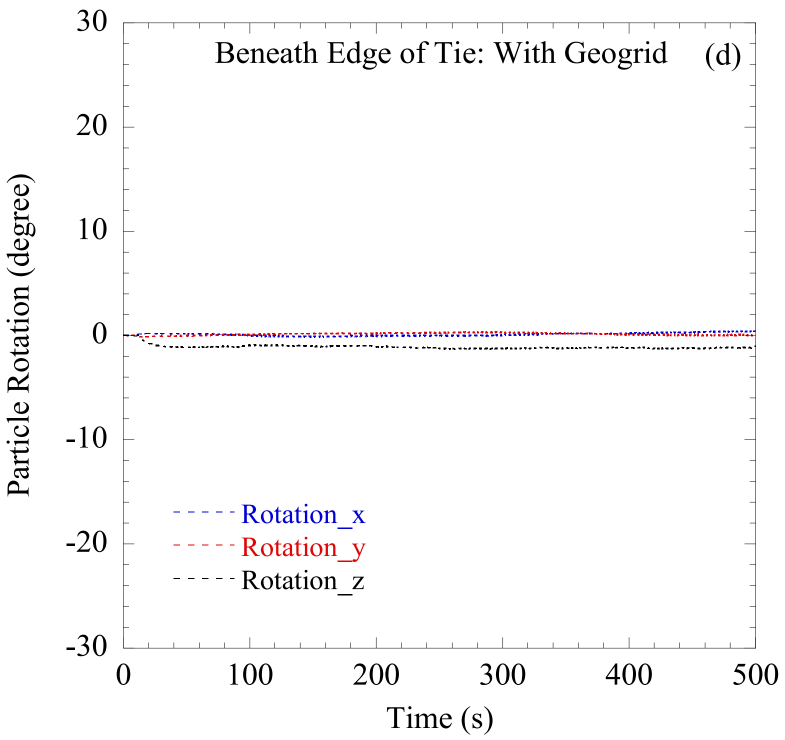


Particle Rotation – beneath Edge of Tie

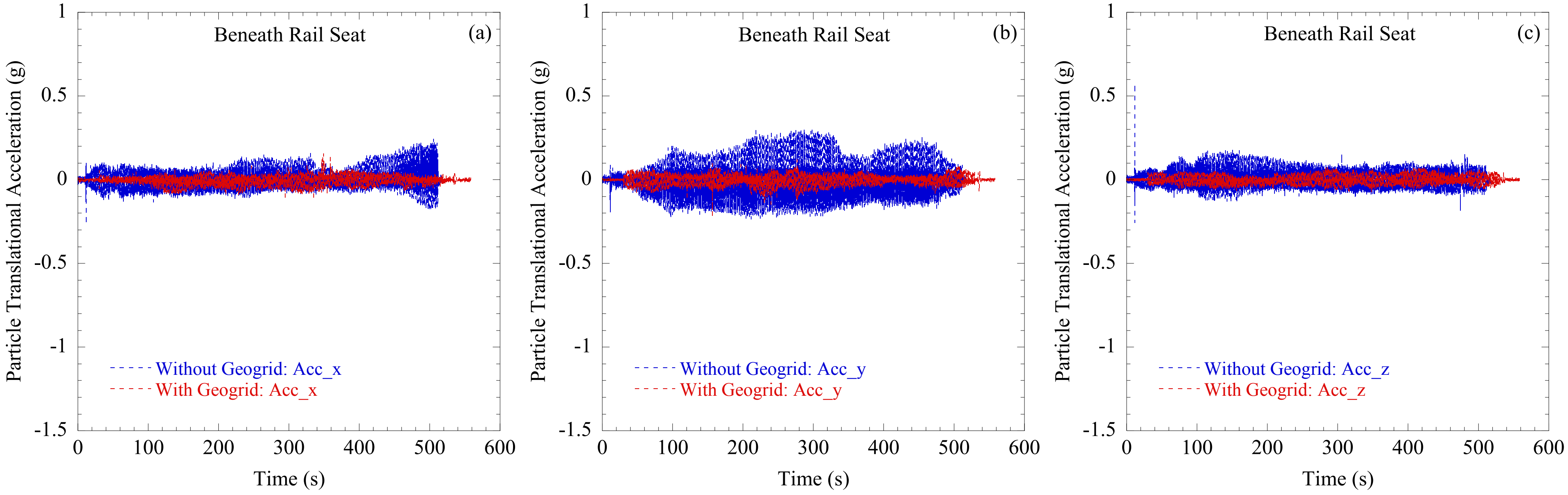
Without Geogrid



With Geogrid

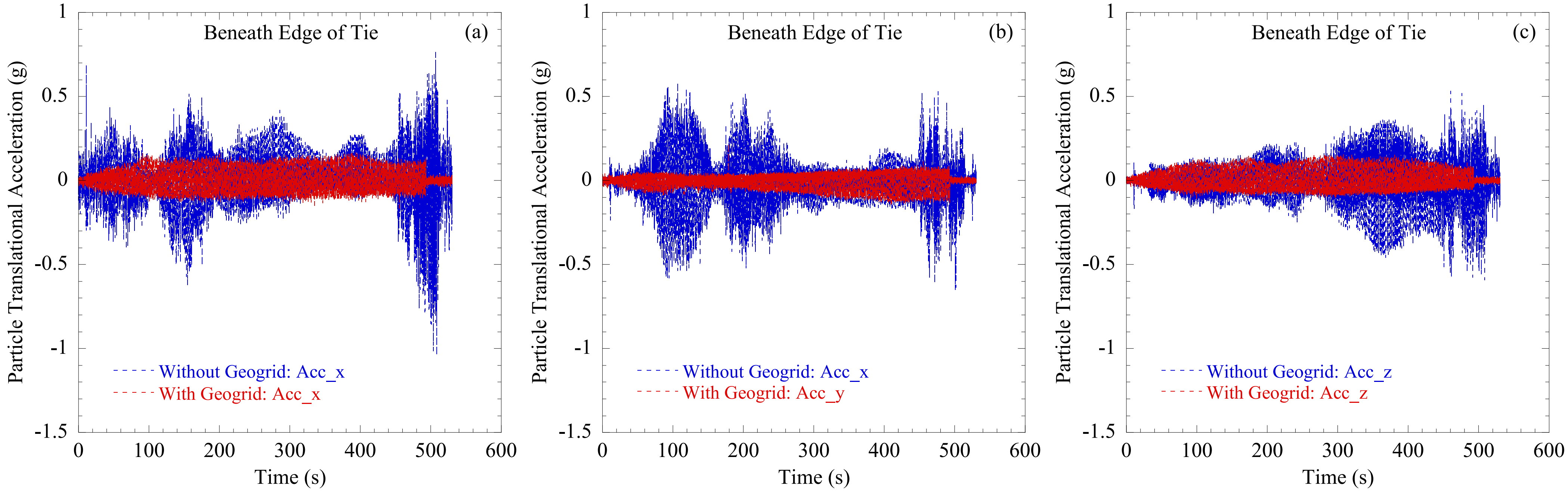


Particle Acceleration – beneath Rail Seat



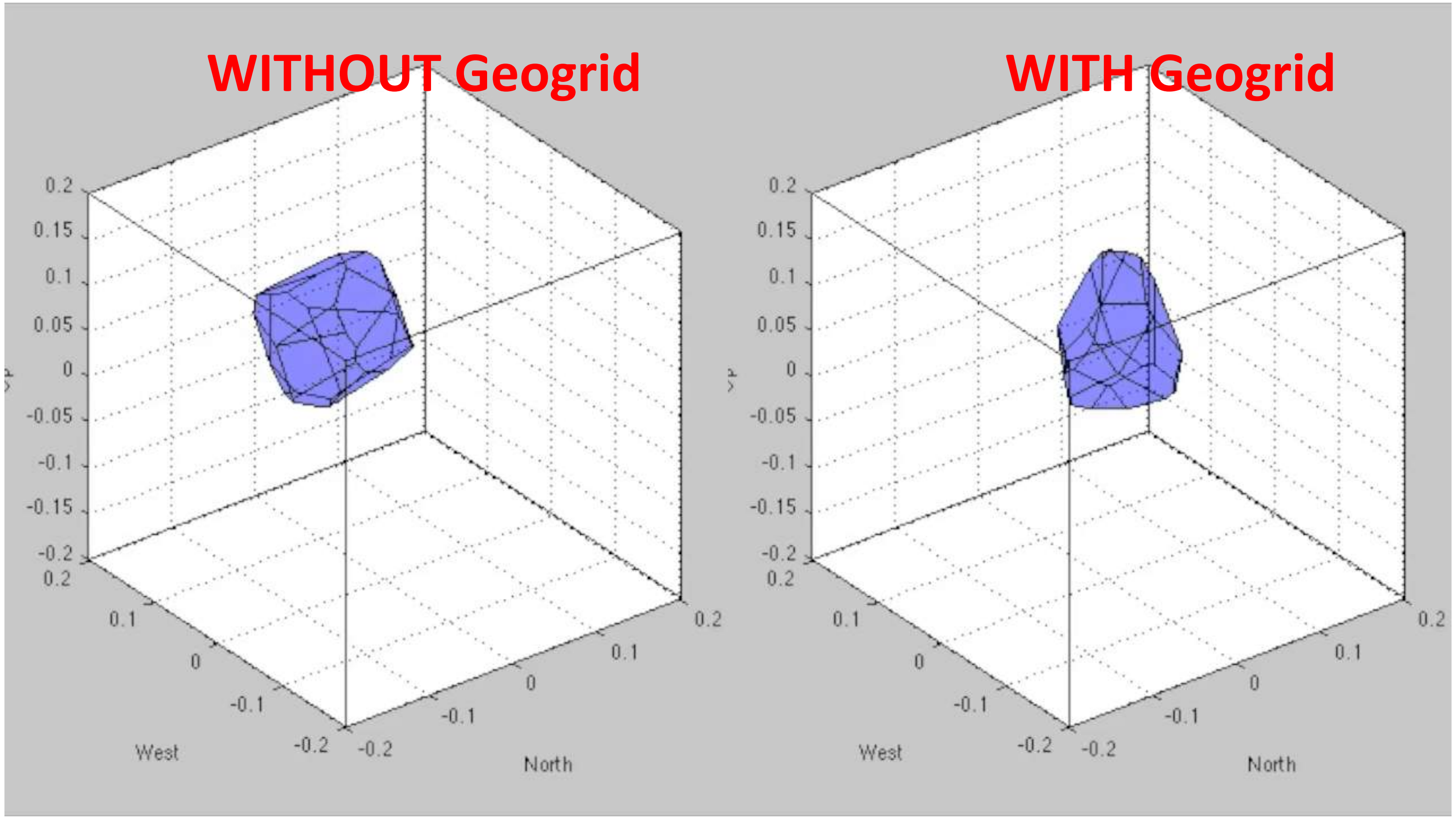
(a) X direction; **(b) Y direction;** **(c) Z direction**

Particle Acceleration – beneath Edge of Tie



(a) X direction; (b) Y direction; (c) Z direction

Visualization: Without/With Geogrid



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!

8

3rd ICTG 2016
4-7 September 2016, Guimarães, Portugal

Universidade do Minho
Escola de Engenharia

SIMSG ISSMGE

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

DOT Sponsors:

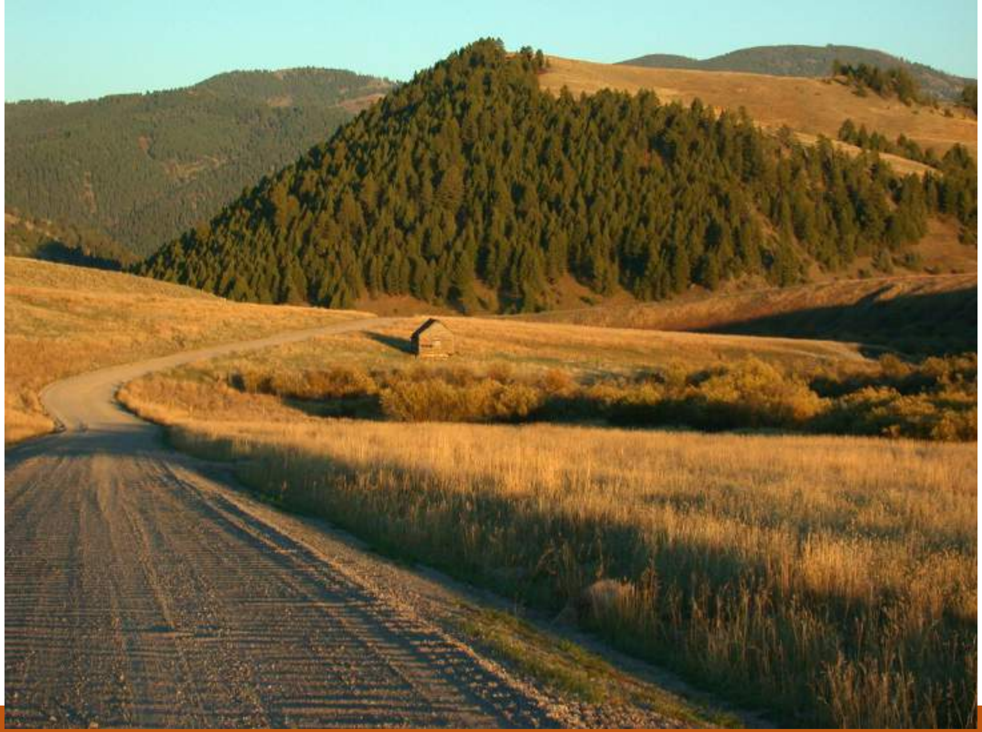
- Idaho
- Montana
- New York
- Ohio
- Oklahoma
- Oregon
- South Dakota
- Texas
- Wyoming

Manu. Donations:

- Colbond
- Huesker
- NAUE
- Propex
- Synteen
- TenCate

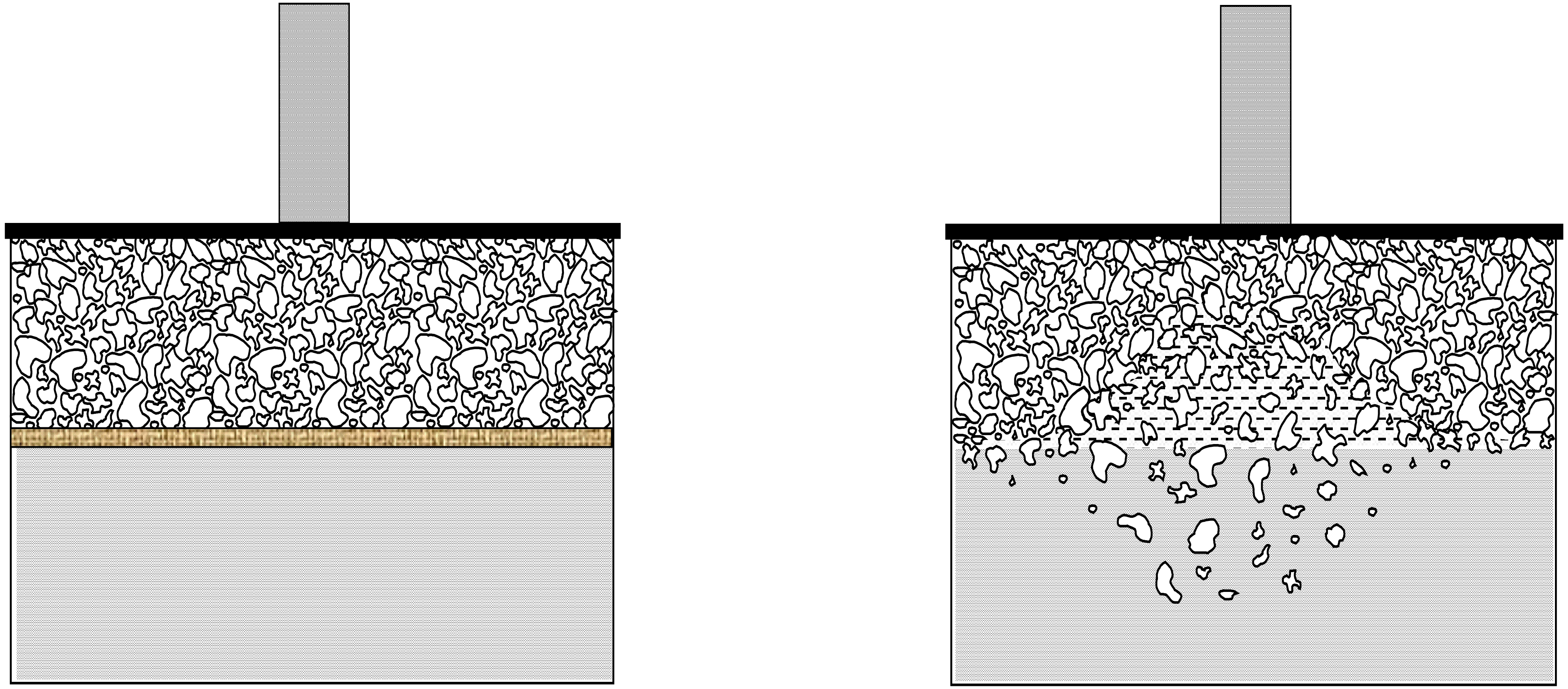
Background

- 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



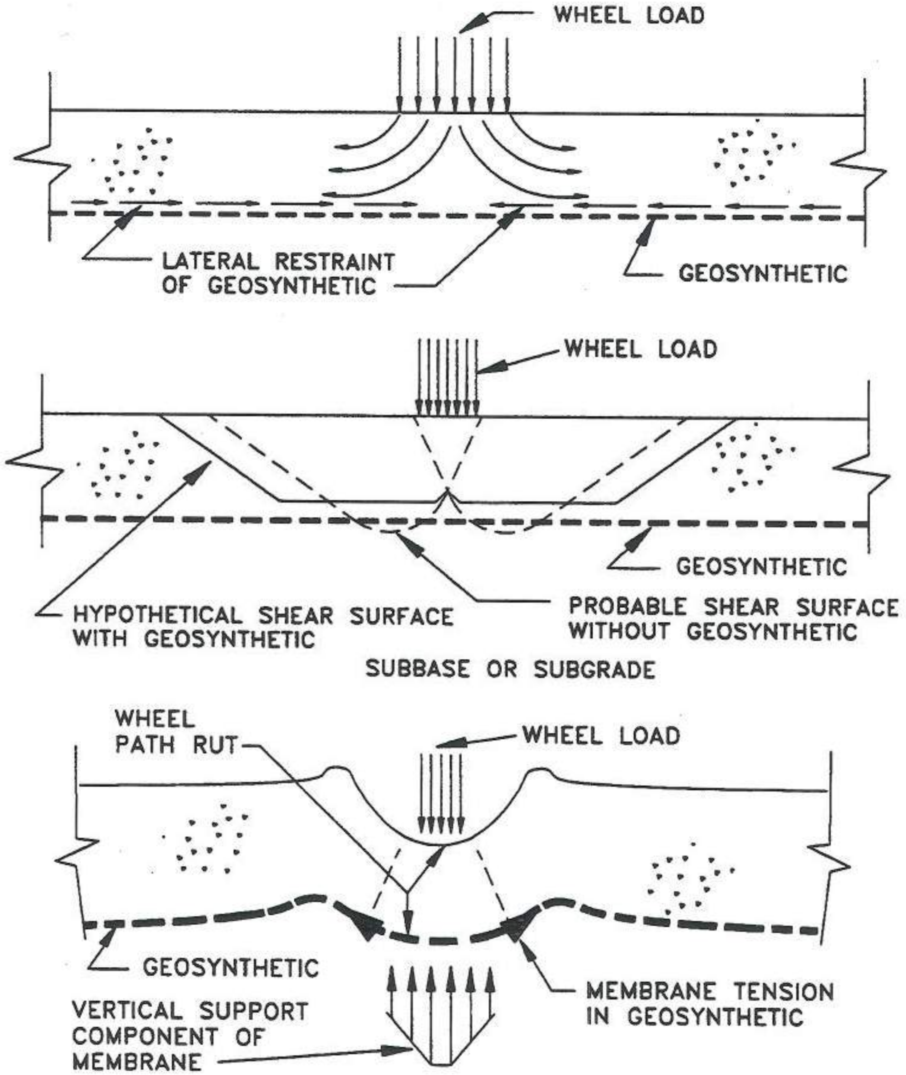
Geosynthetic Benefit on Soft Subgrades

Stabilization: Separation Function



Stabilization: Reinforcement Function

- Lateral Restraint
- Bearing Capacity Increase
- Membrane Tension Support



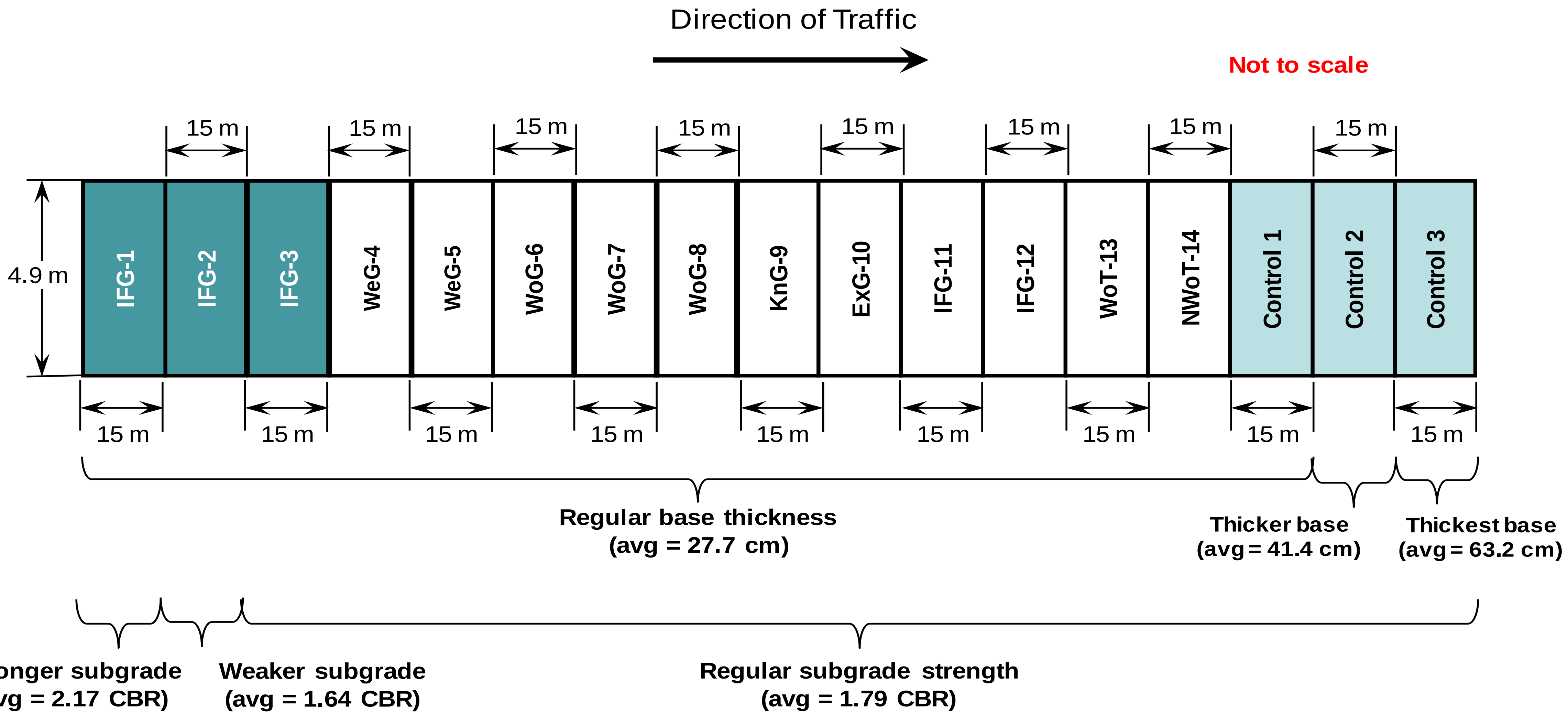
Study Objective

- 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
WoG-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)

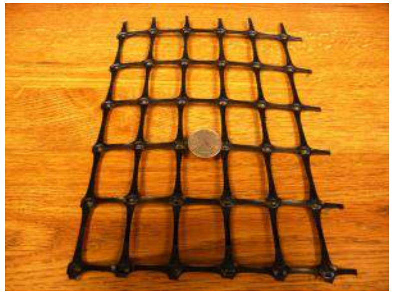
Thickest base
(avg = 63.2 cm)

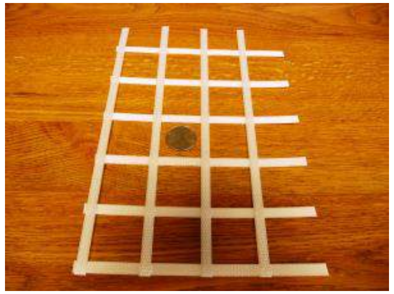
Stronger subgrade
(avg = 2.17 CBR)

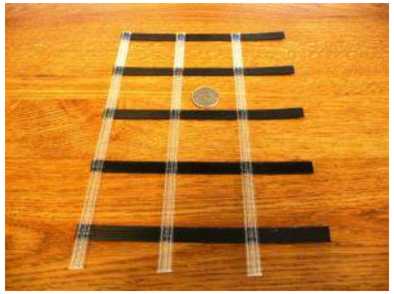
Weaker subgrade
(avg = 1.64 CBR)

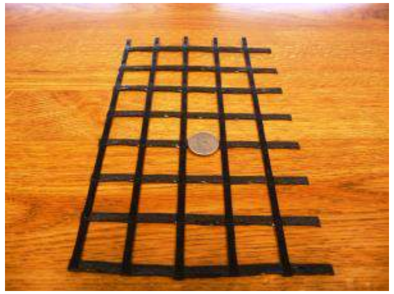
Regular subgrade strength
(avg = 1.79 CBR)

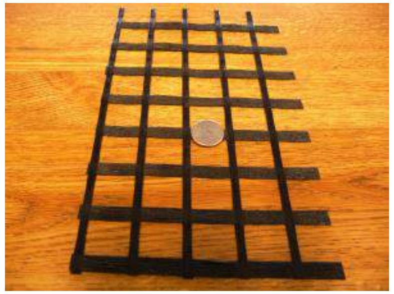
Geosynthetics

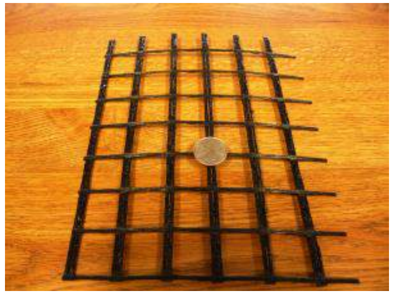

IFG-1, IFG-2, IFG-3

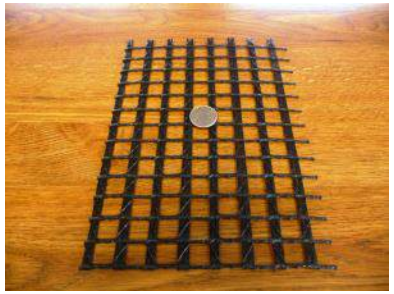

WeG-4

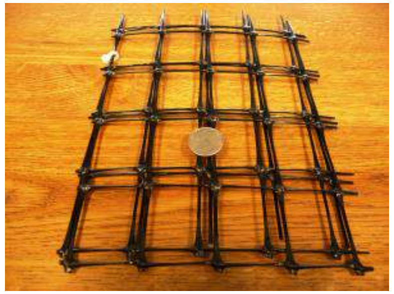

WeG-5

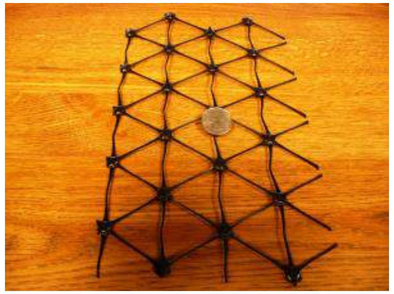

WoG-6

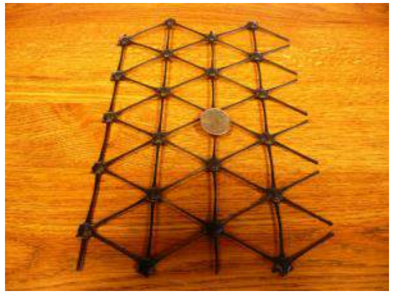

WoG-7

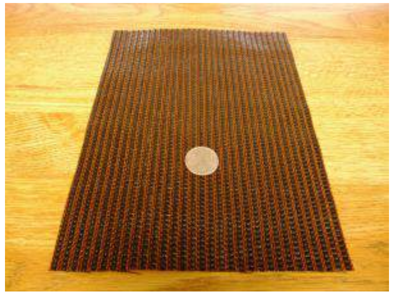

WoG-8

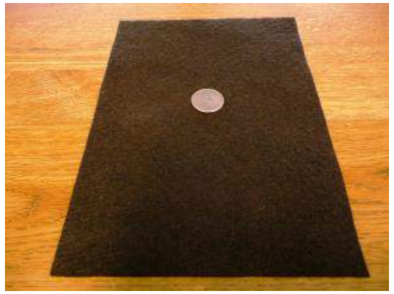

KnG-9


ExG-10

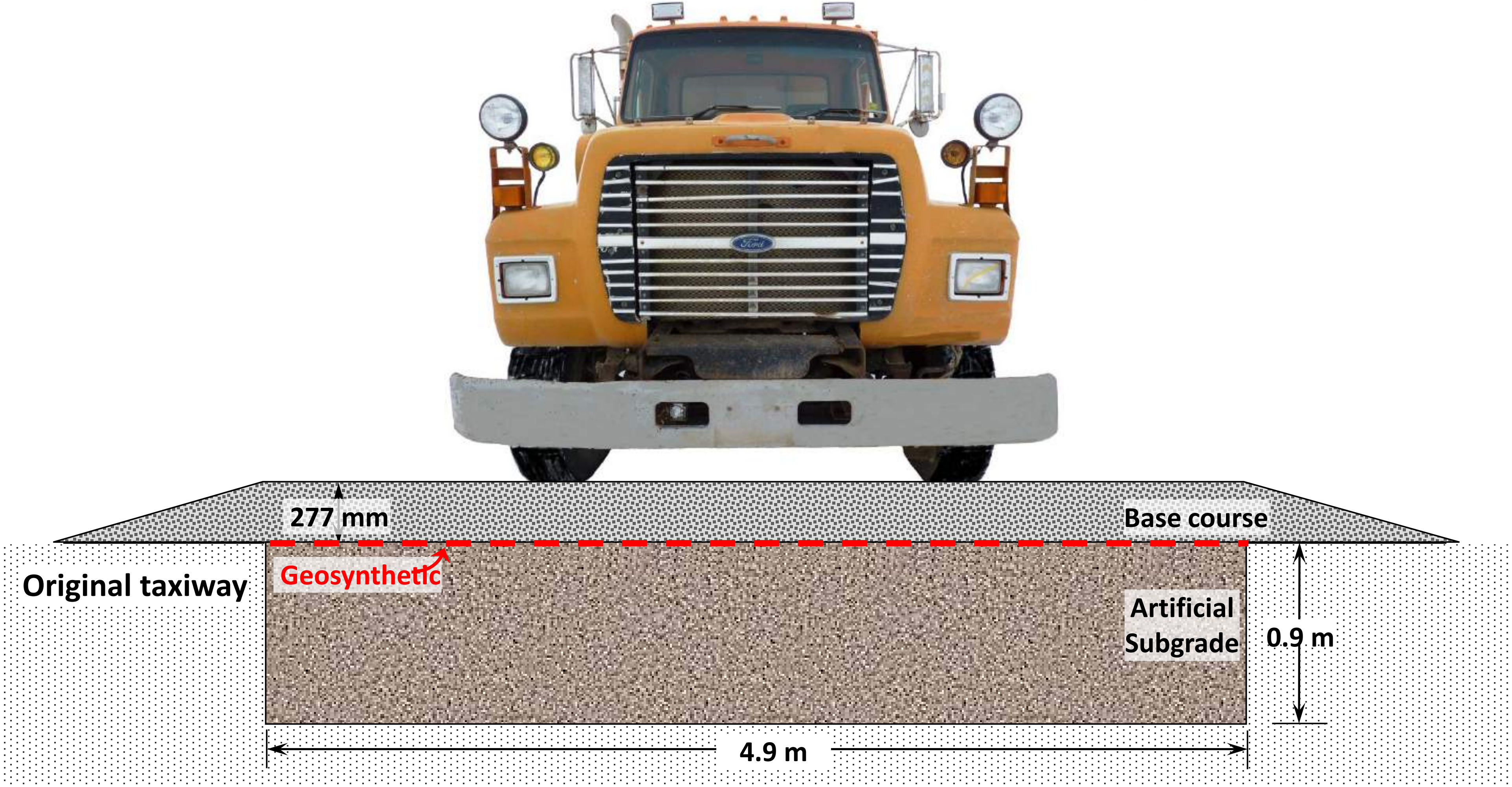

IFG-11


IFG-12


WoT-13


NWoT-14

Idealized Cross-Section (to scale)



Trench

Preparing Subgrade

Placing Subgrade

Compacted Subgrade

Screeding Subgrade

Installing Geosynthetics

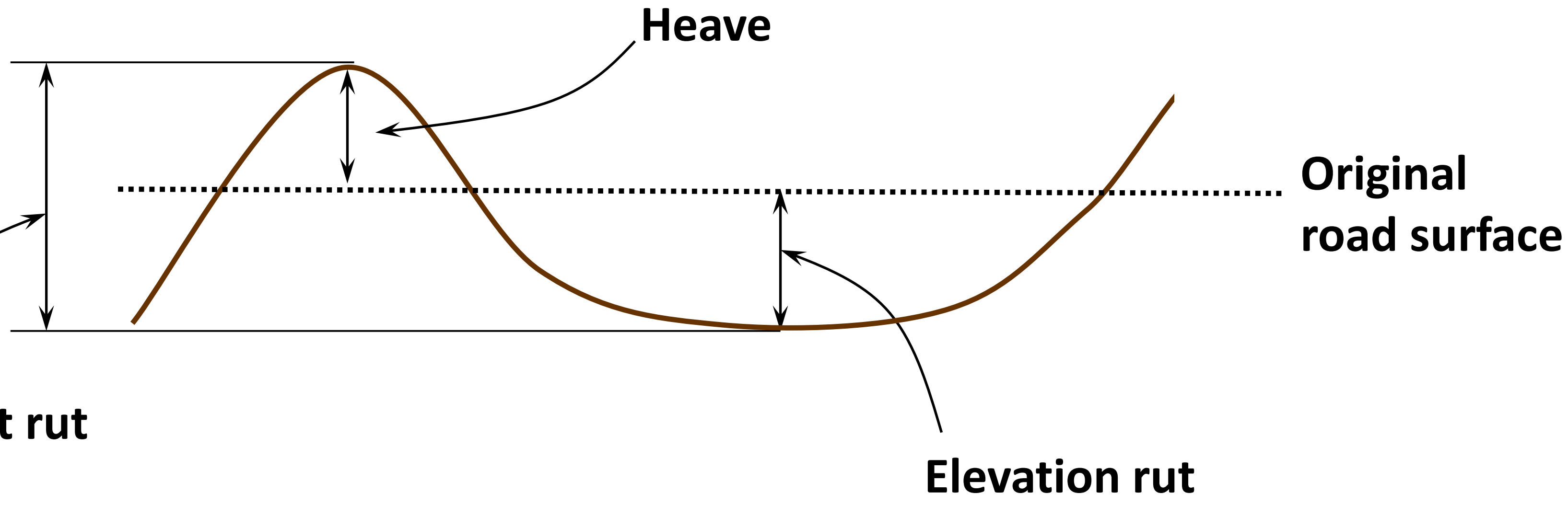
Constructing Base Layer

Finished Construction

Test Vehicle

- 20.6 metric tons
- 8 kph

Rut Measurement

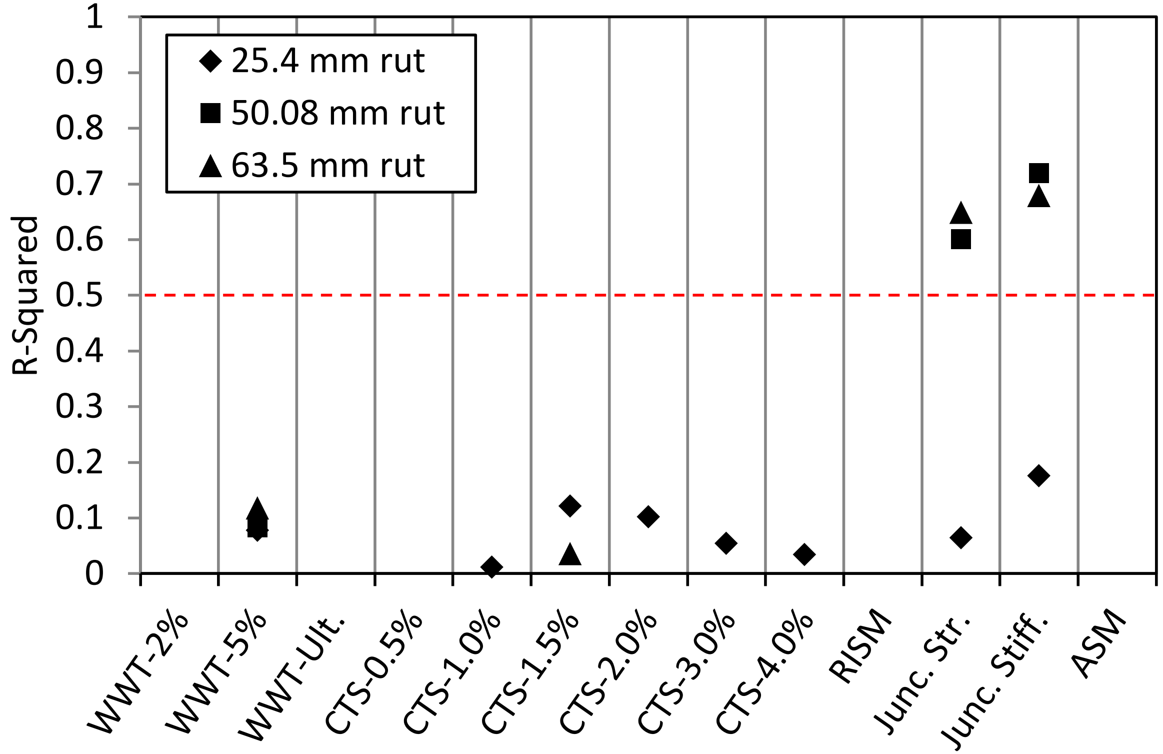


*Measurements were made at 0, 3, 10 20, 40, 70, 80, 102, 125, 175, 250, 300, 325, 351, 395, 440, 540, 640, and 740 truck passes

Linear Regression Analysis

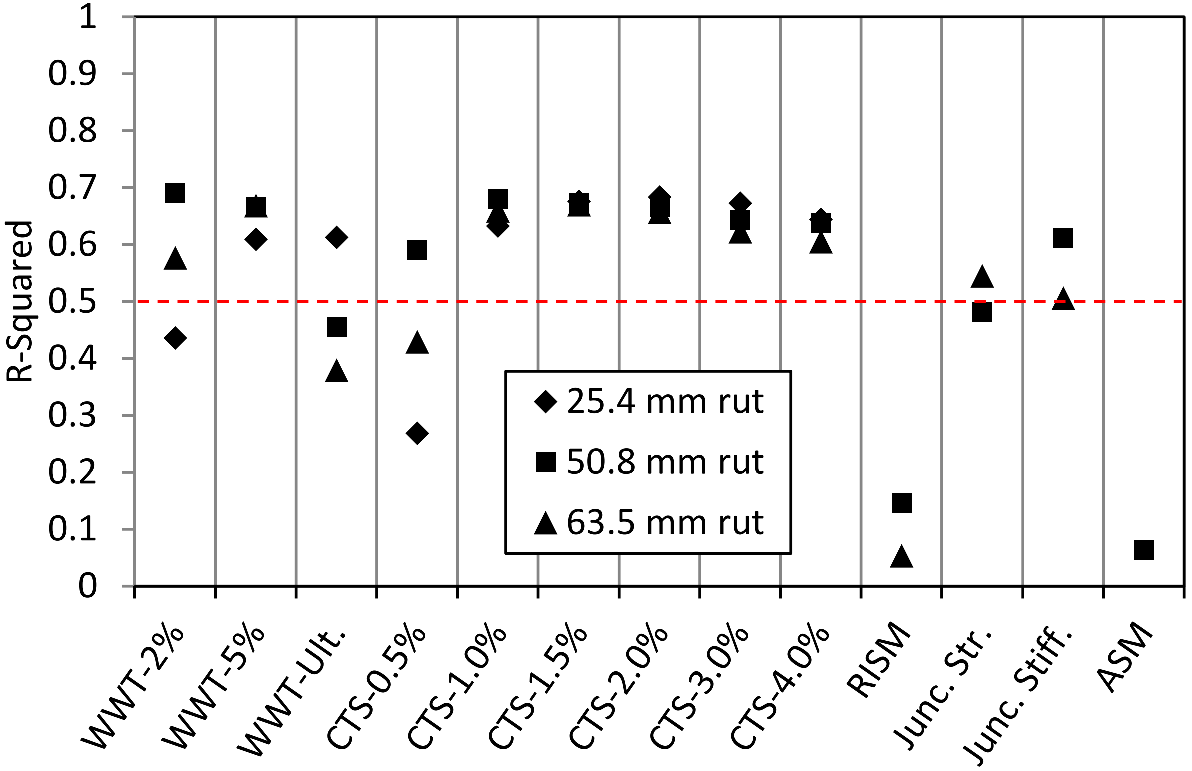
- Determine material properties most related to performance
- Evaluated at 1.0, 2.0 and 2.5 in. of rut
- Material properties evaluated
  - Wide-width strength at 2%
  - Wide-width strength at 5%
  - Ultimate wide-width strength
  - Cyclic tensile stiffness at 0.5, 1.0, 1.5, 2.0, 3.0 and 4.0% strain
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

Regression Analysis Results in XMD

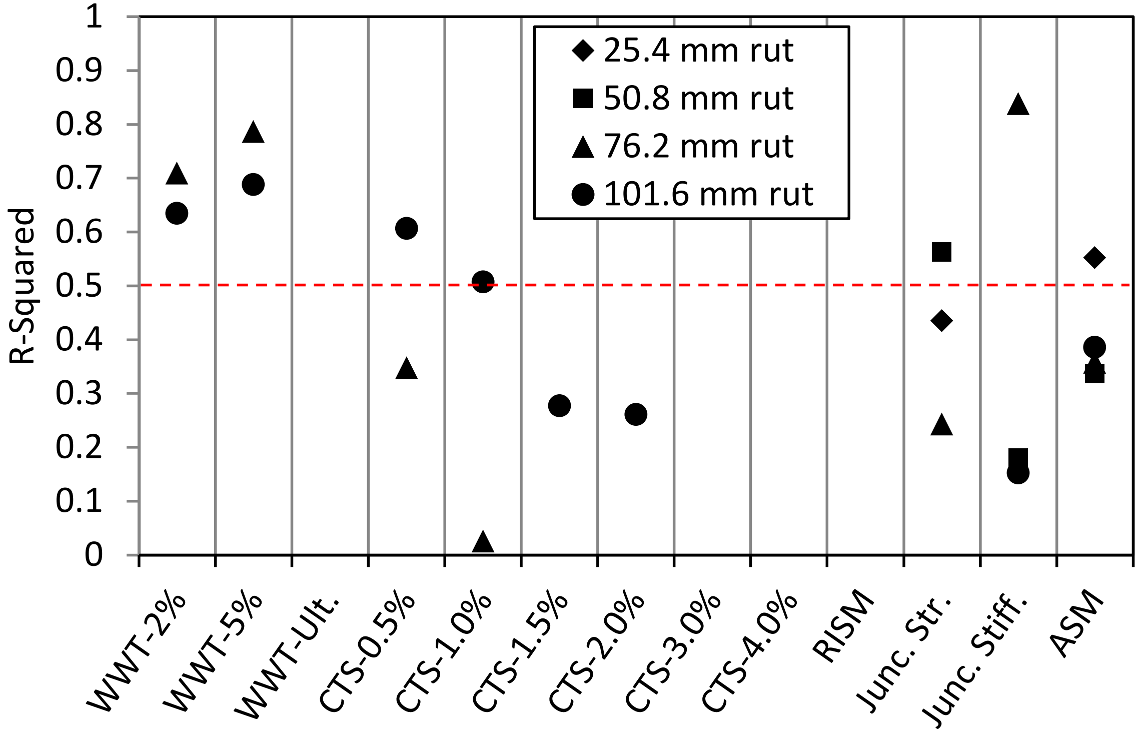


Results using all data

Results using select data

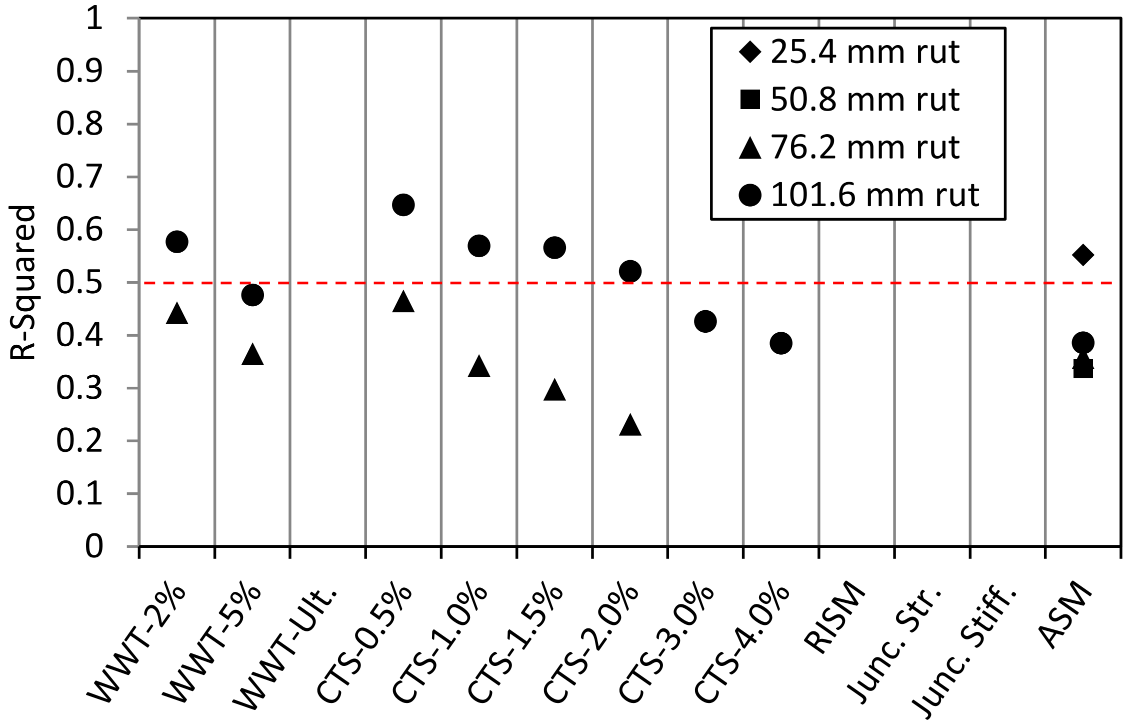


Regression Results from Phase I Study



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

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

h = compacted base course thickness {m}

N = number of axle passes

k = constant dependent on base thickness and reinforcement

α_0 = initial stress distribution angle = 38.5°

$$R_E = \min\left(\frac{E_{bc}}{E_{sg}}, 5.0\right) = \min\left(\frac{3.48CBR_{bc}^{0.3}}{CBR_{sg}}, 5.0\right)$$

$$k = (0.96 - 1.46J^2)\left(\frac{r}{h}\right)^{1.5}$$

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)

ξ, ω, and n are constants calibrated by Giroud and Han (2004b) using data from unpaved, unreinforced roads (ξ = 0.9, ω = 1.0, and n = 2.0)

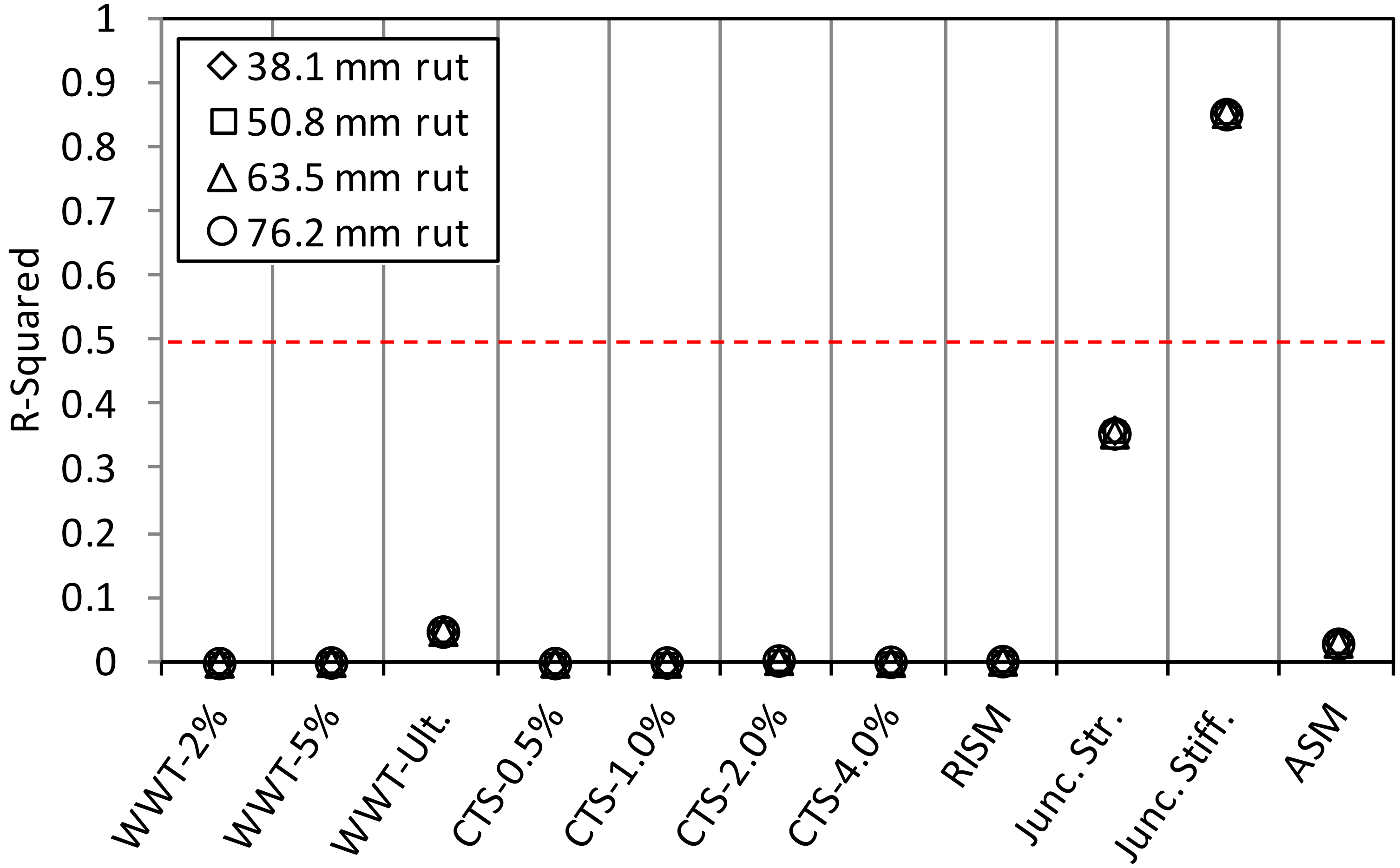
Back-Calculate *k'*

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

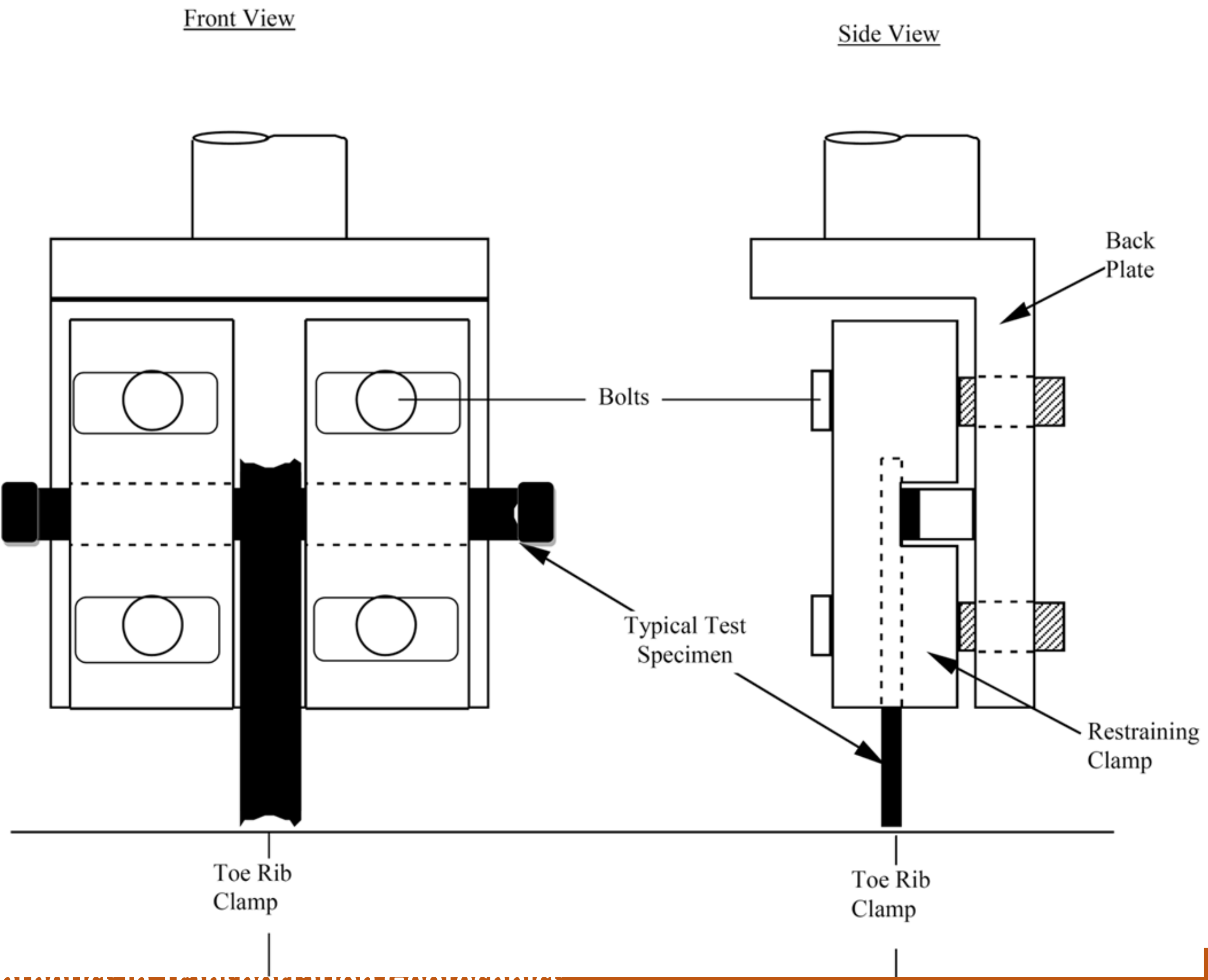
h = 0.276 m; average thickness of base course layer
R_E = 4.8; average $CBR_{bc.\ field}$ = 20, average CBR_{sg} = 1.79
P = 37.63 kN
r = 0.139 m
N_c = 5.71
c_u = 62.7 kPa
f_s = 75 mm

- 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

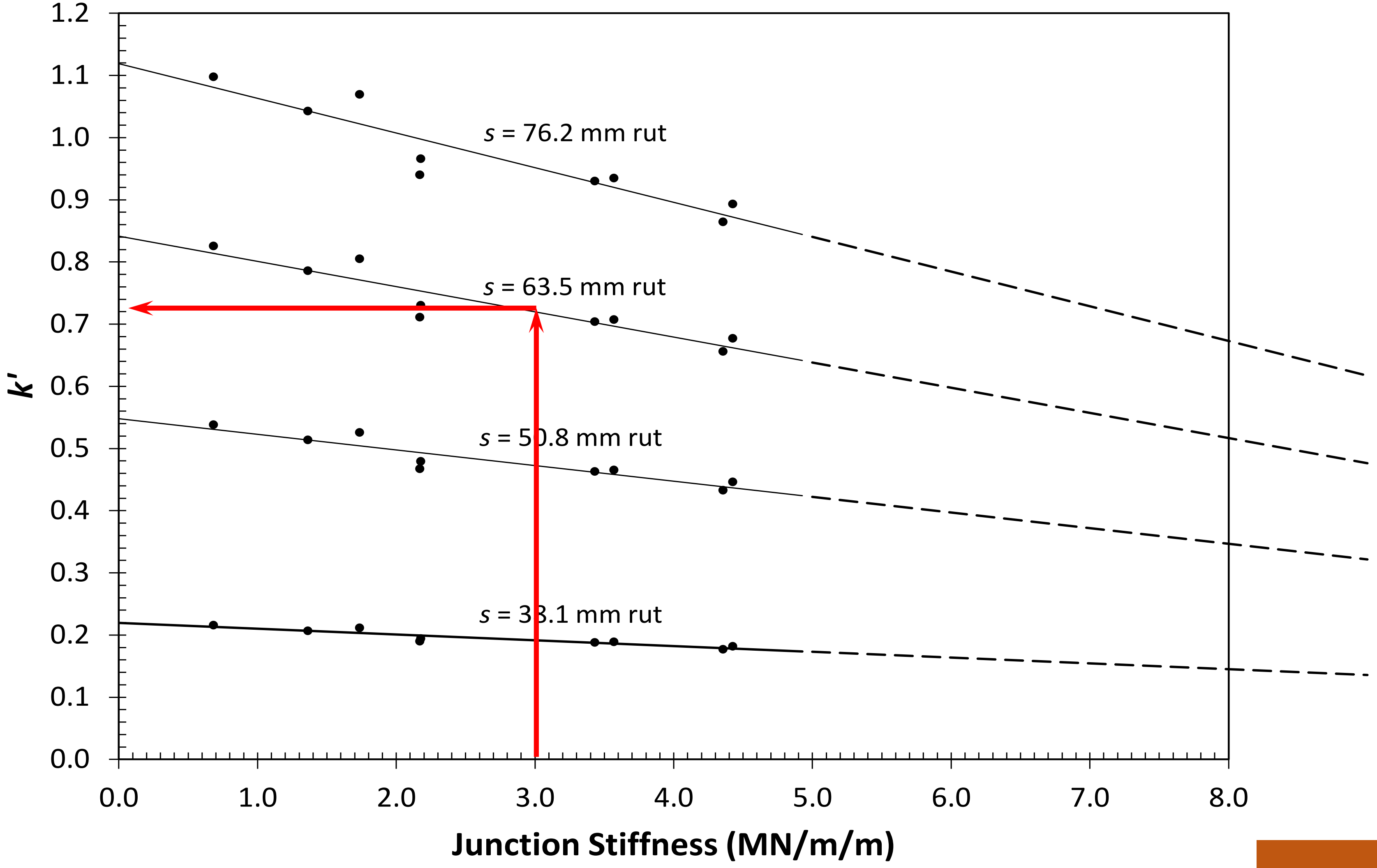


Junction Strength/Stiffness in XMD

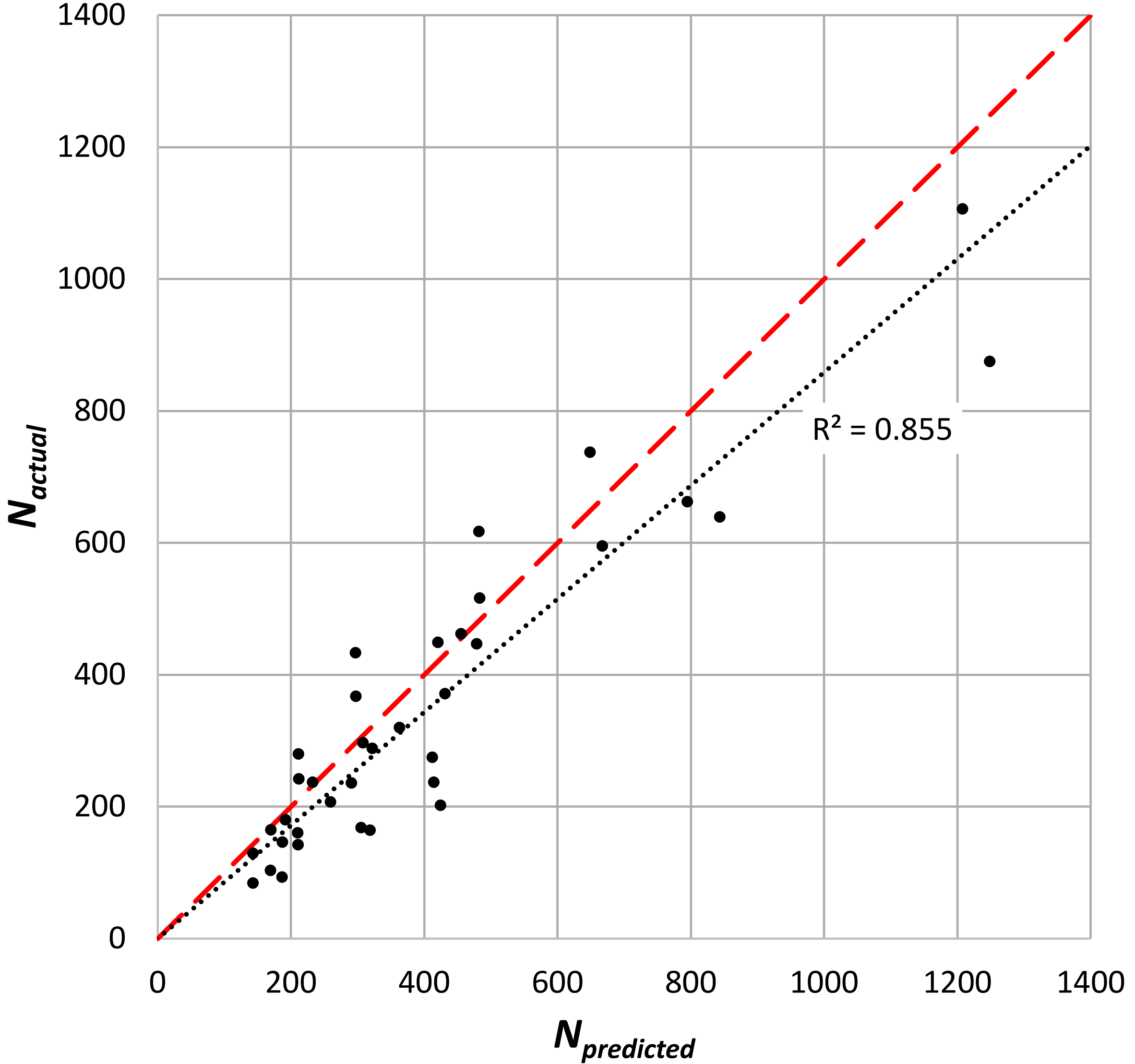


- Ultimate strength of junction in shear
- Junction stiffness = secant stiffness at 1.3 mm displacement {MN/m/m}

Junction Stiffness versus *k'*



Validation



Final form of design equation:

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

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

MONTANA
STATE UNIVERSITY
College of
ENGINEERING
Western Transportation Institute

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3rd ICTG 2016

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Workshop 1 – Geosynthetics in Transportation Geotechnics

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

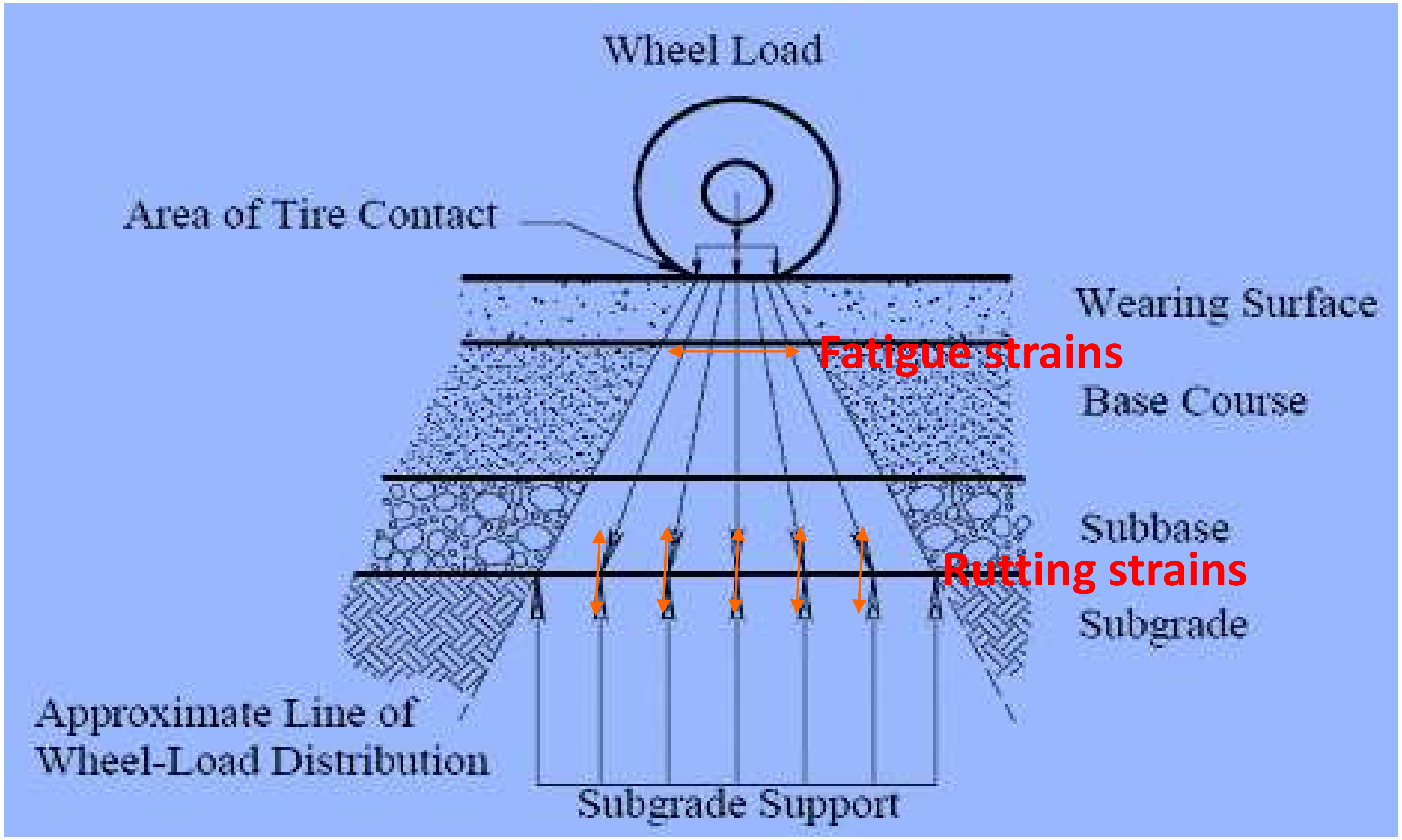
Sireesh Saride[1], Jorge Zornberg[2]

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

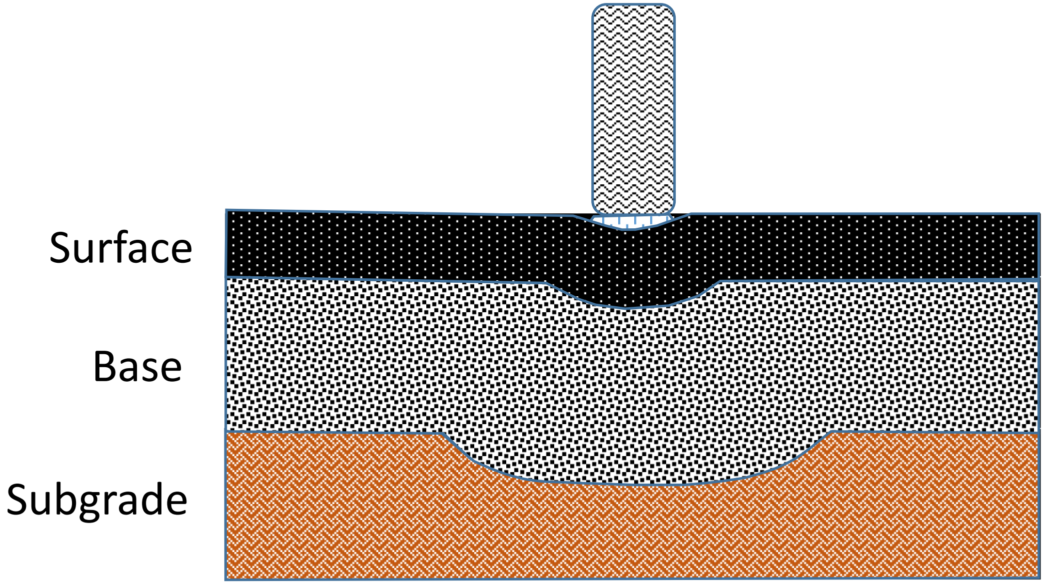
Issues with Flexible Pavements

Rutting

Fatigue cracking

Factors affecting Pavement Performance

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



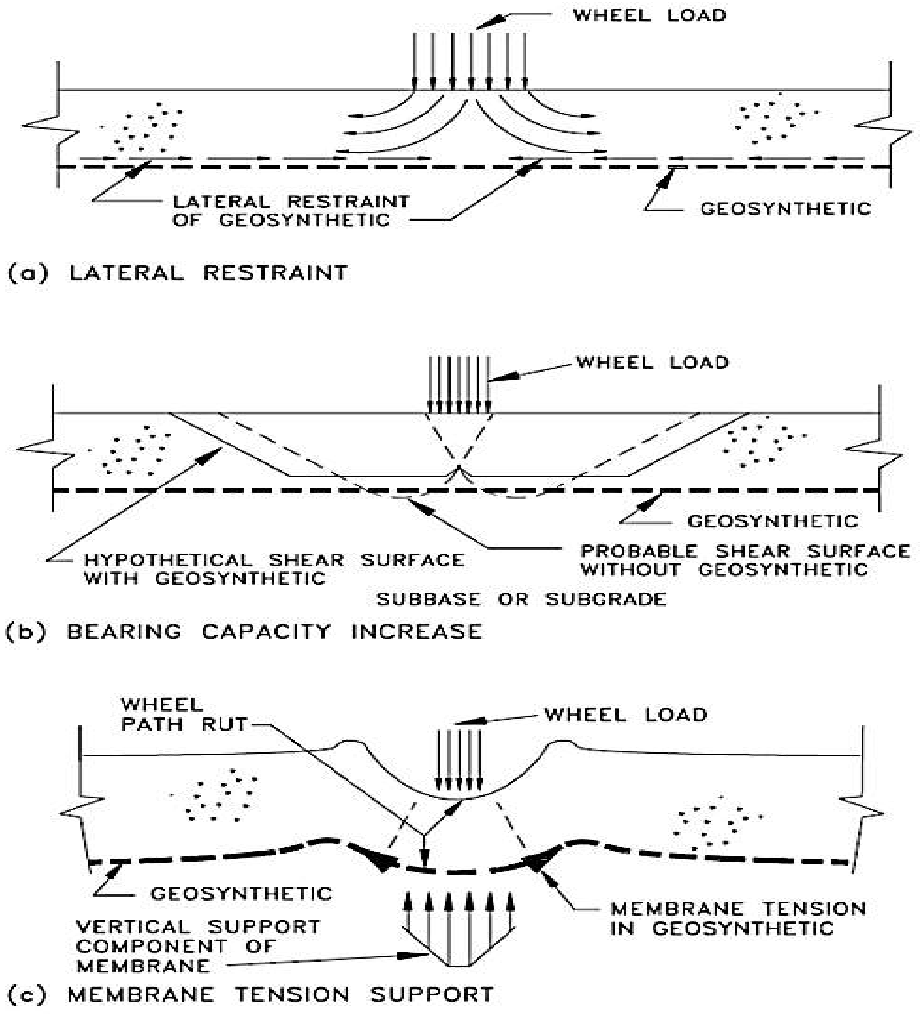
- **Hence, higher contact stresses would transfer to the weak subgrades**
- **Leads to high rutting…**

Improvement Techniques

- Stabilization Techniques
  - Subgrade level
  - Base/subbase level
- Geosynthetic Reinforcements
  - Geogrids
  - Geocells

Classification of Geosynthetics

Possible Reinforcement Functions Provided by Geosynthetics



(After Haliburton et al., (1981)

Why Geocell?

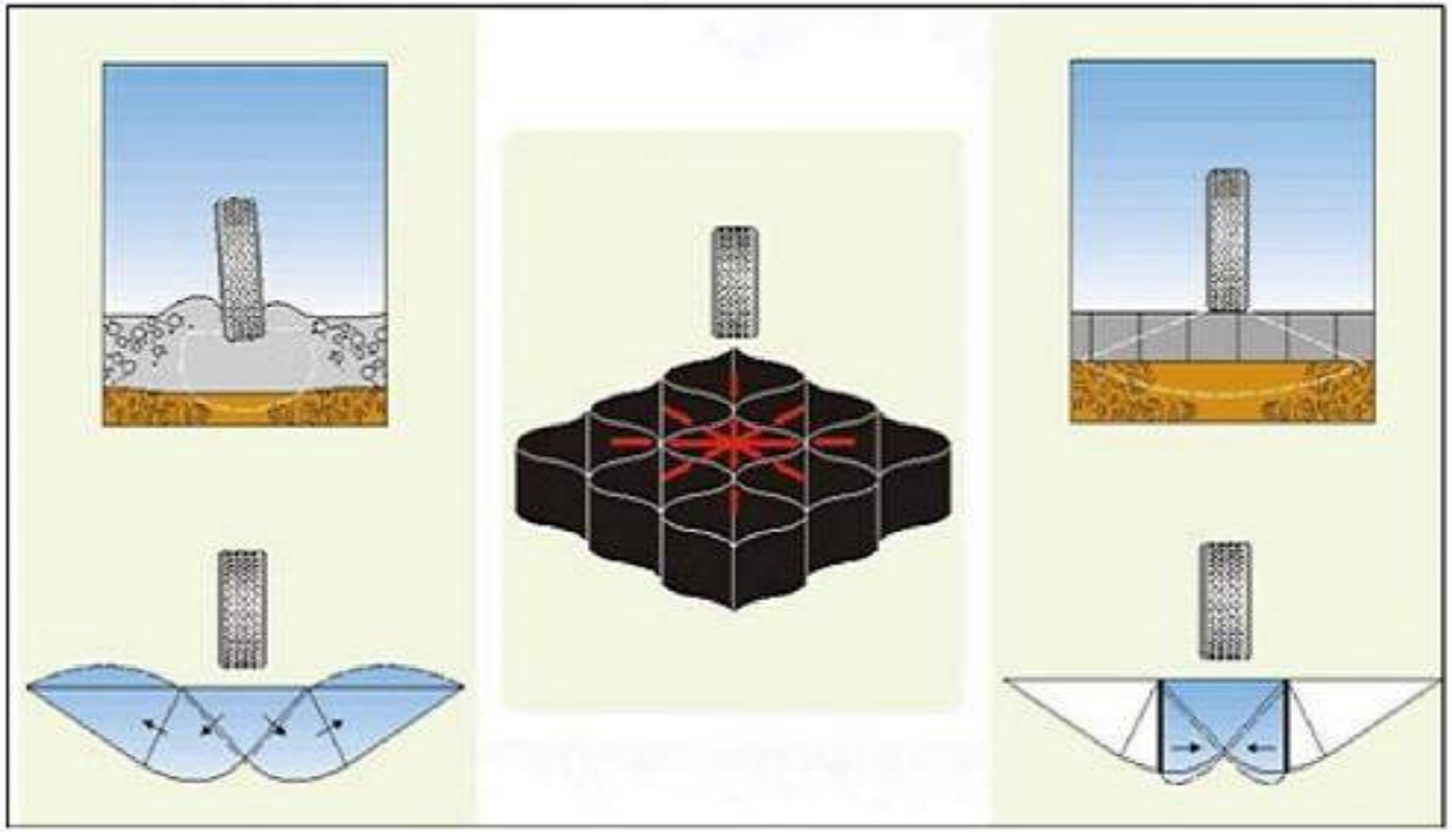


Image Source: www.esi.info

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

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

Horizontal Movement Arrangement

200 kN Capacity Actuator Frame

Hoses Connected to Actuator

Computer to Control the Actuator

DAQ Arrangement

100 kN Actuator

Actuator Movement Controller

LVDT's

Plate and Plunger Arrangement

Test Box
(1 M x 1 M X 1 M)

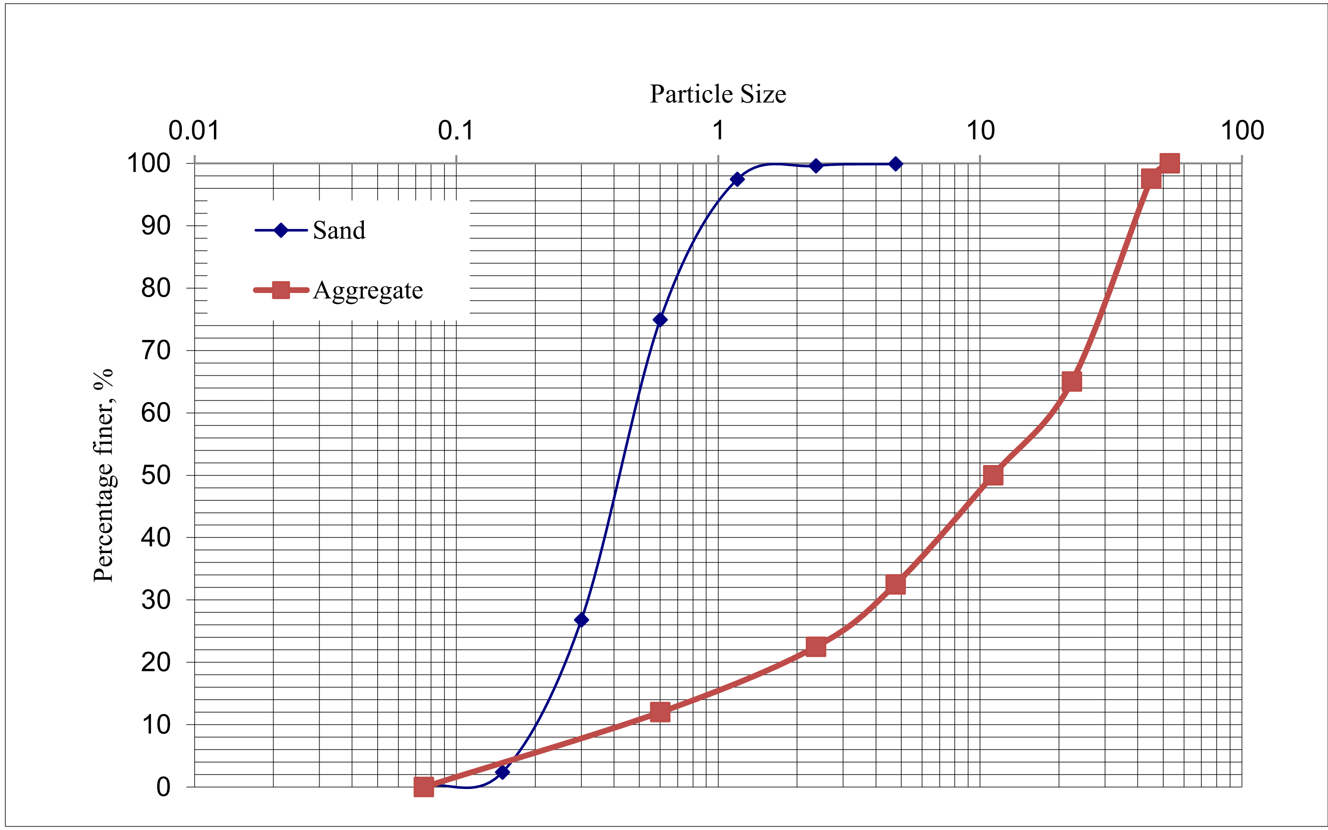
Computer Controlled Servo Hydraulic Actuator Test Setup

Materials used

- Sand
- Aggregate
- Clay
- HDPE Geocell
- Surface Layer

Properties of Dry Sand & Aggregates

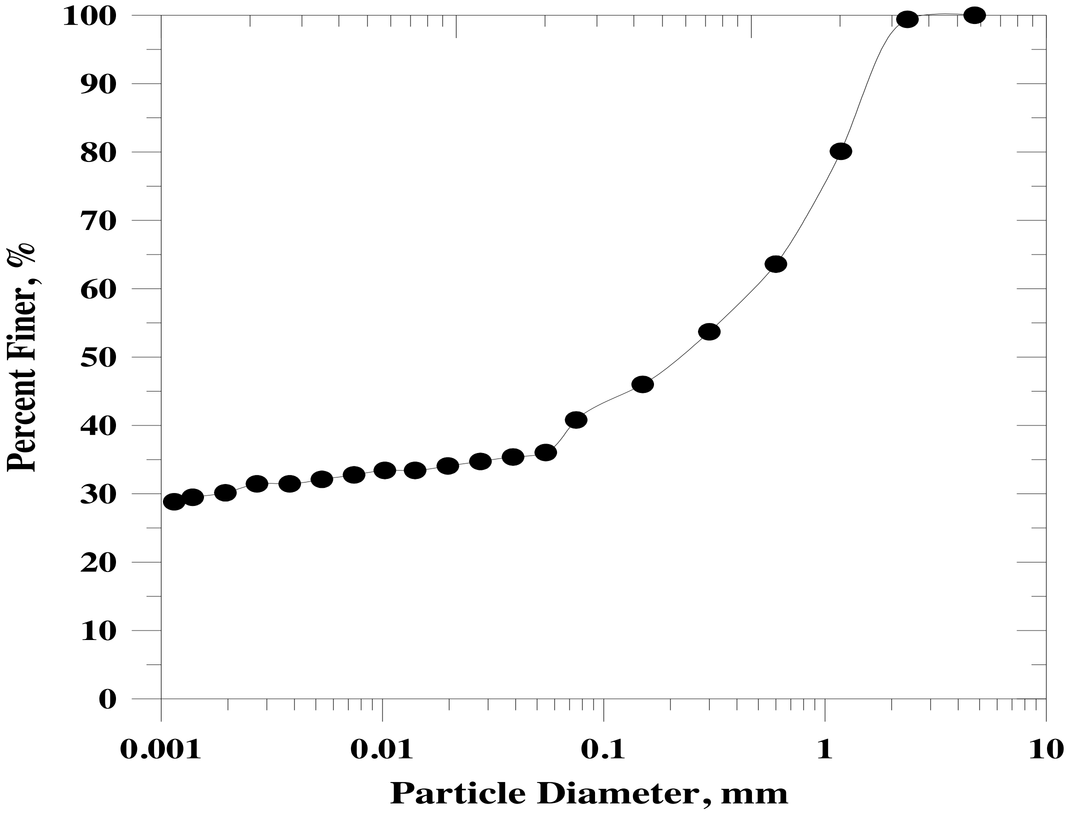
Properties	Values
D_{10}, mm	0.20
D_{30}, mm	0.32
D_{60}, mm	0.48
Sand Classification (USCS)	SP
c_u	2.40
c_c	1.07
Specific gravity	2.63
e_{max}	0.74
e_{min}	0.51
Φ at 75, 70, 30 % R_D	41°, 37°,34°



Material	Classification
Aggregate	MoRTH’s Base Grade III

Properties of Clayey Soil

Properties	Values
Clay Classification (USCS)	SC
Plastic Limit	21
Liquid Limit	46
Specific gravity	2.69
MDD (gm/cm^3)	1.72
OMC (%)	15



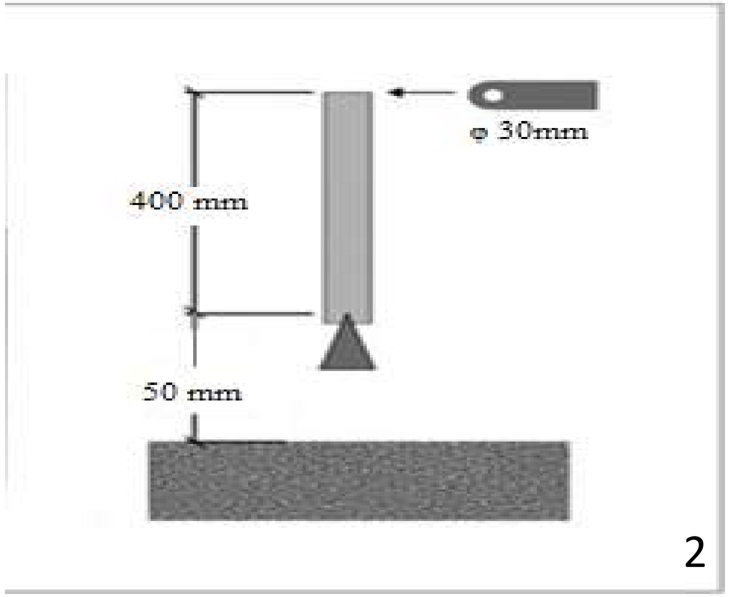
Engineering Properties of Geocells

Properties	**Values**
Material Composition	Polymer – High Density Polythylene (HDPE) with density of 0.935-0.965 g/cm^3
Weld Spacing (mm)	356
Cell Depth (mm)	75, 100, 150, 200
Cell Size (±10%) (mm)	259 x 224
Cell Area (± 4%)	290
Min. Cell Seam Strength (N)	2100

Preparation of Test Section

1. A 5 kg static compactor – Clay subgrade
2. Pluviation / raining technique – Sand bases
3. A plate vibrator - Aggregate bases


1



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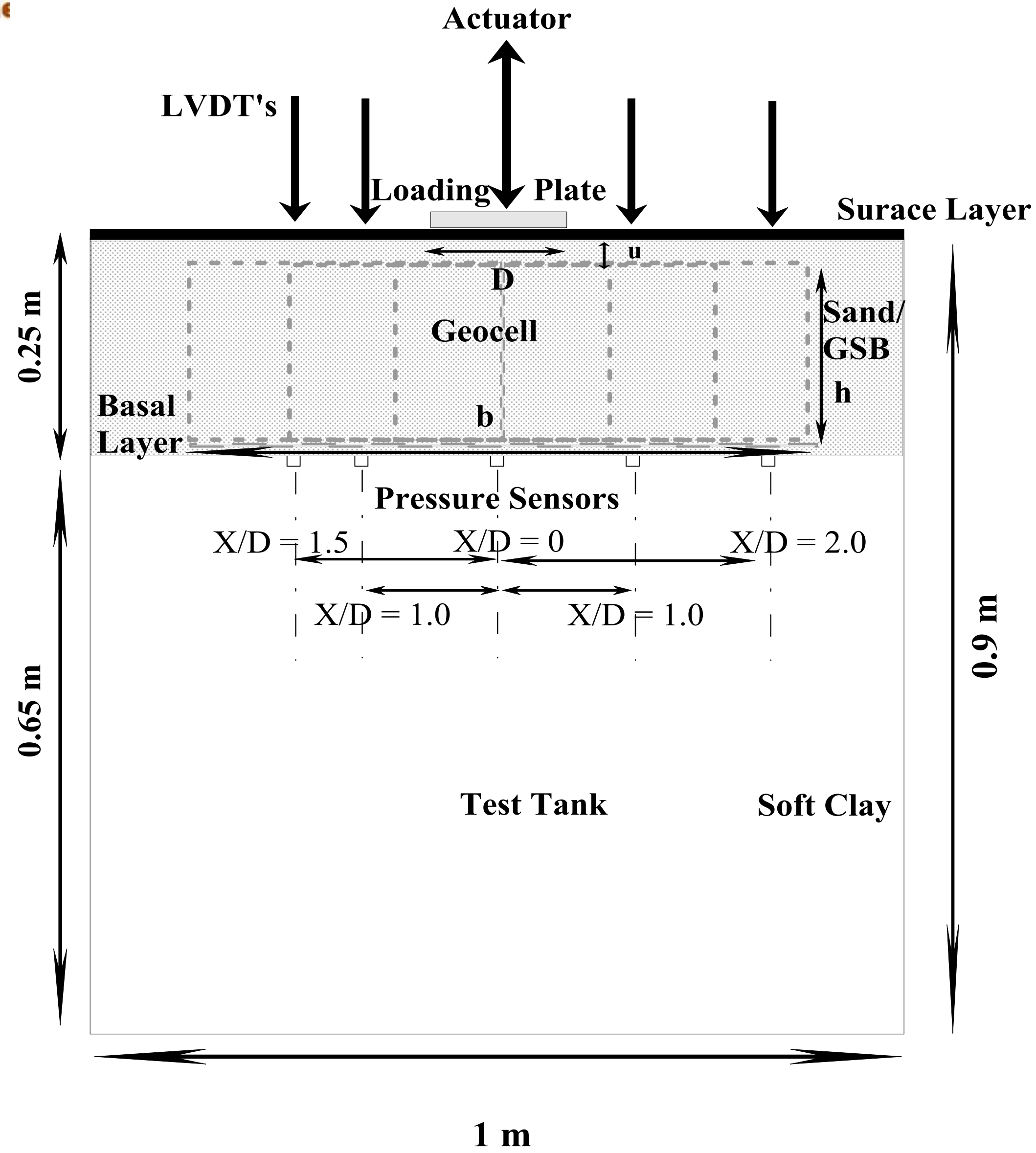
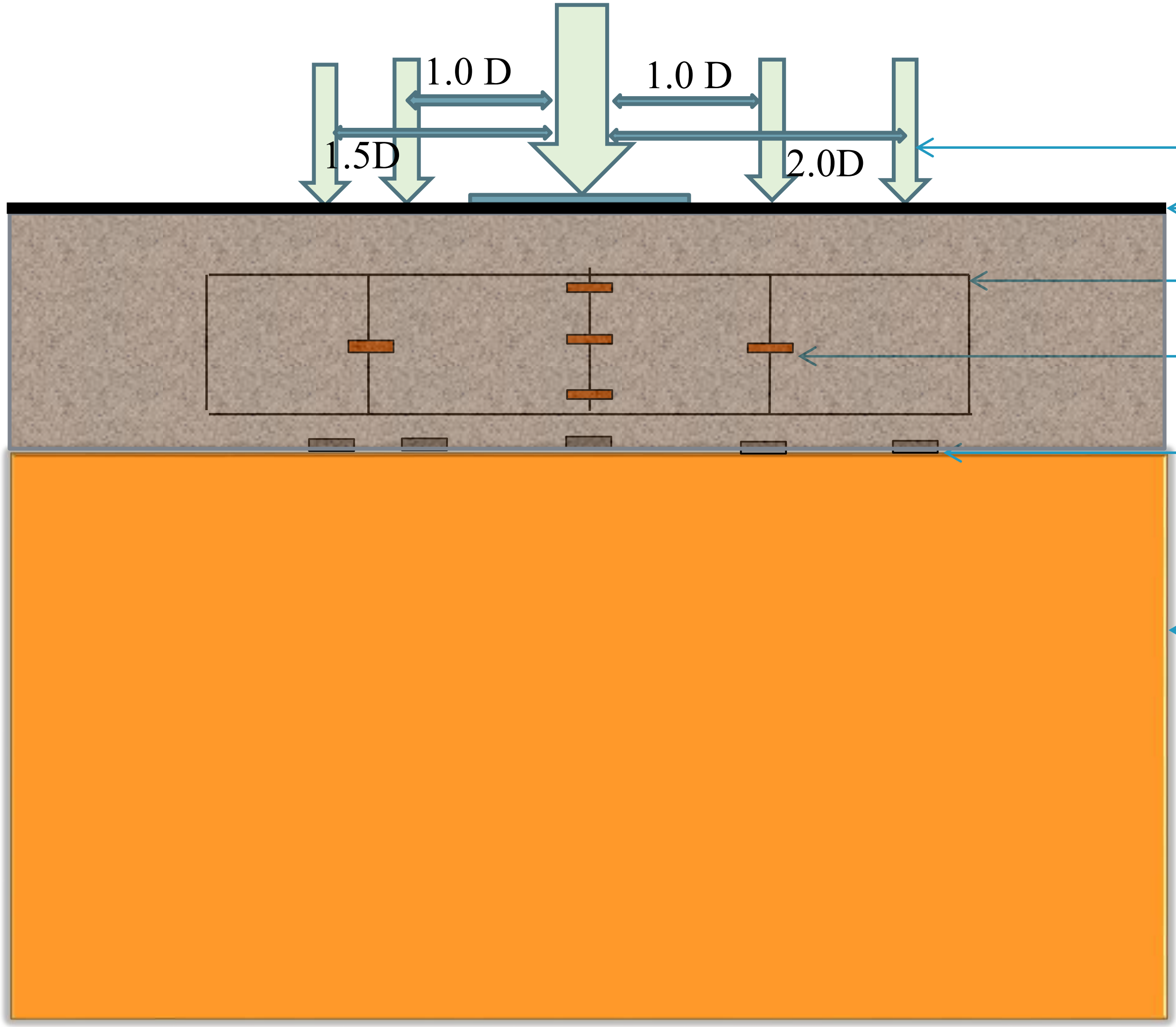
3



Typical Test Setup With Instrumentation



LVDT's
Surface layer
Geocell

Strain Gauges

Pressure Cells

Test Tank
(1m x 1m x1m)

Schematic of Test Setup

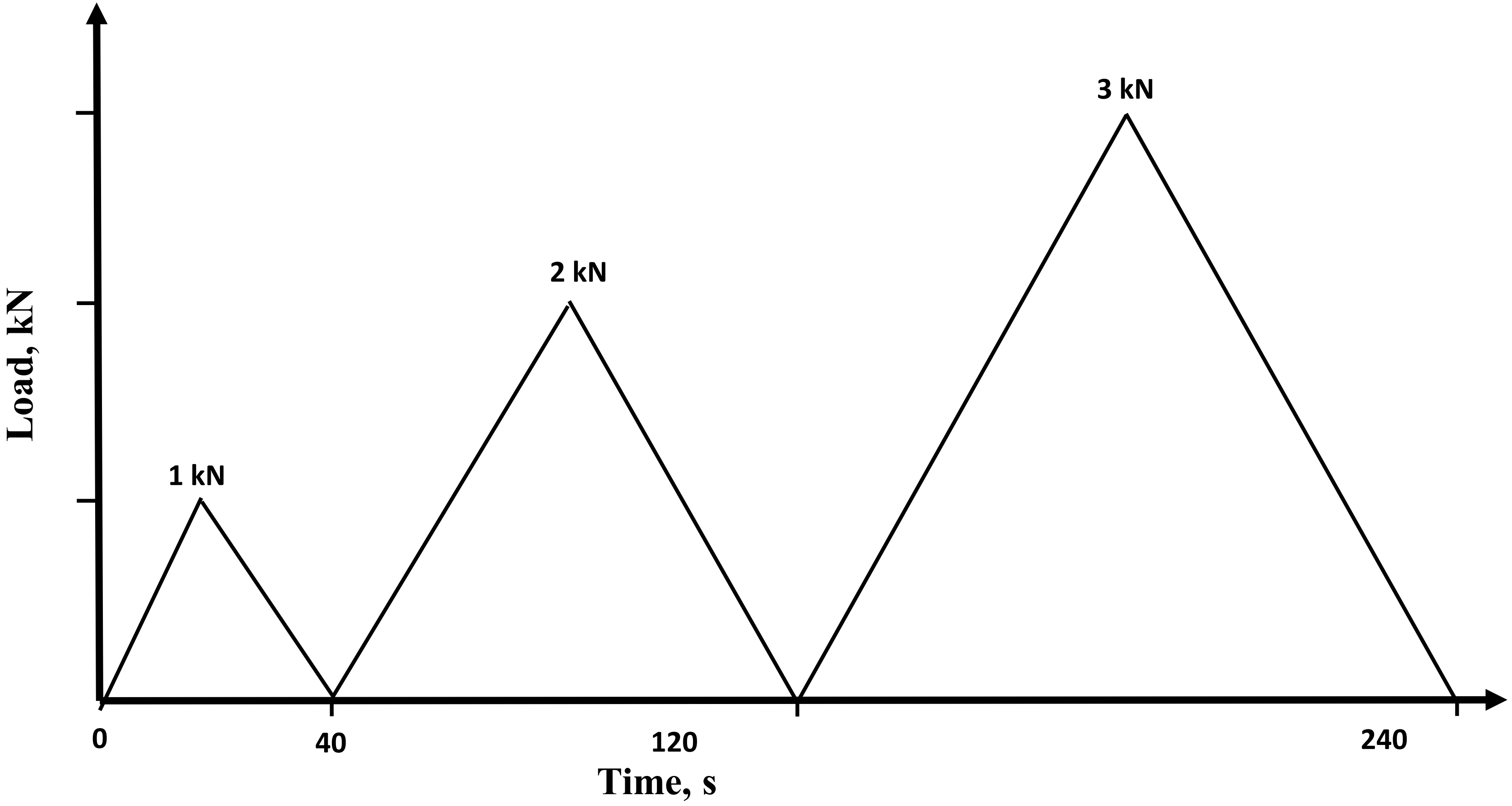
Instrumentation

Geocell with Strain Gauges

Earth pressure Cells

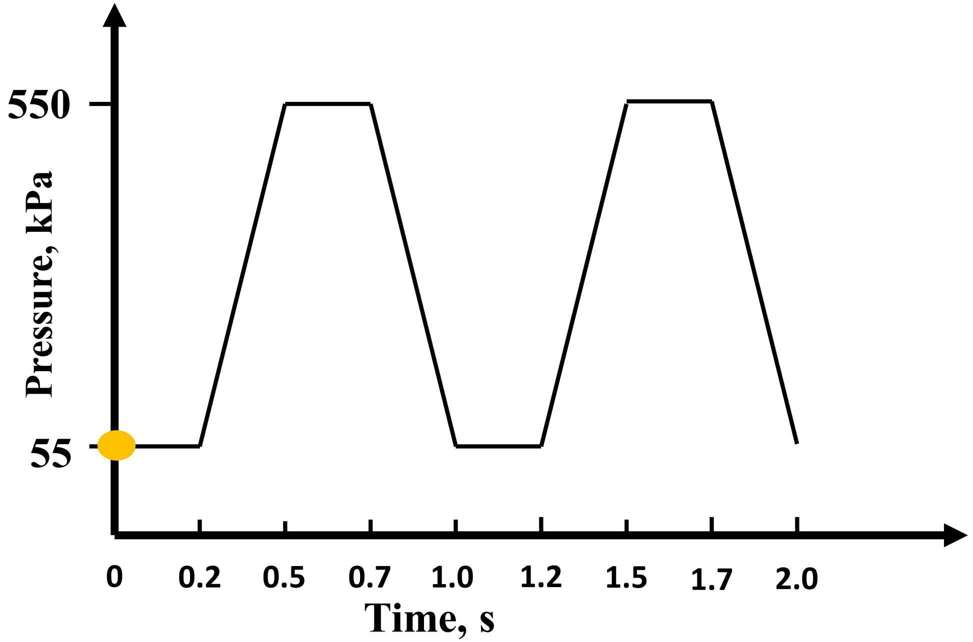
Cyclic Load Tests

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



Repetitive Load Tests

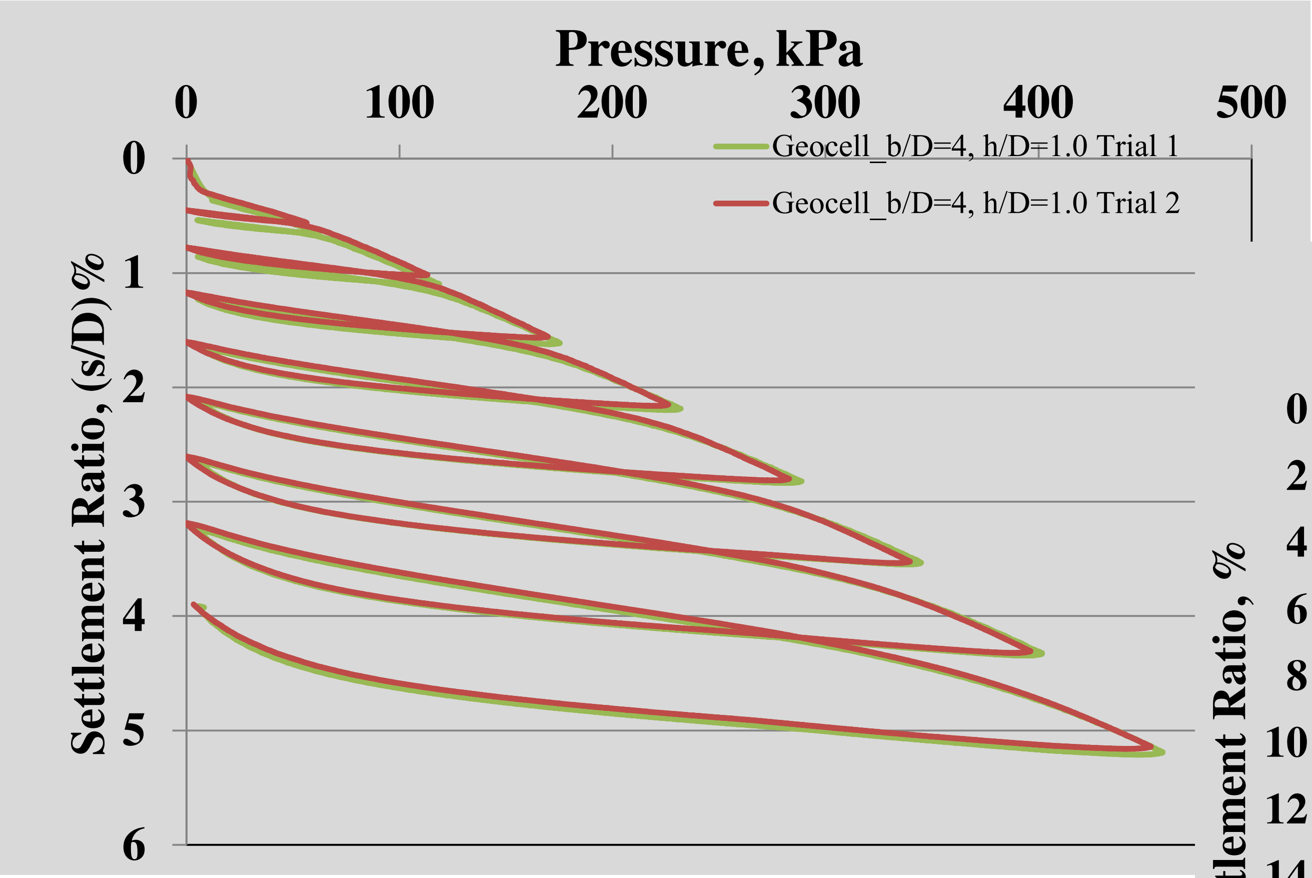
- Haversine load pulse at 1 Hz frequency



Testing Scheme - Granular Bases

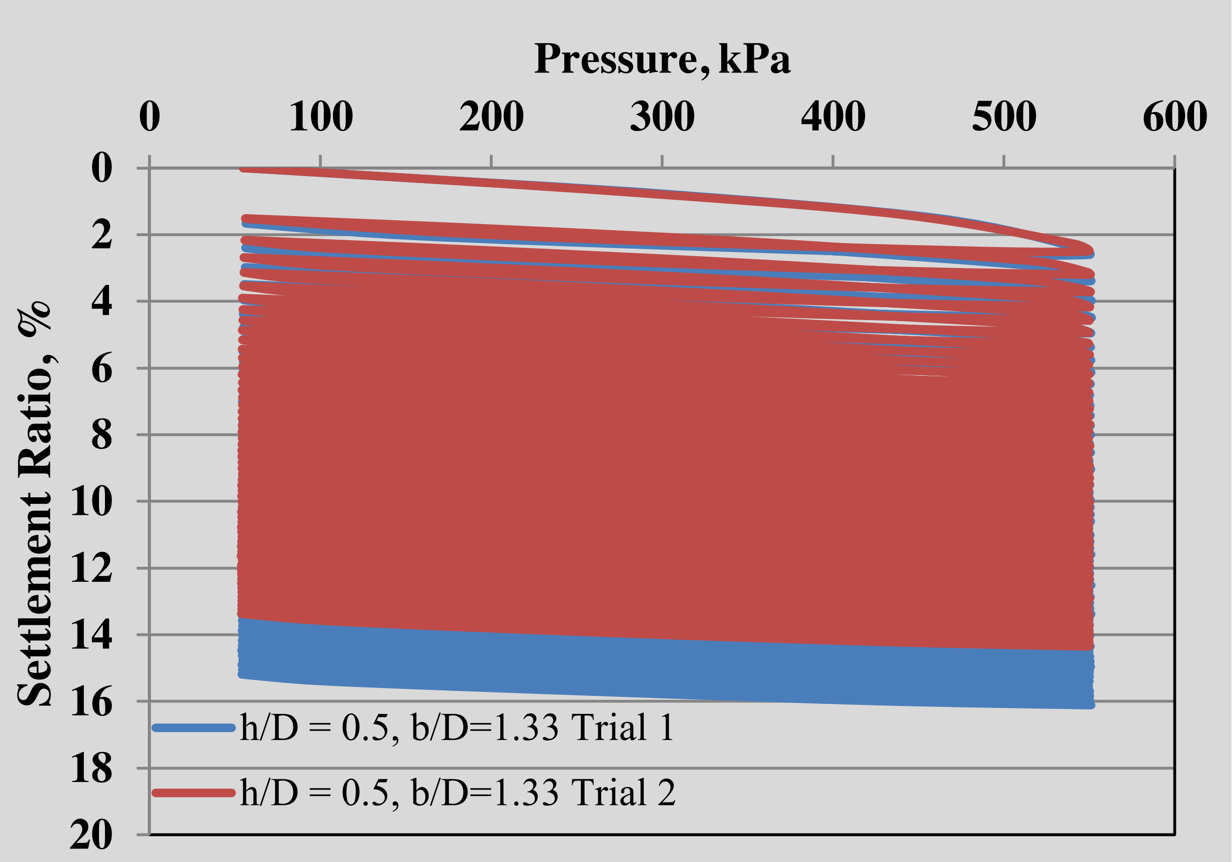
Description	Test Nomenclature	Constant Parameters
Unreinforced Granular aggregate base over Clayey Soil Subgrade	**U G C**	Υ_d = 23.1 kN/m^3 , C_u=10 kPa H/D = 1.67
Unreinforced Granular aggregate base over Clayey Soil Subgrade and Surface Layer	**U G C SL**	Surface Layer, Υ_d = 23.1 kN/m^3, C_u=10 kPa H/D = 1.67
Geocell Reinforced Granular aggregate base over Clayey Soil Subgrade	**G G C**	Υ_d = 23.1 kN/m^3 , C_u=10 kPa H/D = 1.67, b/D = 4, h/D = 1.33.
Geocell and Basal Geogrid Reinforced Granular aggregate base over Clayey Soil Subgrade	**G BG G C**	Υ_d = 23.1 kN/m^3 , C_u=10 kPa H/D = 1.67, b/D = 4, h/D = 1.33, B/D = 4.33.
Geocell Reinforced Granular aggregate base over Clayey Soil Subgrade and Surface Layer	**G G C SL**	Surface Layer, Υ_d = 23.1 kN/m^3 , C_u=10 kPa H/D = 1.67, b/D = 4, h/D = 1.33.
Geocell and Basal Geogrid Reinforced Granular aggregate base over Clayey Soil Subgrade and Surface Layer	**G BG G C SL**	Surface Layer Υ_d = 23.1 kN/m^3 , C_u=10 kPa H/D = 1.67, b/D = 4, h/D = 1.33, B/D = 4.33.

Repeatability of Tests



Cyclic load

Repeated load



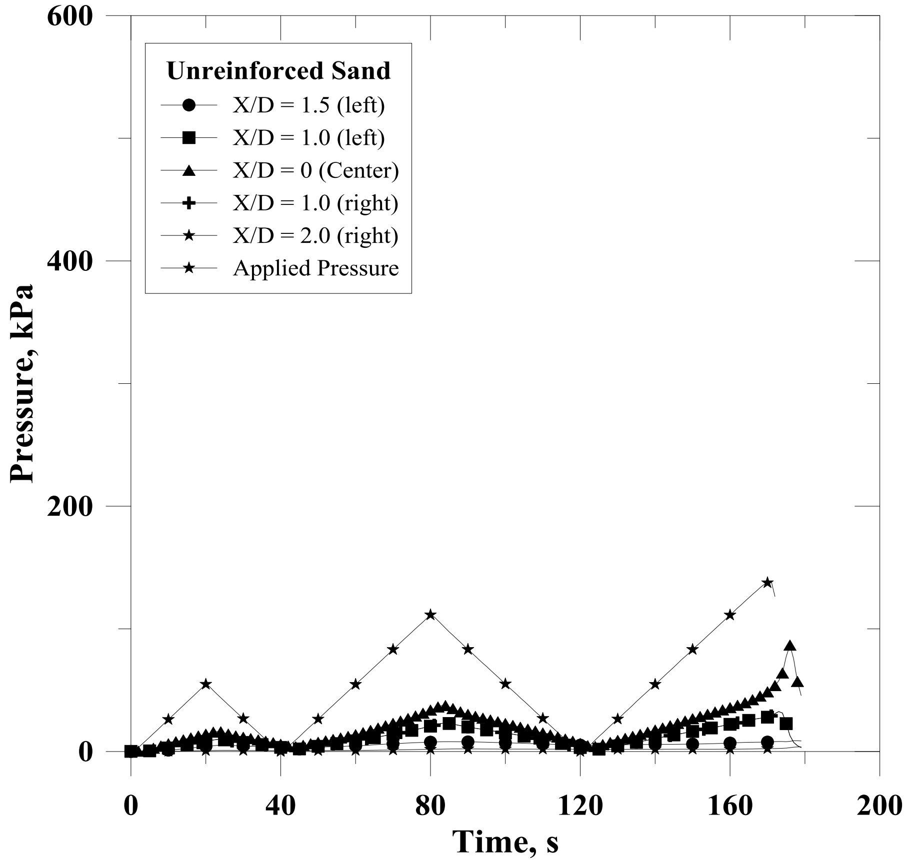
Performance Indicator

CPR: Contact pressure reduction

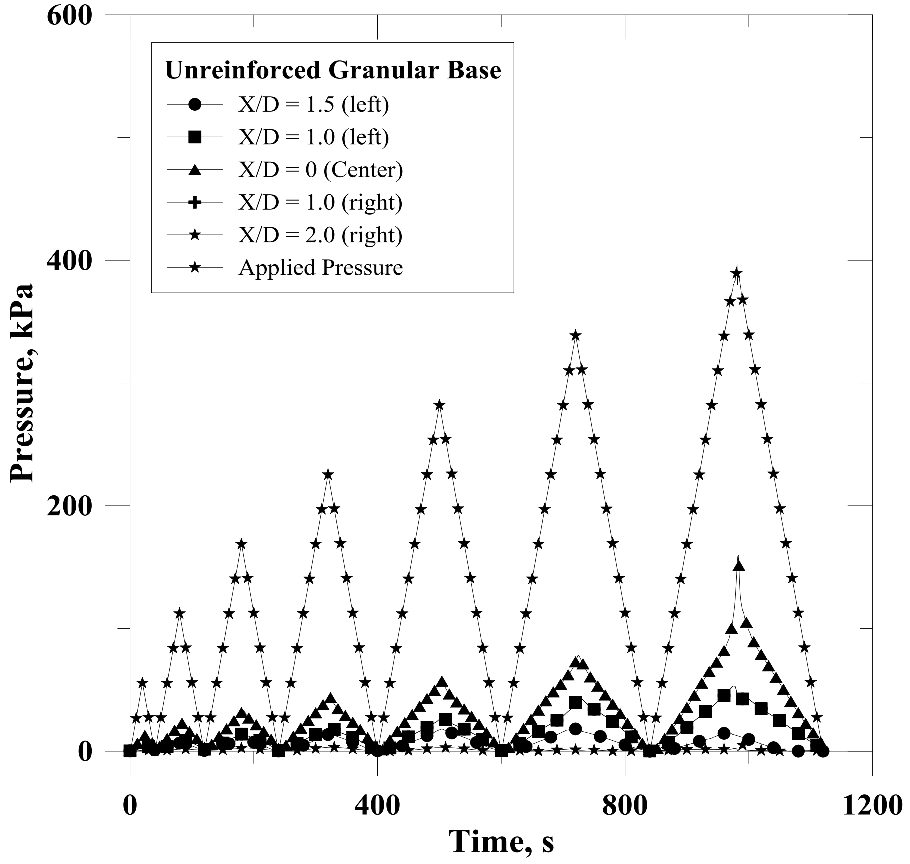
$$(CPR) = \left(1 - \frac{CP_{interface}}{AP}\right) \quad X\ 100$$

CP: Contact Pressure at the base-subgrade interface (kPa)
AP: Applied Pressure (kPa)

Contact pressure distribution in unreinforced beds without surface layer

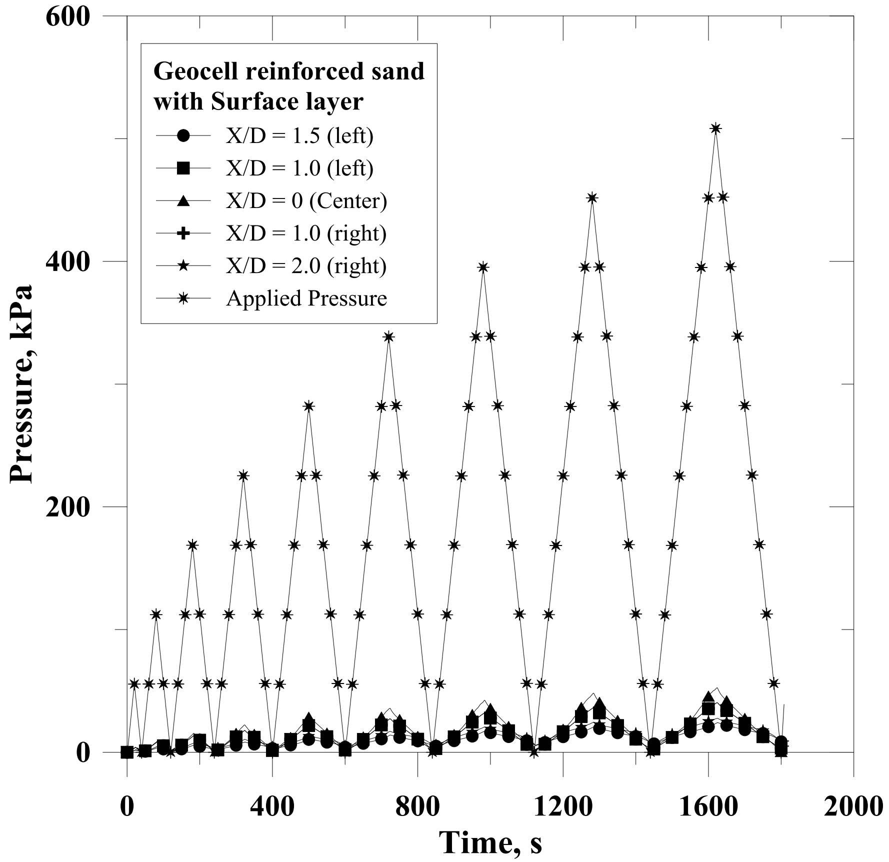


Sand Base

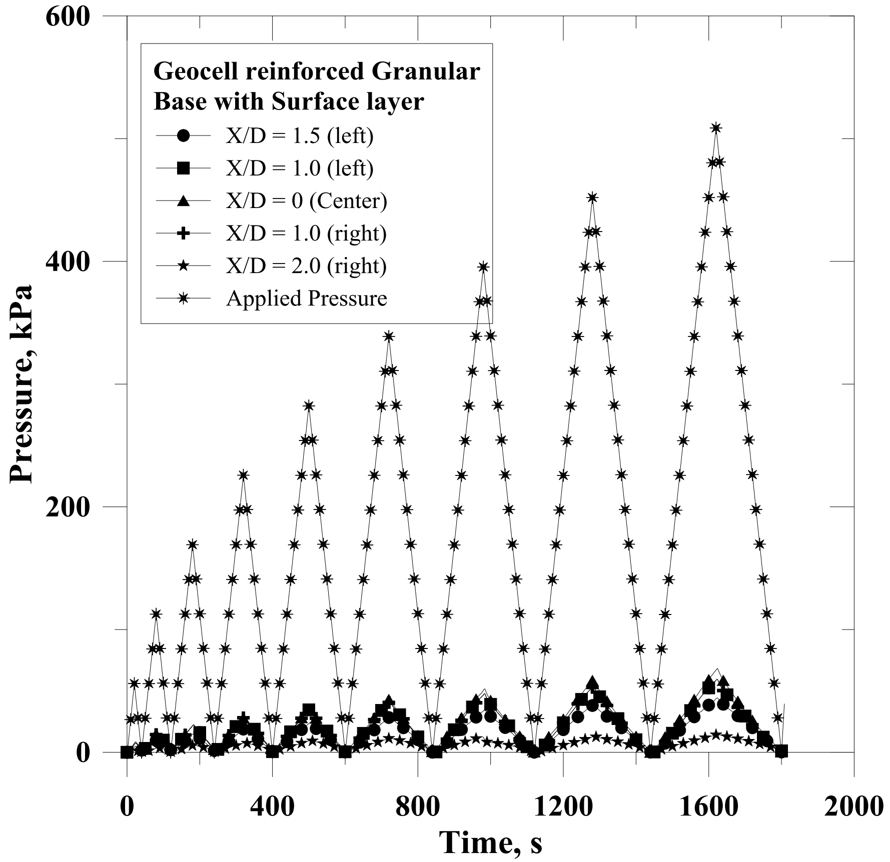


Granular Base

Contact pressure distribution in unreinforced beds with surface layer

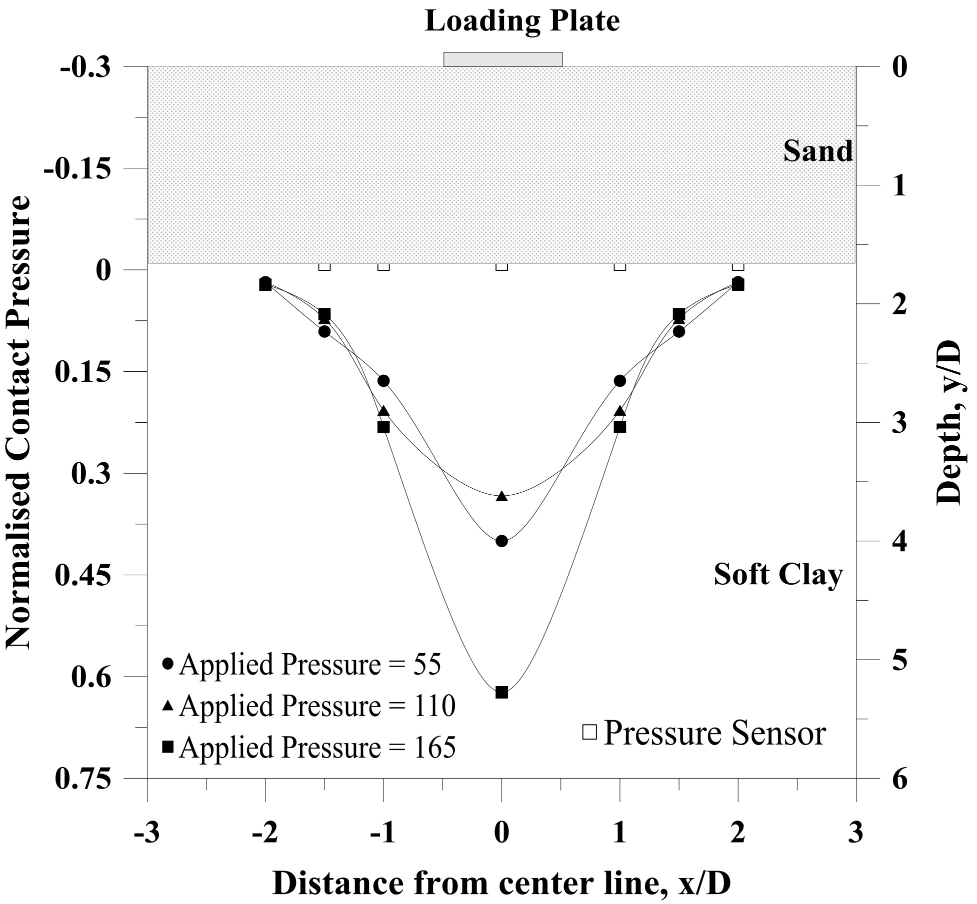


Sand Base

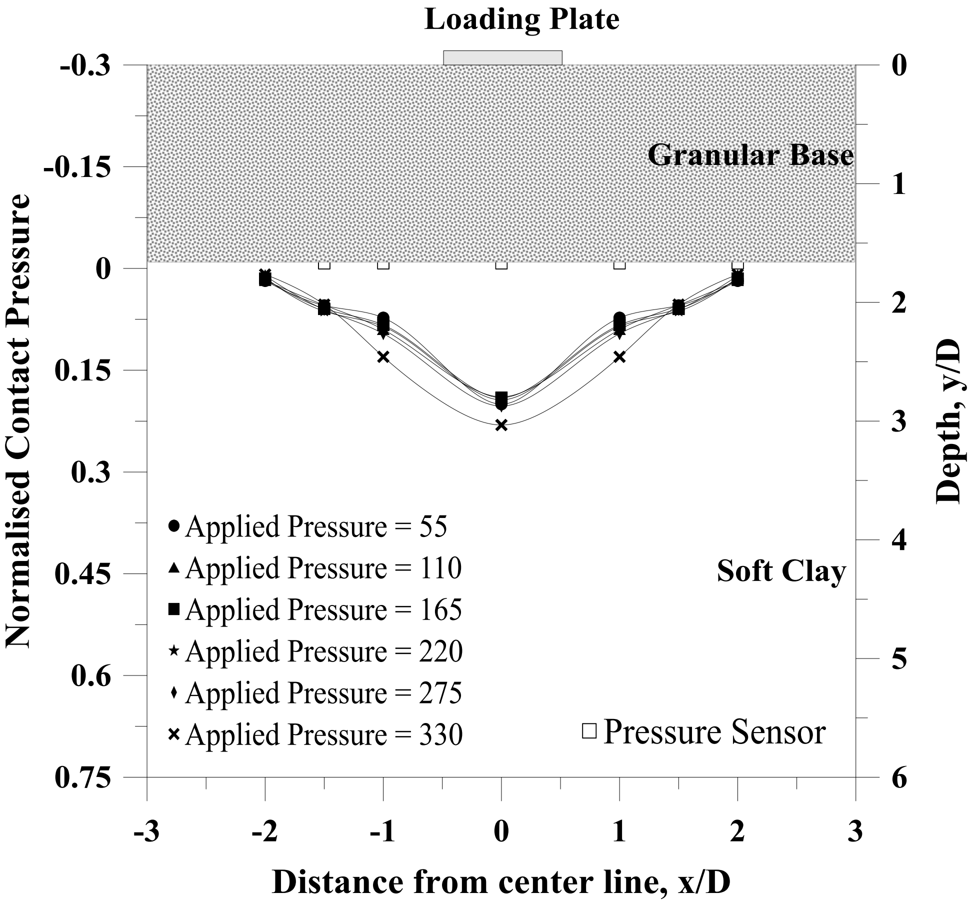


Granular Base

Contact pressure distribution in unreinforced beds without Surface Layer

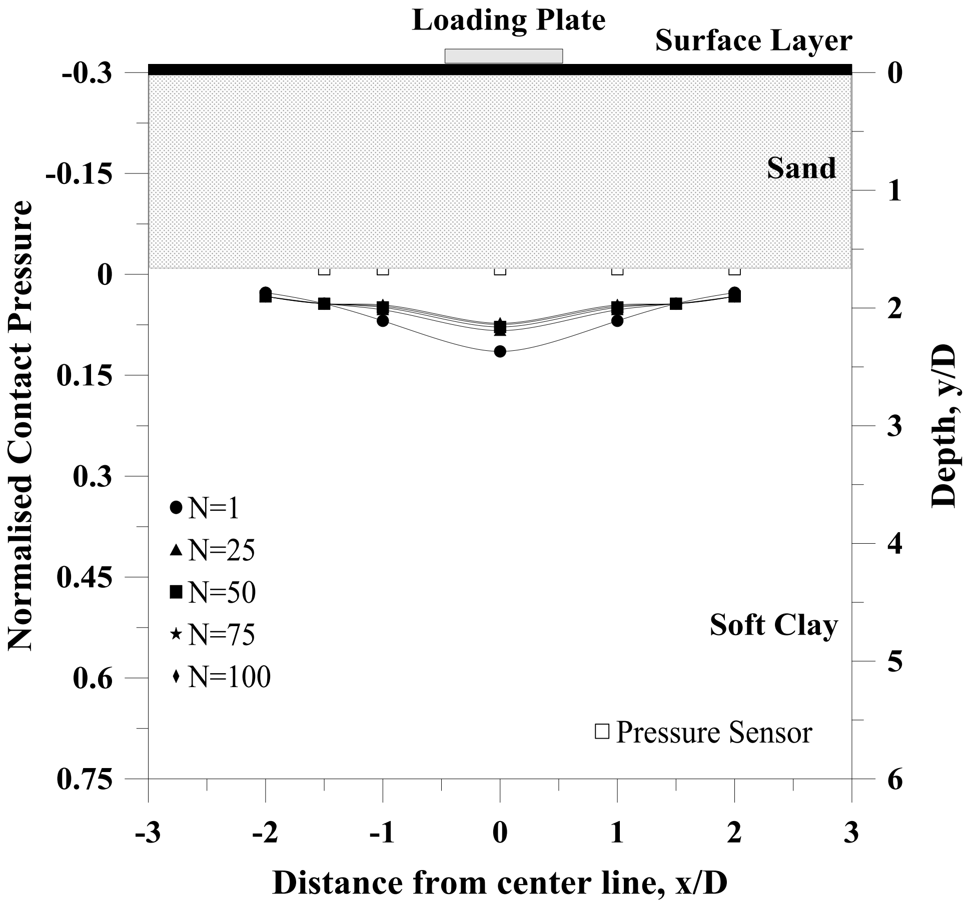


Sand Base

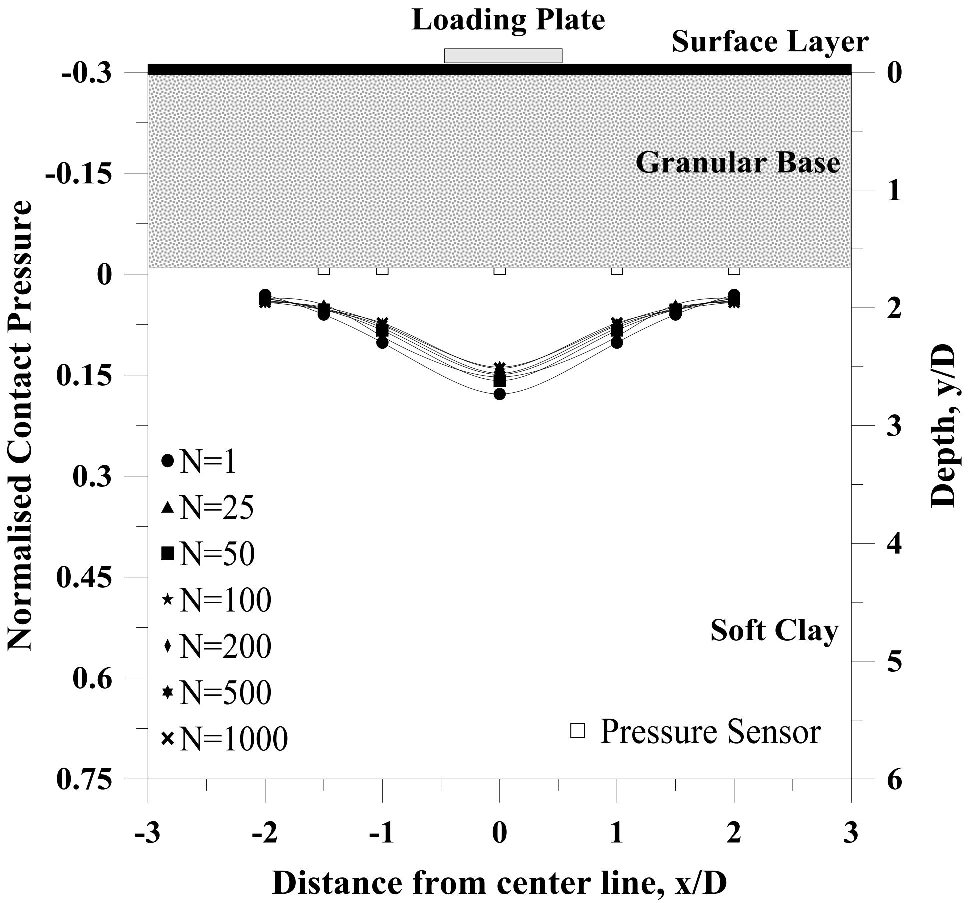


Granular Base

Contact pressure distribution in unreinforced beds with Surface Layer

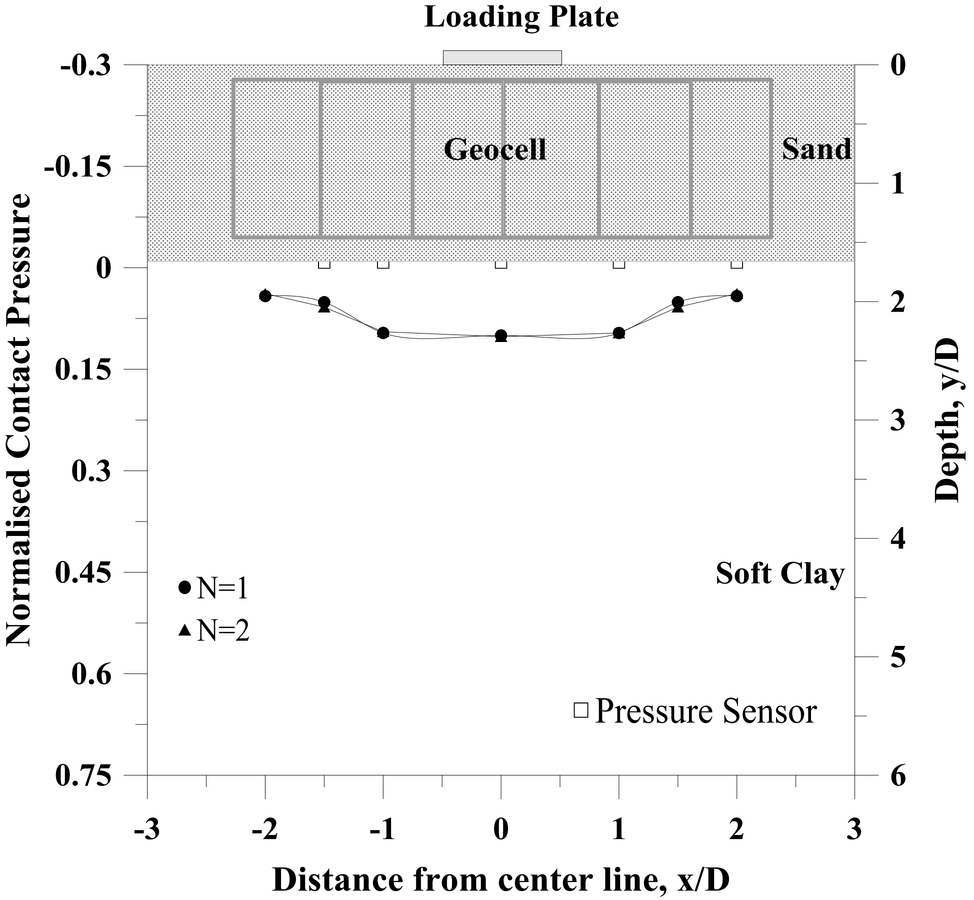


Sand Base

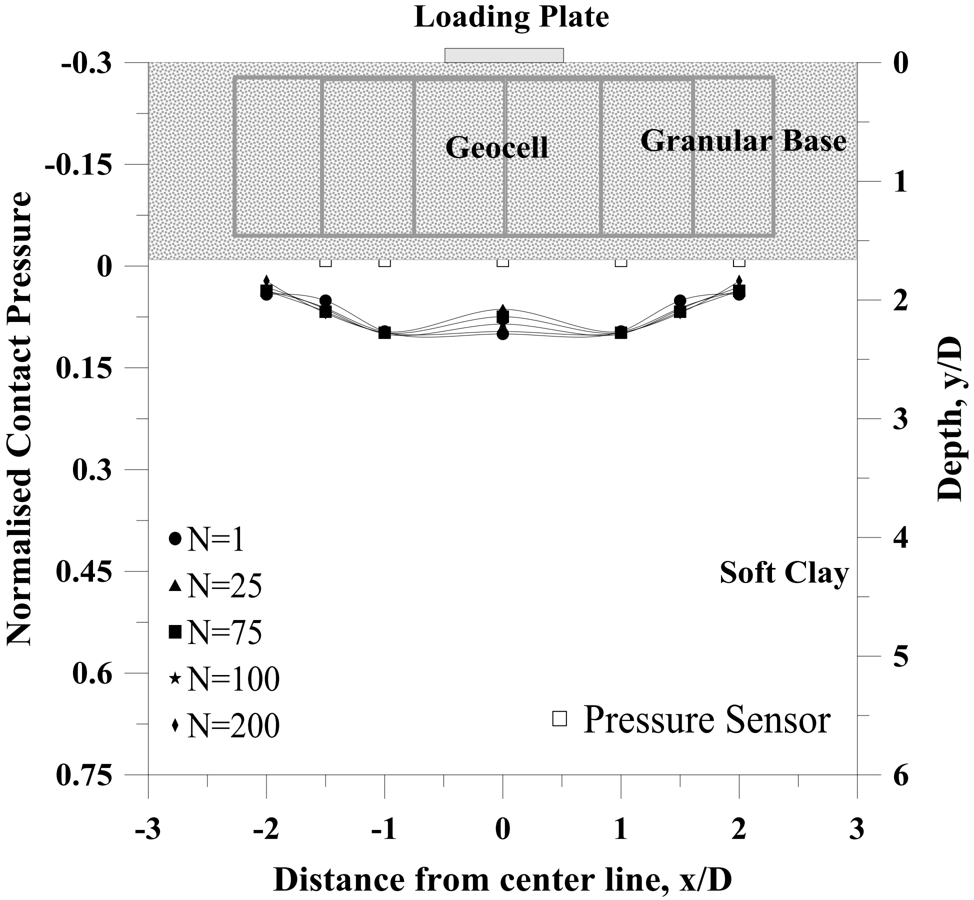


Granular Base

Contact pressure distribution in geocell reinforced beds without Surface Layer

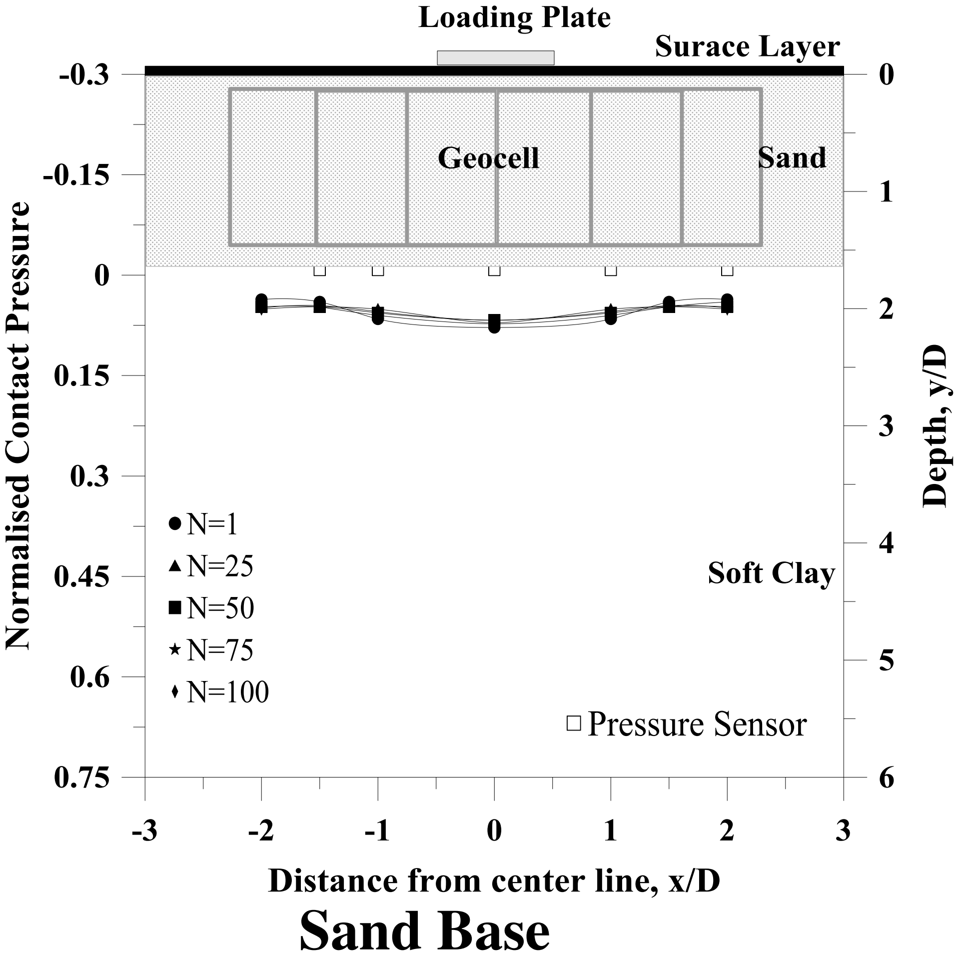


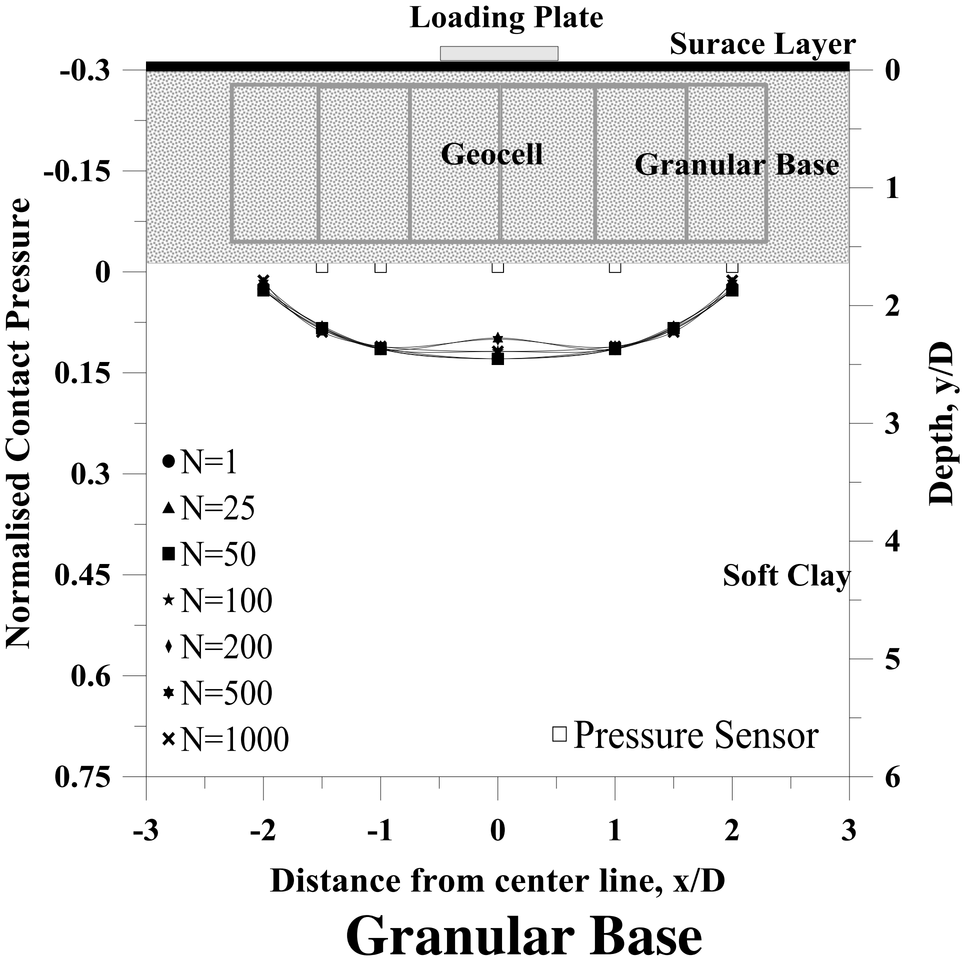
Sand Base



Granular Base

Contact pressure distribution in geocell reinforced beds with Surface Layer





Test Results

Test Case	CPR (%)		M_r
	CLT	RLT	
U S C	33.3	35	20
G S C	90.2	92	43
U G C	55	76	25
G G C	89	90	48

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

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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

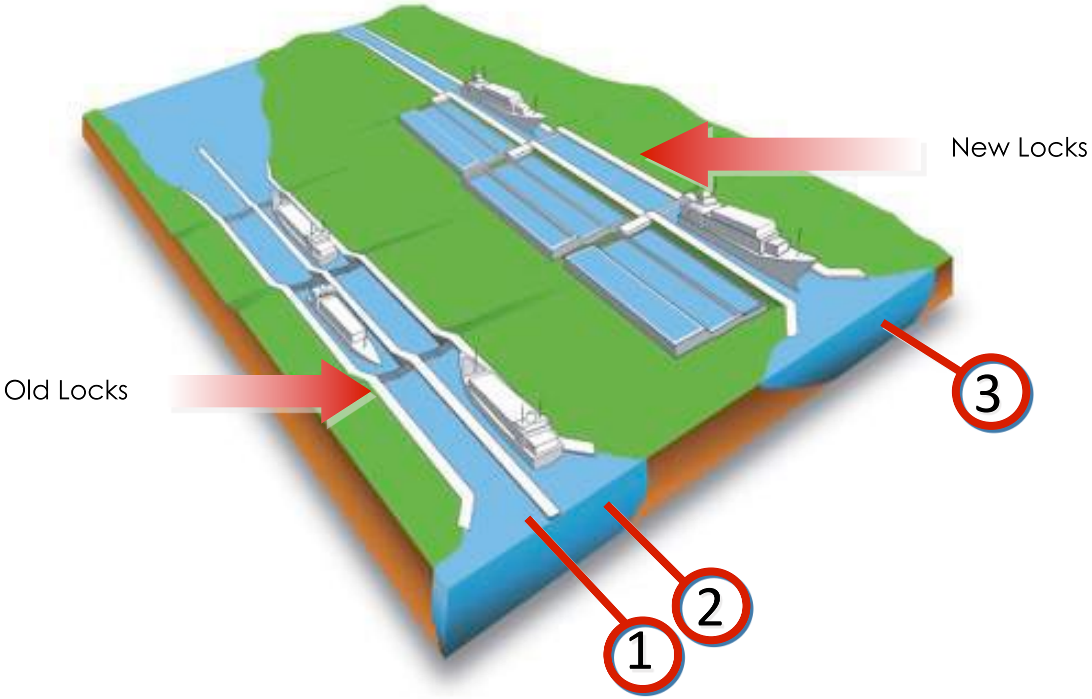
Water Saving Basins Panama

- 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

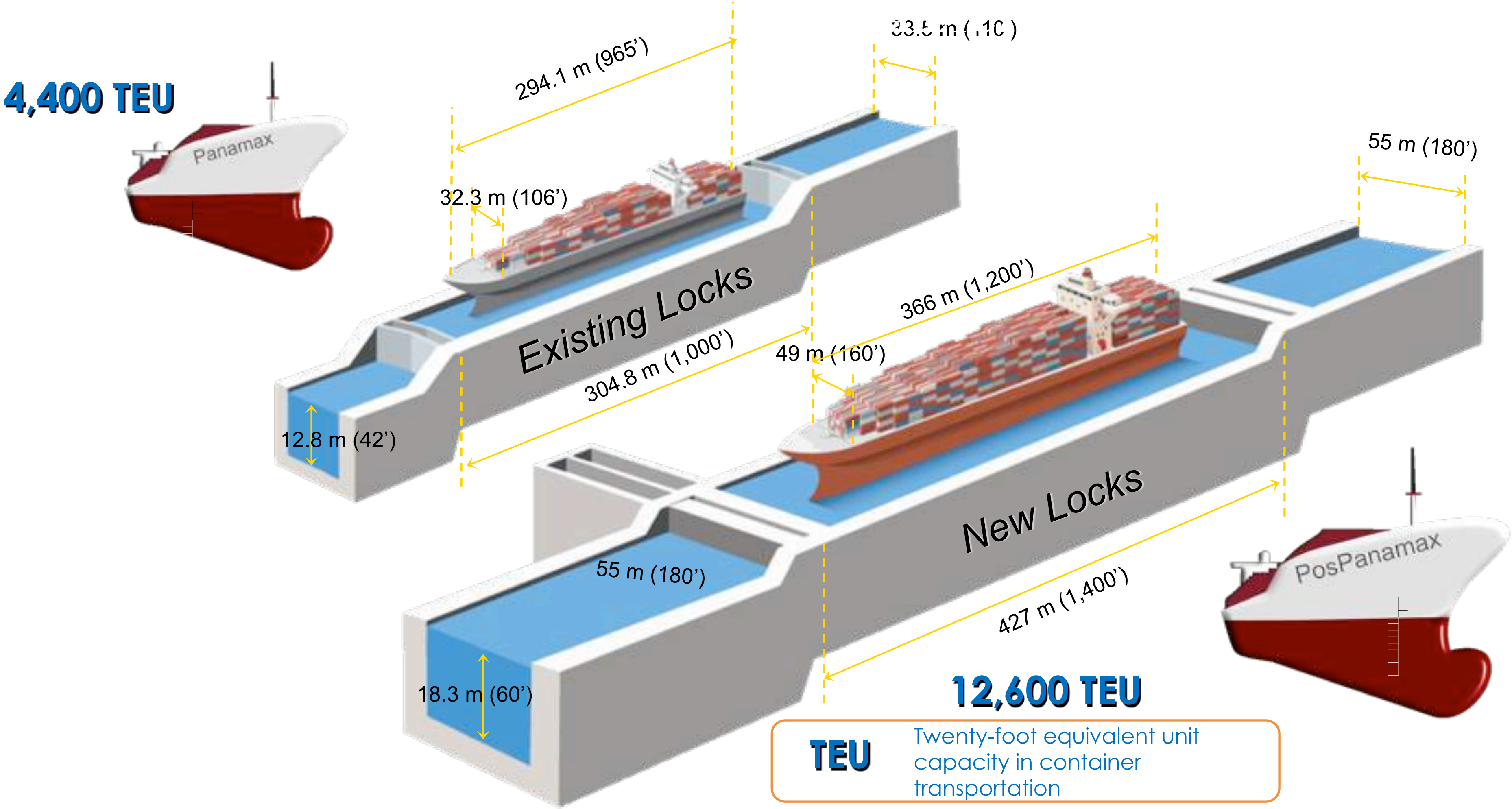
Localization of the Locks

Ubicación de las nuevas esclusas

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.

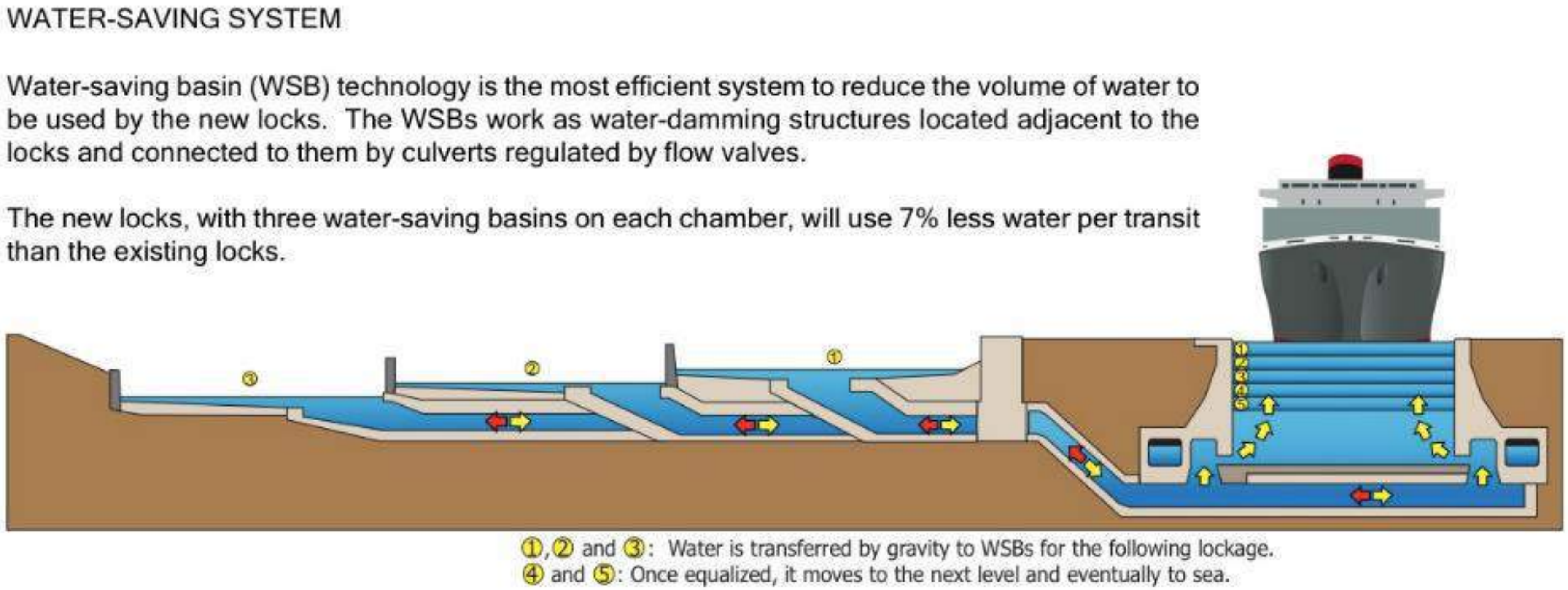


TEU Twenty-foot equivalent unit capacity in container transportation


LAKE GATUN
EXISTING CANAL
PACIFIC ENTRANCE
THE PANAMA CANAL
THIRD SET OF LOCKS PROJECT

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.



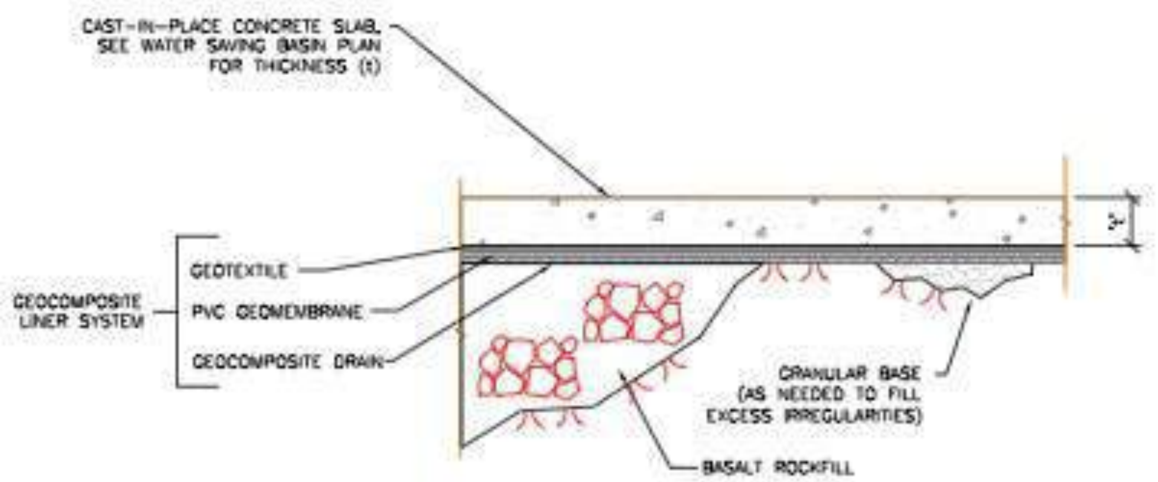
WATER-SAVING SYSTEM

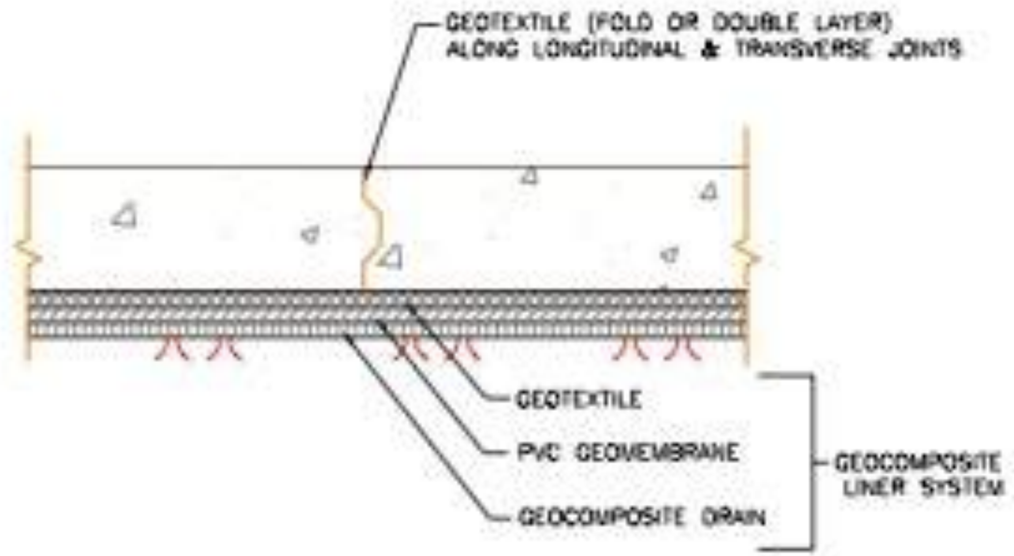
Water-saving basin (WSB) technology is the most efficient system to reduce the volume of water to be used by the new locks. The WSBs work as water-damming structures located adjacent to the locks and connected to them by culverts regulated by flow valves.

The new locks, with three water-saving basins on each chamber, will use 7% less water per transit than the existing locks.

①,② and ③: Water is transferred by gravity to WSBs for the following lockage.
④ and ⑤: Once equalized, it moves to the next level and eventually to sea.

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)

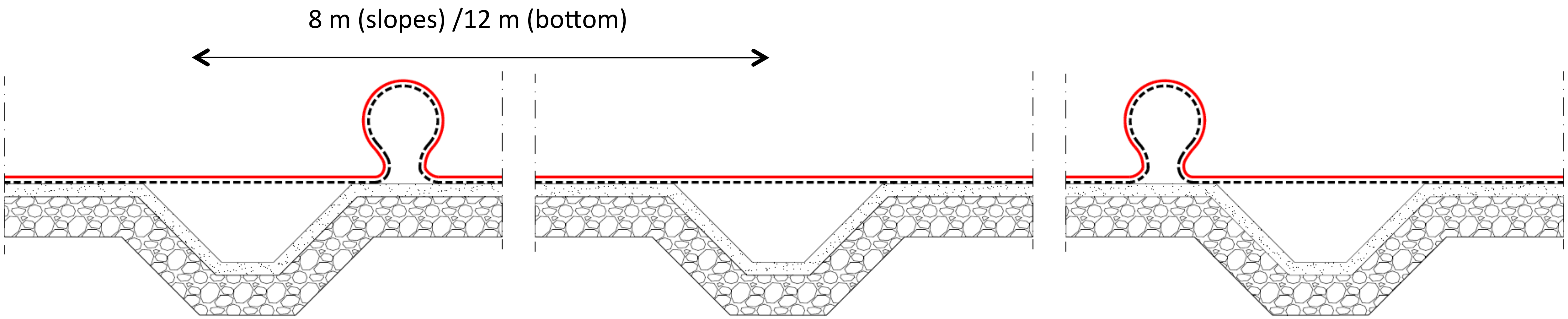
Carpi Solution Advantages

- 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

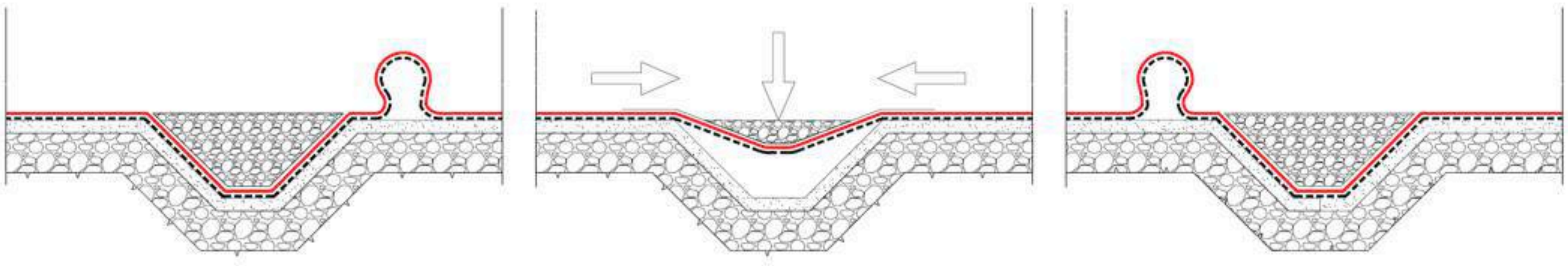
Tensioning Trenches Installation Sequence (Carpi patent)

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



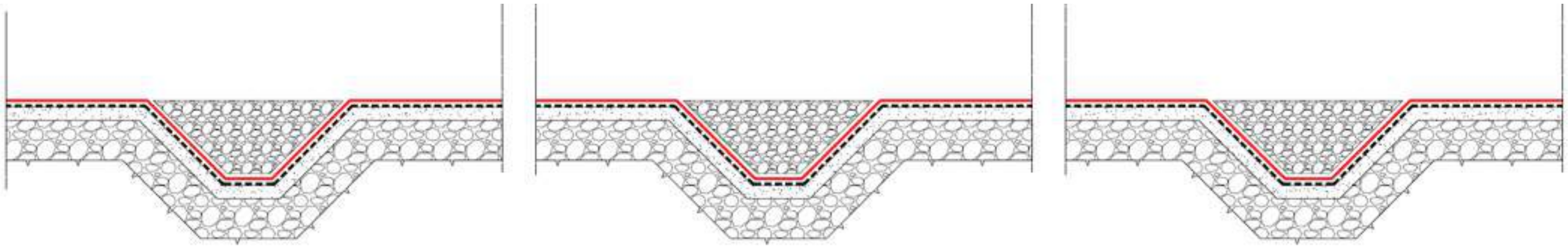
Tensioning Trenches Installation Sequence (Carpi patent)

- Stage 4: Tensioning of geocomposite by filling remaining alternate trenches



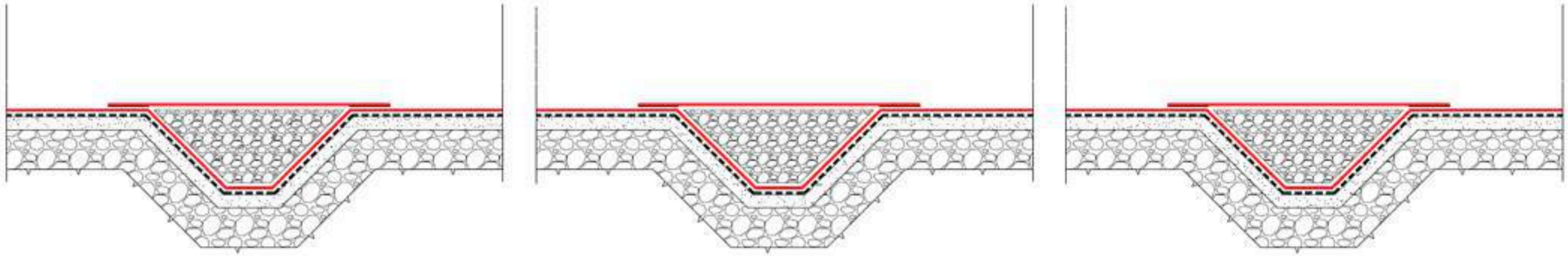
Tensioning Trenches Installation Sequence (Carpi patent)

- Stage 5: Tensioning of geocomposite by filling remaining alternate trenches



Tensioning Trenches Installation Sequence (Carpi patent)

- Stage 6: Installation of geocomposite over ballasted trenches

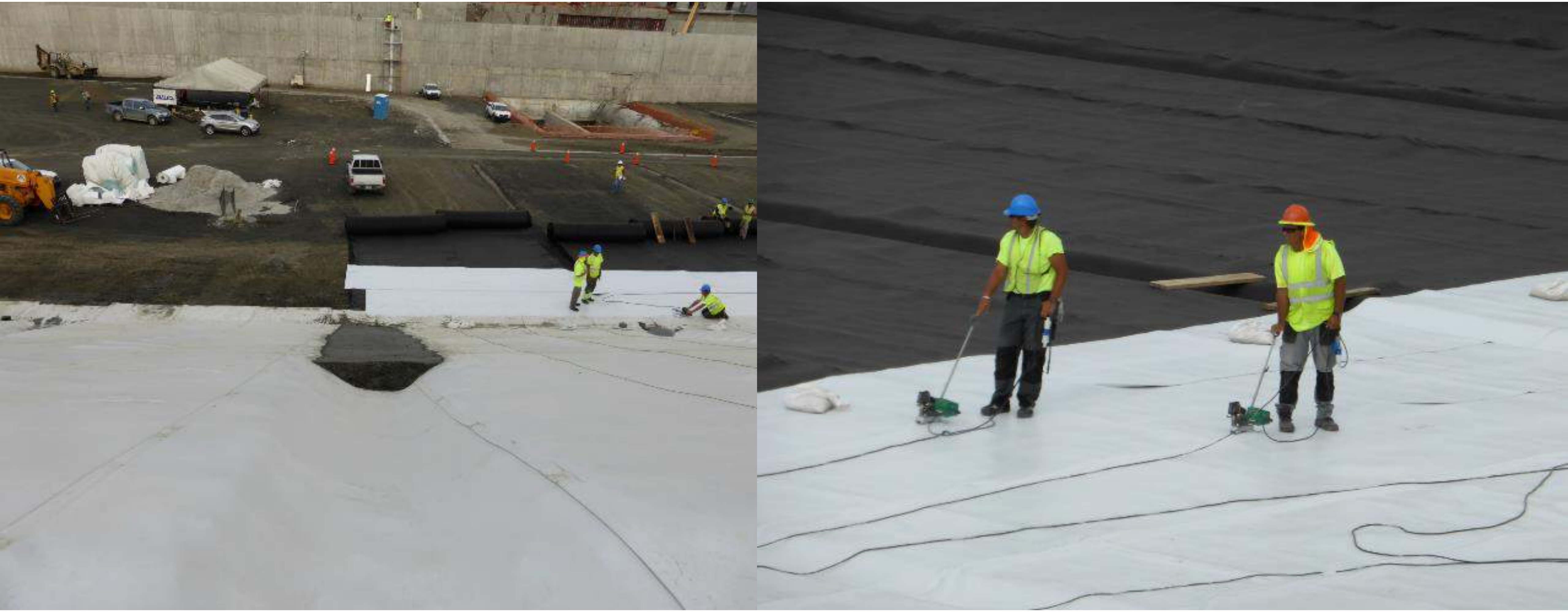


Tensioning Trenches Installation Sequence

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

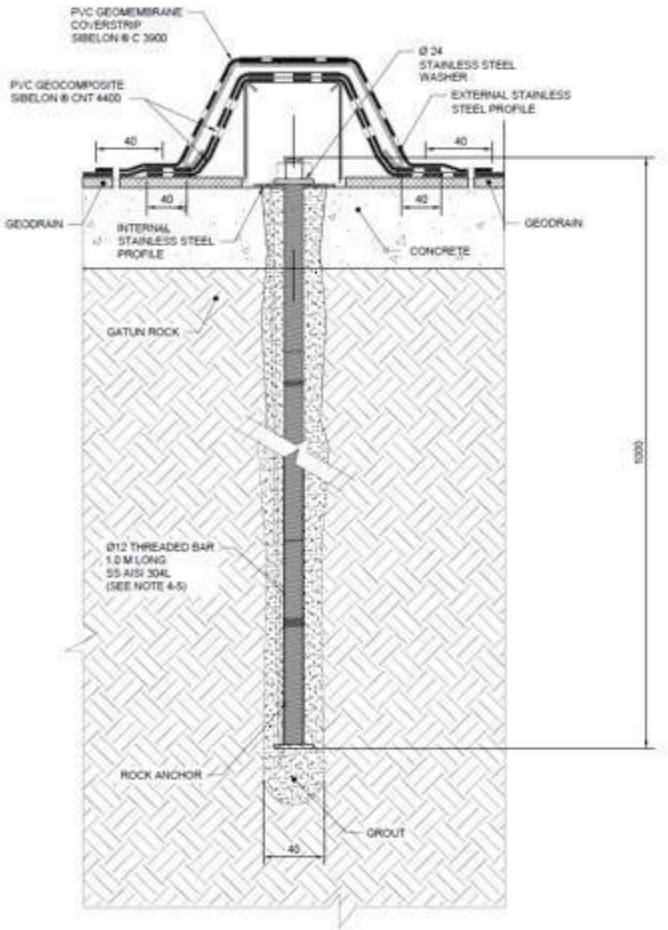
- Punctual Rock anchors for vertical anchoring profiles
- Punctual Soil Nailing Anchors on Slopes
- Mechanical Perimeter Seal around concrete Structures
- Anchor trenches in the rock fill embankments

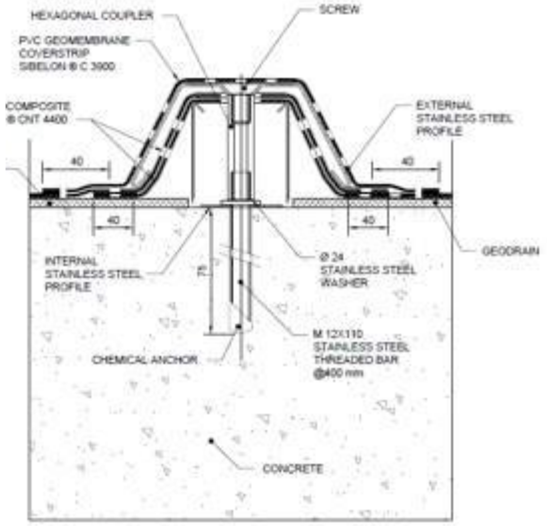
Carpi Anchoring Solutions

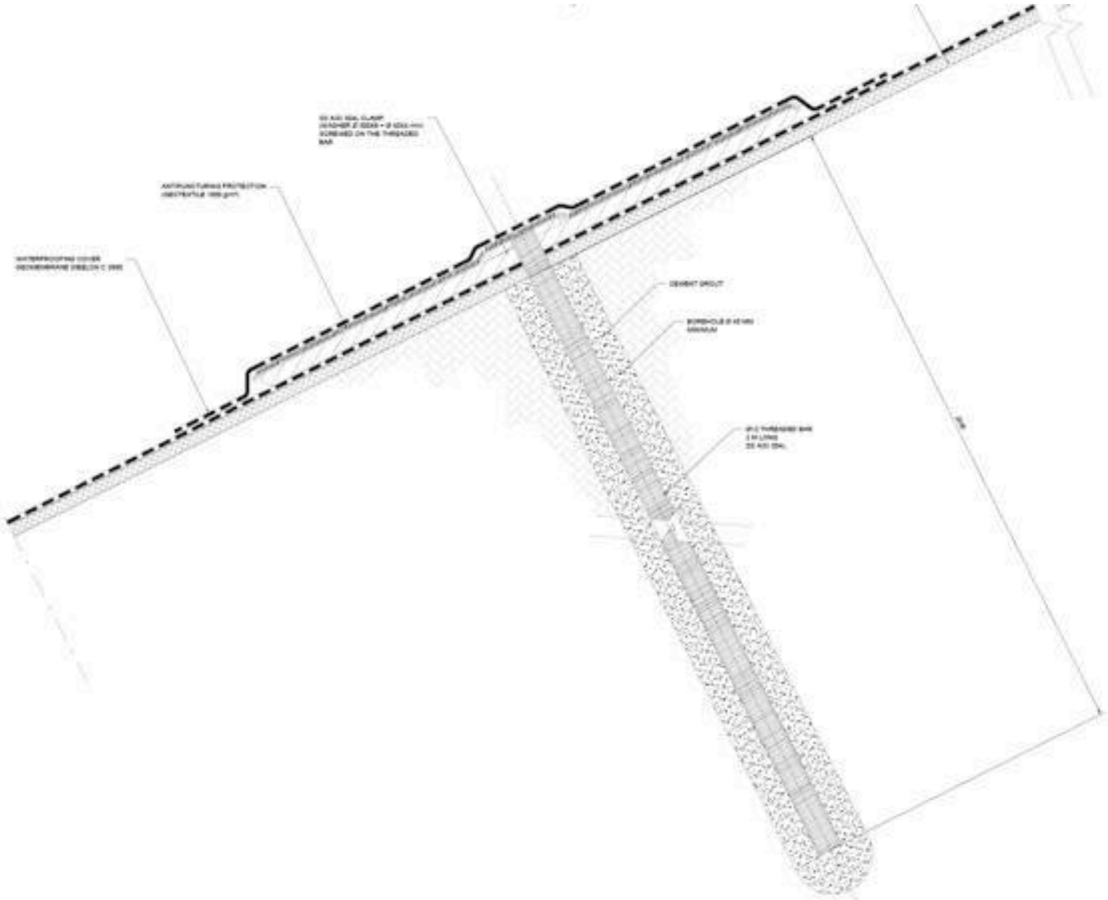
- **Mechanical Perimeter Seal around concrete Structures, Joint treatment at the Dividing walls.**

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

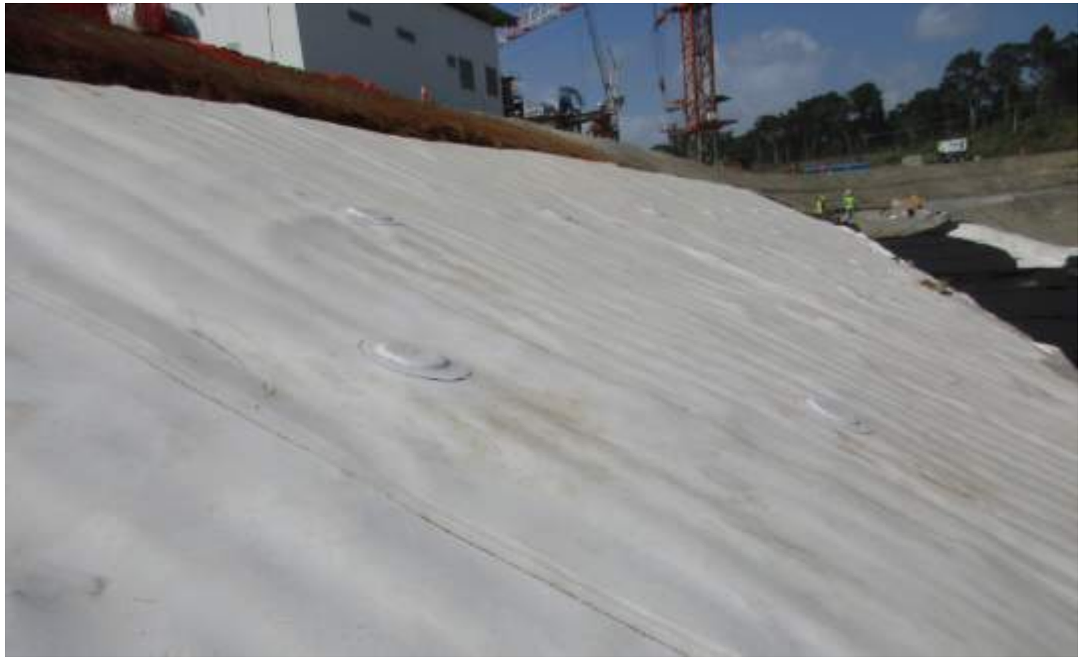
- **Punctual Soil Nailing Anchors on Slopes.**

Carpi Anchoring Solutions

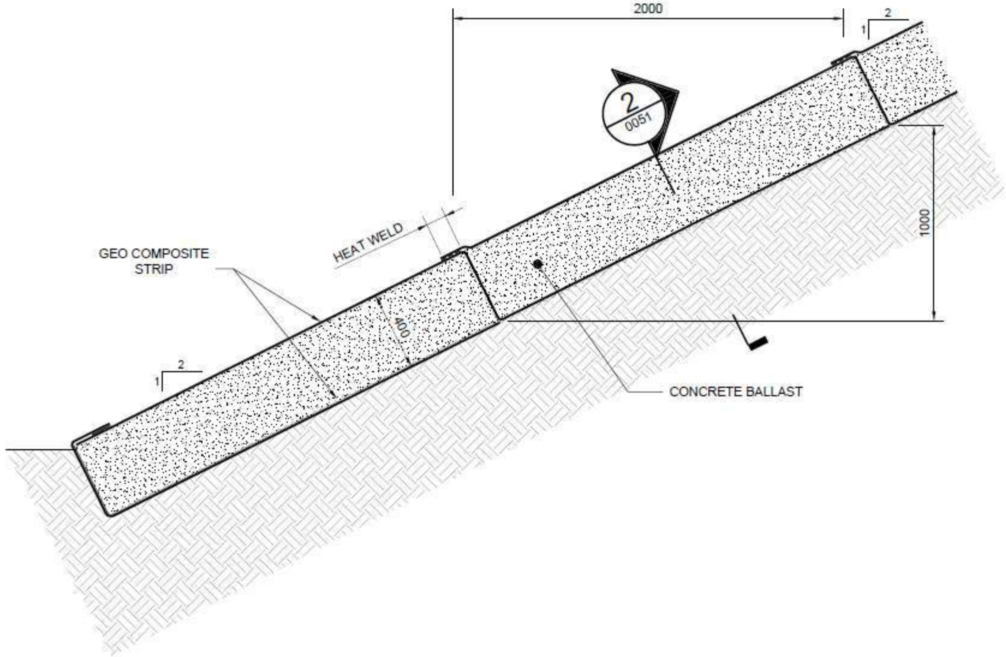
- **Punctual Soil Nailing Anchors on Slopes.**

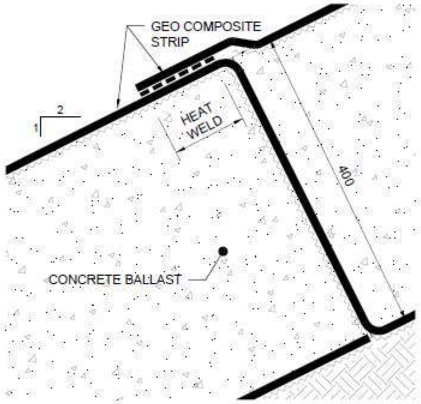
Carpi Anchoring Solutions

- **Punctual Soil Nailing Anchors on Slopes.**



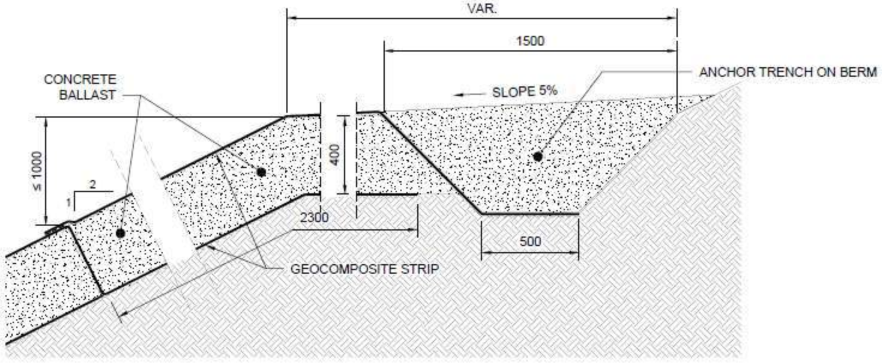
Anchor trenches at the rock fill embankments (Carpi patent)





DETAIL

SCALE: 1 = 5

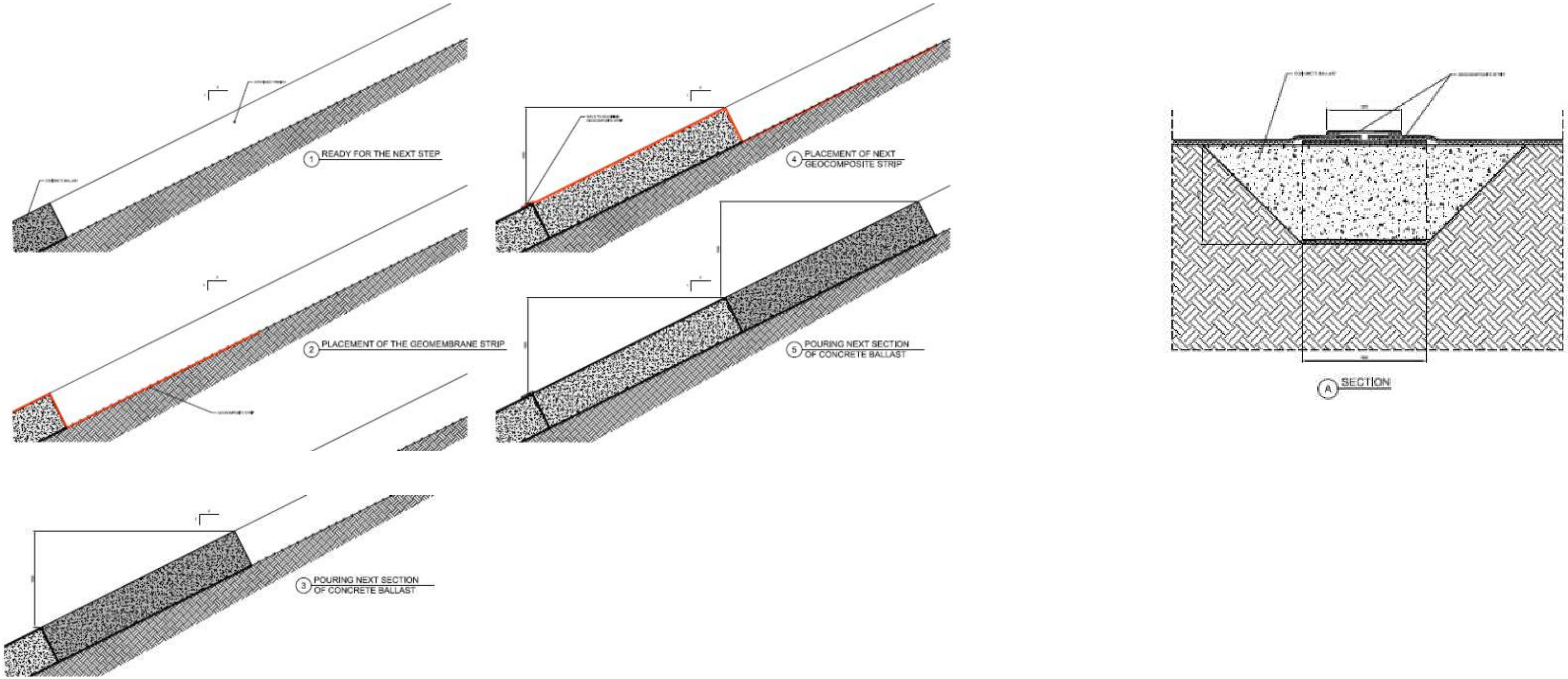


LONGITUDINAL SECTION AT TOP OF LINING

SEE DETAIL 2.

SCALE: 1 = 20

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 m^2

The use of geosynthetics in water conveyance structures

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

Thanks for your attention

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3rd ICTG 2016

4-7 September 2016, Guimarães, Portugal

Universidade do Minho
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Workshop 1 – Geosynthetics in Transportation Geotechnics

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

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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

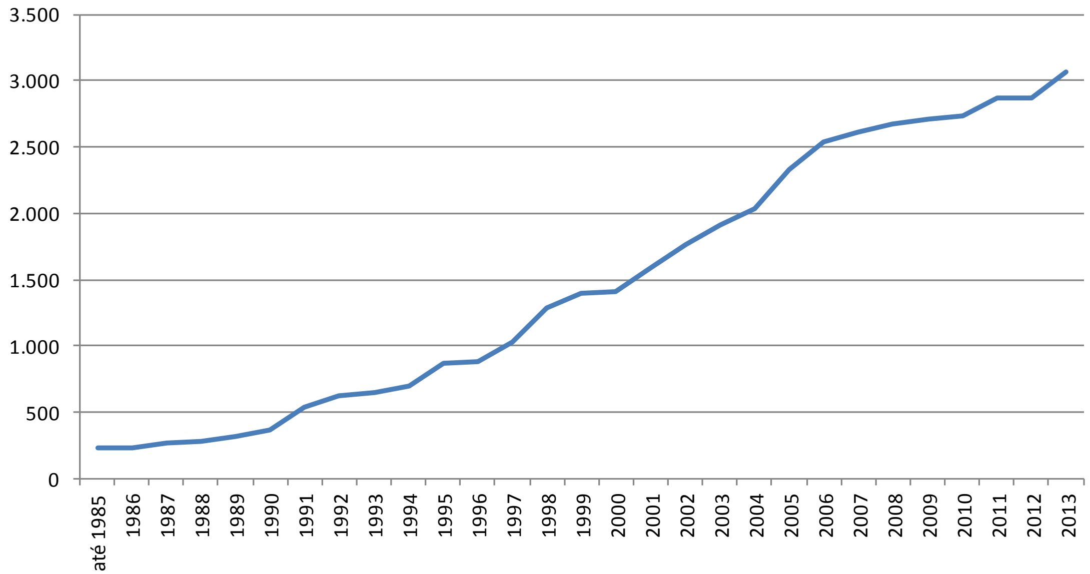
1. Introduction
2. Road pavements
3. Rail tracks
4. Conclusions

Source: Google images, 2016

1. Introduction 1/4

Road network in Portugal: motorways

1990	2012
316 km	2,988 km (Total length of road network - 14,284 km)

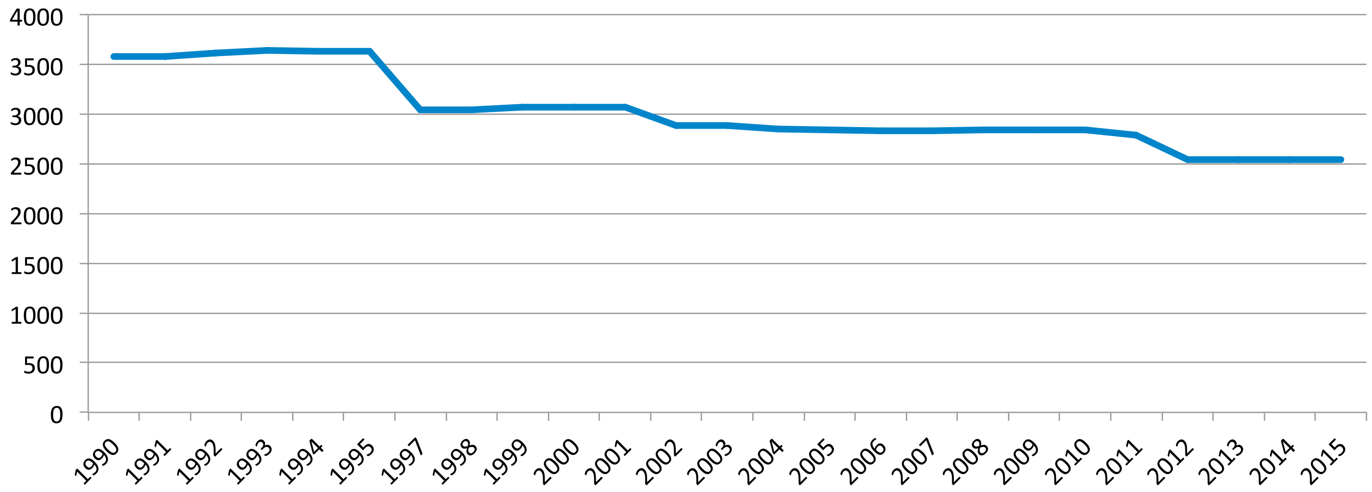


Source: Portuguese Network Directory, 2016

1. Introduction 2/4

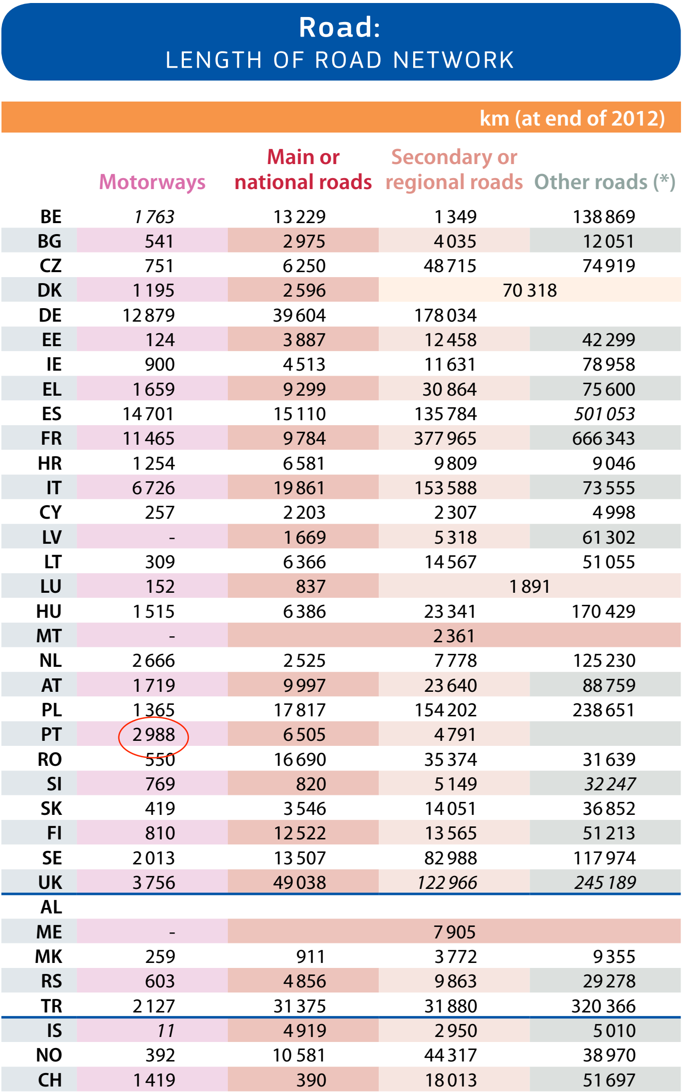
Rail network in Portugal: railway lines in operation

1990	2015
3,582 km	**2,546 km** **1,935 km** (single track) **611 km** (multiple track)



Source: Portuguese Network Directory, 2016

1. Introduction 3/4

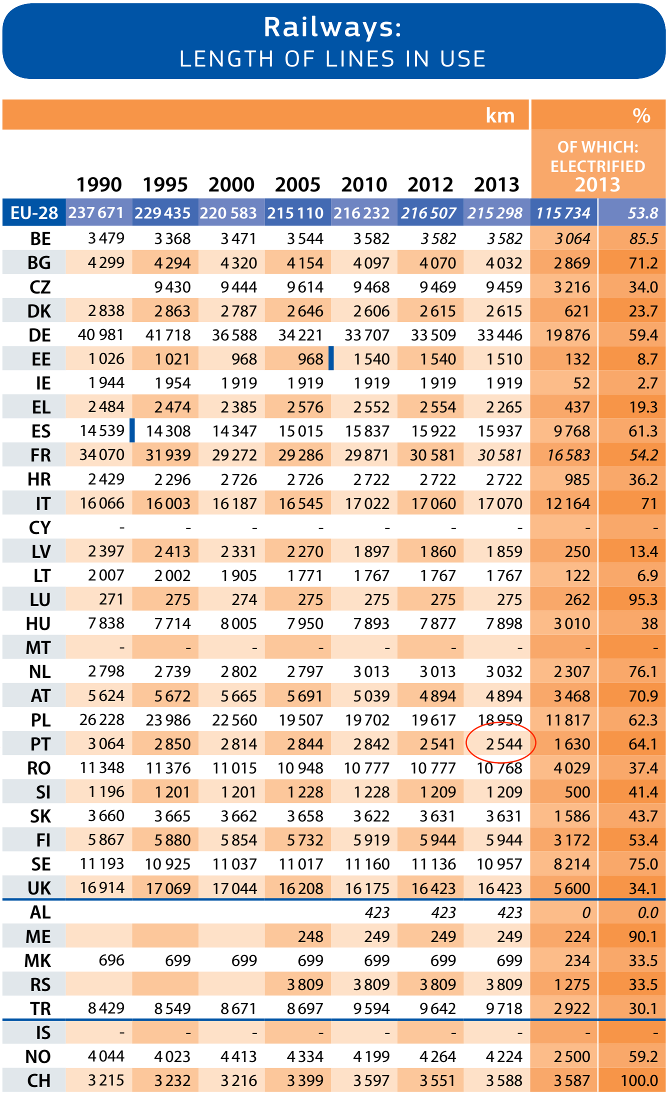


Road:
LENGTH OF ROAD NETWORK

km (at end of 2012)

	Motorways	Main or national roads	Secondary or regional roads	Other roads (*)
BE	*1 763*	13 229	1 349	138 869
BG	541	2 975	4 035	12 051
CZ	751	6 250	48 715	74 919
DK	1 195	2 596	70 318	
DE	12 879	39 604	178 034	
EE	124	3 887	12 458	42 299
IE	900	4 513	11 631	78 958
EL	1 659	9 299	30 864	75 600
ES	14 701	15 110	135 784	*501 053*
FR	11 465	9 784	377 965	666 343
HR	1 254	6 581	9 809	9 046
IT	6 726	19 861	153 588	73 555
CY	257	2 203	2 307	4 998
LV	-	1 669	5 318	61 302
LT	309	6 366	14 567	51 055
LU	152	837	1 891	
HU	1 515	6 386	23 341	170 429
MT	-	2 361		
NL	2 666	2 525	7 778	125 230
AT	1 719	9 997	23 640	88 759
PL	1 365	17 817	154 202	238 651
PT	2 988	6 505	4 791	
RO	550	16 690	35 374	31 639
SI	769	820	5 149	*32 247*
SK	419	3 546	14 051	36 852
FI	810	12 522	13 565	51 213
SE	2 013	13 507	82 988	117 974
UK	3 756	49 038	*122 966*	*245 189*
AL				
ME	-	7 905		
MK	259	911	3 772	9 355
RS	603	4 856	9 863	29 278
TR	2 127	31 375	31 880	320 366
IS	*11*	4 919	2 950	5 010
NO	392	10 581	44 317	38 970
CH	1 419	390	18 013	51 697

6th



Railways:
LENGTH OF LINES IN USE

	km								%
								OF WHICH: ELECTRIFIED 2013	
	1990	1995	2000	2005	2010	2012	2013		
EU-28	237 671	229 435	220 583	215 110	216 232	*216 507*	*215 298*	*115 734*	*53.8*
BE	3 479	3 368	3 471	3 544	3 582	*3 582*	*3 582*	*3 064*	*85.5*
BG	4 299	4 294	4 320	4 154	4 097	4 070	4 032	2 869	71.2
CZ		9 430	9 444	9 614	9 468	9 469	9 459	3 216	34.0
DK	2 838	2 863	2 787	2 646	2 606	2 615	2 615	621	23.7
DE	40 981	41 718	36 588	34 221	33 707	33 509	33 446	19 876	59.4
EE	1 026	1 021	968	968	1 540	1 540	1 510	132	8.7
IE	1 944	1 954	1 919	1 919	1 919	1 919	1 919	52	2.7
EL	2 484	2 474	2 385	2 576	2 552	2 554	2 265	437	19.3
ES	14 539	14 308	14 347	15 015	15 837	15 922	15 937	9 768	61.3
FR	34 070	31 939	29 272	29 286	29 871	30 581	*30 581*	*16 583*	*54.2*
HR	2 429	2 296	2 726	2 726	2 722	2 722	2 722	985	36.2
IT	16 066	16 003	16 187	16 545	17 022	17 060	17 070	12 164	71
CY	-	-	-	-	-	-	-	-	-
LV	2 397	2 413	2 331	2 270	1 897	1 860	1 859	250	13.4
LT	2 007	2 002	1 905	1 771	1 767	1 767	1 767	122	6.9
LU	271	275	274	275	275	275	275	262	95.3
HU	7 838	7 714	8 005	7 950	7 893	7 877	7 898	3 010	38
MT	-	-	-	-	-	-	-	-	-
NL	2 798	2 739	2 802	2 797	3 013	3 013	3 032	2 307	76.1
AT	5 624	5 672	5 665	5 691	5 039	4 894	4 894	3 468	70.9
PL	26 228	23 986	22 560	19 507	19 702	19 617	18 959	11 817	62.3
PT	3 064	2 850	2 814	2 844	2 842	2 541	2 544	1 630	64.1
RO	11 348	11 376	11 015	10 948	10 777	10 777	10 768	4 029	37.4
SI	1 196	1 201	1 201	1 228	1 228	1 209	1 209	500	41.4
SK	3 660	3 665	3 662	3 658	3 622	3 631	3 631	1 586	43.7
FI	5 867	5 880	5 854	5 732	5 919	5 944	5 944	3 172	53.4
SE	11 193	10 925	11 037	11 017	11 160	11 136	10 957	8 214	75.0
UK	16 914	17 069	17 044	16 208	16 175	16 423	16 423	5 600	34.1
AL					*423*	*423*	*423*	*0*	*0.0*
ME				248	249	249	249	224	90.1
MK	696	699	699	699	699	699	699	234	33.5
RS				3 809	3 809	3 809	3 809	1 275	33.5
TR	8 429	8 549	8 671	8 697	9 594	9 642	9 718	2 922	30.1
IS	-	-	-	-	-	-	-	-	-
NO	4 044	4 023	4 413	4 334	4 199	4 264	4 224	2 500	59.2
CH	3 215	3 232	3 216	3 399	3 597	3 551	3 588	3 587	100.0

19th

Source: EU Transport in figures, Statistical Pocketbook, 2015

1. Introduction 4/4

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

GEO reinforce

Reforço de geomateriais e suas implicações no projeto de pavimentos e vias férreas

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

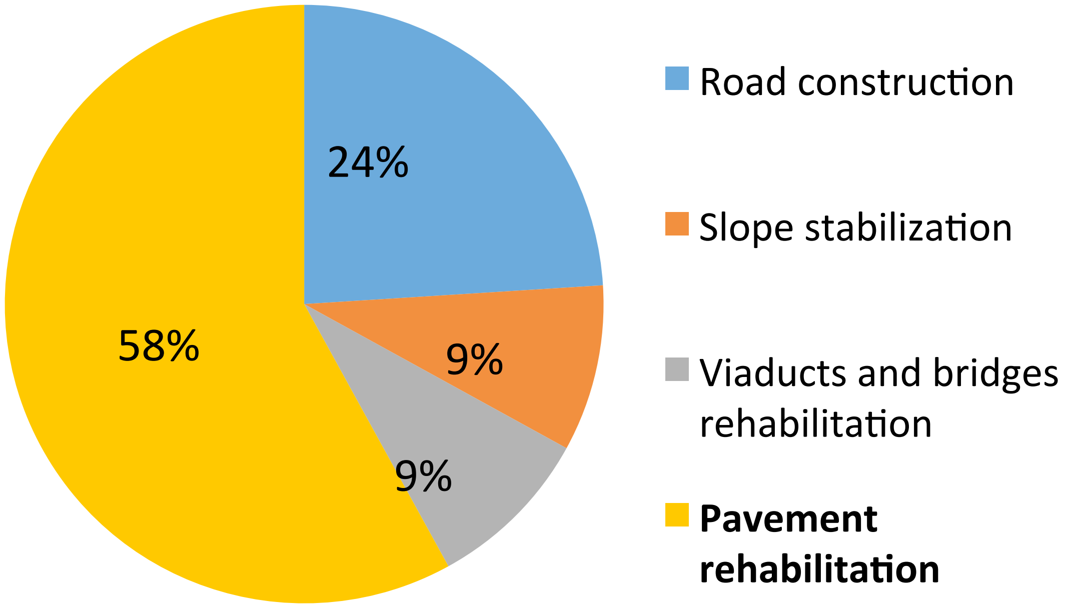
PLATAFORMA DO GRUPO DE TRABALHO PORTUGUÊS

PLATFORM OF PORTUGUESE WORKING GROUP

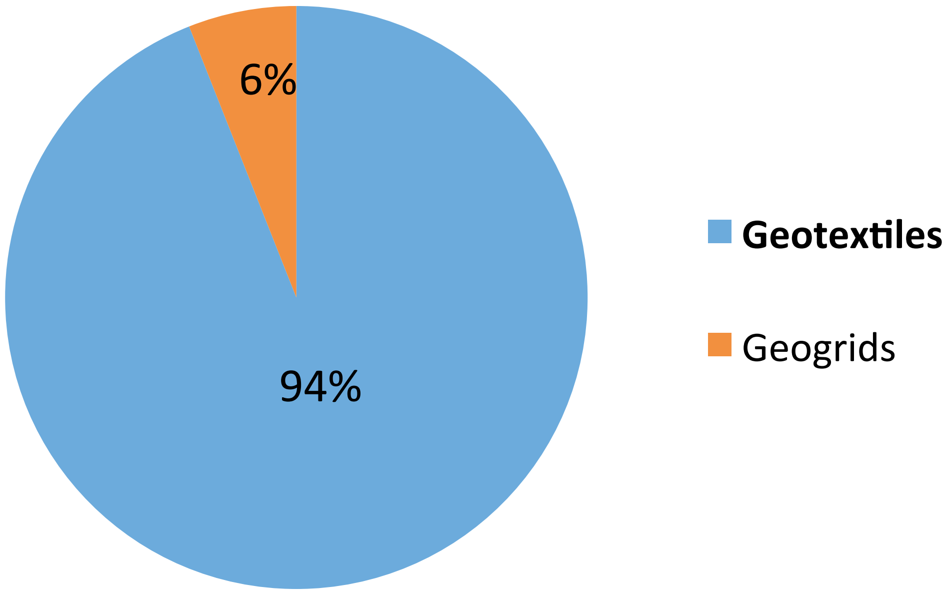
2. Road pavements 1/2

Subgrade

Distribution of the use of geosynthetics (2001-2012): 500,000 m^2; 45 road works



Type of use

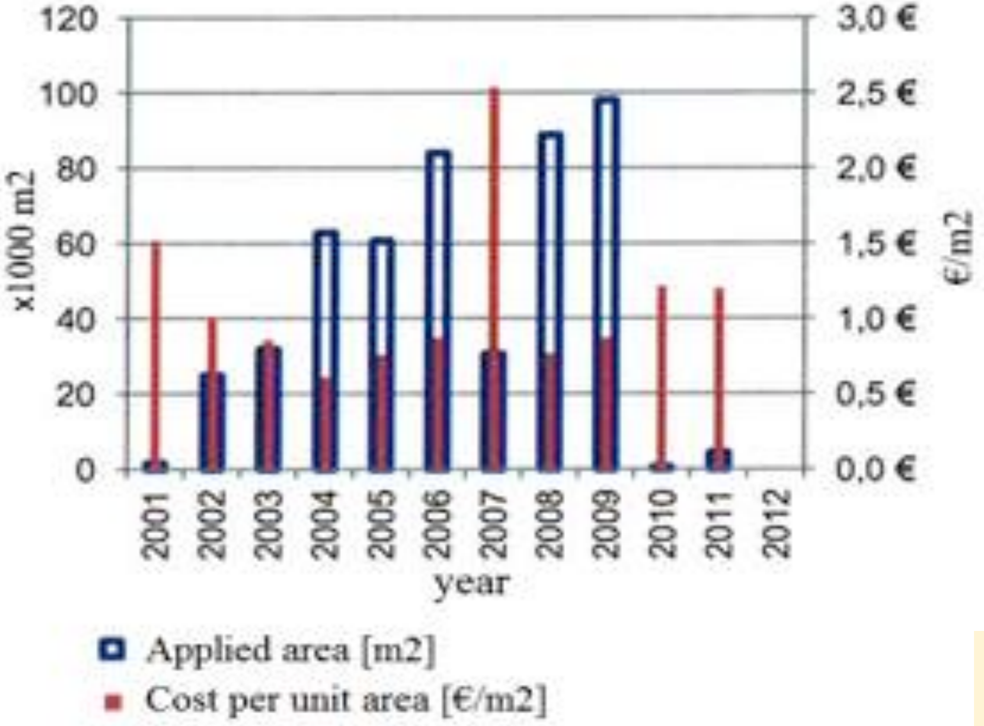


Type of geosynthetics

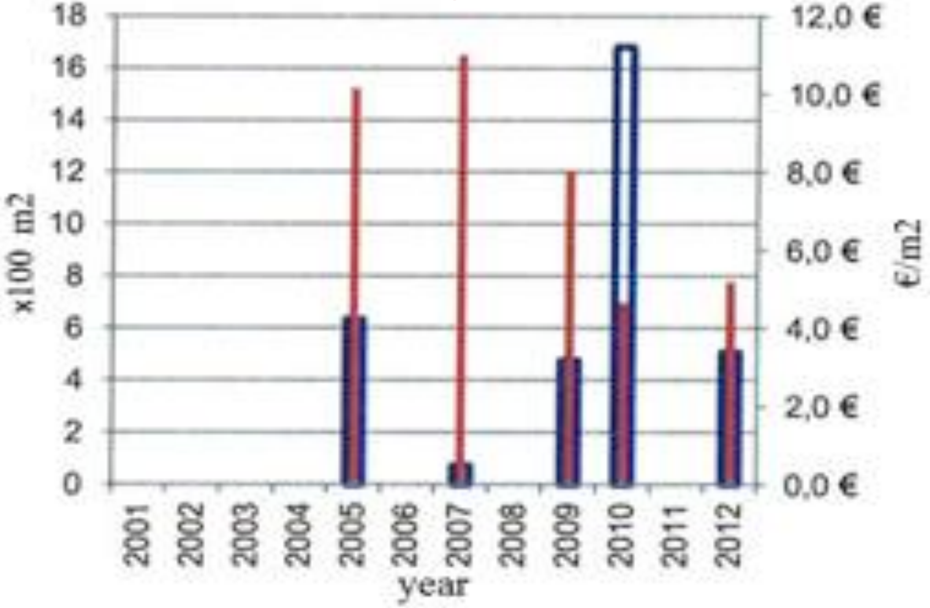
2. Road pavements 2/2

Subgrade - Quantities and costs

Geotextiles



Geogrids



45,000 m^2/year:
- 44,300 m^2 geosynthetics
- 700 m^2 geogrids

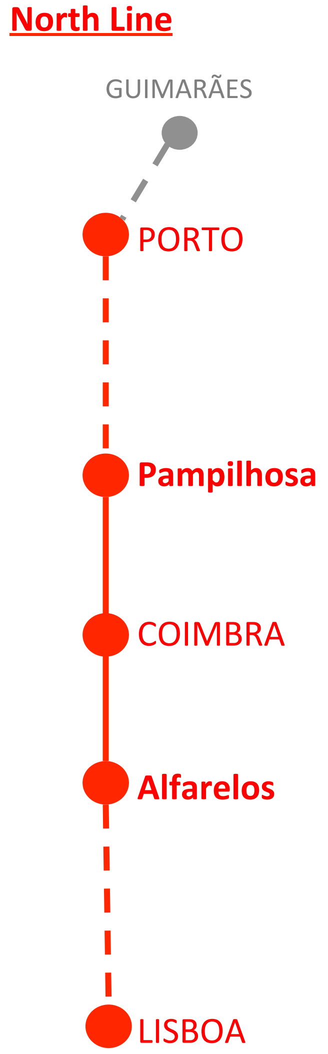
- 1.50 €/m^2 geosynthetics
- 7.80 €/m^2 geogrids

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 m^2
    - Composite of geogrid and nonwoven geotextile: 34,580 m^2



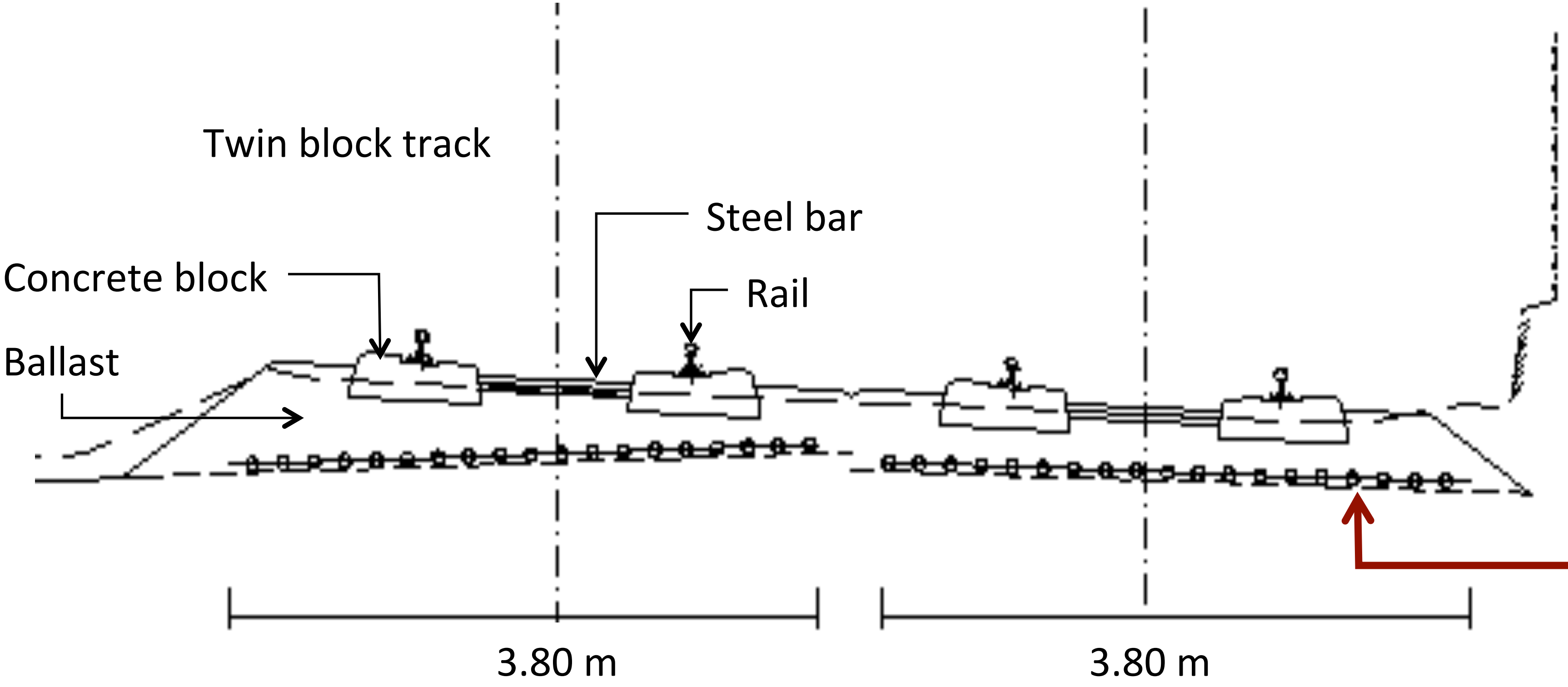
3. Rail tracks 2/3

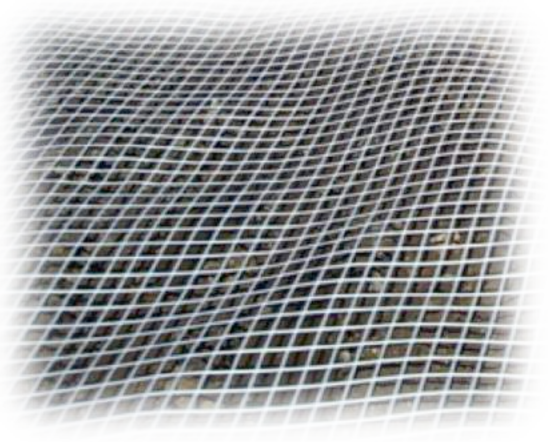
Passengers/Cargo
Loads: 22.5 ton/axle

Ballast: granite
Geogrid: biaxial

Cross–section of rehabilitated rail track

North Line Railway - Alfarelos/Pampilhosa - km 194,500 to km 218,000





Geogrid

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 !**