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Dynamic behaviour and a-seismic design of reinforced soil retaining wall

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The content of the presentation today is from a long-term research at the University of Tokyo and the Railway Technical Research Institute, Japan, that has been performed together with, in particular:

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- **Dr TATEYAMA Masaru, RTRI, and**
- **Dr UCHIMURA, Taro, Department of Civil Engineering, the University of Tokyo, and**
- **Dr SHINODA, Masahiro, Integrated Geotechnology Institute Limited**

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1. Recent advances in geosynthetic-reinforced retaining soil walls in Japan (1997-1998 Mercer Lecture, revised)

2. Comparison of dynamic stability between reinforced-soil and gravity type RWs

3. A new dynamic earth pressure theory accounting for strain softening and strain localization

4. Seismic stability of soil RWs on slope and remedy measures

5. Lessons for soil RWs from 2004 Niigata-ken Chuetsu Earthquake

6. New type bridge abutments: PL&PS and cement-mixed backfill

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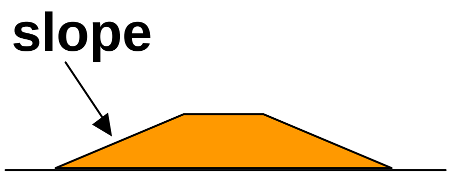
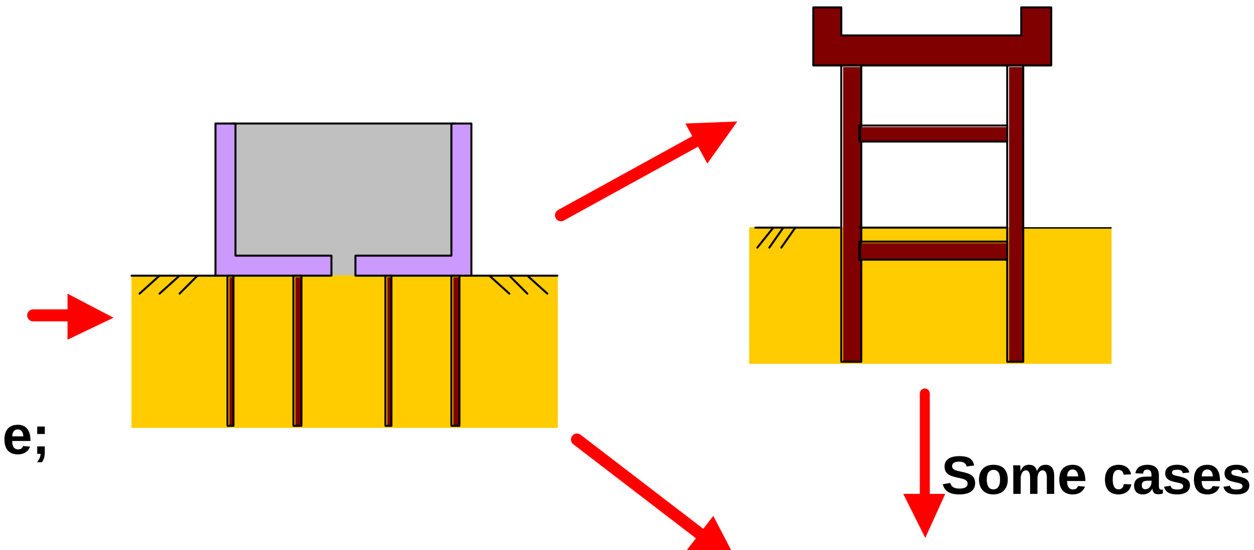
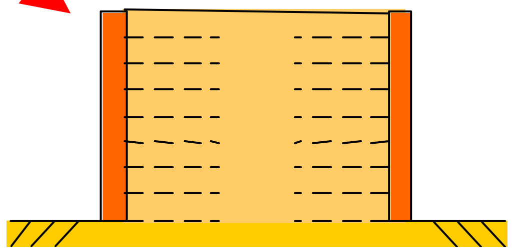
BACKGROUND

· History of elevated railway and highway structures in Japan



· could be unstable;
· could be too deformable;
and
· occupies too large space.





· Higher cost-efficiency

· Sufficiently stable and stiff

(no piles)

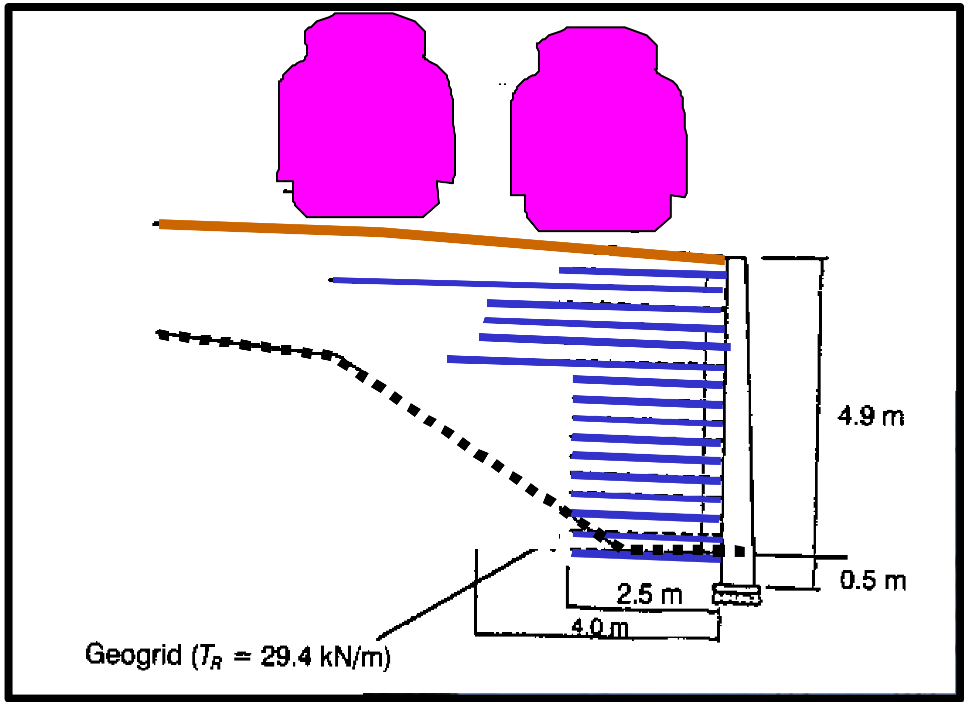
The first test embankment at the University of Tokyo,

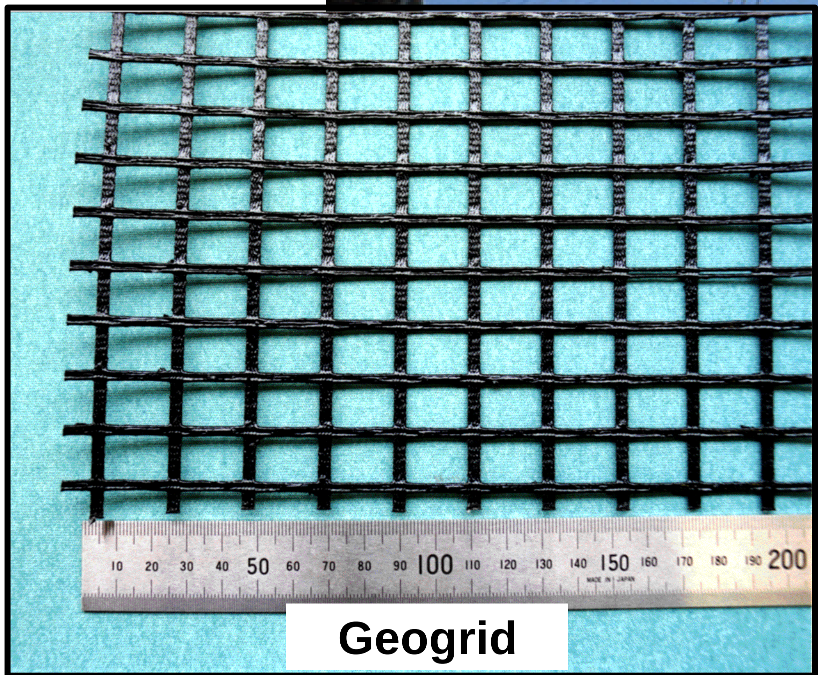
Clay backfill reinforced with a non-woven geotextile with wrapped-around wall face

A GRS-RW with a FHR facing located, closest to my house, supporting a rapid transit (Keio Line)

August 1996

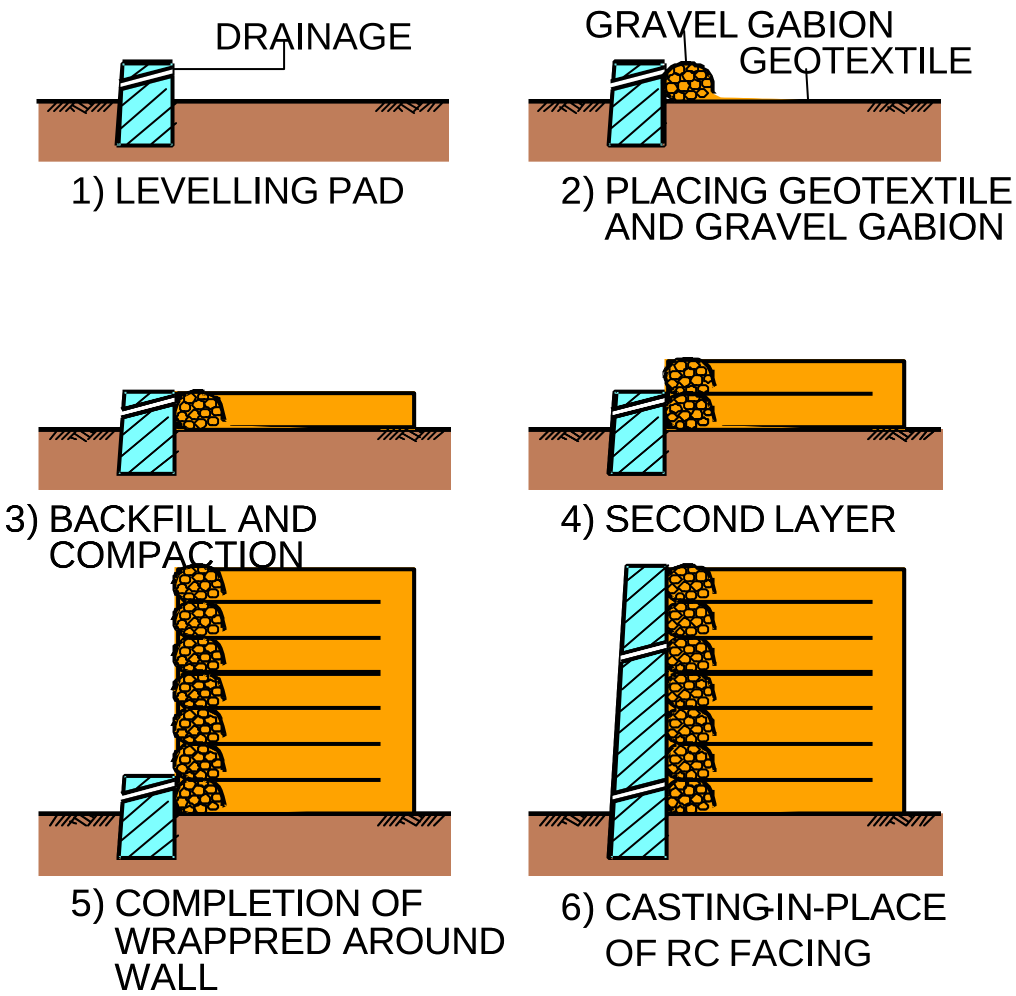
Rapid transit train running on a geogrid-reinforced soil RW





Staged construction - 1;

- **the wall is first constructed with a help of gabions filled with crushed gravel; and**

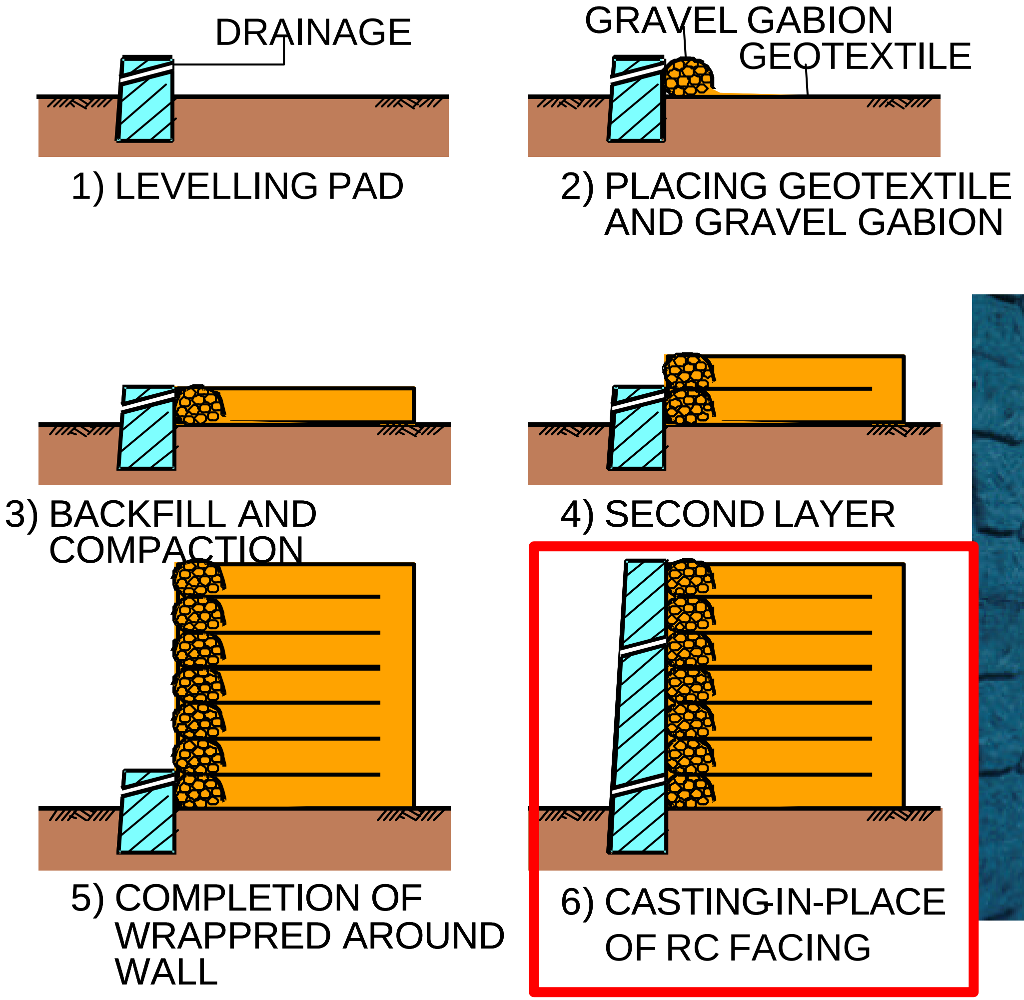


A 5 m-high wall before casting-in-place a FHR facing

5) COMPLETION OF WRAPPRED AROUND WALL

Staged construction - 2;

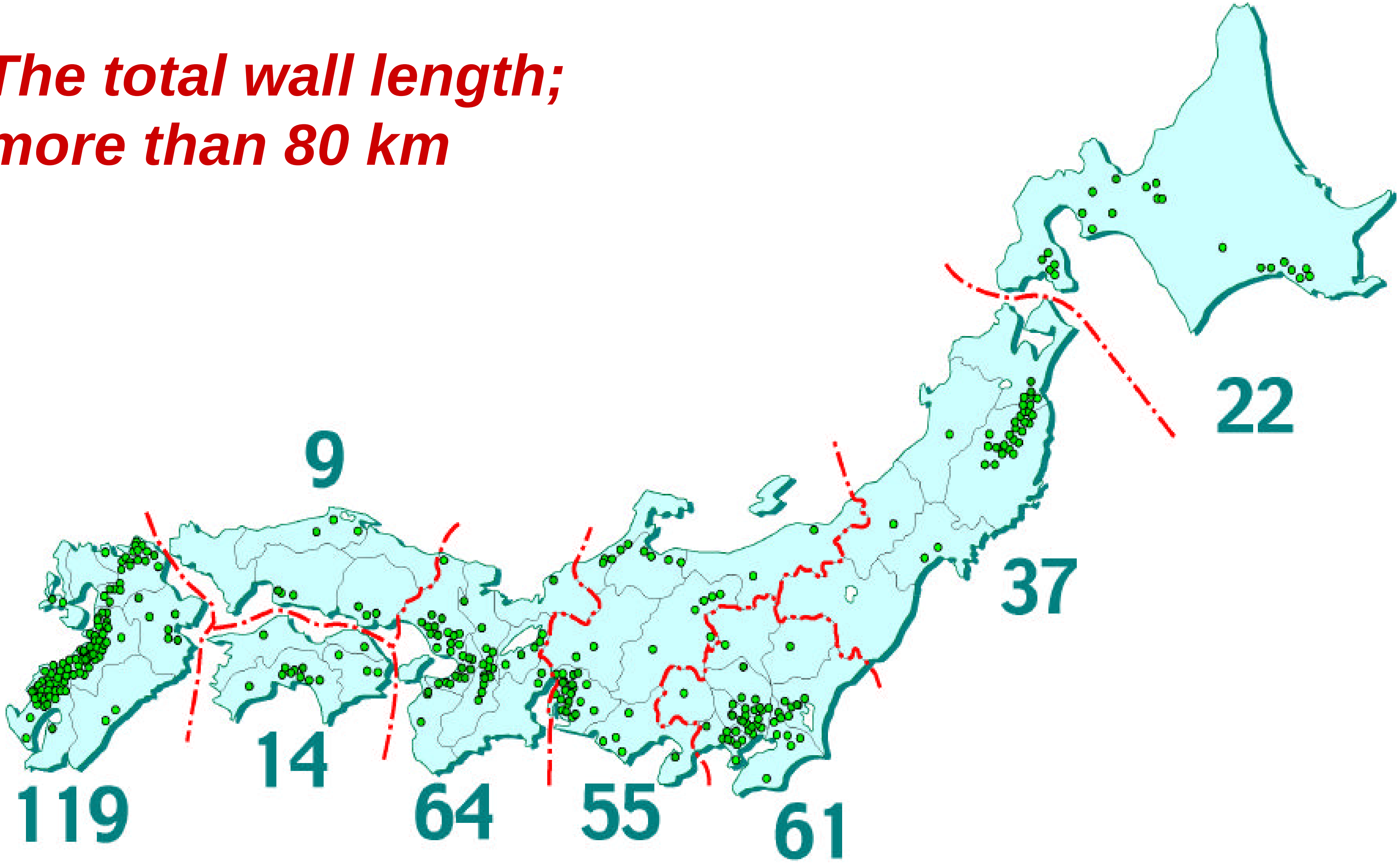
- Then, ***after the deformation of the backfill and supporting ground has taken place***, **a full-height rigid facing is cast-in-place directly on the wrapped- around wall.**



GRS-RWs having a full-height rigid facing constructed by the staged construction procedure

- now supporting railway and highway embankments with a total wall length more than 80 km; and
- one of the standard wall construction procedures for railways and highways in Japan, replacing the conventional wall construction procedures.

The total wall length; more than 80 km



Locations of major GRS-RWs with a full-height rigid facing constructed by the staged construction procedure (by the end of April 2005).

Topics

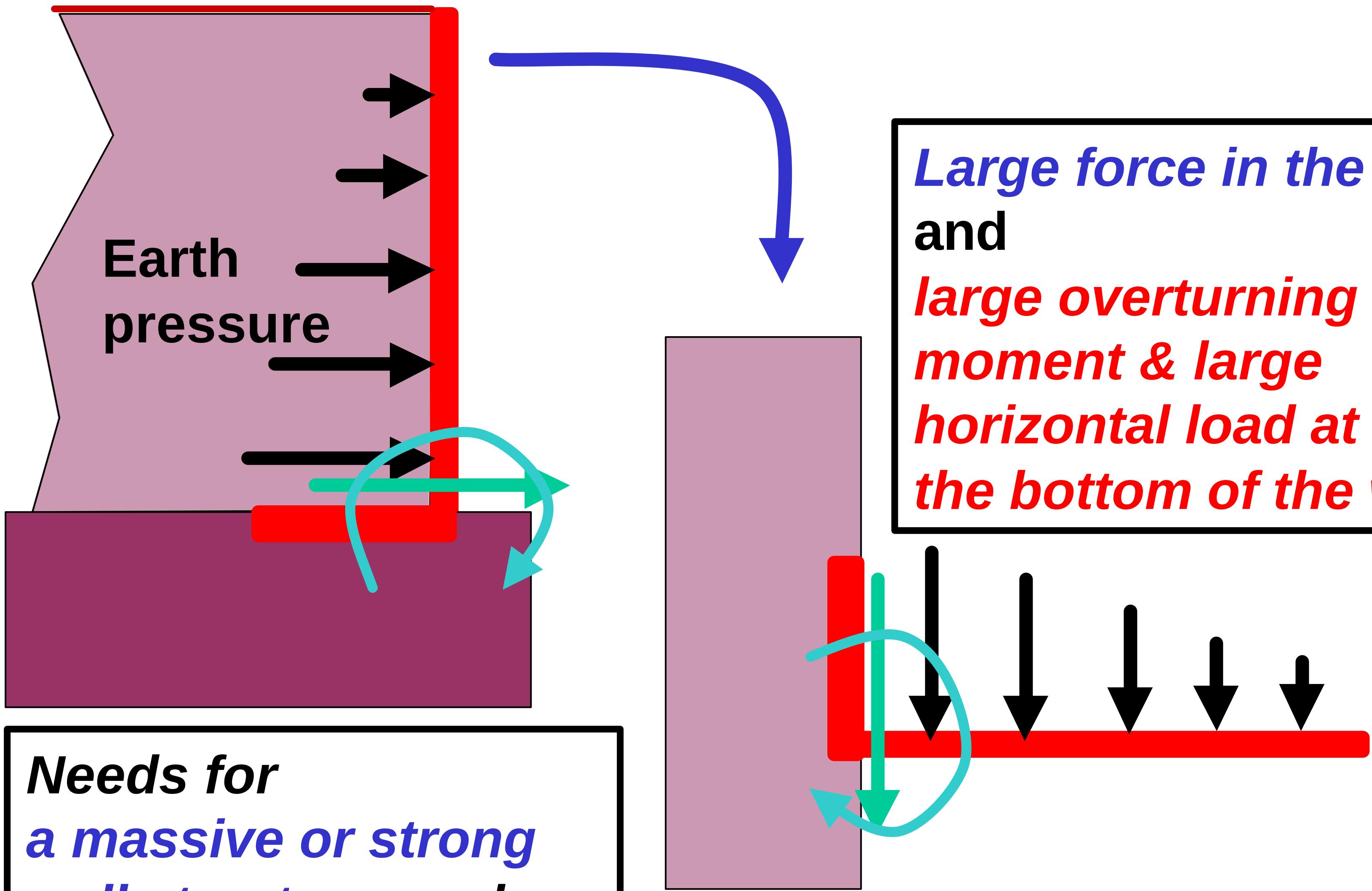
1-1 Re-consideration on the advantages of reinforced soil RWs

1-2 Advantages of using a full-height rigid facing

1-3 Advantages of the staged construction procedure

1-4 Some typical case histories

Conventional type RWs as a cantilever structure



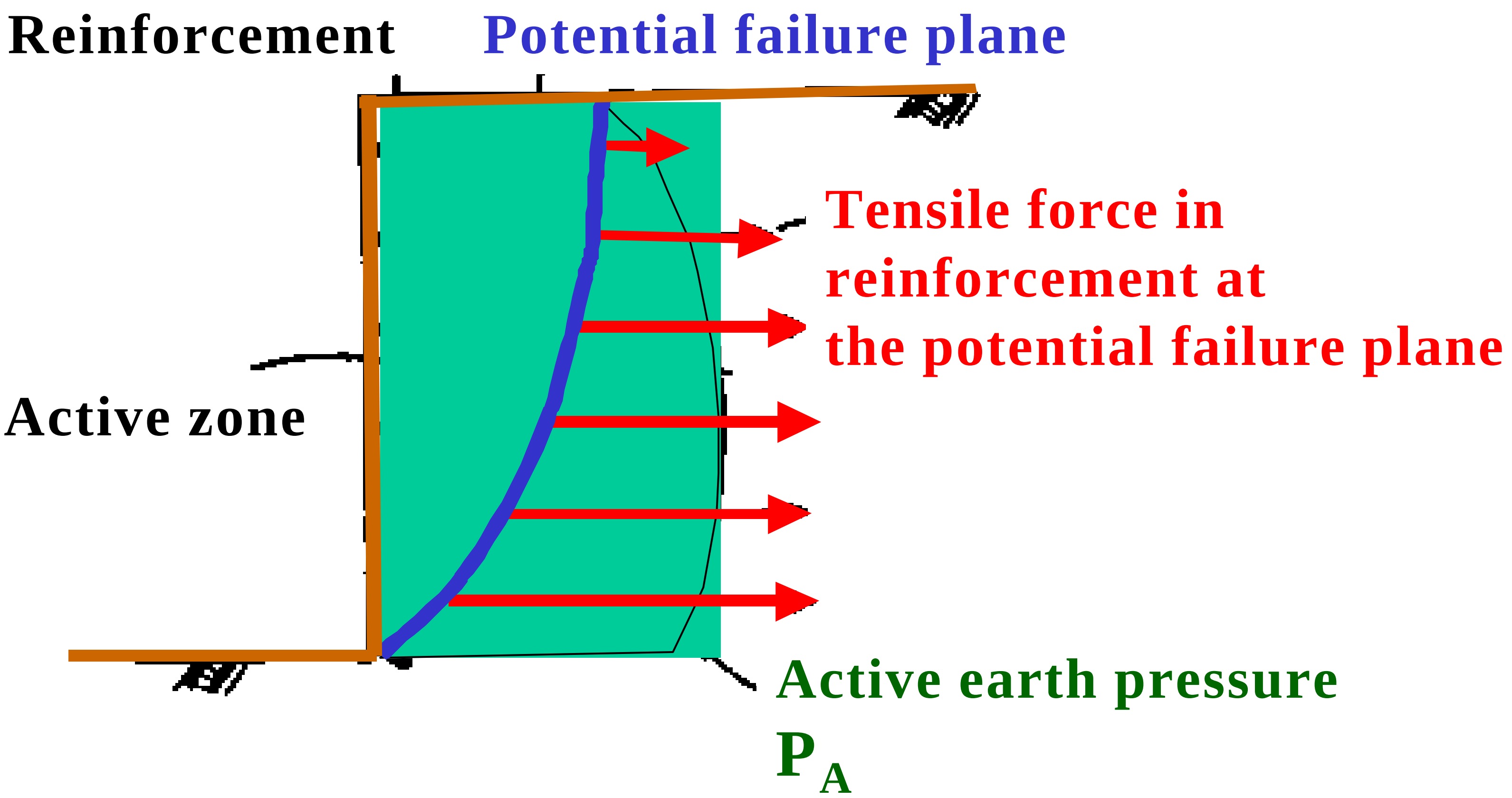
Large force in the wall; **and**
large overturning moment & large horizontal load at the bottom of the wall.

Needs for
a massive or strong wall structure; and
a pile foundation

Two types of force equilibrium with reinforced soil RWs

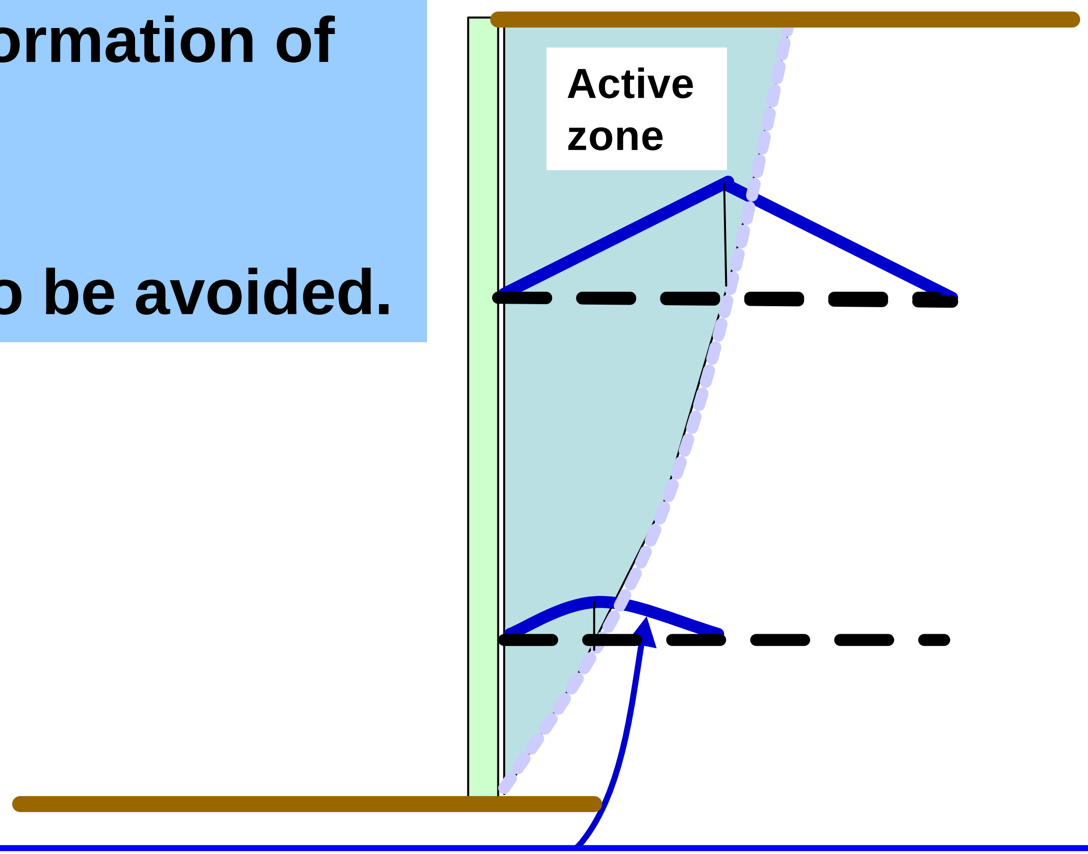
（a） Along the potential failure plane

(b) At the facing



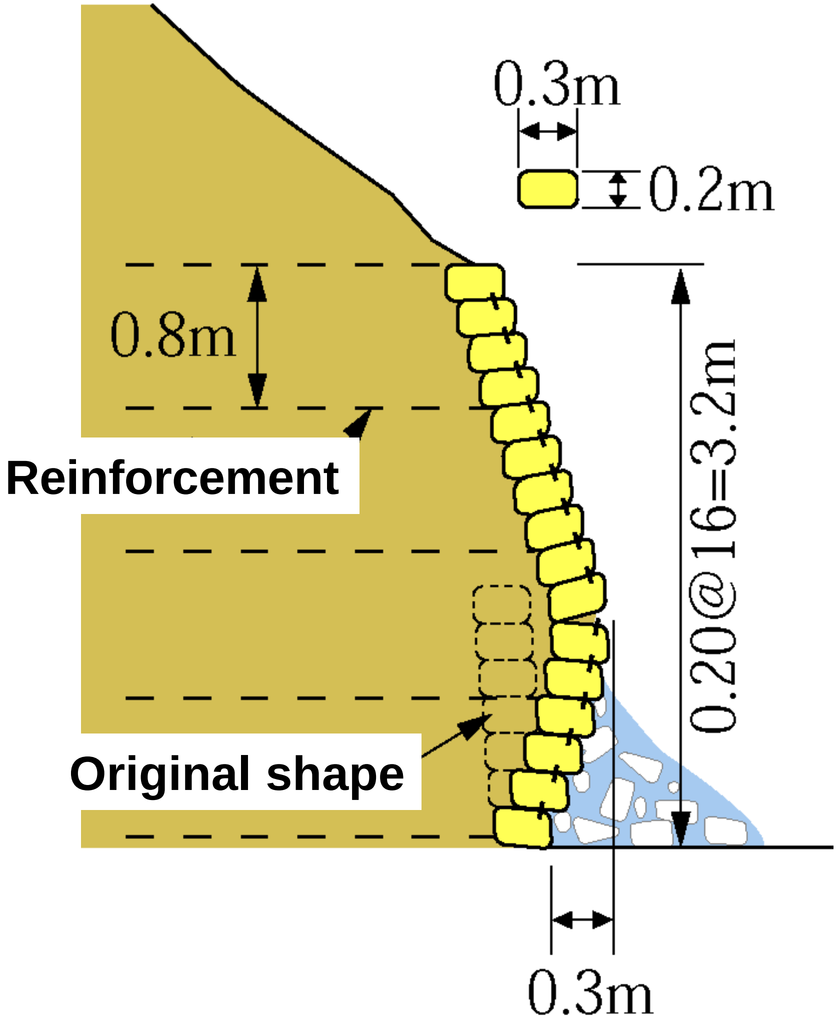
The confining pressure in the active zone is low, resulting in a low stiffness and large deformation of the active zone.

Unstable conditions to be avoided.



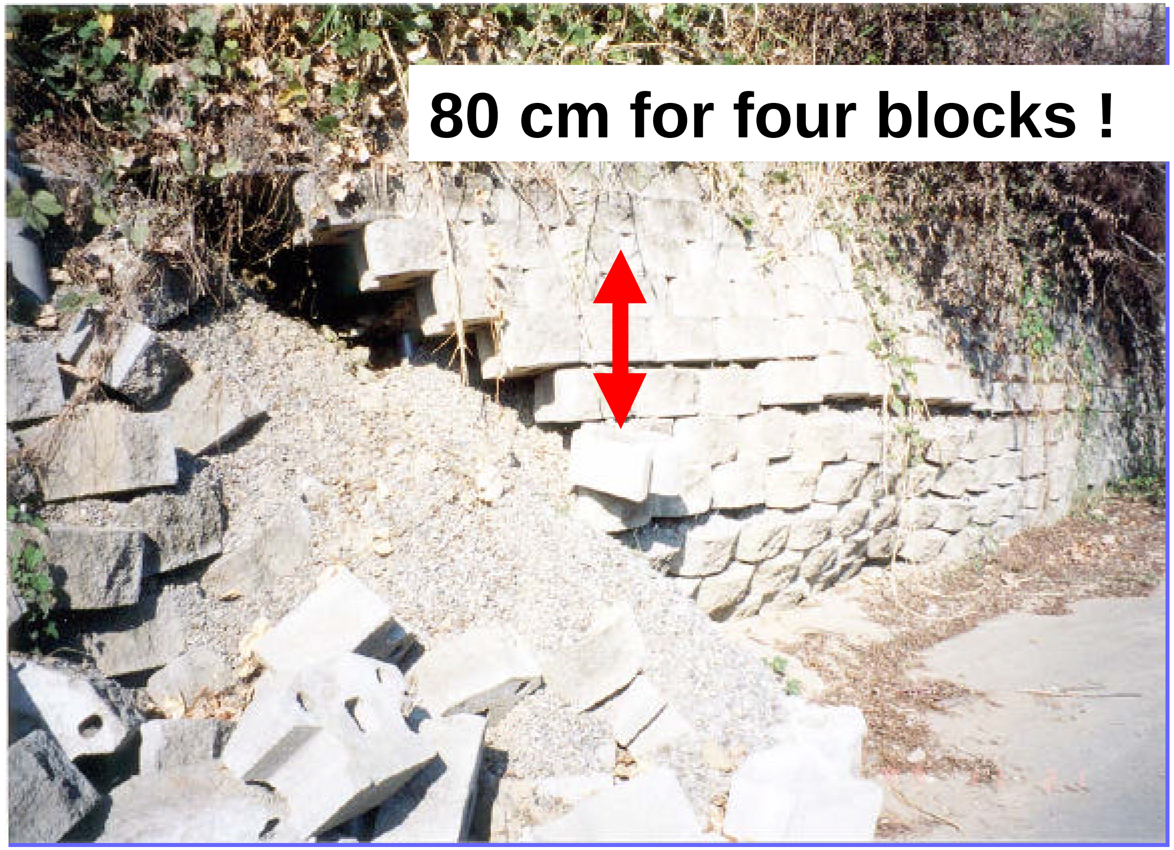
Distribution of tensile force when no facing is used or when the facing and reinforcement are not connected.

Failure of keystone walls during the 1999 ChiChi Earthquake, Taiwan, showing the importance of connection between geogrid and facing



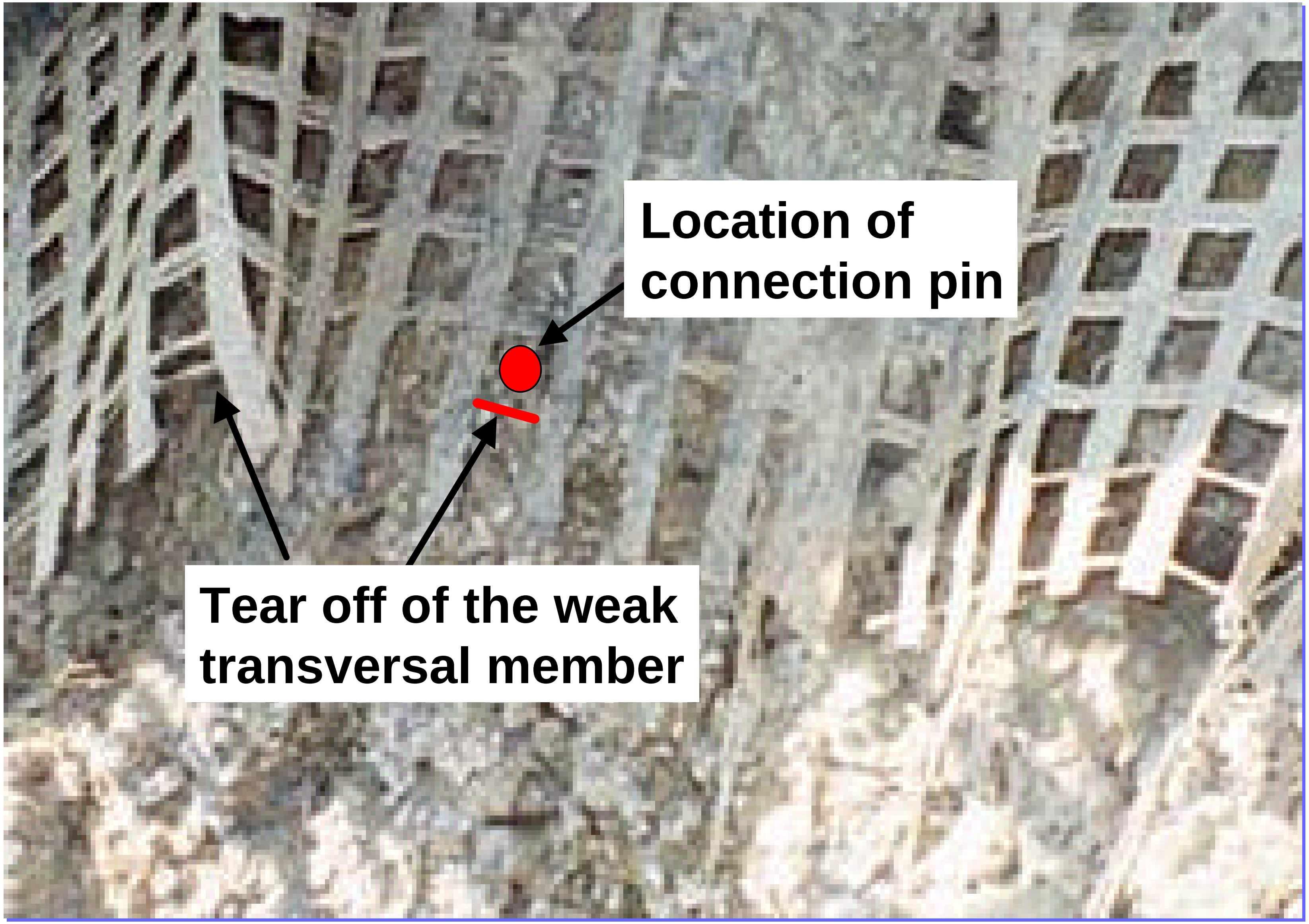
A too large vertical spacing between the reinforcement layers;

designed without paying attentions to the stability of block facing, assuming nearly no earth pressure acting at the back of the facing



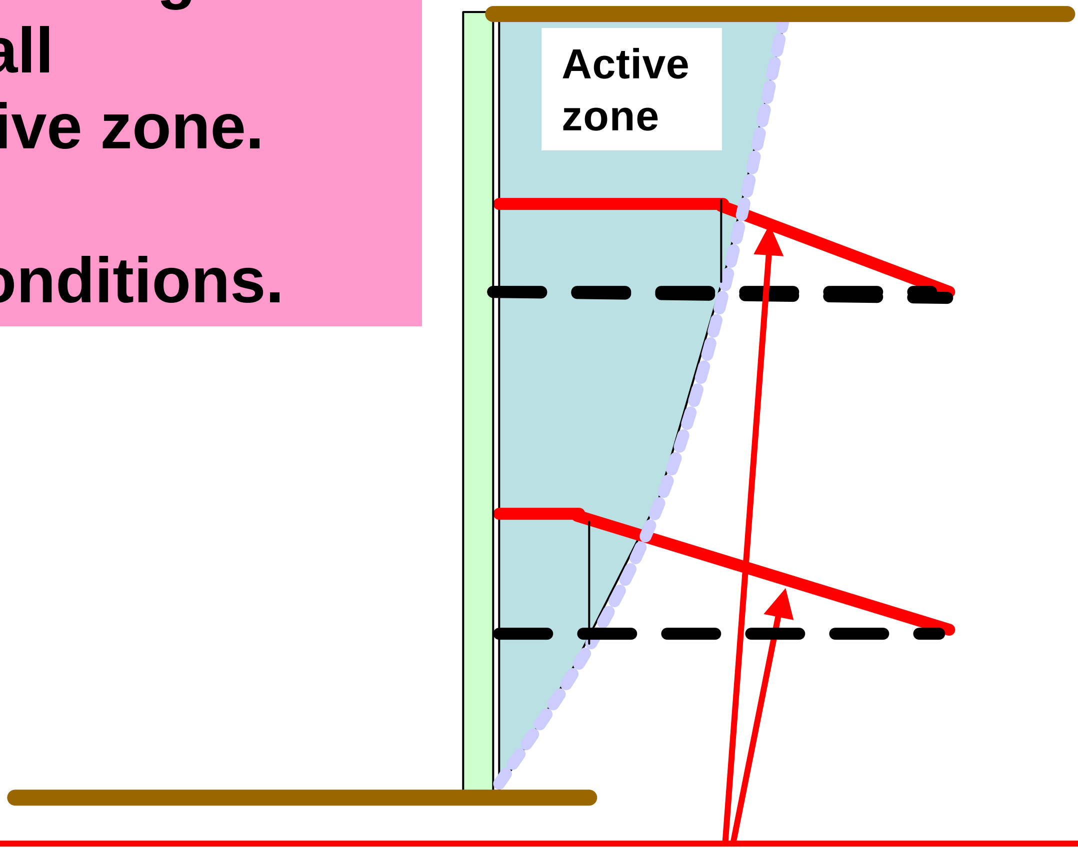
Too short connection pins;

Too low connection strength,



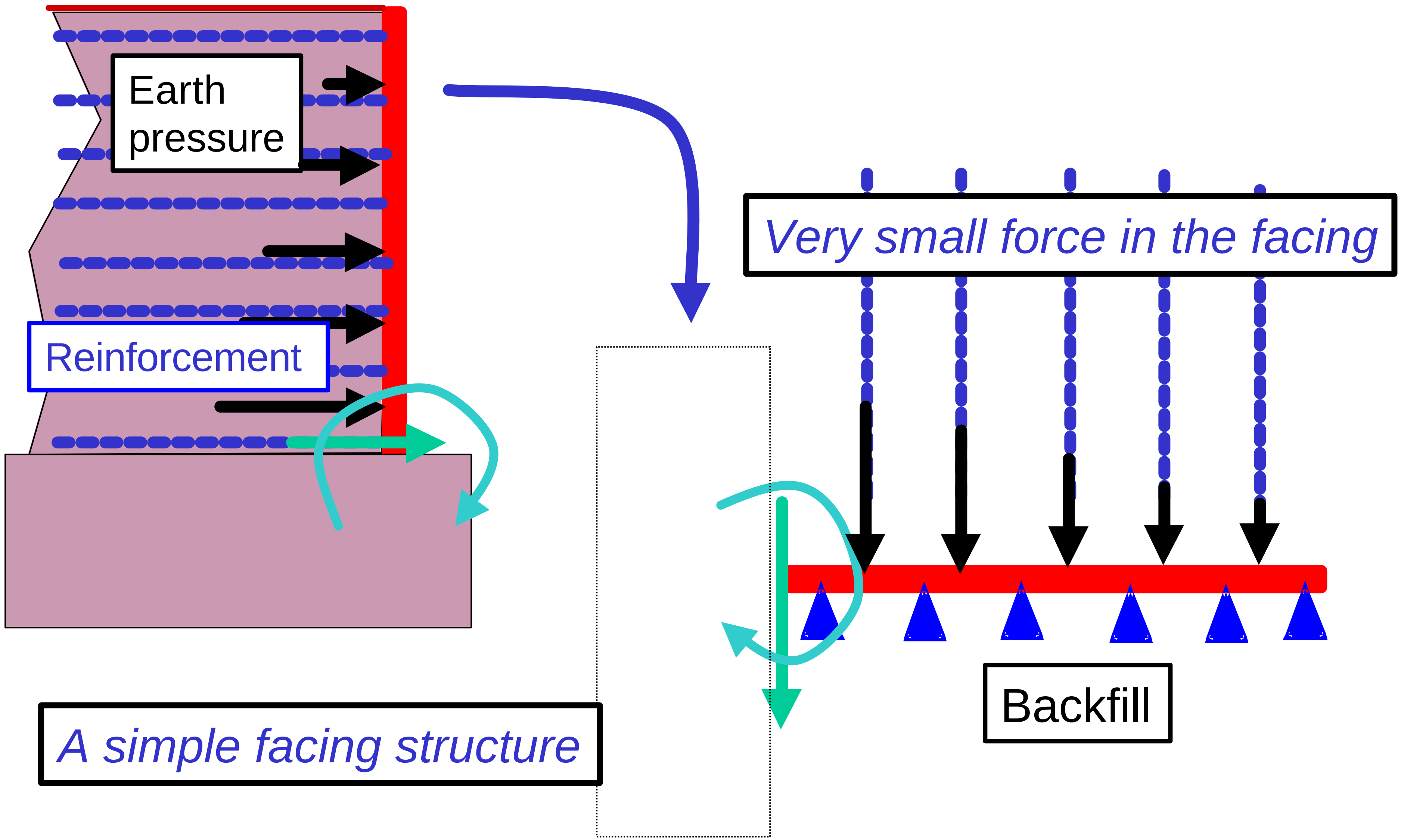
The confining pressure in the active zone is high, resulting in a high stiffness and small deformation of the active zone.

Preferred stable conditions.

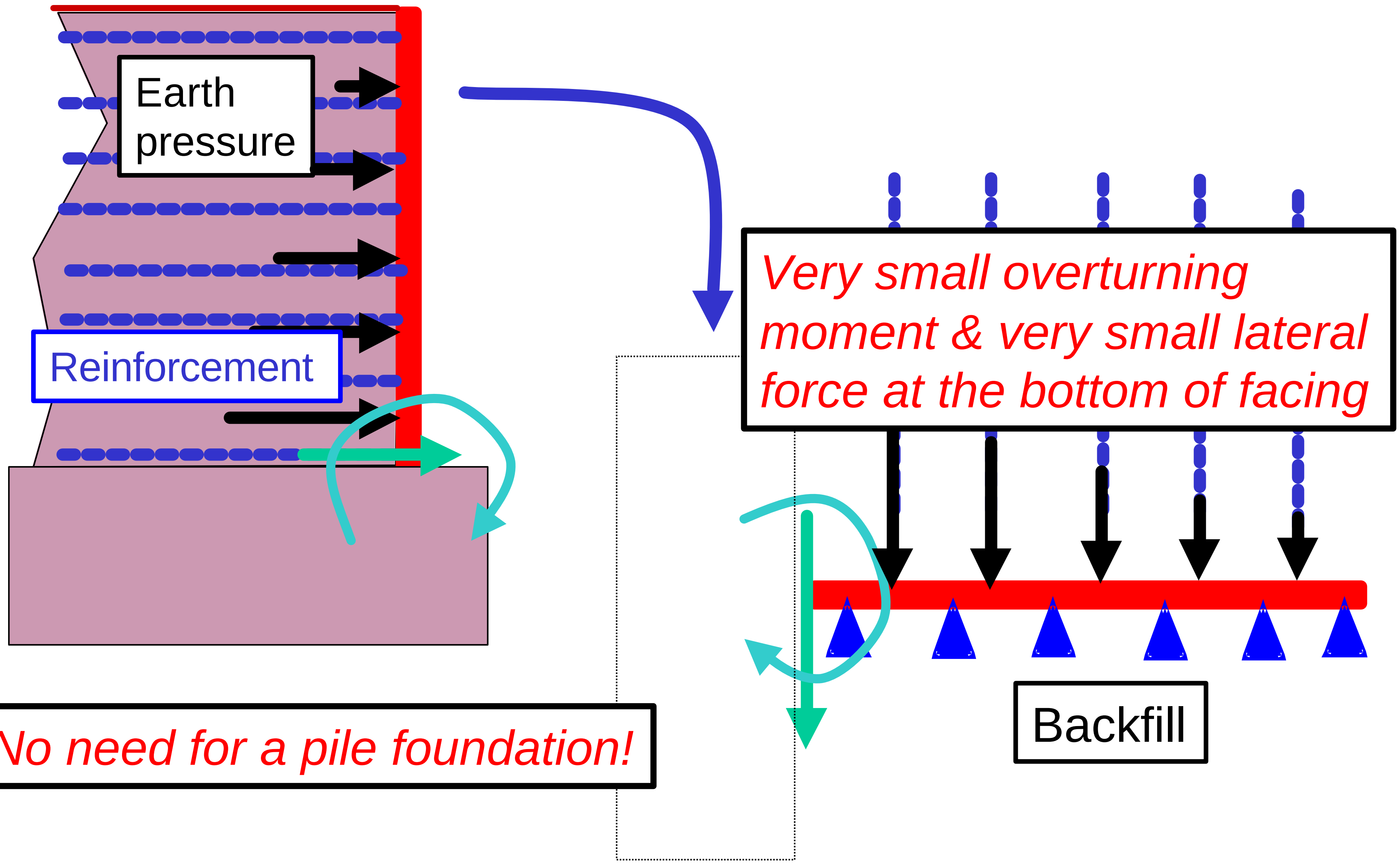


Distribution of tensile force when a rigid facing and reinforcement are connected.

GRS RWs with a full-height rigid facing: *a continuous beam on at a large number of supports with a small span*

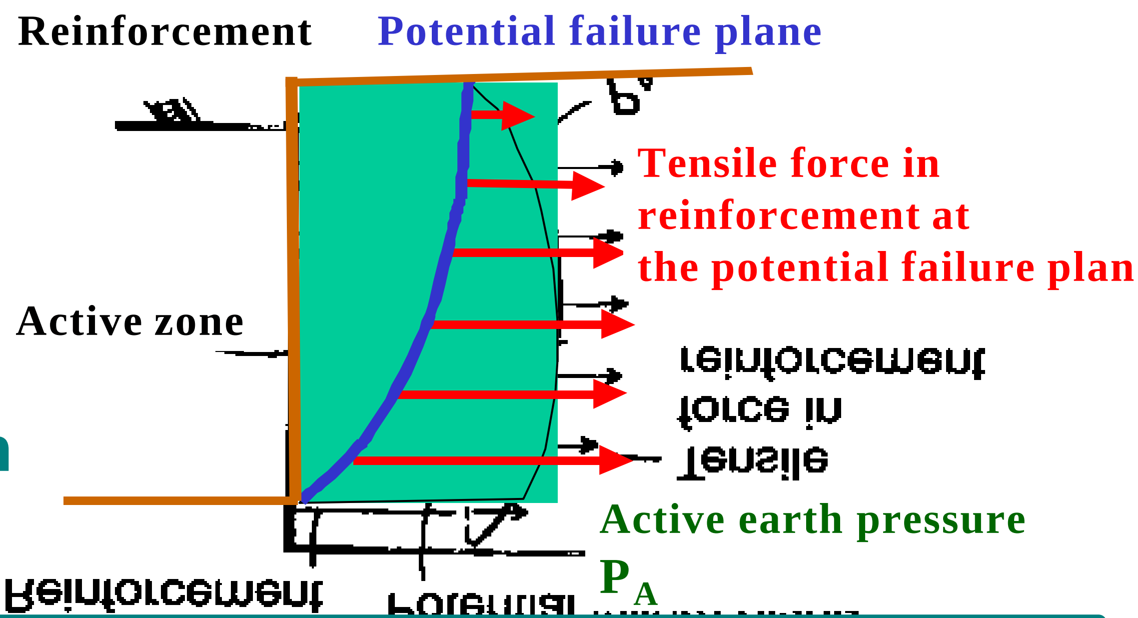


GRS RWs with a full-height rigid facing: *a continuous beam on at a large number of supports with a small span*



No need for a pile foundation!

Conventional explanation of the functions of facing



1) The facing is only to prevent the spilling out of backfill.
2) The earth pressure at the facing should be made low in the reinforced soil retaining wall.
3) The facing should be flexible enough to accommodate the deformation of supporting ground

This explanation is wrong.

Conventional explanation of the functions of facing

1) The facing is only to prevent the spilling out of backfill.
2) The earth pressure at the facing should be made low in the reinforced soil retaining wall.
3) The facing should be flexible enough to accommodate the deformation of supporting ground

The correct explanations

Conventional explanation of the functions of facing

1) The facing is an important and essential **structural component** confining the backfill and developing large tensile force 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 after the start of service**.

Topics

1-1 Re-consideration on the advantages of reinforced soil RWs

1-2 Advantages of using a full-height rigid facing

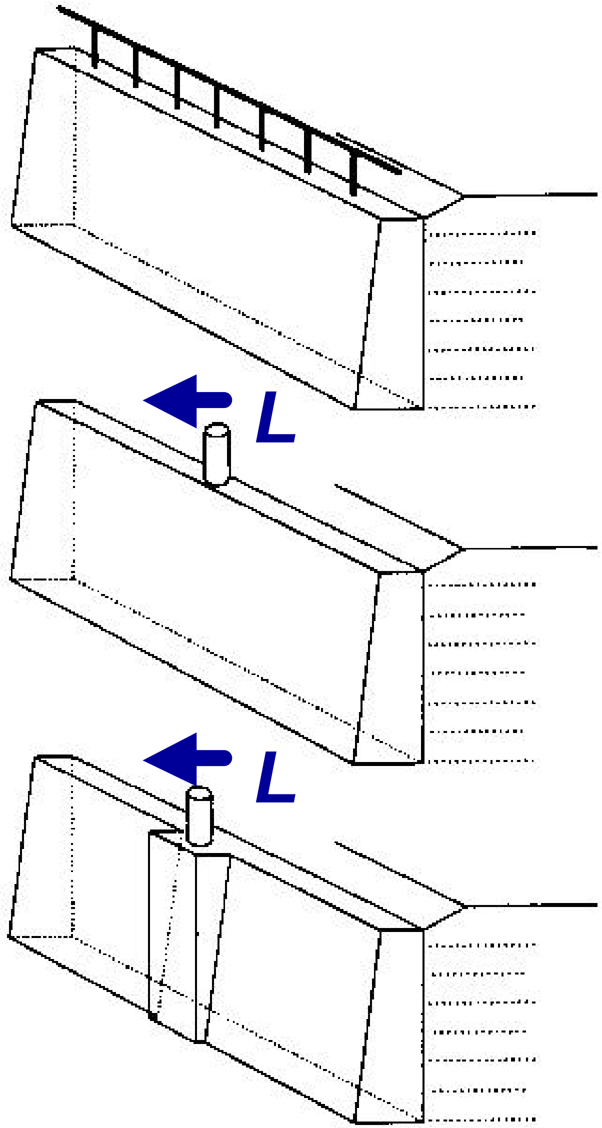
1-3 Advantages of the staged construction procedure

1-4 Some typical case histories

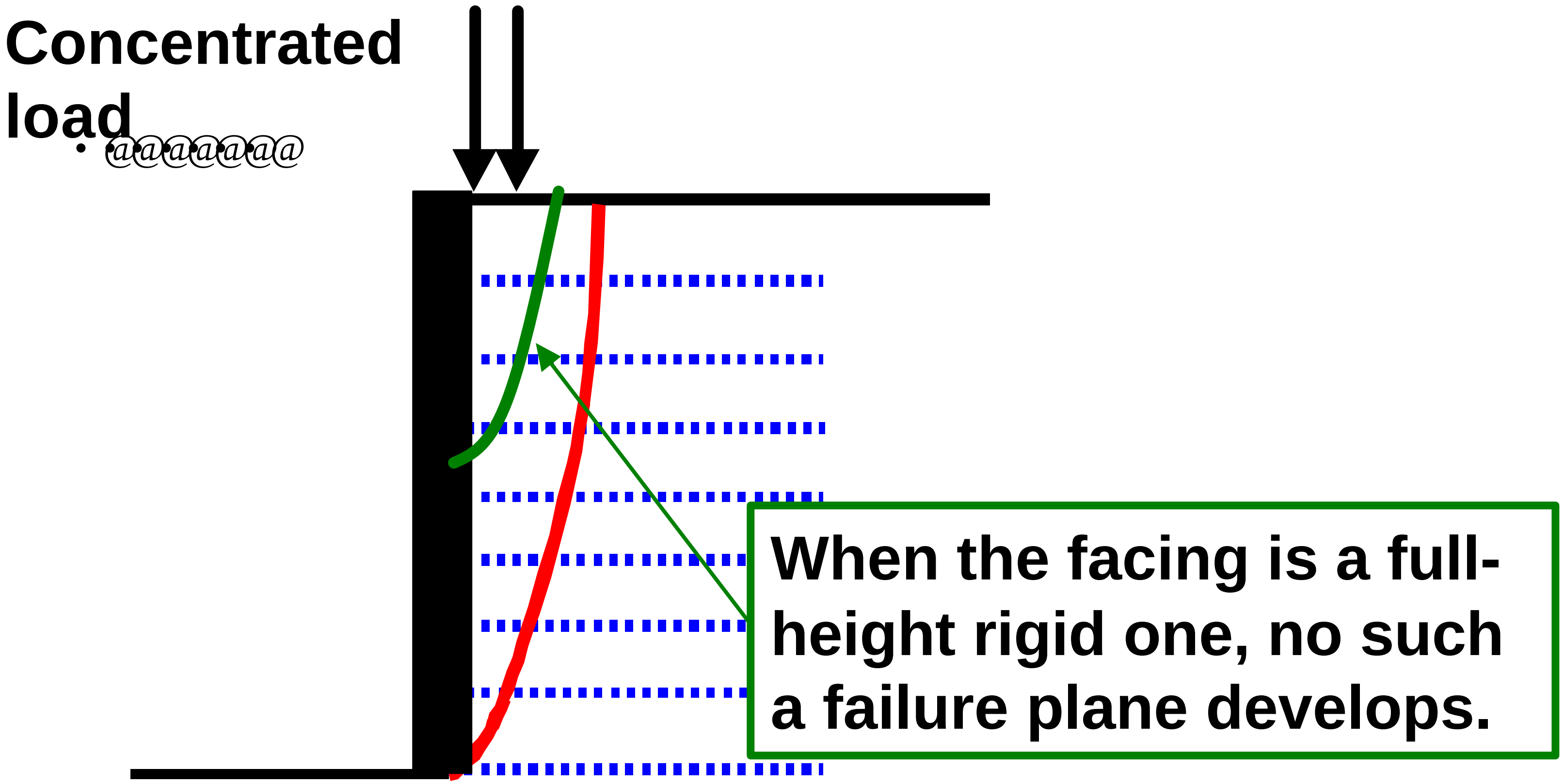
3D effects !

One unit of reinforced backfill and facing resists against the lateral load *L* as a monolith.

?A full-height rigid facing can become a foundation structure to support super-structures, such as electric poles and noise barrier walls

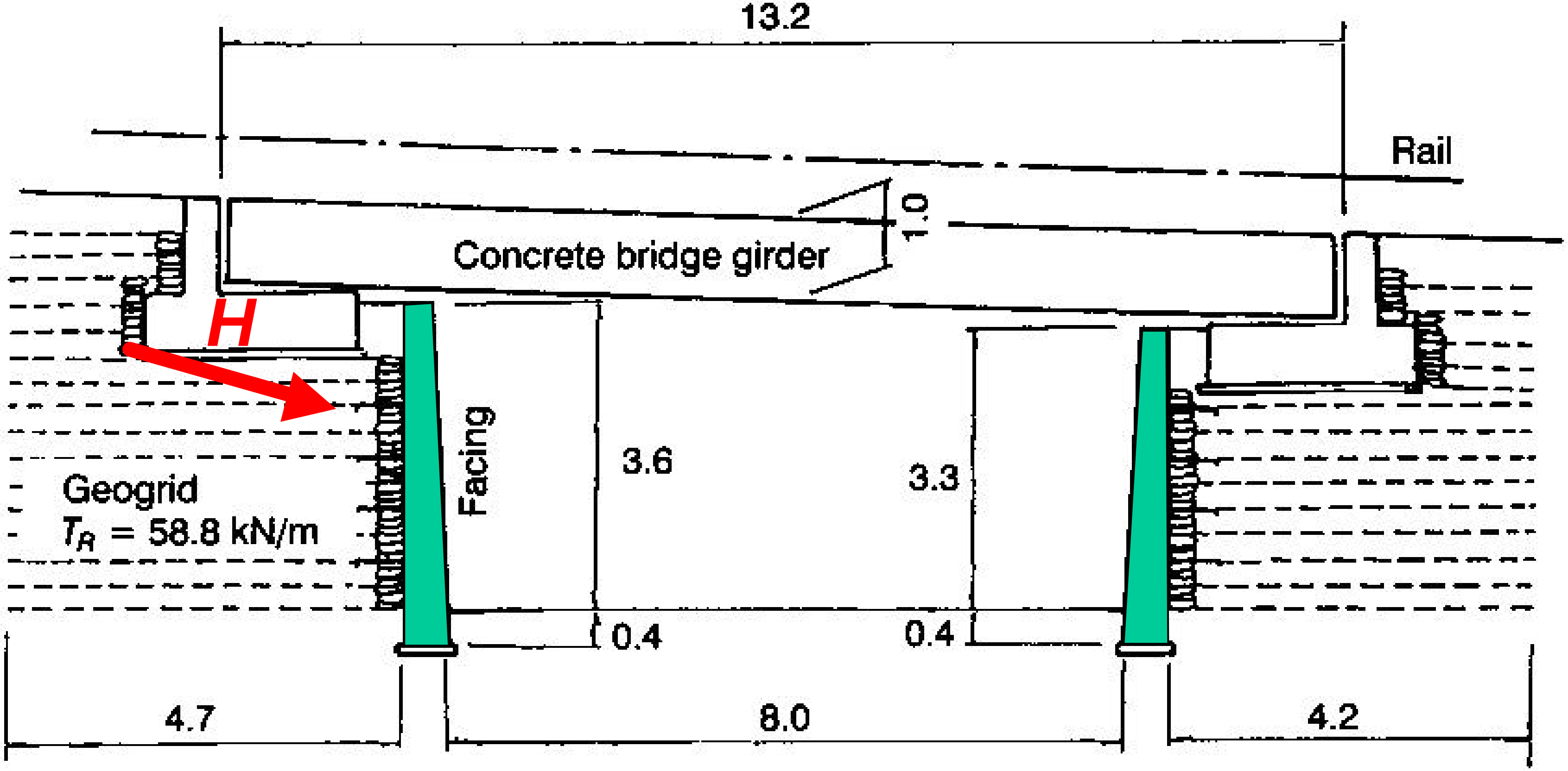


Contributions of the rigidity of facing and connection of reinforcement to the facing:



Bridge abutments of GRS with a full-height rigid facing

Large load H (in particular, seismic load) from the bridge girder is resisted by the facing anchored with the geotextile layers for the full wall height !



A pair of GRS bridge abutments for Seibu Line, Tokyo

Topics

1-1 Re-consideration on the advantages of reinforced soil RWs

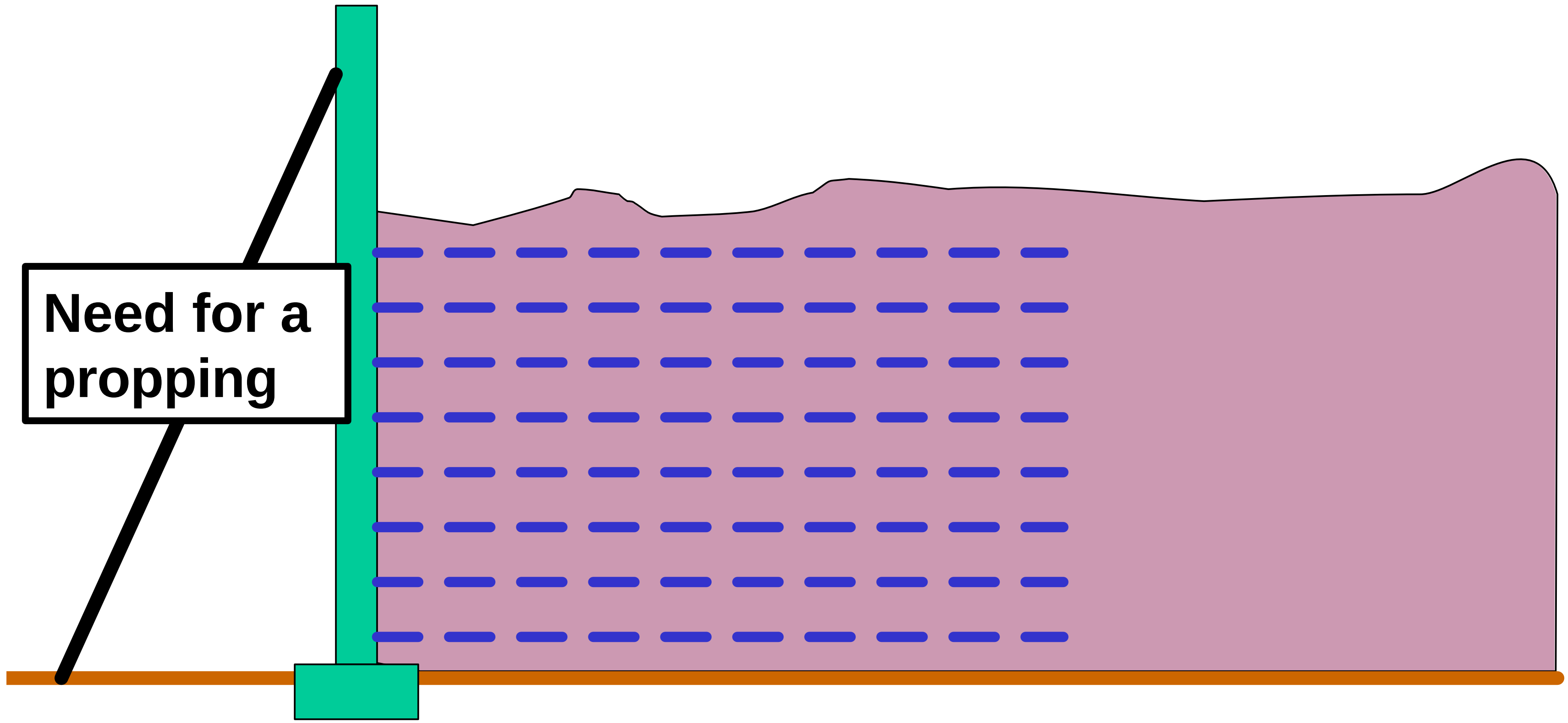
1-2 Advantages of using a full-height rigid facing

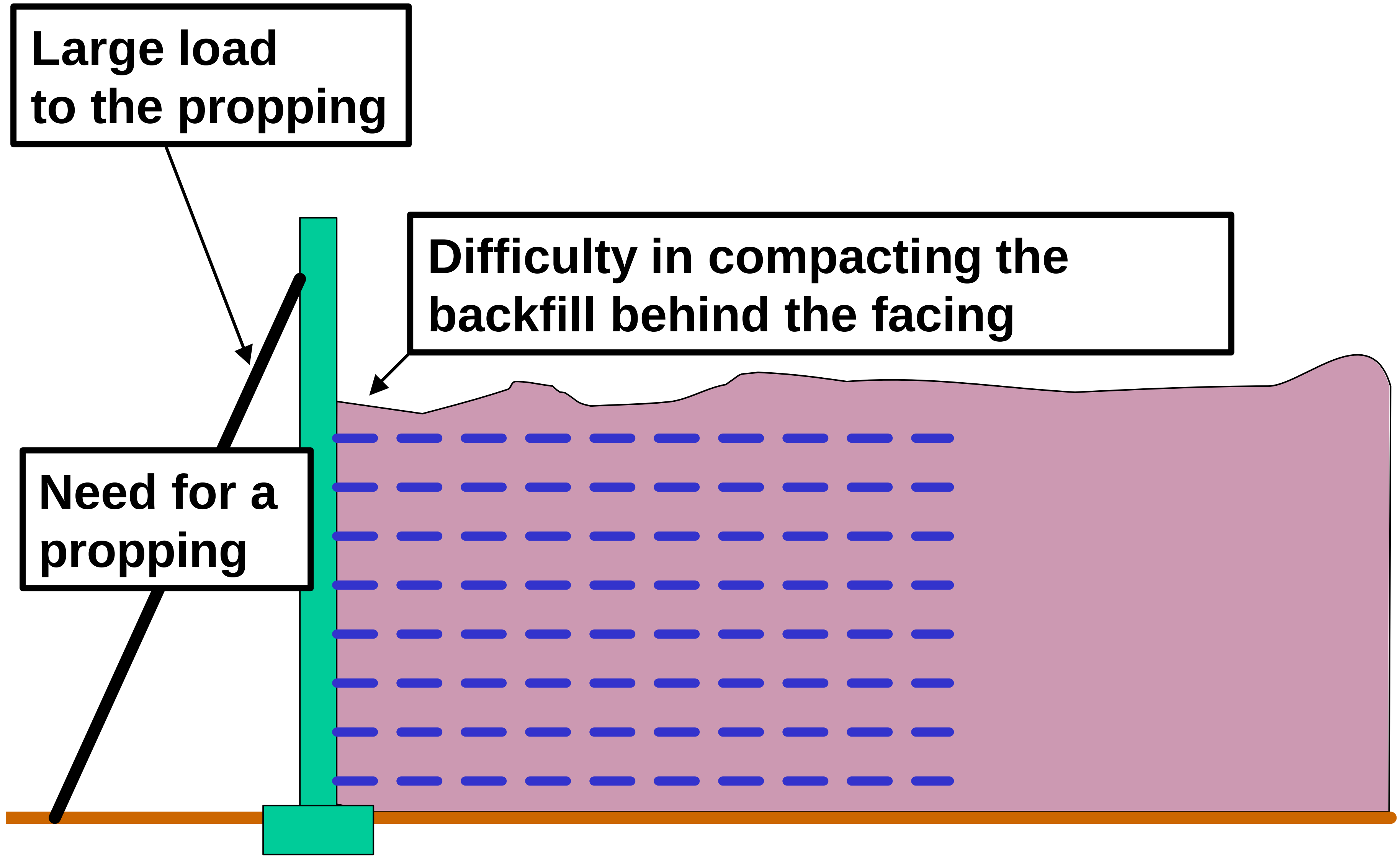
1-3 Advantages of the staged construction procedure

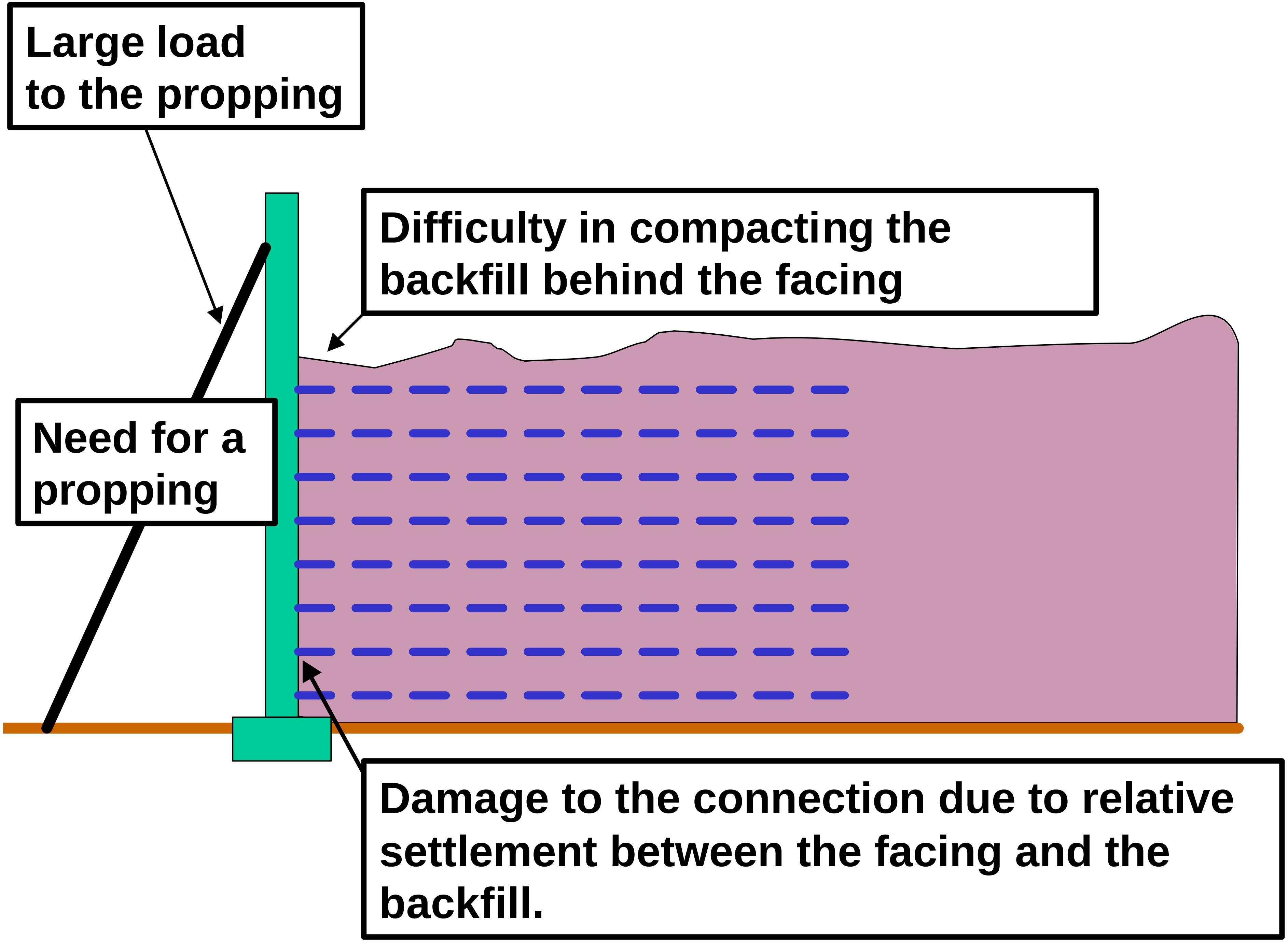
1-4 Some typical case histories

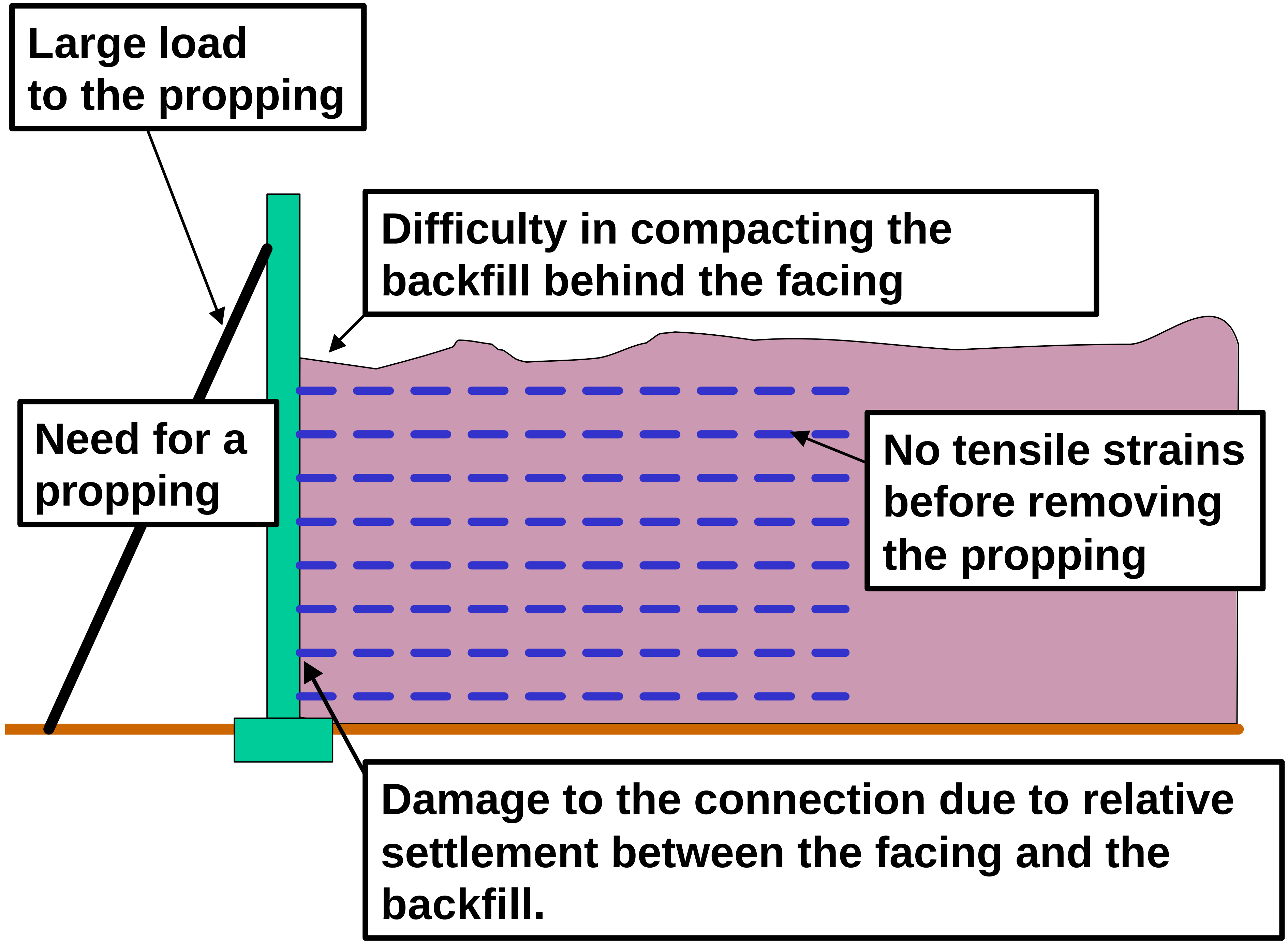
Full-height rigid facing contributes to the wall stability,

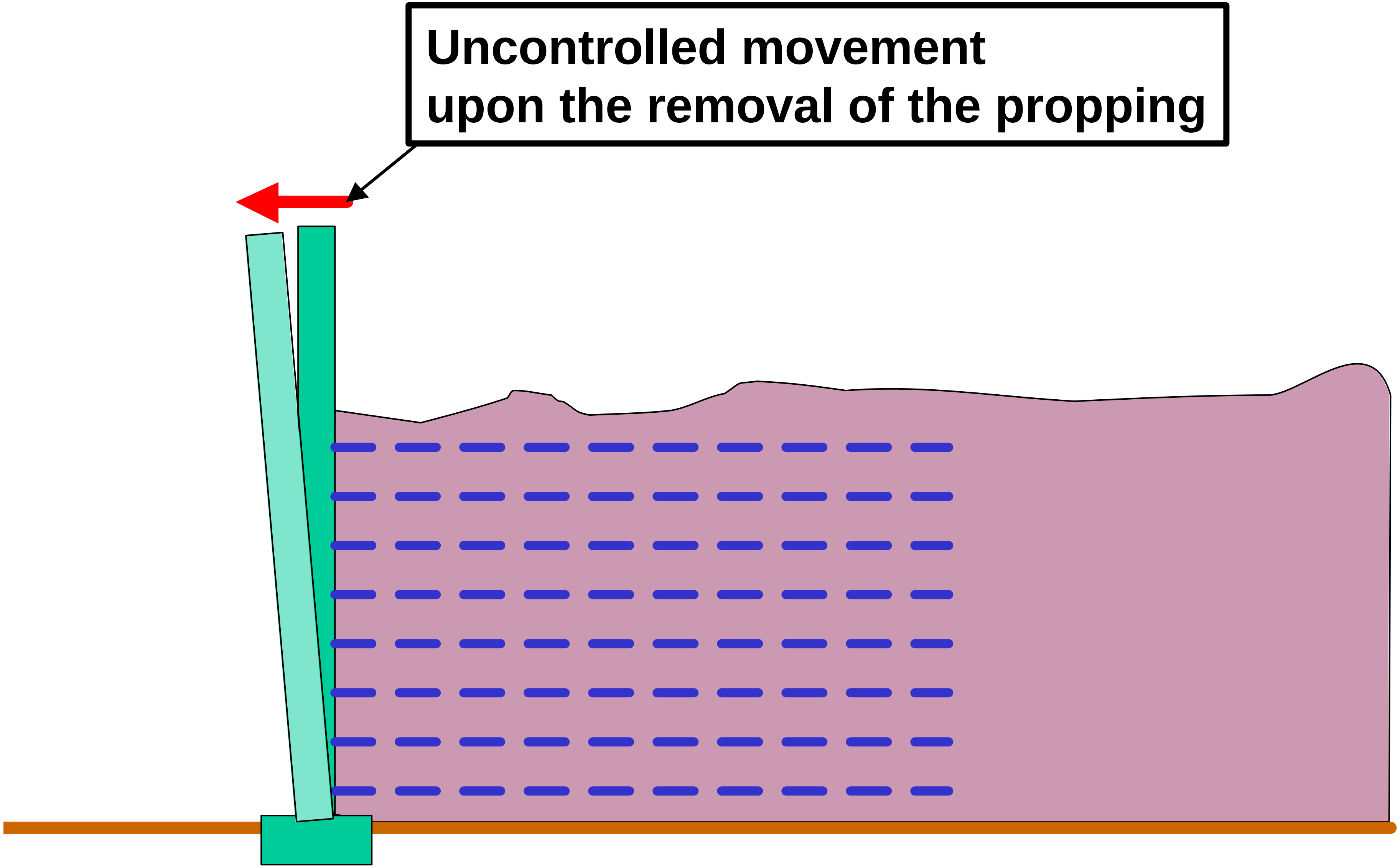
but, several problems during wall construction




Large load
to the propping
Difficulty in compacting the
backfill behind the facing
Need for a
propping


Large load
to the propping
Difficulty in compacting the
backfill behind the facing
Need for a
propping
Damage to the connection due to relative
settlement between the facing and the
backfill.


Large load
to the propping
Difficulty in compacting the
backfill behind the facing
Need for a
propping
No tensile strains
before removing
the propping
Damage to the connection due to relative
settlement between the facing and the
backfill.

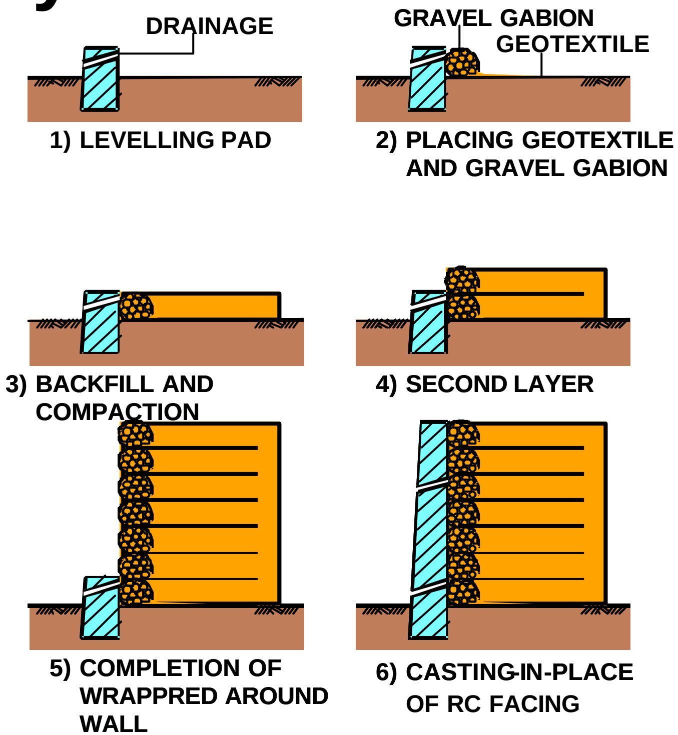


? Most of these problems can be solved by the staged construction procedure …..

The advantages of the staged construction

1) No interaction between a rigid facing and deformable backfill during filling-up and compaction;

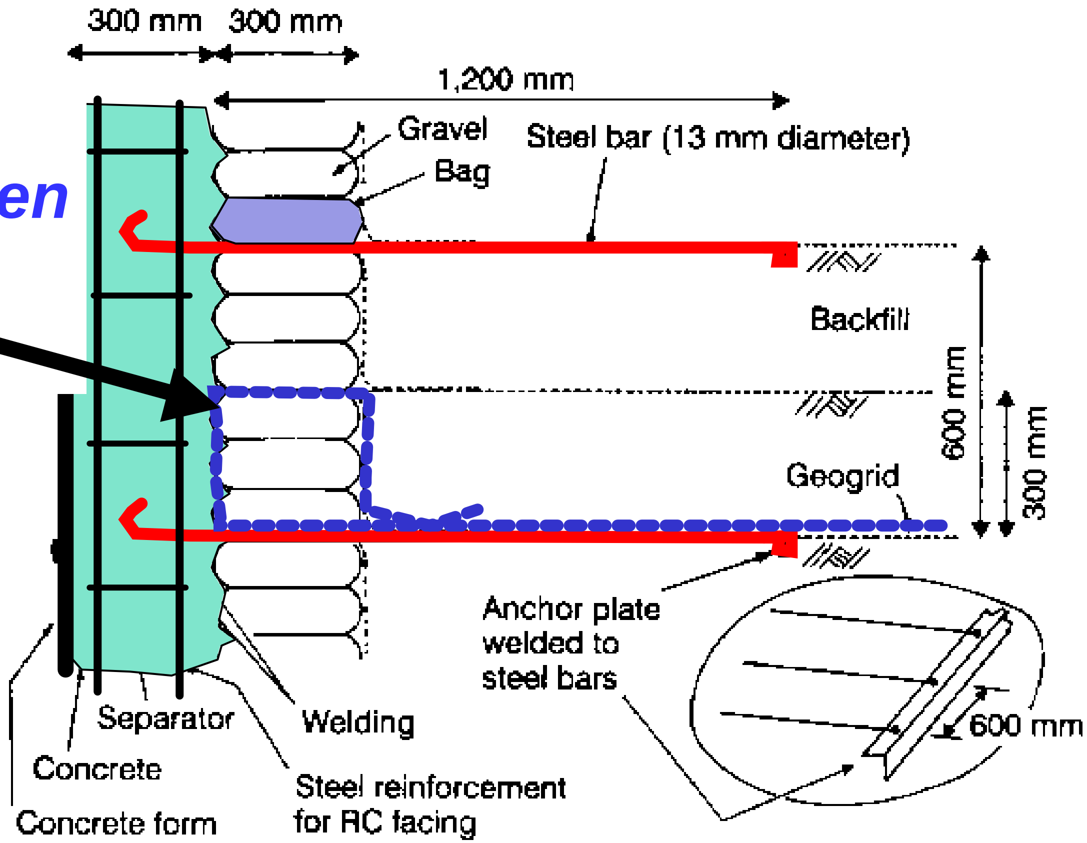
2) Also, large deformation of the supporting ground can be accommodated, without losing the stability of wall.



- **Easy compaction of the backfill back of the wall face**
- **Better mobilization of reinforcement tensile force**

Casting-in-place of full-height rigid facing

Nearly perfect contact and connection between the concrete and the geotexitile !

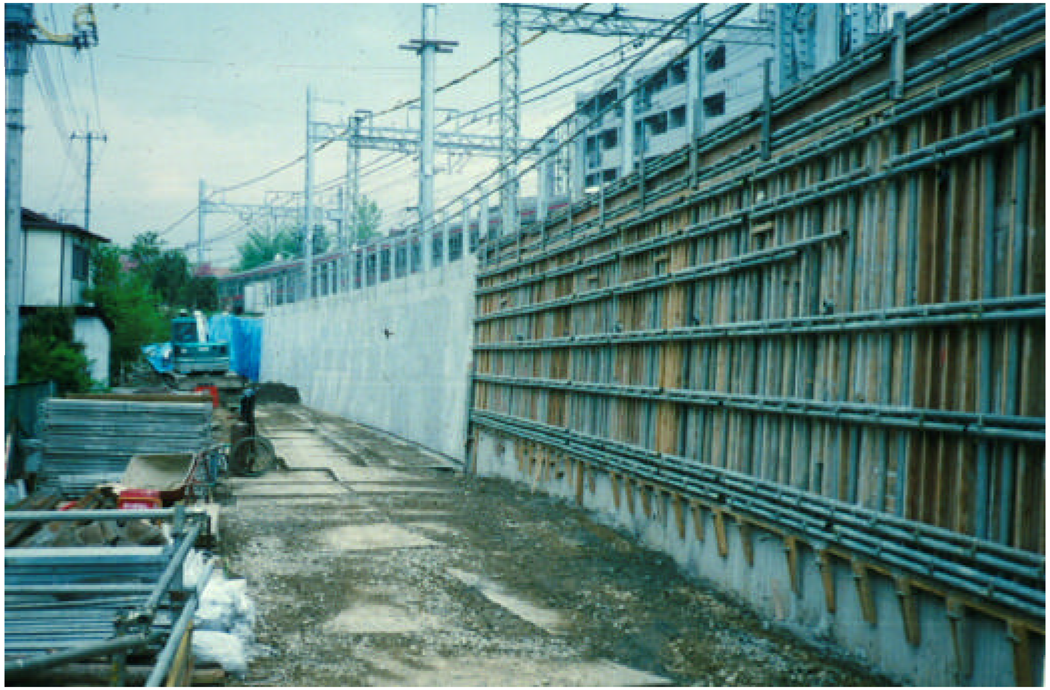


A support of the concrete form anchored inside the backfill

A propping occupying a large space in front of wall for casting-in-place concrete for a conventional cantilever RW.

A propping occupying a large space in front of wall for casting-in-place concrete for a conventional cantilever RW.

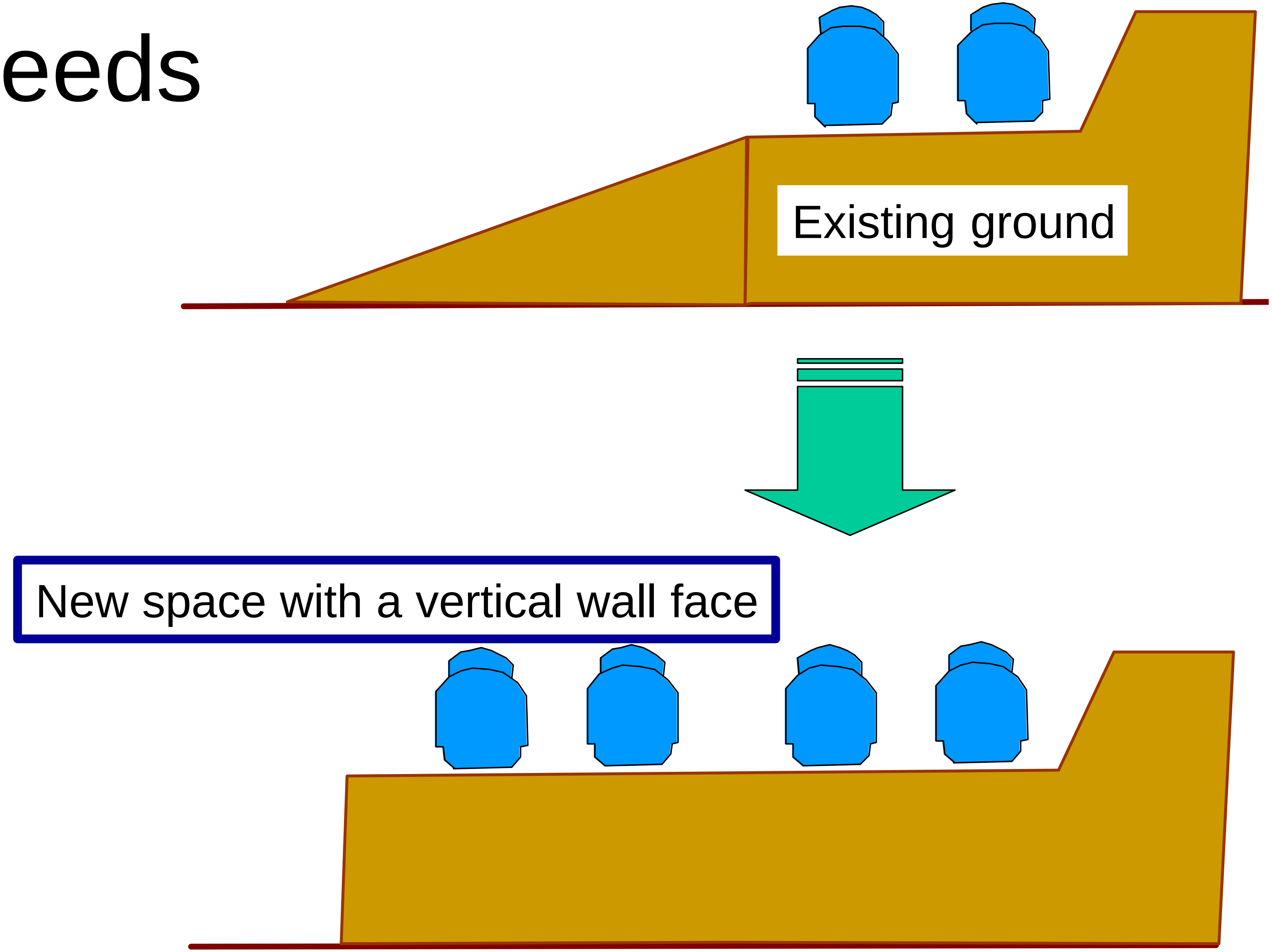
No need for a propping in front of the GRS wall

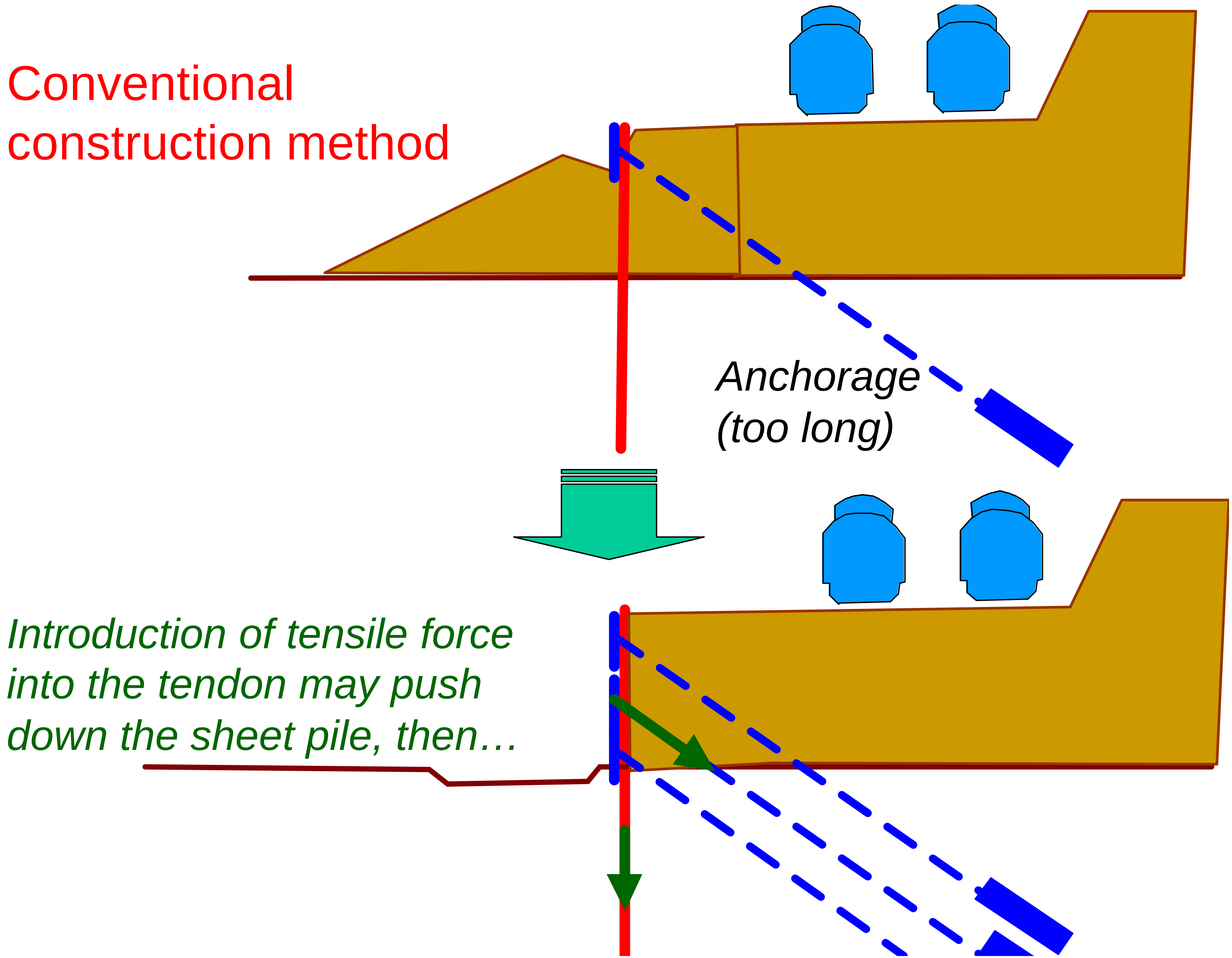


The advantages of the staged construction

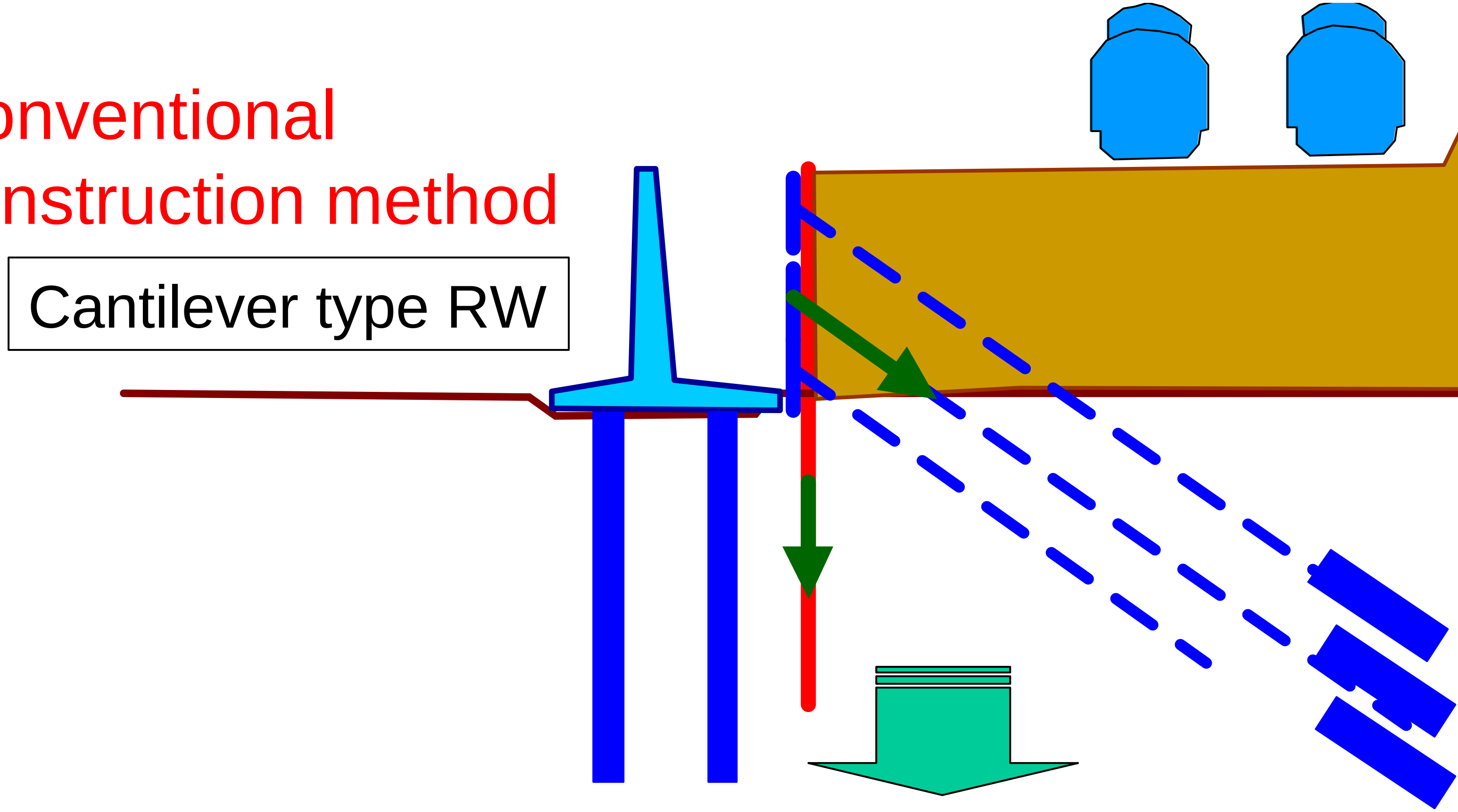
3) Easy alignment of the completed wall face

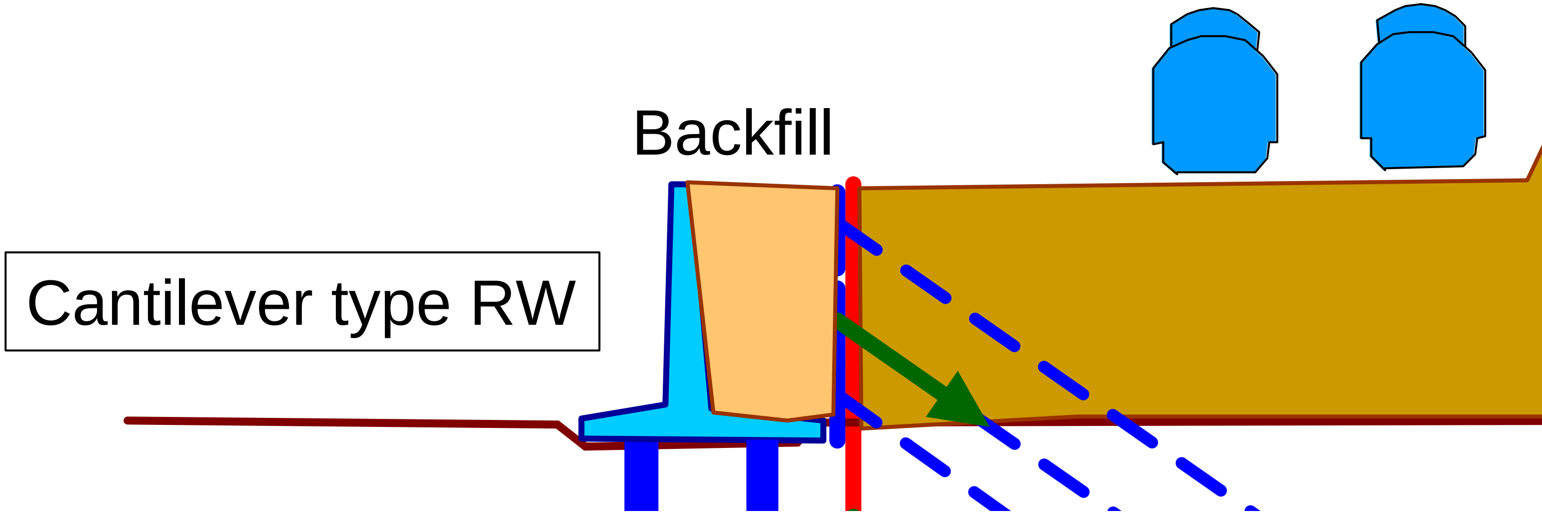
Needs



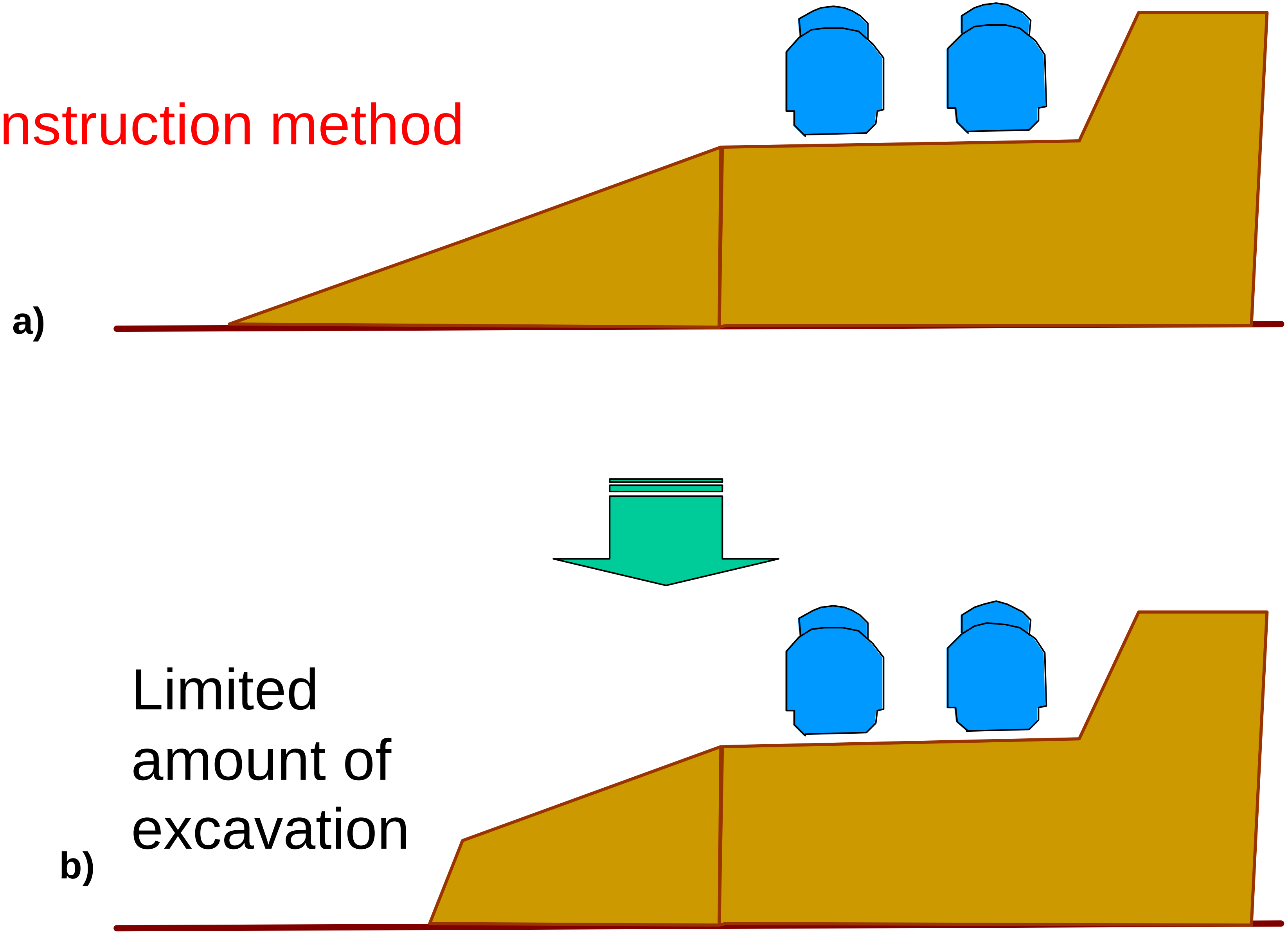

Conventional construction method
Anchorage (too long)
Introduction of tensile force into the tendon may push down the sheet pile, then…

Conventional construction method



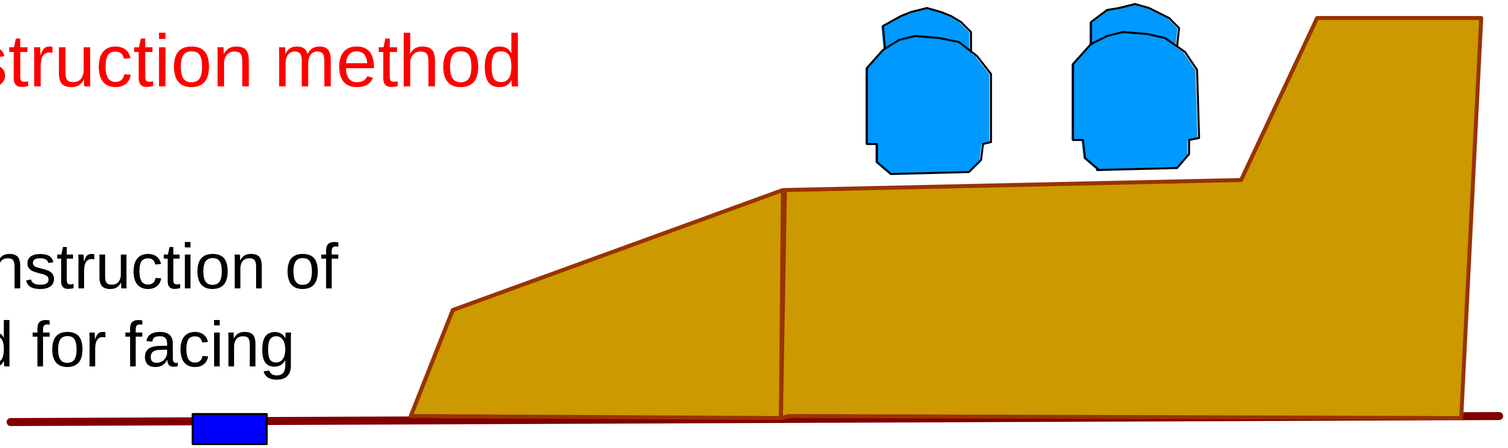


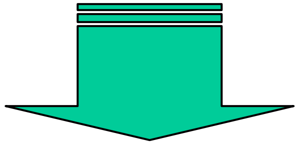
New construction method



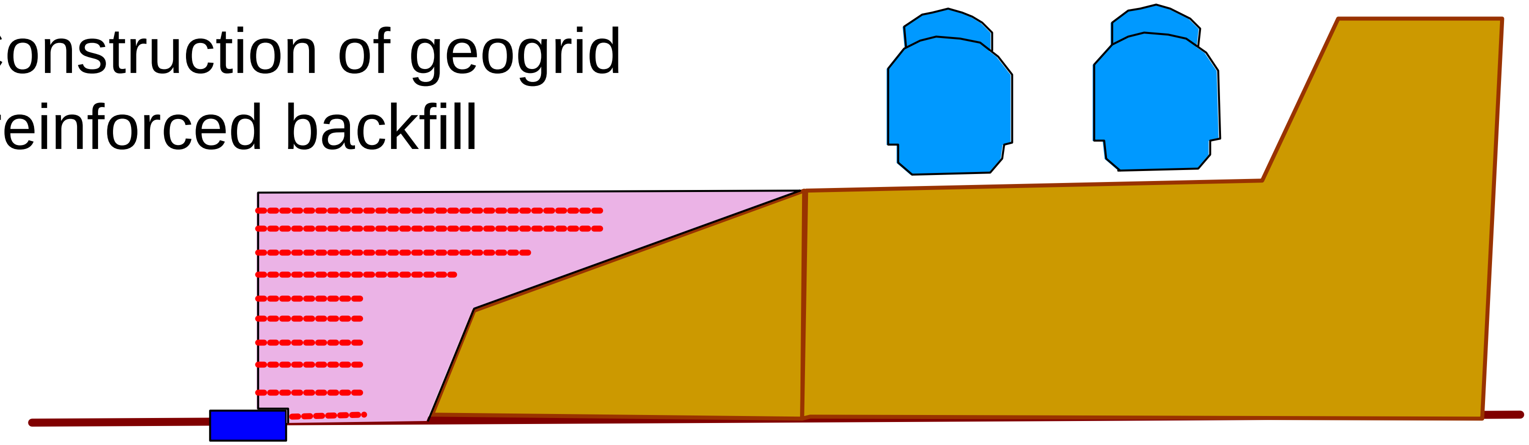
New construction method

c) Construction of pad for facing



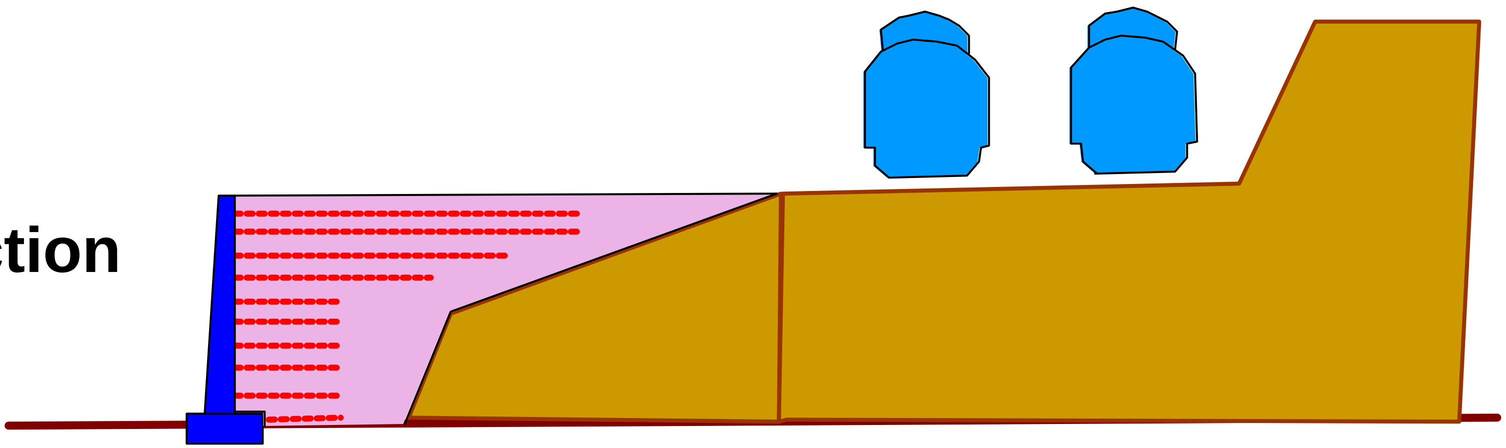


d) Construction of geogrid -reinforced backfill



New construction method

1. A much smaller number of construction steps
2. No use of temporary structure
   (i.e., sheet piles & anchors)
3. Much smaller occupied space
4. Self-supporting wall structure
   (usually no piles needed)



Topics

1-1 Re-consideration on the advantages of reinforced soil RWs

1-2 Advantages of using a full-height rigid facing

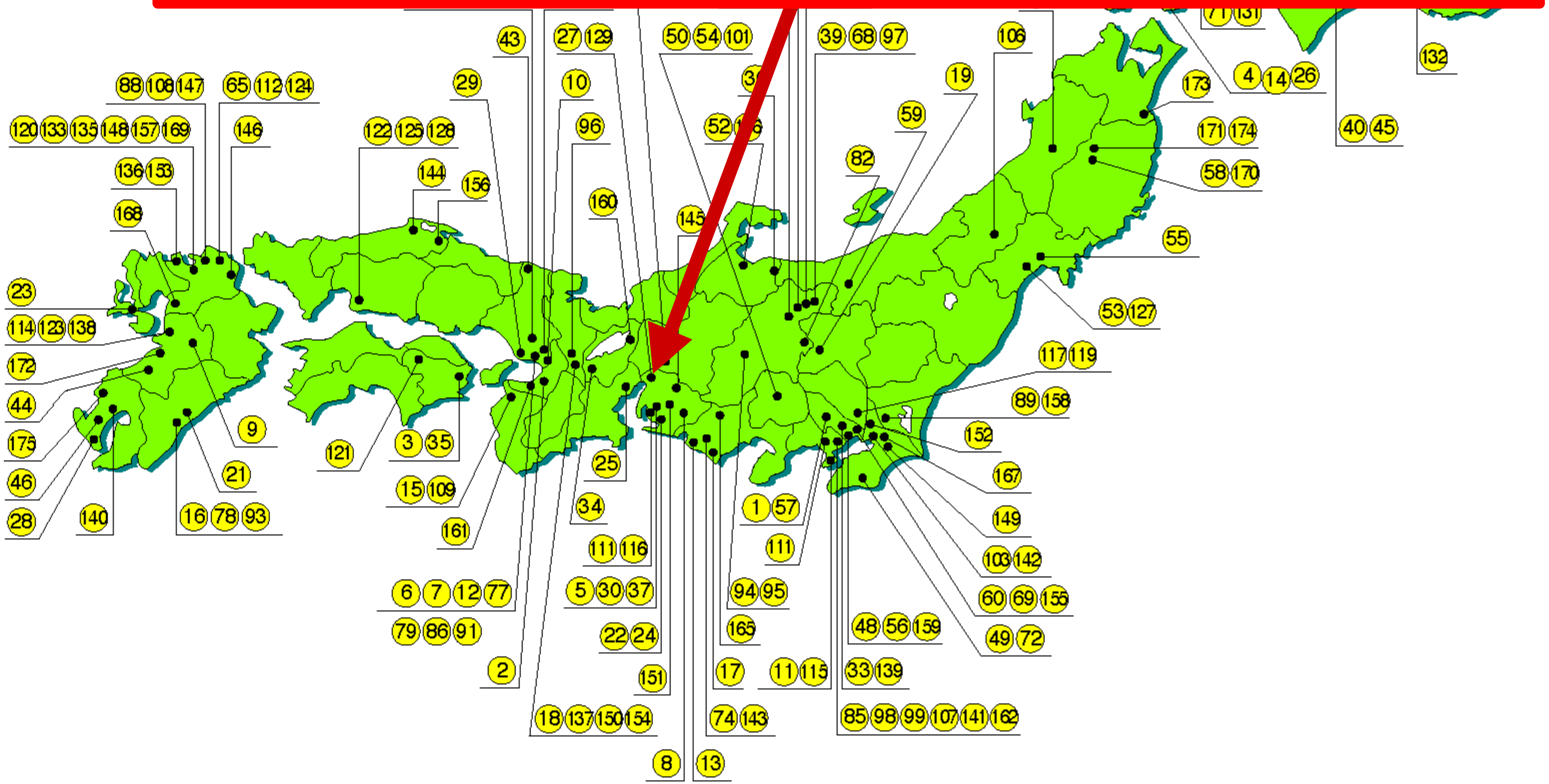
1-3 Advantages of the staged construction procedure

1-4 Some typical case histories

Nagoya wall:

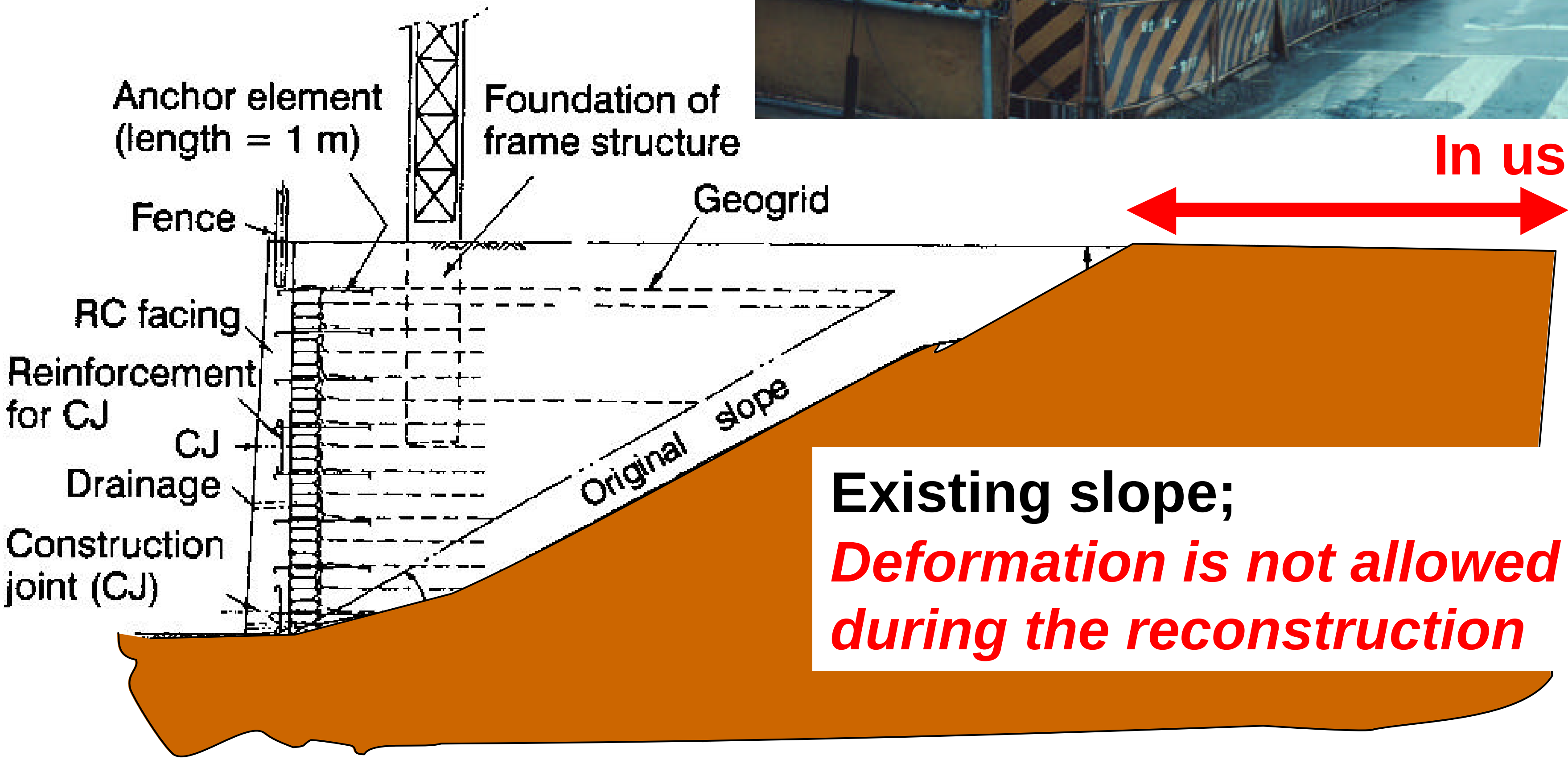
*** the first large scale project**

*** constructed for a period from 1990 to 1991**



Sites of GRS RWs with a full-height rigid facing that have been constructed by the end of April 2000

Reconstruction of an existing slope to a vertical wall



Reconstruction of the slope of embankment
- to GRS-RWs having a FHR facing
- for a yard of bullet trains (Shinkan-Sen)

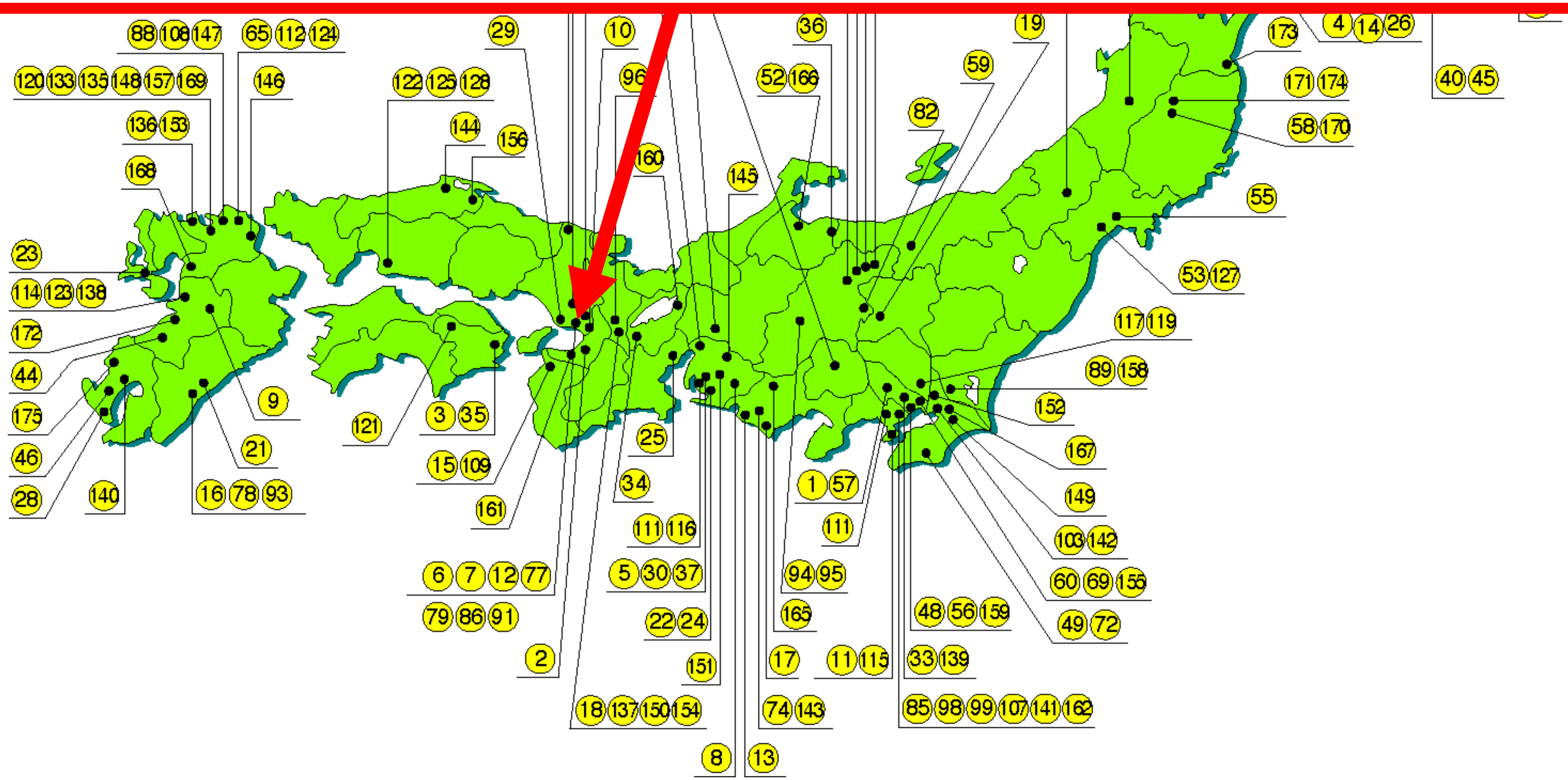
Average wall height= 5 m; and total length= 930 m

The first bridge abutments of geogrid-reinforced soil

Amagasaki wall:

*** The first large scale GRS-RW to support directly tracks for a very busy and rapid railway; &**

*** constructed for a period from 1991 to 1992**



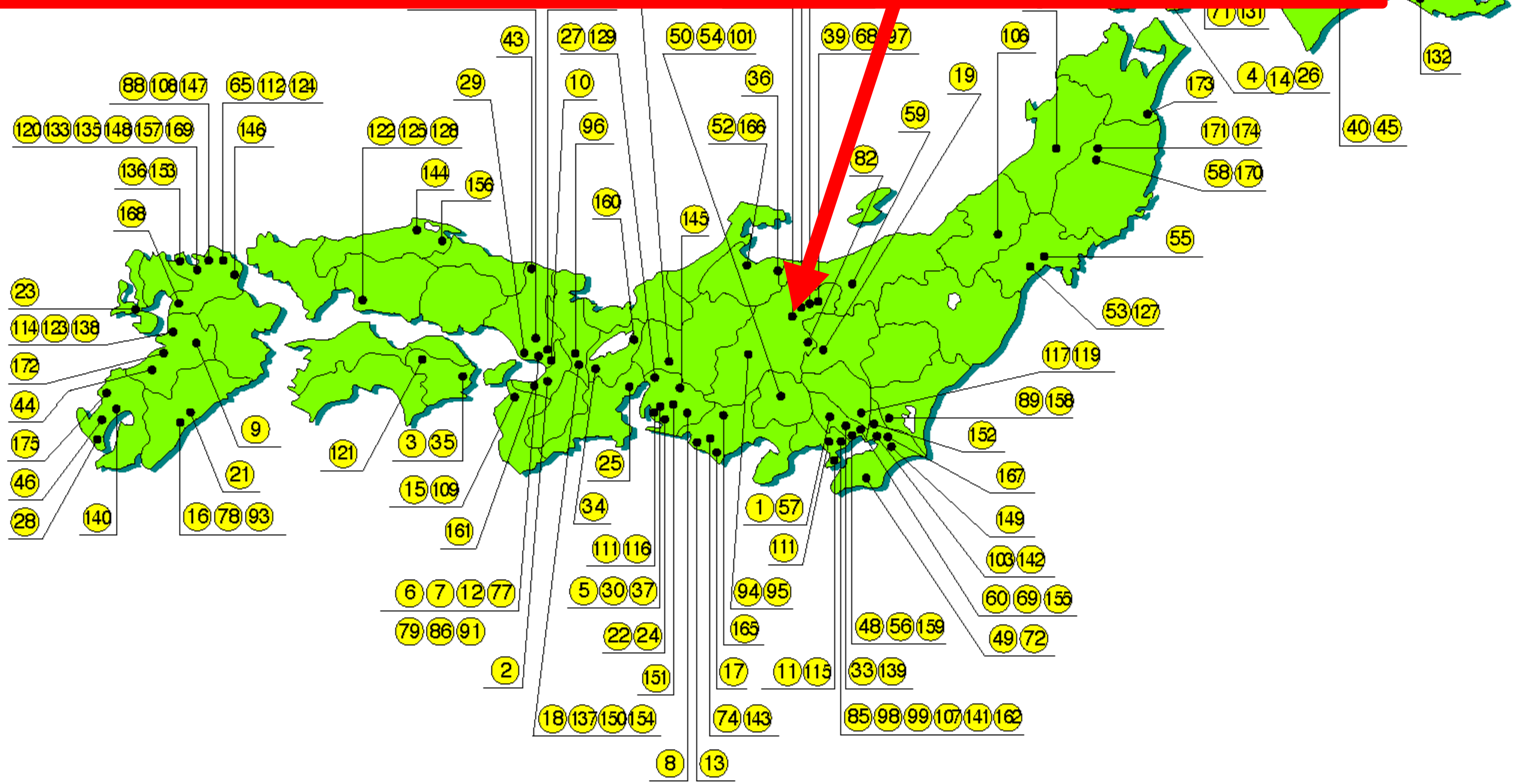
Sites of GRS RWs with a full-height rigid facing that have been constructed by the end of April 2000

Both sides of embankment reconstructed to GRS-RWs having a full-height rigid facing under *a severe space restriction*

Average wall height= 5 m and total length= 1,300 m

Nagano wall:

* for a yard for Shinkansen (bullet train); and
* constructed for a period from 1993 to 1996

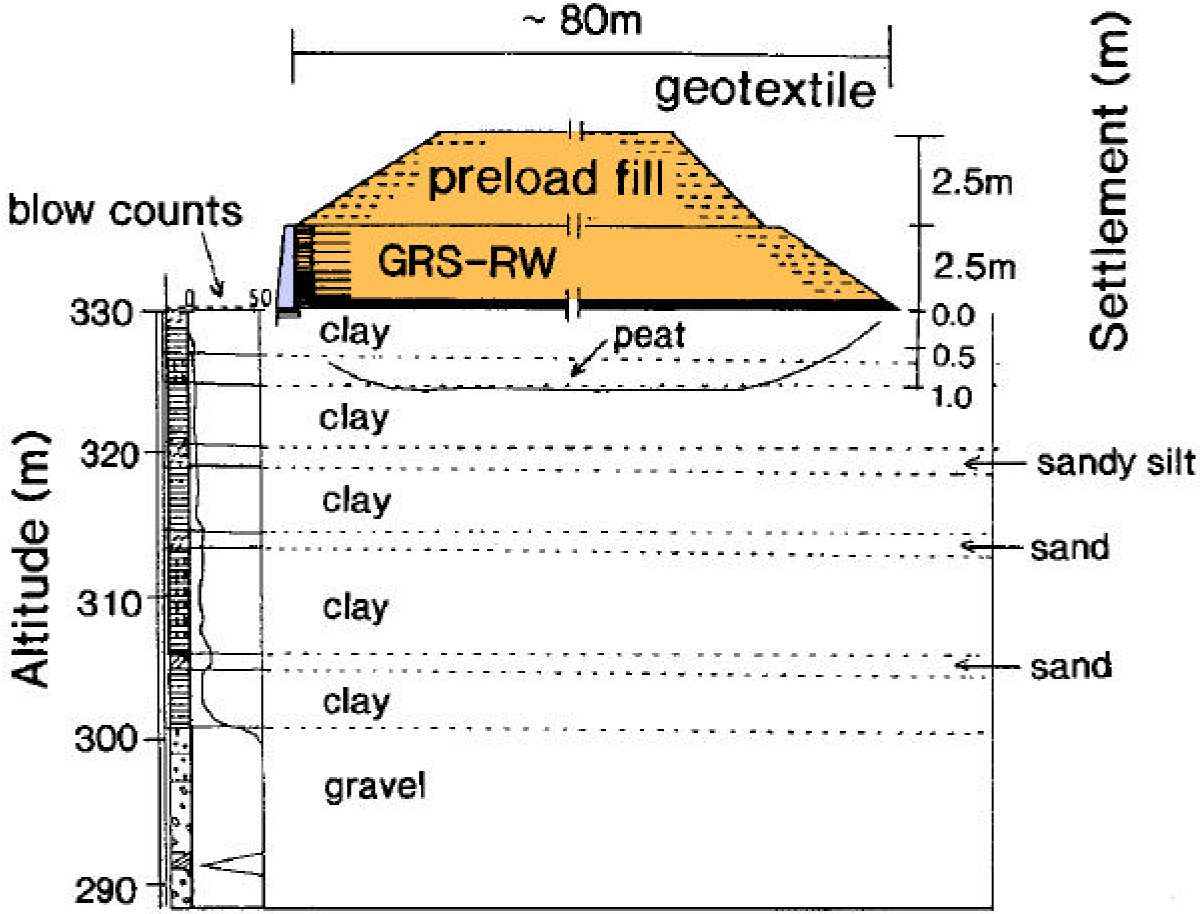


Sites of GRS RWs with a full-height rigid facing that have been constructed by the end of April 2000

Nearly saturated highly weathered tuff – Nagano wall

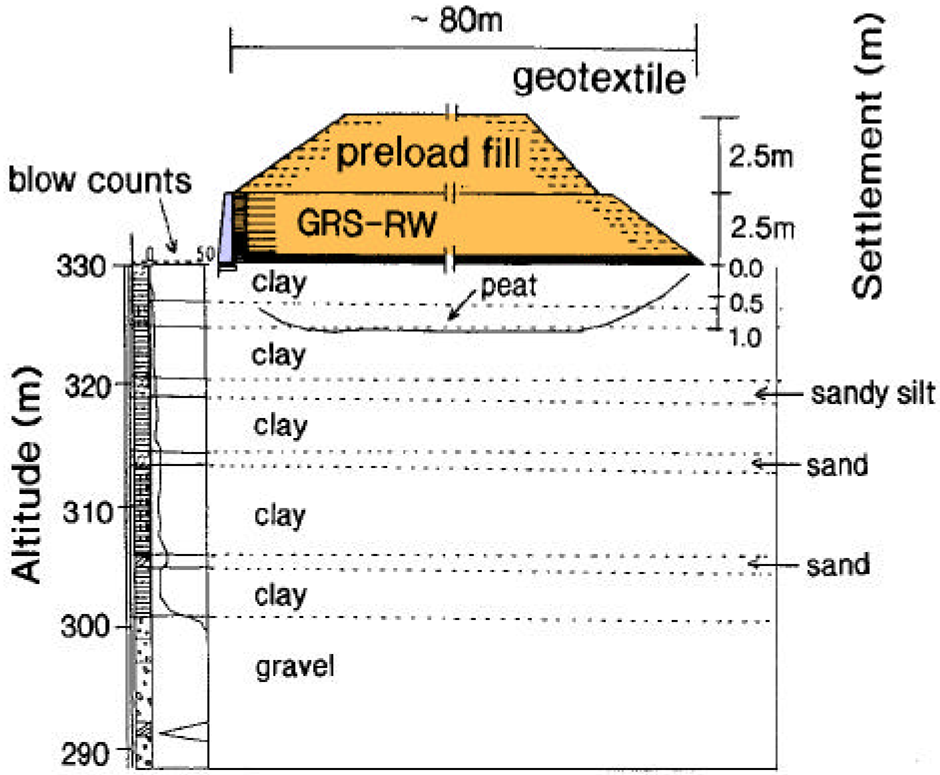
-constructed in 1994 to reconfirm the function of full-height rigid facing;

- in conjunction of the construction of proto-type GRS-RWs for 1993 - 1994.

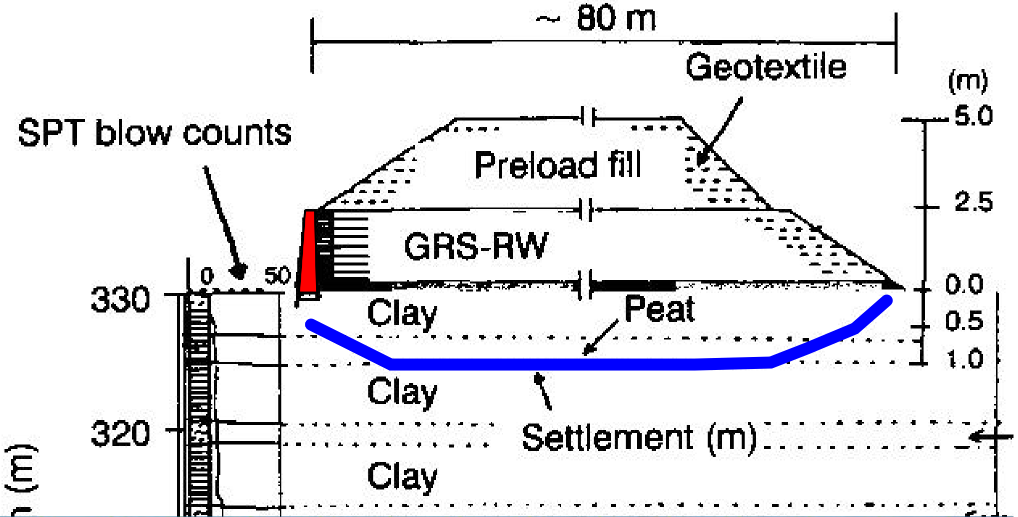


Nearly saturated highly weathered tuff – Nagano wall

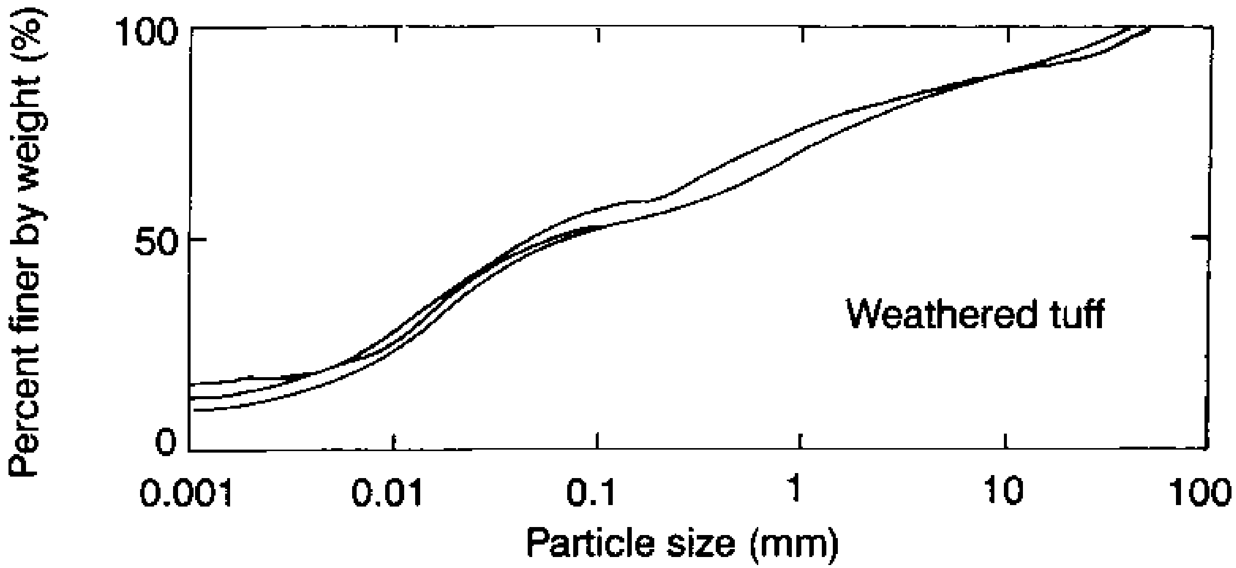
a) a complete wall height of 2 m for a length of 2 km, supporting a yard for Shinkansen (bullet train);
b) ***the first actual clay wall using a nearly saturated soft clay*** as a railway structure in Japan,;
c) constructed on a thick very soft clay deposit;
d) a large ground settlement of about 1 m by preloading before casting-in-place a rigid facing; and
e) no pile foundation.



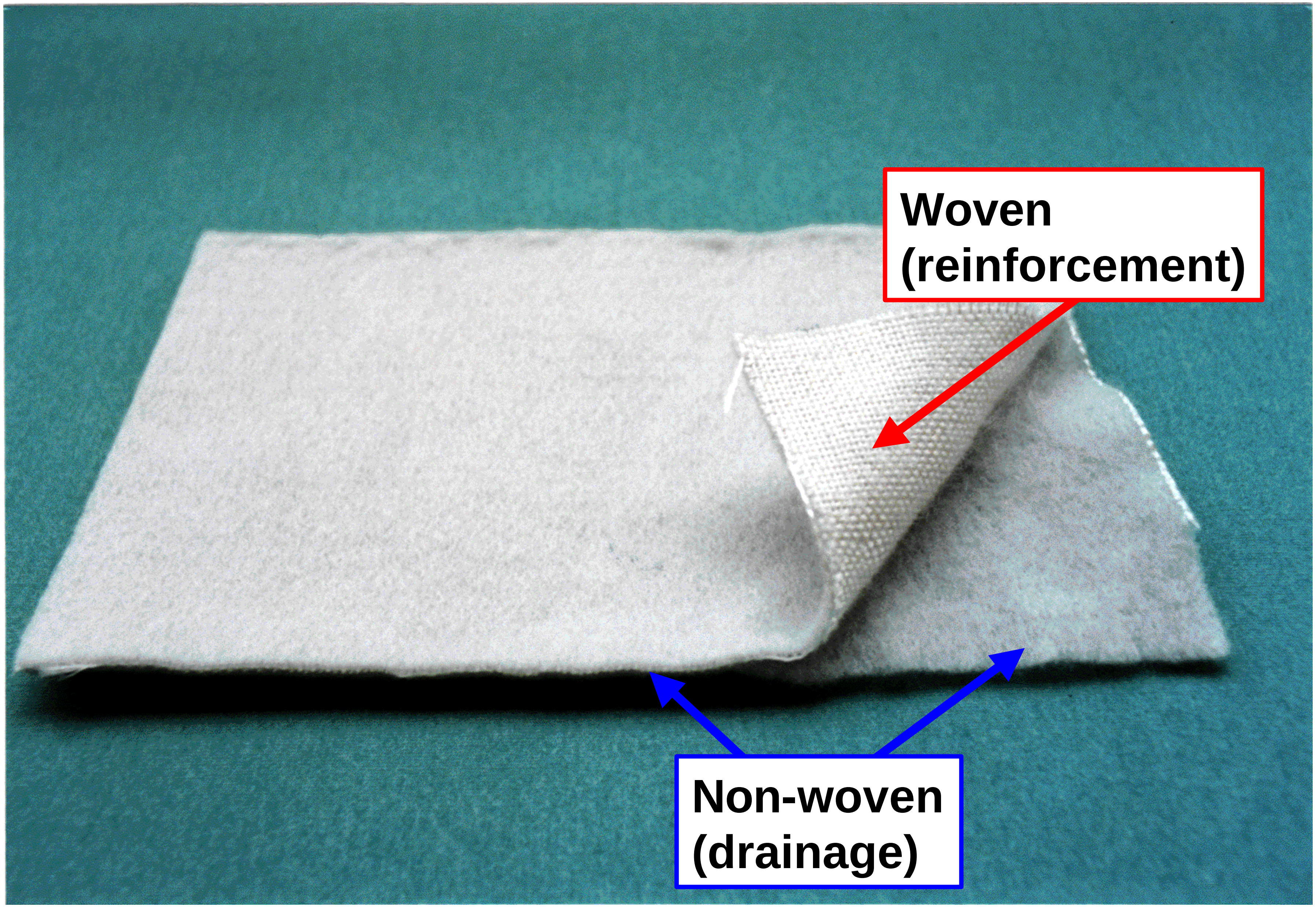
- *Settlement of the embankment by preloading; about 1 m*
- *Casting-in-place of FHR facing was after removing the preload fill.*

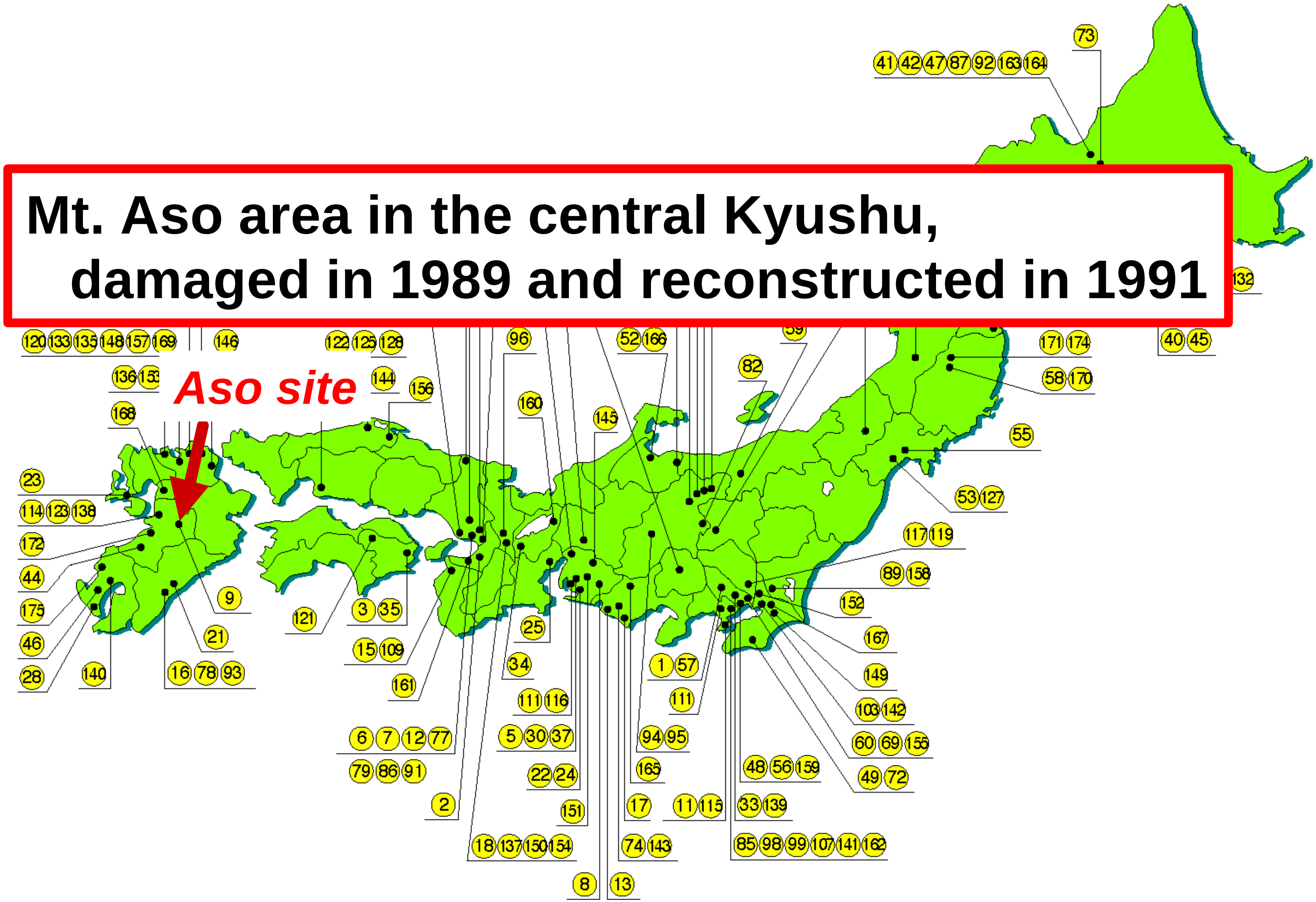


A nearly saturated clay



A composite of non-woven/woven geotextiles



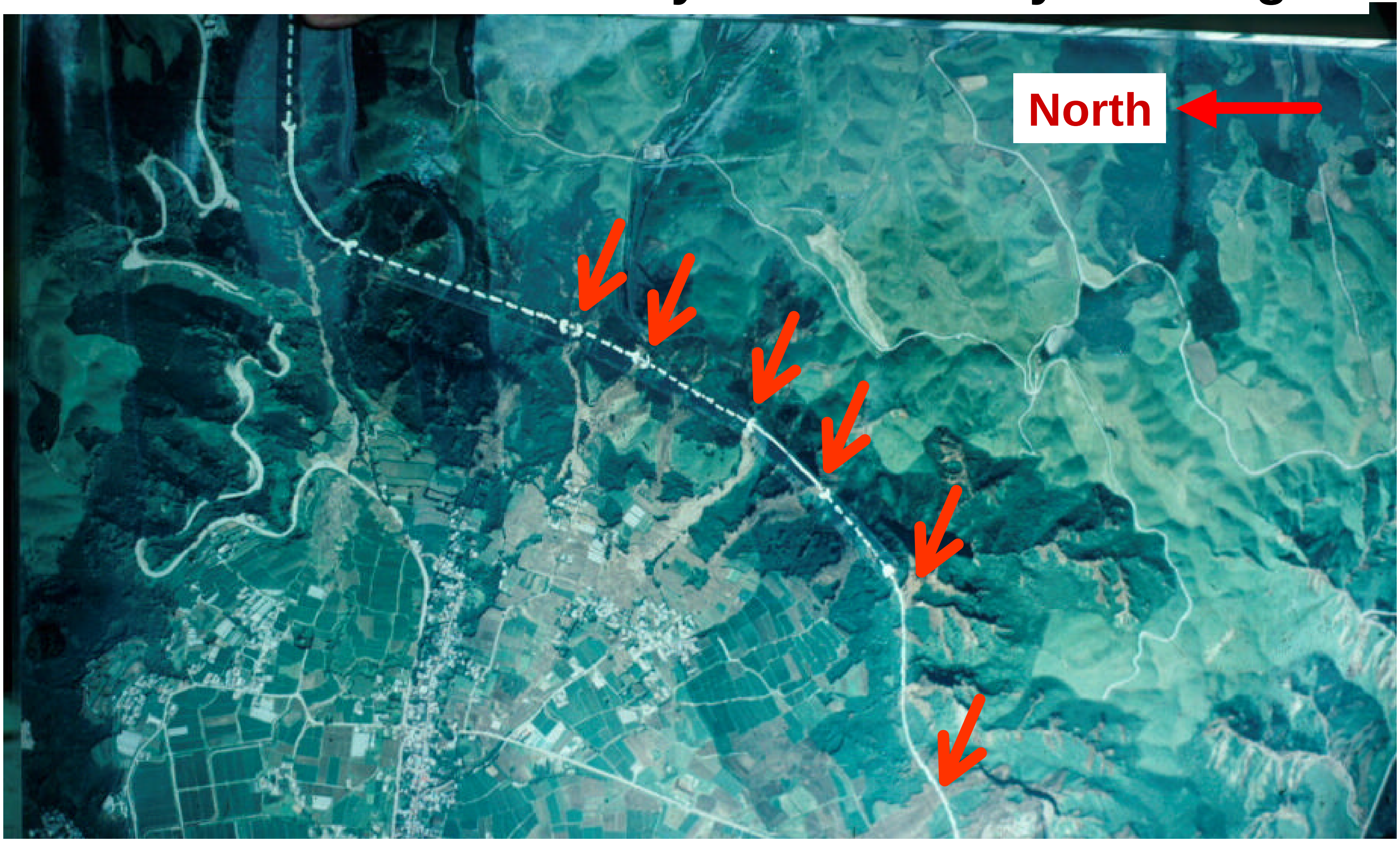


Mt. Aso area in the central Kyushu, damaged in 1989 and reconstructed in 1991

Sites of GRS RWs with a full-height rigid facing that have been constructed by the end of April 2000

Aso site:

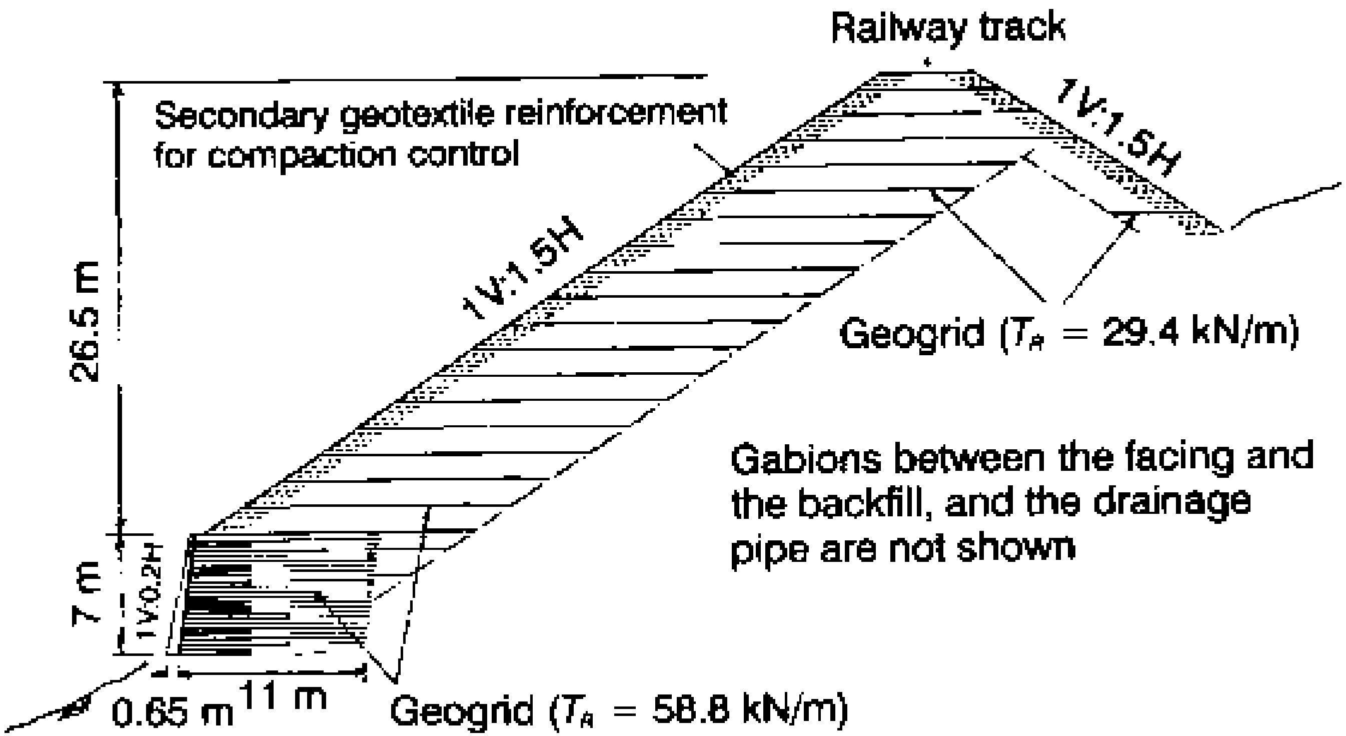
- A series of full sections of railway embankments located in narrow valleys were lost by flooding.



Dam-up of flood water in the upper reach of embankment due to the clogging of a drain pipe crossing each embankment.

A nearly total loss of embankment

- **Six full sections of embankment were reconstructed to geogrid-reinforced slopes supported by GRS-RWs having a FHR facing,**
  - **** to reduce the amount of earthwork;***
  - **** to stabilize the embankment slopes; and***
  - **** to install large diameter drainage pipes.***



A large-diameter corrugated steel drainage pipe.

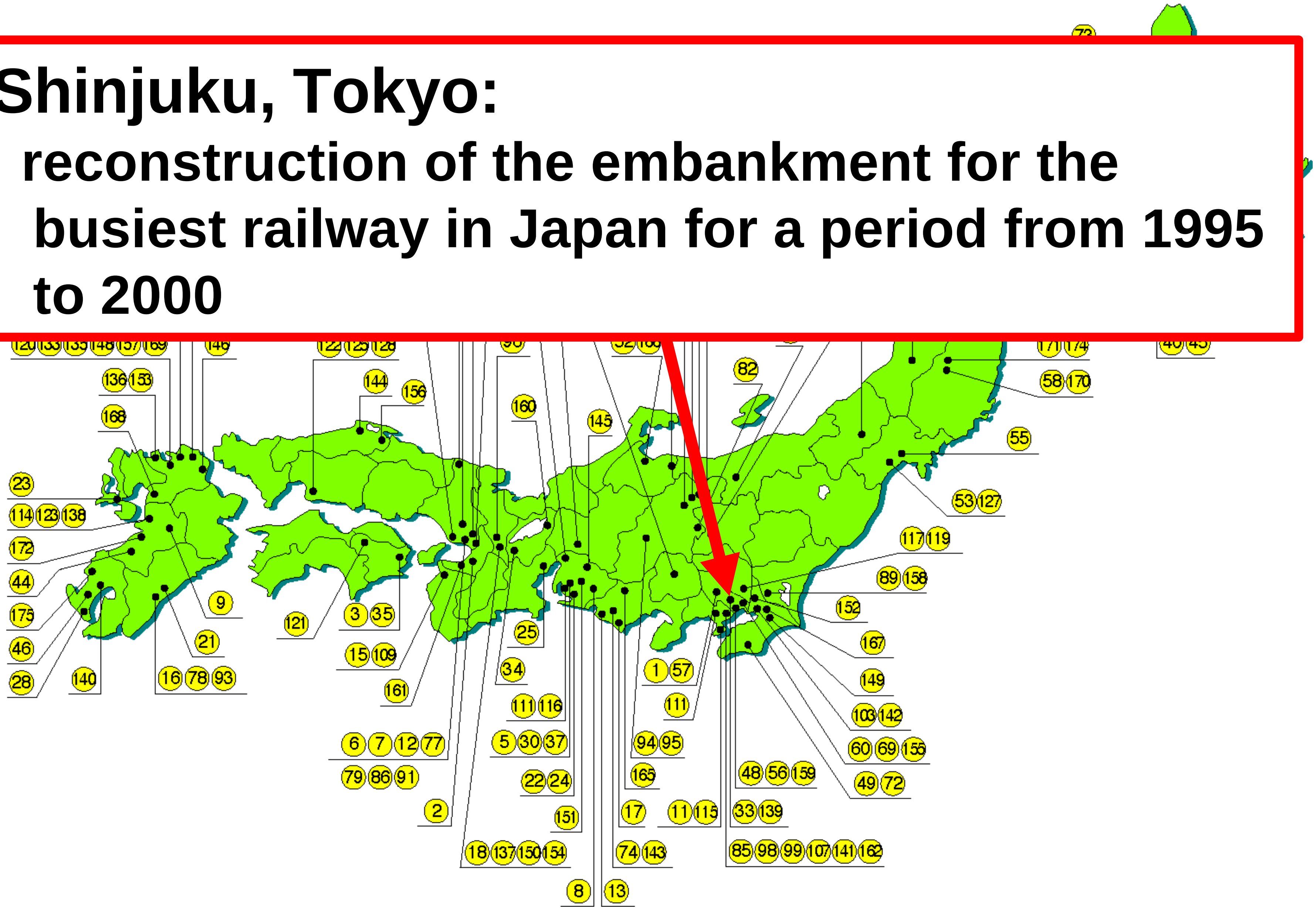
Completed walls and slopes

1991

1996

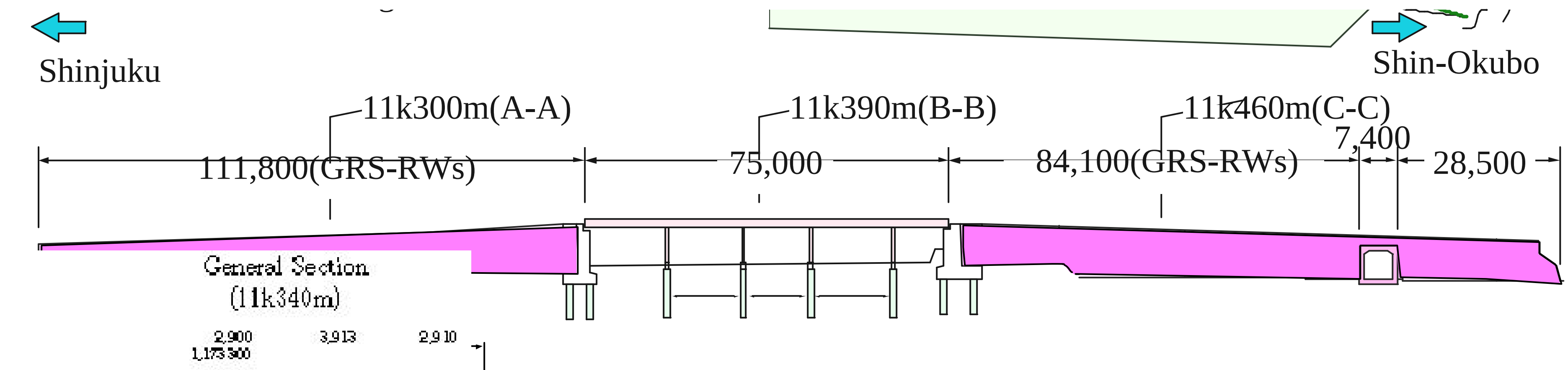
Shinjuku, Tokyo:

reconstruction of the embankment for the busiest railway in Japan for a period from 1995 to 2000



Sites of GRS RWs with a full-height rigid facing that have been constructed by the end of April 2000

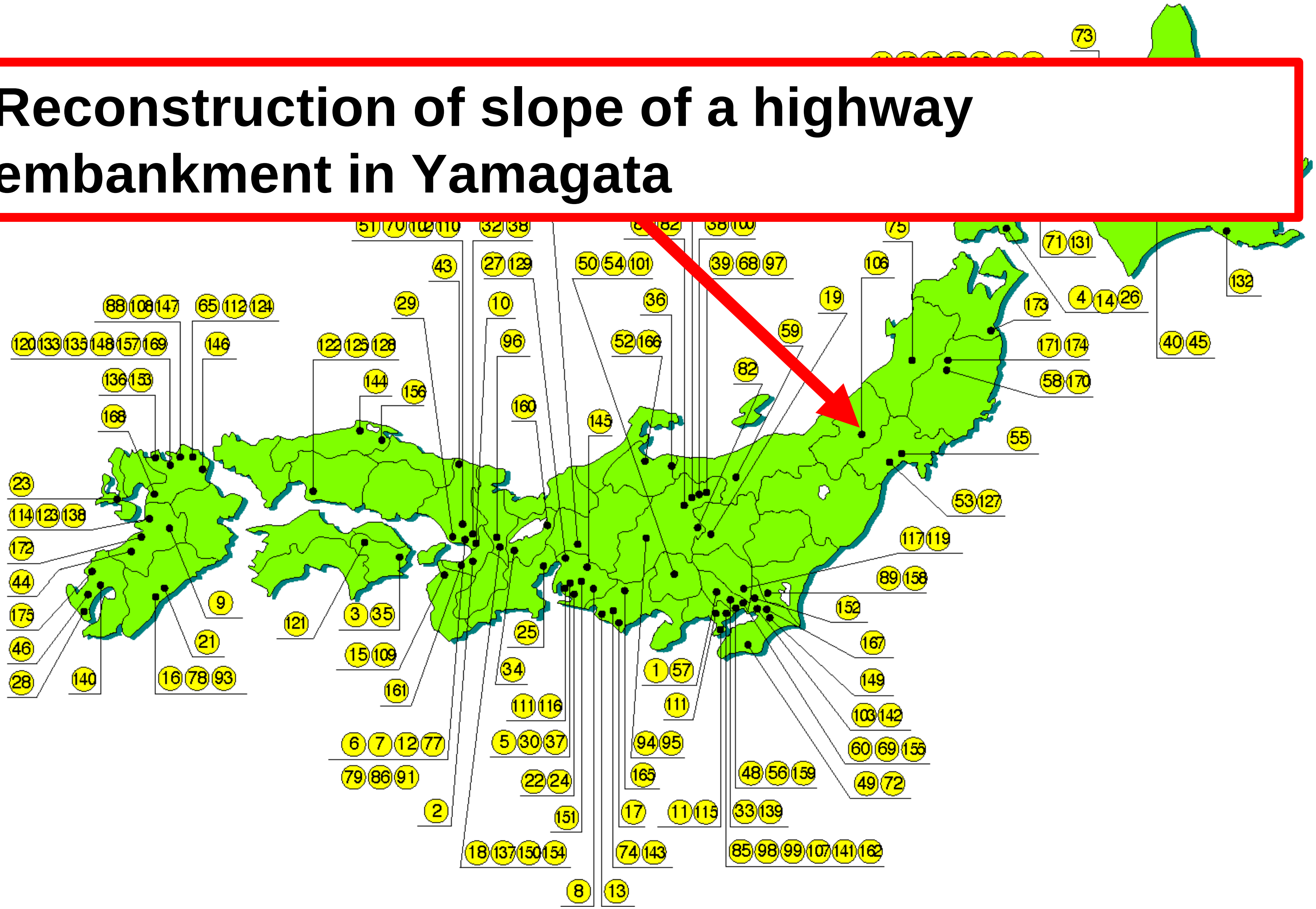
Reconstruction of an old bridge and associated relocation of two railway tracks for the busiest and most important rapid transits in Japan, Chuo and Yamanote Lines.



Wall under construction

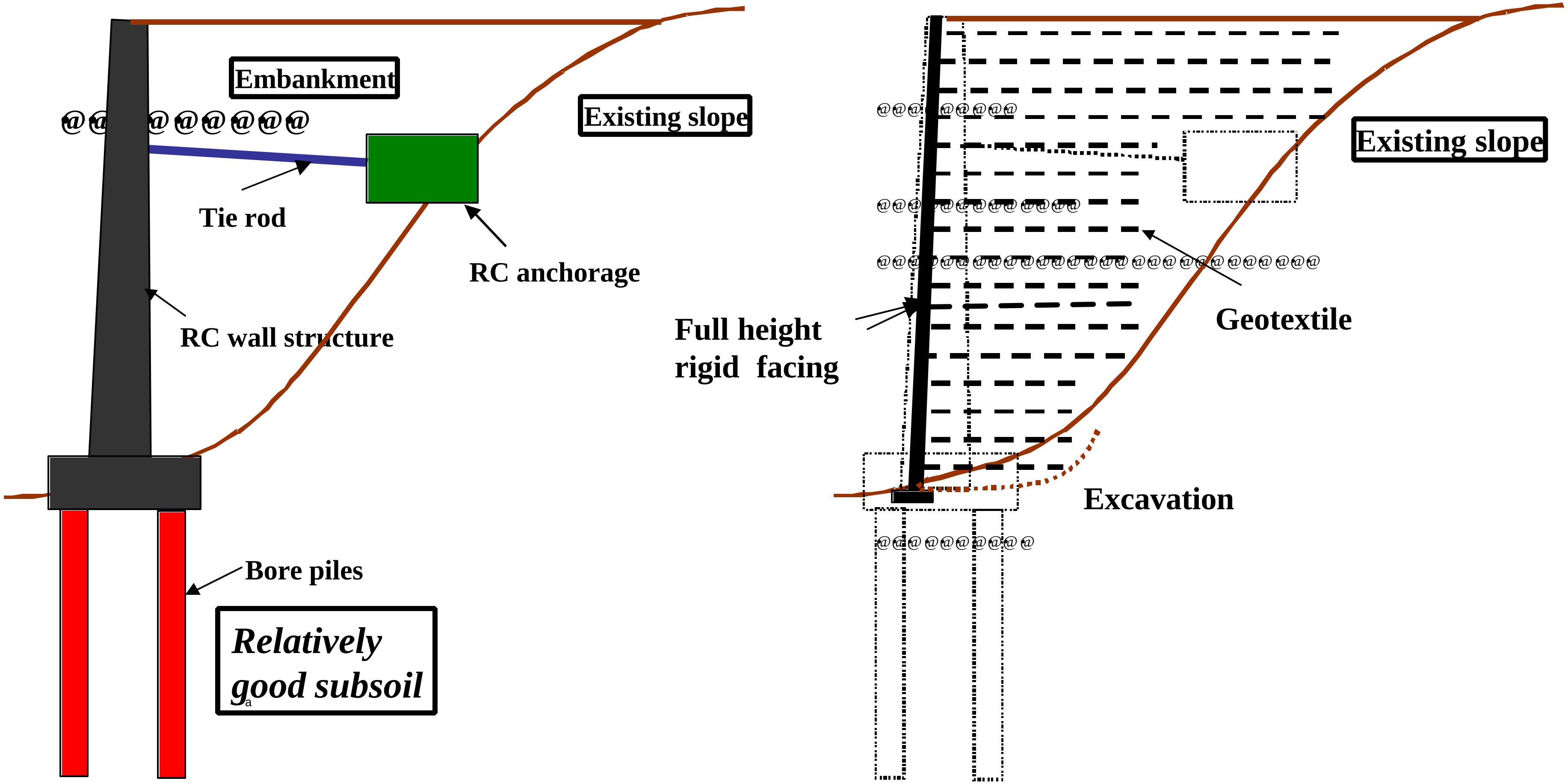
Completed wall

Reconstruction of slope of a highway embankment in Yamagata



Sites of GRS RWs with a full-height rigid facing that have been constructed by the end of April 2000

A case history showing a sharp contrast between conventional cantilever walls and GRS RWs with a FHR facing




Existing cantilever RC RW
Reconstruction
to a GRS RW
with a FHR facing

Reconstruction to a GRS RW with a FHR facing

Summary

1) **GRS-RWs having a FHR facing have been constructed by the staged construction procedure as important permanent soil RWs and bridge abutments for the last two decades in Japan.**

The walls have been mainly for railways so far, but recently many cases also for highway.

2) This success is due mainly to;

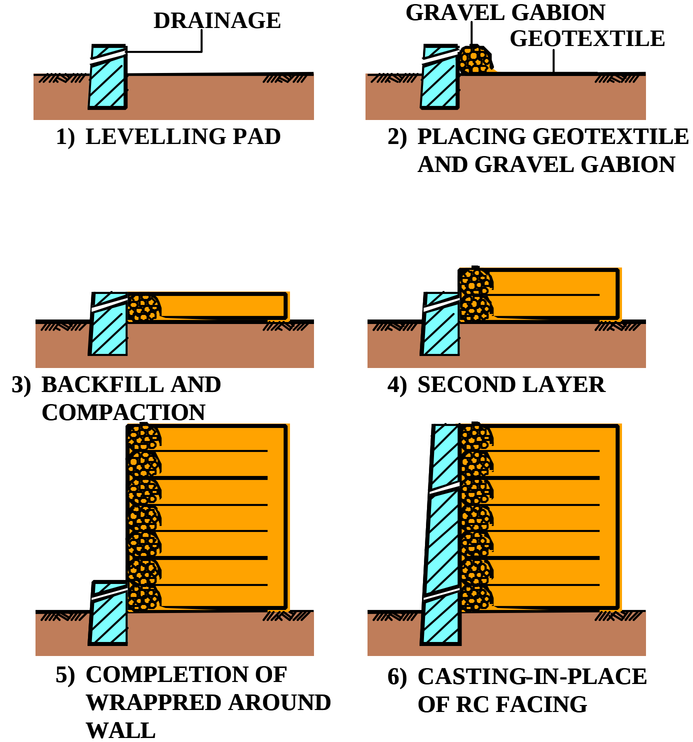
* its high cost-effectiveness; and
* its high performance,

which is equivalent to, or even better than, that of other modern RC retaining walls and RC bridge abutments supported by piles.

3) Some of the keys are as follows;

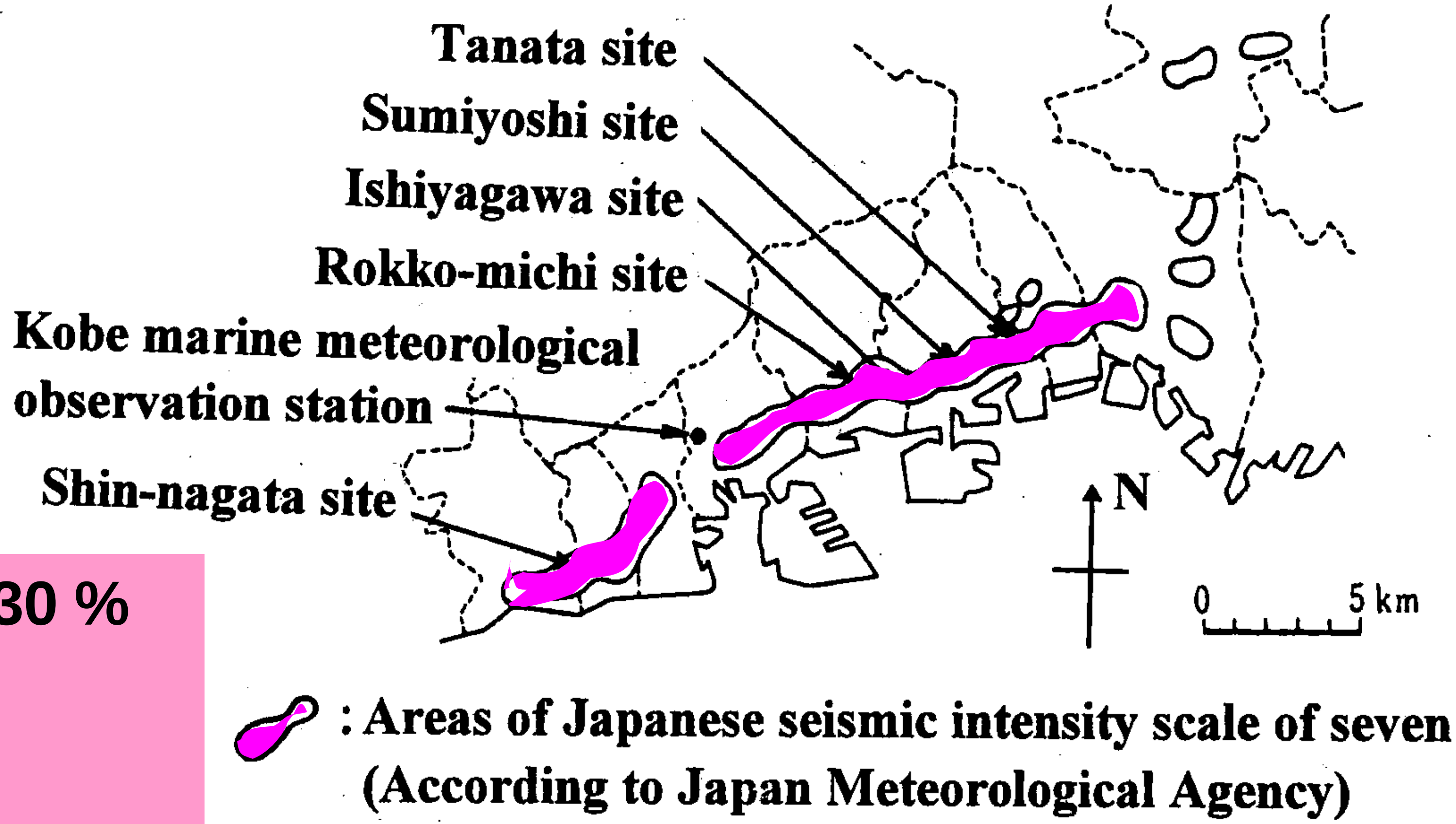
a) the use of a proper type of geosynthetic; grid for cohesionless soils; and nonwoven/woven geotextile composite for nearly-saturated cohesive soils; and

b) the use of a full-height rigid facing that is cast-in-place by a staged construction procedure.



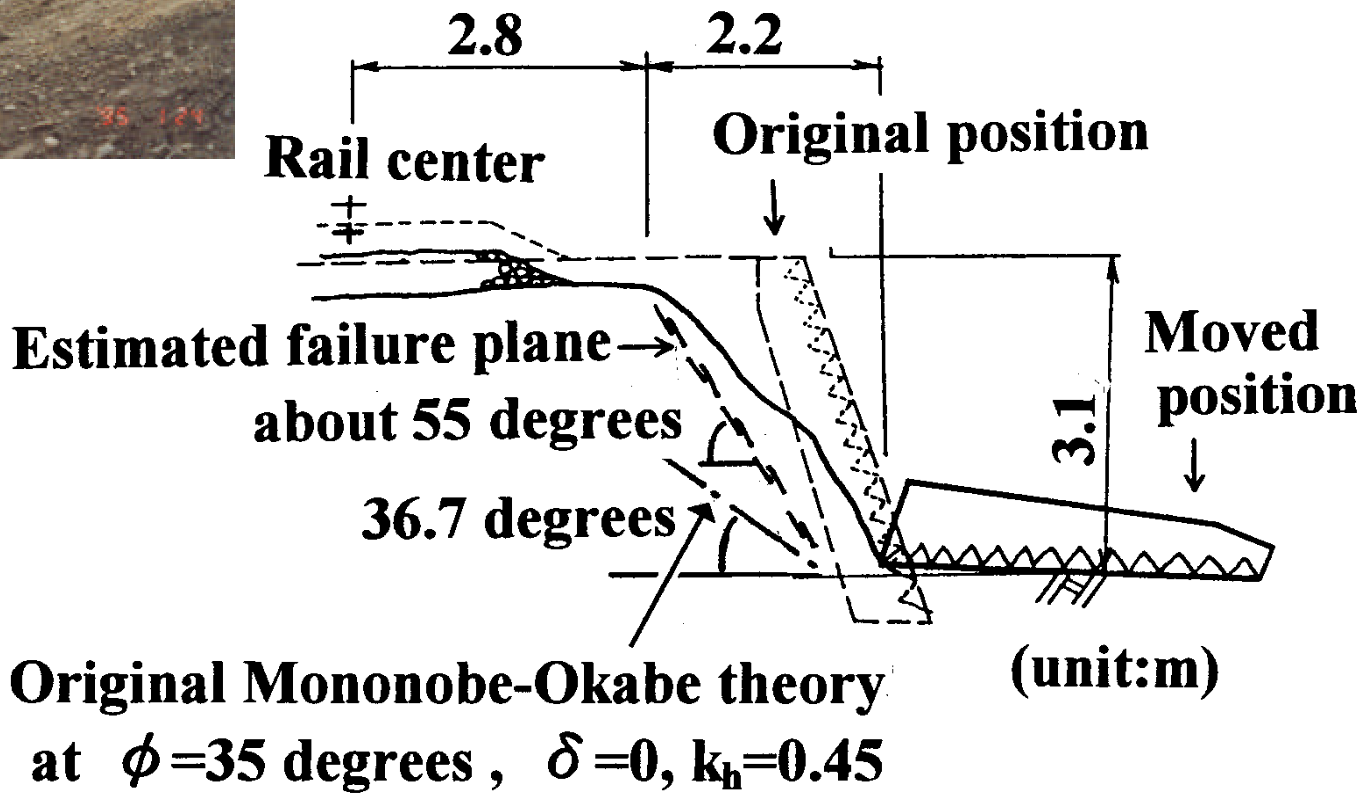
Contents

Lessons from the 1995 Hyogo-ken Nambu (Kobe) Earthquake

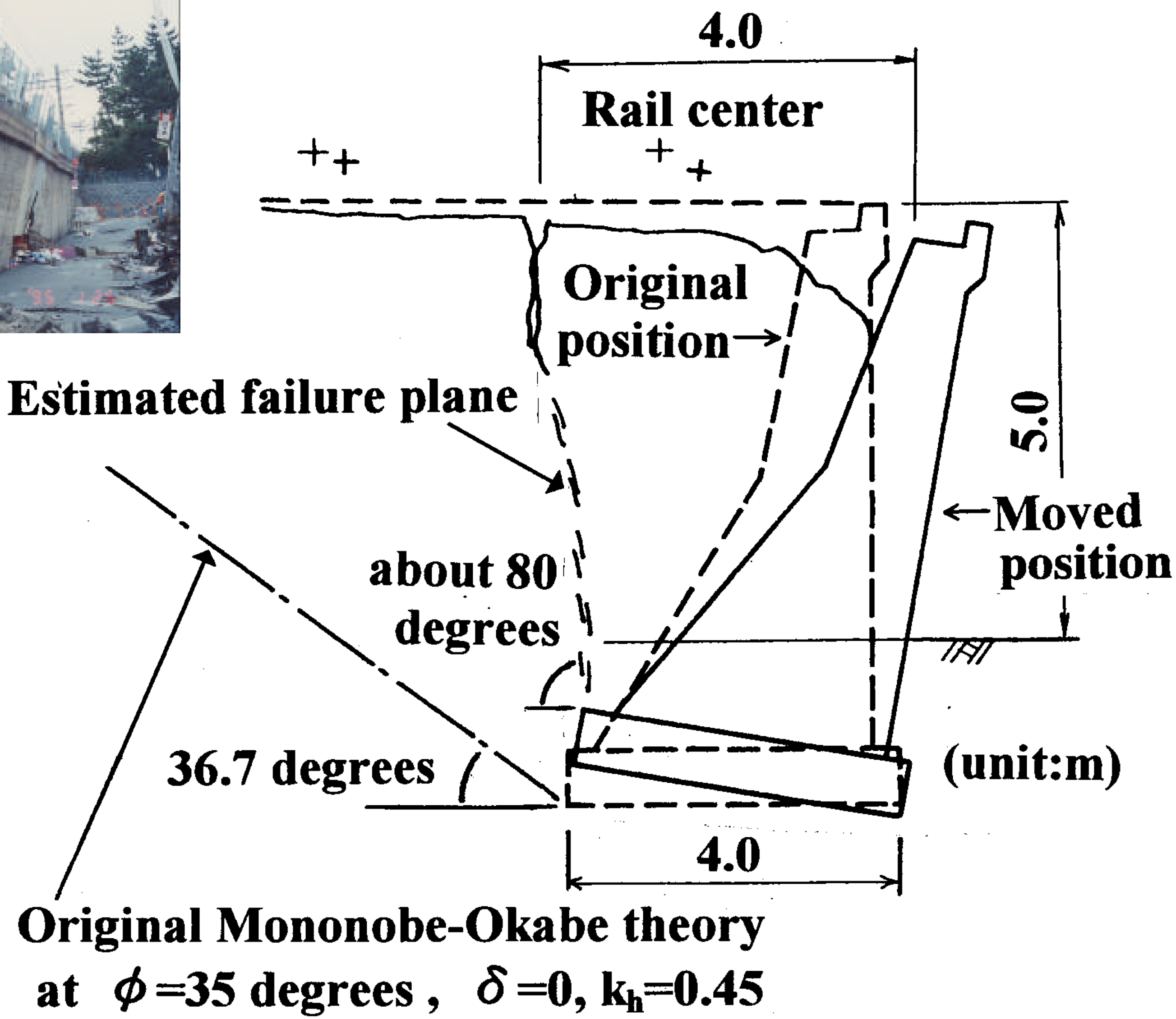


More than 30 % of wooden houses collapsed.

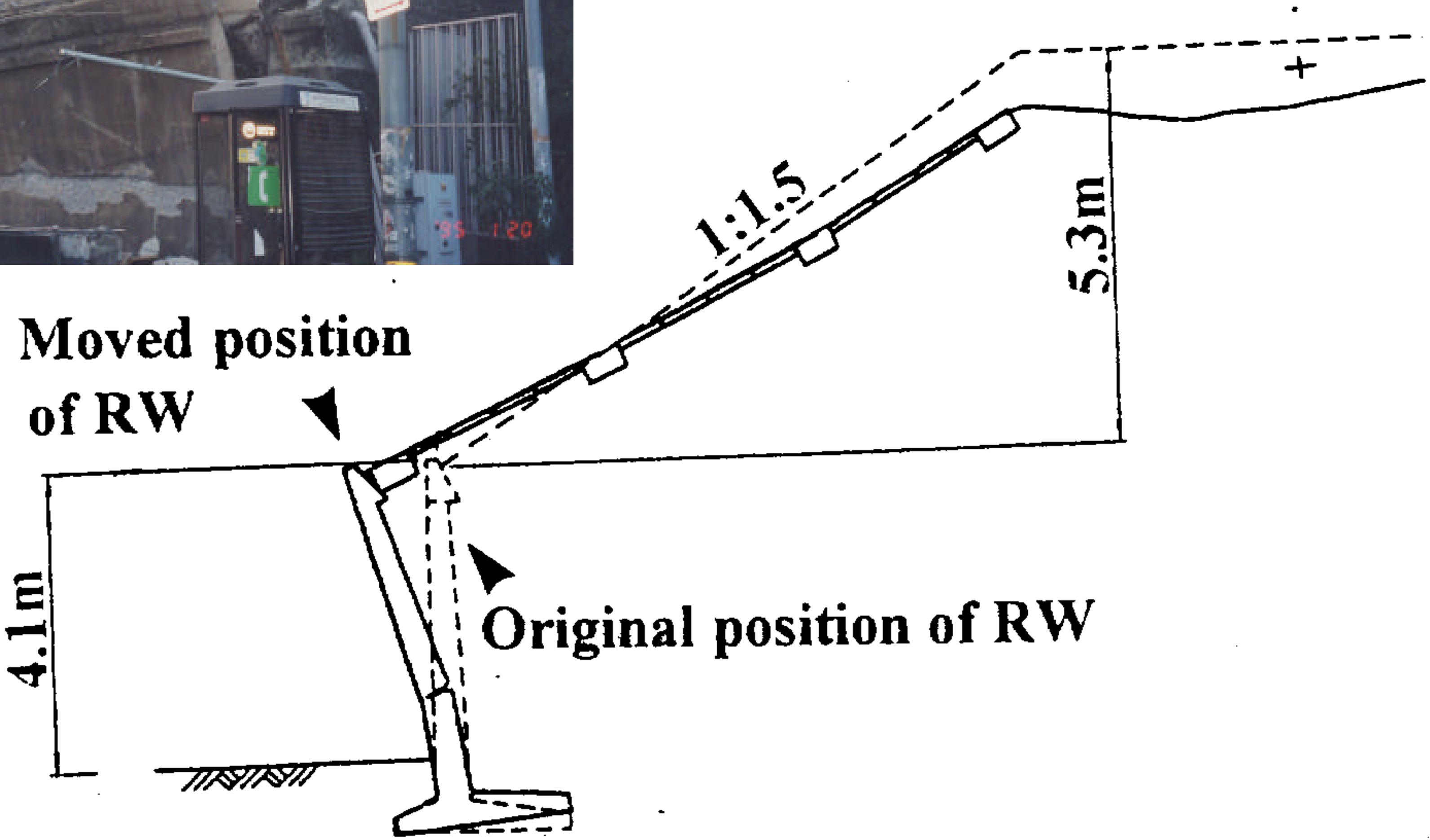
Locations of representative retaining walls damaged during the 1995 Hyogoken-Nanbu earthquake



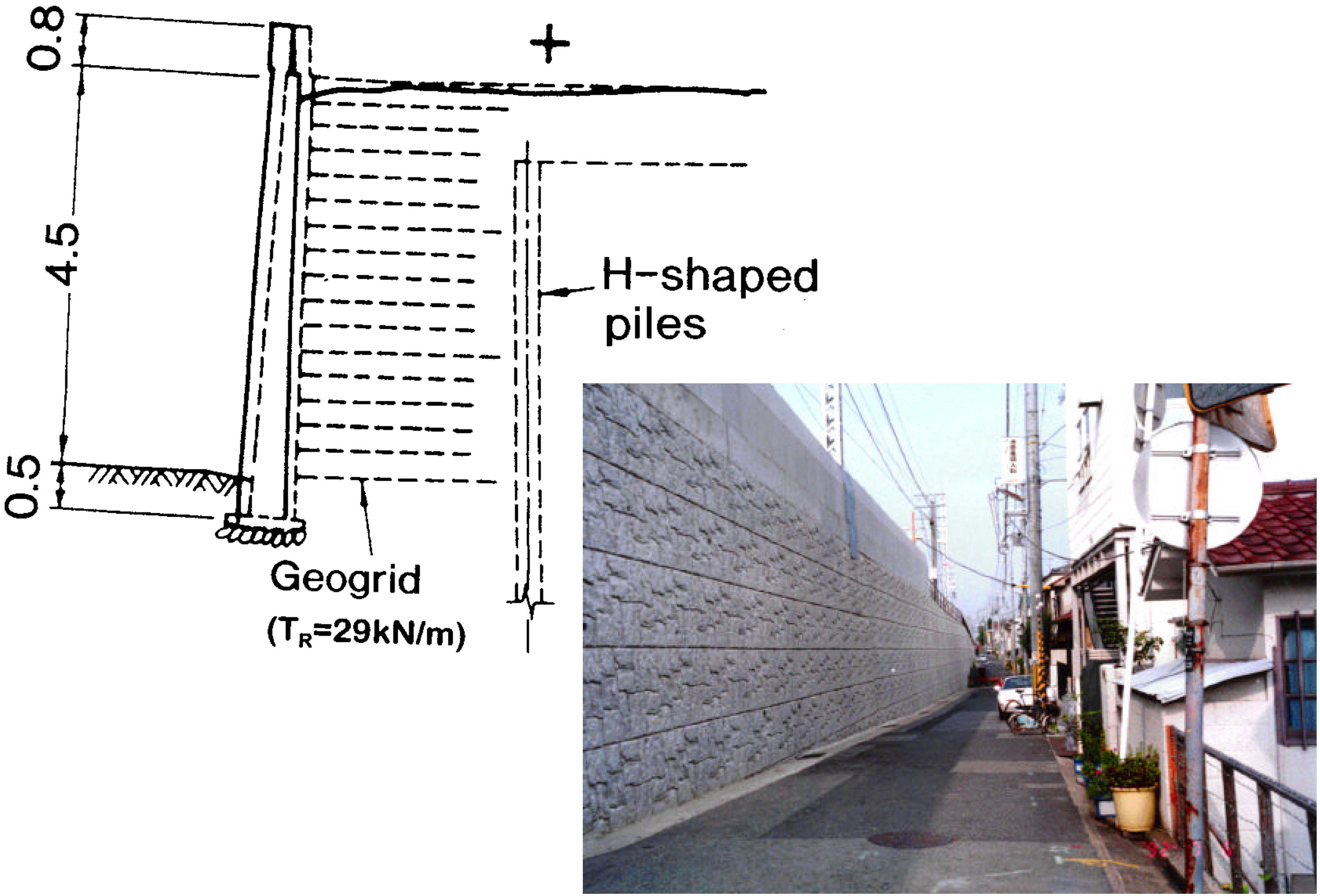
Leaning (gravity) type wall at Sumiyoshi site



Gravity type wall at Ishiyagawa site



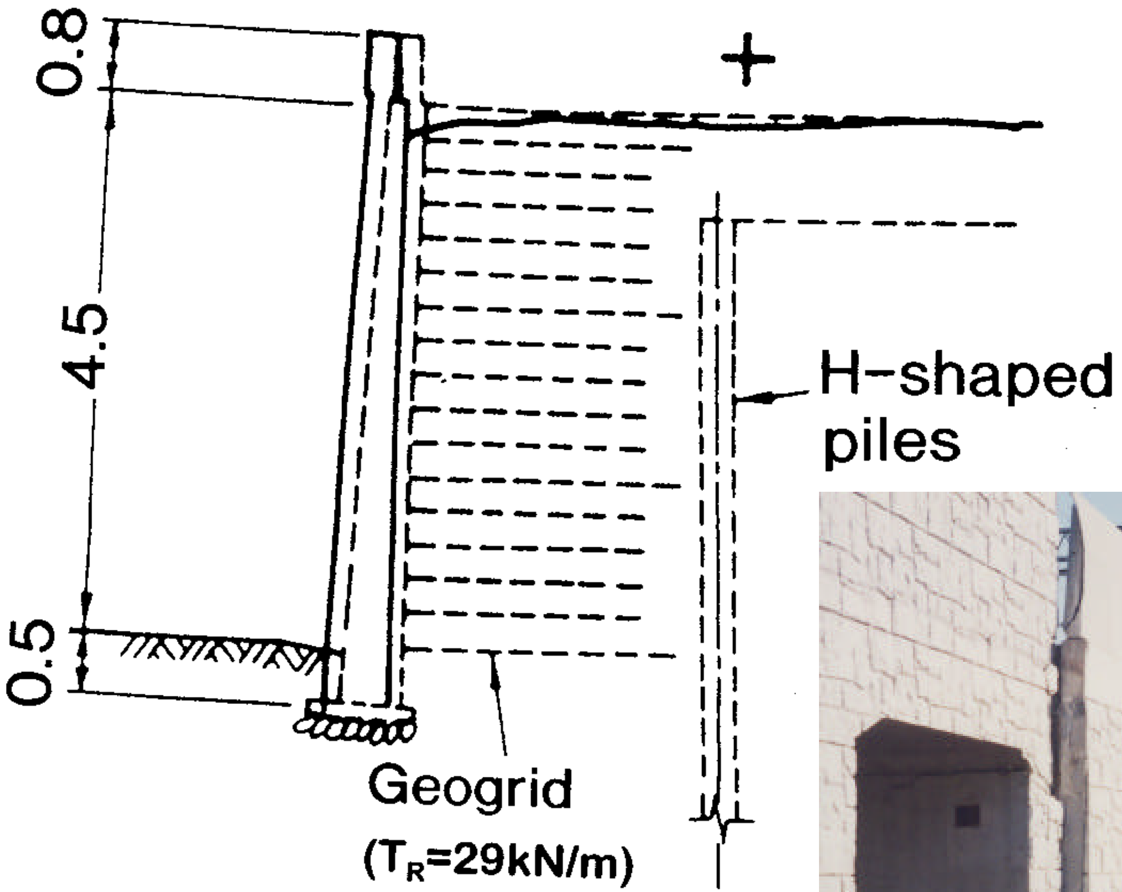
Cantilever type RC wall at Shin-nagata site



Reinforced-soil RW having a full-height facing at Tanata site

Immediately after completion, 1992

Immediately after the 1995 Kobe Earthquake



The wall moved, but did not collapsed !

The wall did not collapsed, but moved !

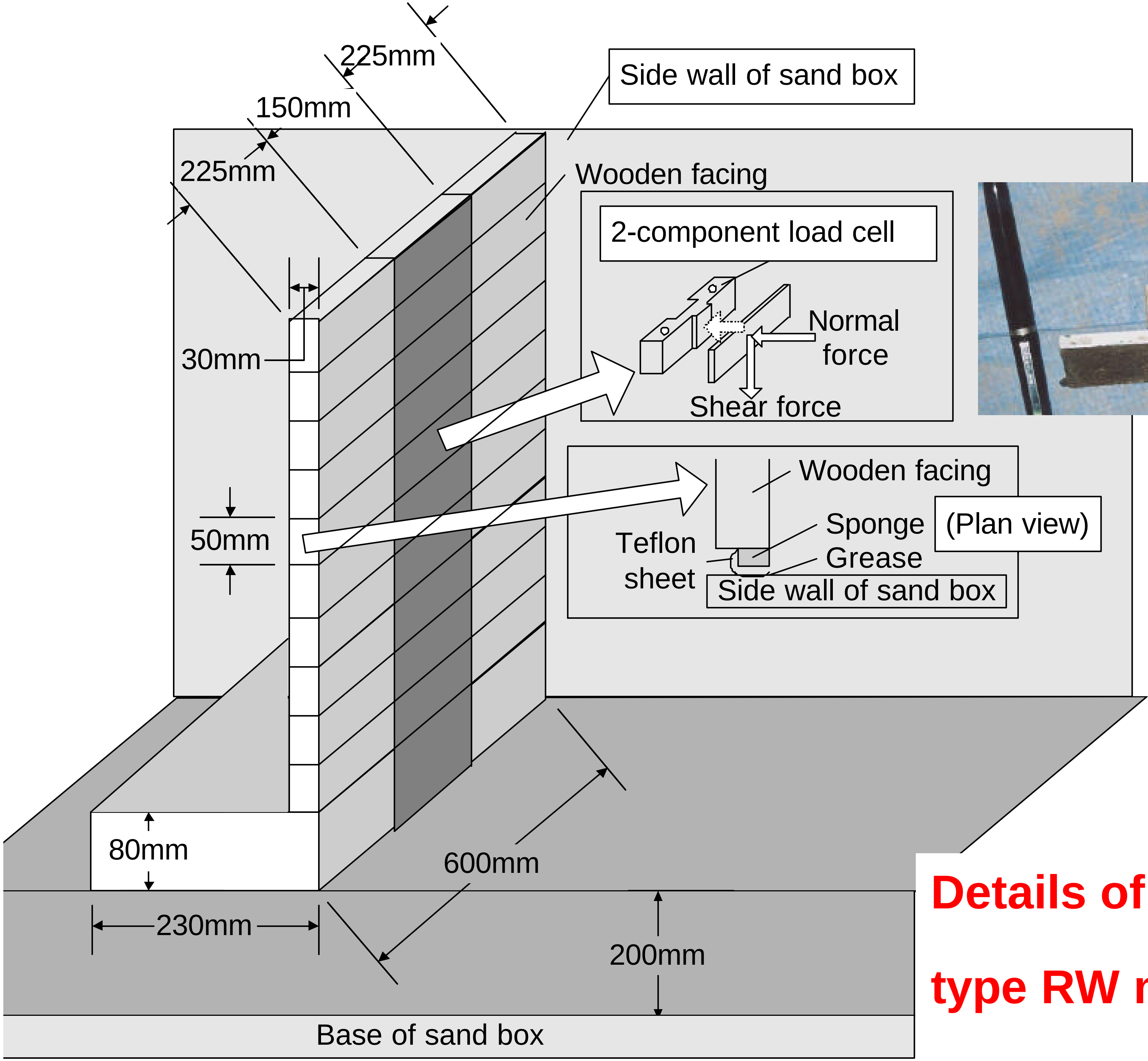
Summary 2-1

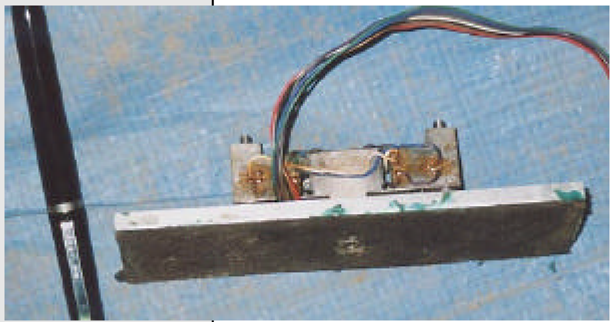
2-1 During the 1995 Hyogoken-Nanbu earthquake, gravity type RWs were severely damaged.

2-2 Cantilever type RWs were moderately damaged.

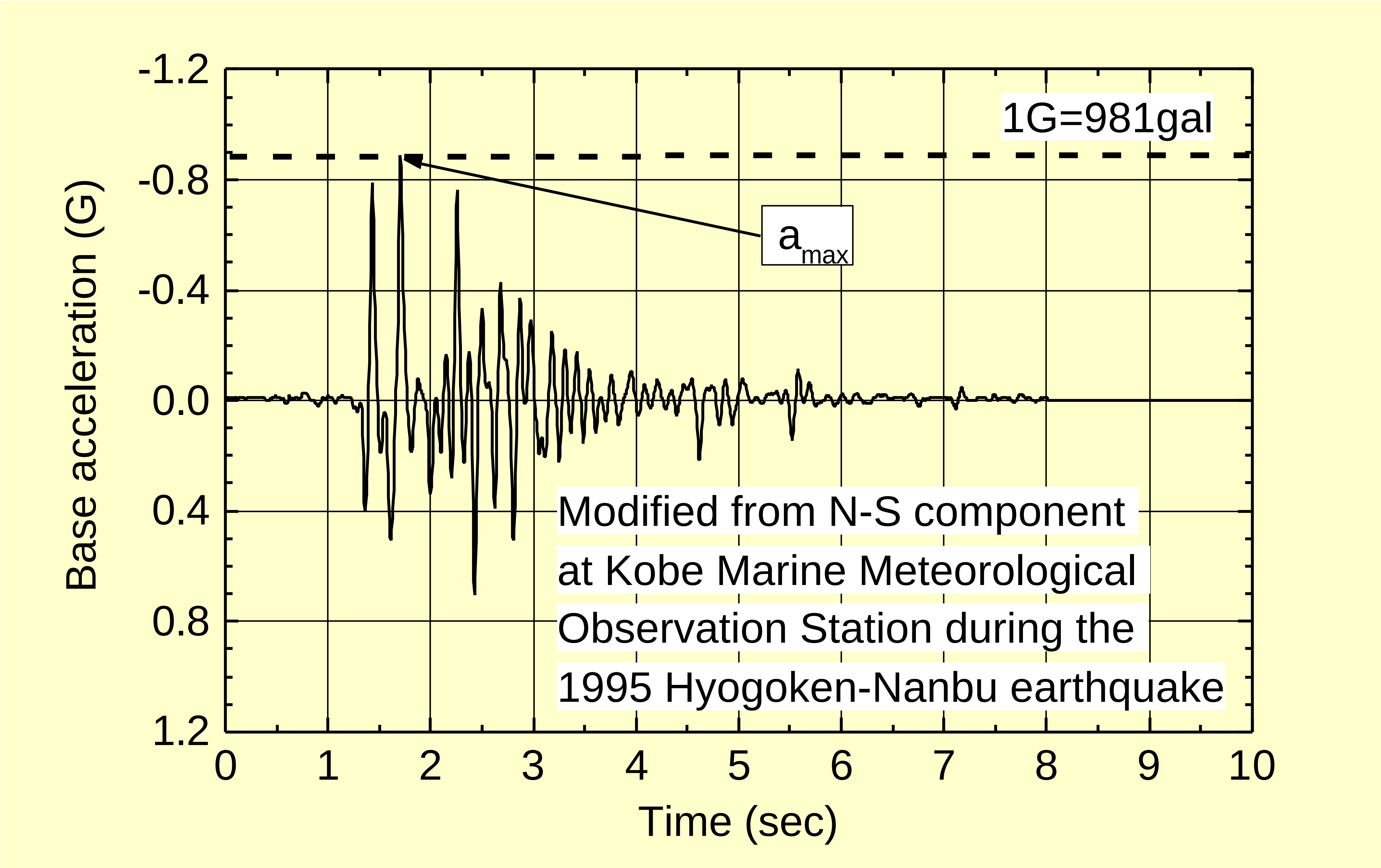
2-3 On the other hand, geosynthetic-reinforced-soil RWs with a full-height rigid facing performed very well and were reused with minor modifications after the earthquake.

? Model shaking table tests to confirm the above.



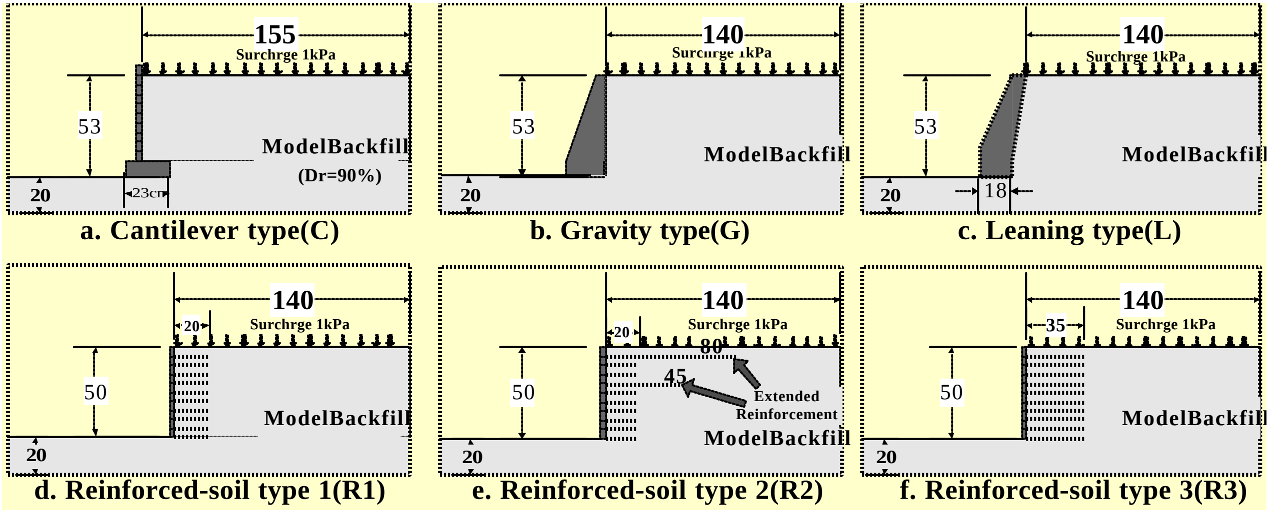


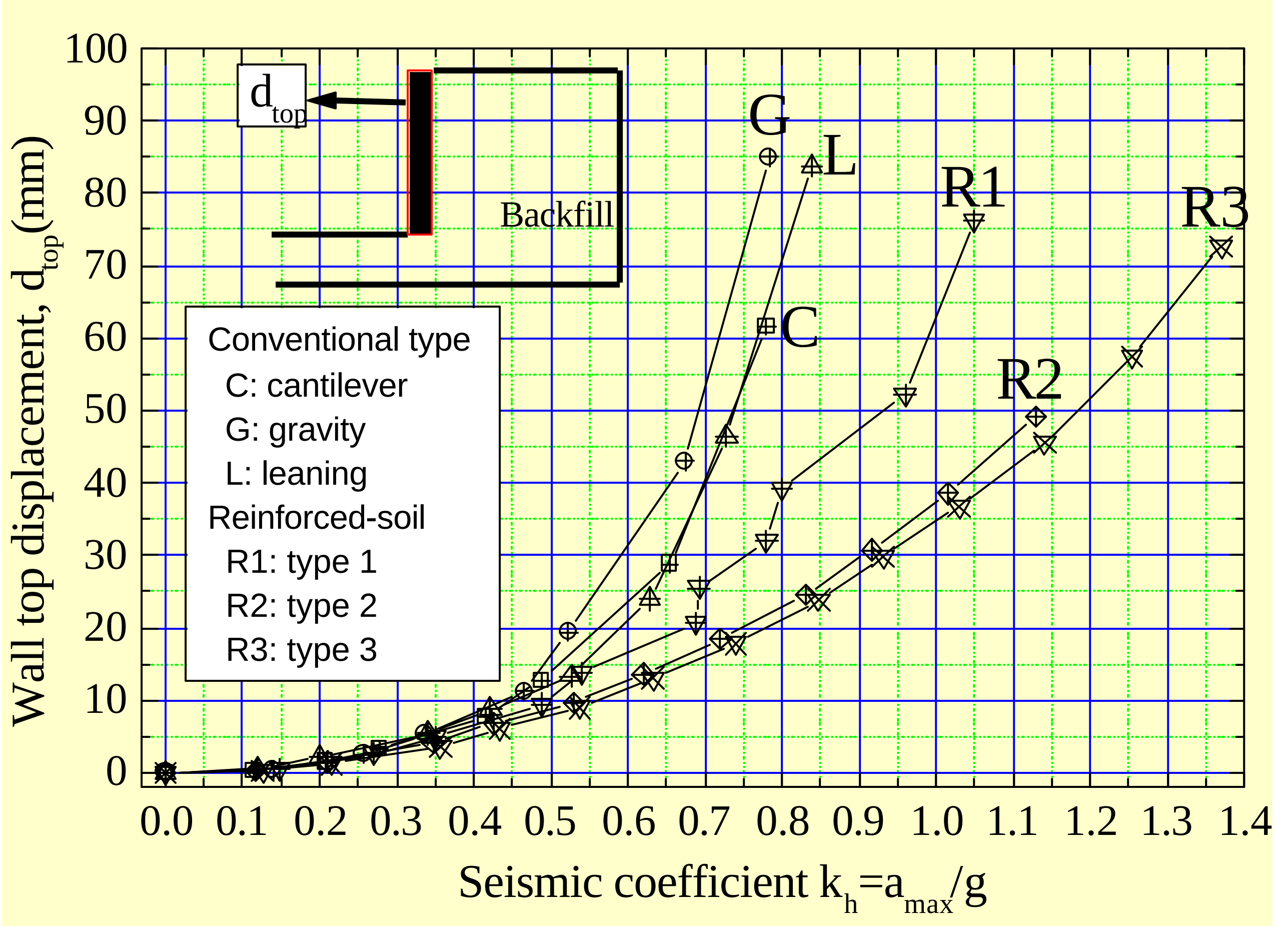
Details of gravity type RW model



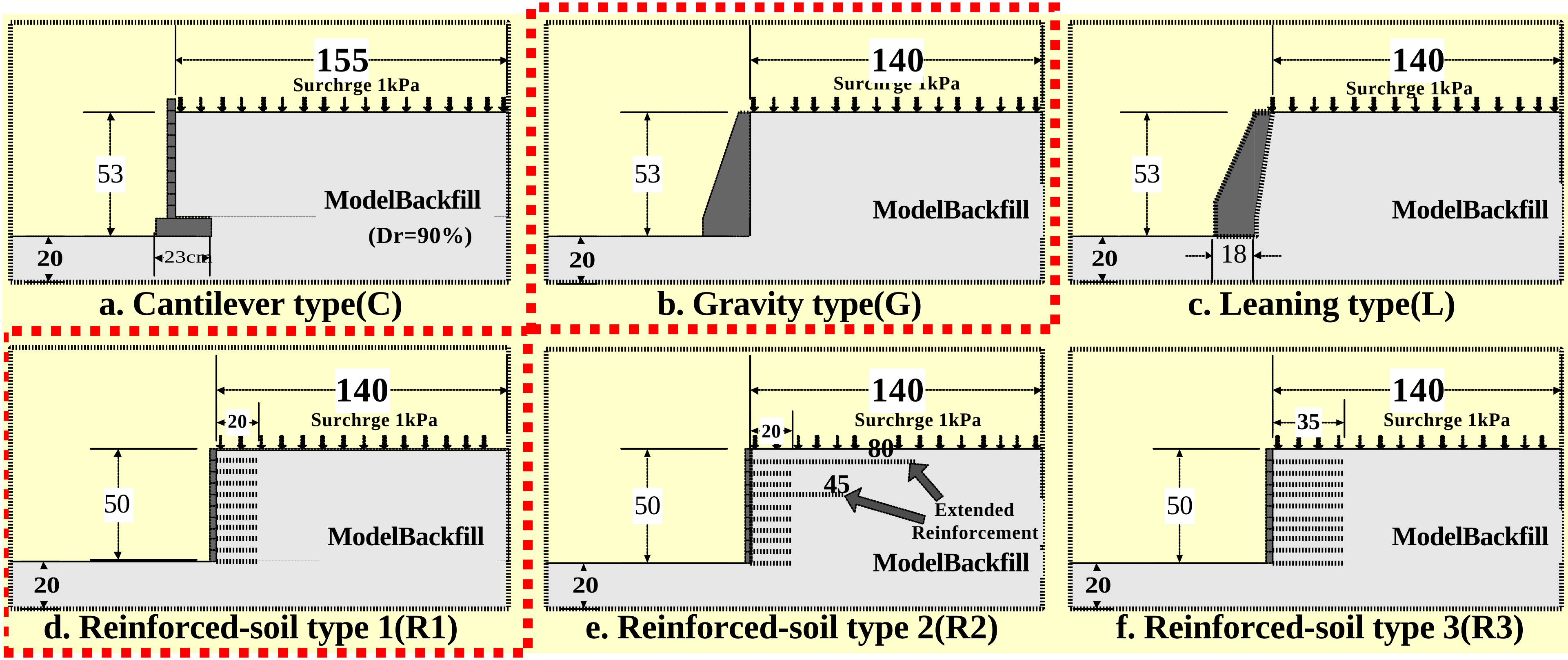
Several shaking steps; a_{max} increased from 0.1G at an increment of 0.1G.

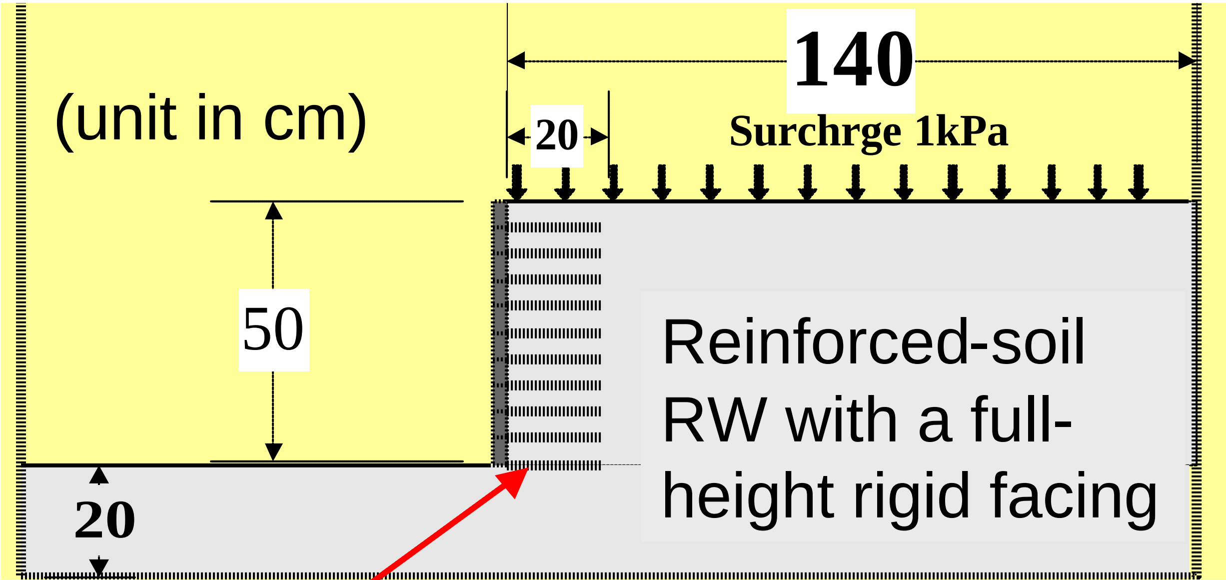
Typical time history of base acceleration


155
Surchrge 1kPa
53
ModelBackfill
(Dr=90%)
20
23cm
a. Cantilever type(C)
140
Surchrge 1kPa
53
ModelBackfill
20
b. Gravity type(G)
140
Surchrge 1kPa
53
ModelBackfill
20
18
c. Leaning type(L)
140
20
Surchrge 1kPa
50
ModelBackfill
20
d. Reinforced-soil type 1(R1)
140
20
Surchrge 1kPa
80
45
Extended
Reinforcement
50
ModelBackfill
20
e. Reinforced-soil type 2(R2)
140
35
Surchrge 1kPa
50
ModelBackfill
20
f. Reinforced-soil type 3(R3)

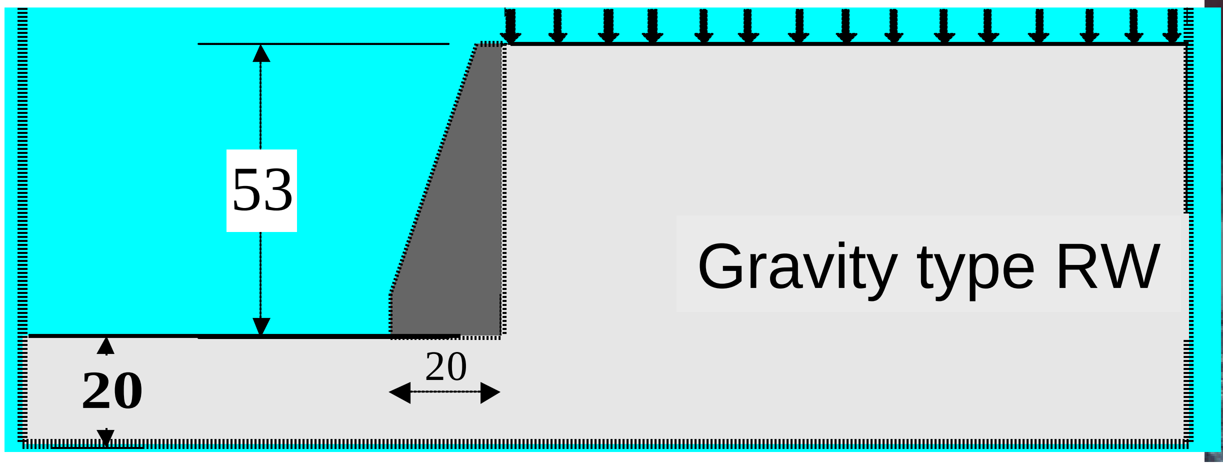

Wall top displacement, dtop(mm)
Seismic coefficient kh=amax/g
dtop
Backfill
G
L
C
R1
R2
R3
Conventional type
C: cantilever
G: gravity
L: leaning
Reinforced-soil
R1: type 1
R2: type 2
R3: type 3

Model shaking table tests (1g)





Reinforcement (10 layers):
grid of phosphor-bronze strips (t=0.2mm)

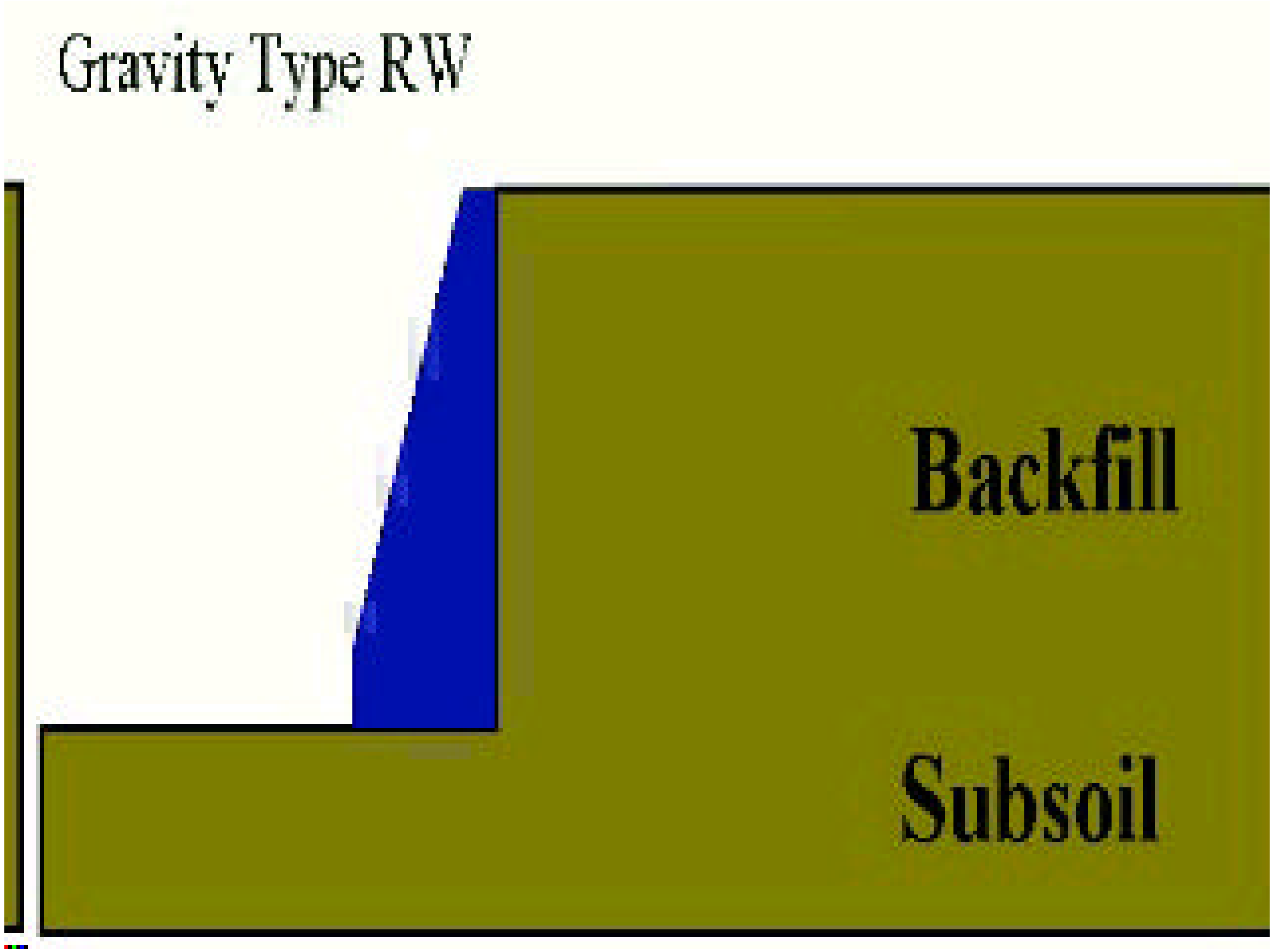


Backfill and subsoil layers:
dry Toyoura sand (D_r=88%)

Performance of retaining walls
(RWs) during 1995 Kobe Eq.

Observed behaviour of gravity type wall

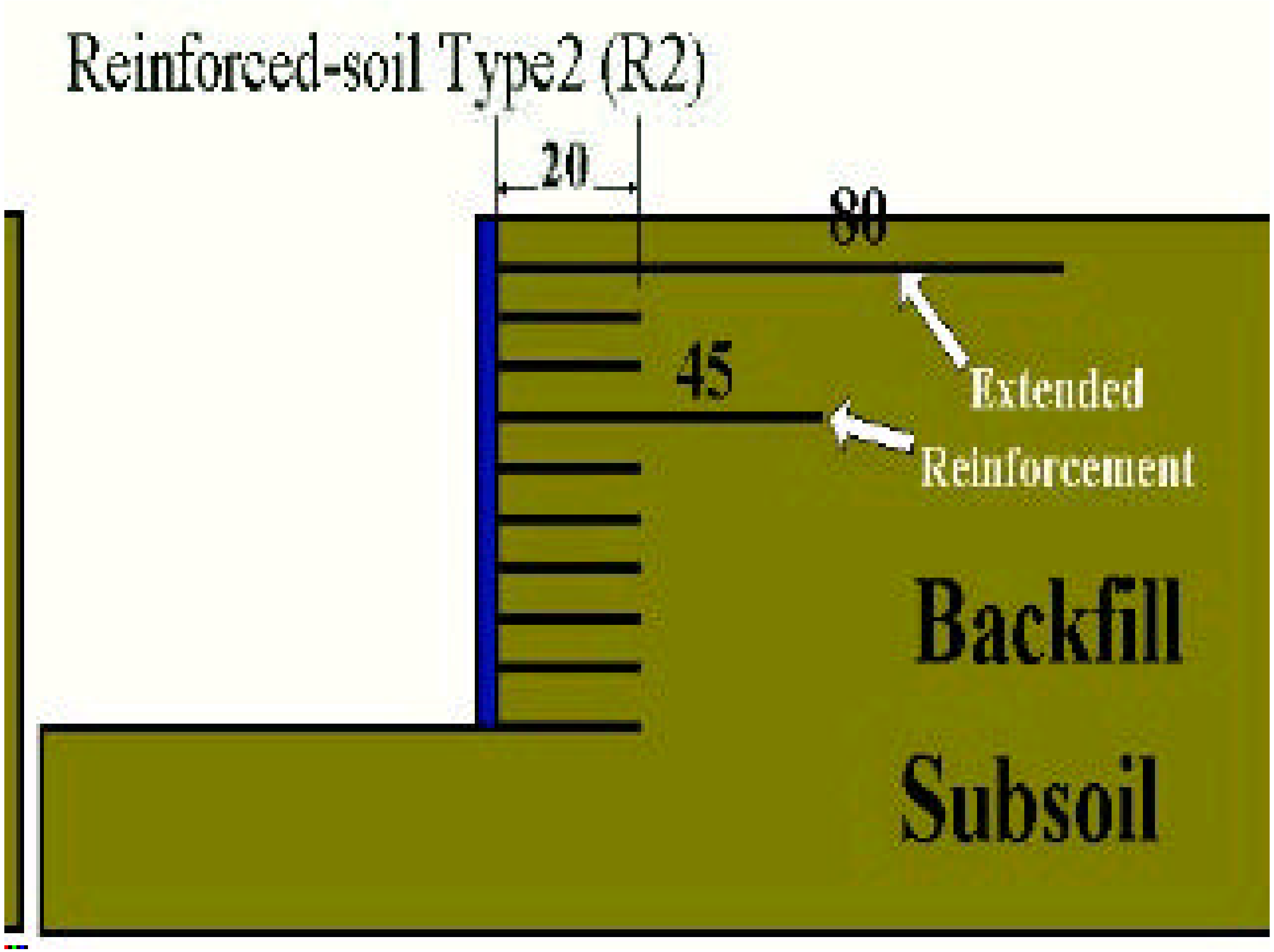
a_{max}: 919gal

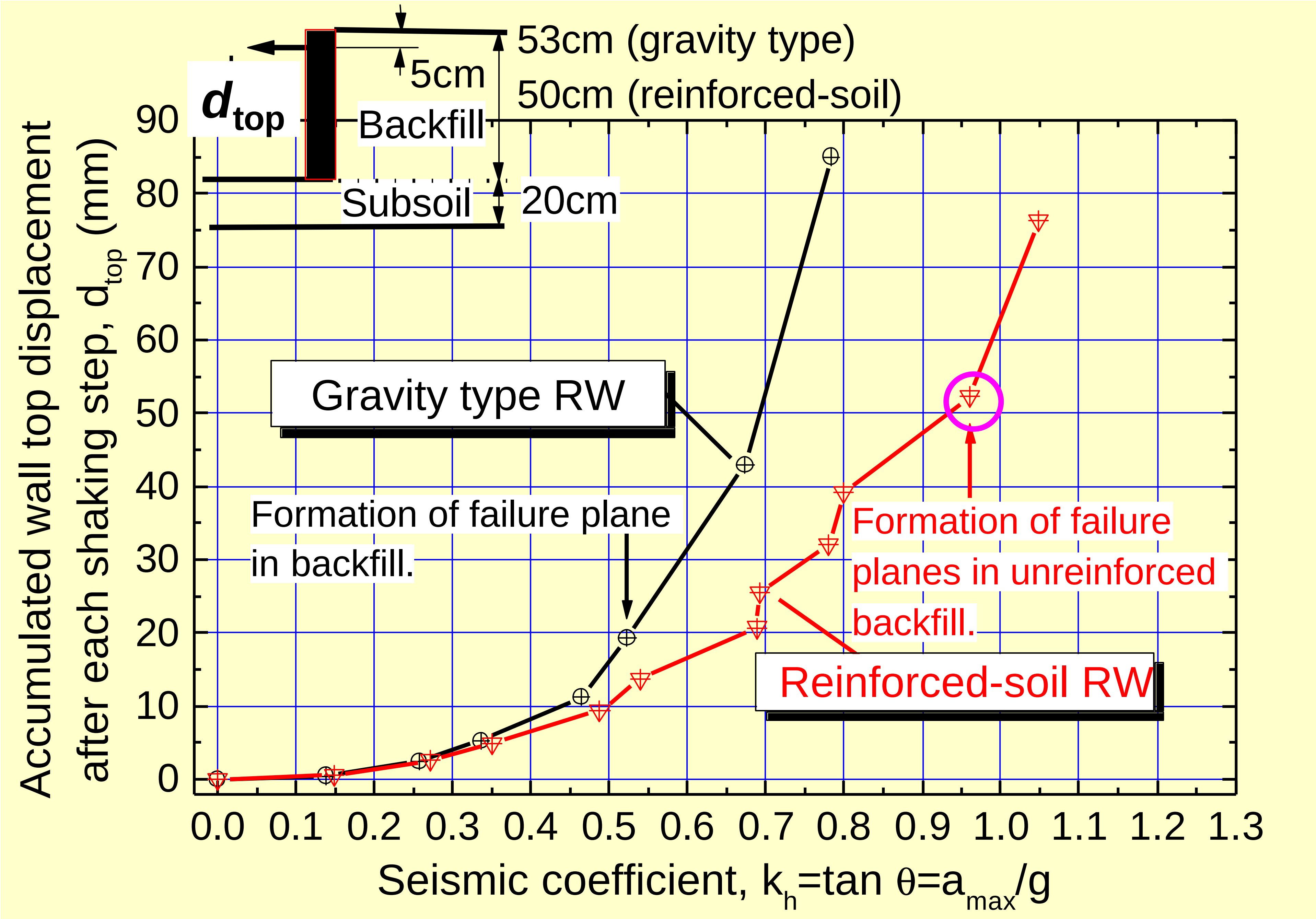


Observed behaviour of reinforced-soil wall

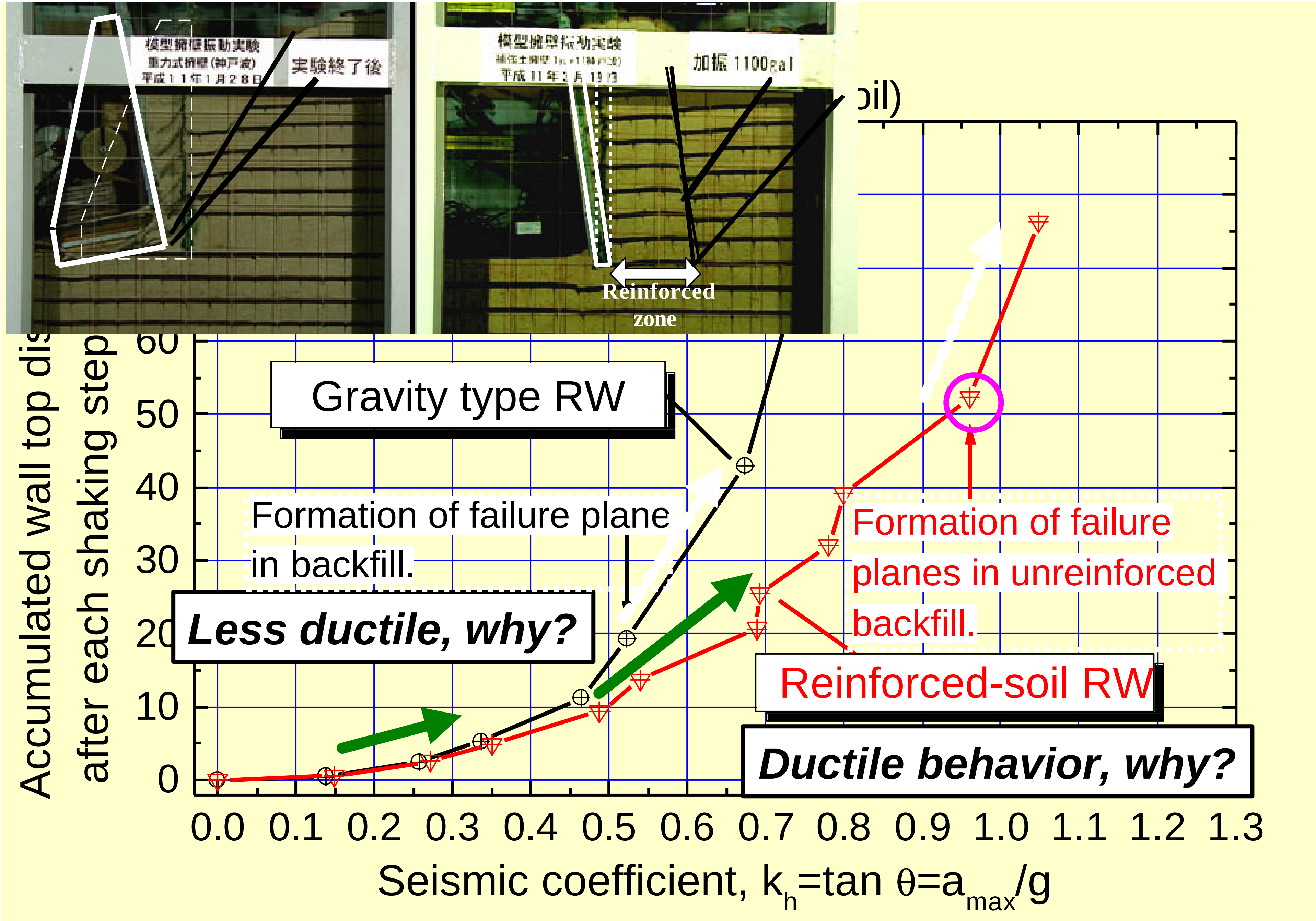
a_{max}:

1,016 gal



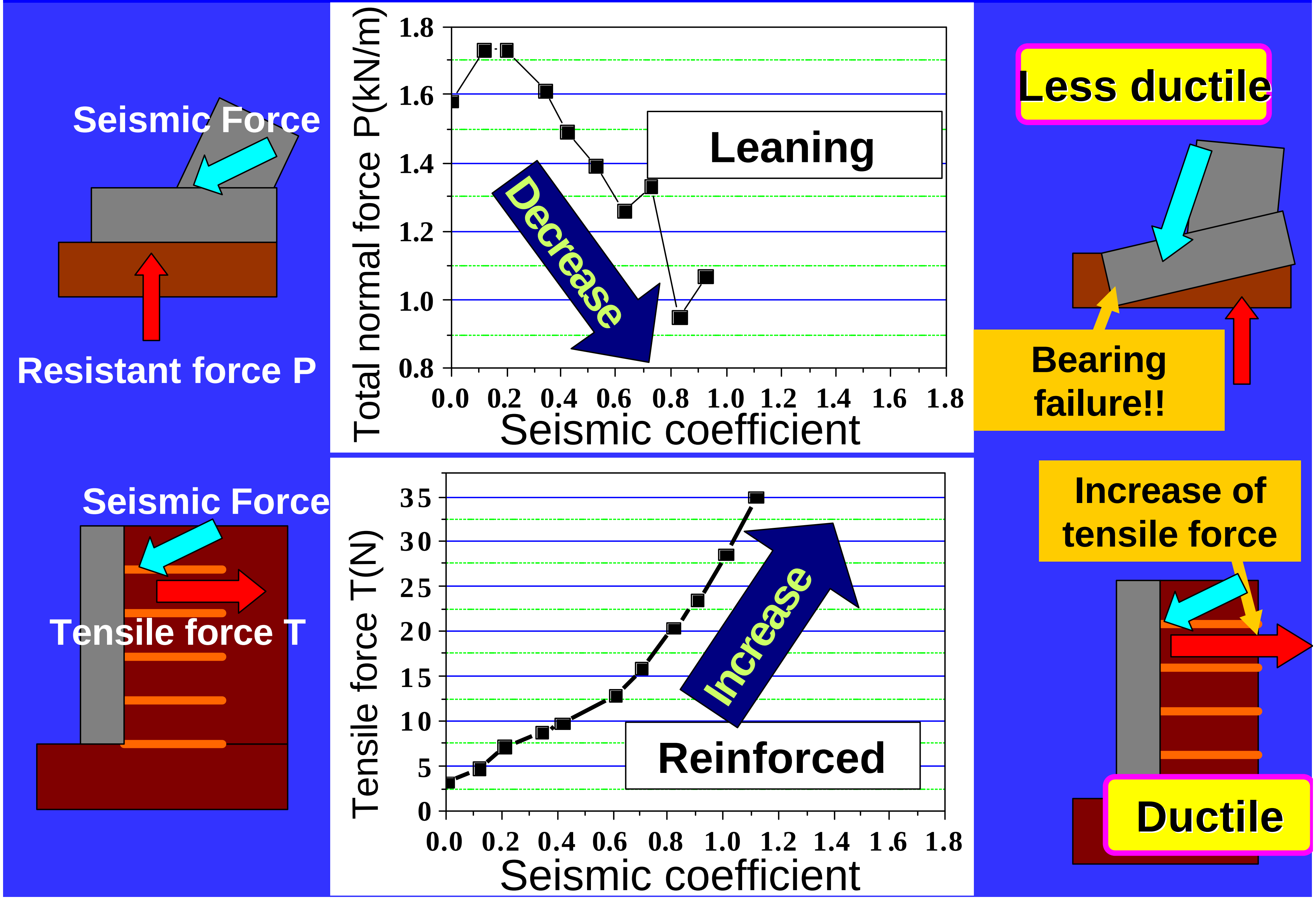


Accumulation of residual wall top displacement, d_{top}



Accumulation of residual wall top displacement, d_{top}

Resistant mechanism



Summary 2-2

2-4 In model tests on level ground, reinforced-soil RWs with a full-height rigid facing showed much more ductile behavior than conventional type RWs.

2-5 Concentration of subgrade reactions at the toe of conventional type walls resulted into a local failure in the subsoil, leading to sudden loss of bearing capacity, thereby brittle behaviour. On the other hand, tensile forces in reinforcement of reinforced-soil RWs could be mobilized effectively to resist against the wall movement in a ductile manner.

2-6 These responses explain the different extents of damage depending on the wall types during the 1995 Hyogoken-Nanbu earthquake.

Contents

Topics:

3-1 Observed failure pattern of backfill soil

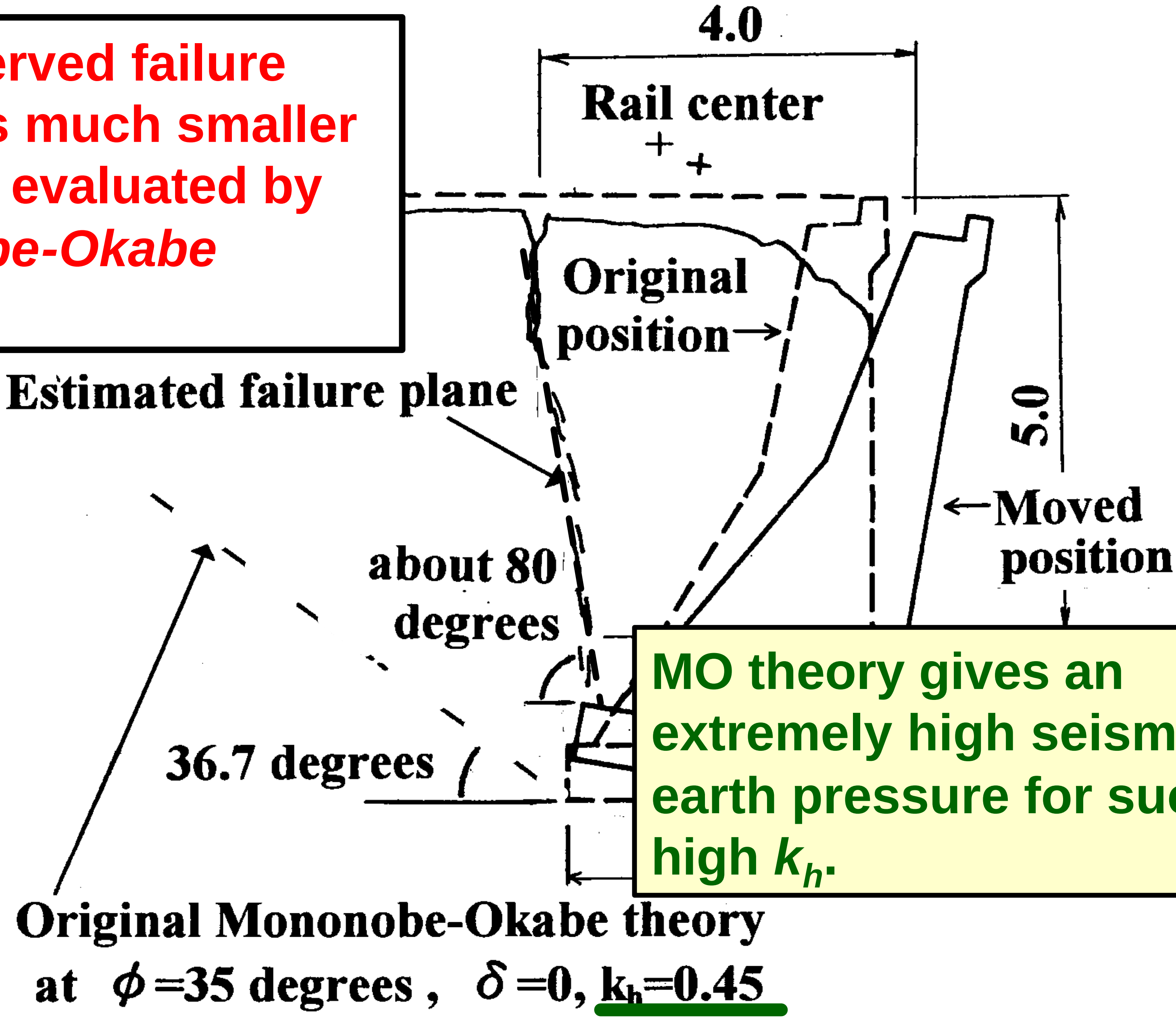
3-2 Modification of *Mononobe-Okabe* method

3-2 Comparison with model test results

A gravity type RW damaged by the 1995 Hyogoken-Nanbu (Kobe) earthquake

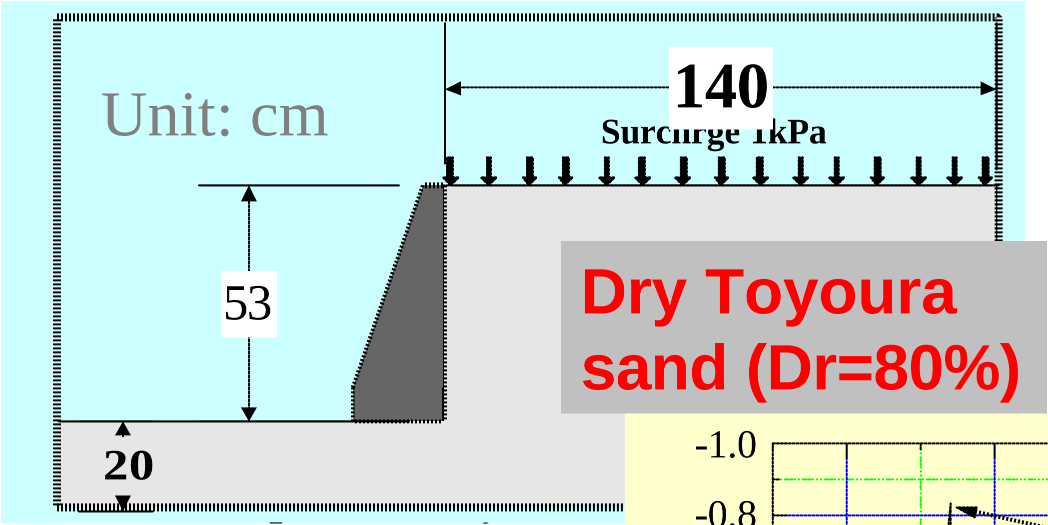
Inconsistency between MO theory and actual seismic behaviour of RW

The observed failure zone was much smaller than that evaluated by *Mononobe-Okabe* theory

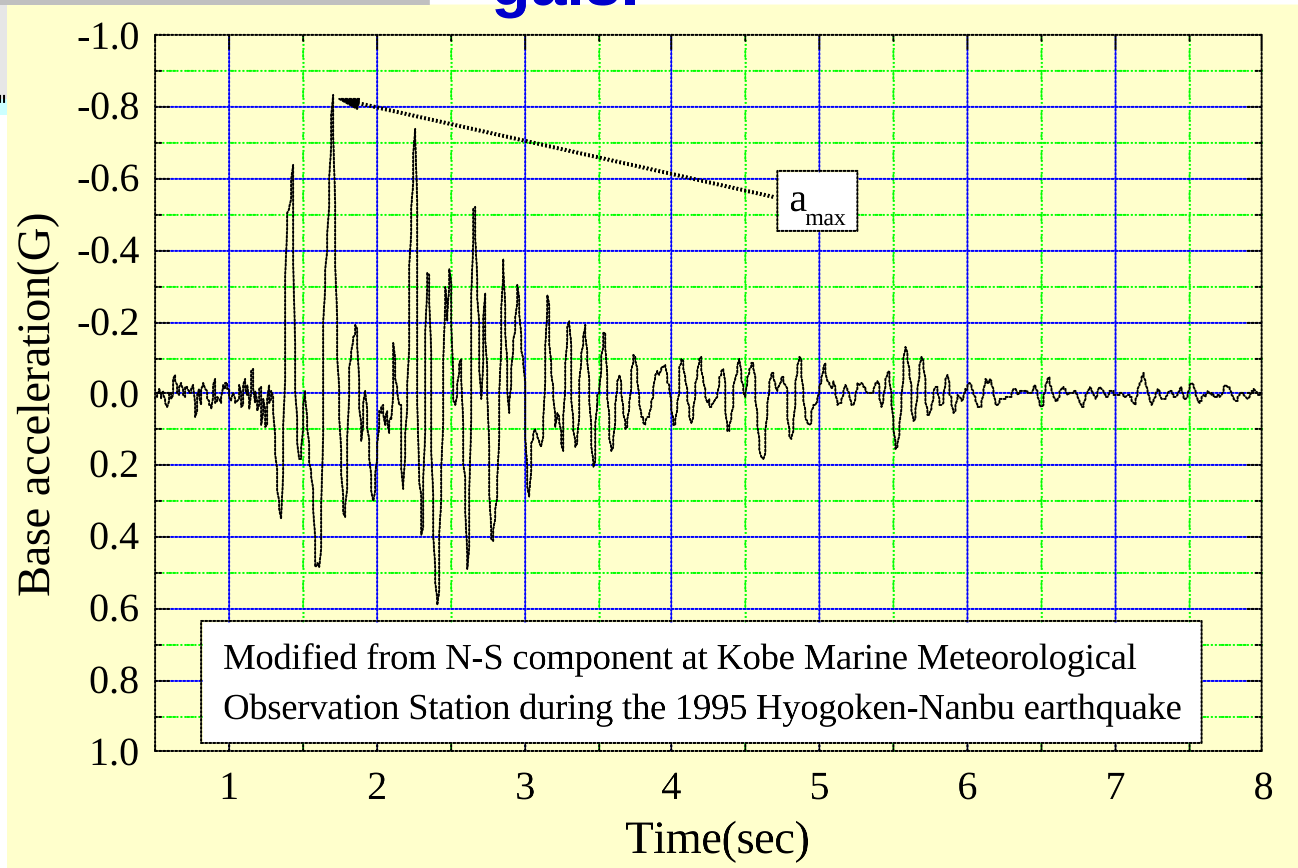


MO theory gives an extremely high seismic earth pressure for such high k_h.

Gravity type RW model in a shaking table test

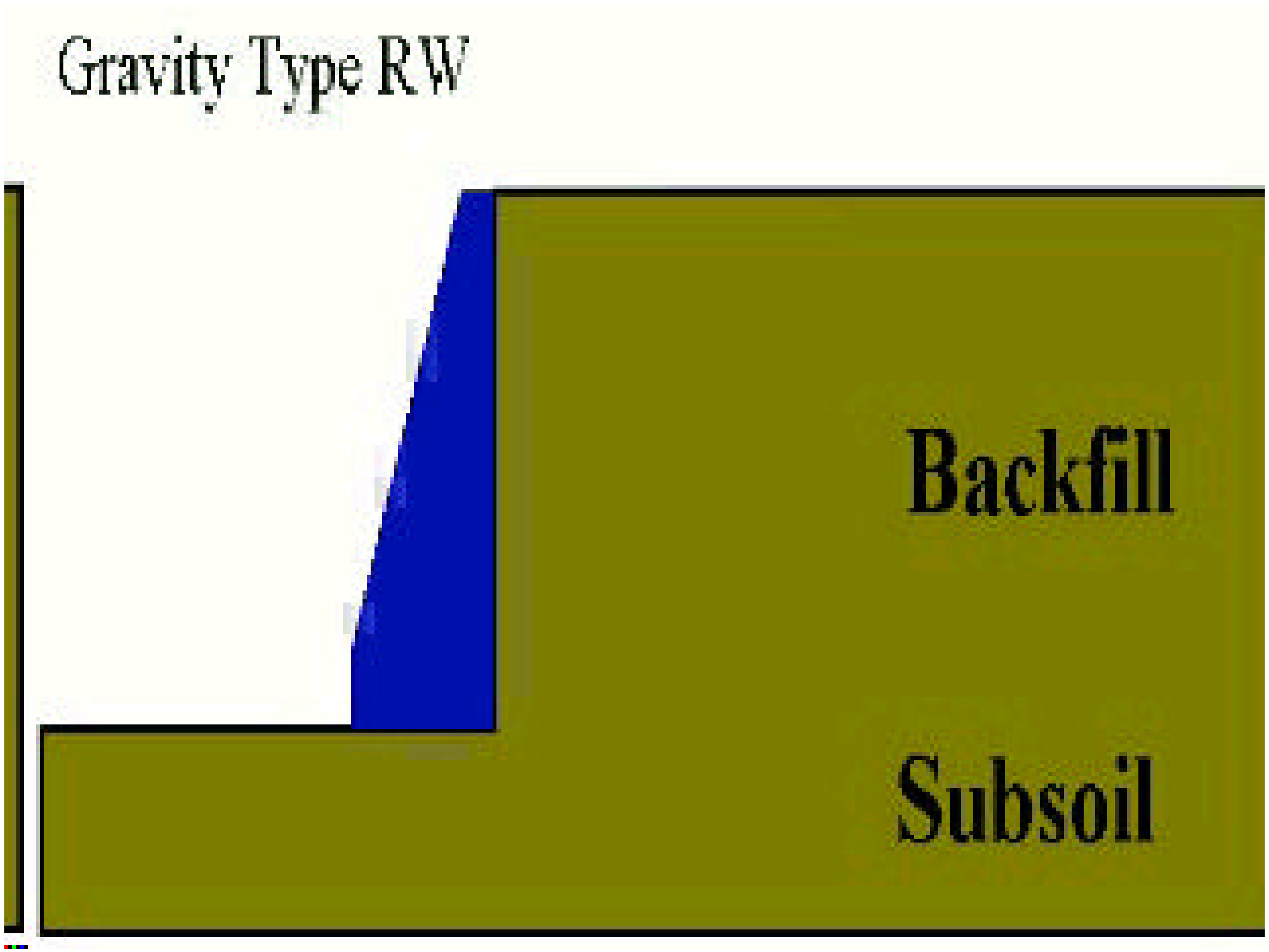


Subjected to several steps of irregular excitation with increasing a_{max} at increment of 100 gals.

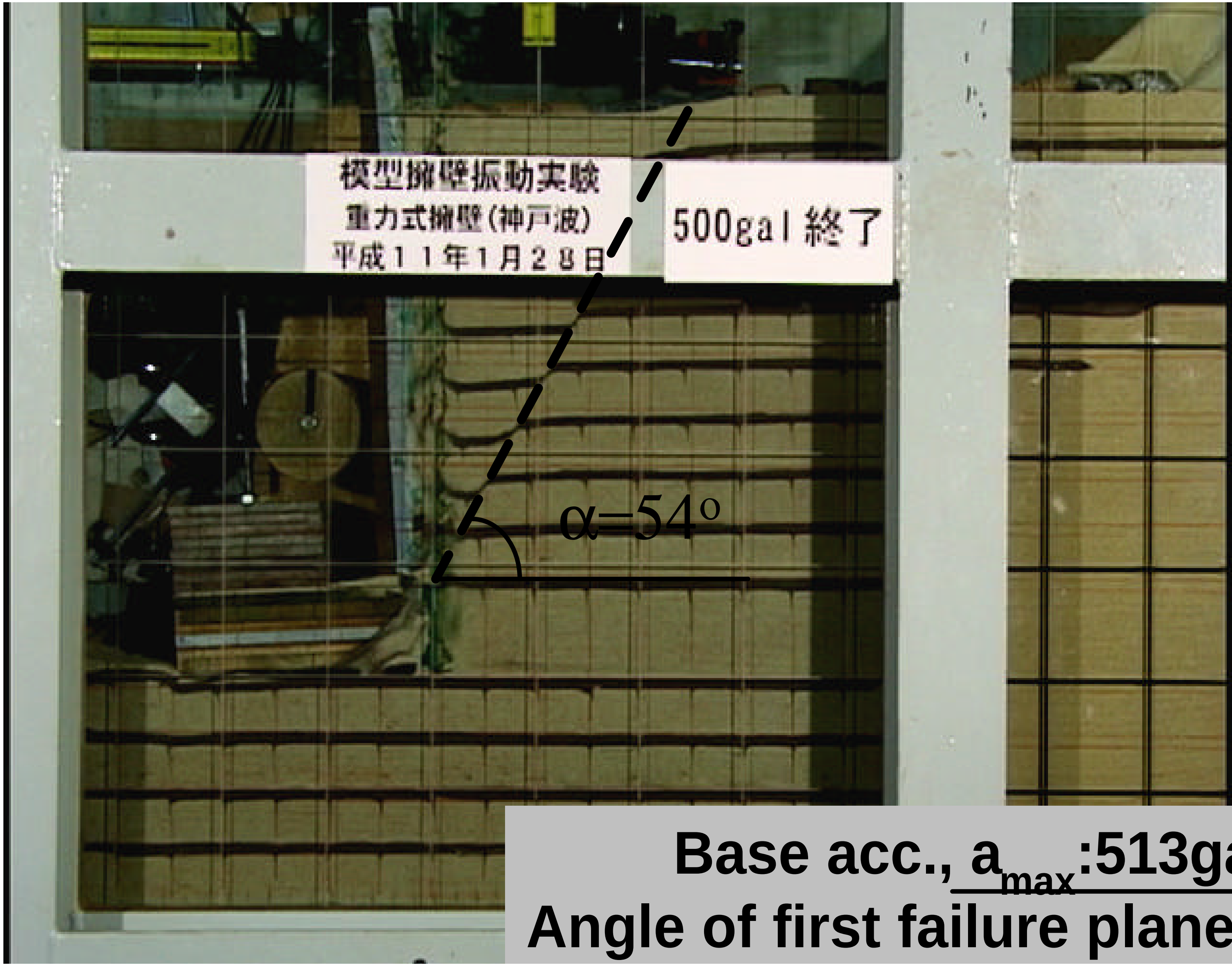


Observed behaviour of gravity type wall

a_{max}: 919gal

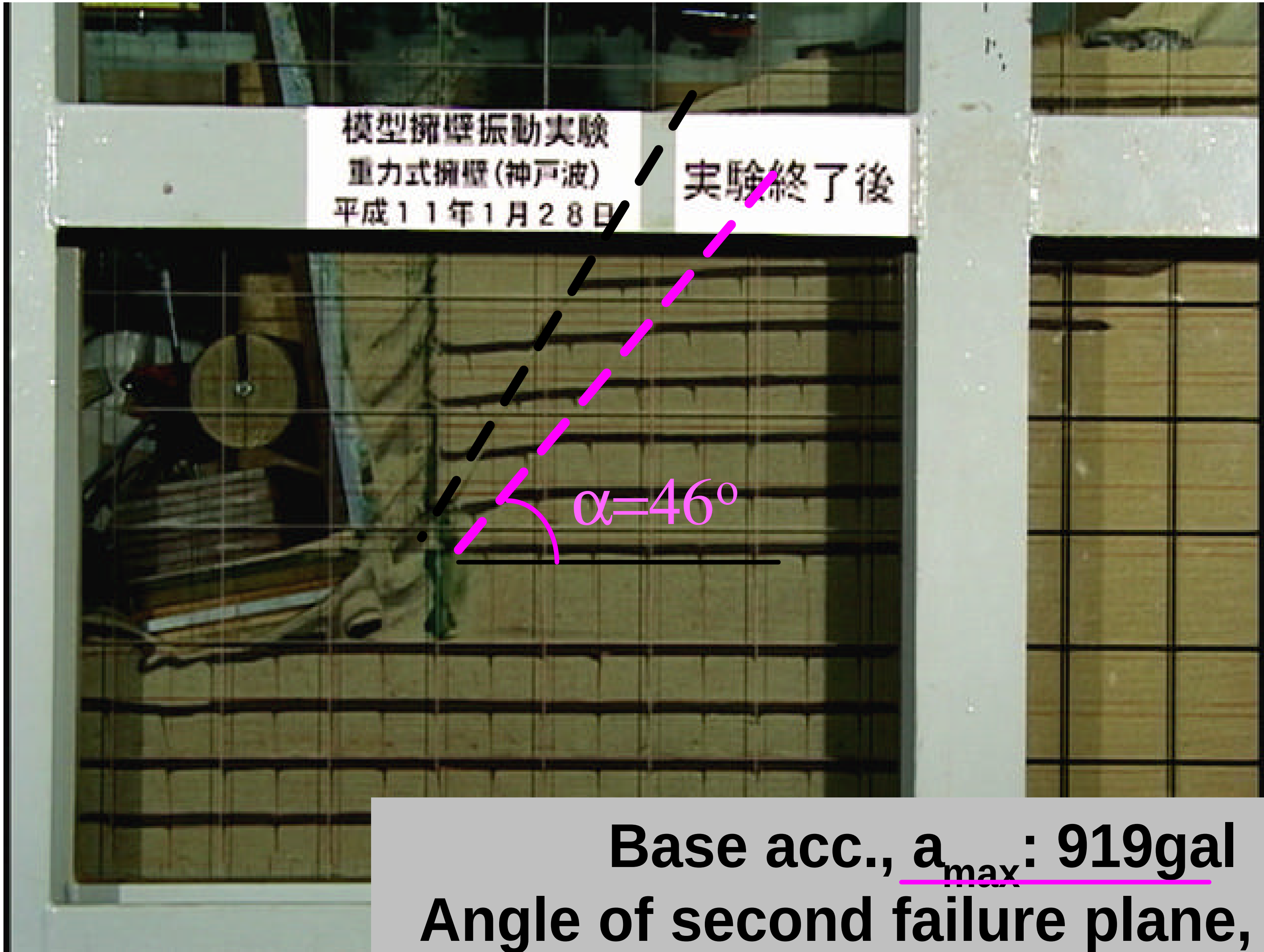


Formation of first failure plane



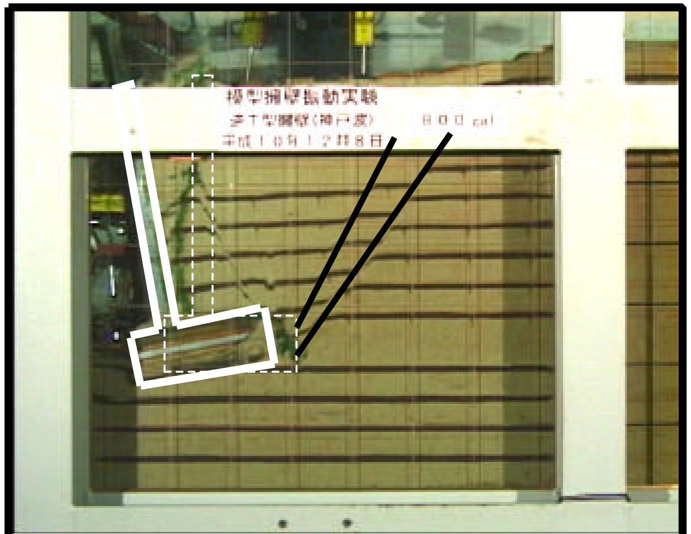
Base acc., a_{max}:513gal
Angle of first failure plane, a : 54º

Formation of <u>second</u> failure plane

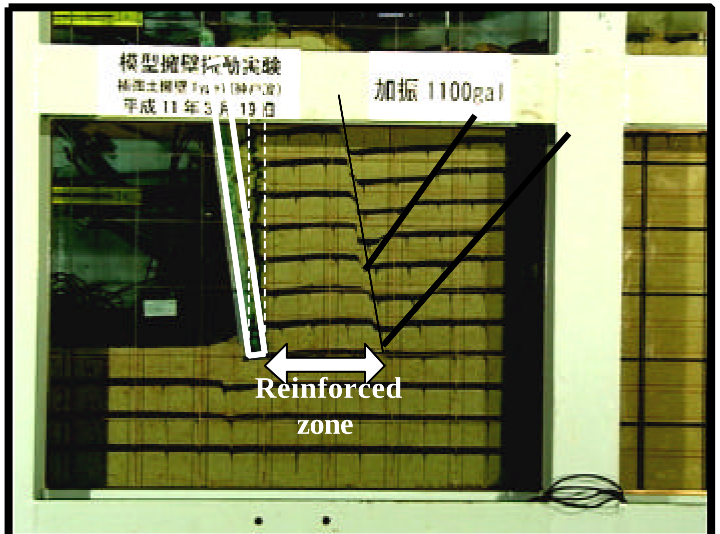


Base acc., a_{max}: 919gal

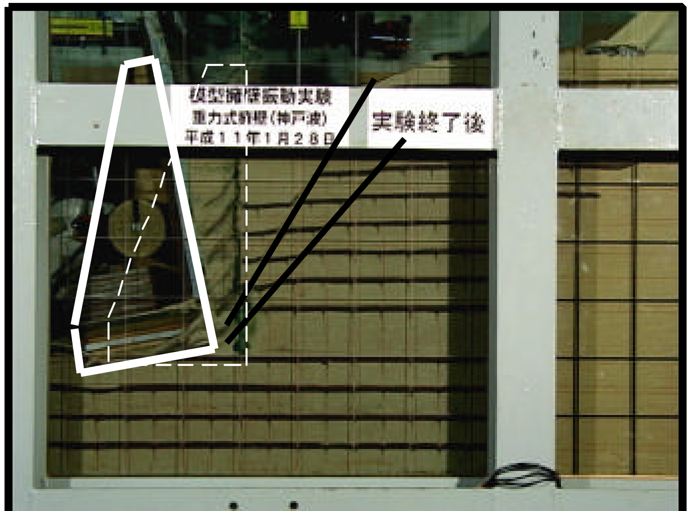
Angle of second failure plane, α : 46°



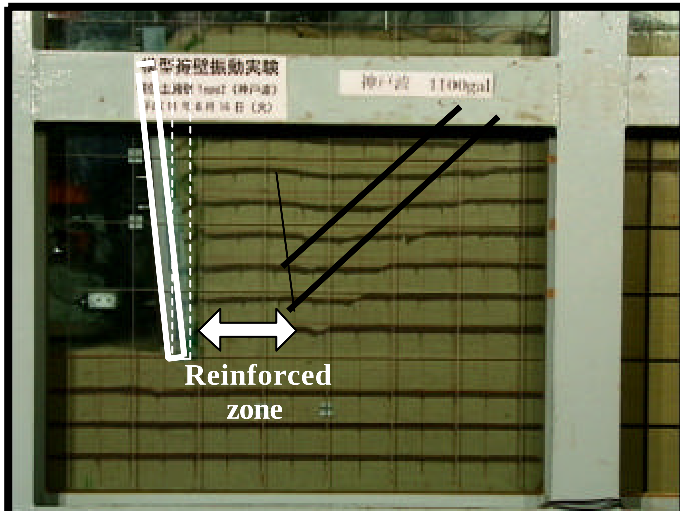
Cantilever type a_{max}=765gal



Reinforced-soil type1 a_{max}=1019gal



Gravity type a_{max}=919gal



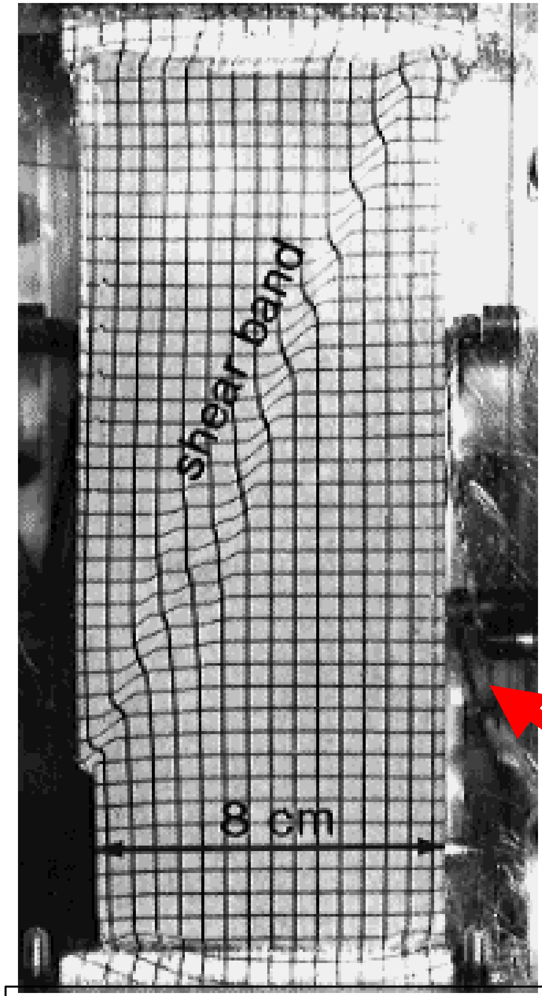
Reinforced-soil type2 a_{max}=1106gal

First and second failure planes

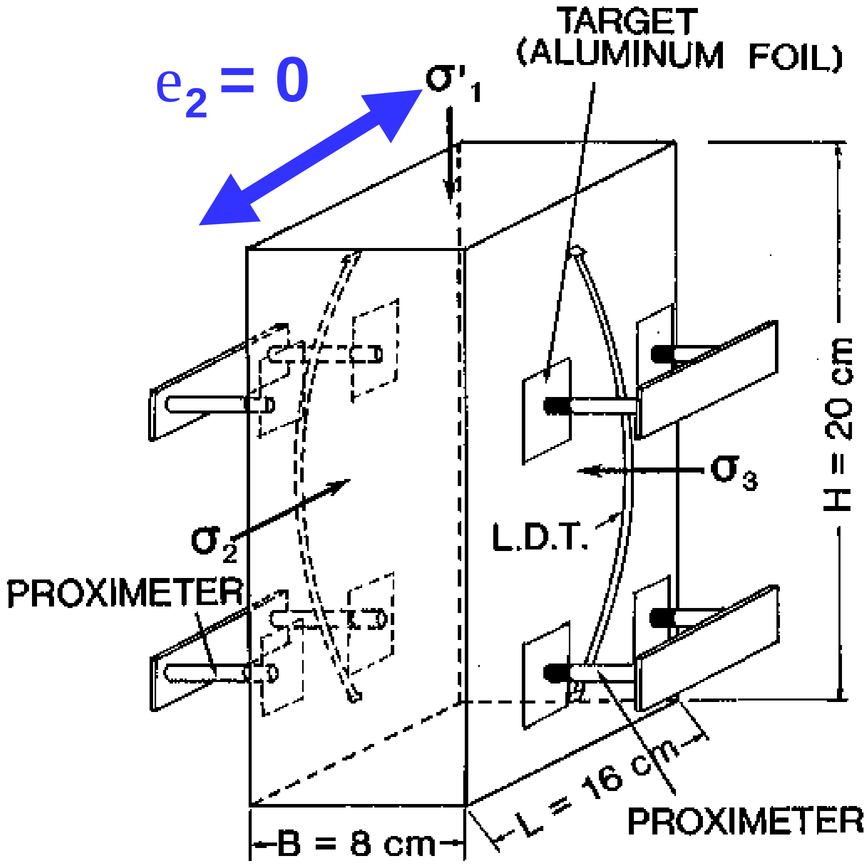
Lessons:

1. **Shear band does not move towards deeper locations continuously with a continuous increase in the input motion.**
2. **The first-developed shear band develops further with an increase in the input motion until the input motion becomes larger enough to develop the second, deeper shear band.**

Why ?



A shear band seen at an axial strain of 11.8 % in a PSC test on Toyoura sand (D_{50}= 0.206 mm; s'_3= 78 kPa) (Yoshida et al., 1995: Yoshida & Tatsuoka 1997).

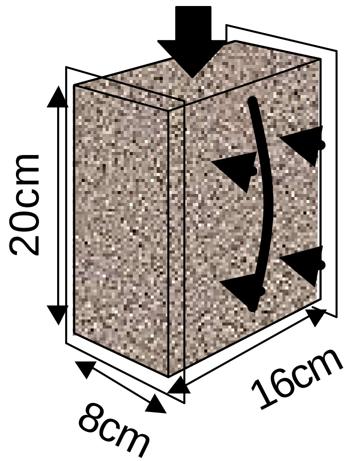


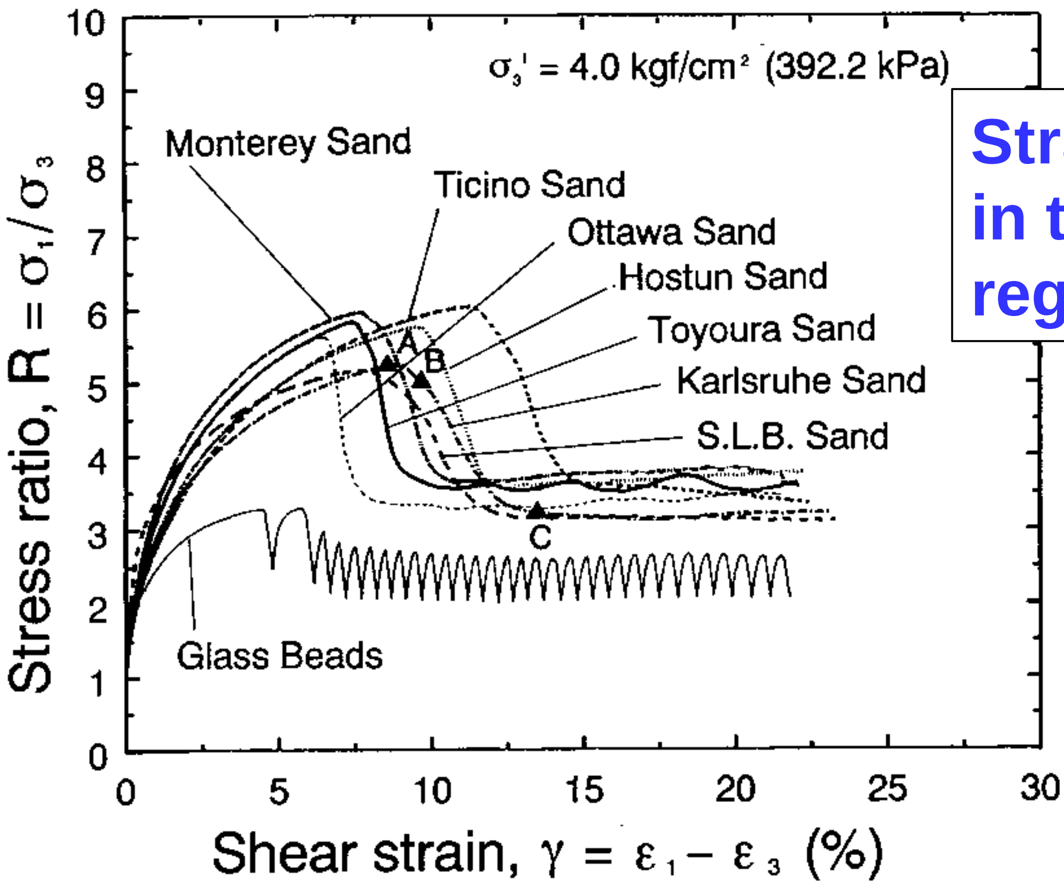
σ'_2 surface seen through the transparent confining platen;

1. Grids drawn on the lateral rubber membrane were made of latex rubber..
2. A number of pictures were taken in each test.
3. Displacements at the nodes were read to an accuracy of the order of 0.01 mm.

Shear stress level:

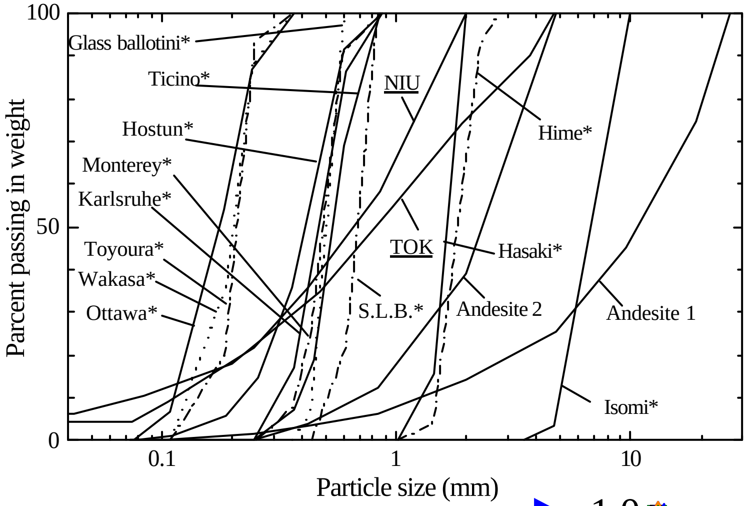
$$R_n = \frac{R - R_{residual}}{R_{peak} - R_{residual}} \quad \begin{cases} R_n = 1.0 \quad when \quad R = R_{peak} \\ R_n = 0.0 \quad when \quad R = R_{residual} \end{cases}$$

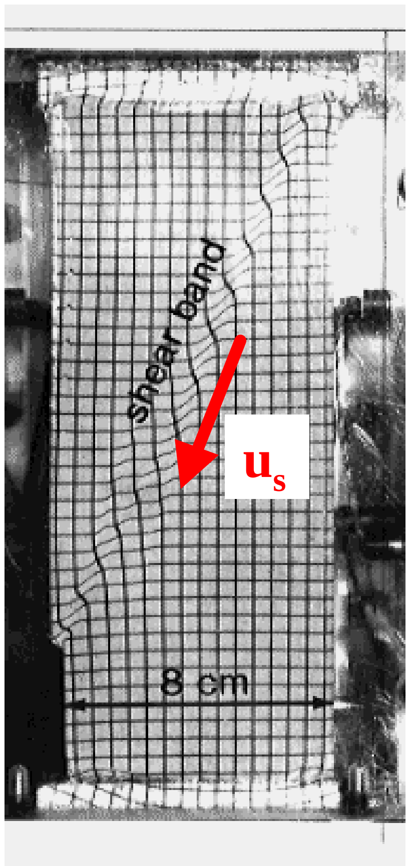


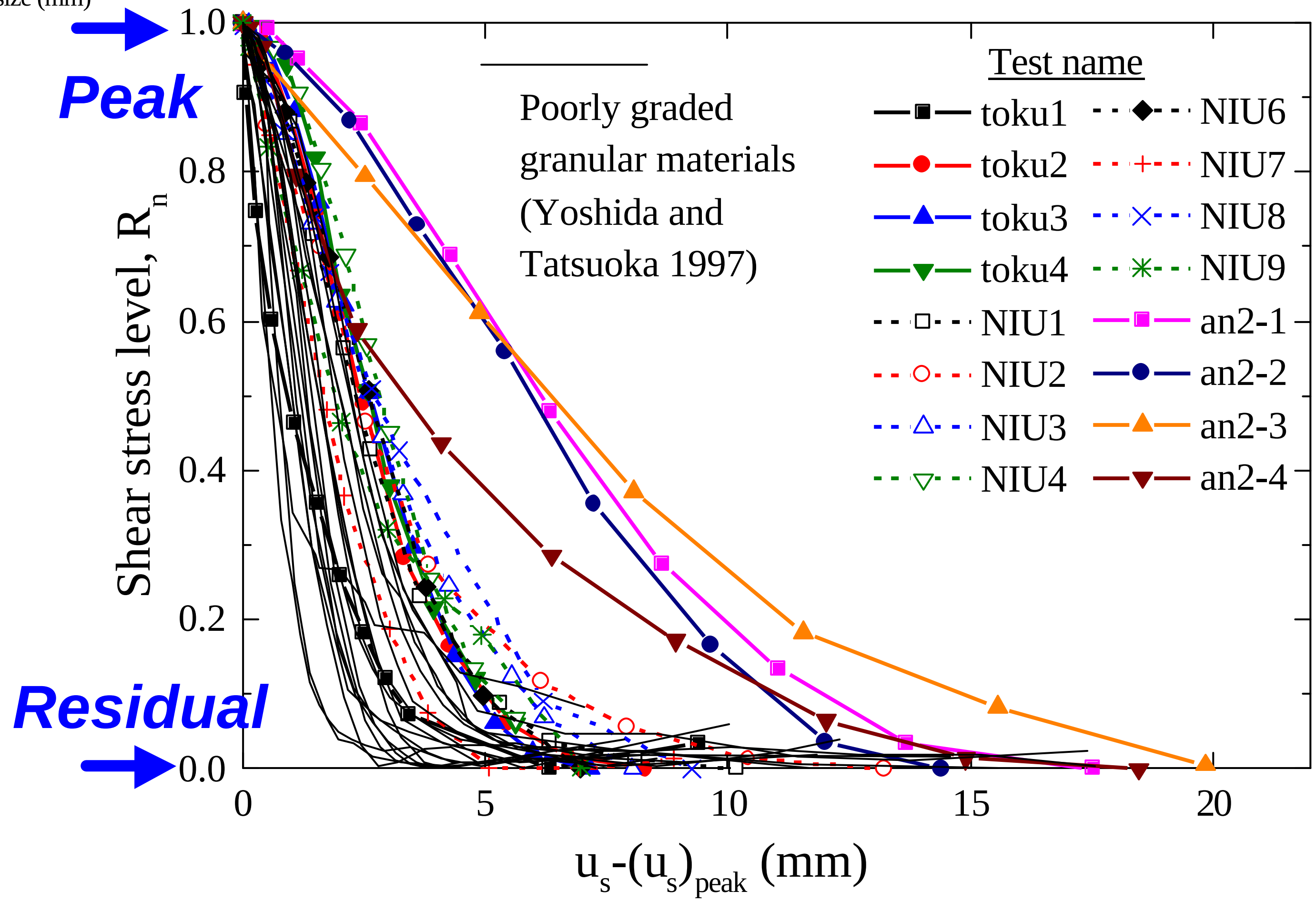


Strain-softening in the post-peak regime

Very different relations for a wide range of particle size







Topics:

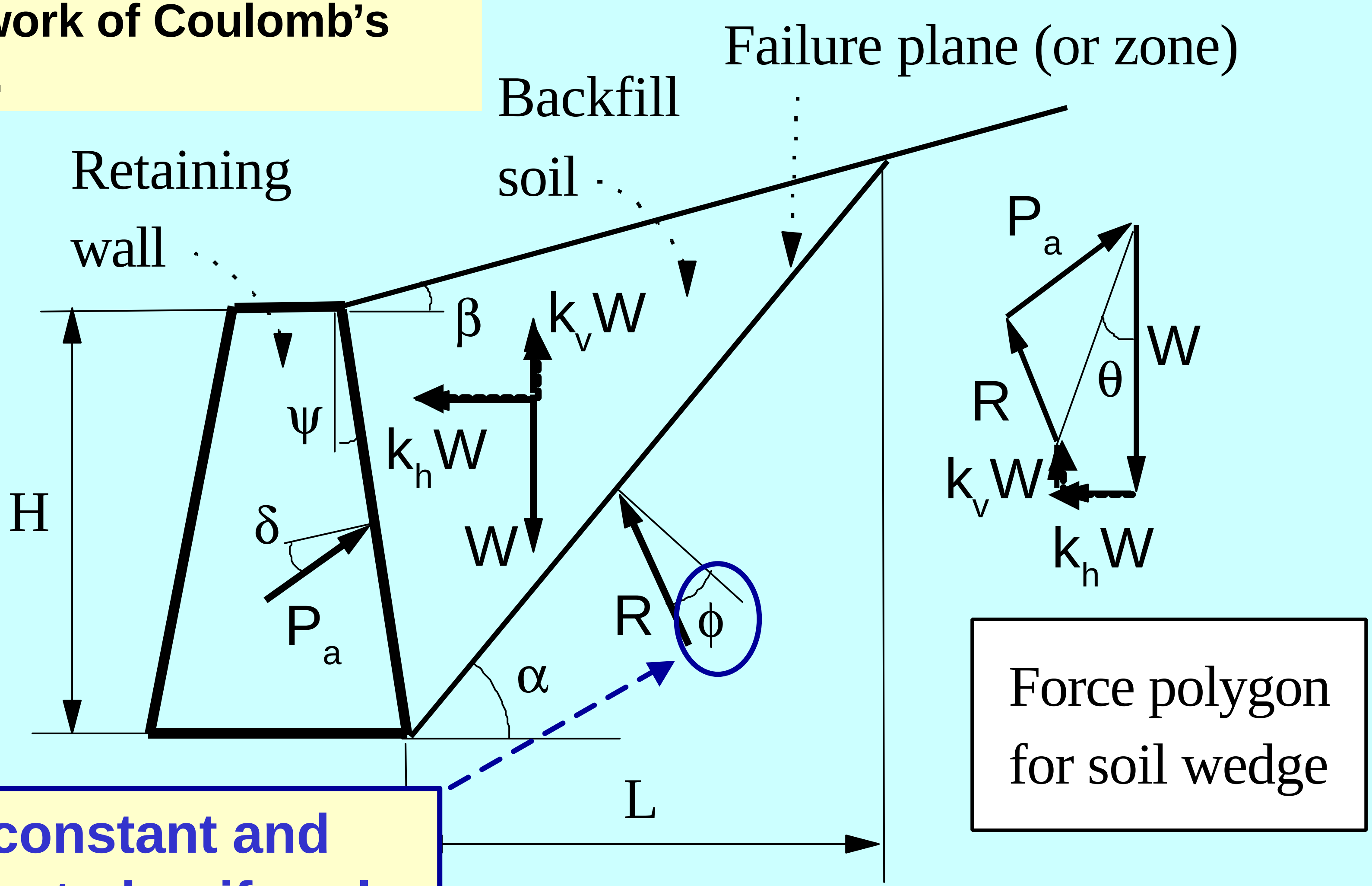
3-1 Observed failure pattern of backfill soil

3-2 Modification of *Mononobe-Okabe* method

3-2 Comparison with model test results

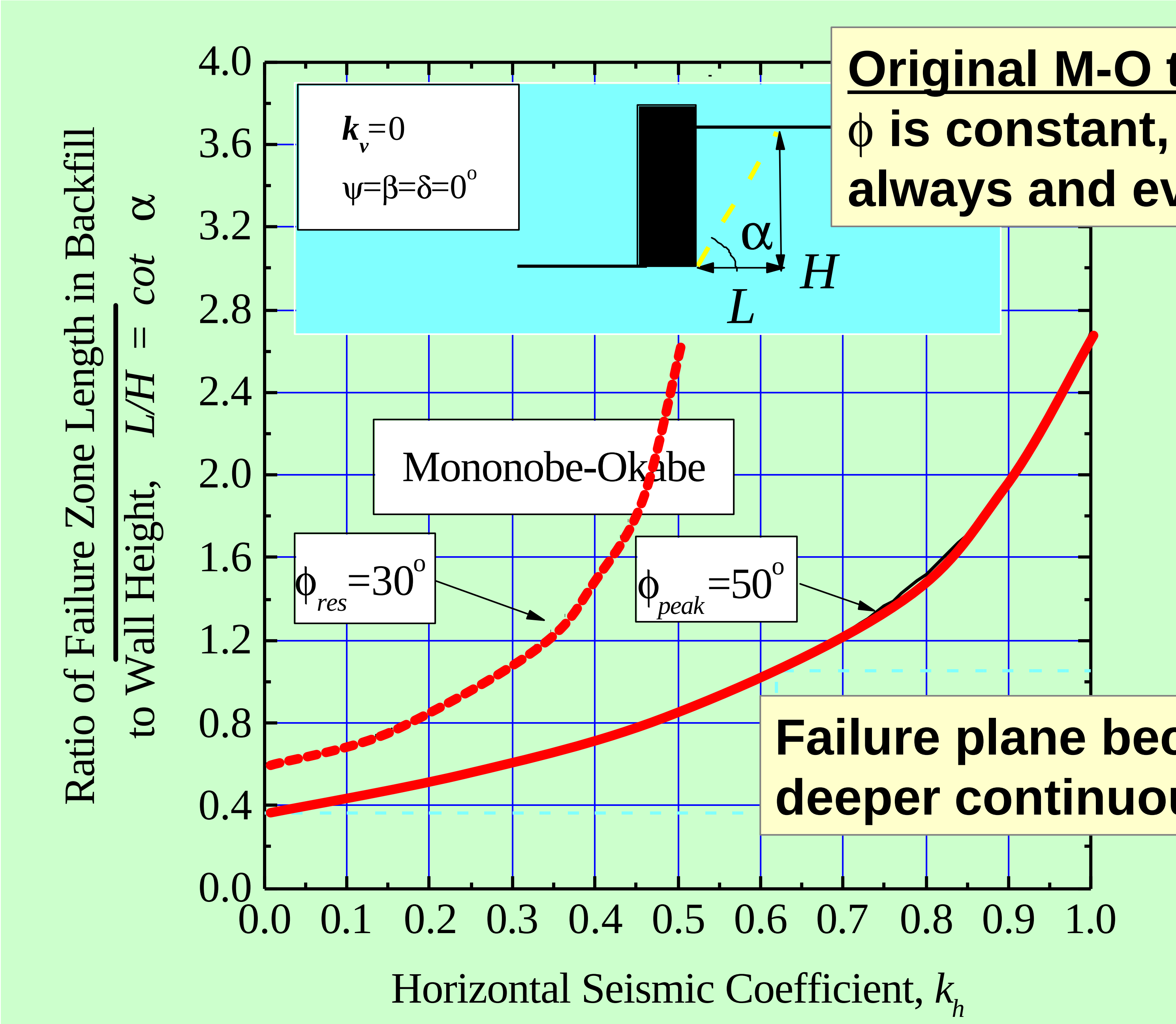
Original *Mononobe-Okabe* theory

Effects of seismic inertia forces considered in the framework of Coulomb's theory.



Kept constant and distributed uniformly.

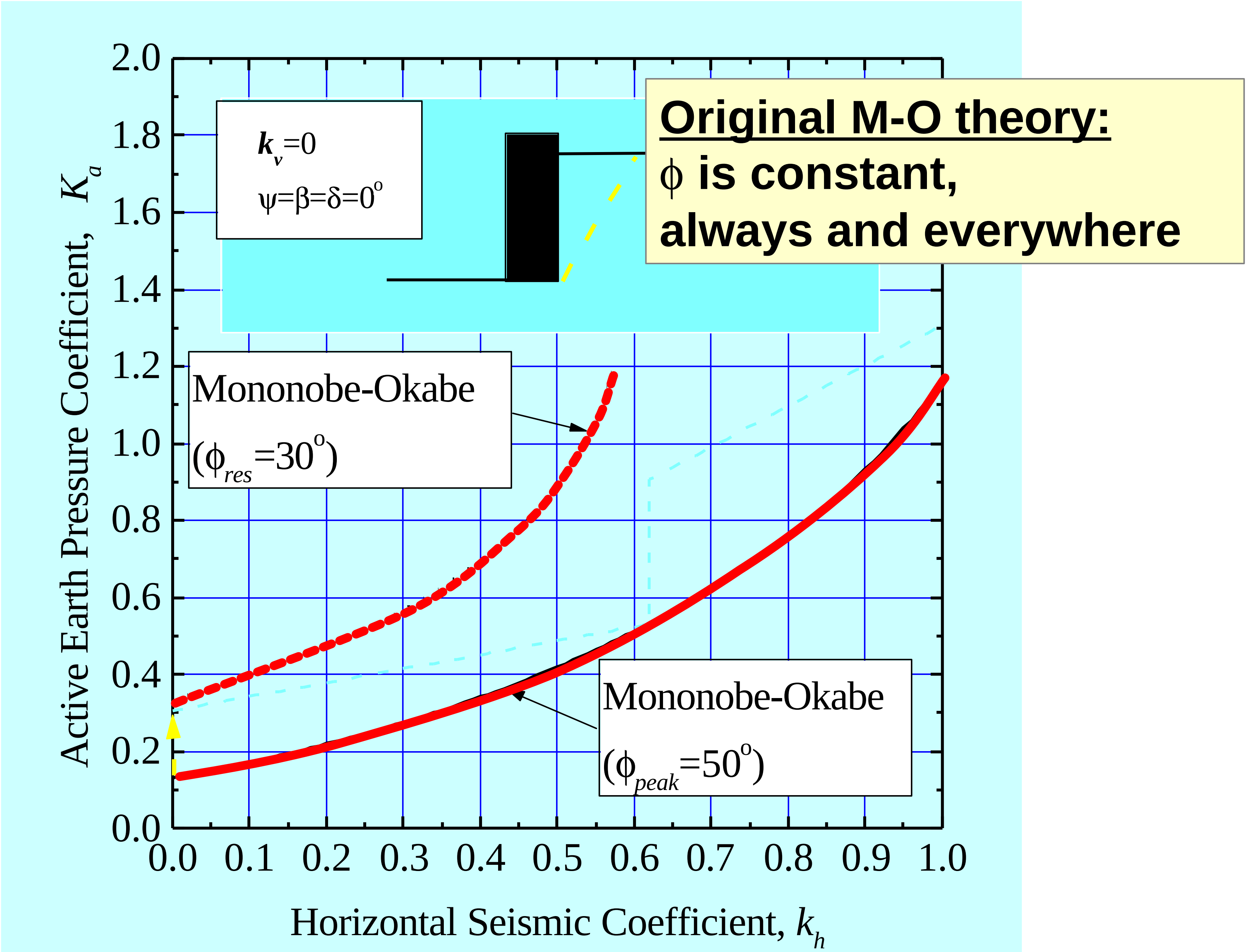
Failure zone vs. seismic coefficient



Original M-O theory:
φ is constant,
always and everywhere.

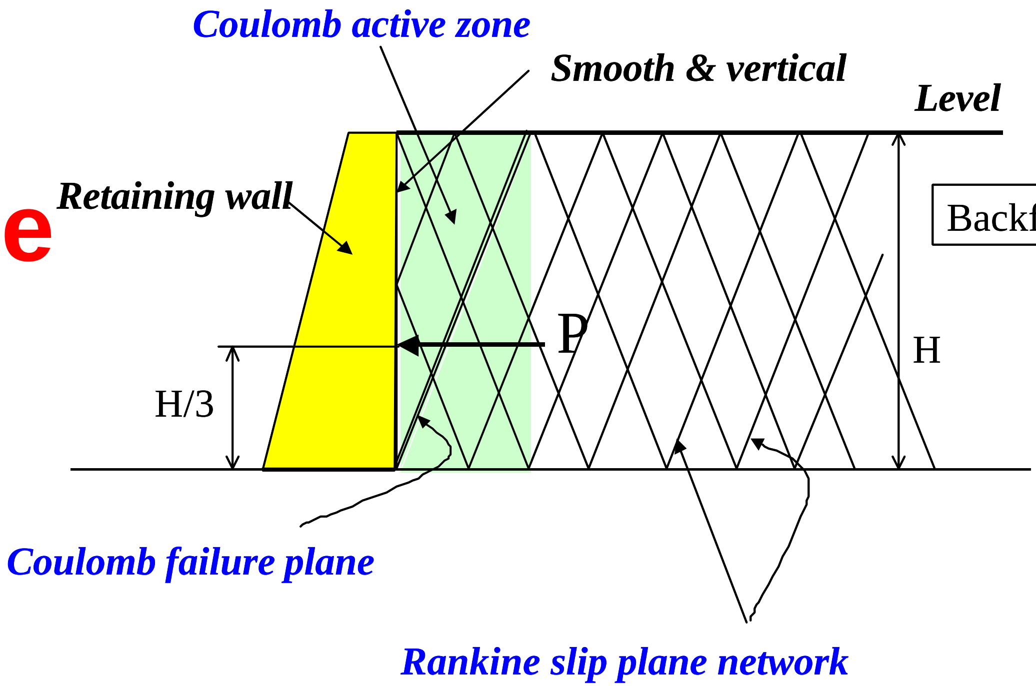
Failure plane becomes deeper continuously with k_h.

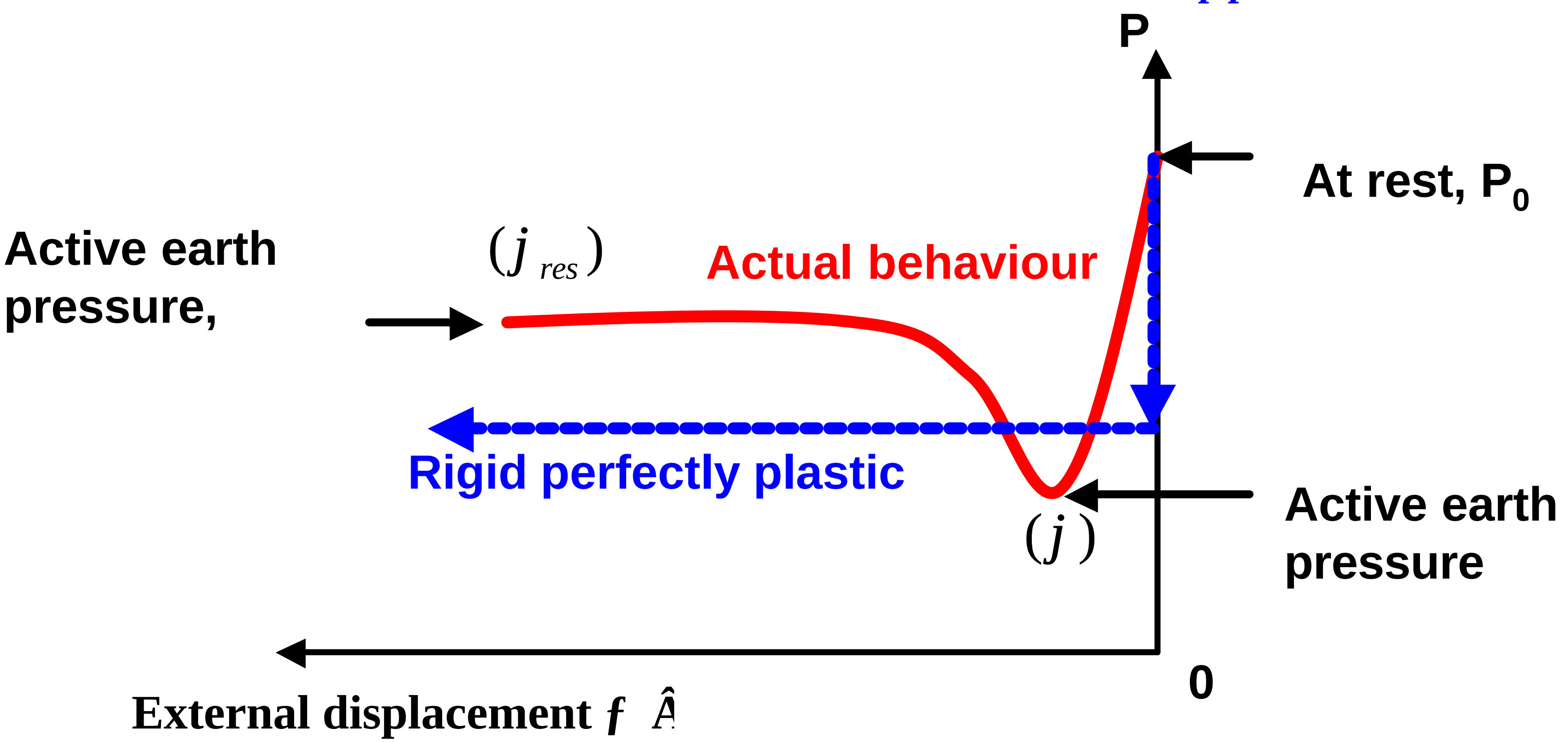
Earth pressure coefficient vs. seismic coefficient



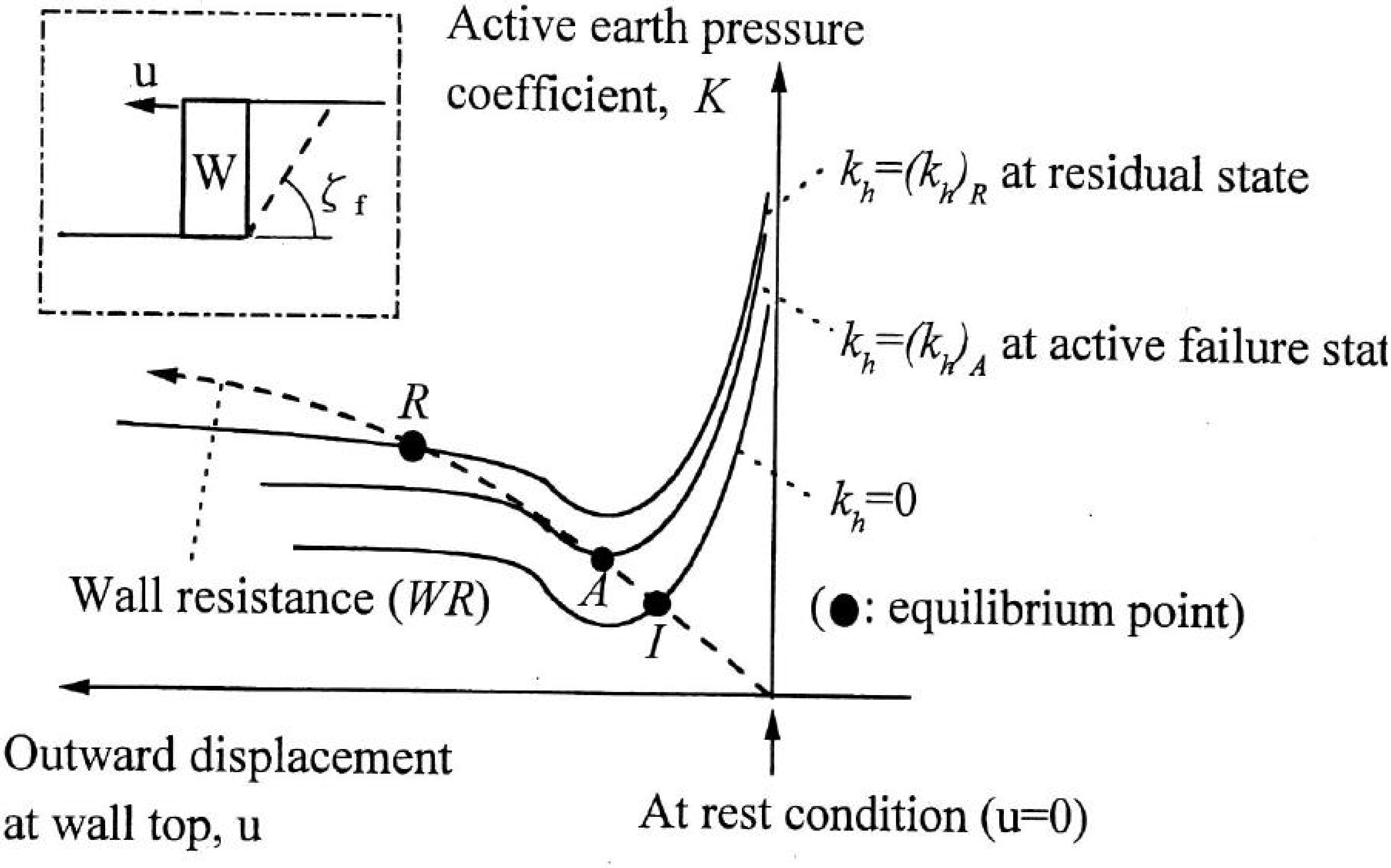
Key 1

Earth pressure





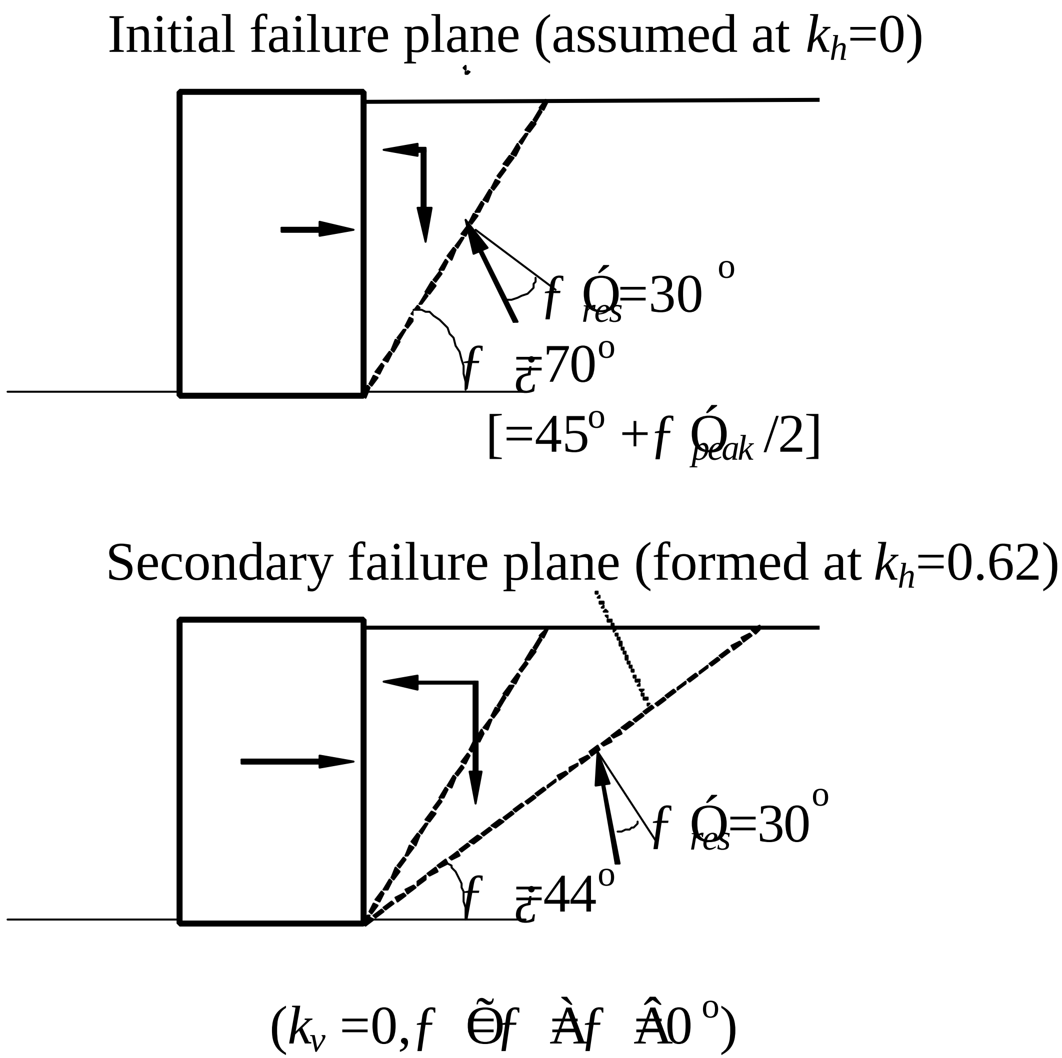
Increase in the seismic earth pressure associated with strain softening by shear banding

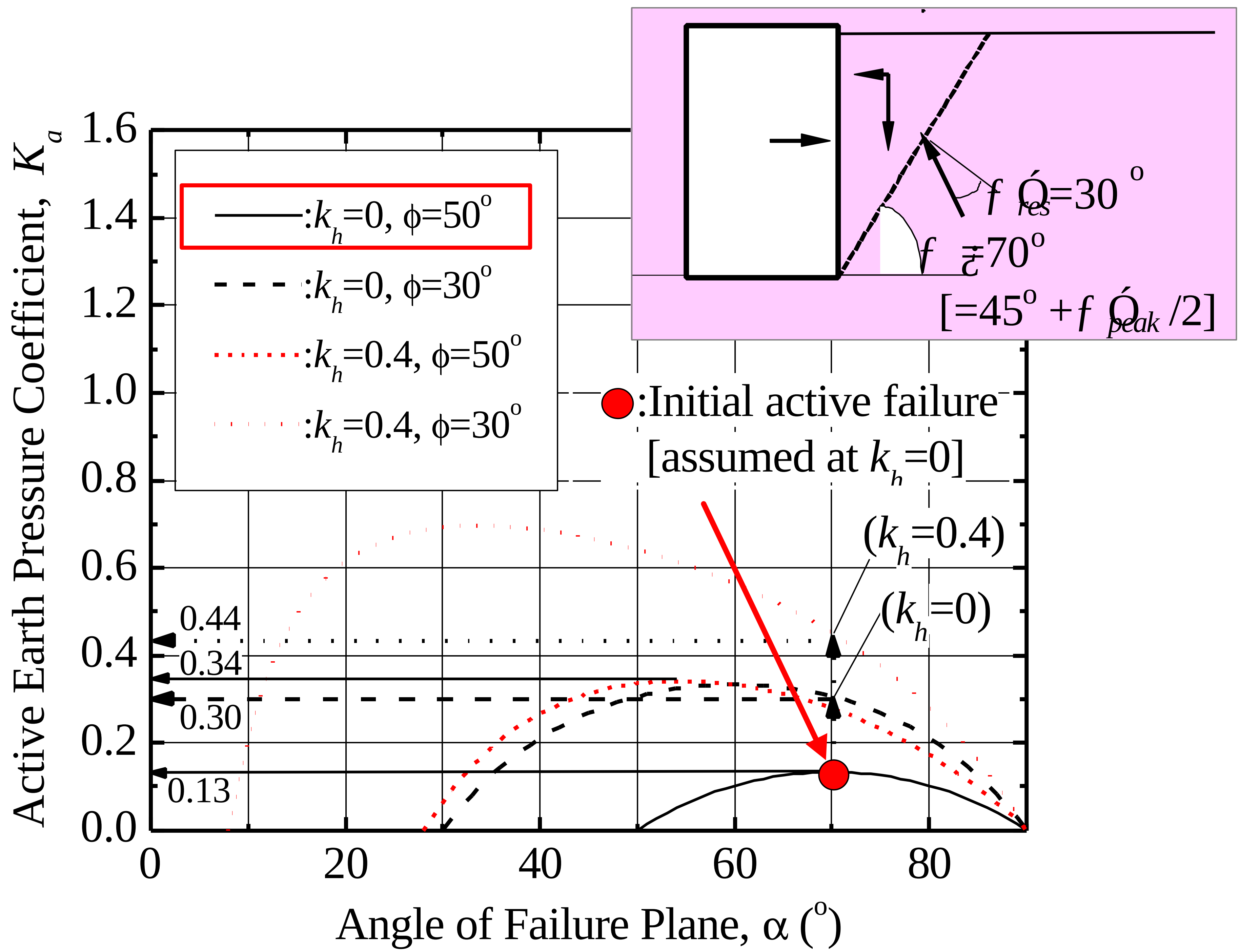


Key 2

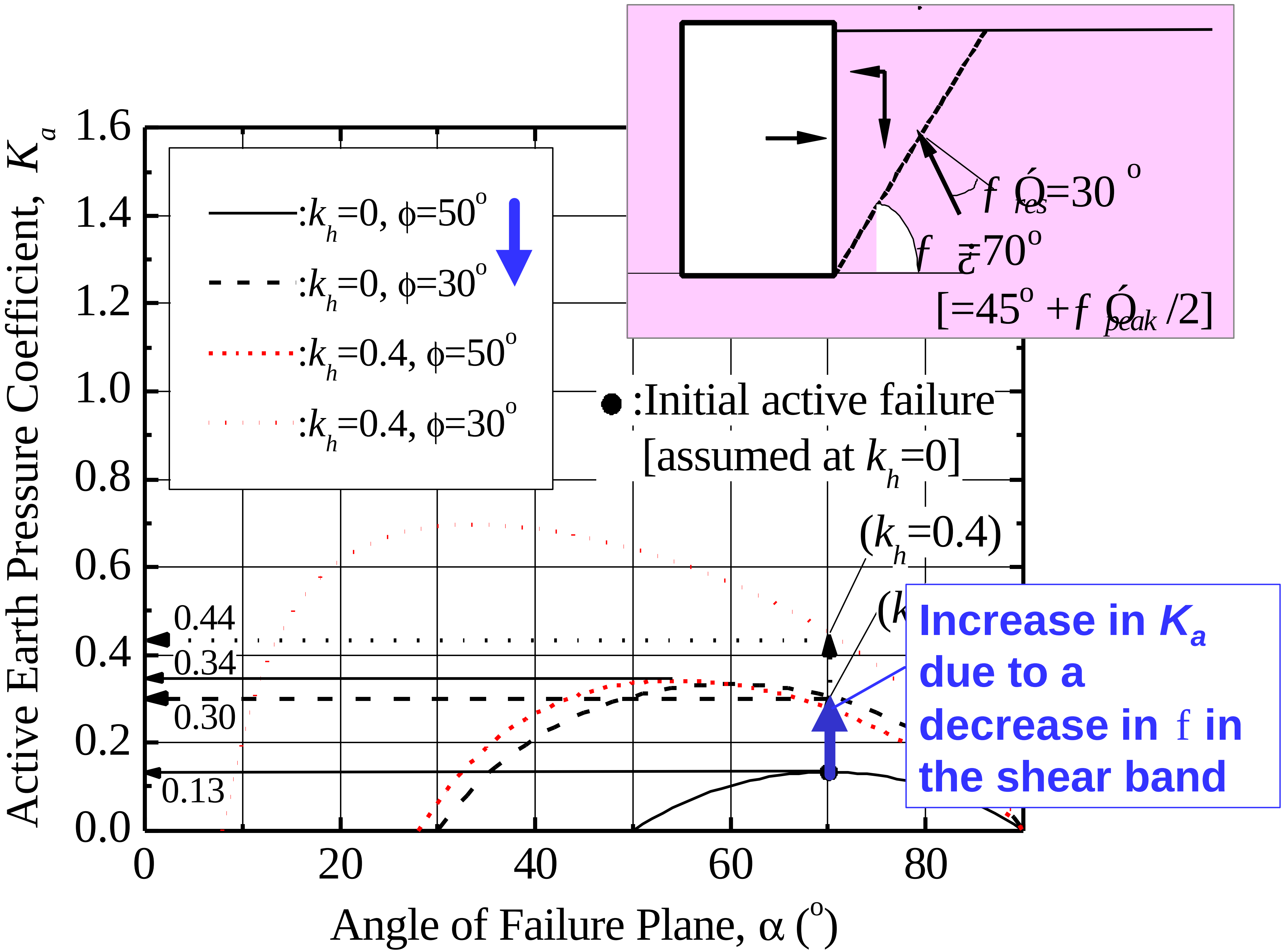
Discontinuous developments of shear band against a continuous increase in the acceleration

$\phi_{peak} = 50^o$
$\phi_{res} = 30^o$
$\delta = 0^o$

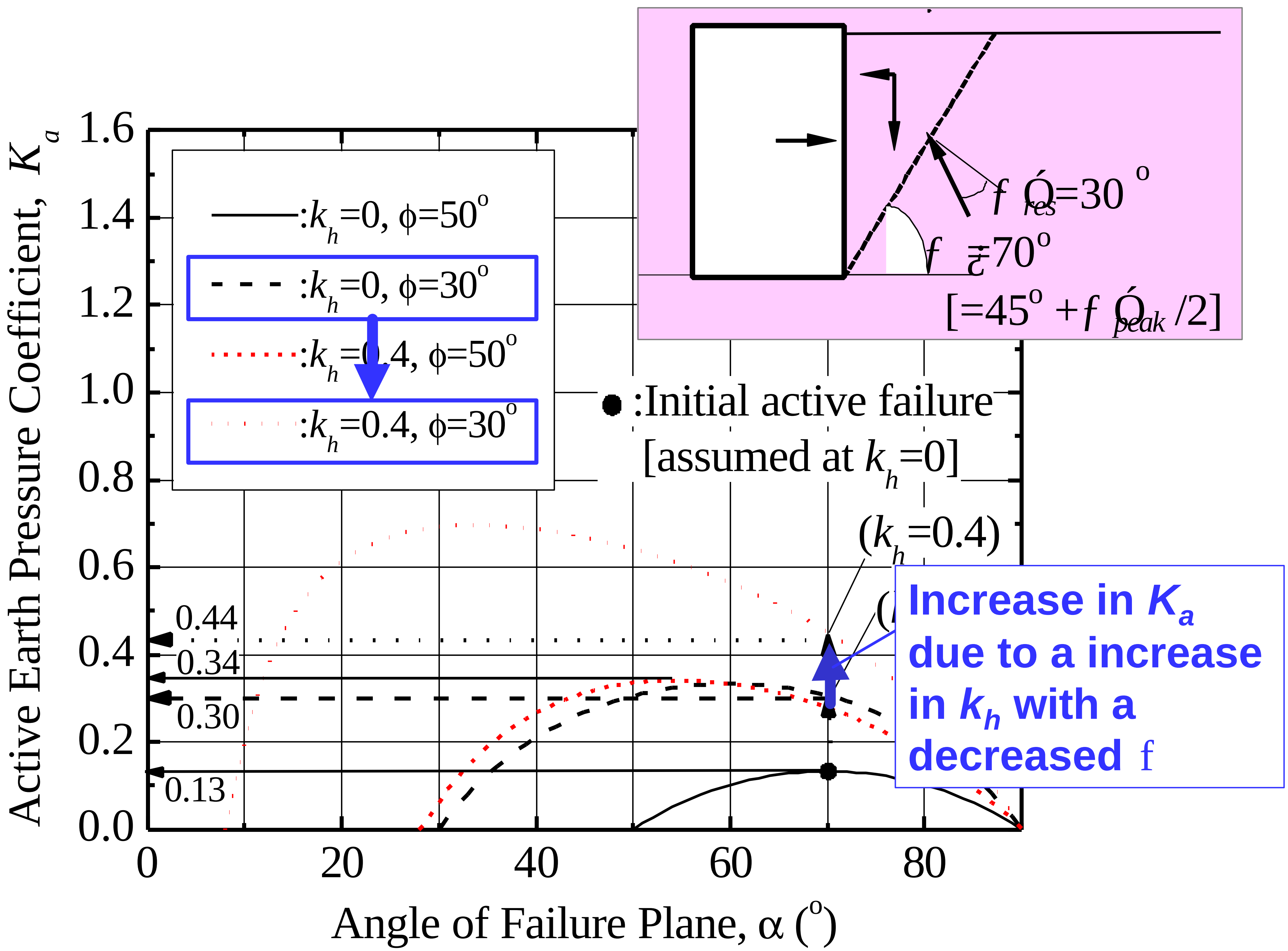




Active Earth Pressure Coefficient, K_a
Angle of Failure Plane, α (°)
:k_h=0, ϕ=50°
:k_h=0, ϕ=30°
:k_h=0.4, ϕ=50°
:k_h=0.4, ϕ=30°
:Initial active failure
[assumed at k_h=0]
(k_h=0.4)
(k_h=0)
0.44
0.34
0.30
0.13
ϕ_res=30°
70°
[=45° + ϕ_peak/2]

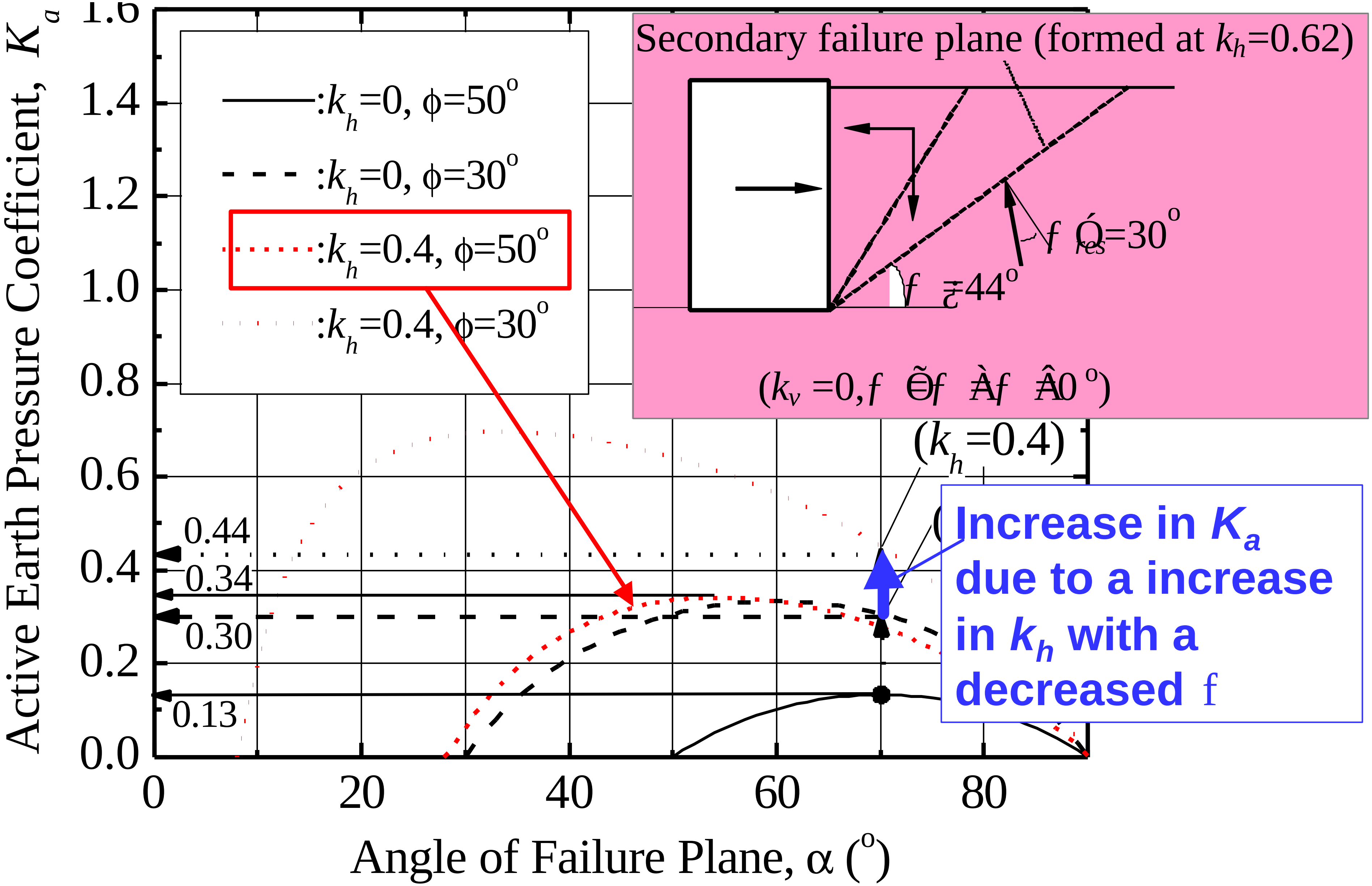


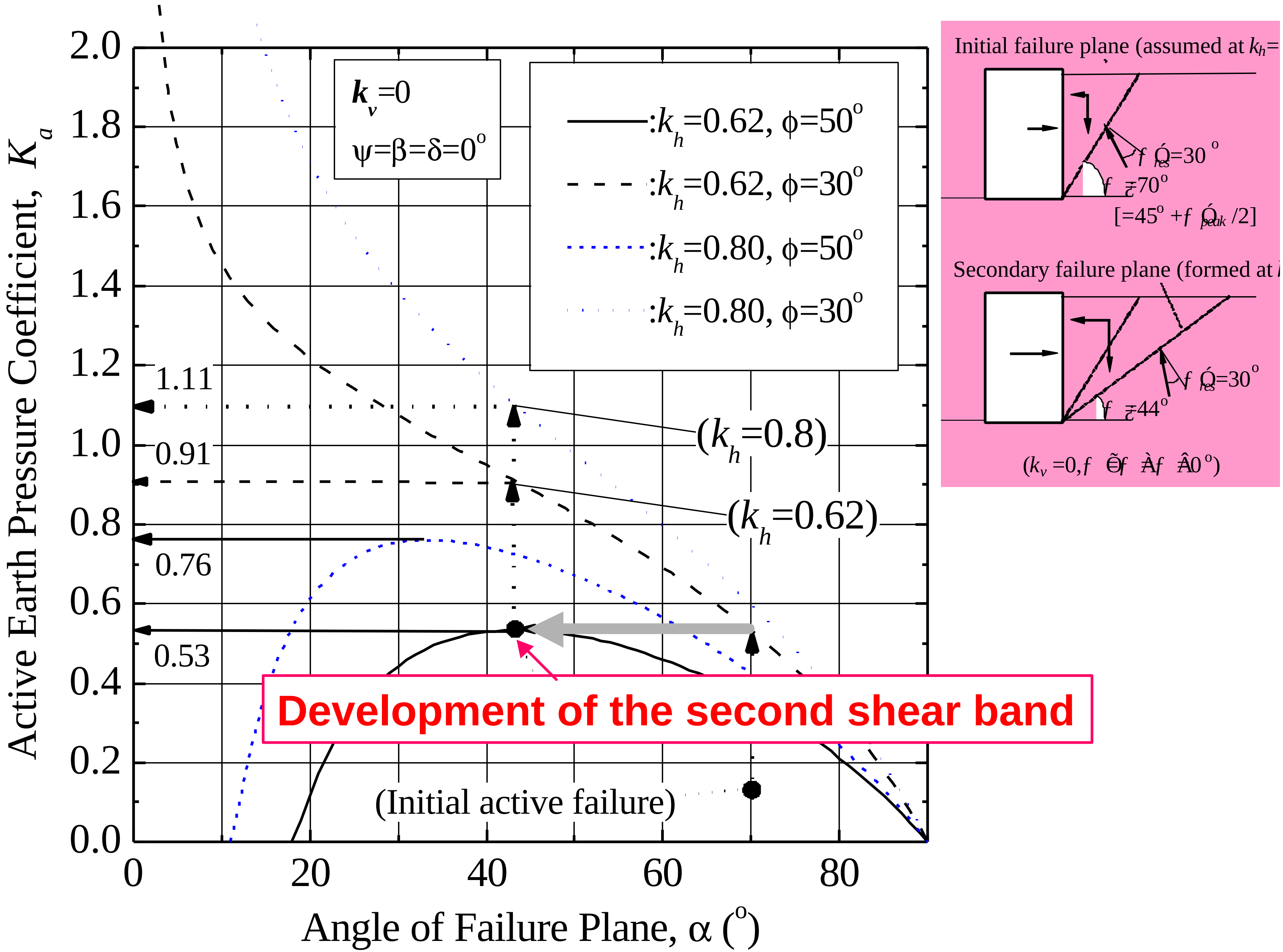
Active Earth Pressure Coefficient, K_a
Angle of Failure Plane, α (°)
:k_h=0, ϕ=50°
:k_h=0, ϕ=30°
:k_h=0.4, ϕ=50°
:k_h=0.4, ϕ=30°
:Initial active failure
[assumed at k_h=0]
(k_h=0.4)
0.44
0.34
0.30
0.13
Increase in K_a due to a decrease in ϕ in the shear band
ϕ_{res}=30°
α=70°
[=45° + ϕ_{peak}/2]



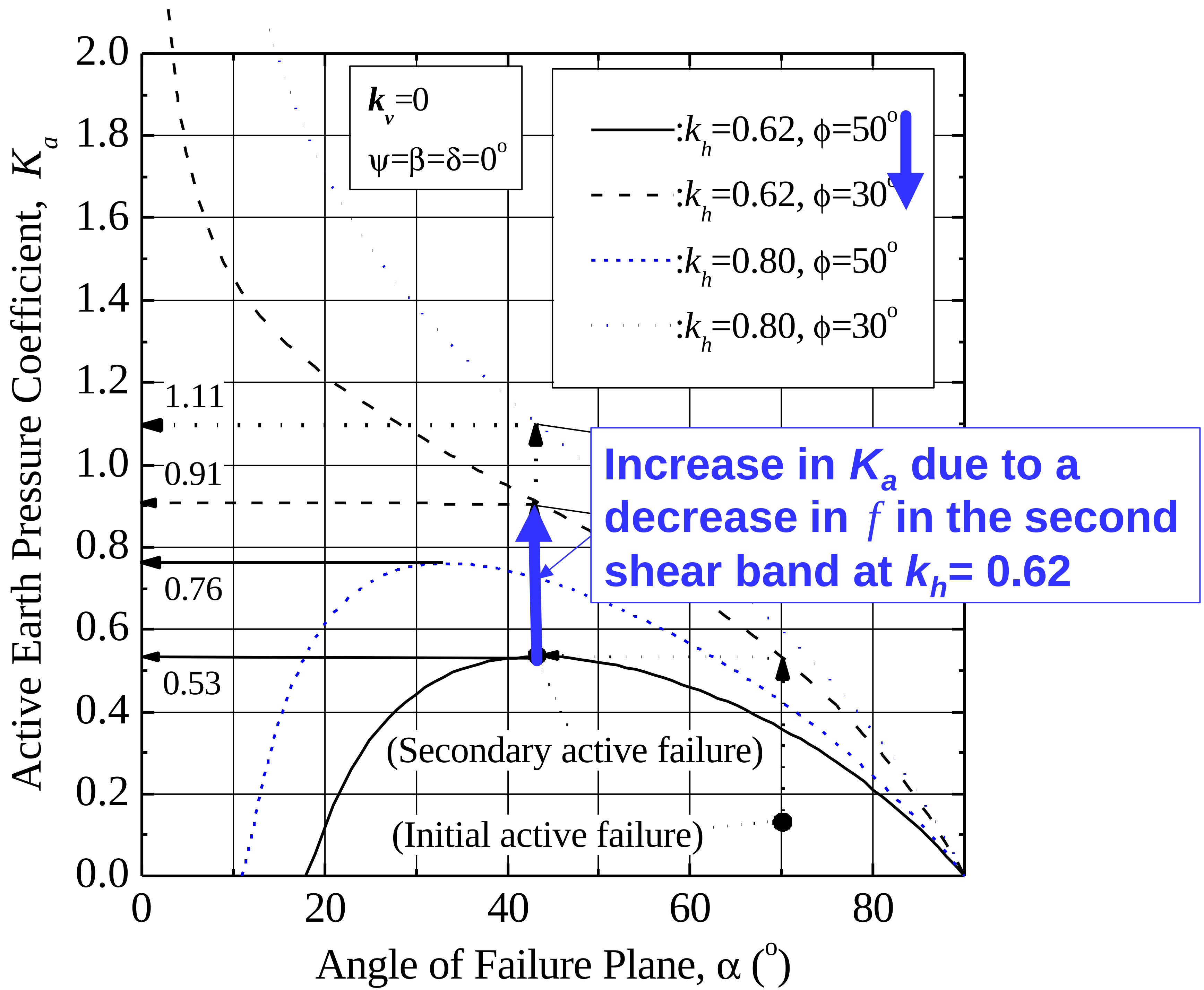
Active Earth Pressure Coefficient, K_a
Angle of Failure Plane, α (°)
:k_h=0, ϕ=50°
:k_h=0, ϕ=30°
:k_h=0.4, ϕ=50°
:k_h=0.4, ϕ=30°
:Initial active failure
[assumed at k_h=0]
(k_h=0.4)
0.44
0.34
0.30
0.13
=30°
70°
[=45° + /2]
Increase in K_a due to a increase in k_h with a decreased

Critical earth pressure in the outside the shear band; still smaller

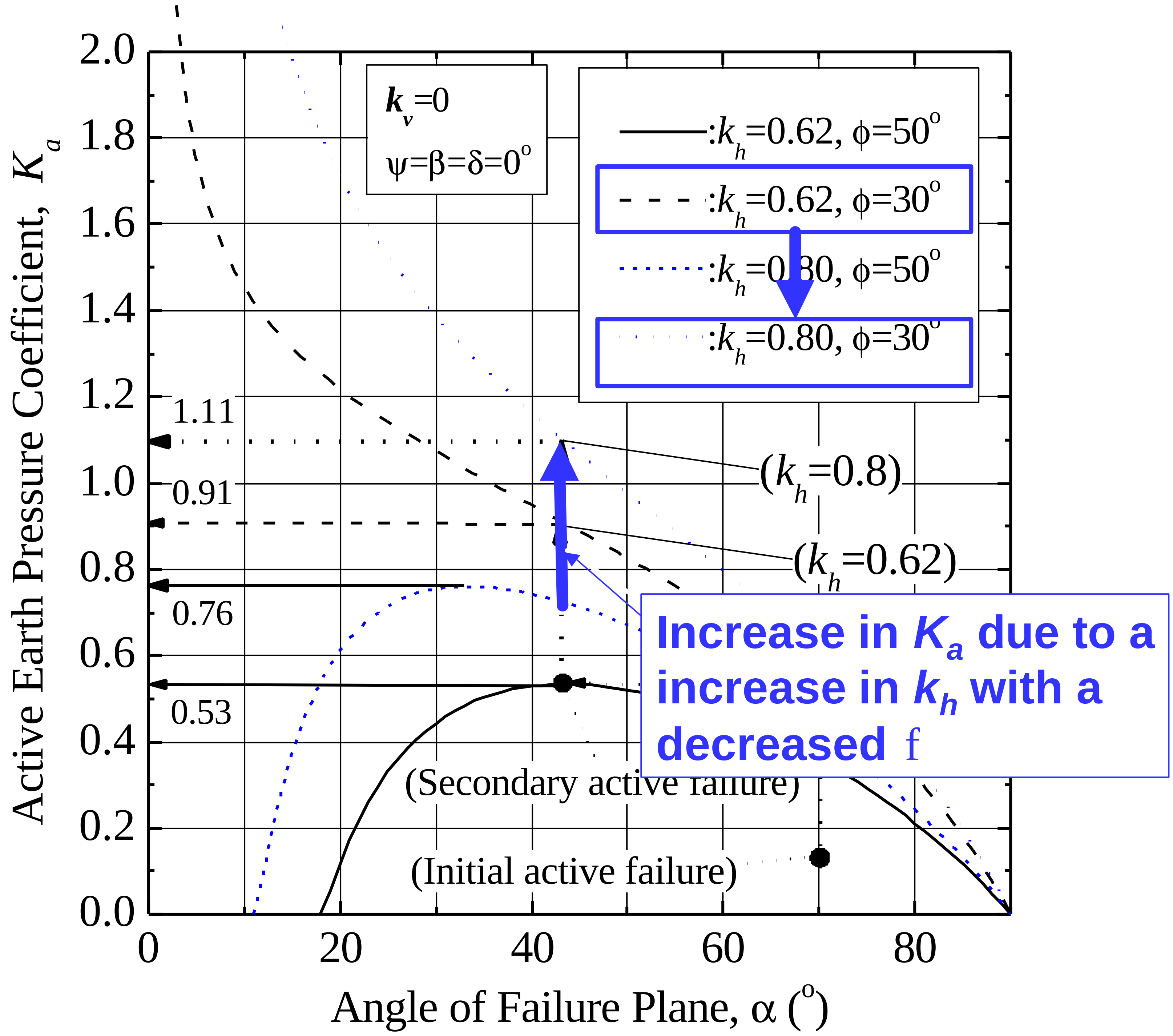




Active Earth Pressure Coefficient, K_a
Angle of Failure Plane, α (°)
k_v=0
ψ=β=δ=0°
:k_h=0.62, φ=50°
:k_h=0.62, φ=30°
:k_h=0.80, φ=50°
:k_h=0.80, φ=30°
1.11
0.91
0.76
0.53
(k_h=0.8)
(k_h=0.62)
(Initial active failure)
Development of the second shear band
Initial failure plane (assumed at k_h=
[=45° +
/2]
Secondary failure plane (formed at

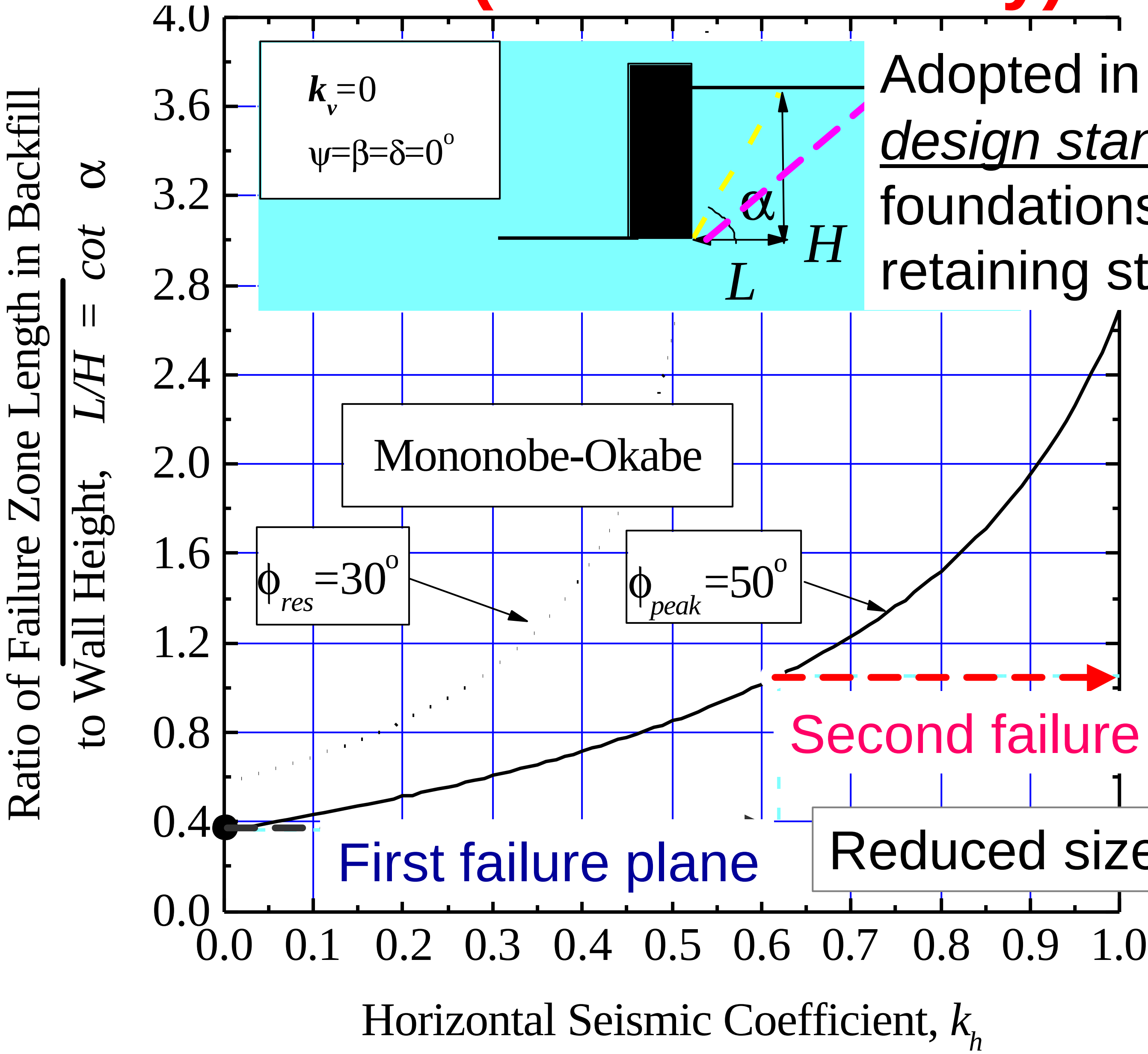


Active Earth Pressure Coefficient, K_a
Angle of Failure Plane, α (°)
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
20
40
60
80
k_v=0
ψ=β=δ=0°
:k_h=0.62, ϕ=50°
:k_h=0.62, ϕ=30°
:k_h=0.80, ϕ=50°
:k_h=0.80, ϕ=30°
1.11
0.91
0.76
0.53
Increase in K_a due to a decrease in f in the second shear band at k_h= 0.62
(Secondary active failure)
(Initial active failure)



Active Earth Pressure Coefficient, K_a
Angle of Failure Plane, α (°)
k_v=0
ψ=β=δ=0°
:k_h=0.62, φ=50°
:k_h=0.62, φ=30°
:k_h=0.80, φ=50°
:k_h=0.80, φ=30°
1.11
0.91
0.76
0.53
(k_h=0.8)
(k_h=0.62)
Increase in K_a due to a increase in k_h with a decreased f
(Secondary active failure)
(Initial active failure)

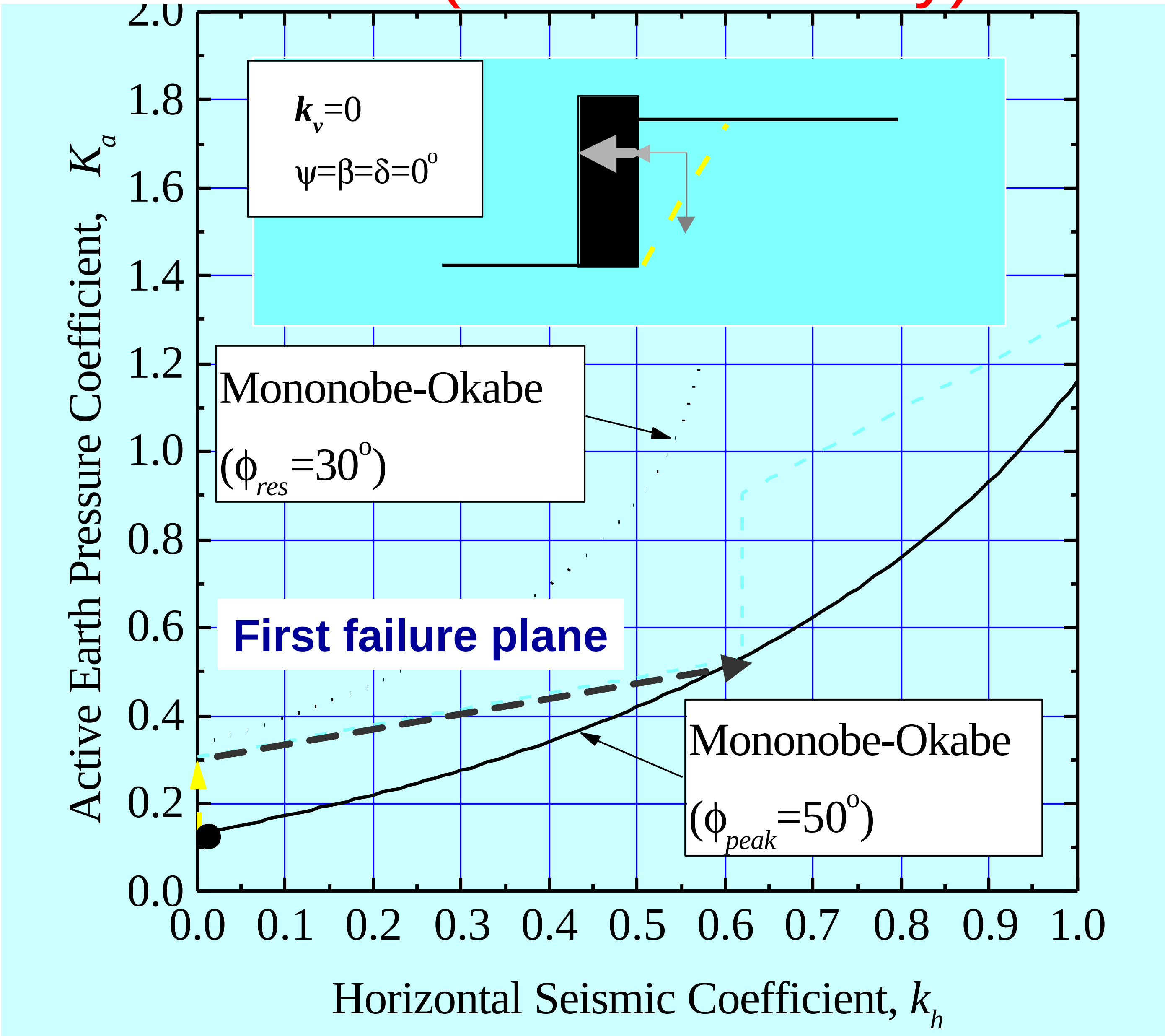
Failure zone vs. seismic coefficient (modified theory)



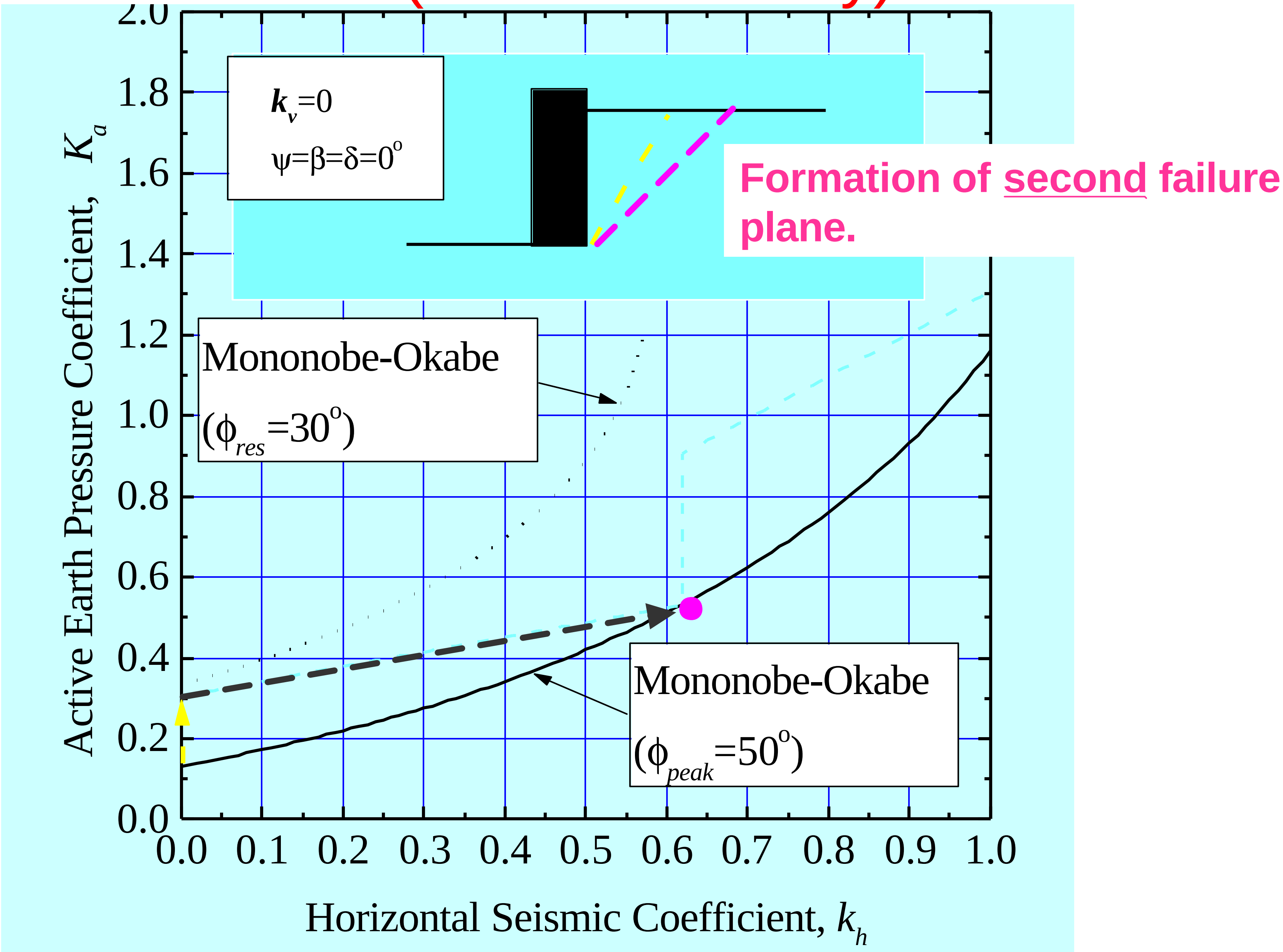
Adopted in Japanese *design standard for railway* foundations and soil retaining structures (1997)

Reduced size of failure zone

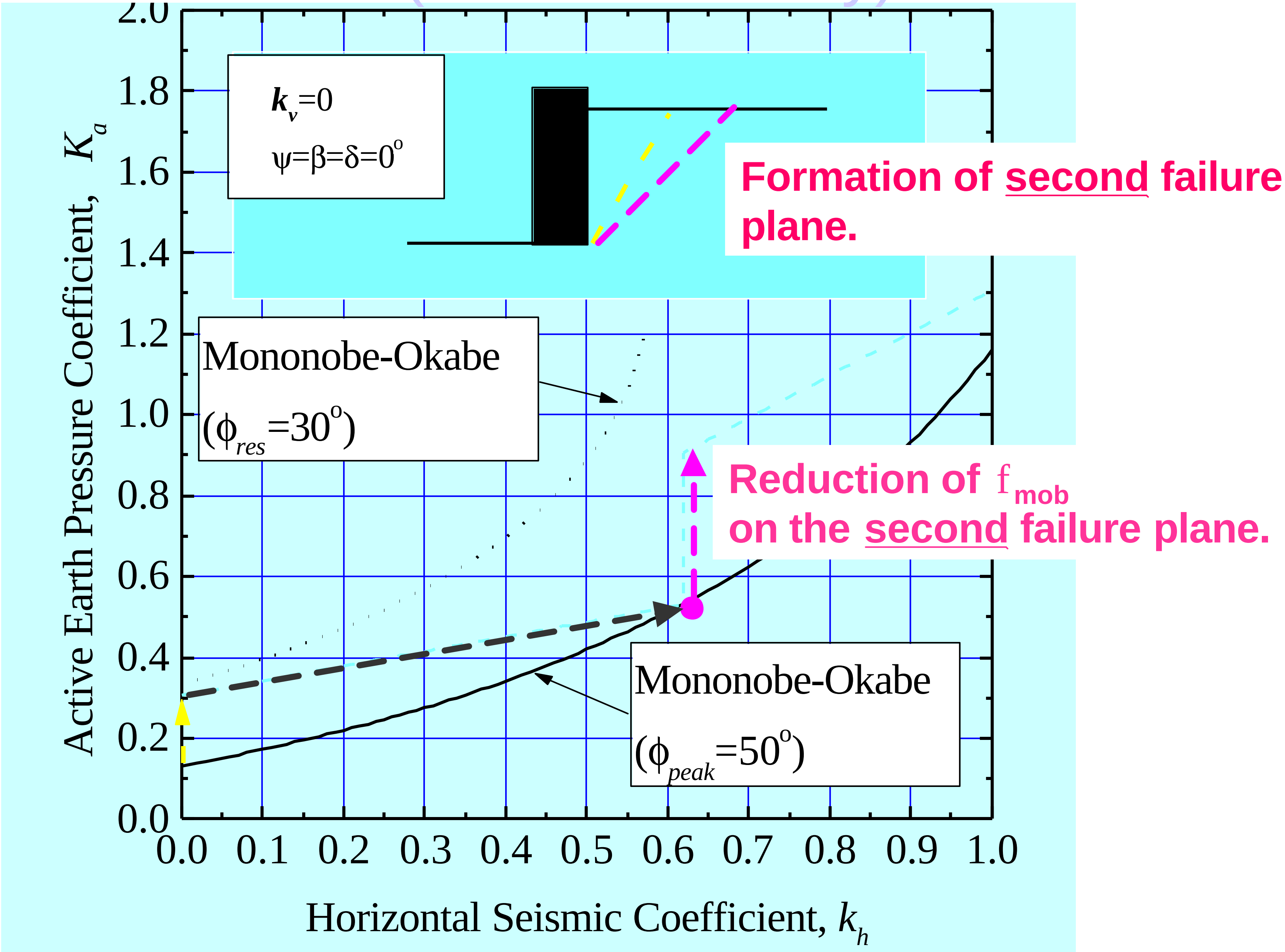
Failure zone vs. seismic coefficient (modified theory)



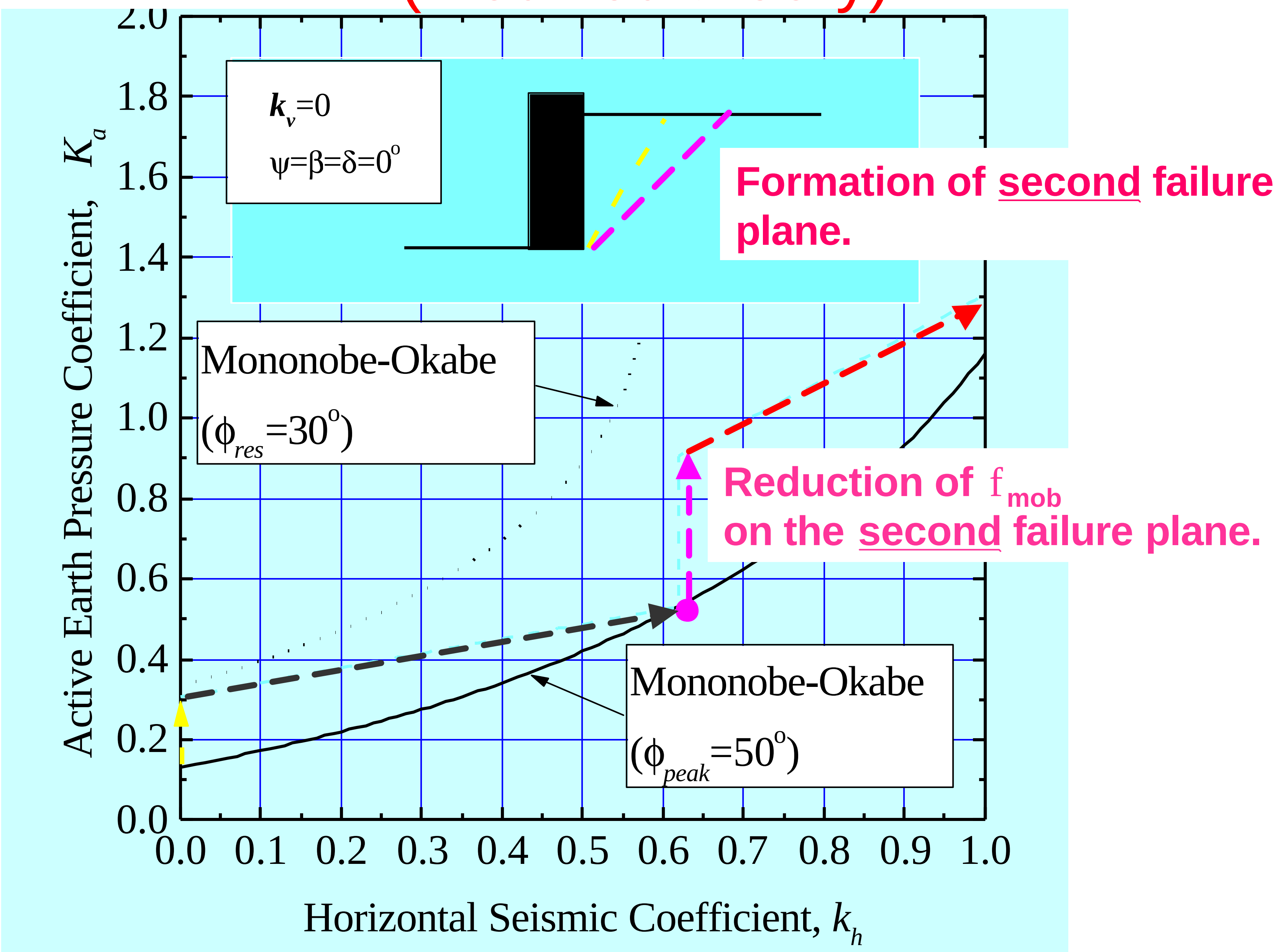
Failure zone vs. seismic coefficient (modified theory)



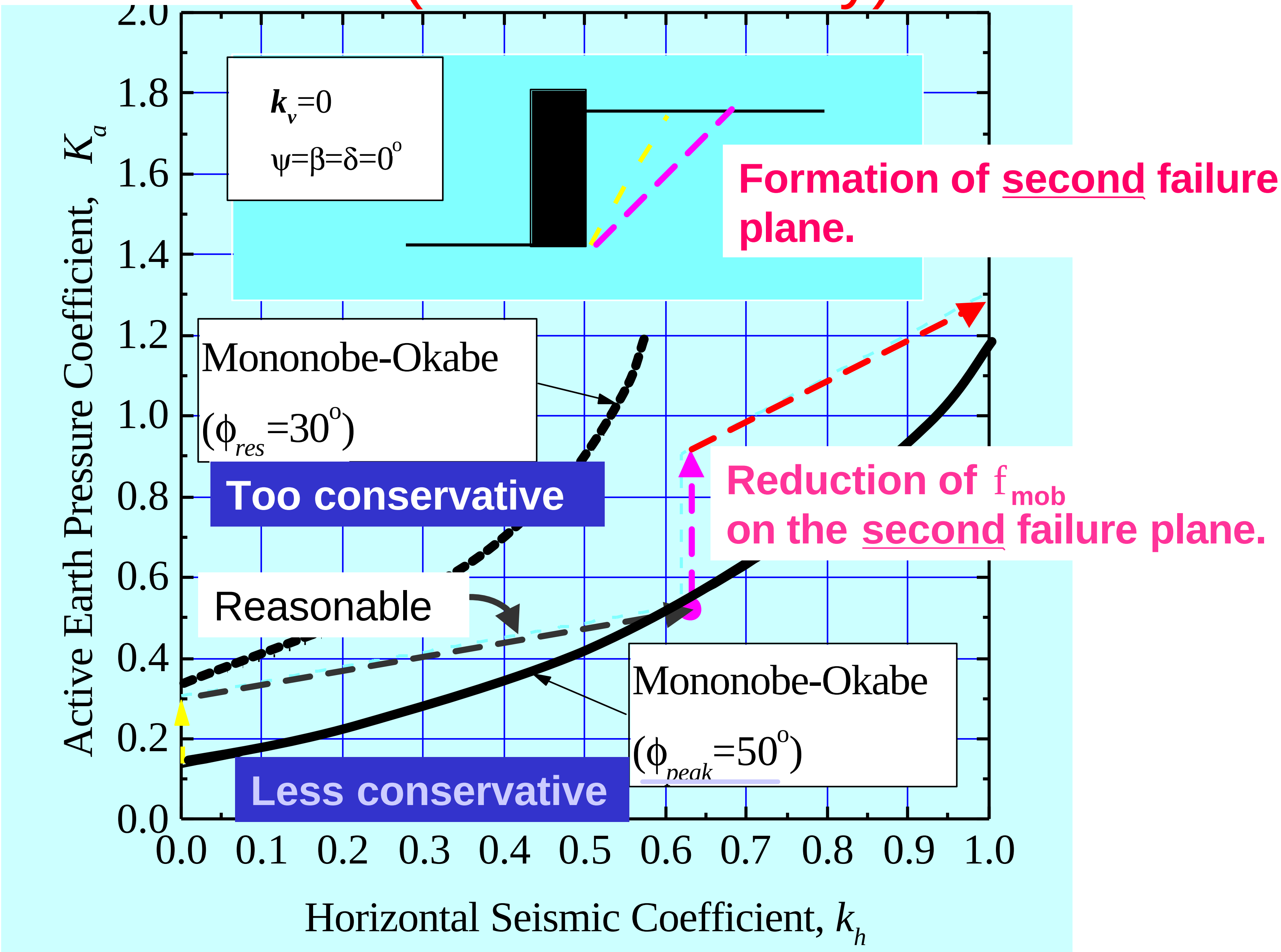
Failure zone vs. seismic coefficient (modified theory)



Failure zone vs. seismic coefficient (modified theory)



Failure zone vs. seismic coefficient (modified theory)

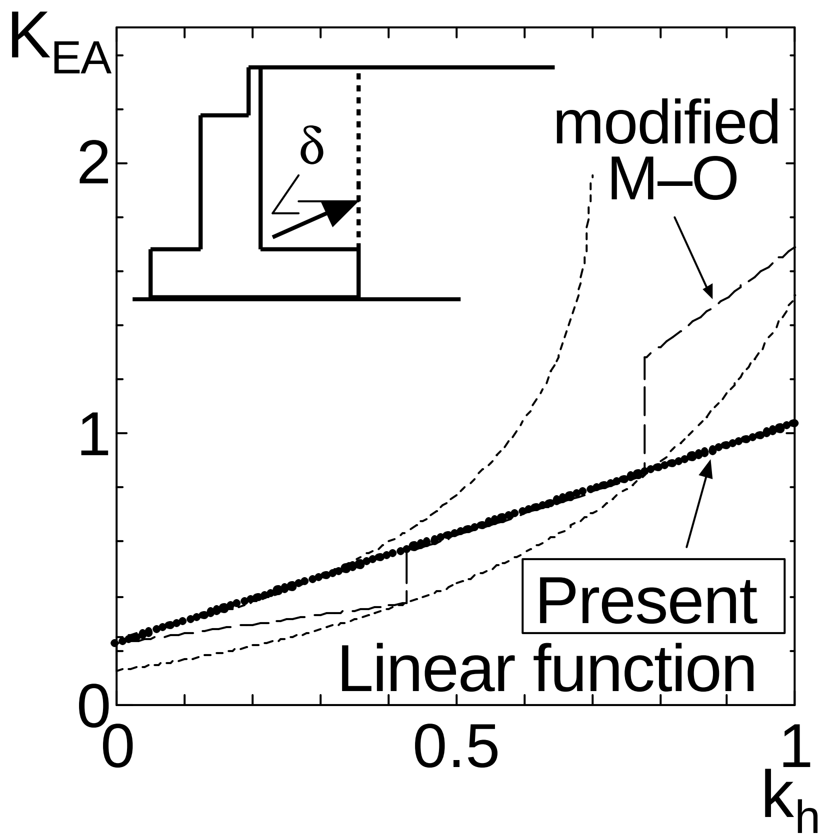


Adopted in Japanese ***design standard for highway bridges*** (2002) after simplification

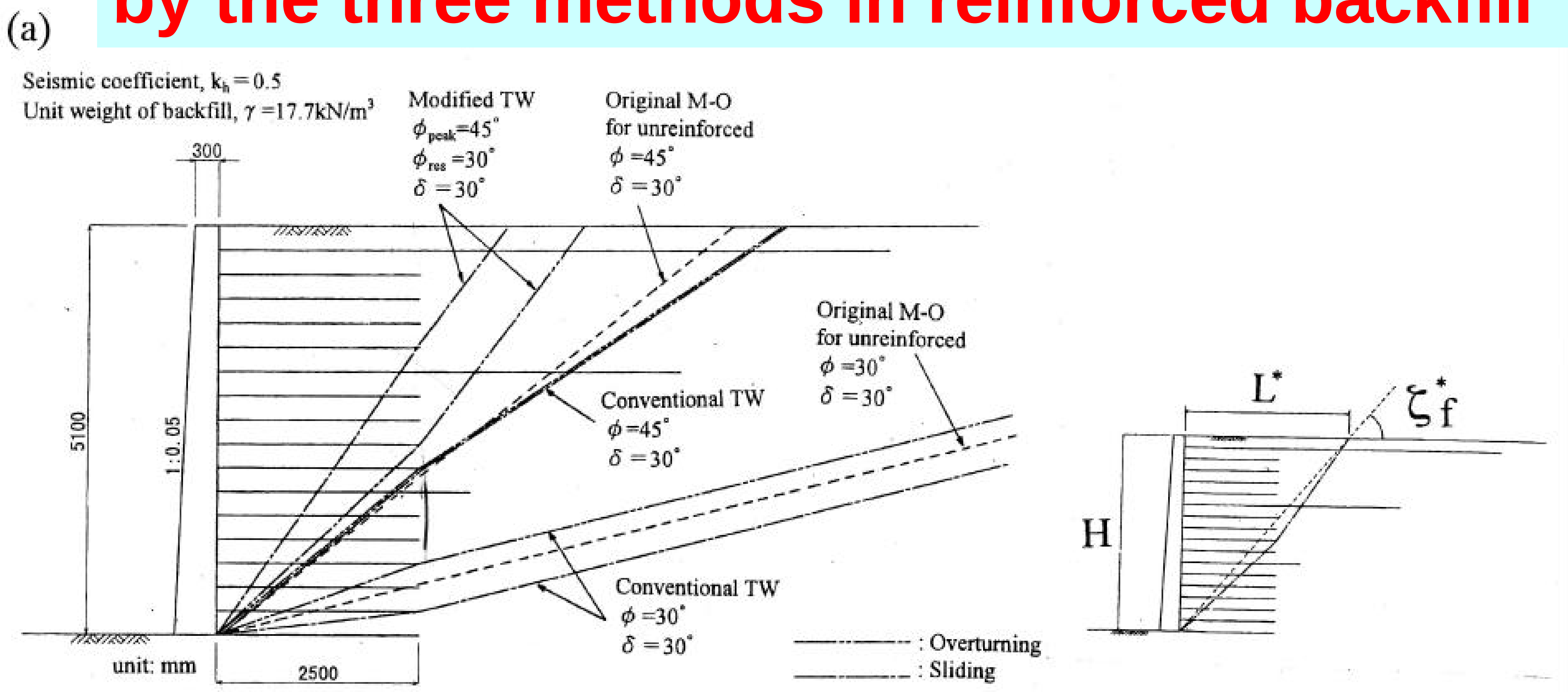
Only the 2nd failure plane is taken into account in estimating K_{EA} for all the values of k_h.

K_{EA} : Linear function w.r.t. k_h

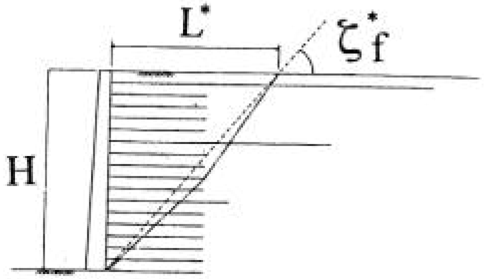
- **When the 3rd failure plane appears, the backfill should have deformed largely, and the seismic earth pressure may not increase.**

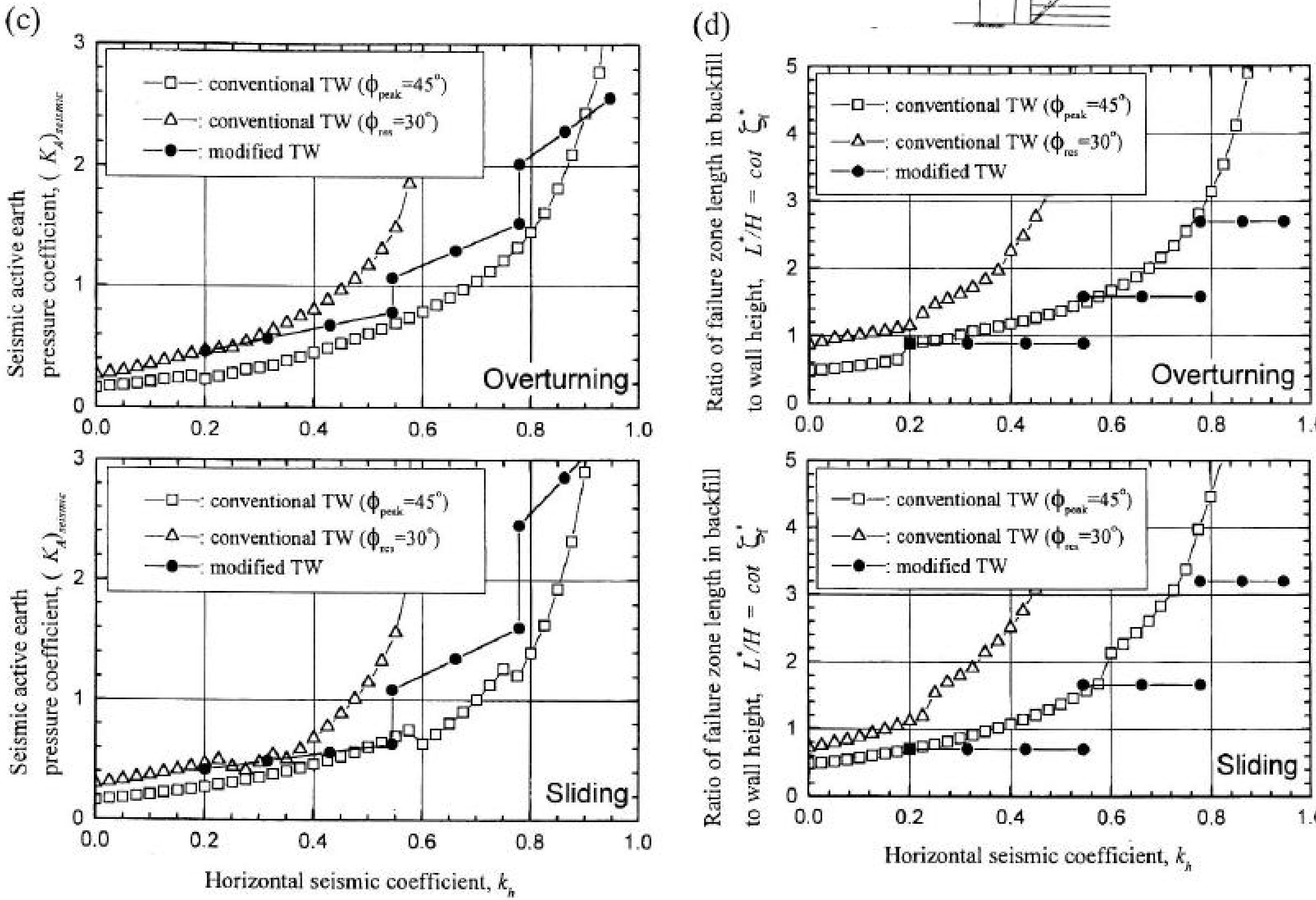


Critical failure planes (i.e., shear bands) by the three methods in reinforced backfill



Comparison between conventional TW method and the new method, $(k_h)_A = 0.2$





Advantages

Modified Mononobe-Okabe method **considering strain softening and strain localization in backfill:**

1. **Reflects ϕ_{peak} and ϕ_{res} values rationally for different backfill conditions (e.g., effects of compaction, soil gradation etc.).**
2. **Yields reasonable seismic active earth pressure even at high seismic loads.**
3. **Provides a realistic and reduced size of failure zone.**

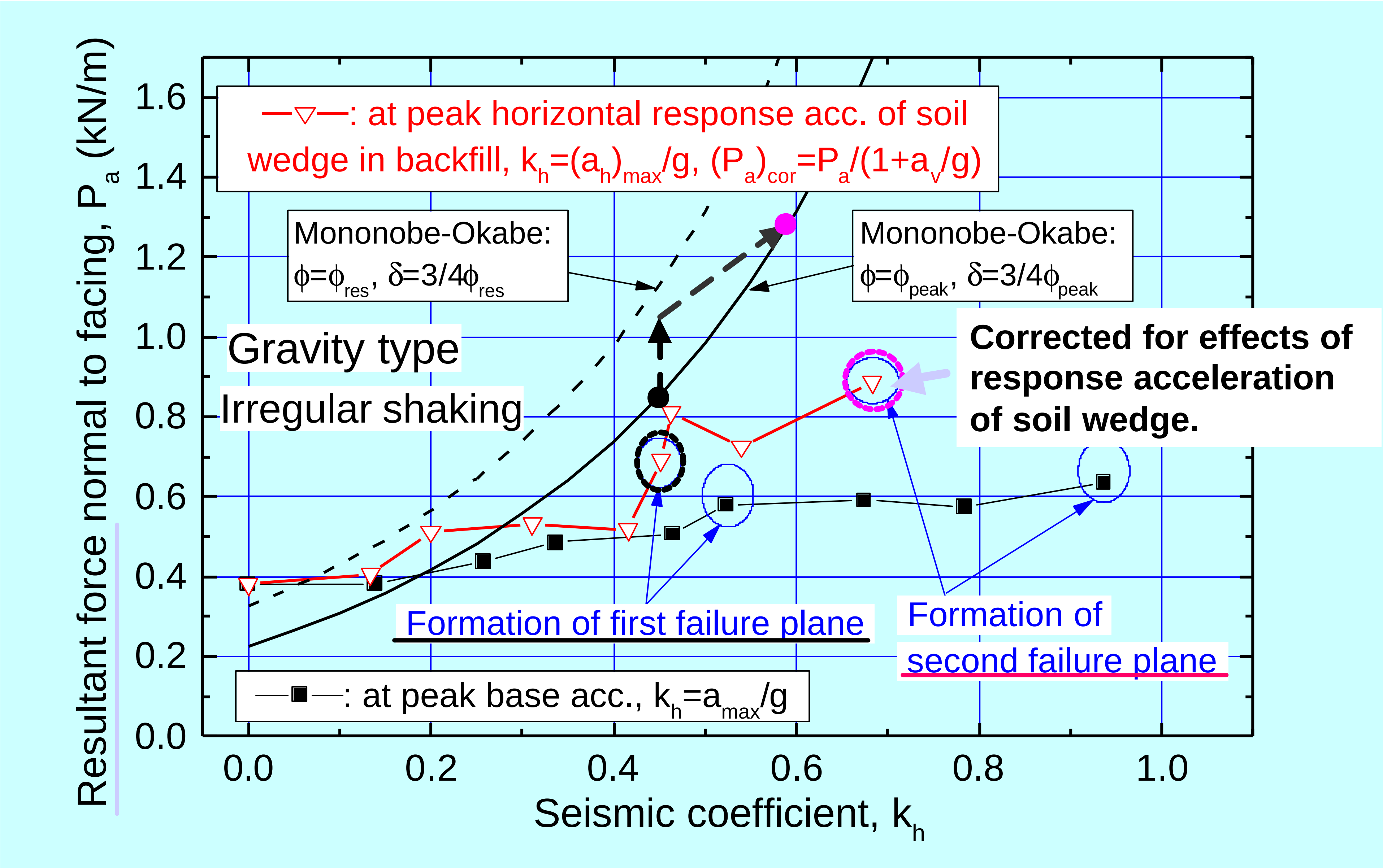
Topics:

3-1 Observed failure pattern of backfill soil

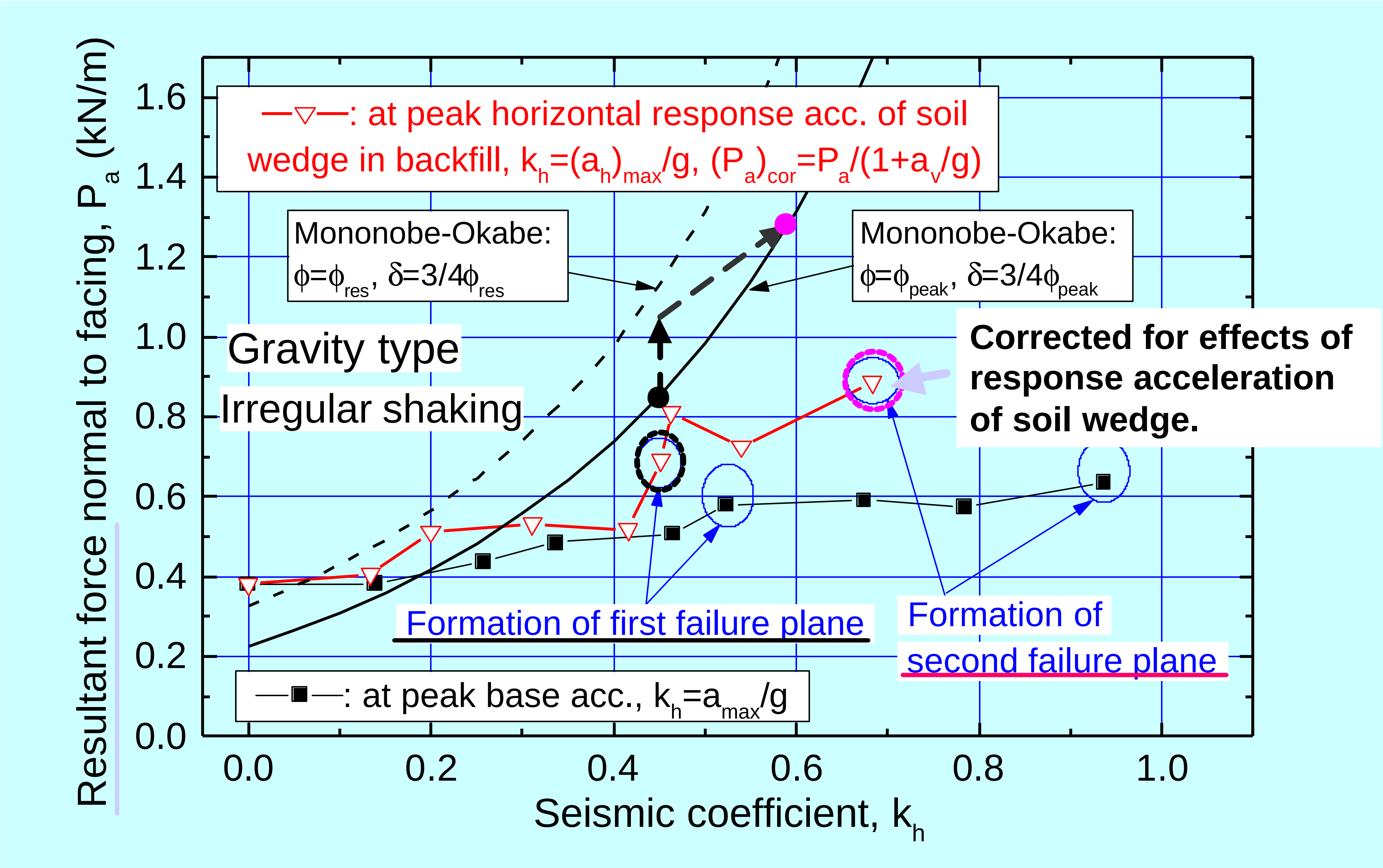
3-2 Modification of *Mononobe-Okabe* method

3-2 Comparison with model test results

Resultant horizontal earth pressure *vs.* seismic coefficient in model shaking tests



The modified M-O; still conservative, because dynamic effects (a large variation of phase inside a RW) are not taken into account.

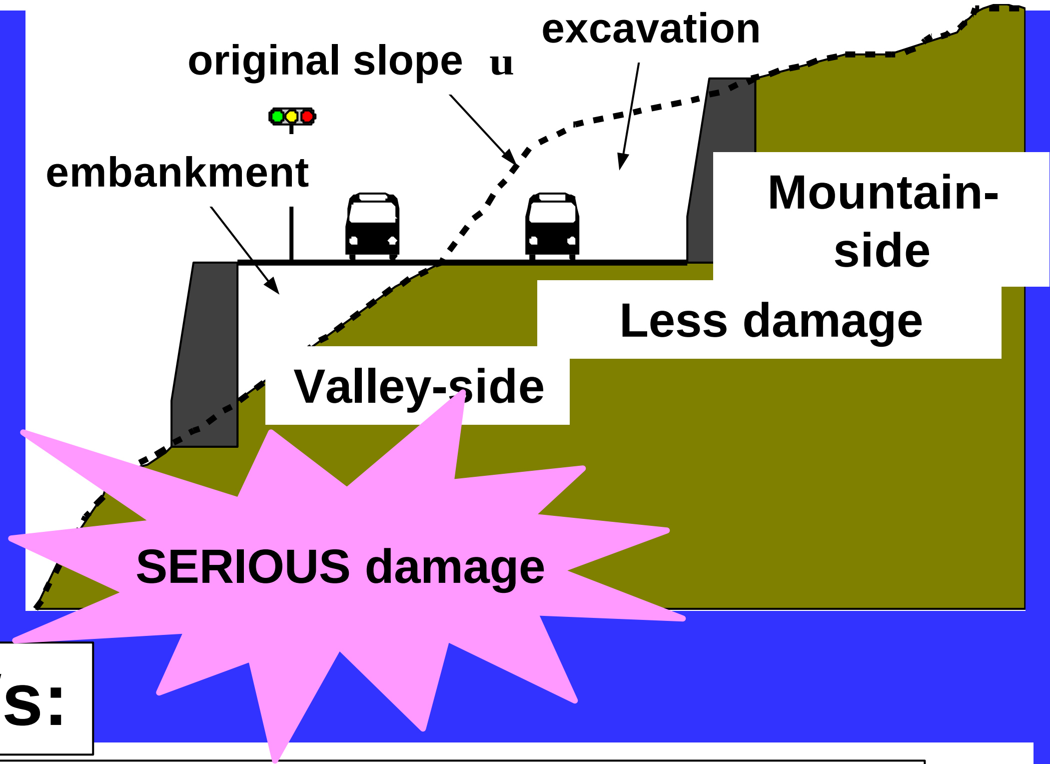


Contents

Background

1995 Hyogoken Nanbu: more serious damage to conventional RWs than reinforced RWs

Chi-Chi(1999): Serious damage to leaning and gravity RWs on slope



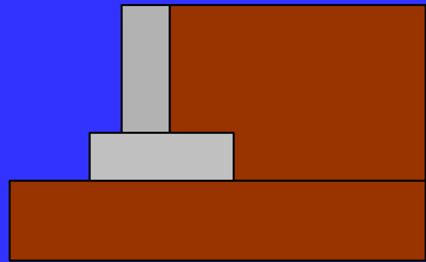
Seismic damage to RWs:

Leaning and Gravity RWs >Reinforced RWs

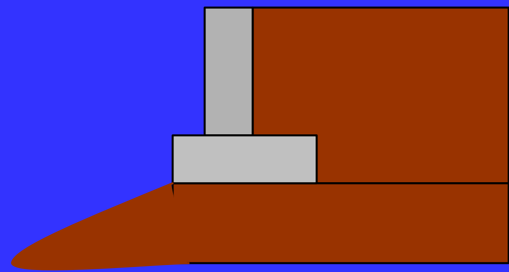
RWs on slope > RWs on level ground

Research prgramme

First stage: shaking table tests of RWs on level ground



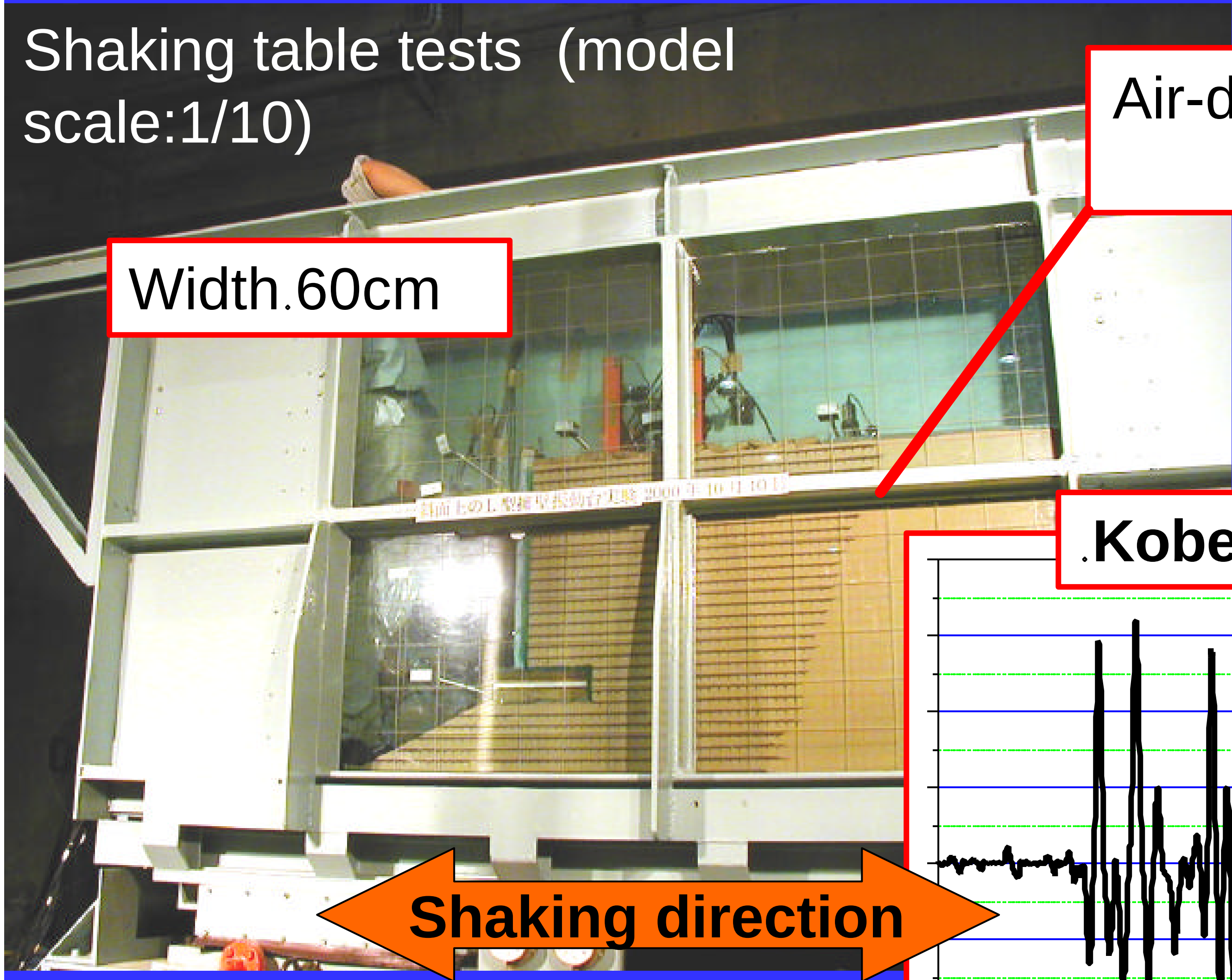
Second stage：　shaking table tests of RWs on slope



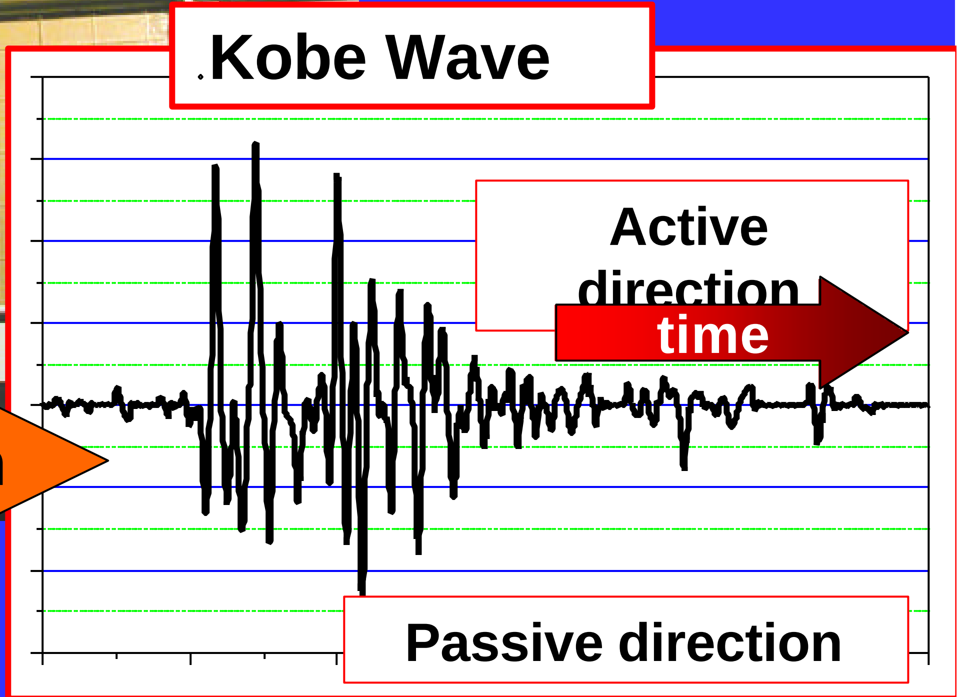
Seismic stability of several types of RWs

Testing method

Shaking table tests (model scale:1/10)

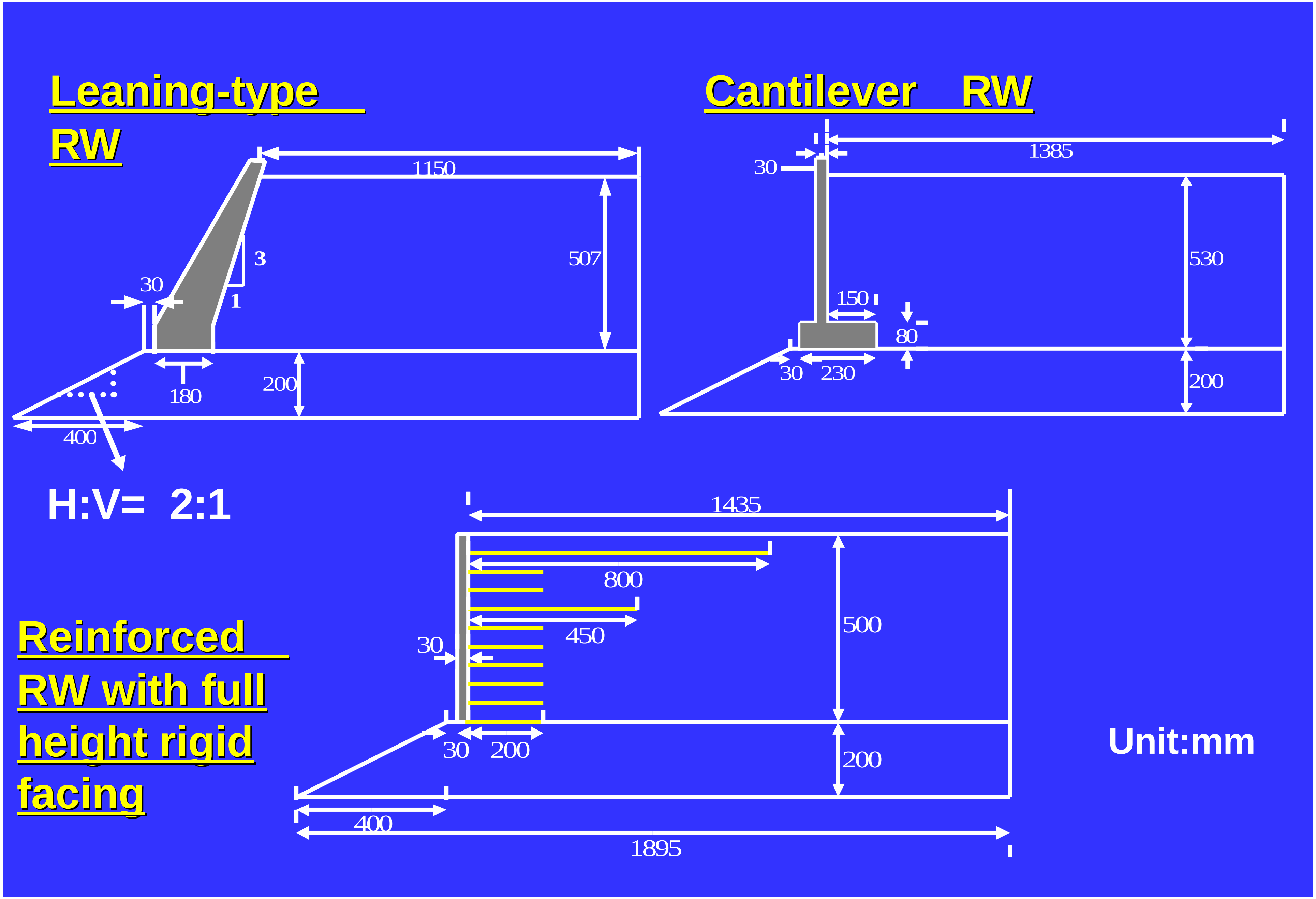


Air-dry Toyoura sand
.D_r= 90 %.

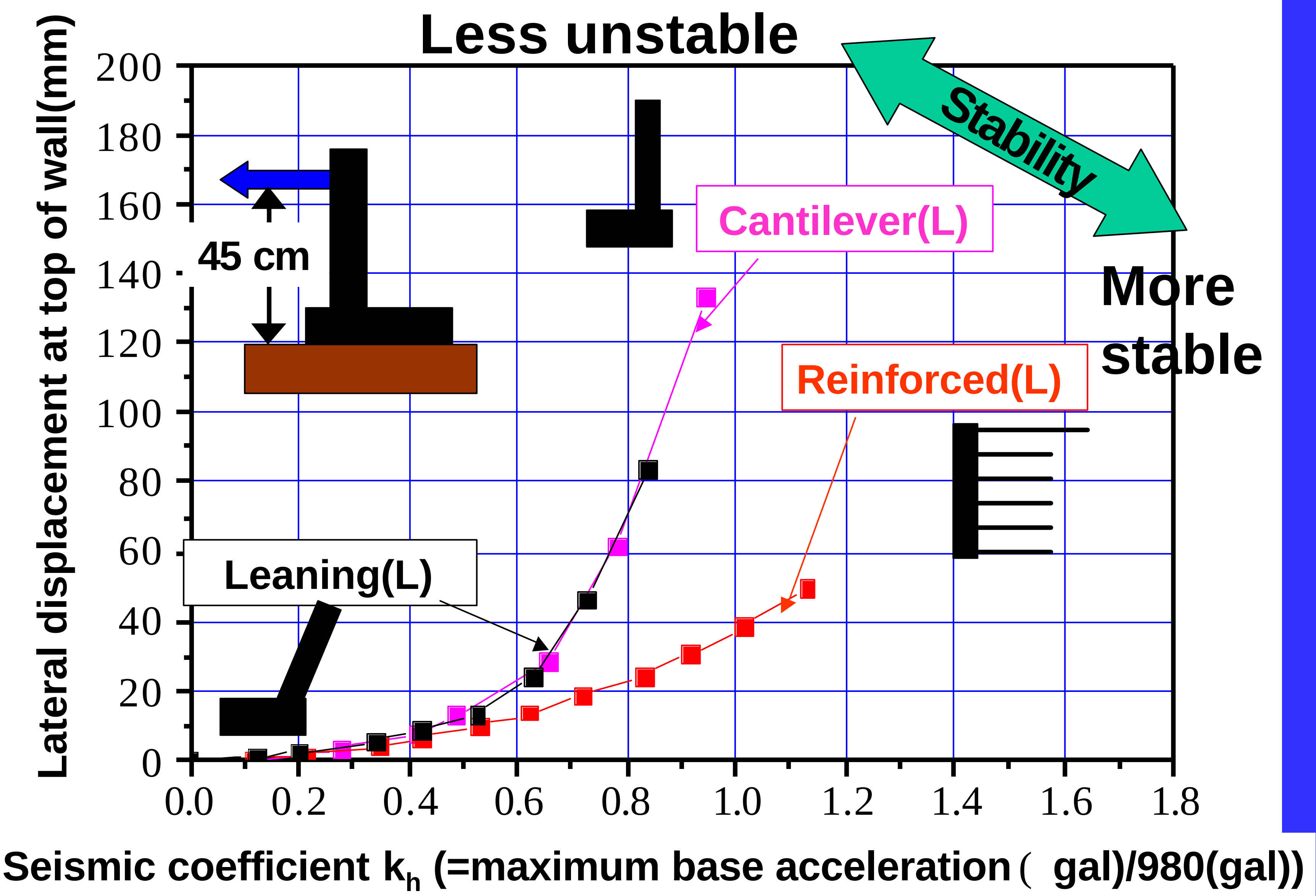


Increment of step :100gal

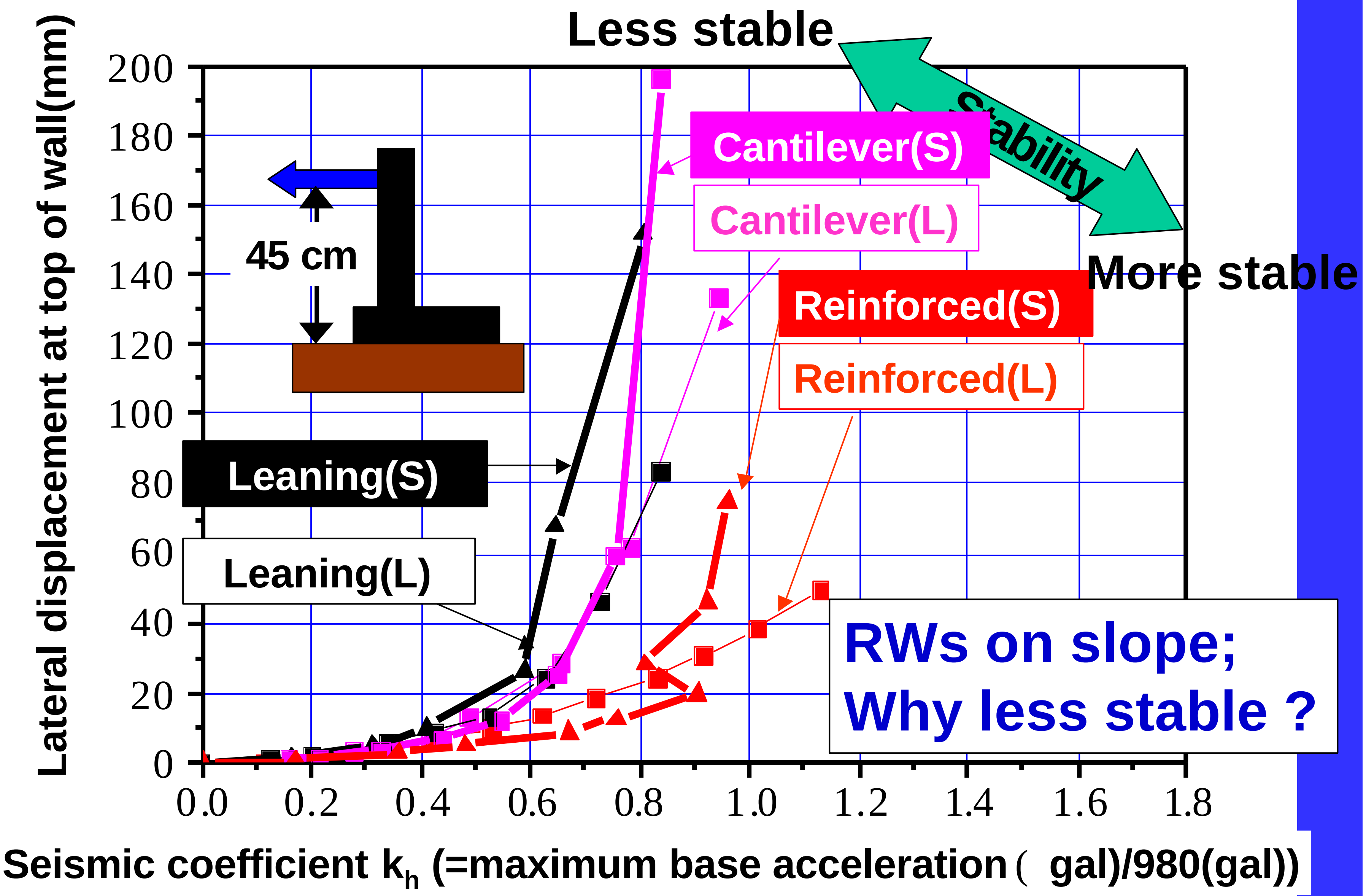
Tested models



Lateral displacement at top of RW on the level ground

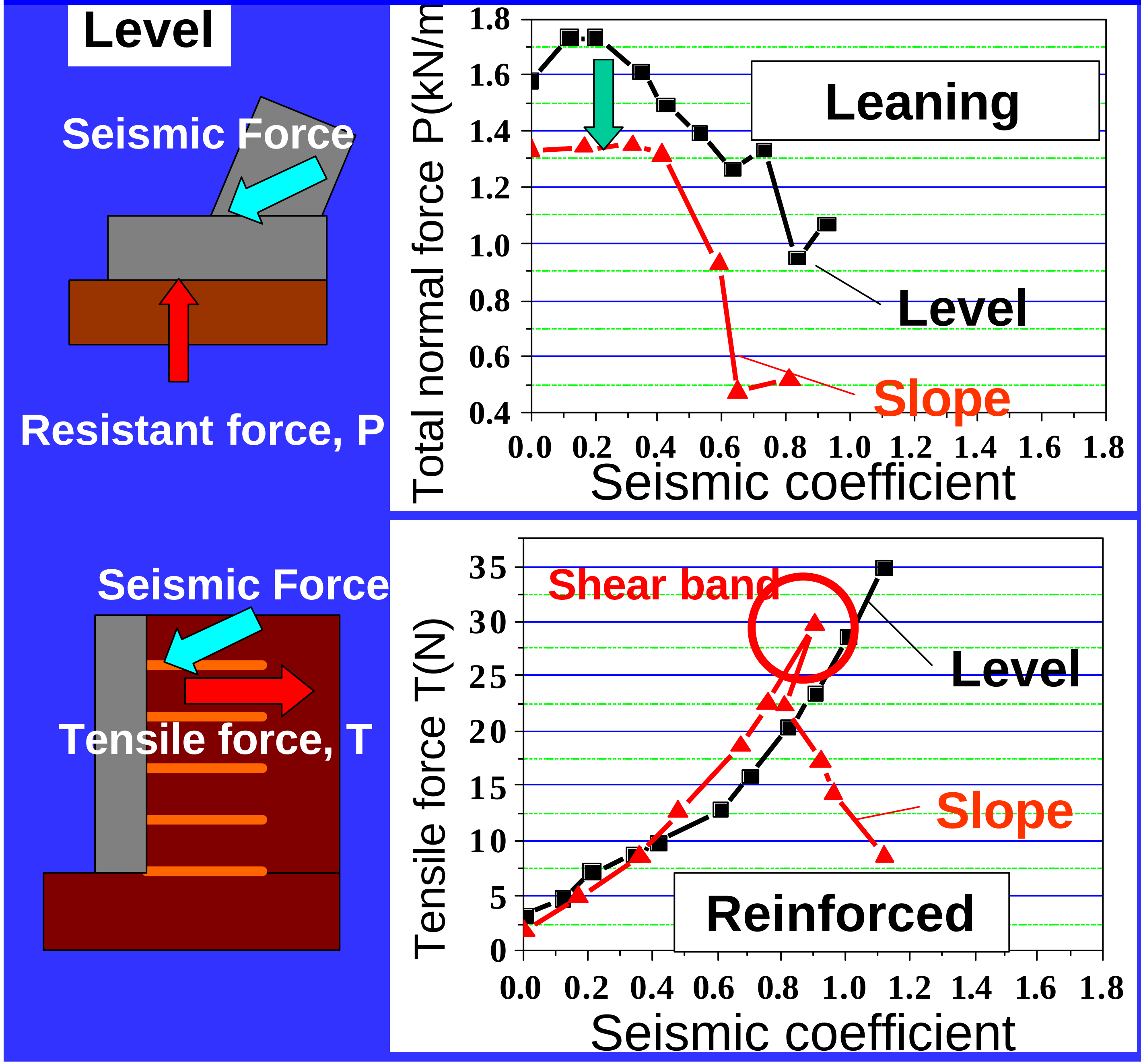


Lateral displacement at top of RW: larger when on slope (S) than when on the level ground (L)

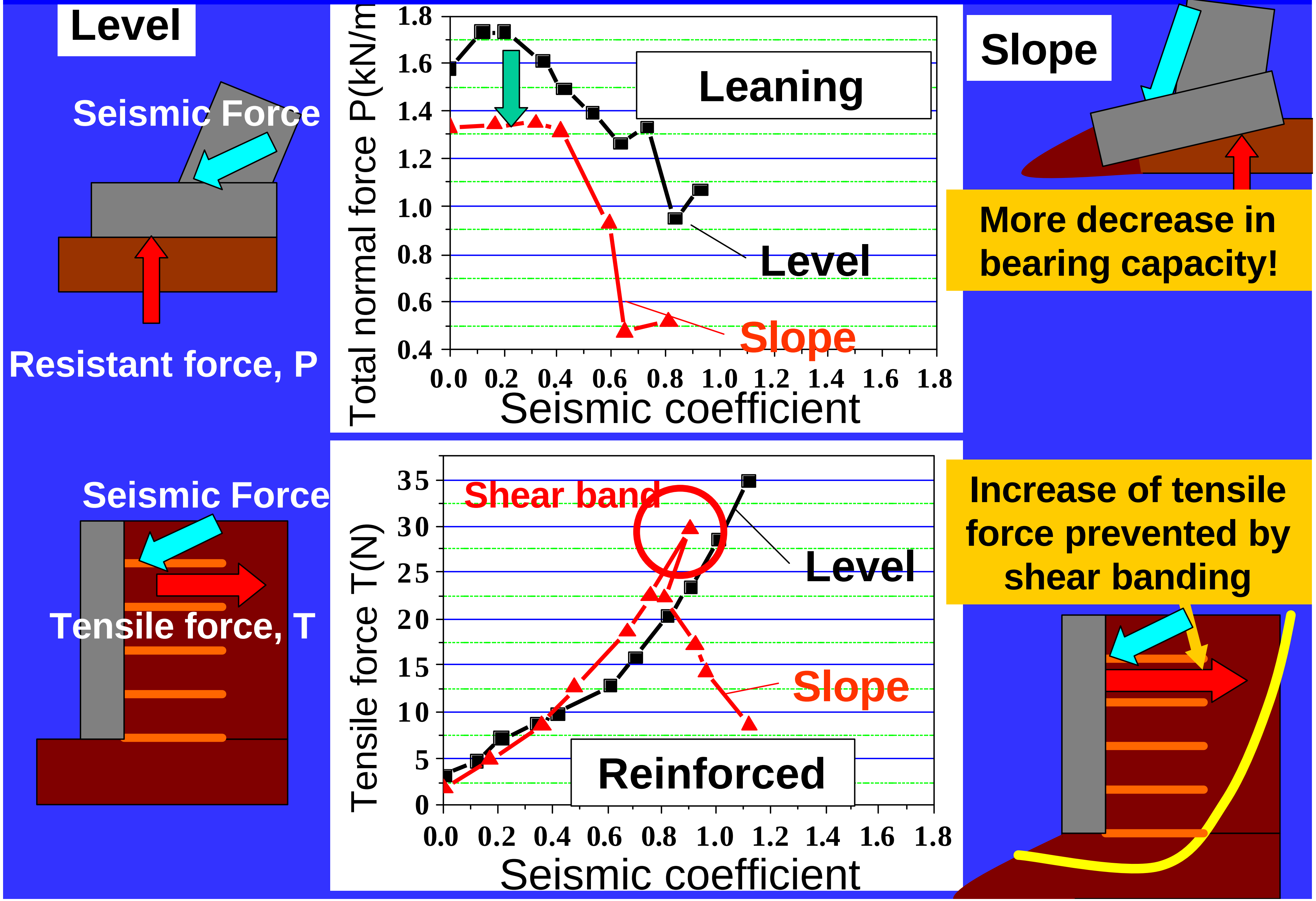


RWs on slope;
Why less stable ?

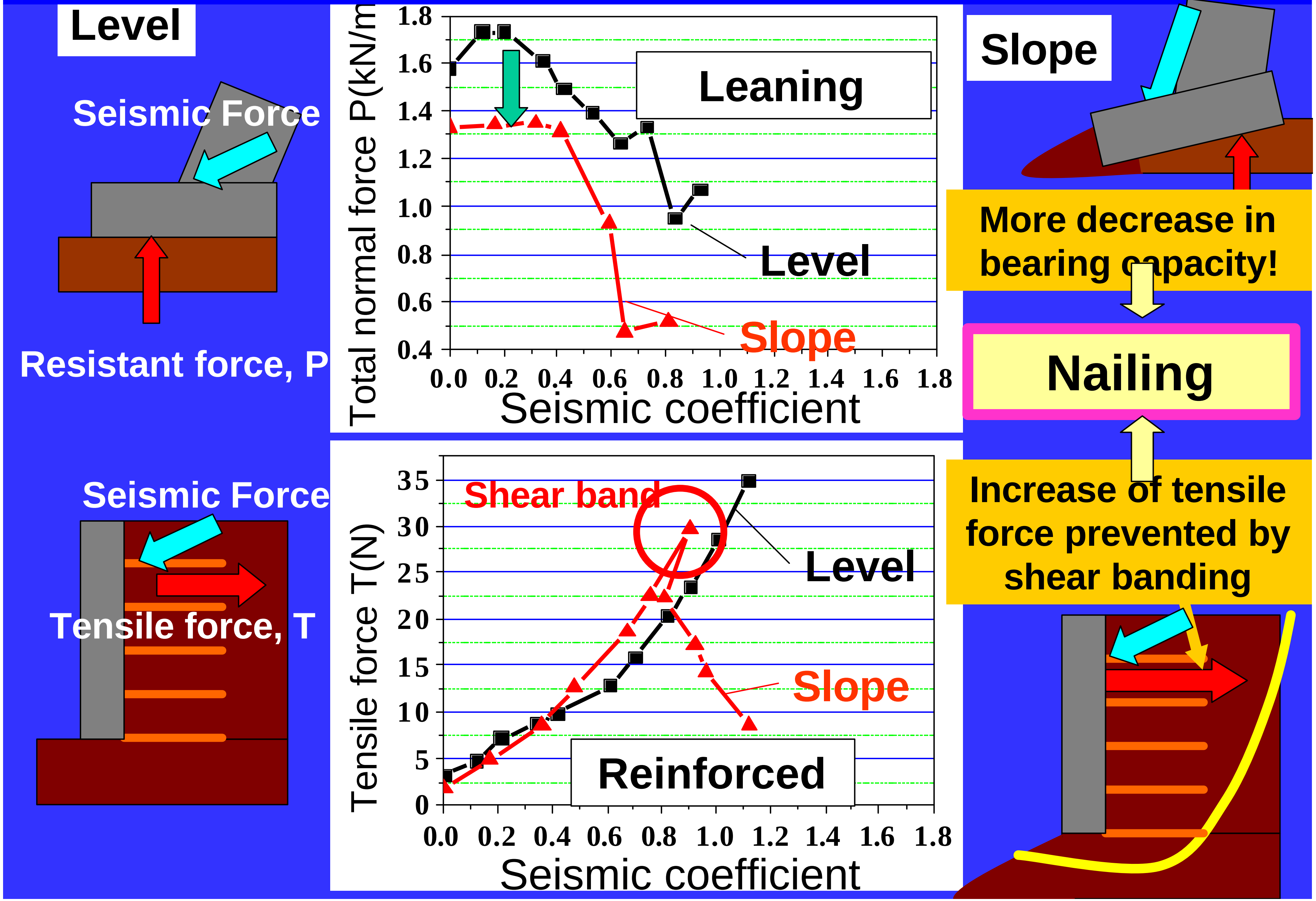
Comparison of resistant force



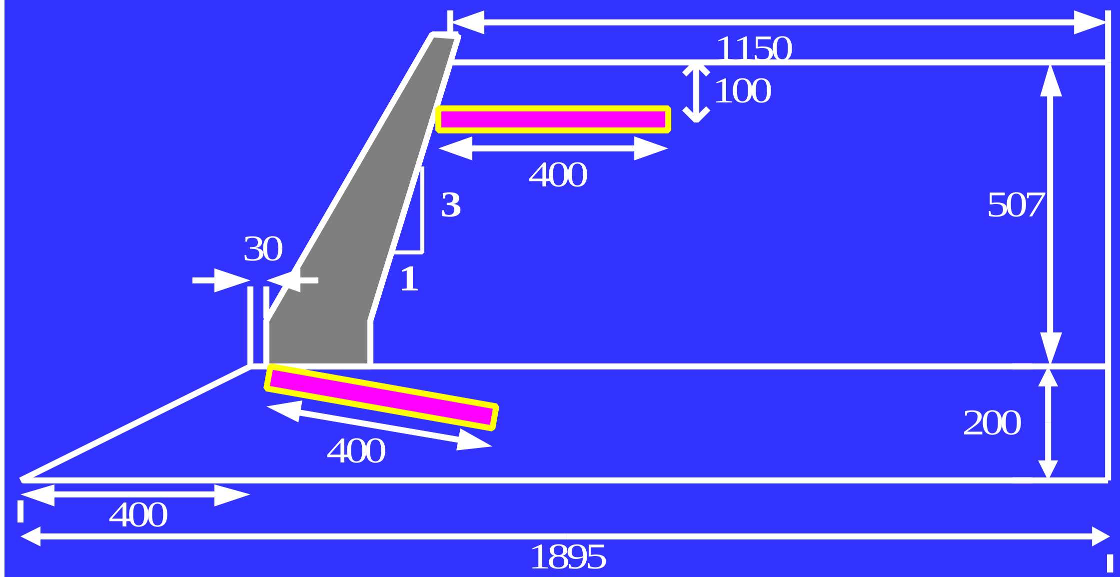
Comparison of resistant force



Comparison of resistant force



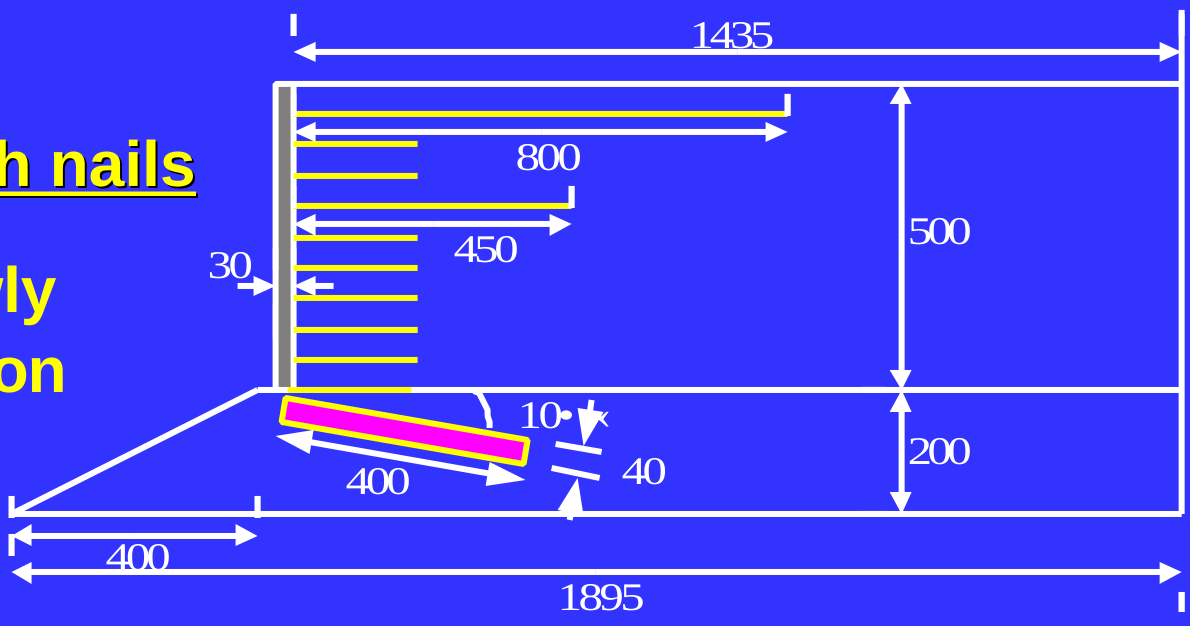
Model RWs types with NAILs



Leaning RWs with nails

Application :
Reinforcement of existing RWs on slope

Unit:mm



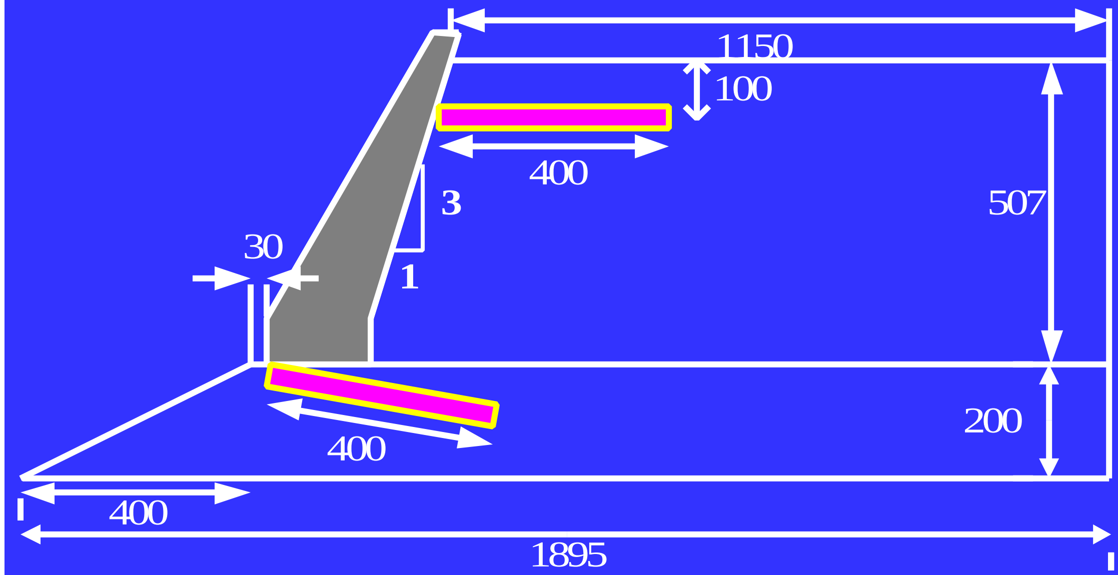
Reinforced RWs with nails

.Application : Newly constructed RWs on slope

Nailing-1

Nailing-2(nails)

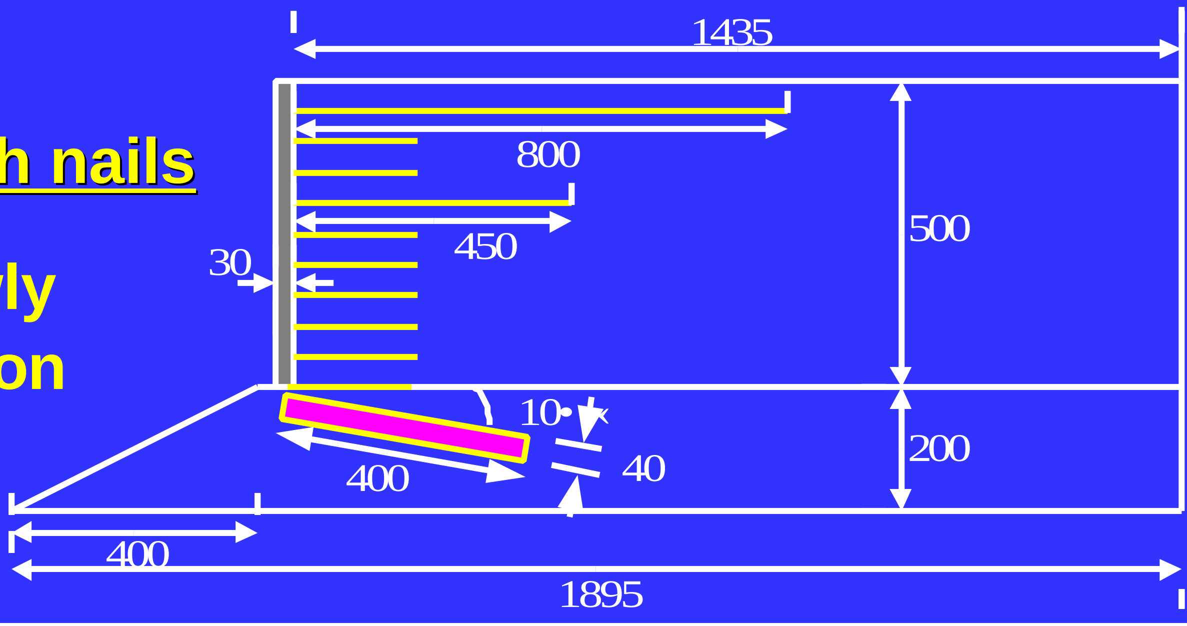
Model RWs types with NAILs



Leaning RWs with nails

Application : Reinforcement of existing RWs on slope

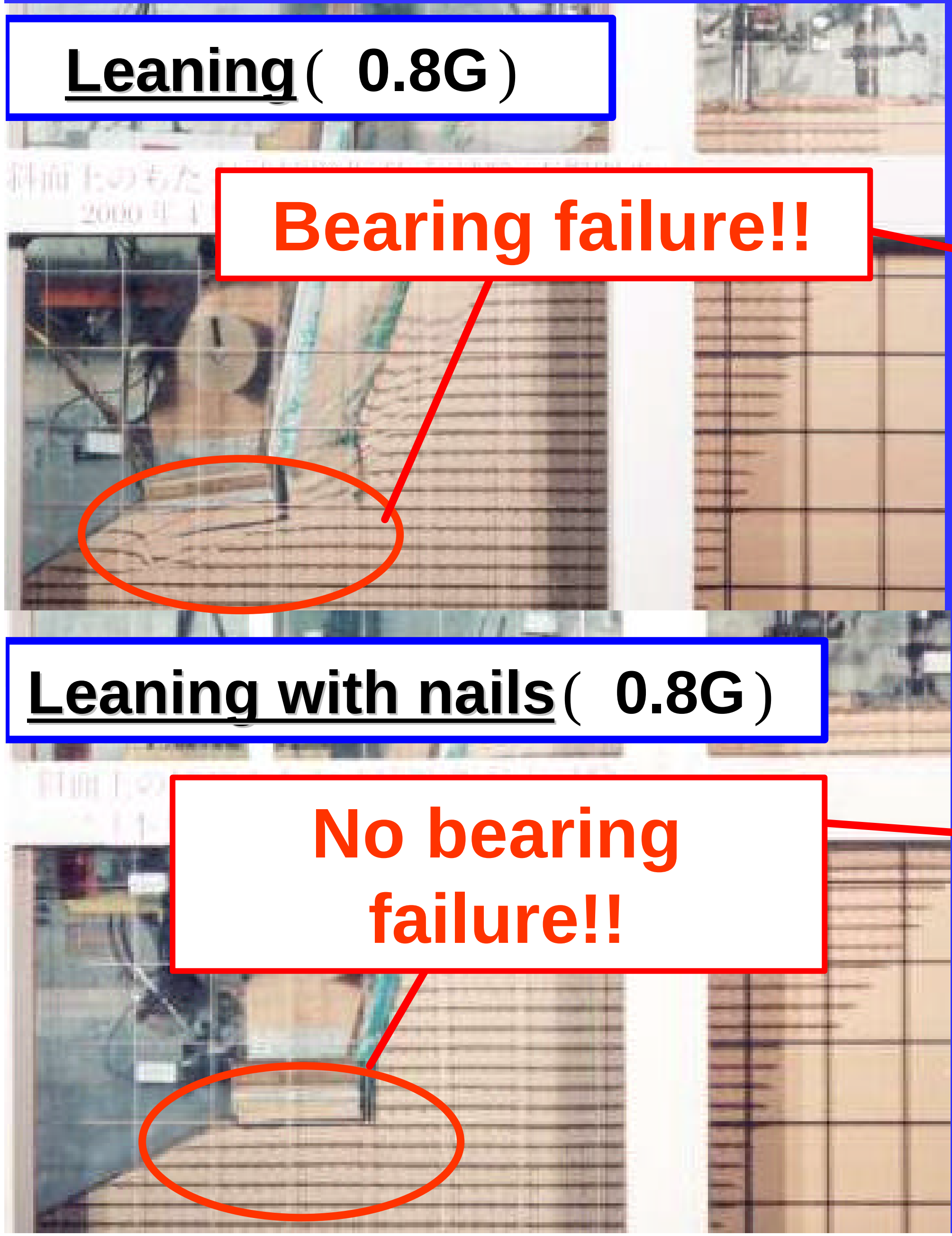
Unit:mm

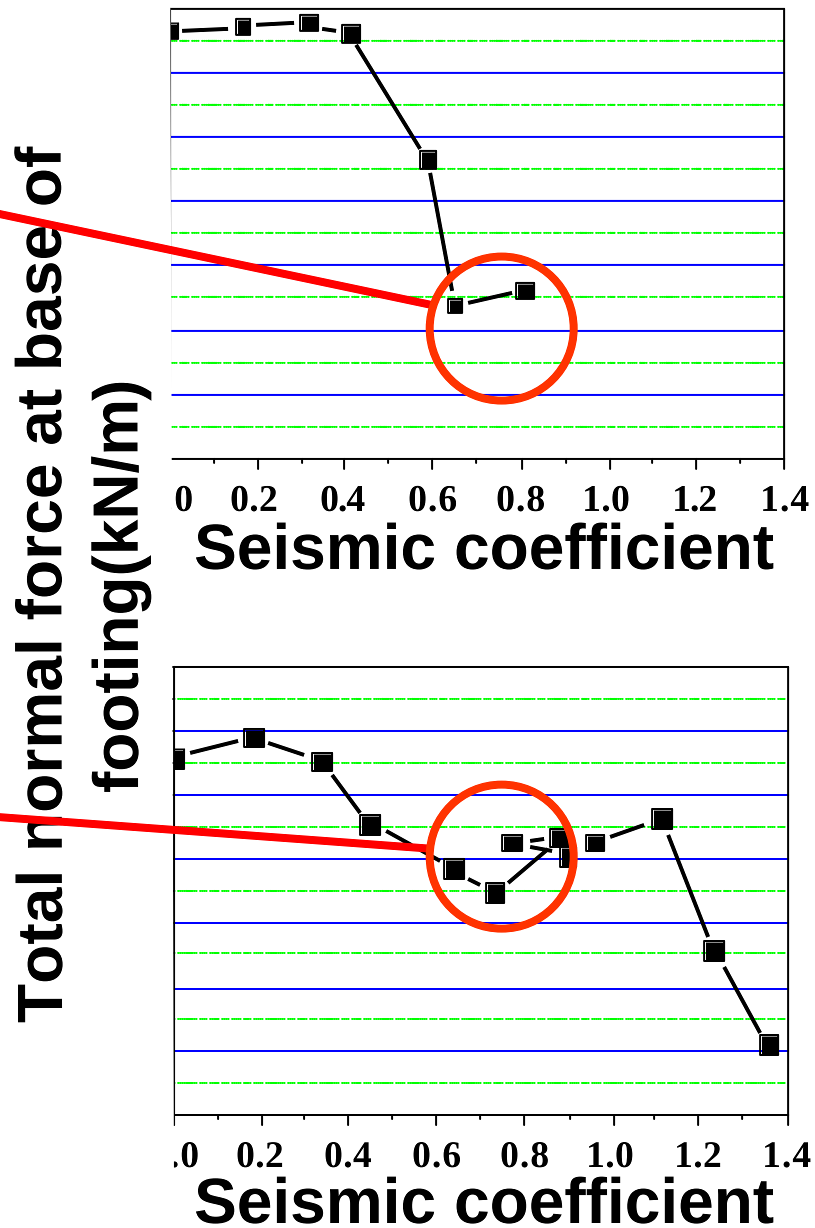


Reinforced RWs with nails

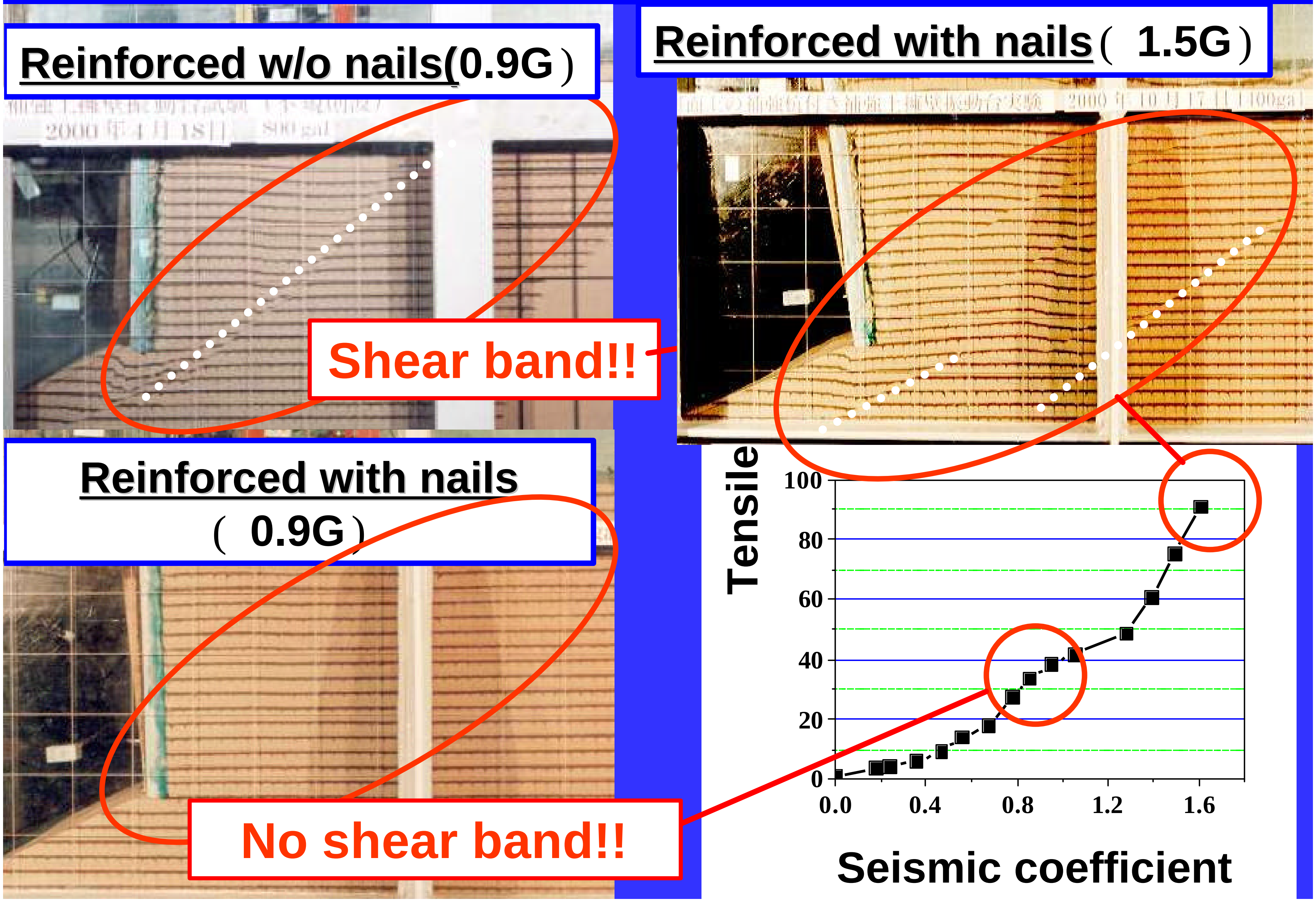
.Application : Newly constructed RWs on slope

Effect of nailing（ leaning type RW）

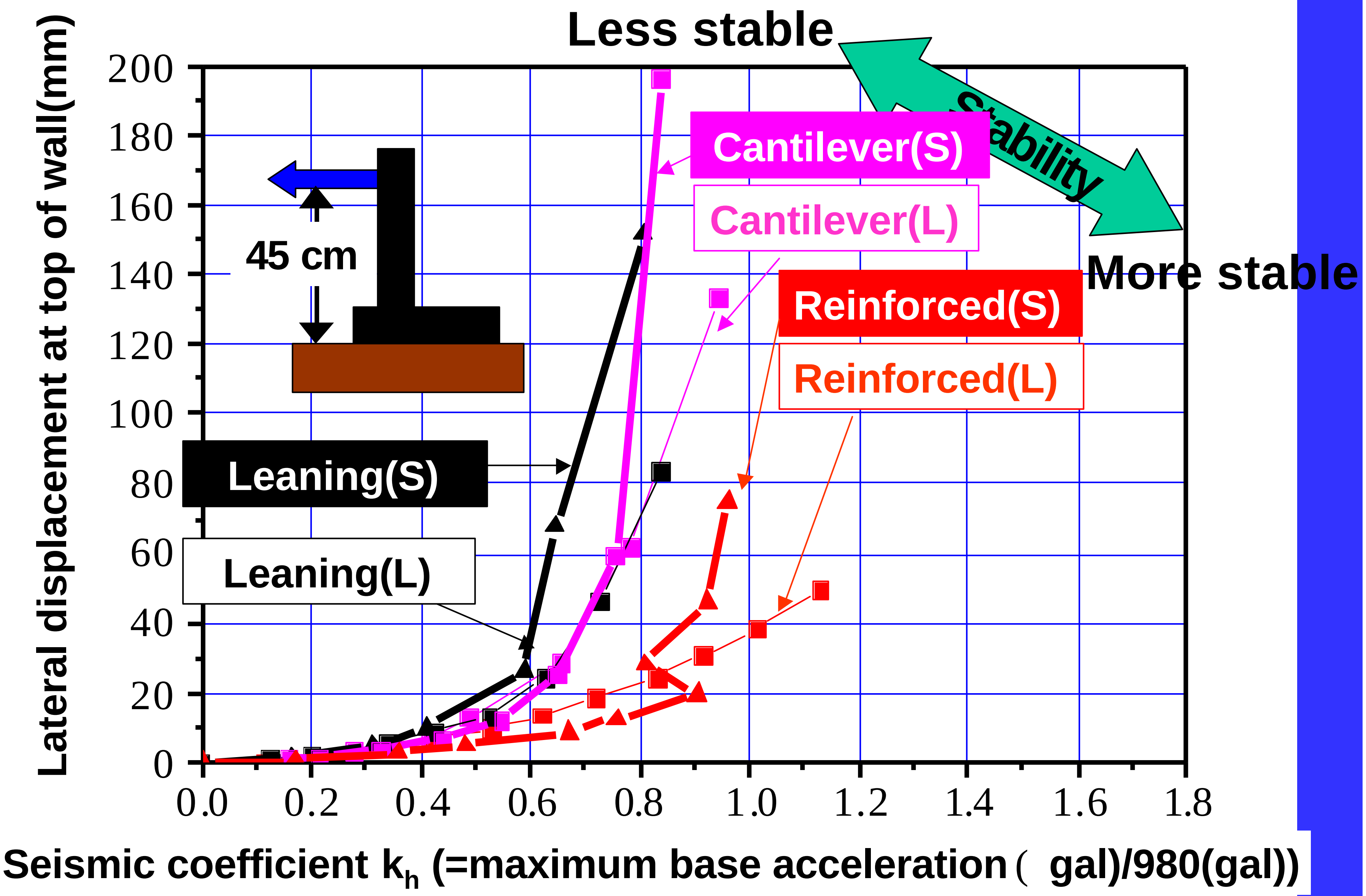




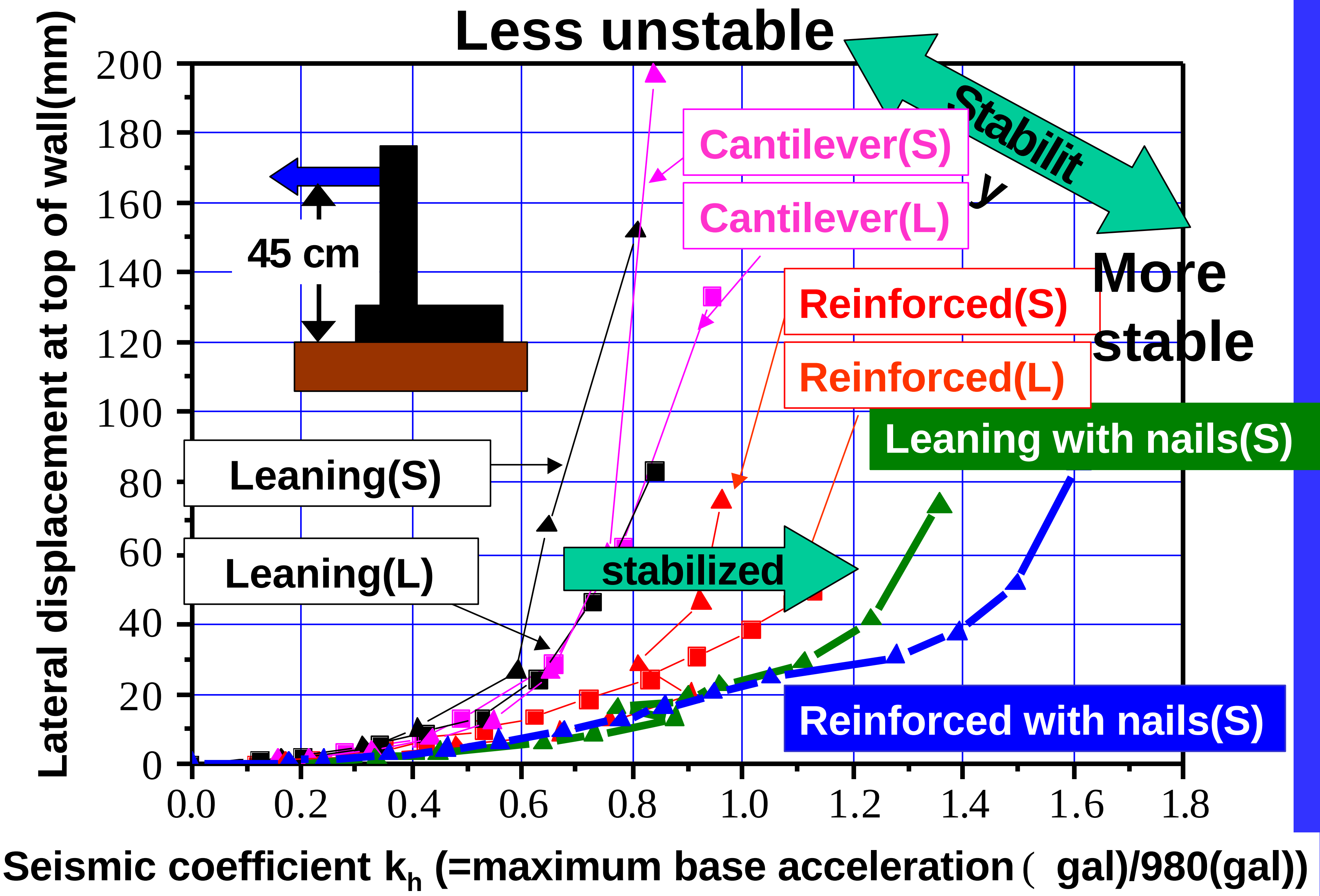
Effect of nailing (reinforced RW)



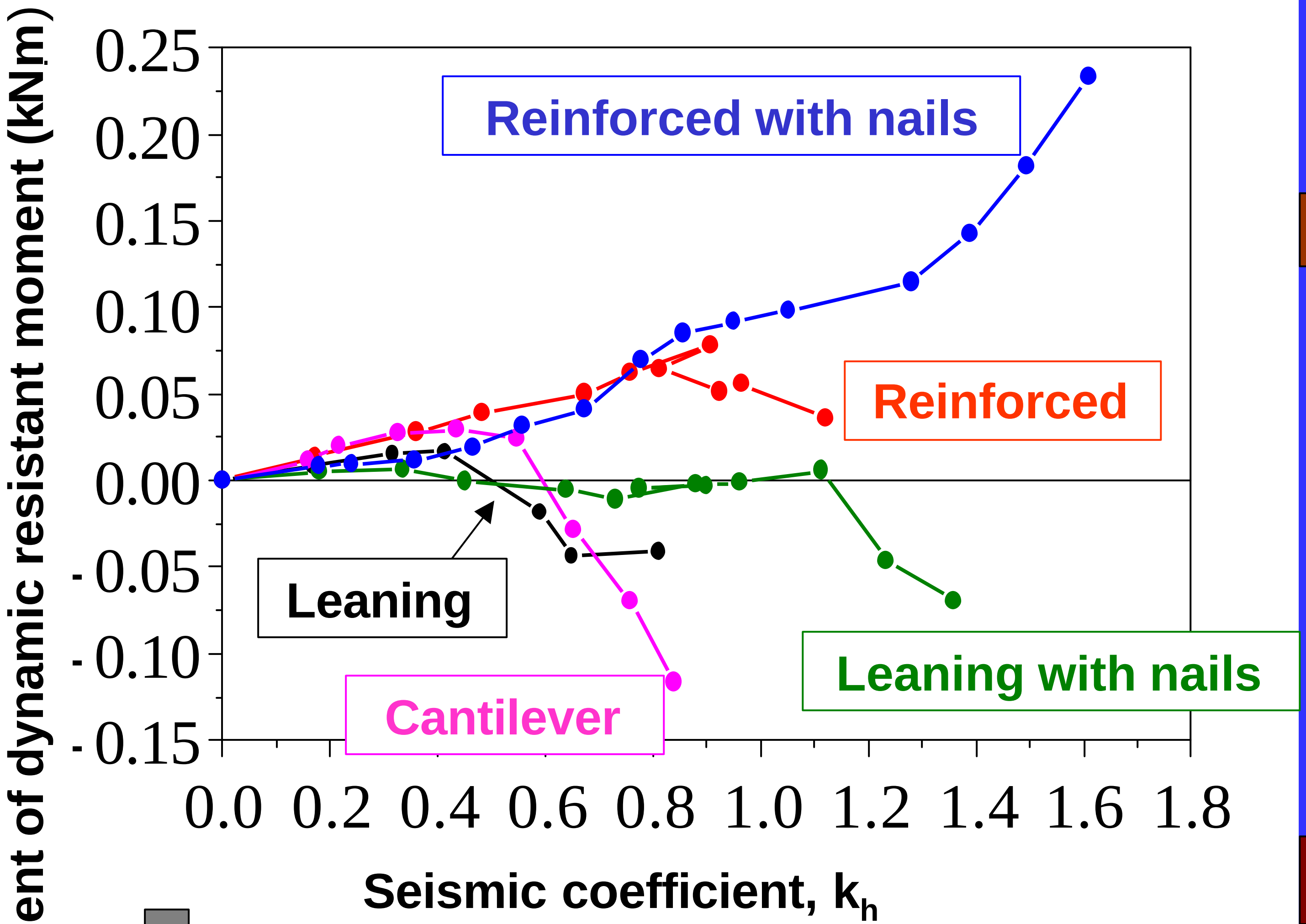
Lateral displacement at top of RW

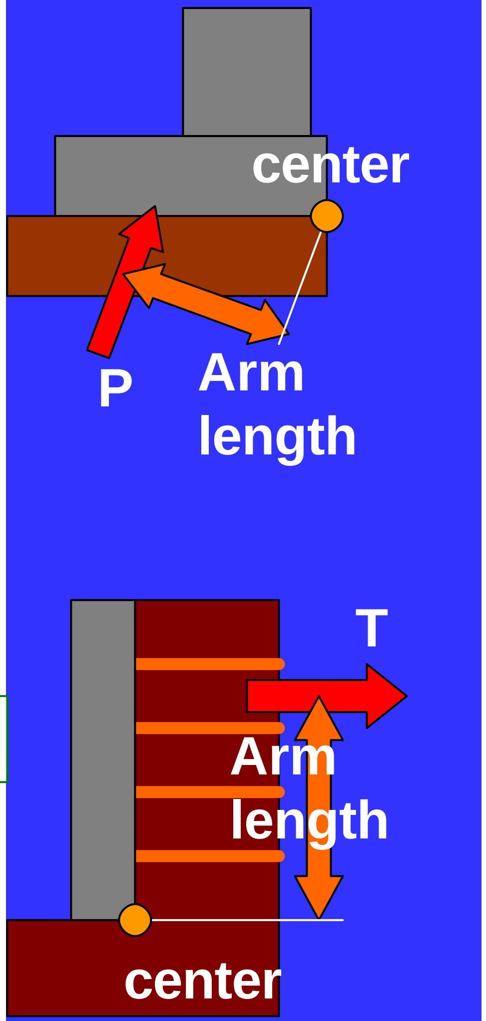


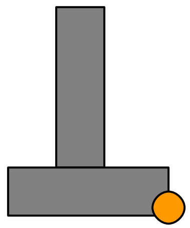
Lateral displacement at top of RW



Dynamic resistant moment







Fs = Resistant moment / Rotation moment

Increment of this part

Summary of model shaking table tests

3-1 The reinforced soil RWs showed more ductile and higher seismic-resistance than conventional RWs.

3-2 The seismic stability of RWs on slope is much lower than those on the level ground, due to:

a) low bearing capacity of slope for conventional RWs; and

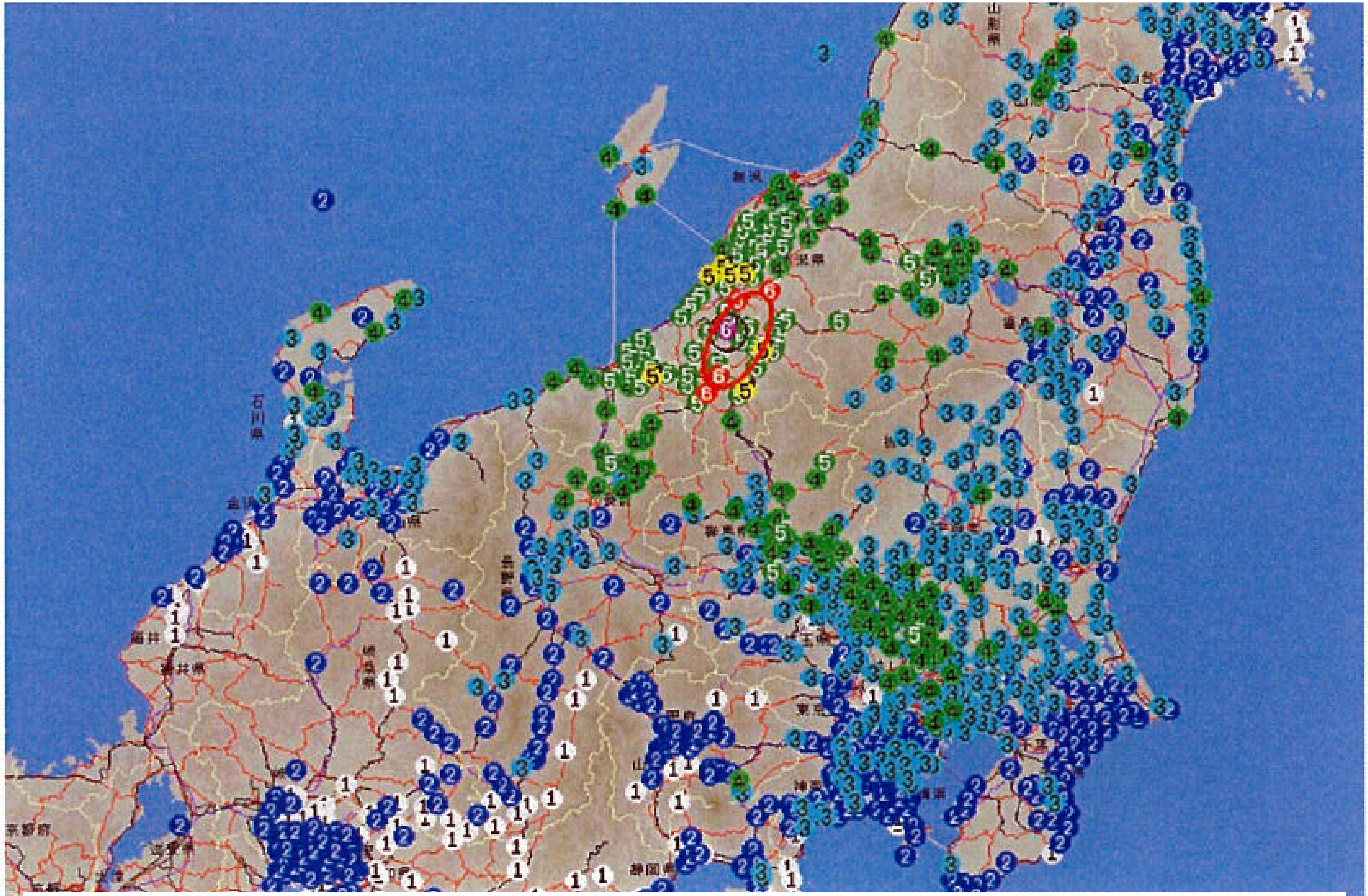
b) premature development of shear band for reinforced RWs on slope with relatively short reinforcement.

3-3 Nailing can reinforce RWs on slope effectively.

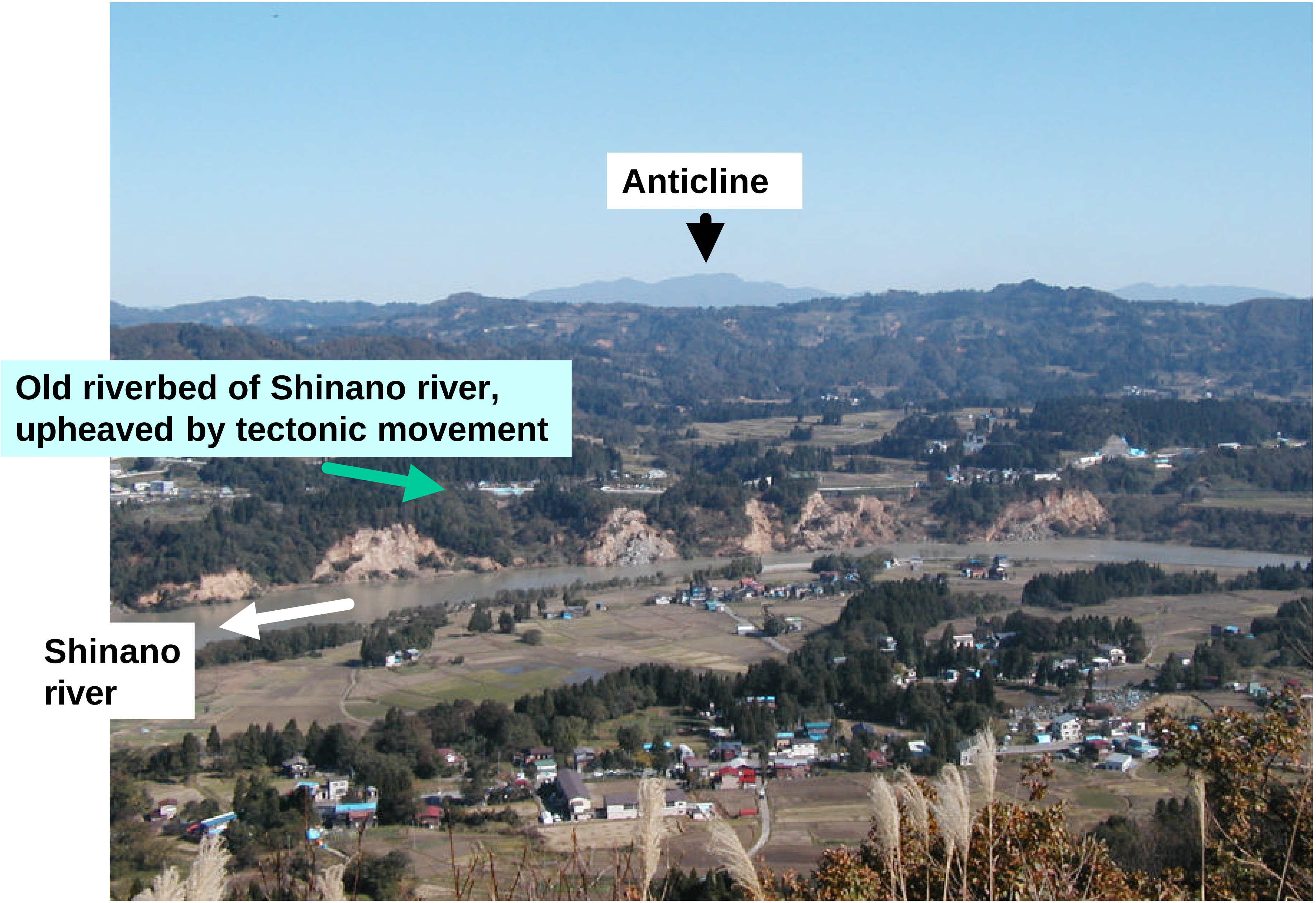
- **Reinforced RWs with nails could be the most suitable solution for newly constructed RWs on slope.**
- **Nailing could be one of the most effective ways to stabilize existing conventional type RWs on slope.**

2004 Niigata ken Chuetsu Earthquake

Mainshock (M= 6.8),17:56PM 23 October 2004

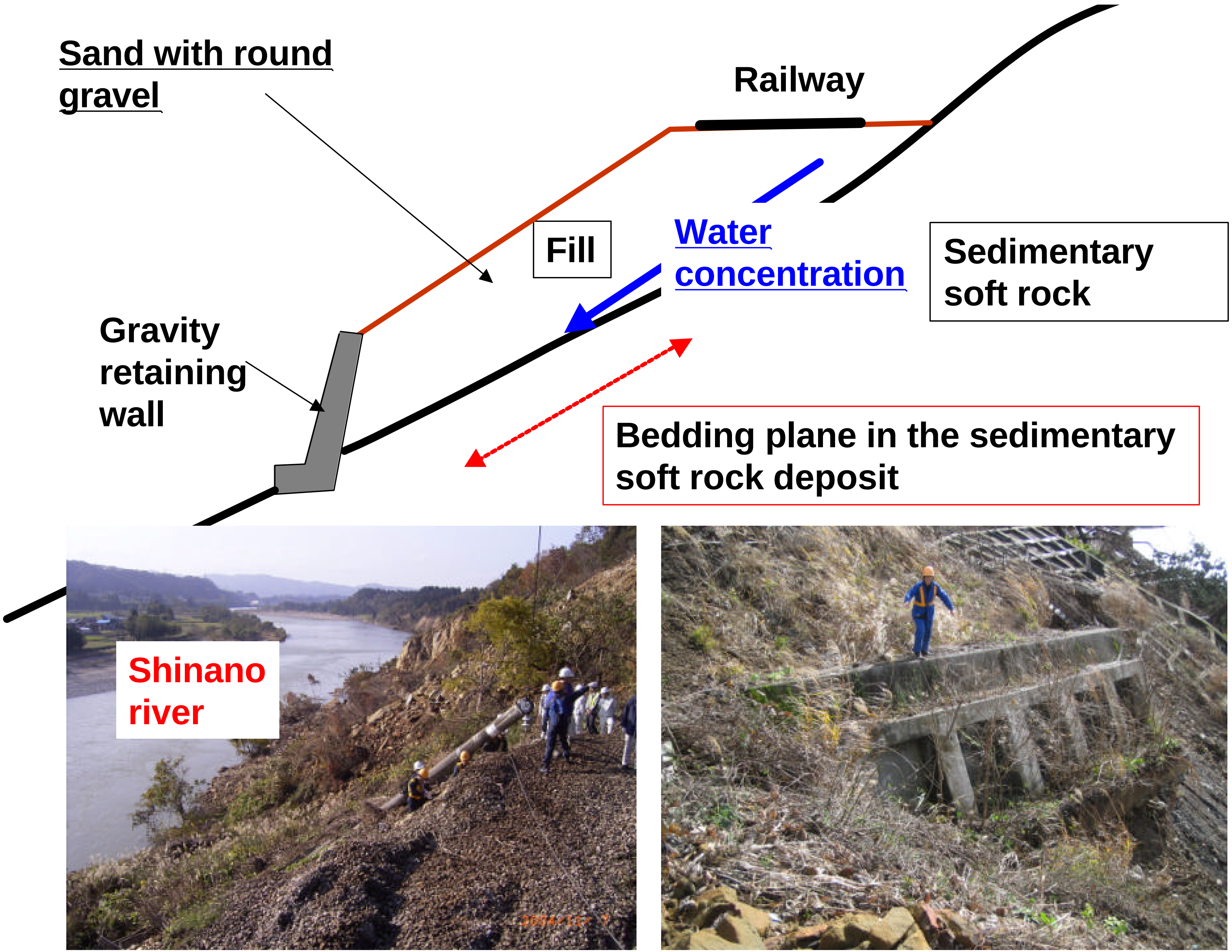


Distribution of the JMA-scale seismic intensity


Anticline
Old riverbed of Shinano river, upheaved by tectonic movement
Shinano river

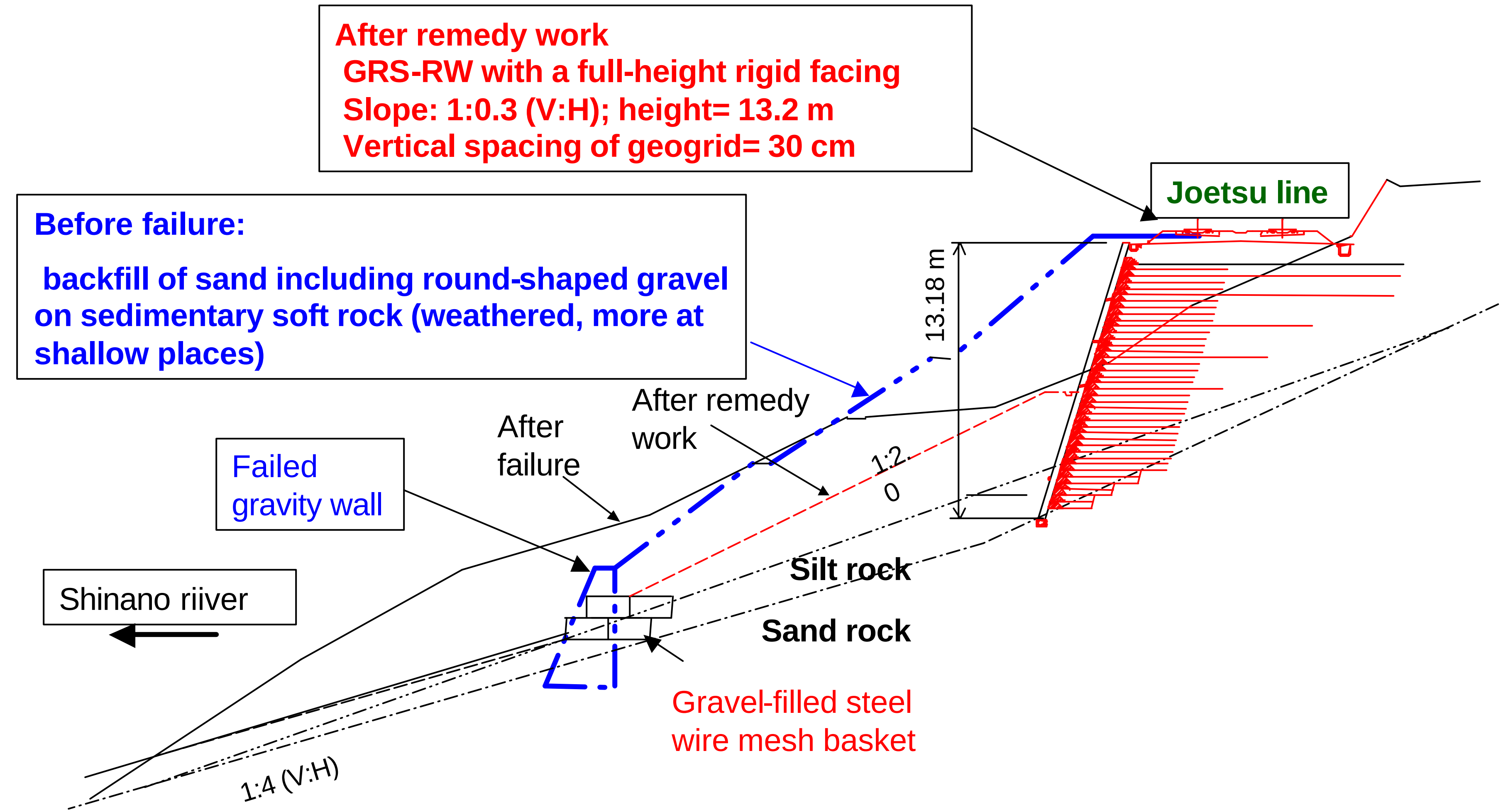
Failure of a railway embankment


2004/11/ 7


Sand with round gravel
Railway
Fill
Water concentration
Sedimentary soft rock
Gravity retaining wall
Bedding plane in the sedimentary soft rock deposit
Shinano river

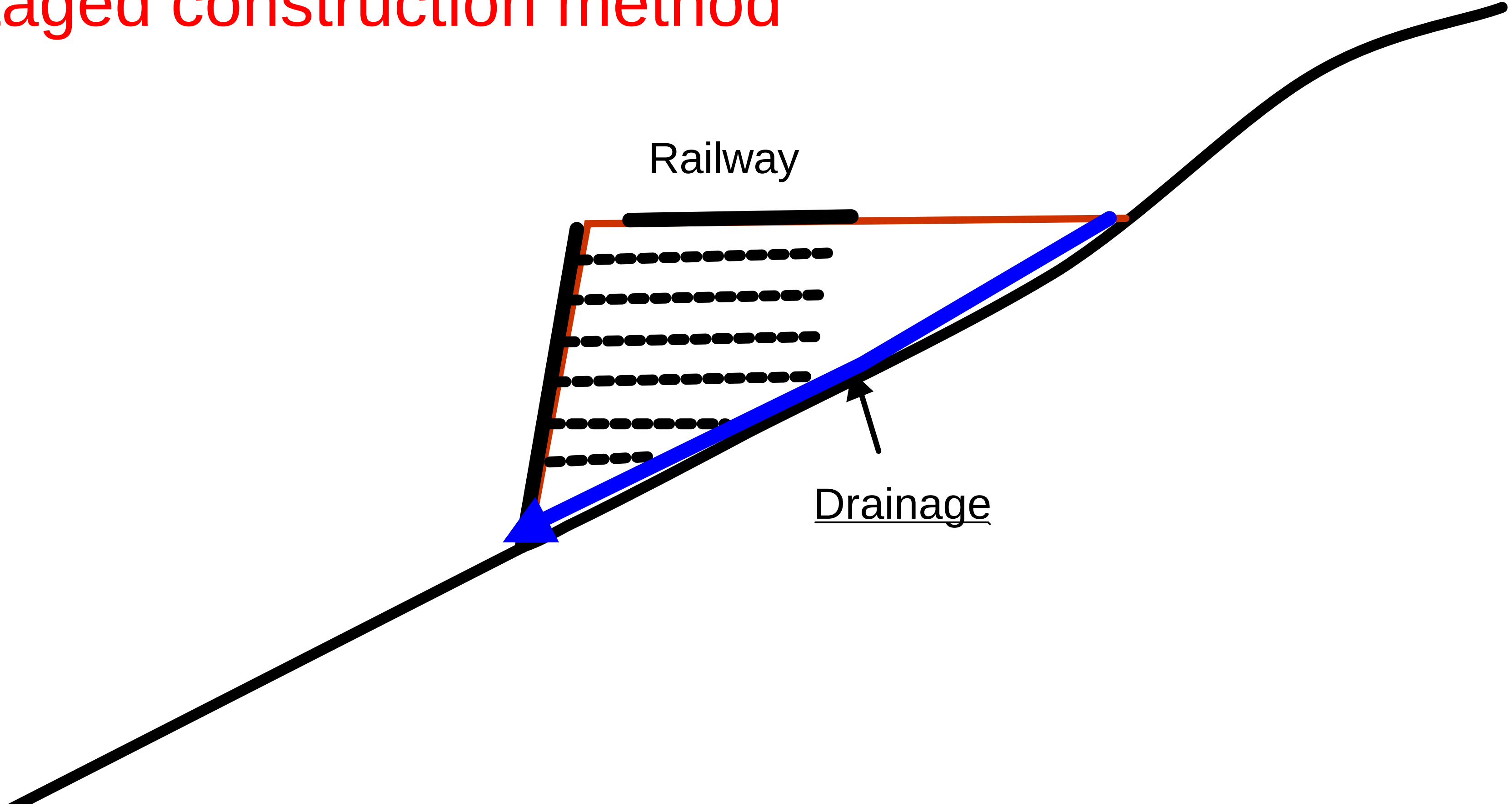
Site 2

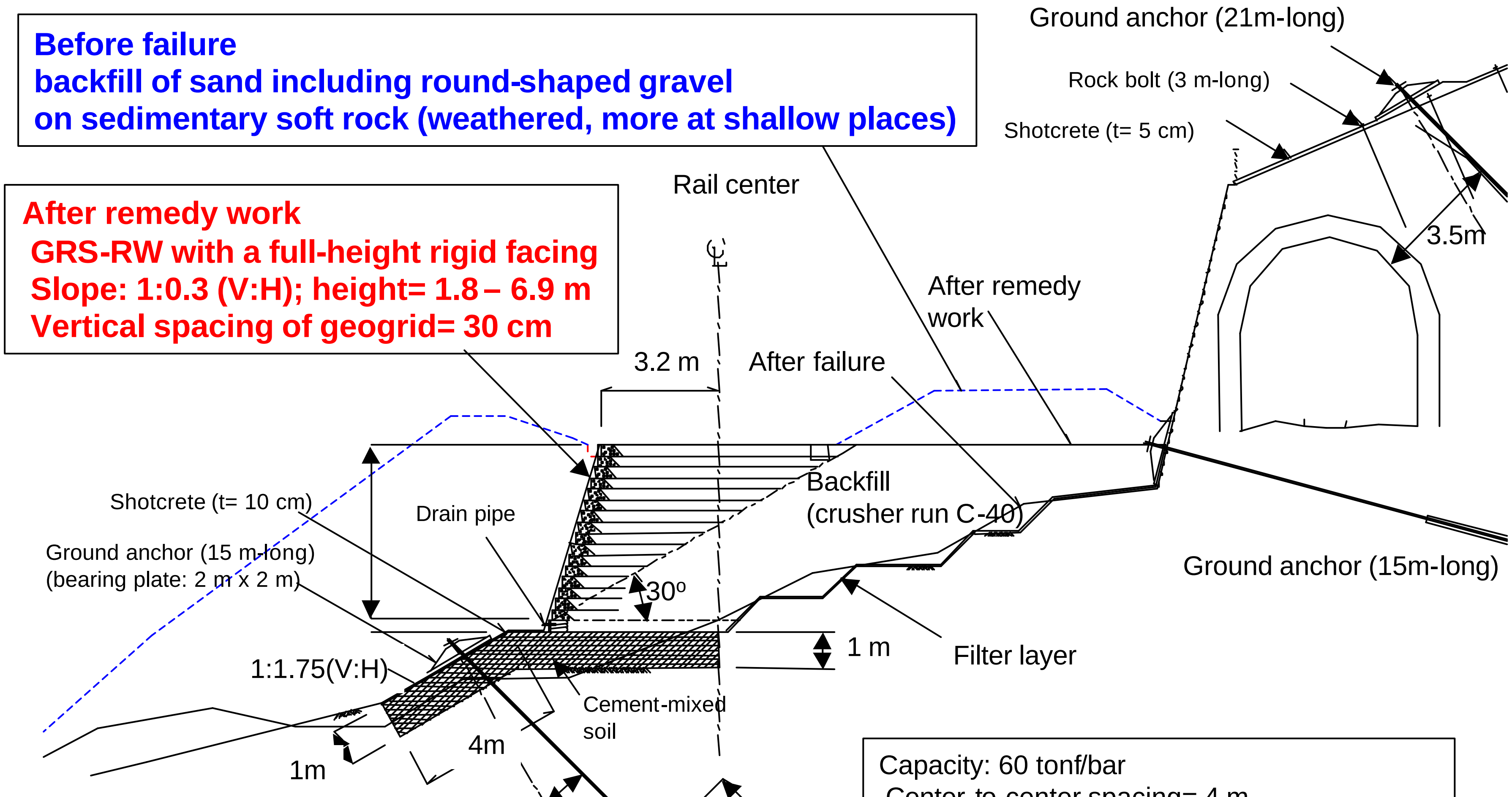
Site 2


After remedy work
GRS-RW with a full-height rigid facing
Slope: 1:0.3 (V:H); height= 13.2 m
Vertical spacing of geogrid= 30 cm
Joetsu line
Before failure:
backfill of sand including round-shaped gravel on sedimentary soft rock (weathered, more at shallow places)
13.18 m
After remedy work
After failure
Failed gravity wall
1:2.0
Silt rock
Sand rock
Shinano riiver
Gravel-filled steel wire mesh basket
1:4 (V:H)

Site 3

At these three sites, four failed embankments (one for the national road and three for the railway) were reconstructed to four geogrid-reinforced soil (GRS) retaining walls having a full-height rigid facing (i.e., thin lightly steel-reinforced concrete facing) by the staged construction method

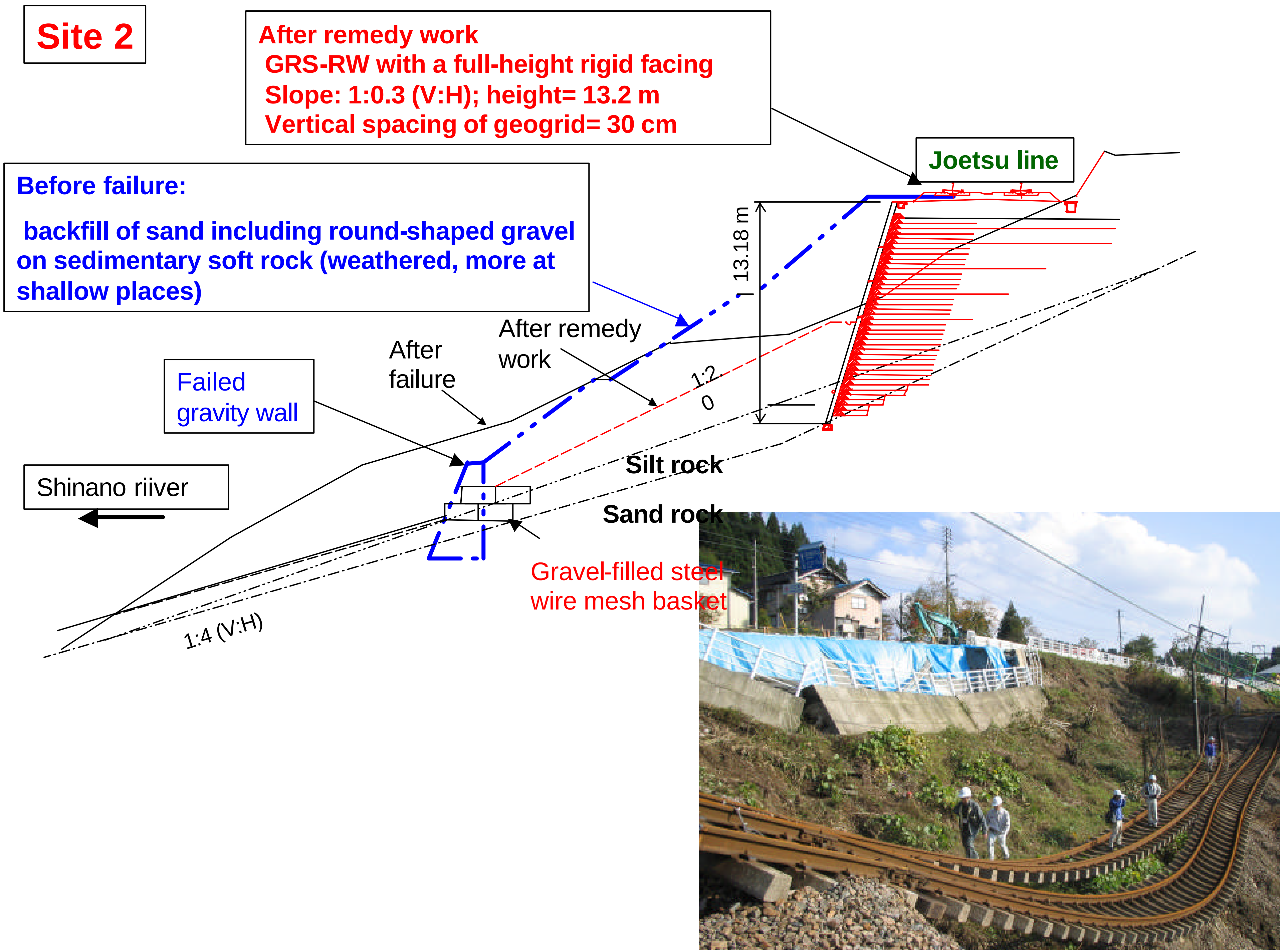



Before failure
backfill of sand including round-shaped gravel
on sedimentary soft rock (weathered, more at shallow places)
After remedy work
GRS-RW with a full-height rigid facing
Slope: 1:0.3 (V:H); height= 1.8 – 6.9 m
Vertical spacing of geogrid= 30 cm
Ground anchor (21m-long)
Rock bolt (3 m-long)
Shotcrete (t= 5 cm)
3.5m
Rail center
After remedy work
3.2 m
After failure
Backfill
(crusher run C-40)
Shotcrete (t= 10 cm)
Drain pipe
Ground anchor (15 m-long)
(bearing plate: 2 m x 2 m)
30º
1 m
Filter layer
Ground anchor (15m-long)
1:1.75(V:H)
Cement-mixed soil
1m
4m
15º
Capacity: 60 tonf/bar
Center-to-center spacing= 4 m
Anchorage:
11.5 cm in dia. times 7 m-long
(the same for all the ground anchor at this site)

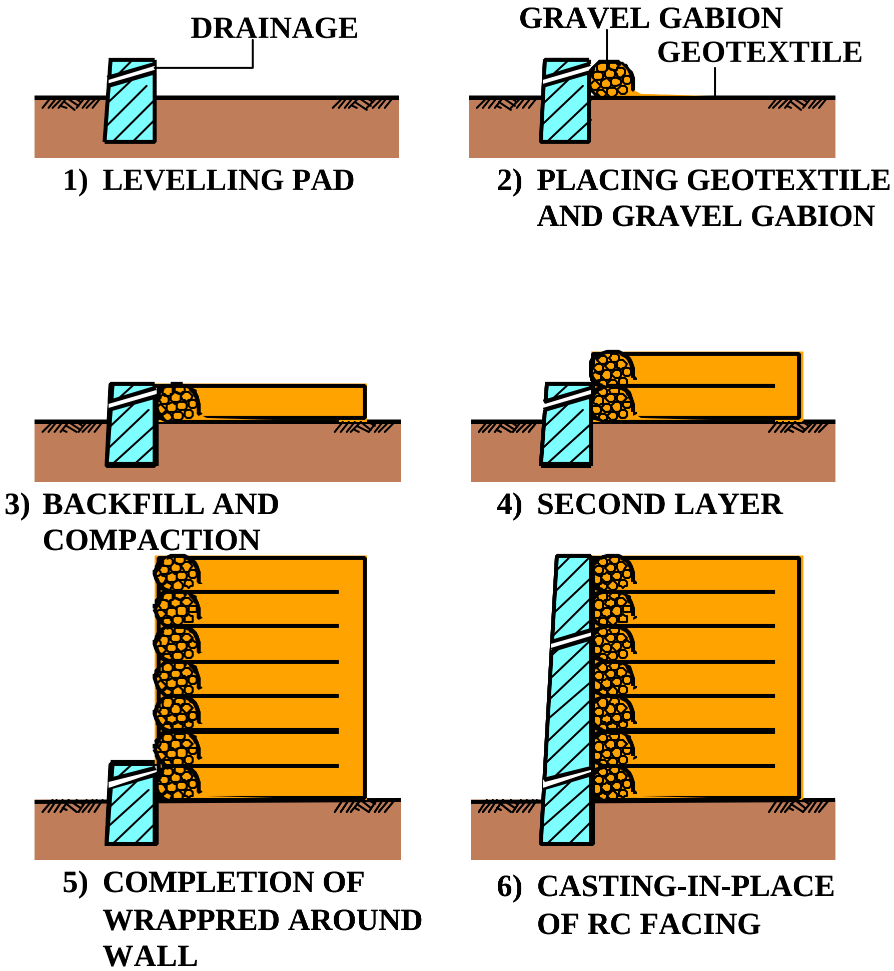
Site 1

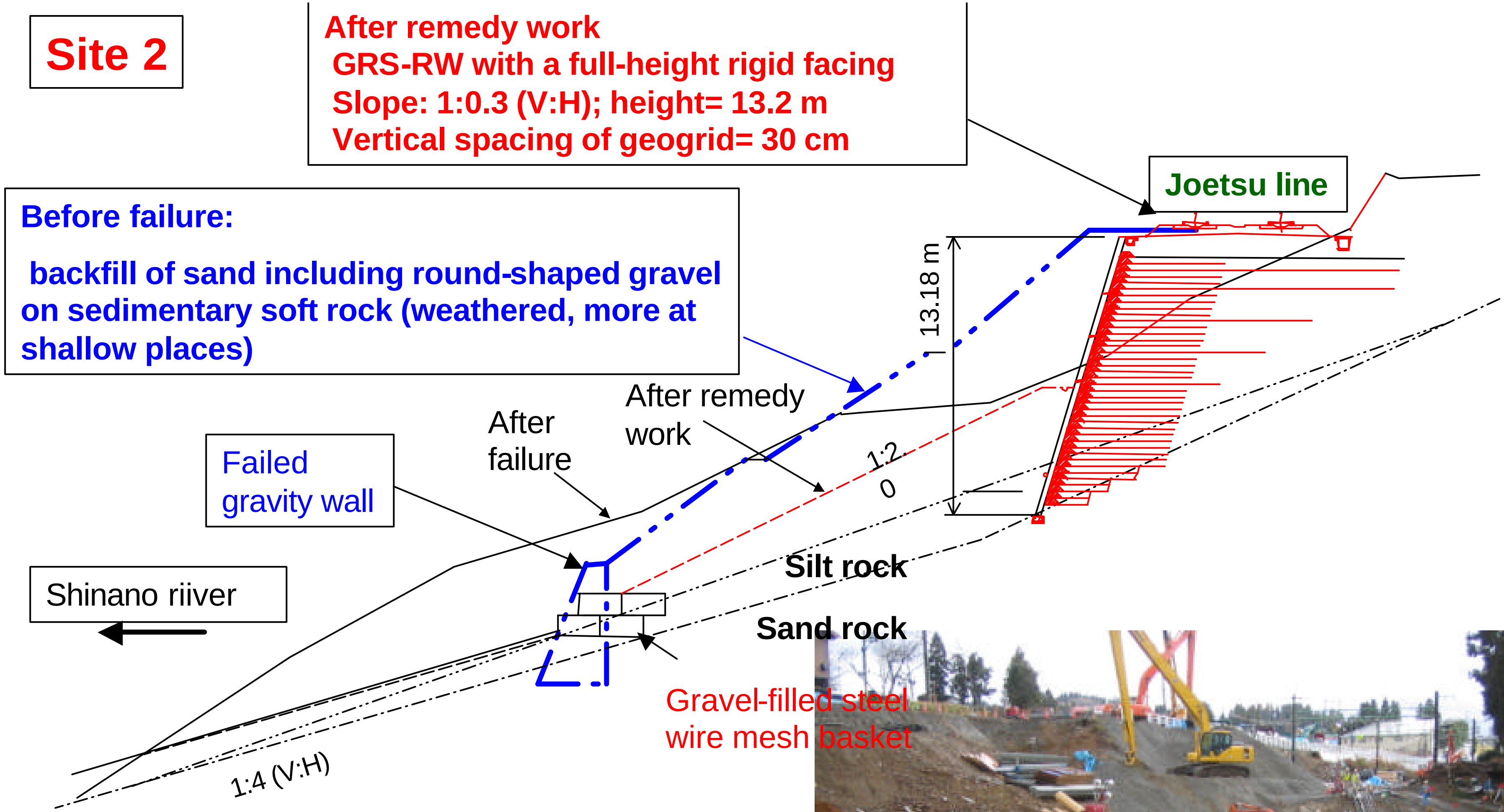

2004/12/14

Geogrid reinforced-soil retaining wall before casting concrete facing


Site 2
After remedy work
GRS-RW with a full-height rigid facing
Slope: 1:0.3 (V:H); height= 13.2 m
Vertical spacing of geogrid= 30 cm
Joetsu line
Before failure:
backfill of sand including round-shaped gravel on sedimentary soft rock (weathered, more at shallow places)
13.18 m
After remedy work
After failure
Failed gravity wall
1:2.0
Silt rock
Sand rock
Shinano riiver
Gravel-filled steel wire mesh basket
1:4 (V:H)

Geosynthetic-reinforced soil retaining wall having a full-height rigid facing that is staged-constructed





Site 2
After remedy work
GRS-RW with a full-height rigid facing
Slope: 1:0.3 (V:H); height= 13.2 m
Vertical spacing of geogrid= 30 cm
Before failure:
backfill of sand including round-shaped gravel on sedimentary soft rock (weathered, more at shallow places)
Joetsu line
13.18 m
After remedy work
After failure
Failed gravity wall
1:2.0
Silt rock
Sand rock
Shinano riiver
Gravel-filled steel wire mesh basket
1:4 (V:H)

The first train running on the geosynthetic-reinforced wall, 26 December 2004

Site 3

Summary:

The three railway embankments that failed totally during the 2004 Niigata Chuetsu earthquake were reconstructed within two months after the failure.

It was validated again by this new case history that the geosynthetic-reinforced soil retaining wall could be very competitive to construct wall structures for such important structures (i.e., railway and highway).

Contents

1. Recent advances in geosynthetic-reinforced soil structures in Japan (1997-1998 Mercer Lecture, revised)

2. Comparison of dynamic stability between reinforced-soil and gravity type retaining walls

3. A new dynamic earth pressure theory accounting for strain softening and strain localization

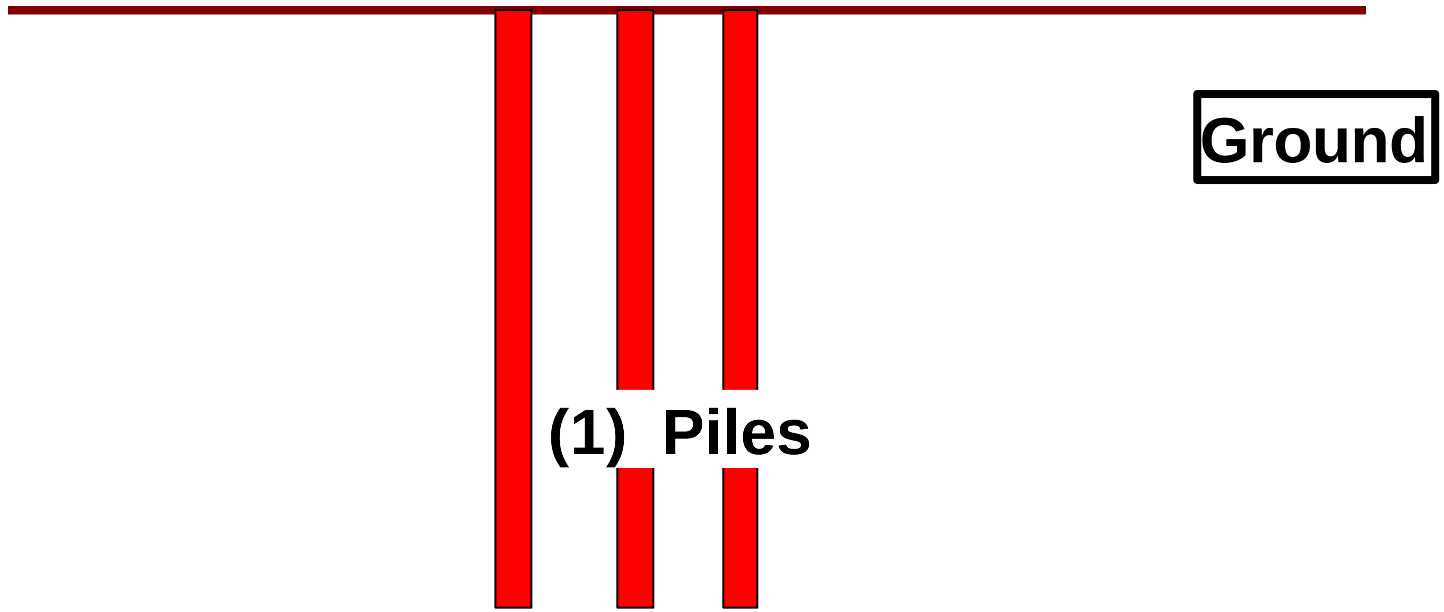
4. Seismic stability of soil retaining walls on slope

5. Lessons from 2004 Niigata-ken Chuetsu Earthquake

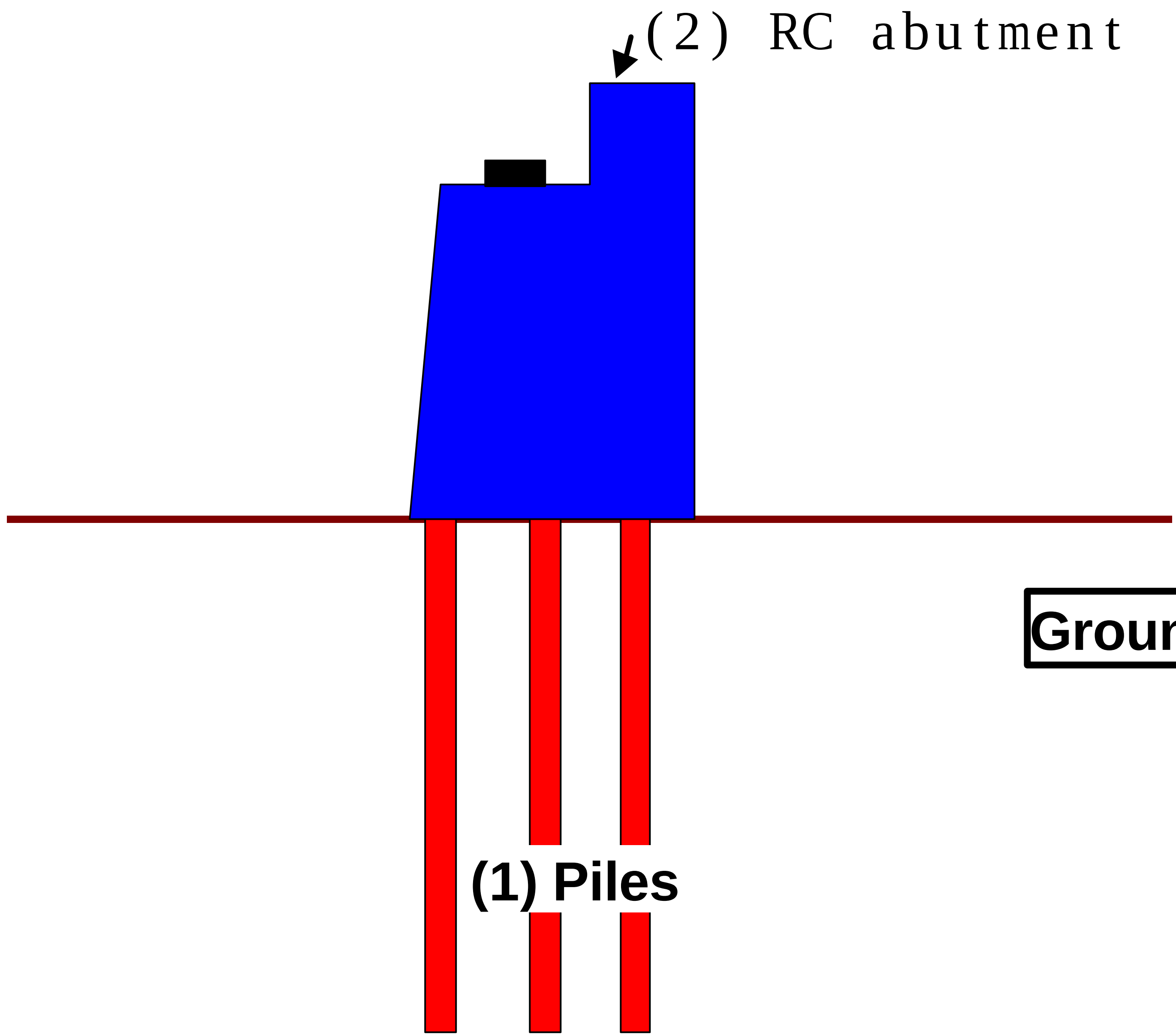
6. **New type bridge abutments: PL&PS and cement-mixed backfill**

Ground

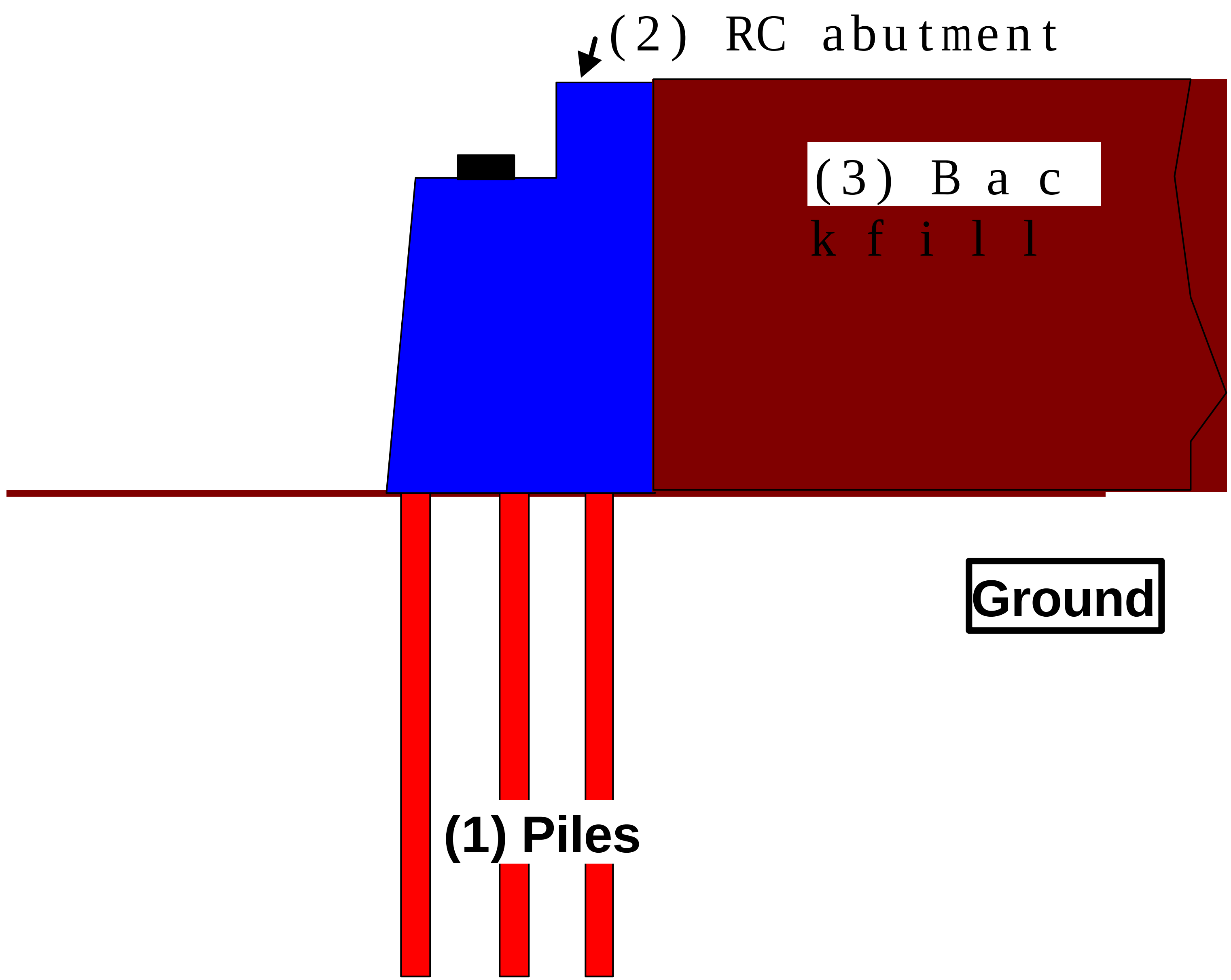
Several technical problems with conventional bridge abutment



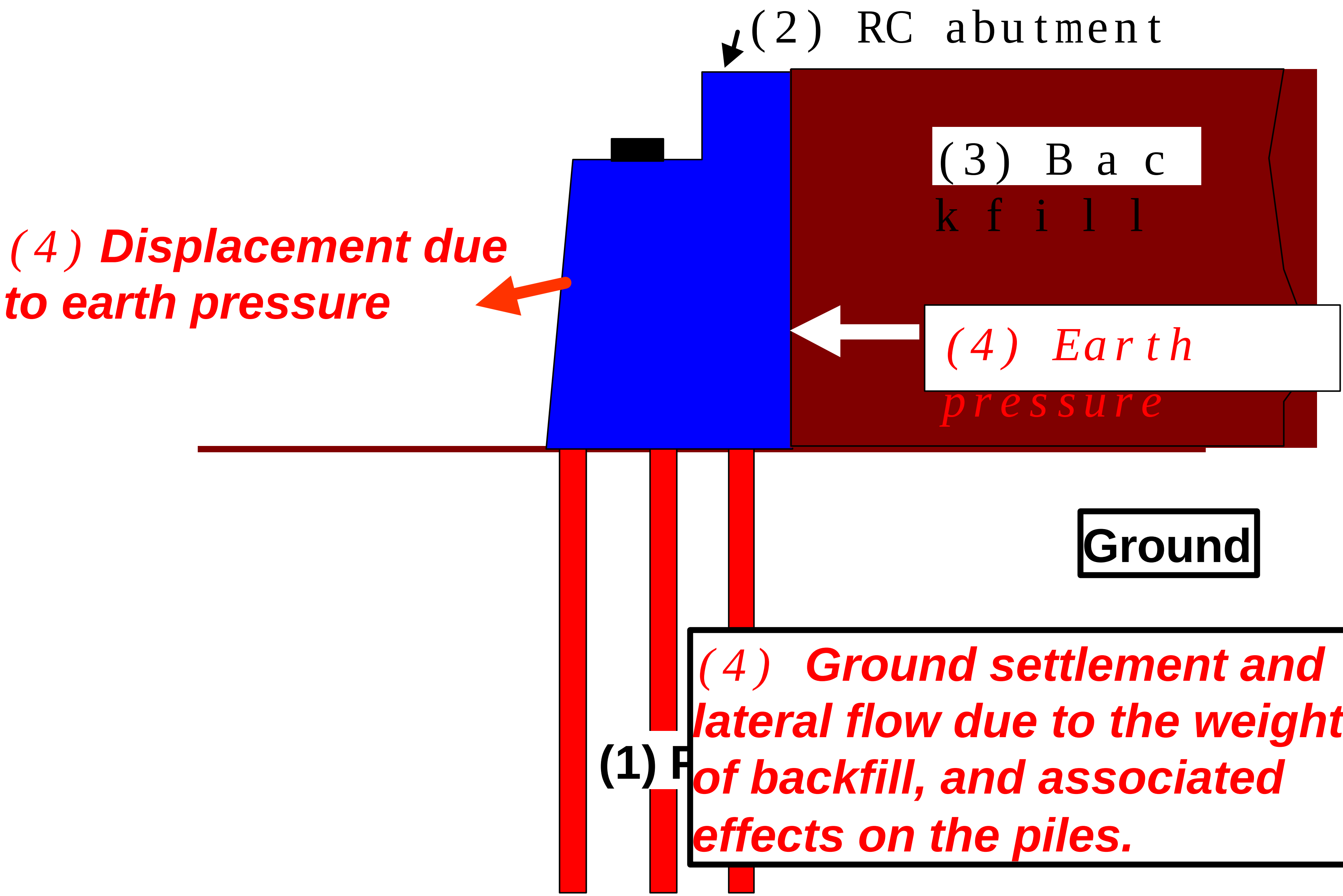
Several technical problems with conventional bridge abutment



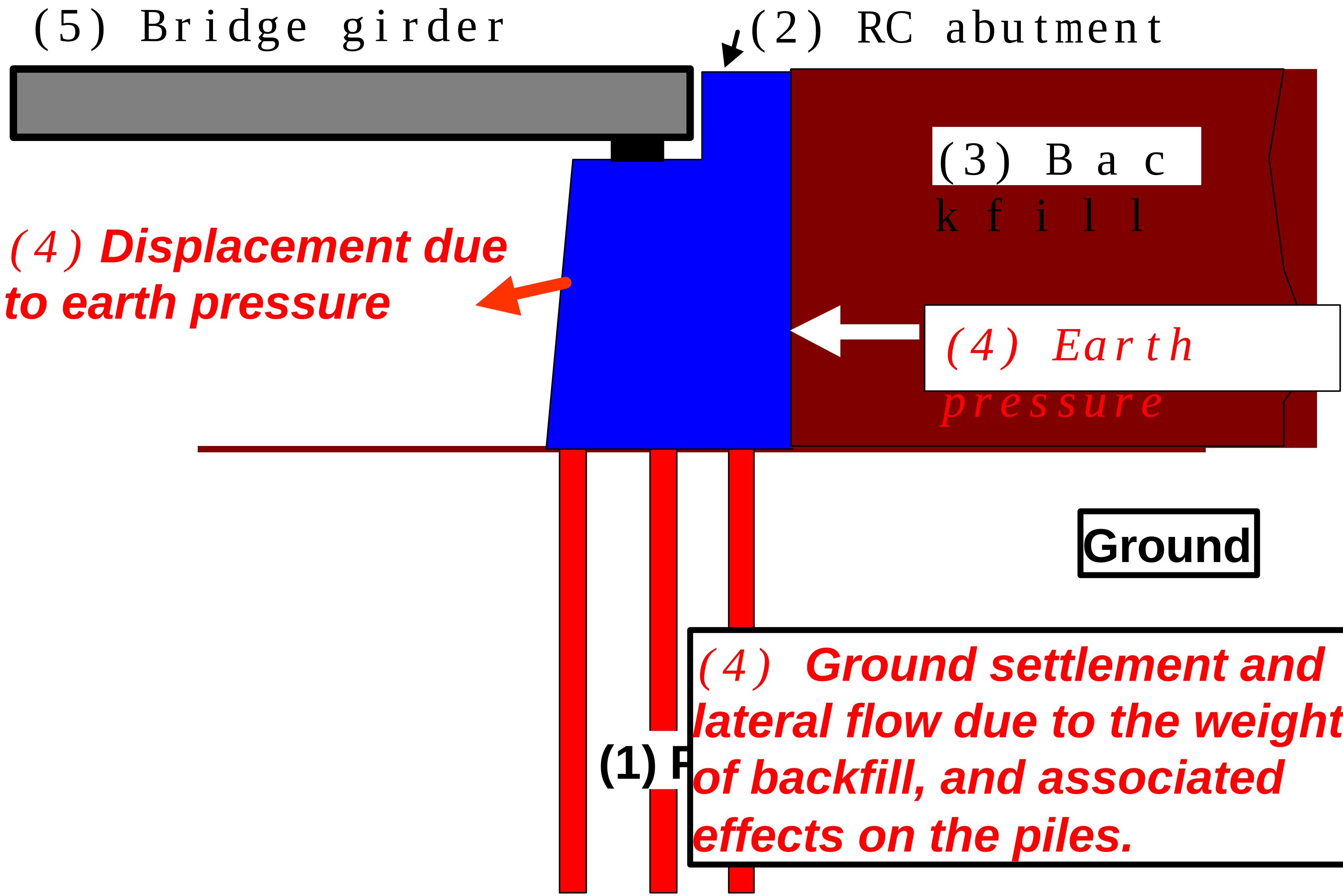
Several technical problems with conventional bridge abutment



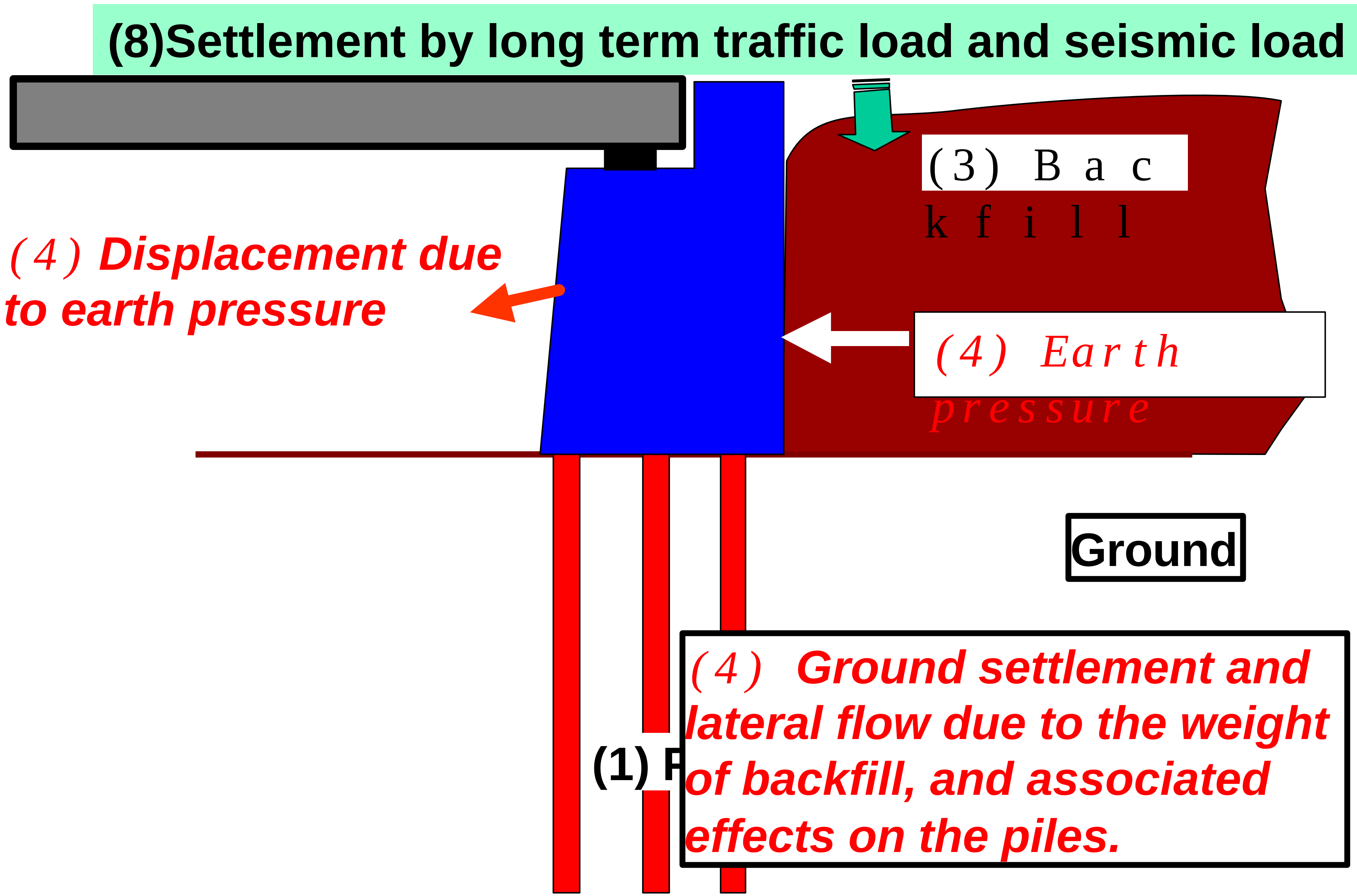
Several technical problems with conventional bridge abutment



Several technical problems with conventional bridge abutment



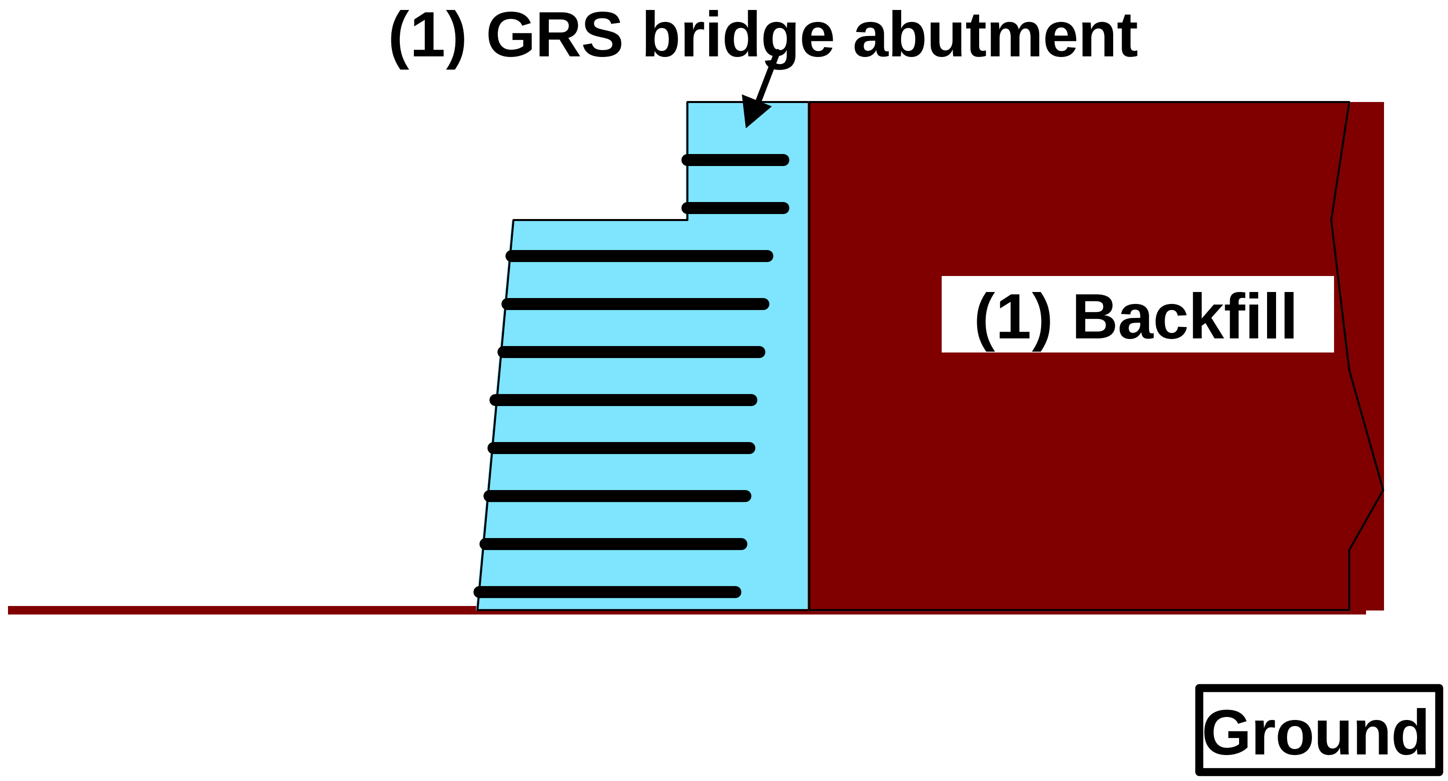
Several technical problems with conventional bridge abutment



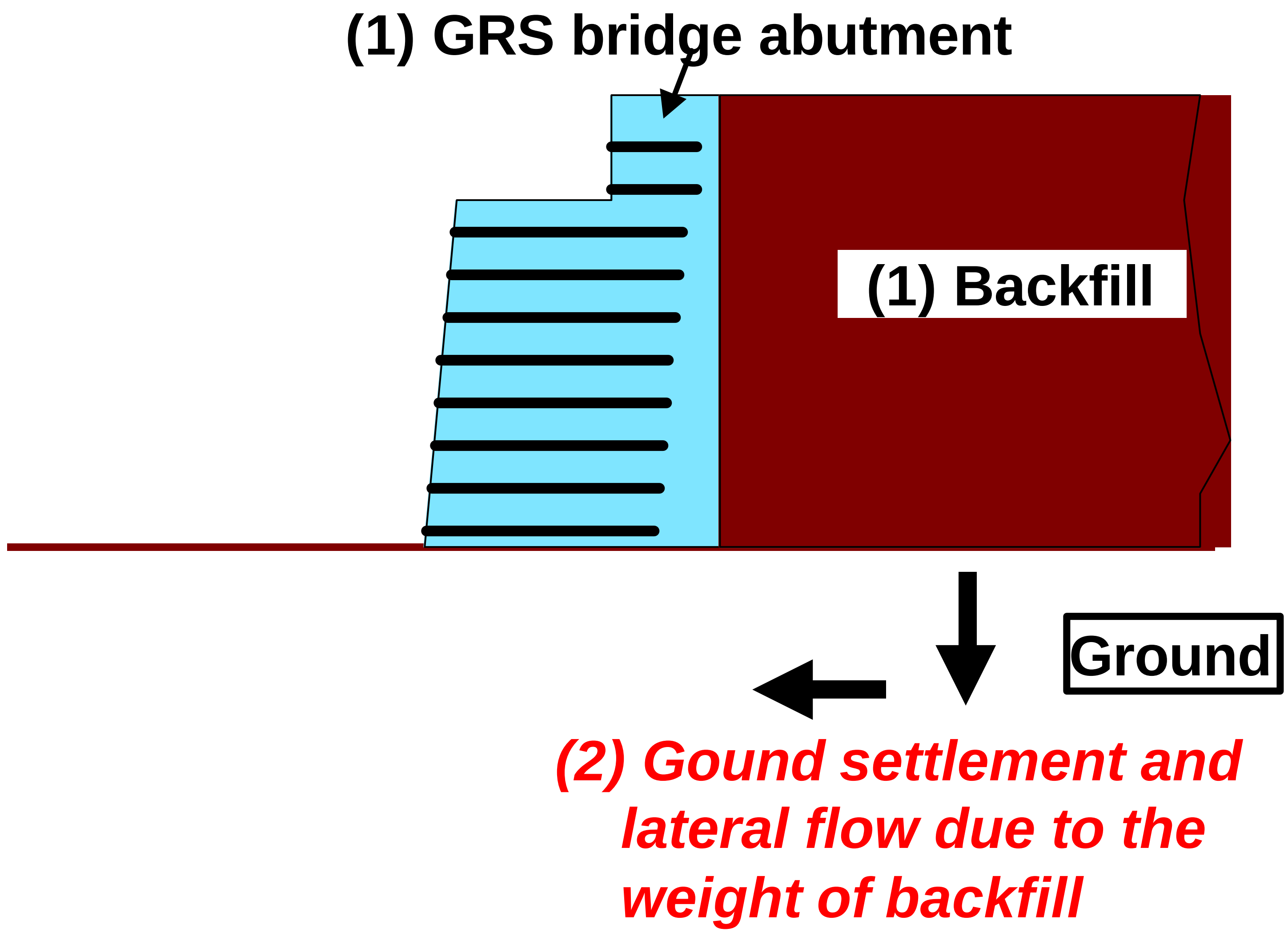
Several technical problems with conventional bridge abutment

Ground

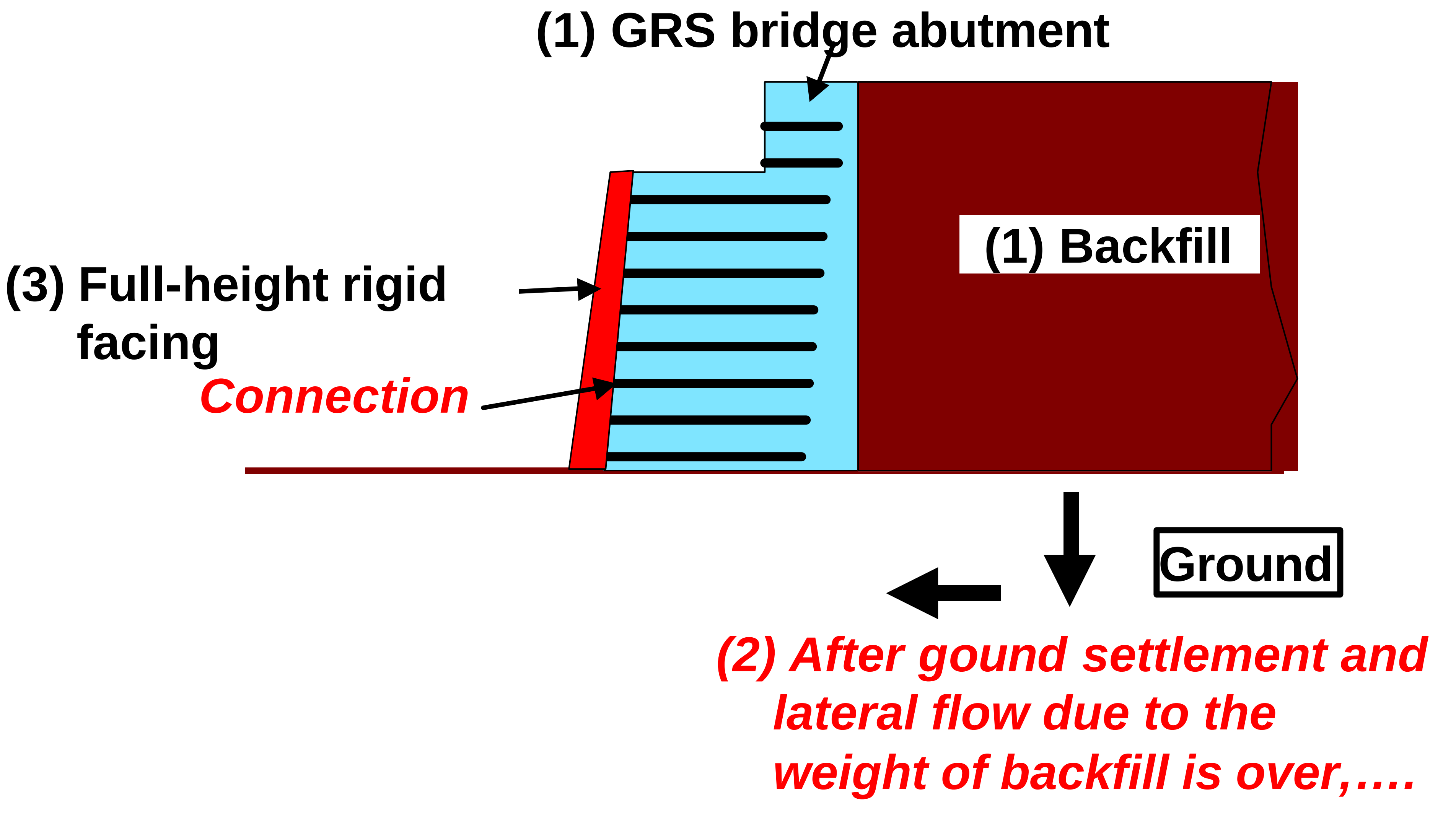
Relevance of geosynthetic-reinforced soil bridge abutment



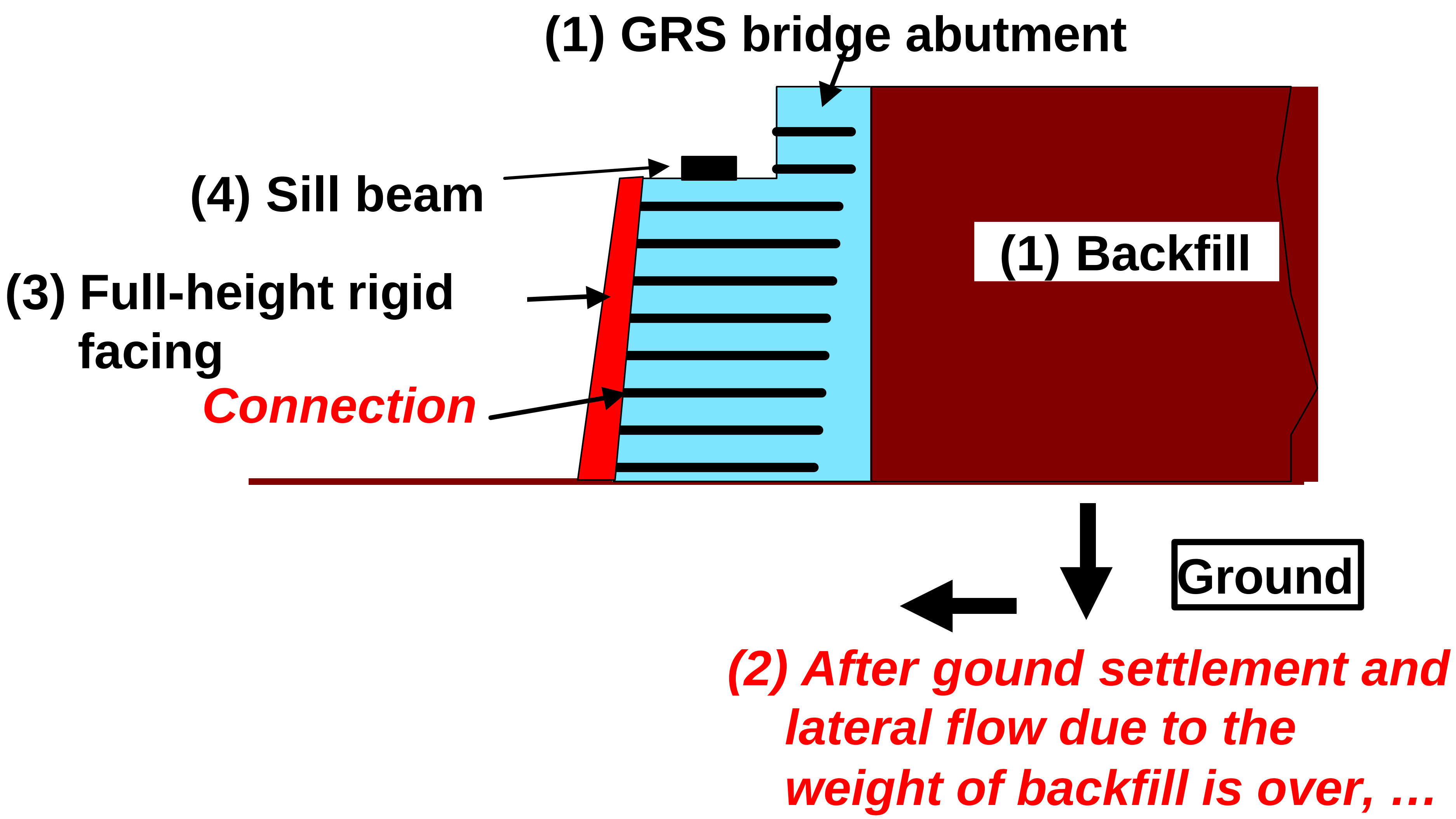
Relevance of geosynthetic-reinforced soil bridge abutment



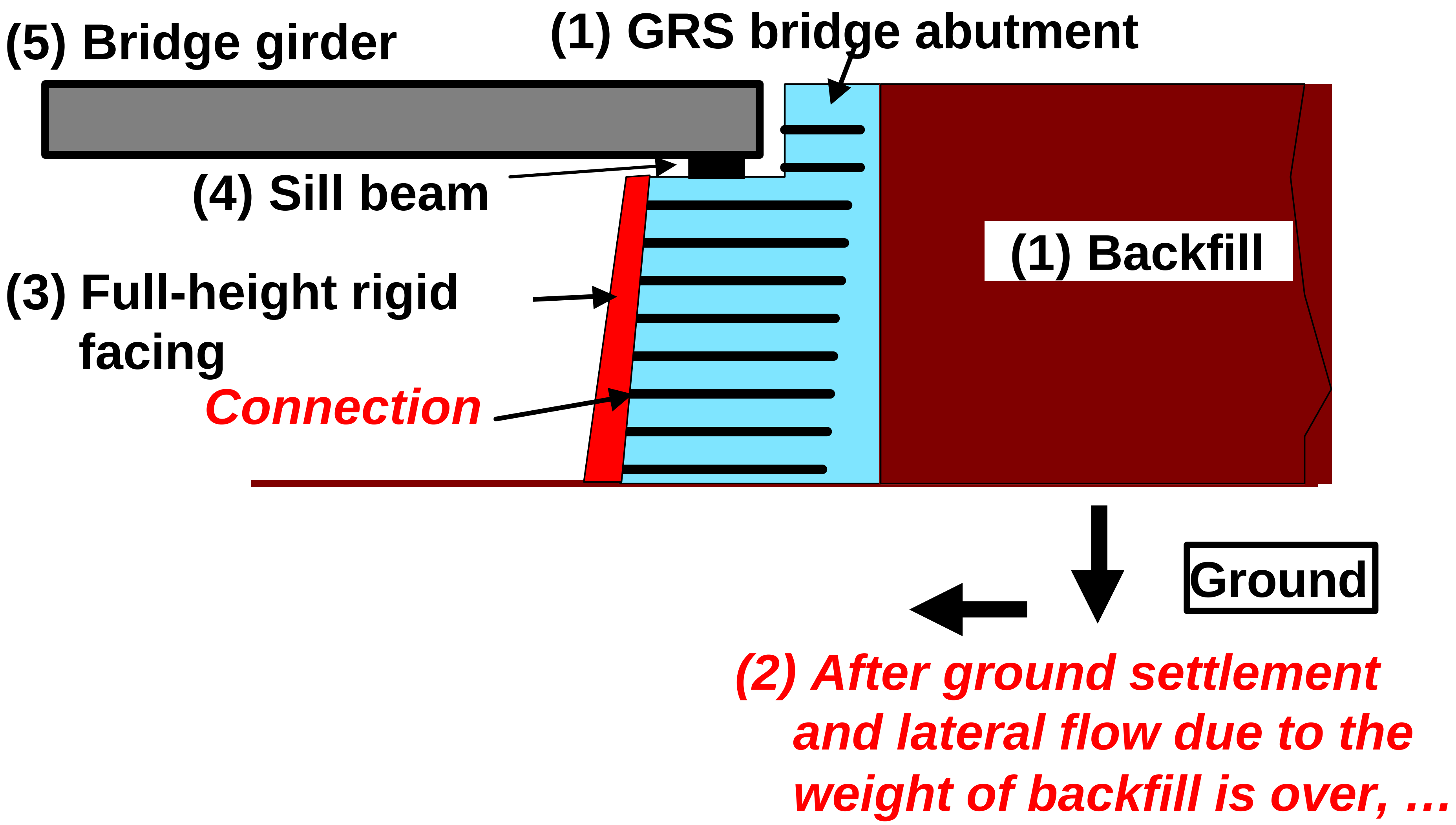
Relevance of geosynthetic-reinforced soil bridge abutment



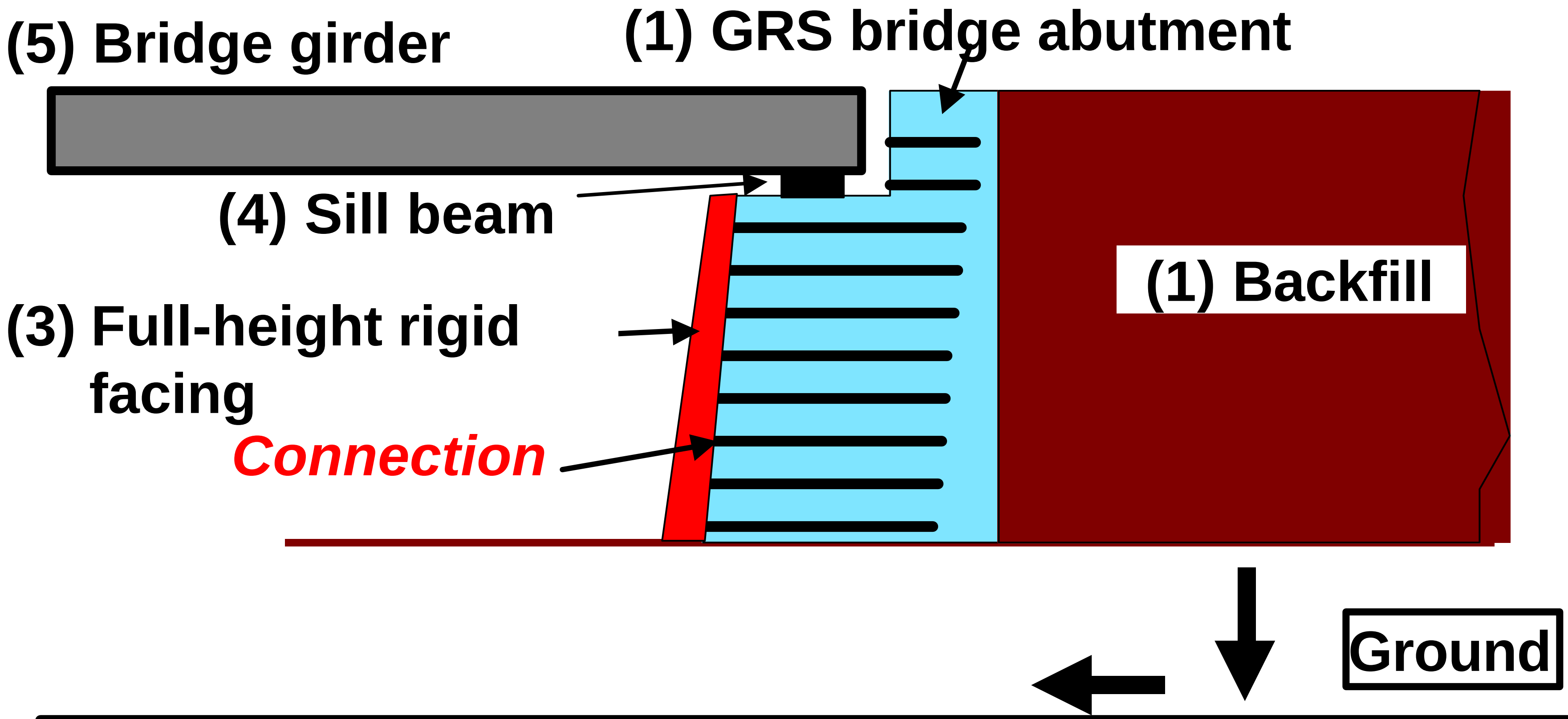
Relevance of geosynthetic-reinforced soil bridge abutment



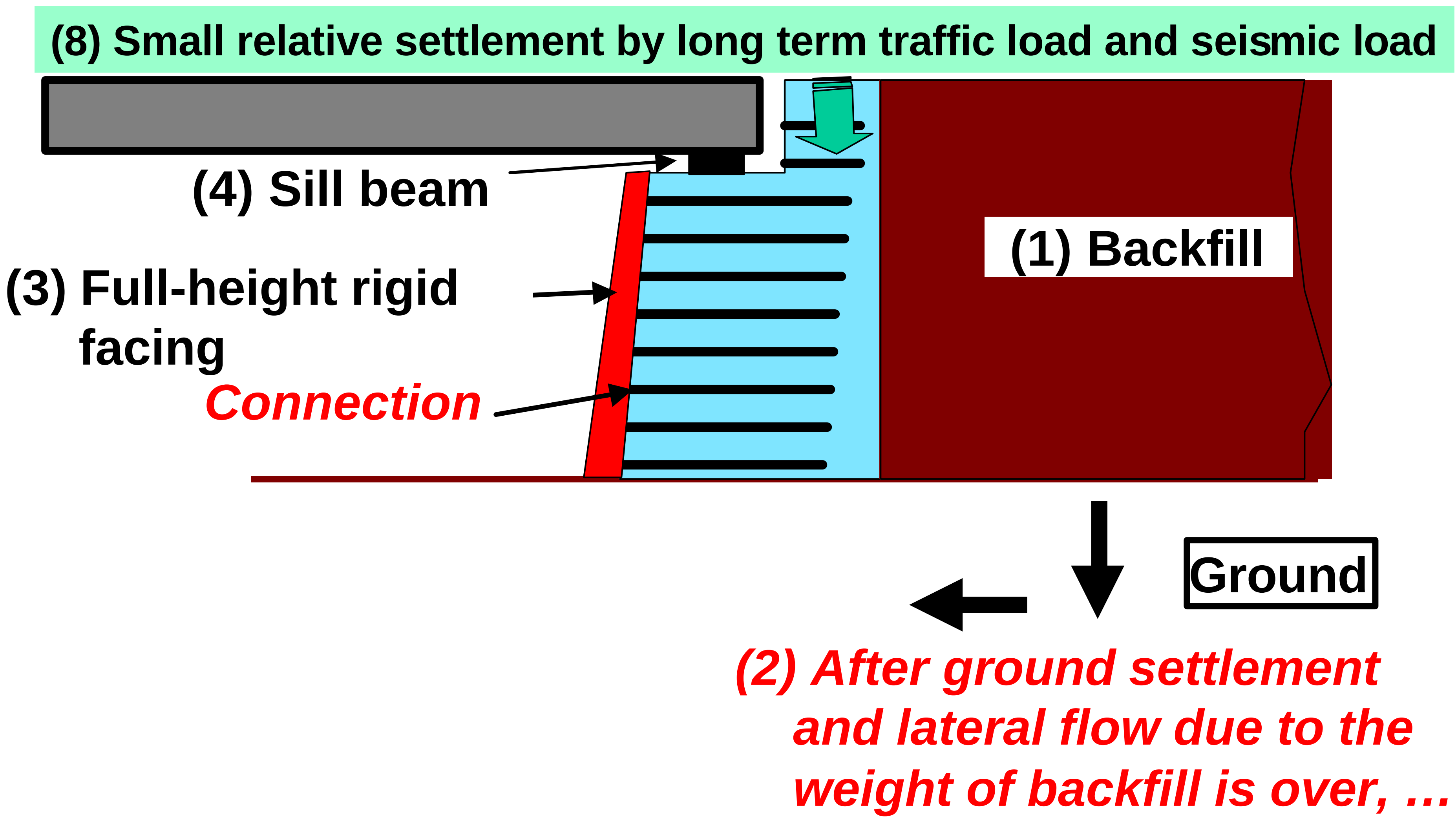
Relevance of geosynthetic-reinforced soil bridge abutment



Relevance of geosynthetic-reinforced soil bridge abutment



As the full-height rigid facing is constructed after the deformation of backfill and ground takes place and the location of the sill beam can be adjusted when the bridge girder is installed, some amount of displacement of the wall during construction can be allowed. Therefore, a pile foundation usually becomes unnecessary.



Relevance of geosynthetic-reinforced soil bridge abutment

A pair of GRS bridge abutments

Seibu Ikebukuro Line, Tokyo

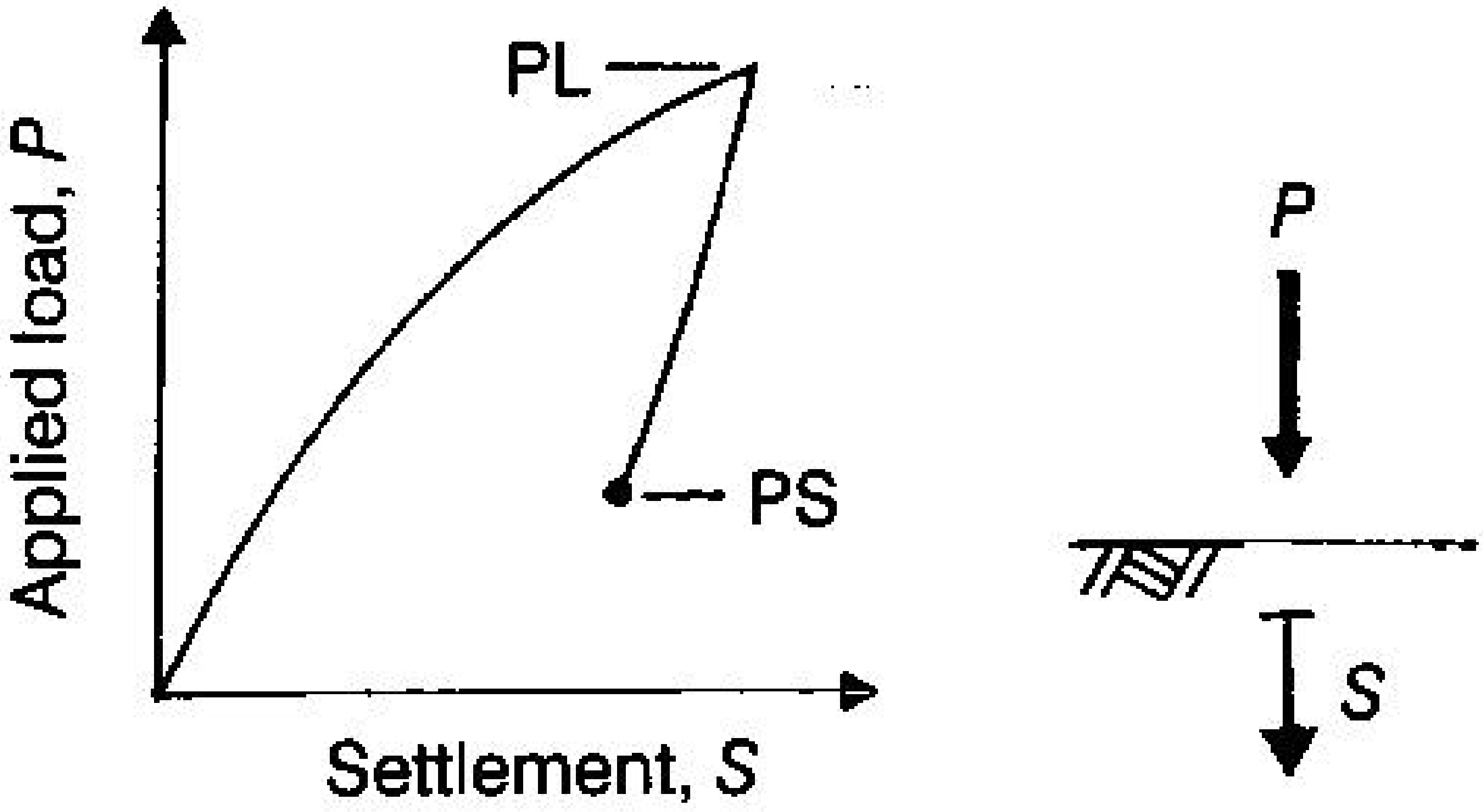
Needs for GRS bridges abutments (and piers);

1) supporting longer girders than the present ones; and

2) being seismically more stable to survive even very severe earthquakes like the1995 Great Kobe Earthquake.

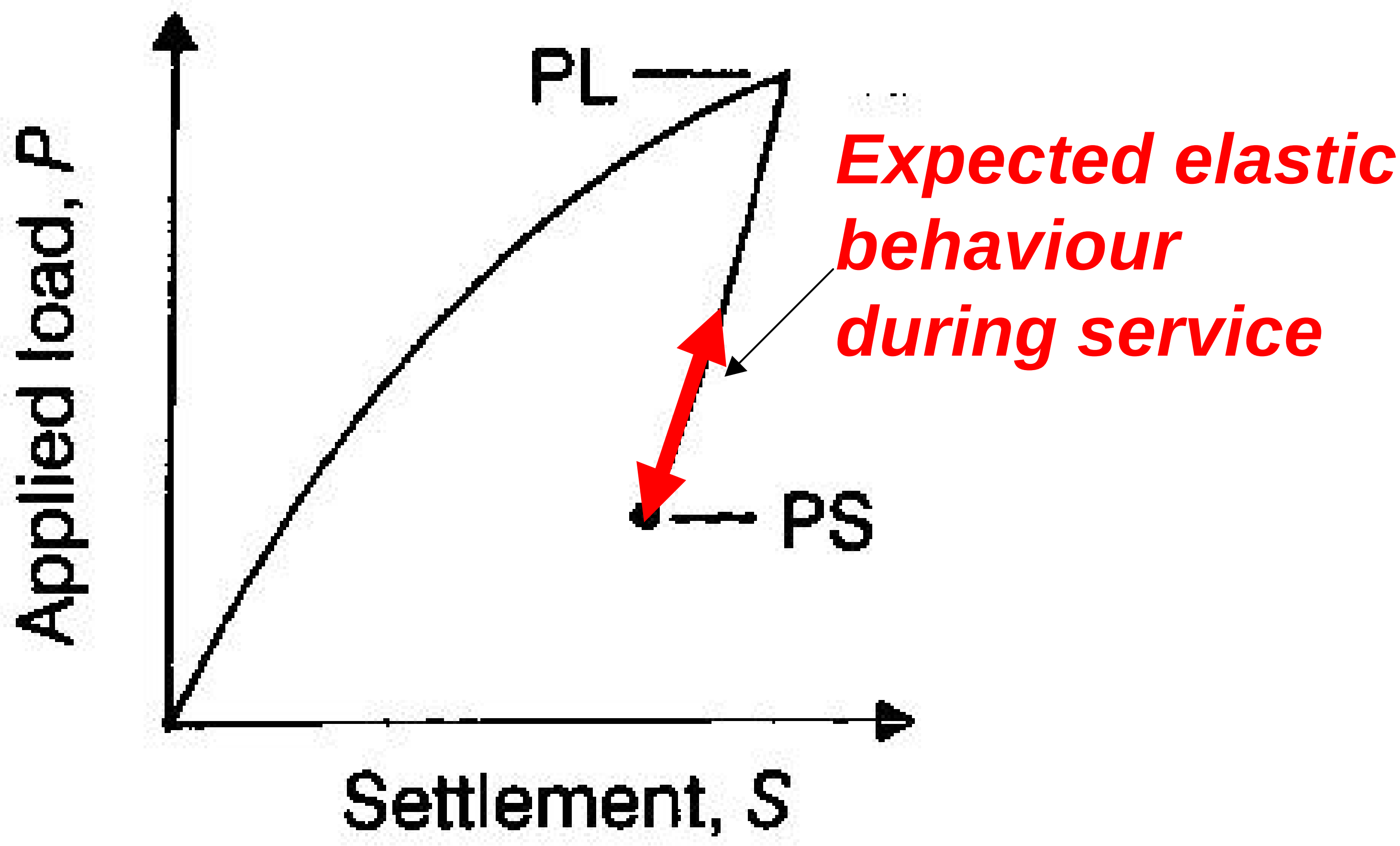
Preloading and prestressing procedures

1) Application of sufficiently large preload ***PL***;
2) Unloading to the initial prestress load level ***PS***; and
3) Fixing the top ends of the tie rods to the top reaction block.

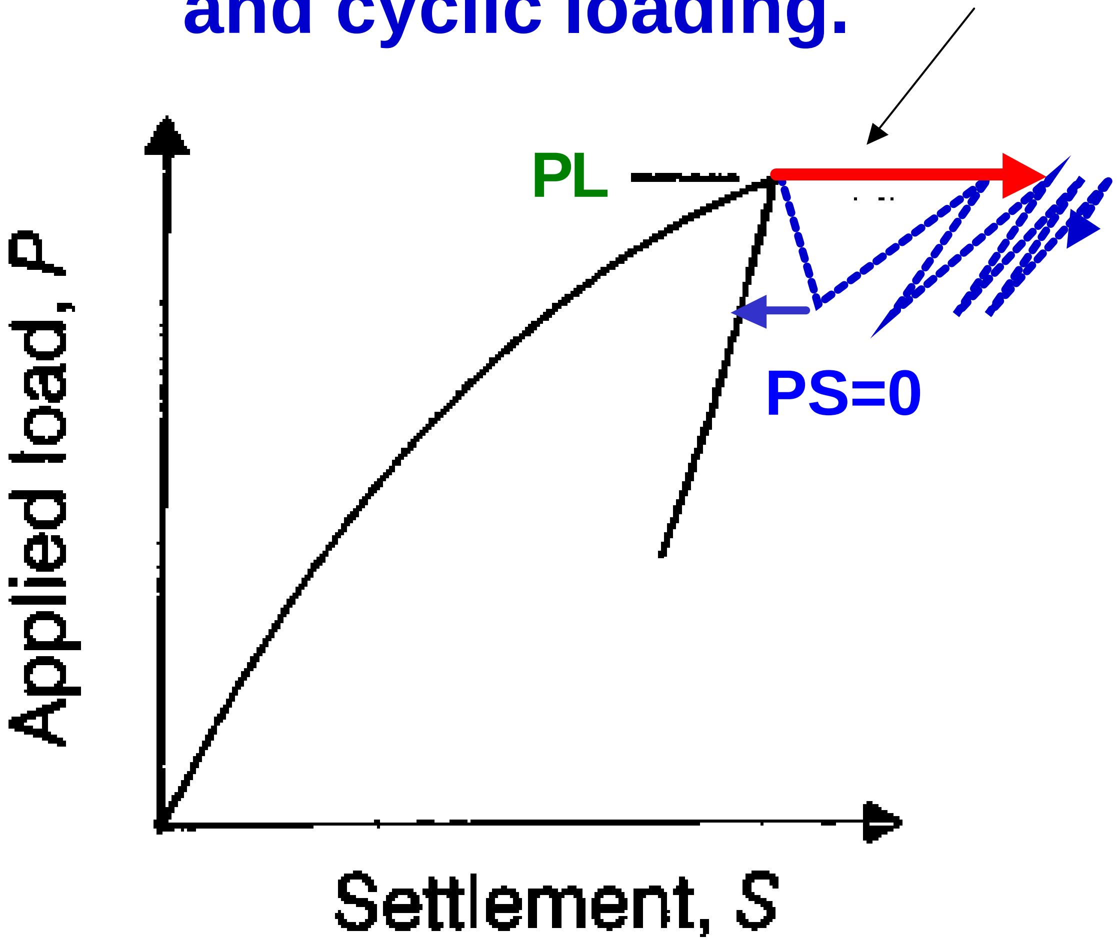


Preloading and prestressing procedures

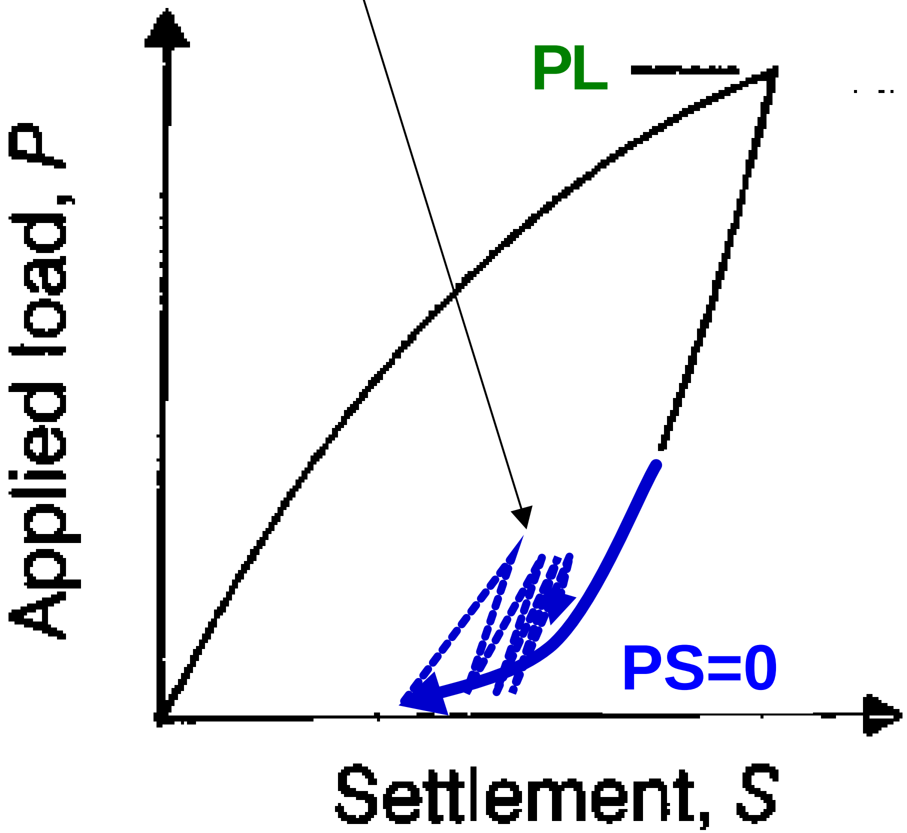
1) High *PL* for elastic deformation of backfill elastic;
2) High *PS* for high stiffness of backfill



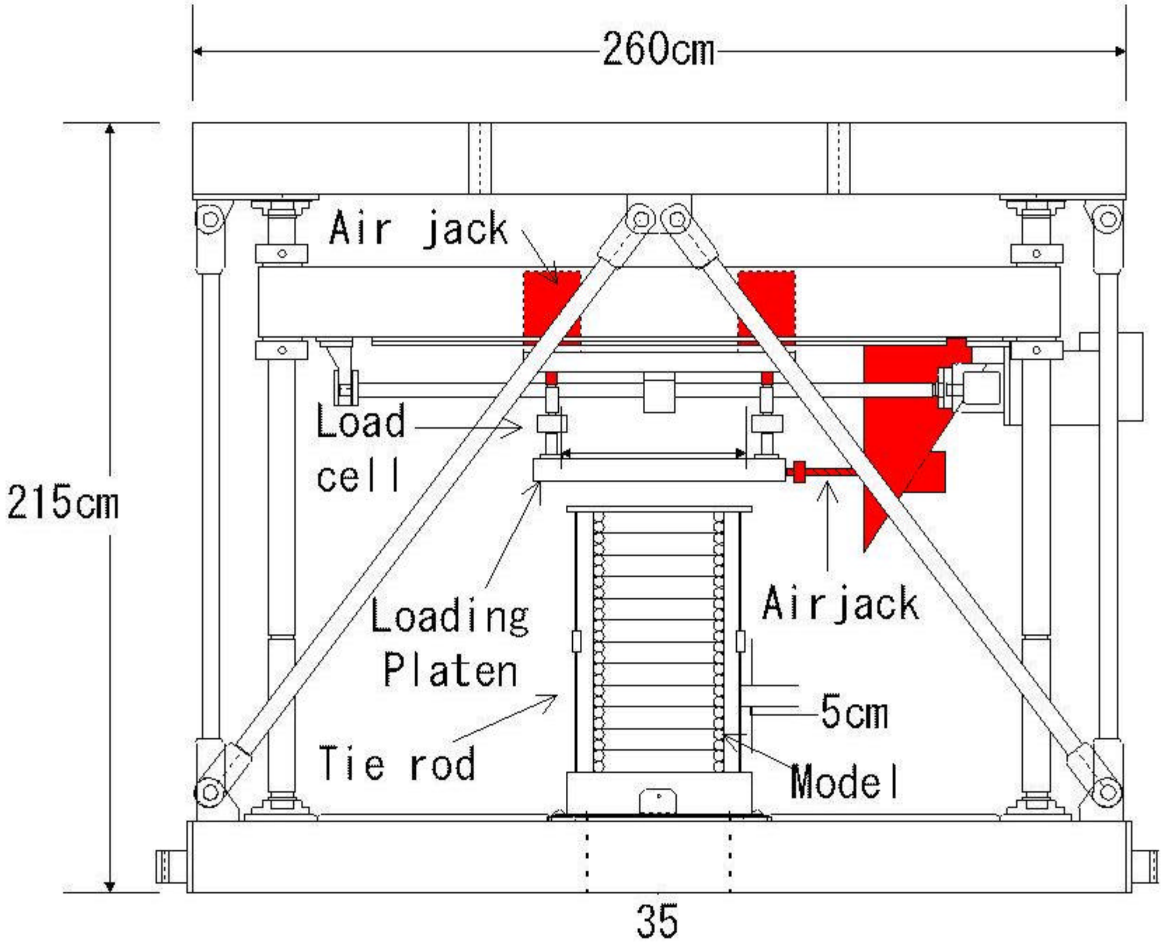
When PS= PL, the backfill exhibits large residual deformation by sustained and cyclic loading.



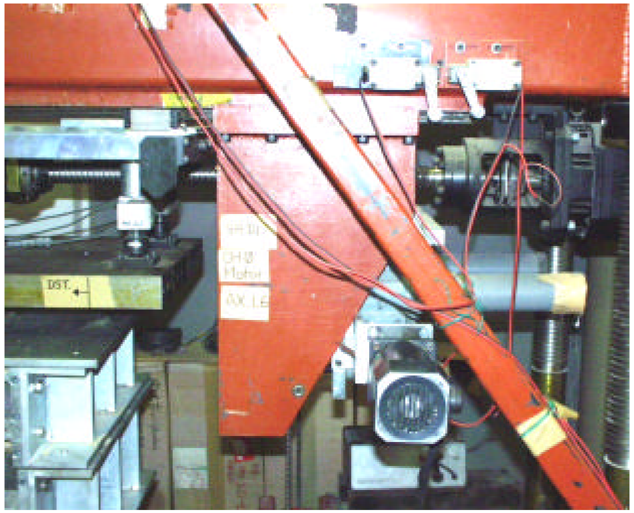
When PS= 0, the backfill exhibits large residual deformation by sustained and cyclic loading.



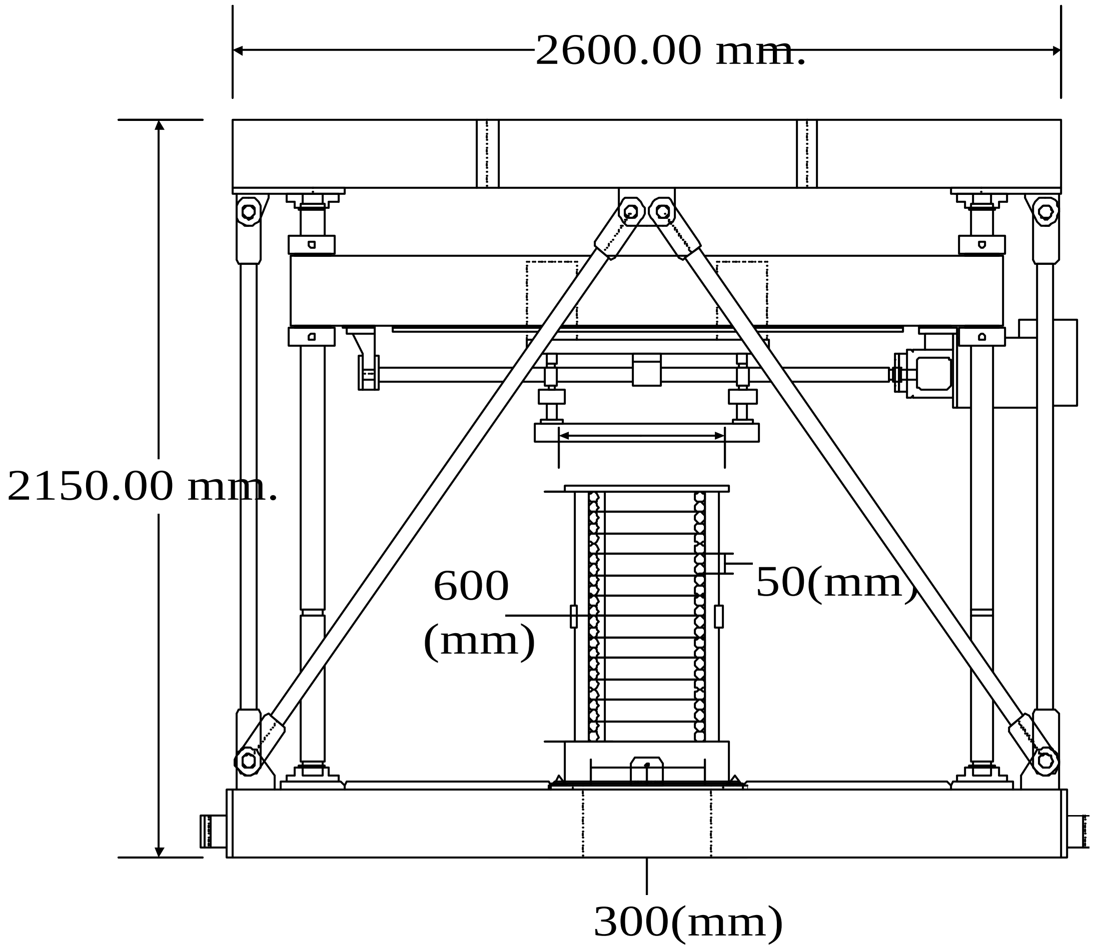
Model tests on a reinforced soil structure

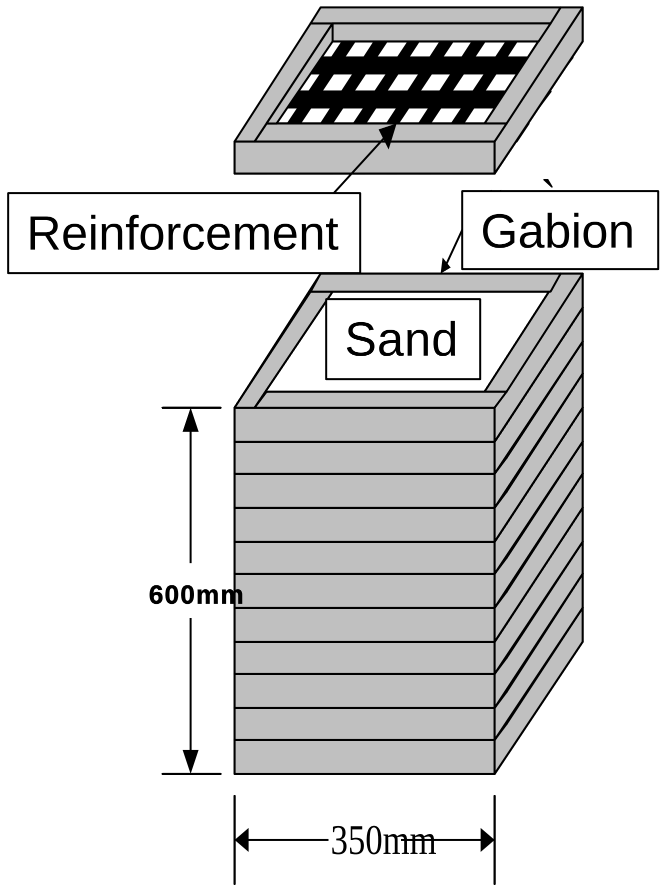


Independent control of vertical and horizontal load

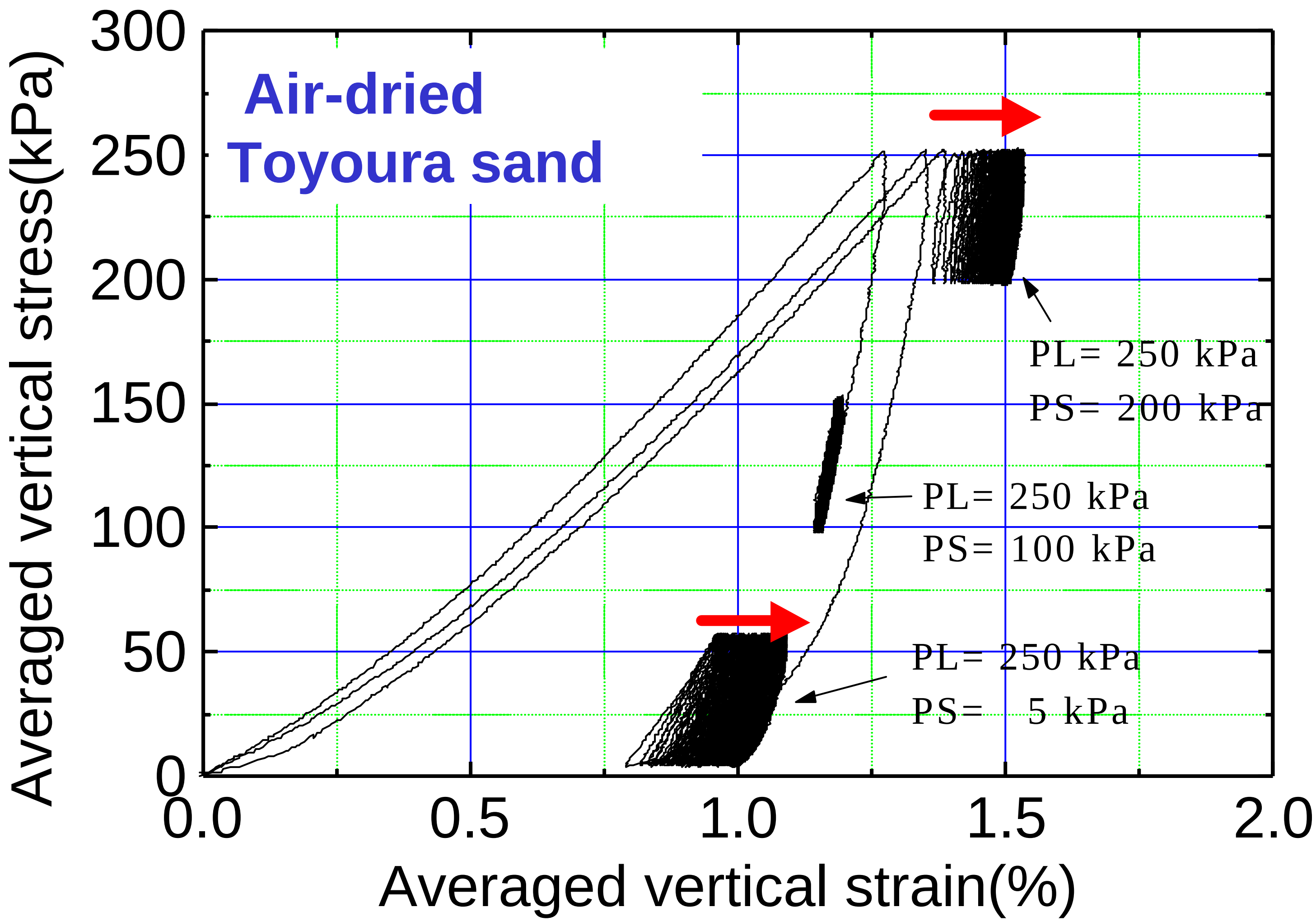


Model of reinforced soil structure



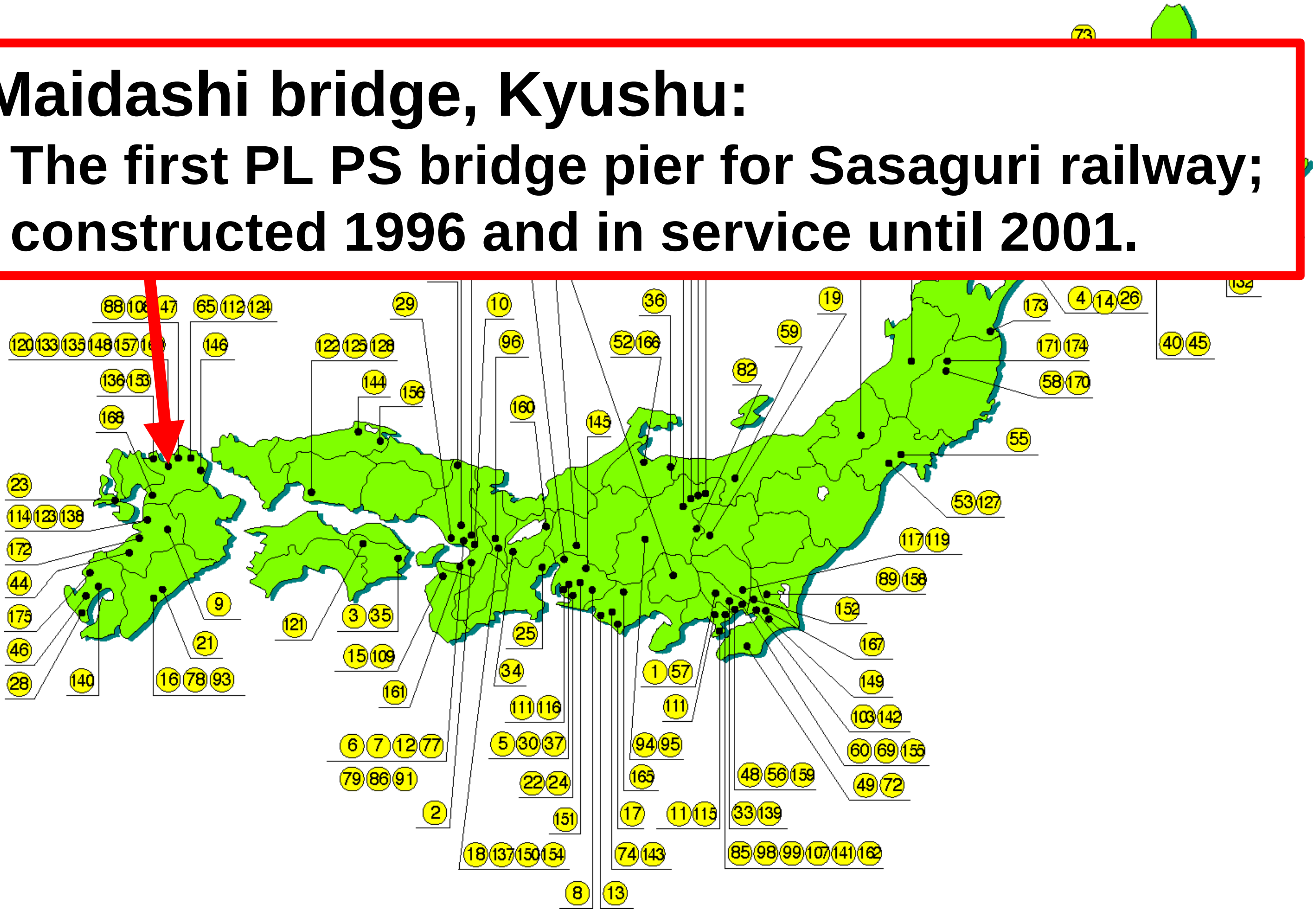


The residual deformation by cyclic loading of the backfill decreases substantially by applying both preload and prestress.



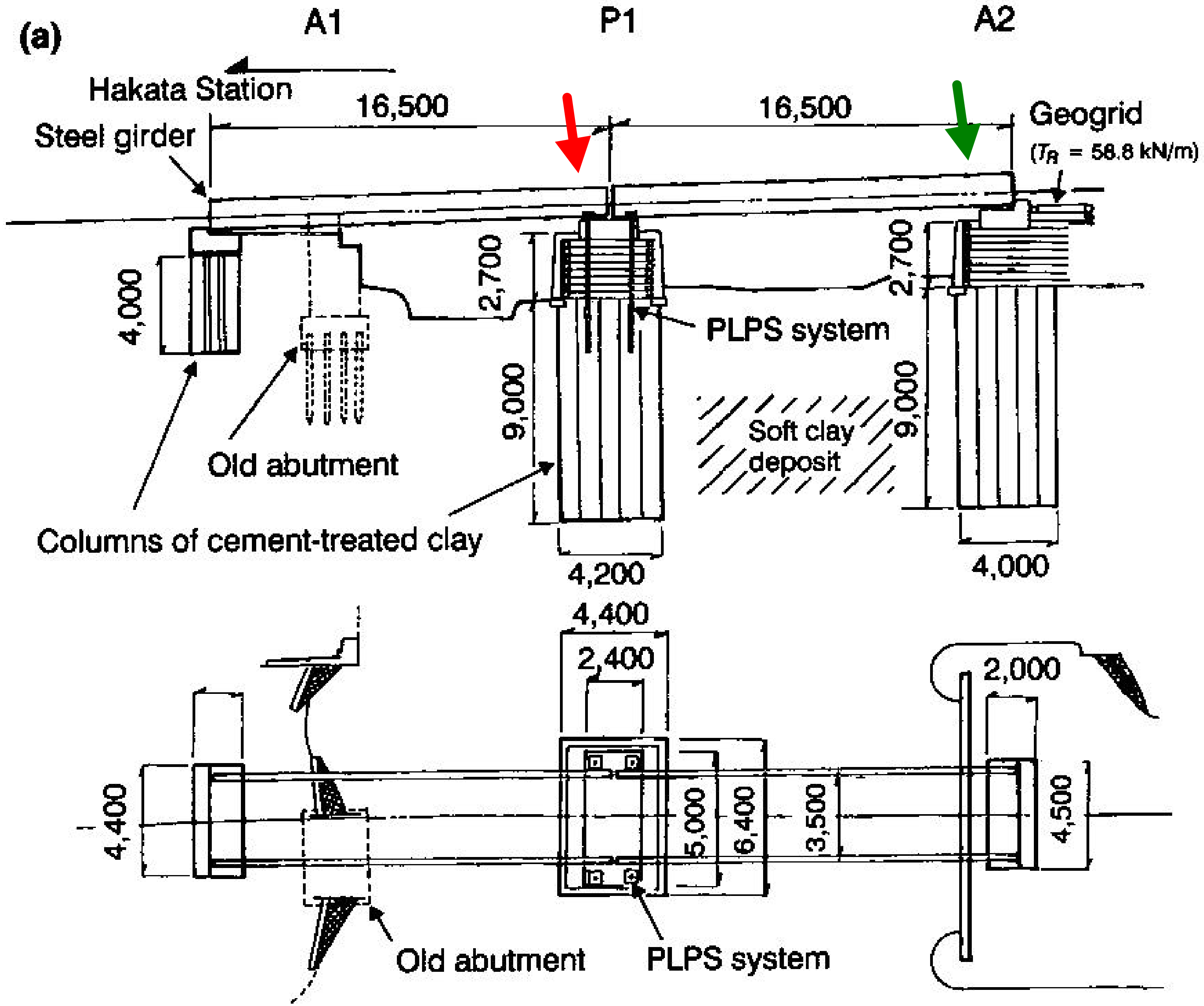
Maidashi bridge, Kyushu:

The first PL PS bridge pier for Sasaguri railway; constructed 1996 and in service until 2001.



Sites of GRS RWs with a full-height rigid facing that have been constructed by the end of April 2000

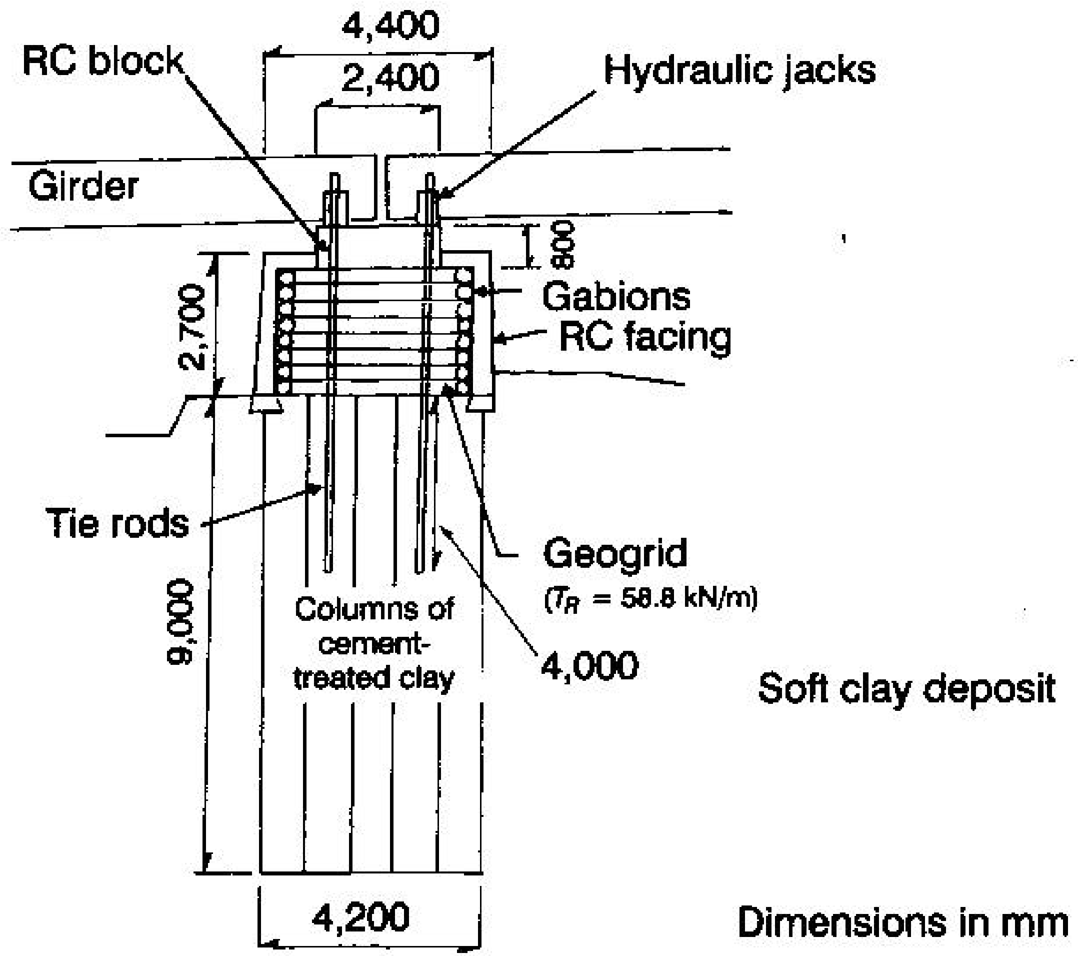
PL PS GRS bridge pier vs. *GRS abutment without PL & PS*



PL PS GRS pier

GRS abutment without PLPS

Backfill; a well-graded gravel supporting two 16.5 m-long girder



Dead load of the girder
= 20 tonf;
Design live load by train
= 136 tonf;
Preload= 240 tonf*; and
Initial prestress= 100 tonf.

*** 240= 100 + 136 +** a

Construction of the PL PS GRS bridge pier

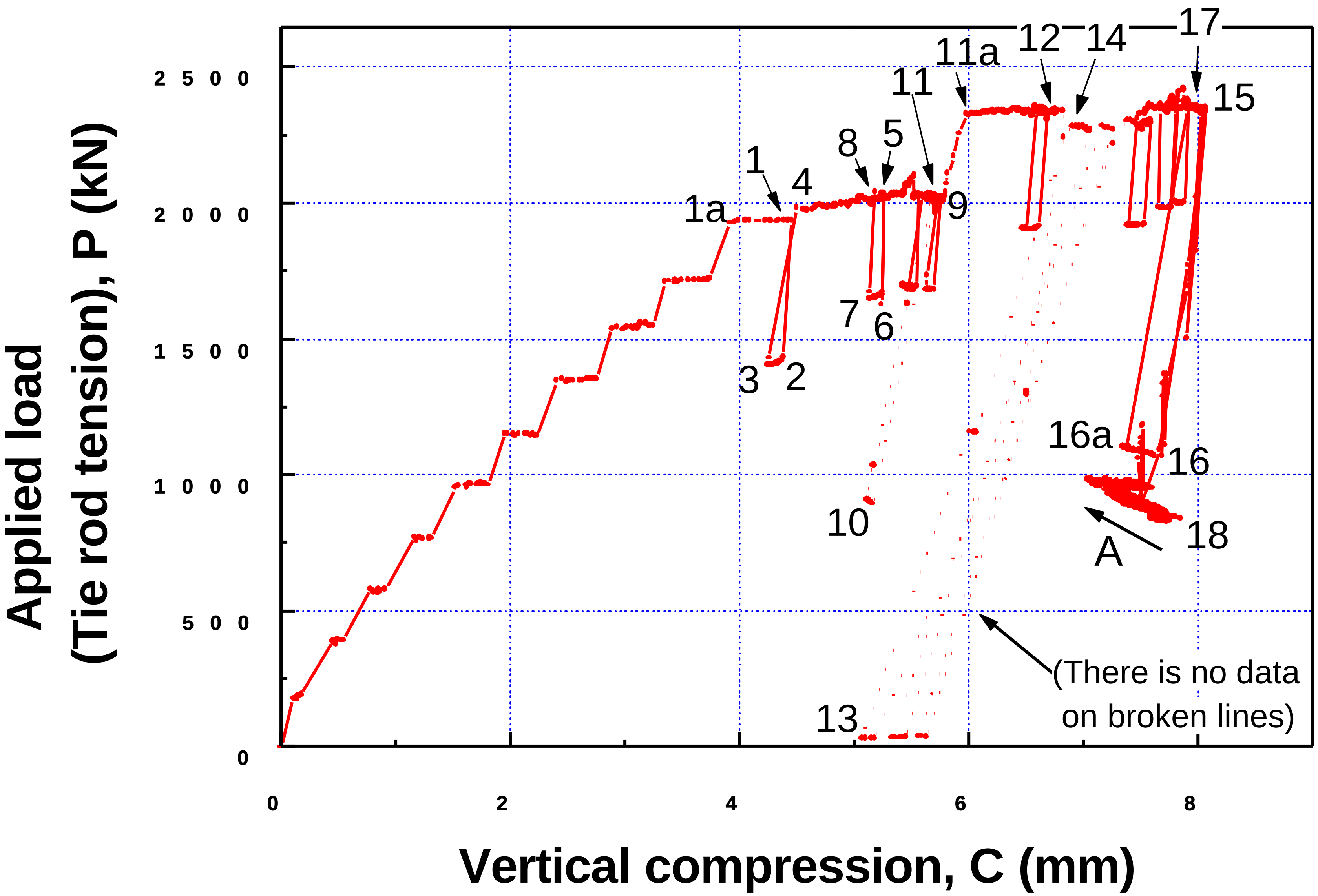
Construction of the PL PS GRS bridge pier

Construction of the PL PS GRS bridge pier

Construction of the PL PS GRS bridge pier

Application of PL PS GRS by using hydraulic jacks

Application of PL PS GRS by using hydraulic jacks



How much is the compression of the backfill ?

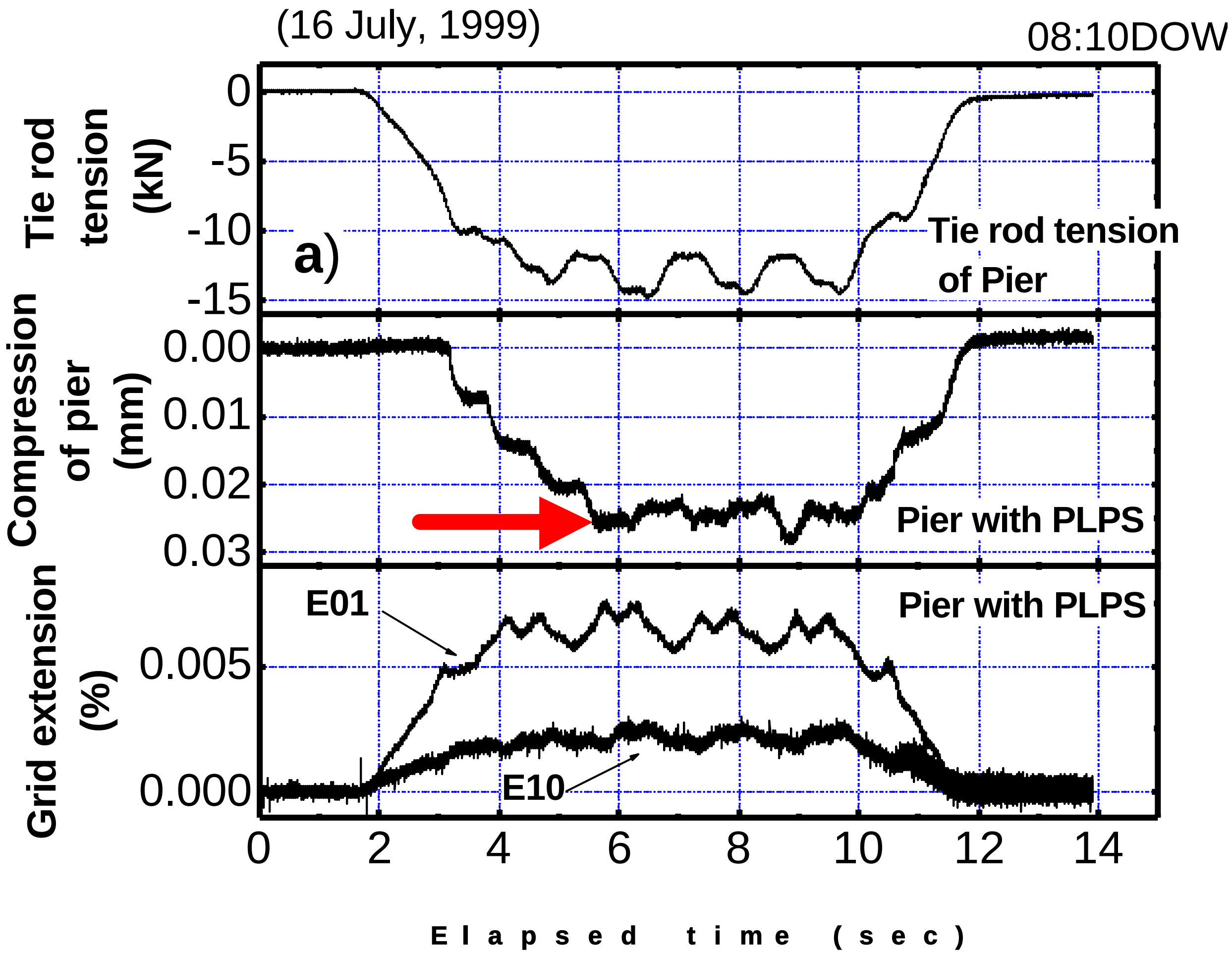
The compression of the backfill is 0.025 mm

Behaviour during train passing of PLPS GRS pier:

a compression of only 0.025 mm !

or

a compressive strain of only 0.001 % (within the elastic limit)!



PLPS GRS pier:

a compression of only 0.025 mm !

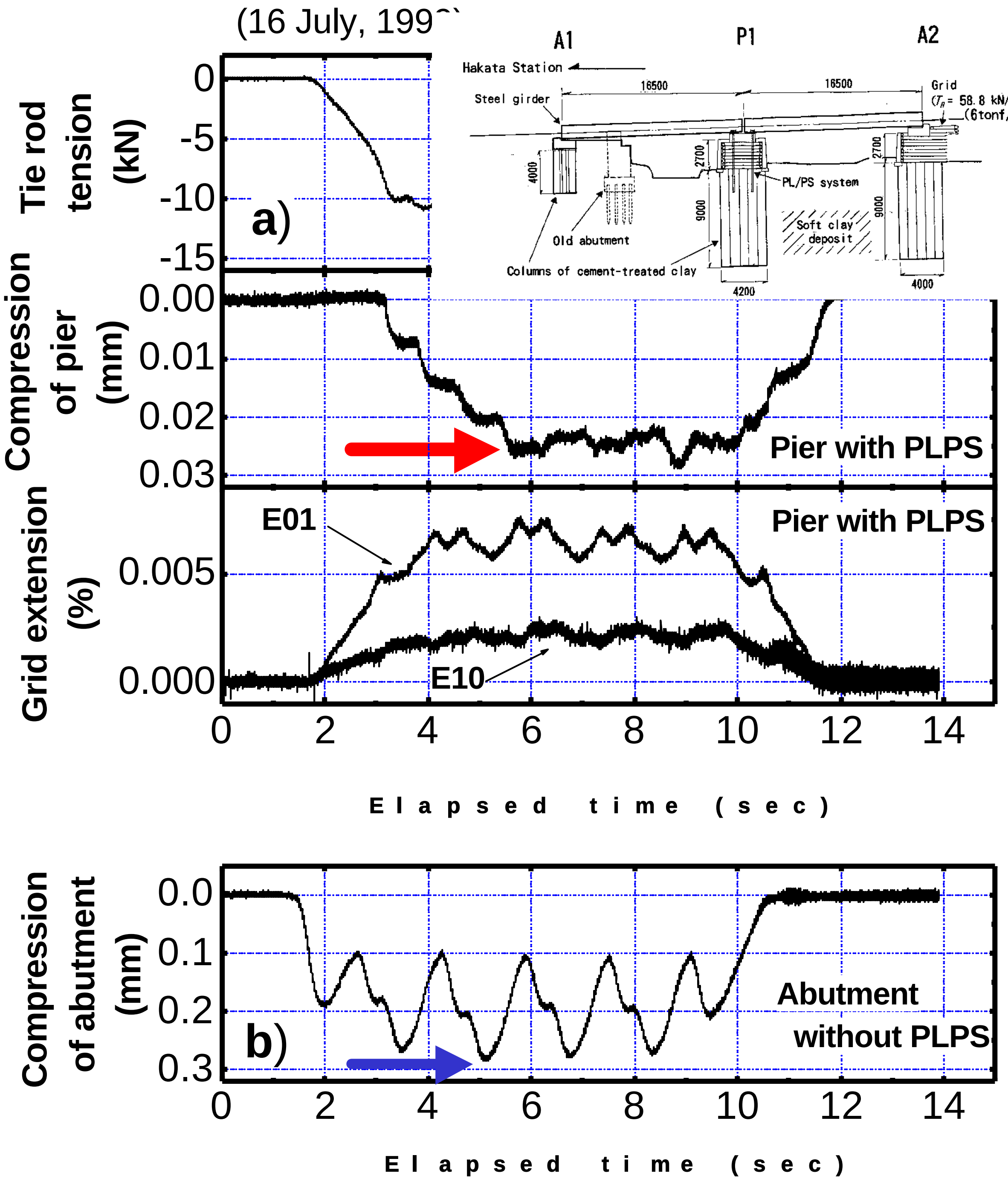
or

a compressive strain of only 0.001 % (within the elastic limit)!

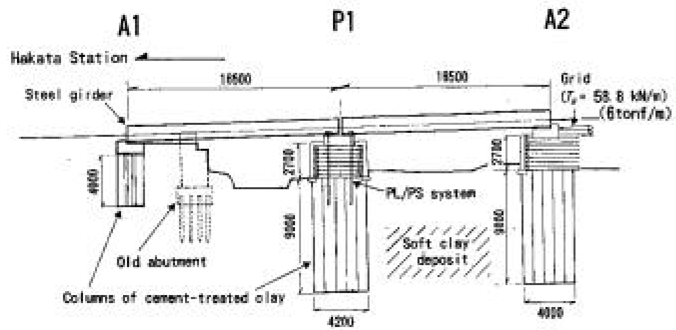
GRS abutment without PLPS:

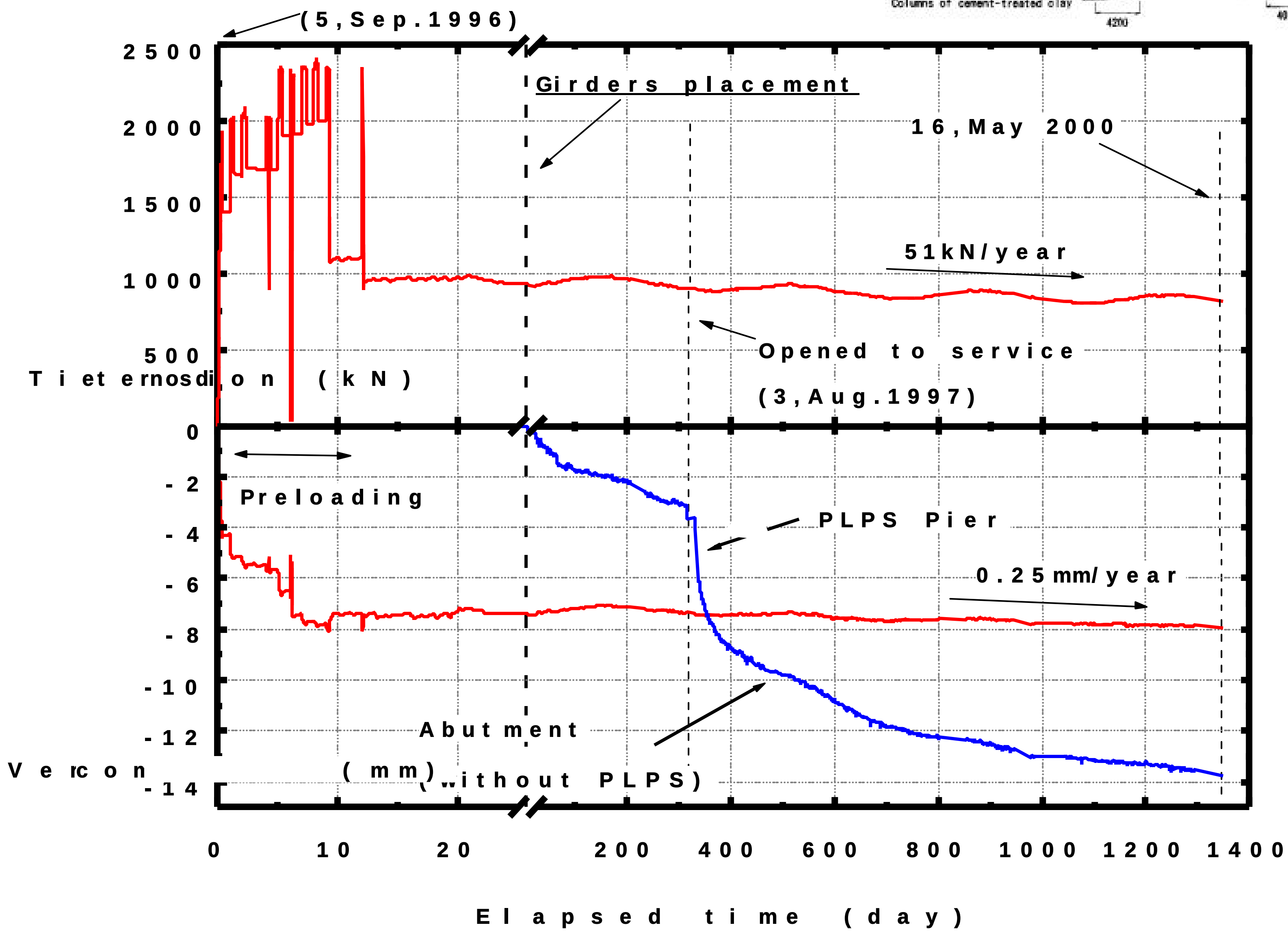
a compression of 0.25 mm or

a compressive strain of 0.01 % (exceeding the elastic limit !)



Long-term behaviour





A very high performance of the PLPS GRS bridge pier for daily train load,

without showing;

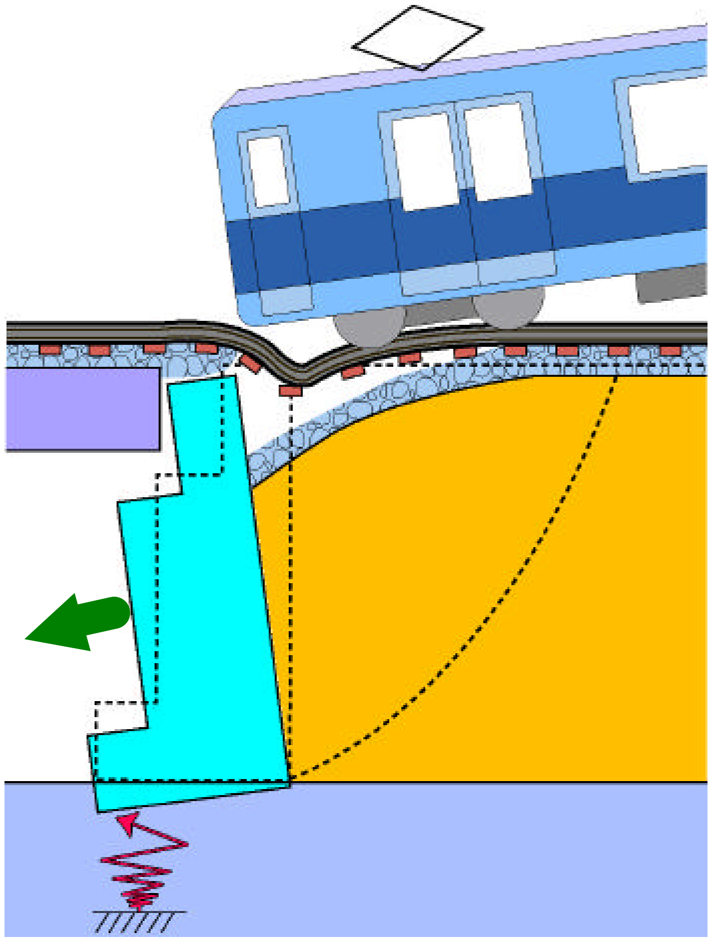
1) noticeable settlement; and

2) noticeable reduction in the prestress.

DEVELOPMENT OF NEW TYPE ASEISMIC BRIDGE ABUTMENT

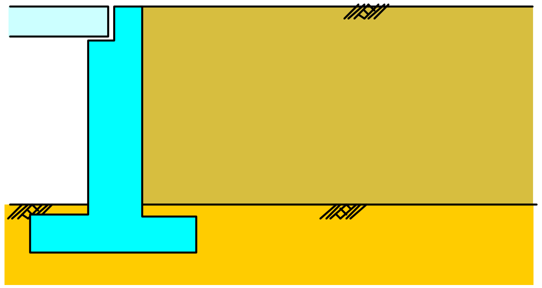
Contents:

Background



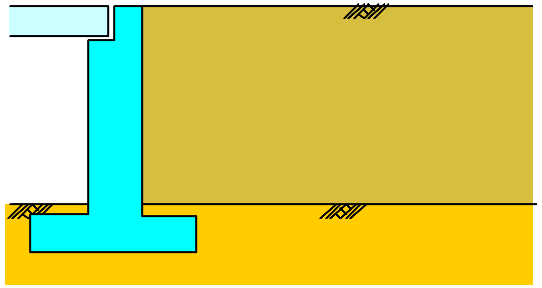
- Loss of the stability of abutment & backfill by seismic loading.
- Differential lateral displacement and settlement between the abutment and the backfill.

Strong needs for the development of new bridge abutment systems having a substantial high seismic stability and a high-cost effectiveness

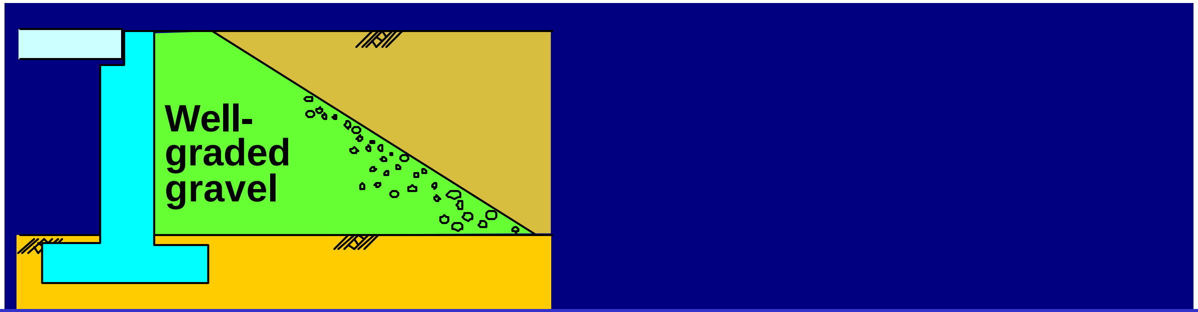


Ordinary backfill without improvement

Possible solutions at different levels-1

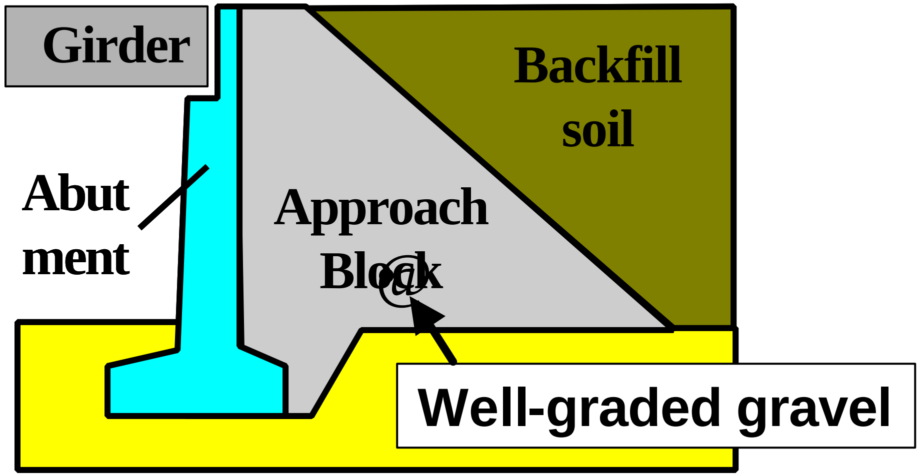


Ordinary backfill without improvement



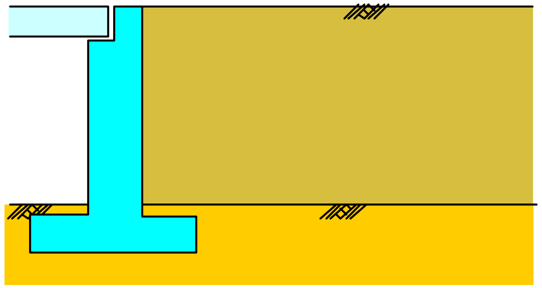
Measures to prevent a large settlement of backfill (already adopted)

Not satisfactory performance during previous earthquakes

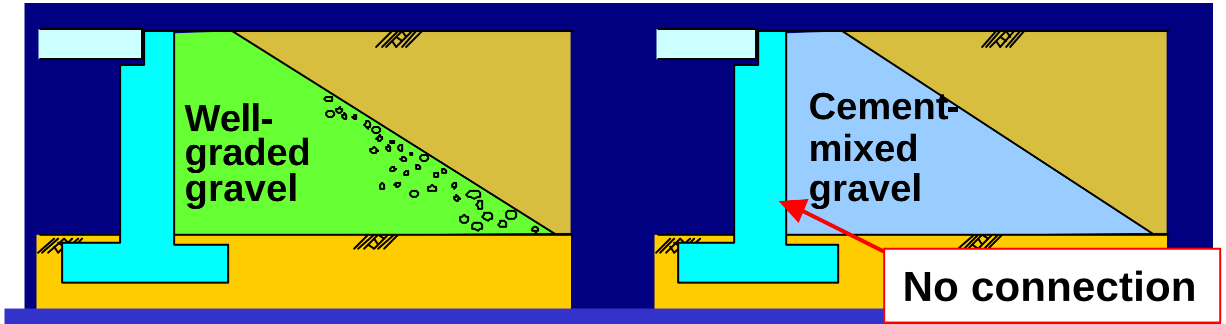


A settlement of the backfill relative to an abutment, Arikawa bridge, Tsugaru-Kaikyo Line, East Japan Railway, Hokkaido Nansei-oki Earthquake, 12 July 1993

Possible solutions at different levels-2

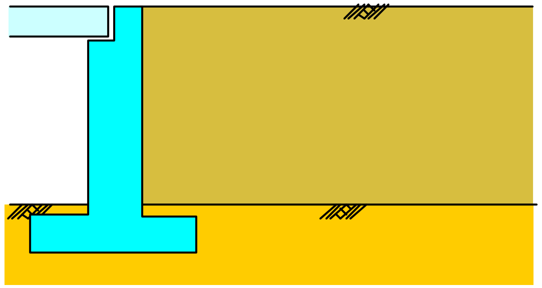


Ordinary backfill
without improvement

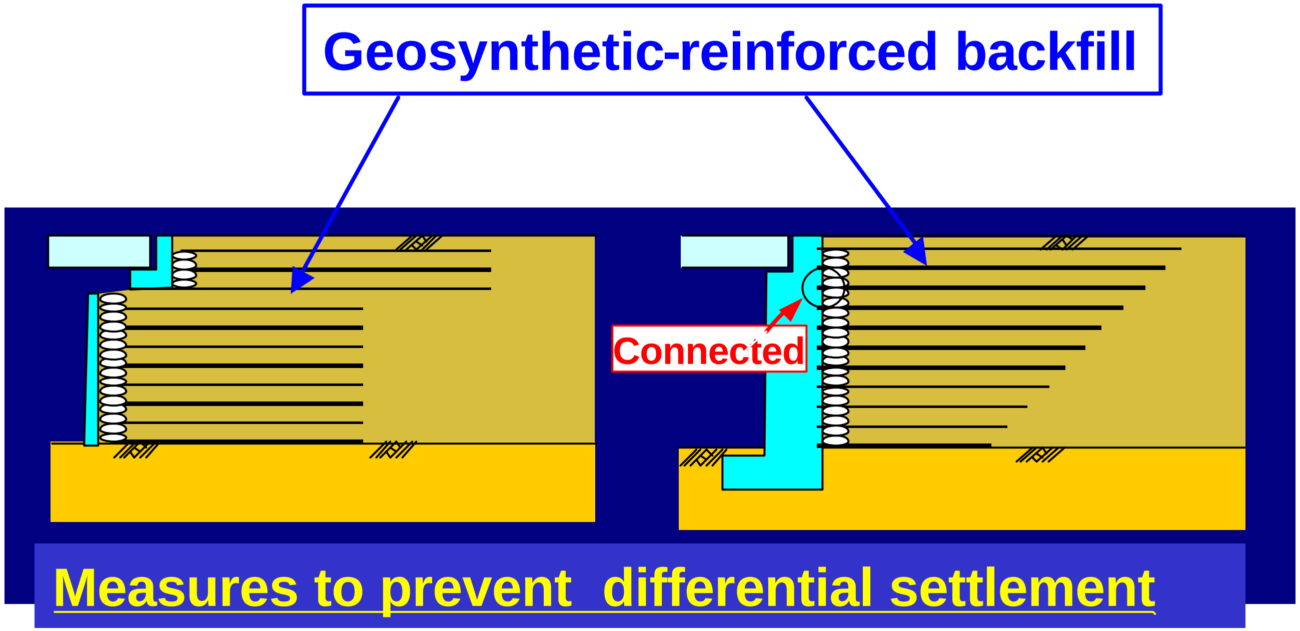


Measures to prevent a large settlement of backfill (already adopted)

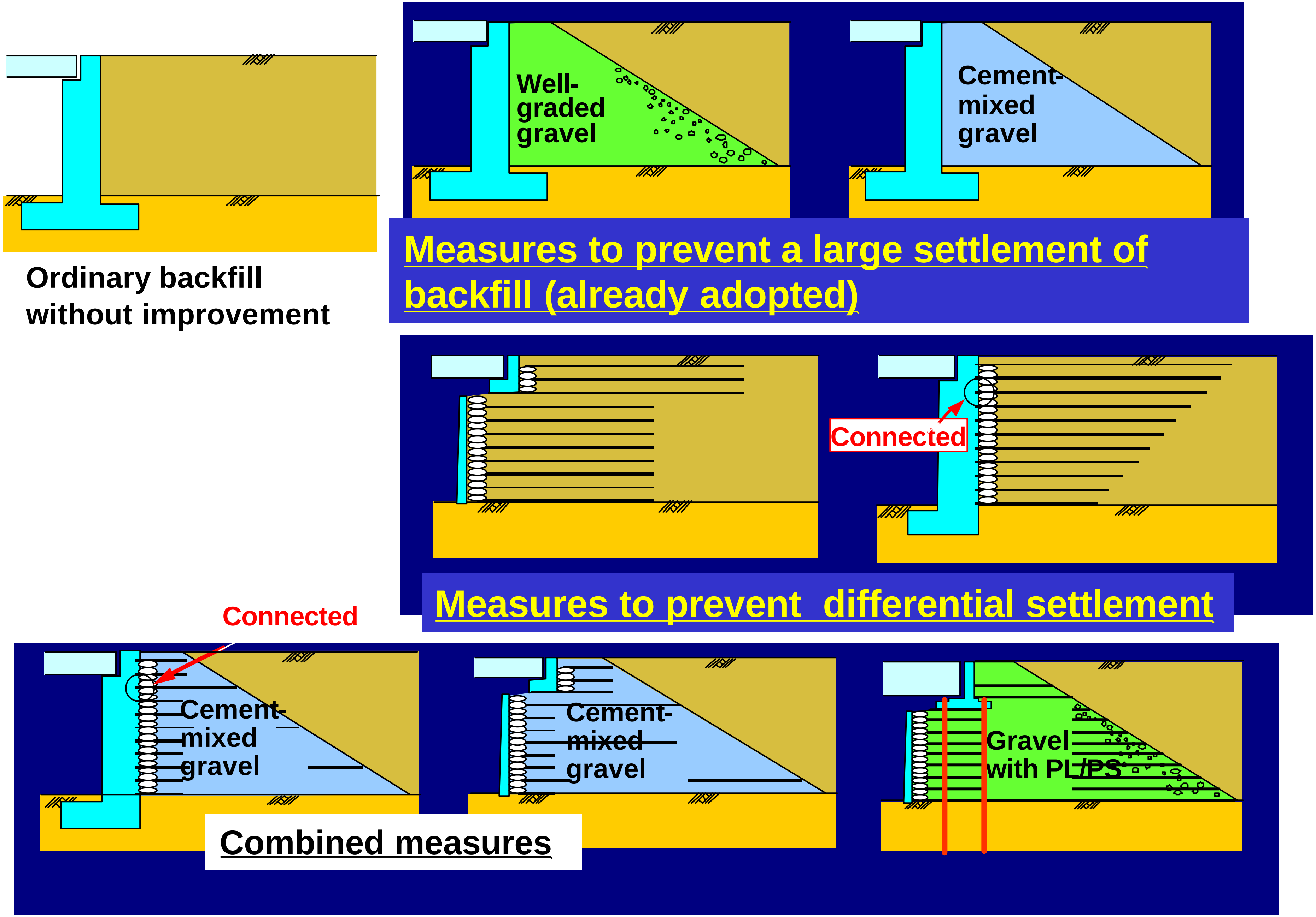
Possible solutions at different levels-3



Ordinary backfill without improvement

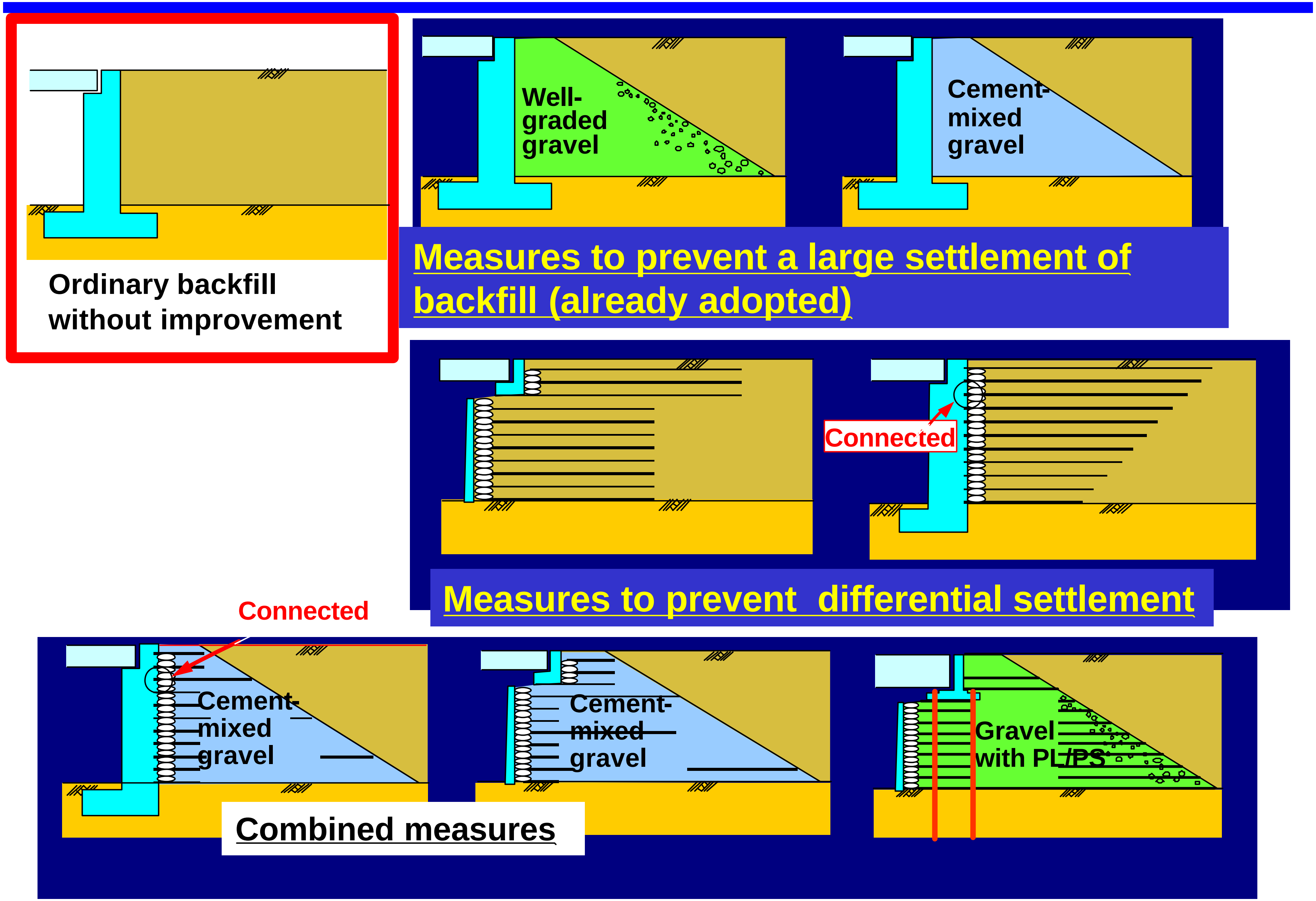


Possible solutions at different levels-4

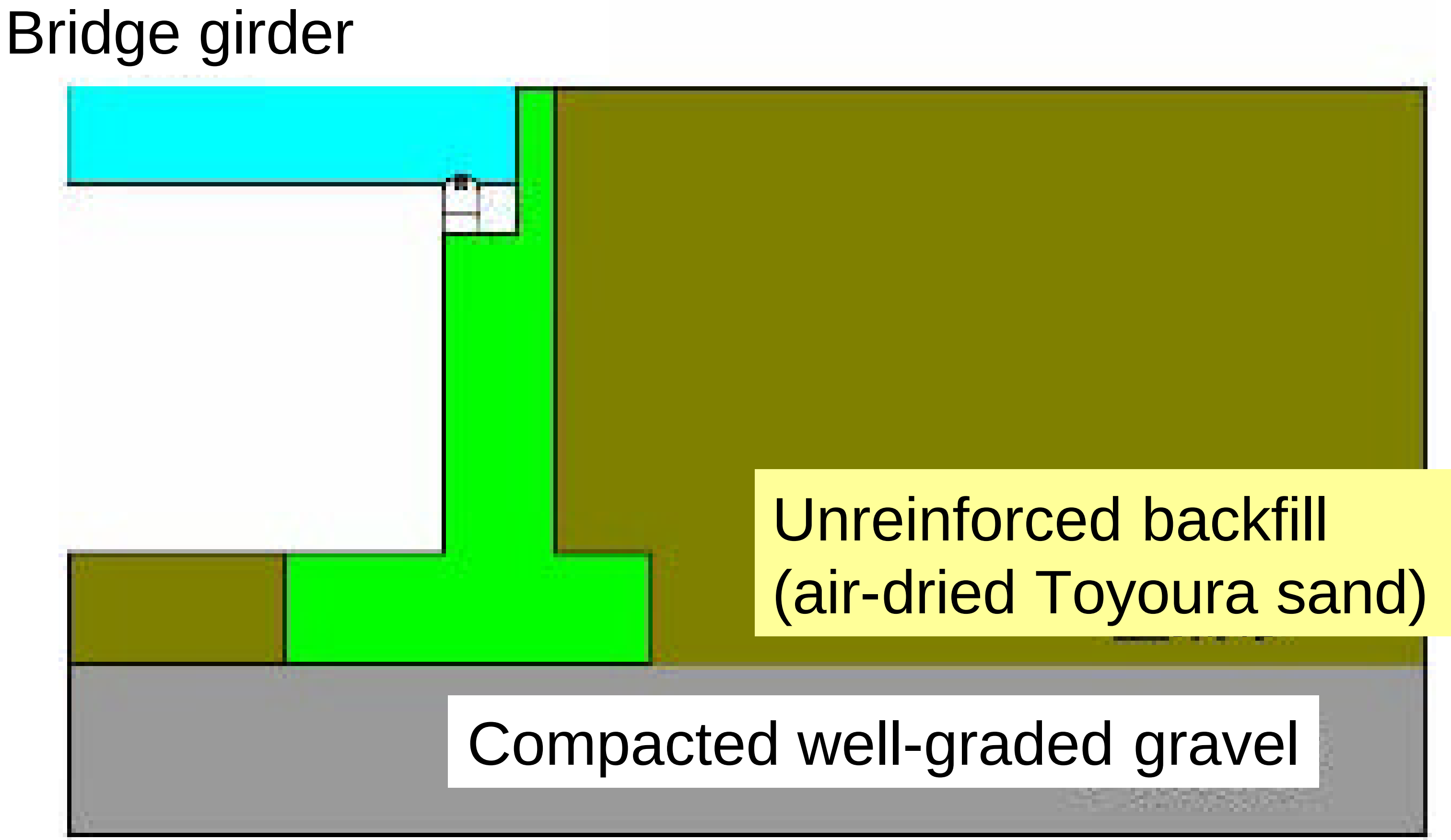


Contents:

Shaking table tests to evaluate the possible solutions

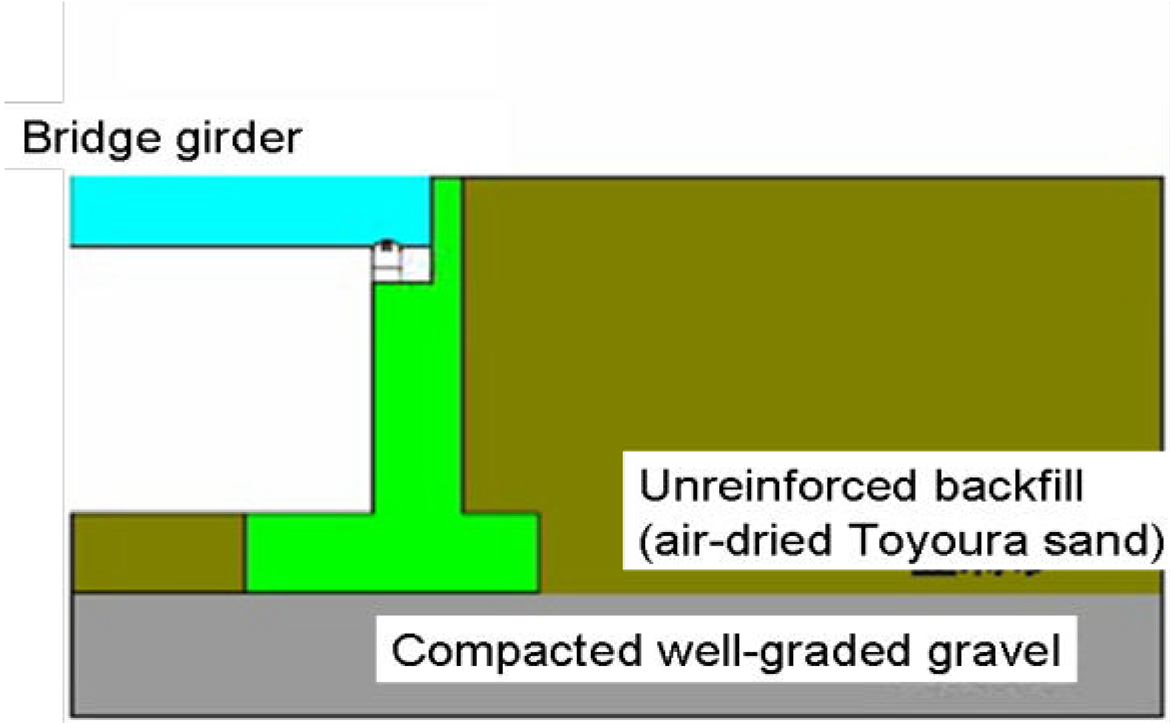


Most conventional



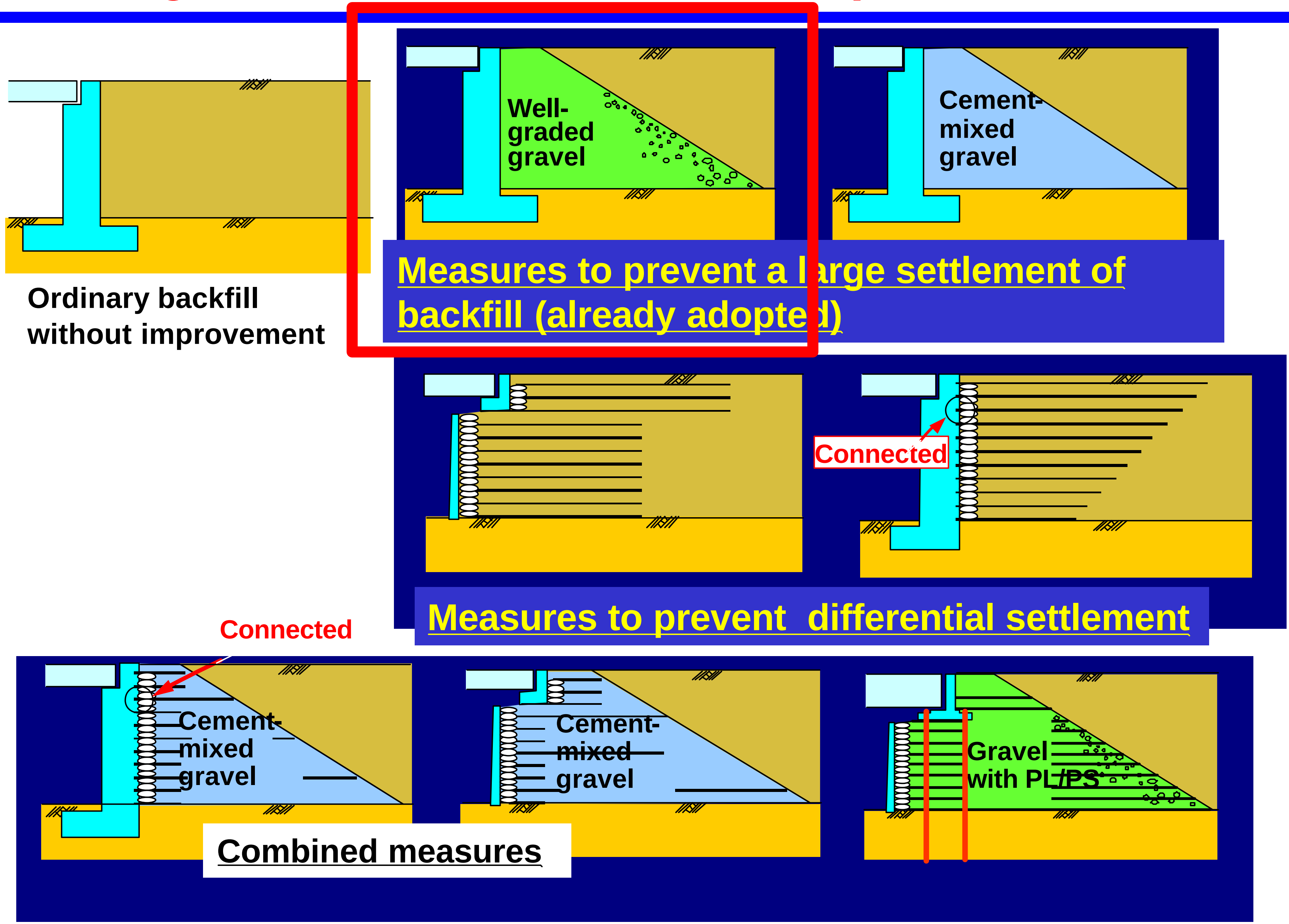
Sinusoidal 350 gals

Most conventional

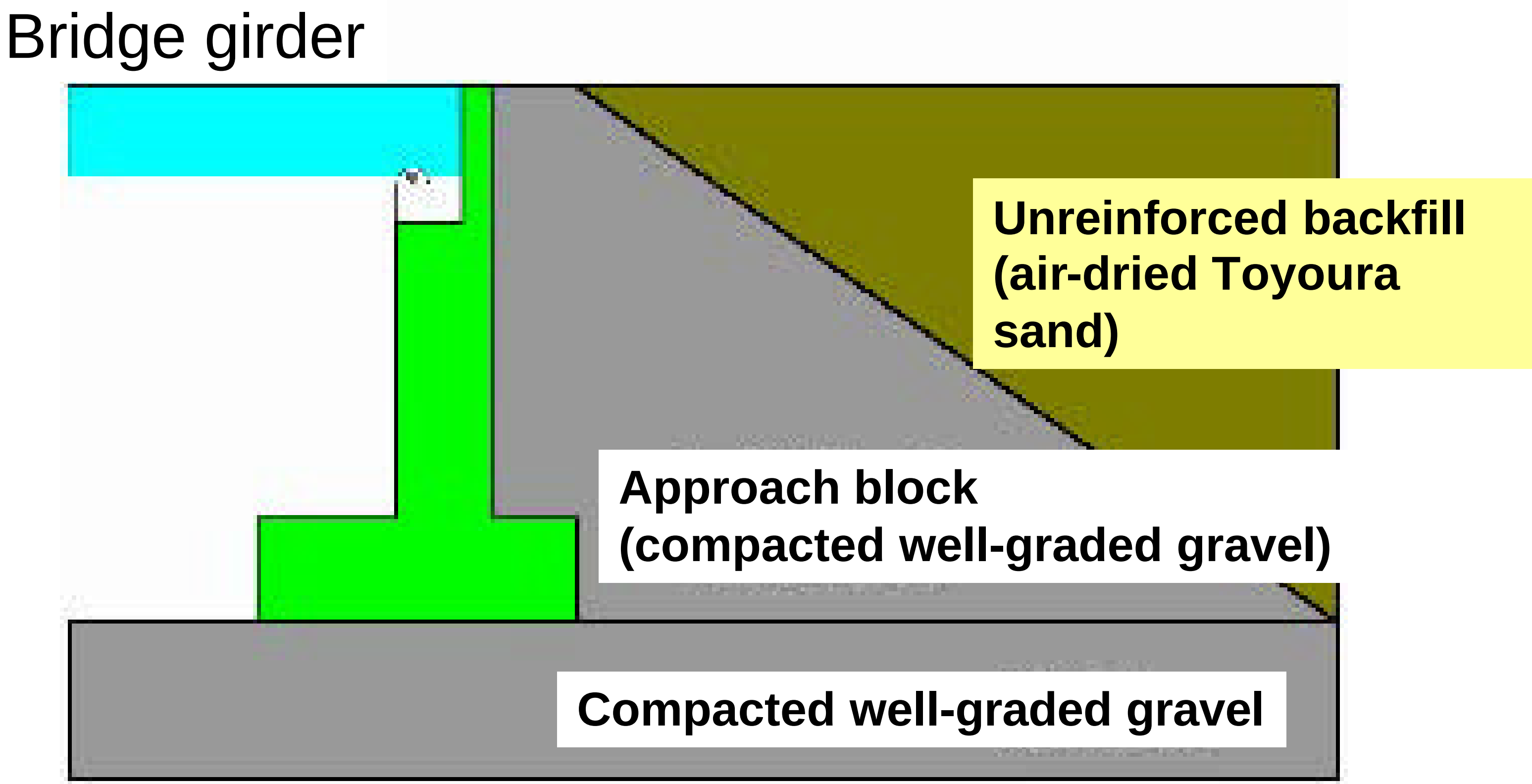


-A very low seismic stability !
-The backfill is less stable than the abutment !
-The active earth pressure increases during dynamic loading !

Shaking table tests to evaluate the possible solutions

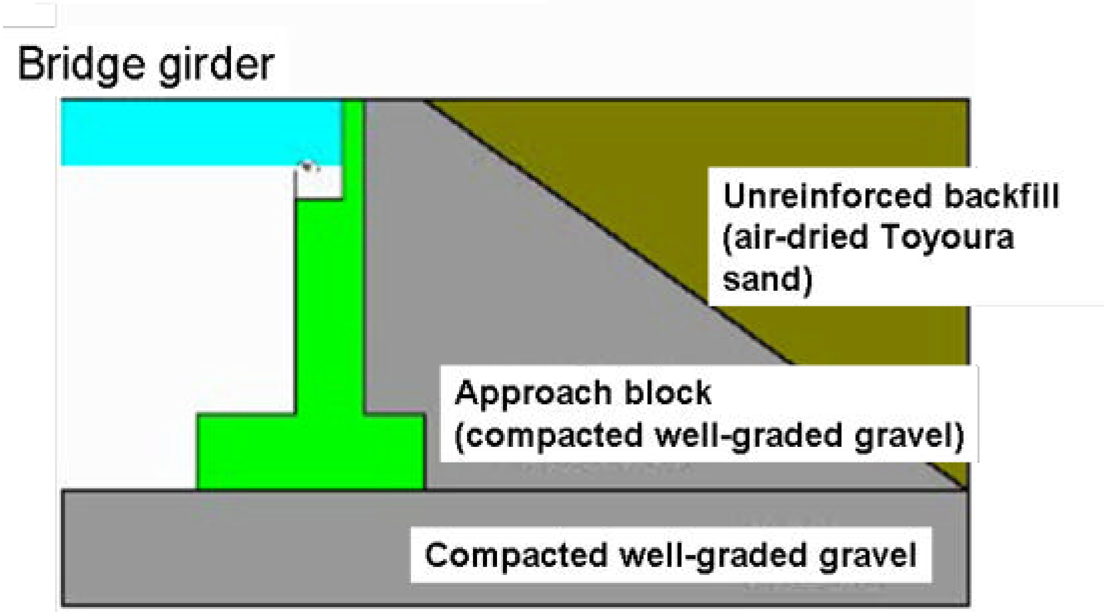


With an approach block of a well-graded gravel to prevent a large settlement of backfill (already adopted)



Sinusoidal 450 gals

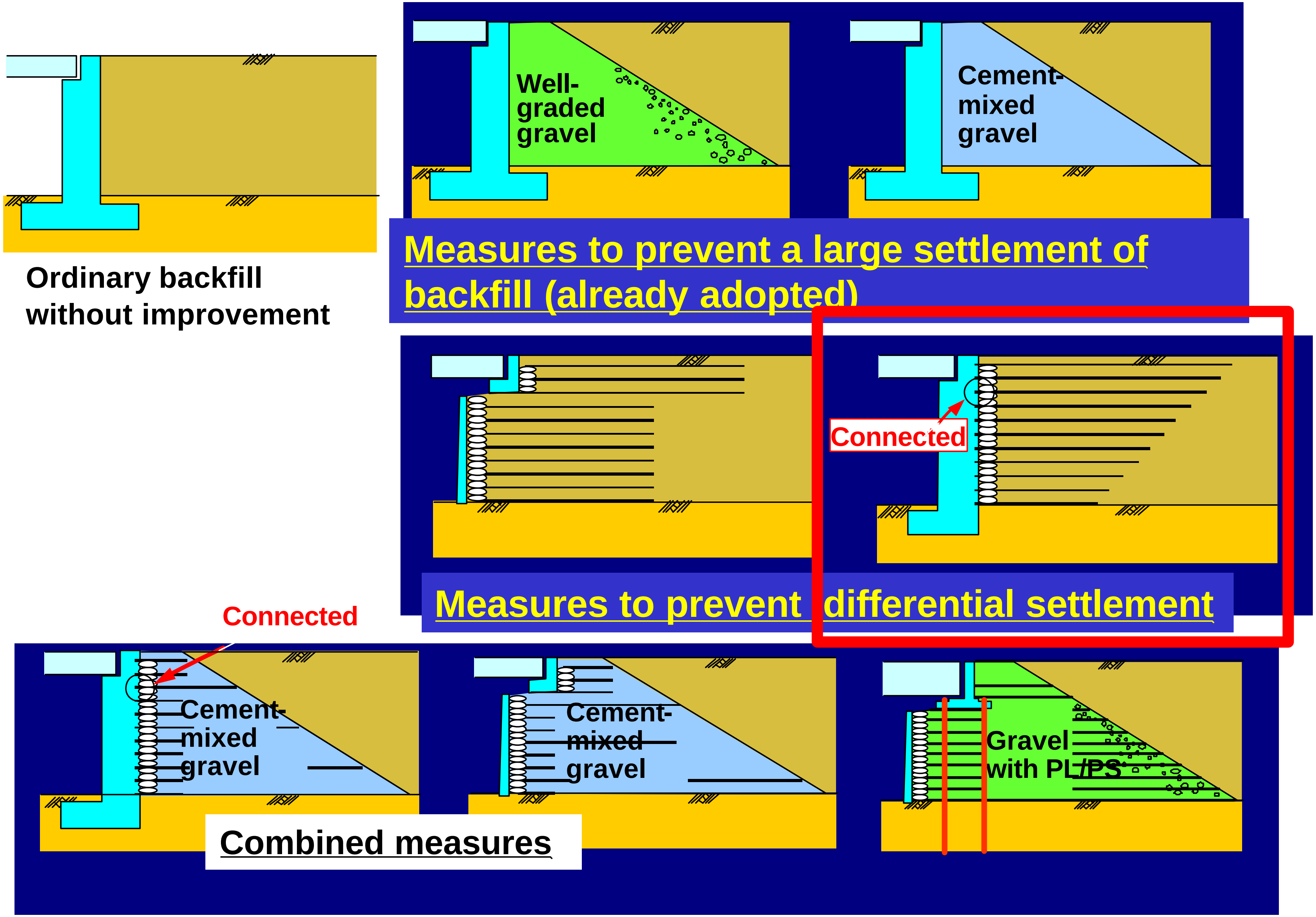
With an approach block of a well-graded gravel



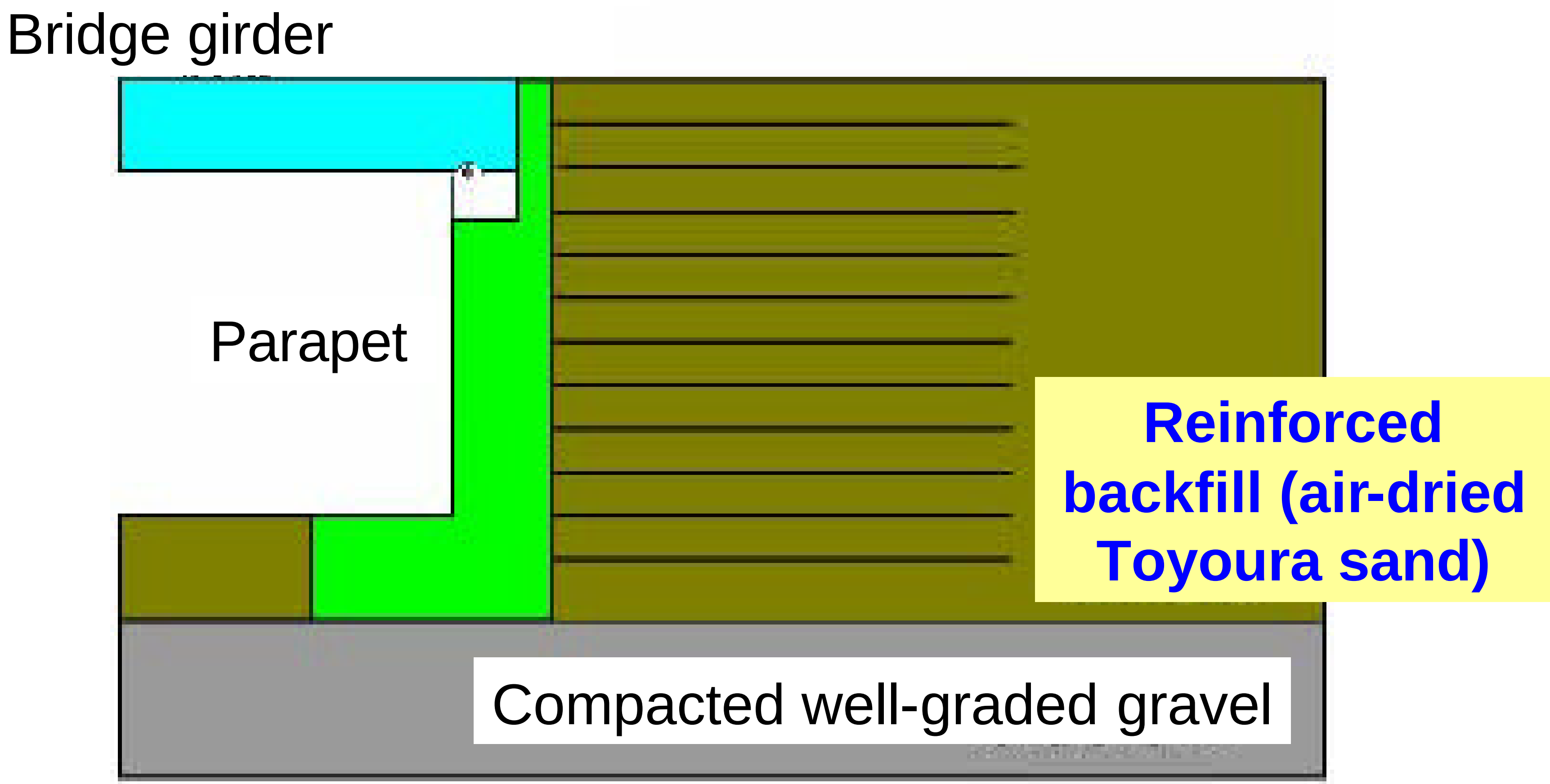
-Not sufficiently stable !

-Too large relative movement between the RC facing structure (parapet) and the backfill

Shaking table tests to evaluate the possible solutions

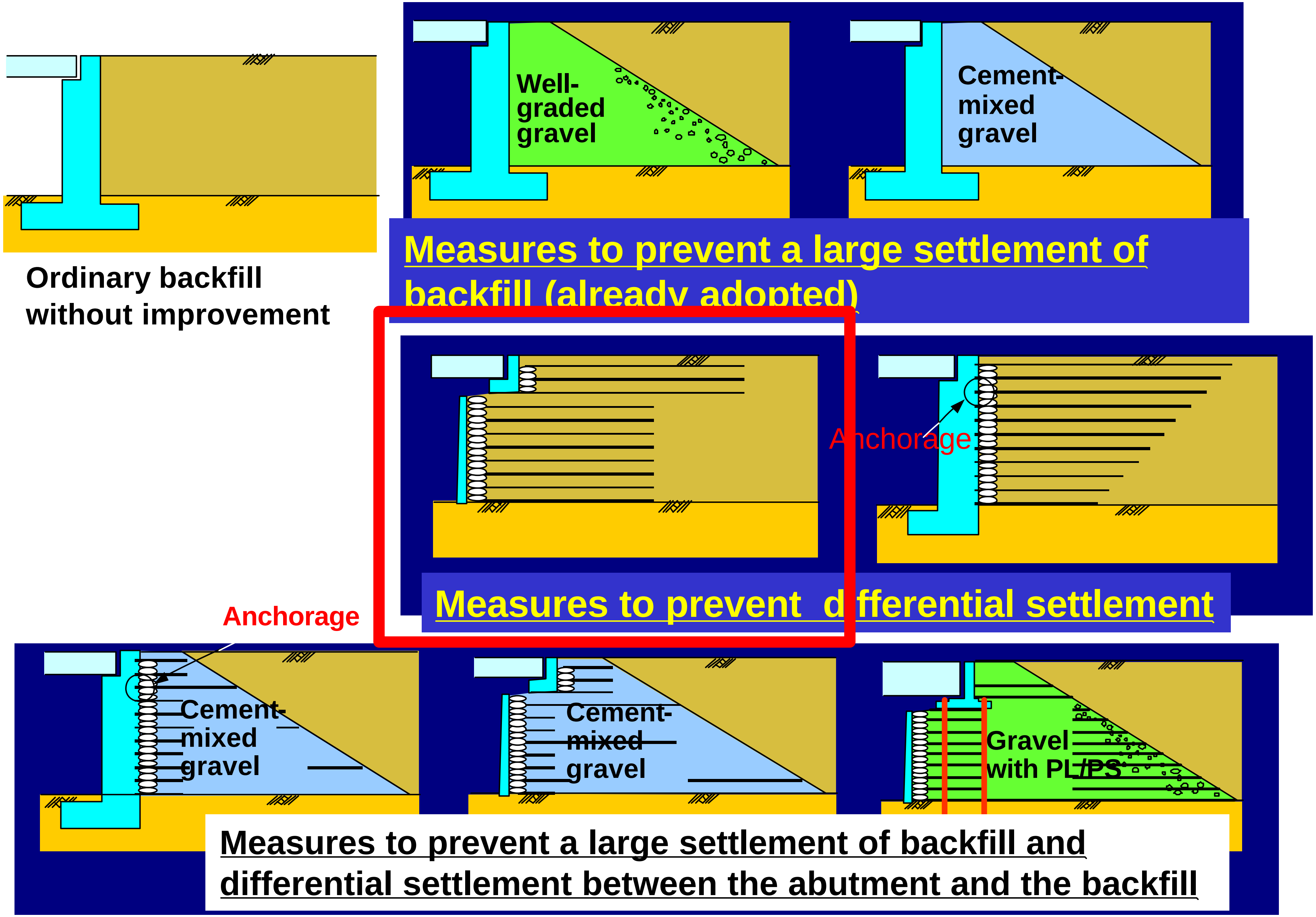


A parapet with geotextile-reinforced backfill with a firm connection between reinforcement and parapet to prevent a relative settlement; *the bridge girder on the parapet*

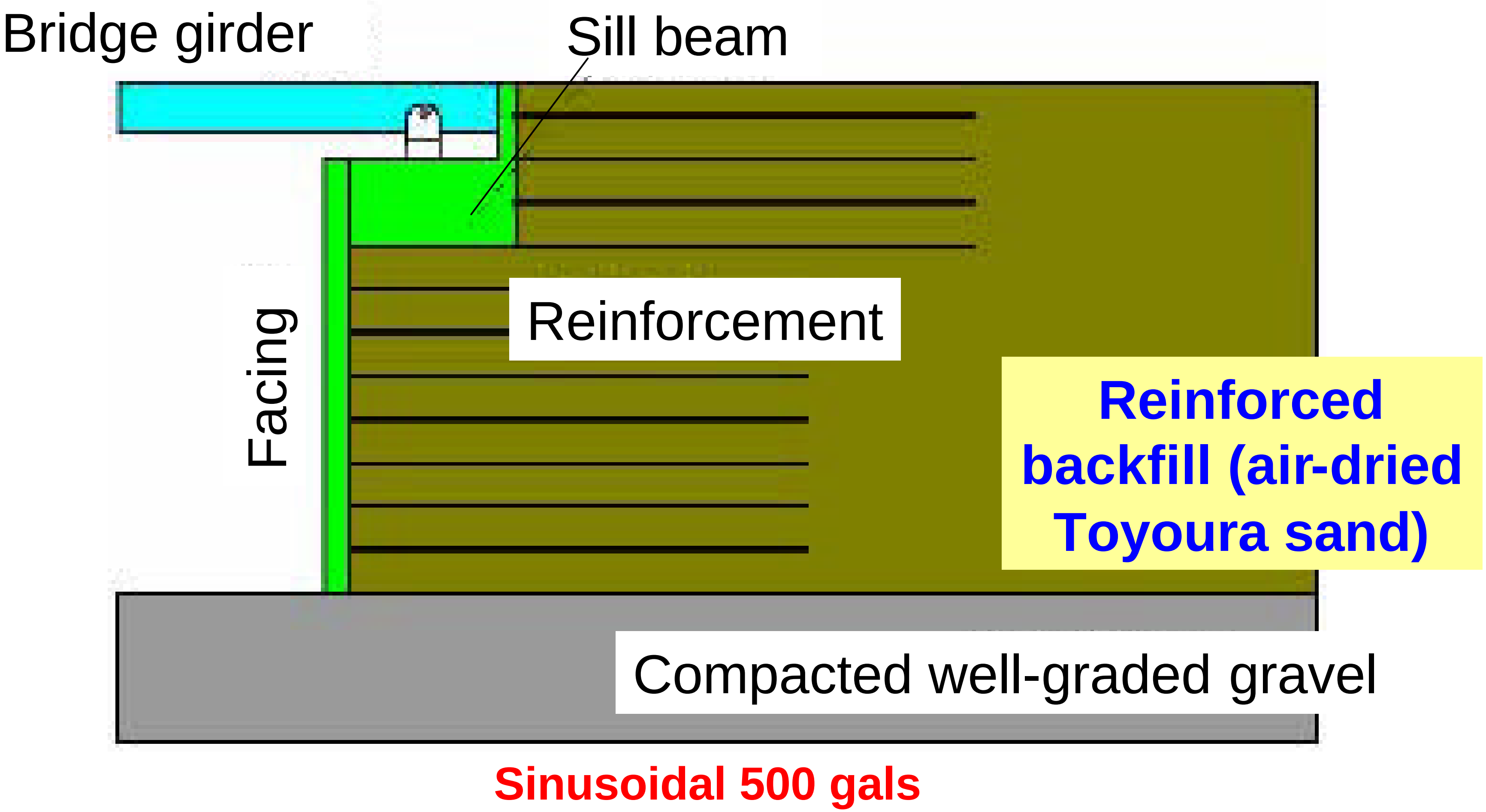


Sinusoidal 500 gals

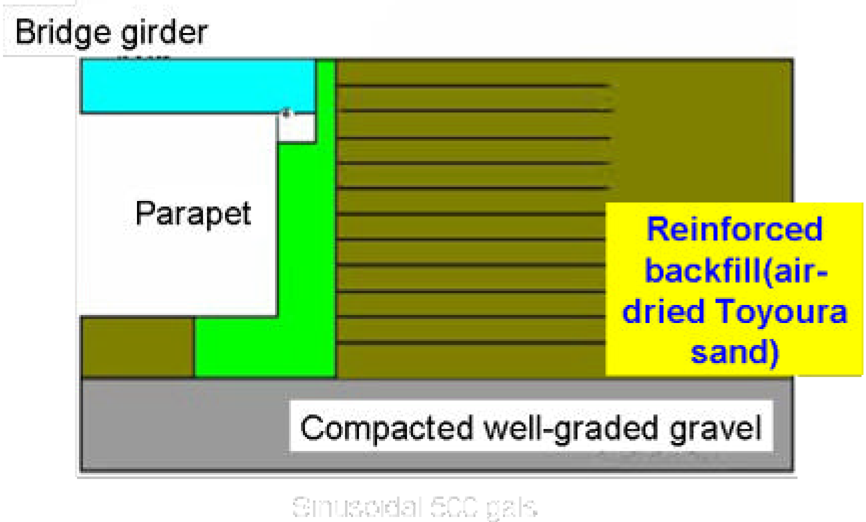
Shaking table tests to evaluate the possible solutions

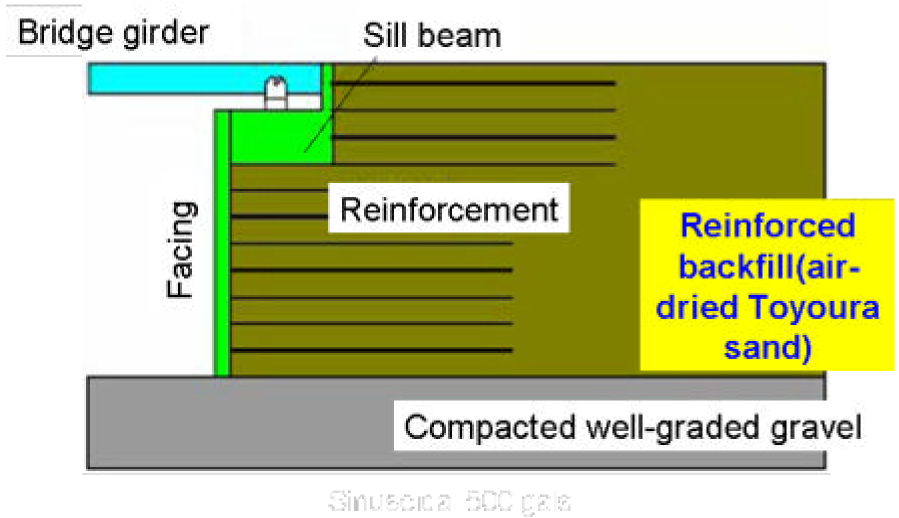


A parapet with geotextile-reinforced backfill with a firm connection between reinforcement and parapet to prevent a relative settlement;
the bridge girder on the backfill



A parapet with geotextile-reinforced backfill with a firm connection between reinforcement and parapet to prevent a relative settlement

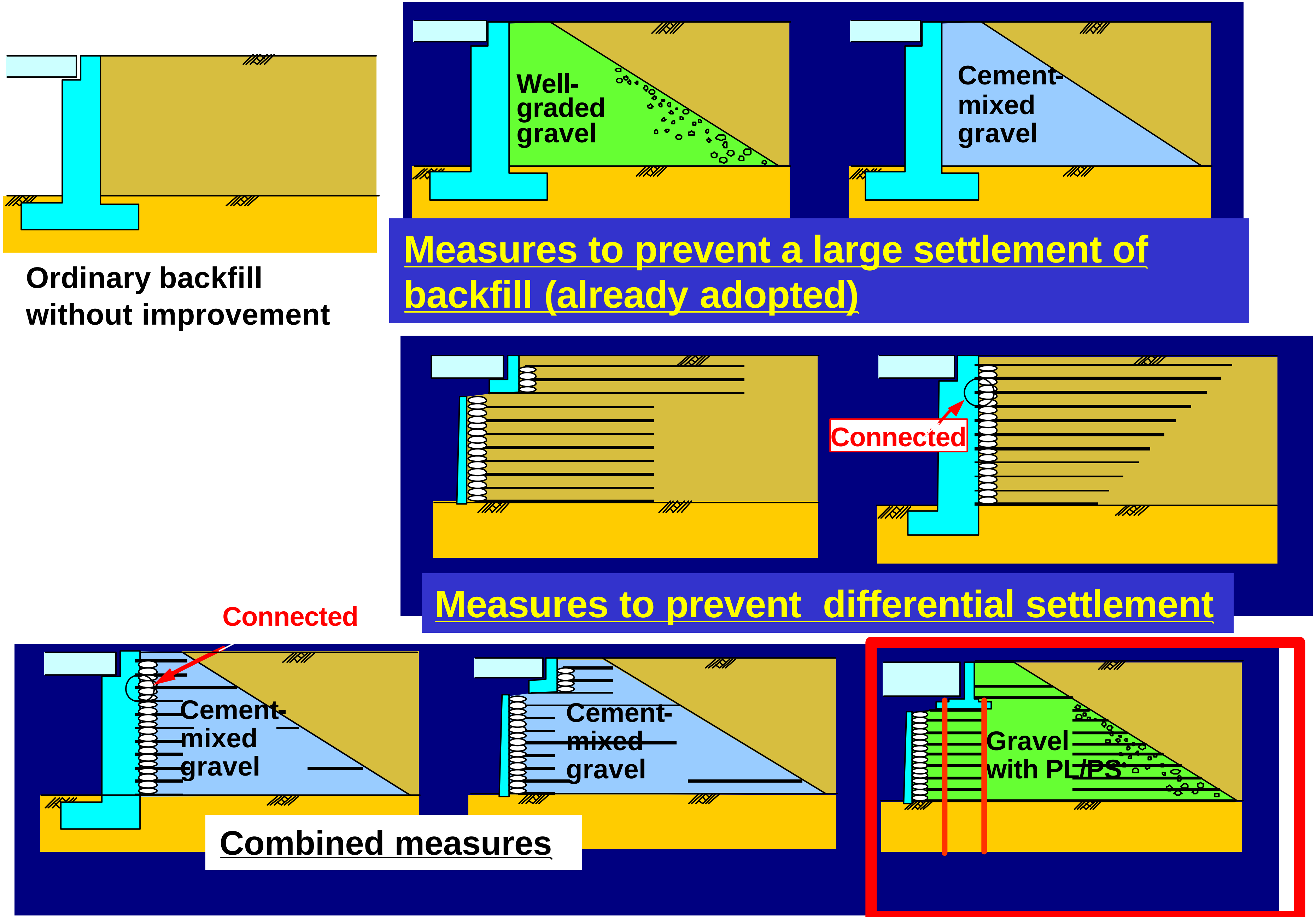




-Reasonably stable,

-But, too deformable to be used as a bridge abutment !

Shaking table tests to evaluate the possible solutions



PLPS bridge abutment worked very well against static load for about five years.

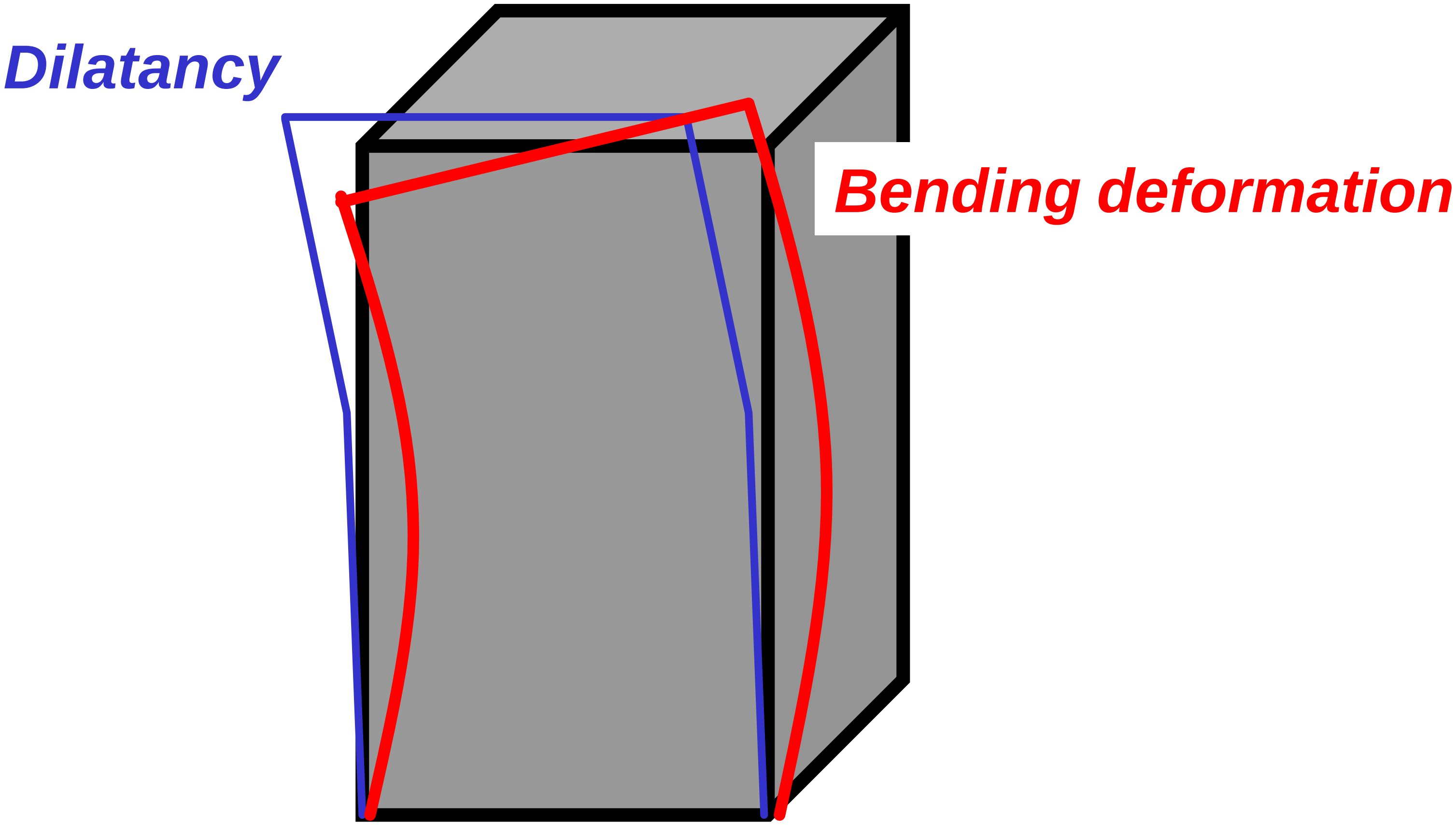
But how about for a long life time ?

How about against strong seismic load ?

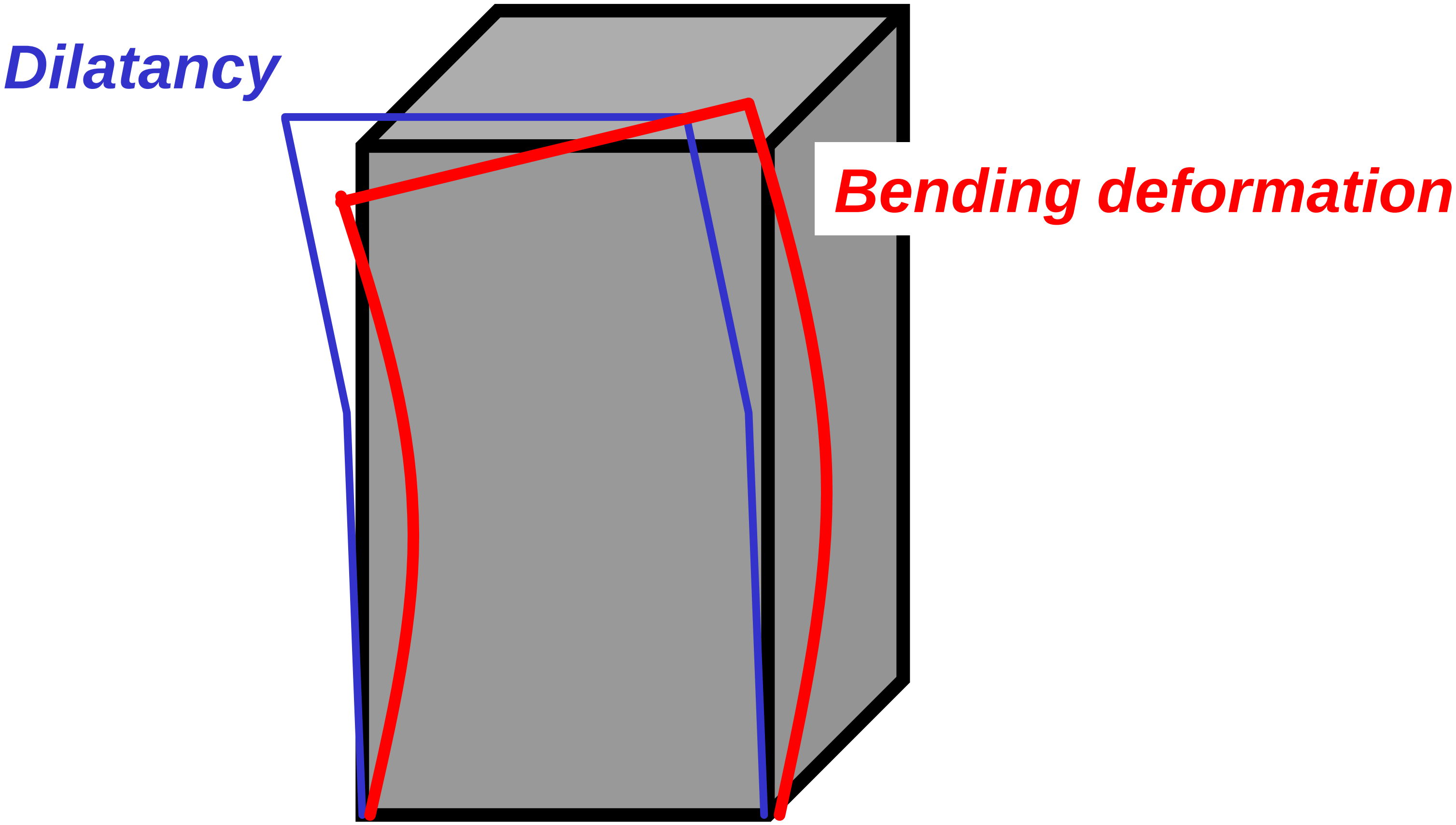
PS can survive these events?

In addition,

to reduce the bending deformation, the increase in the backfill height should be restrained !



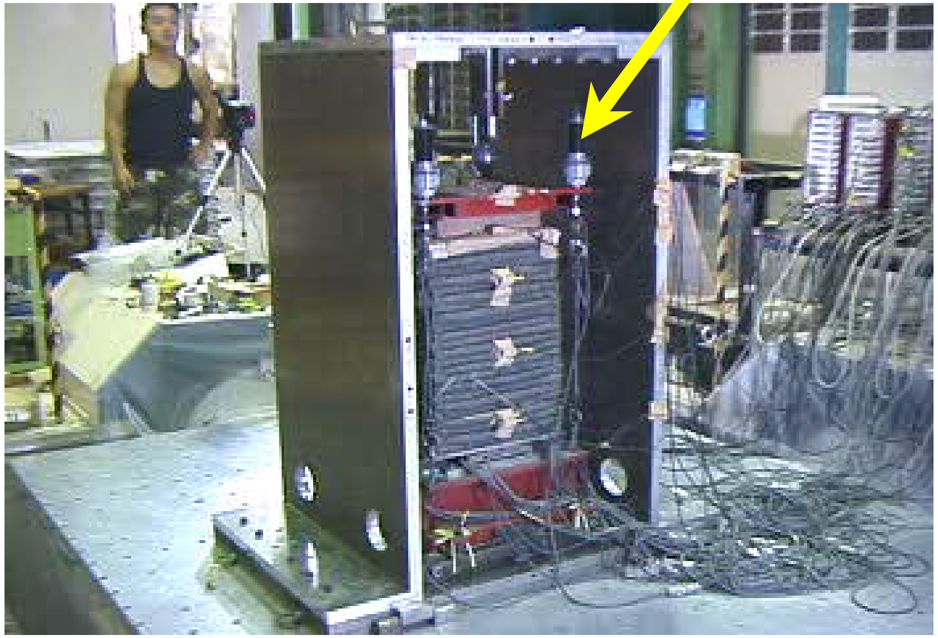
Furthermore, the vertical stress largely increases by restraining the increase in the height due to dilatancy, which makes the strength of the backfill very large.



To achieve a substantially high seismic stability of PLPS structure: the use of a ratchet system

Shaking table tests (700 gal, 5Hz, 25sec)

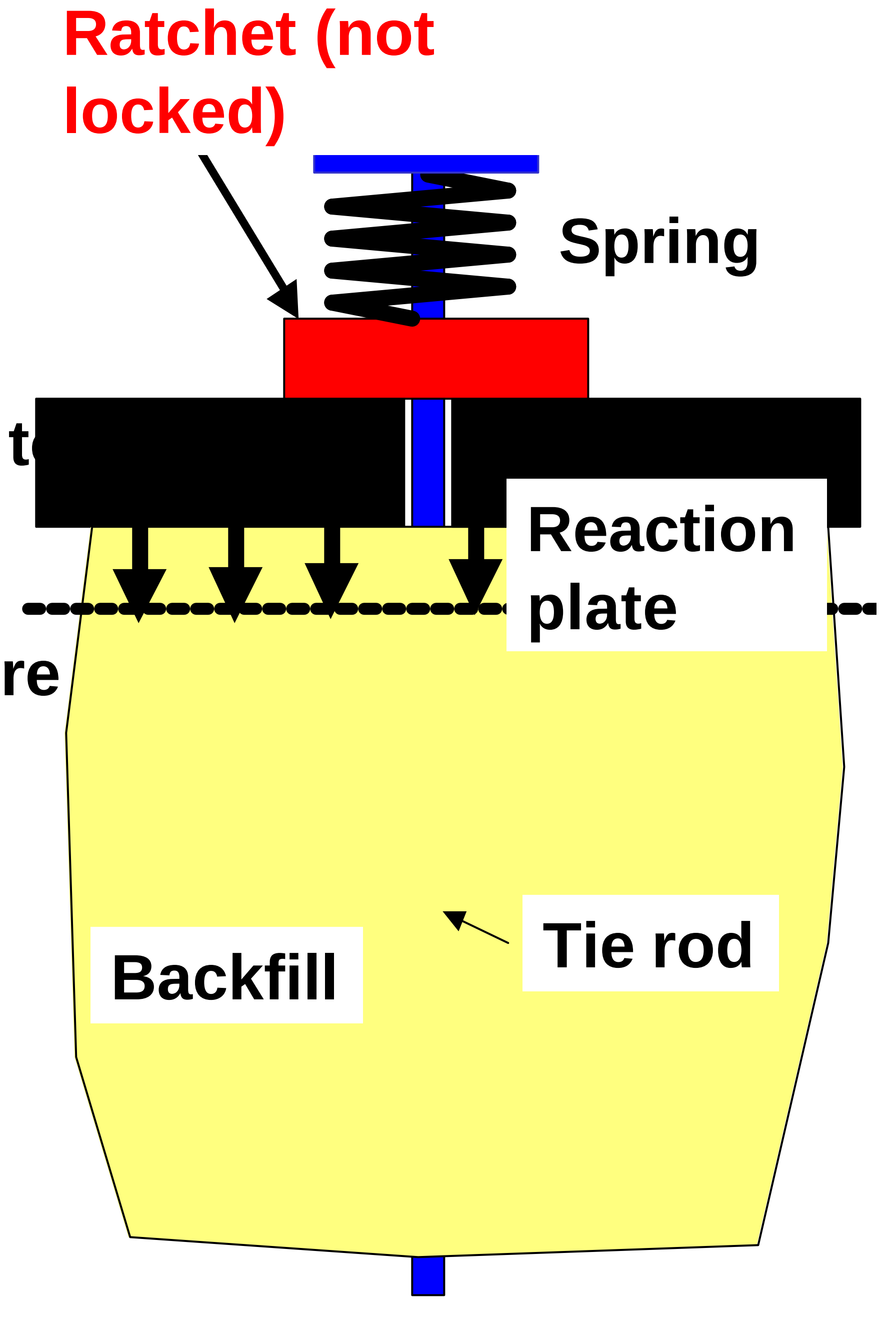
Without a ratchet system



With a ratchet system

A ratchet system:

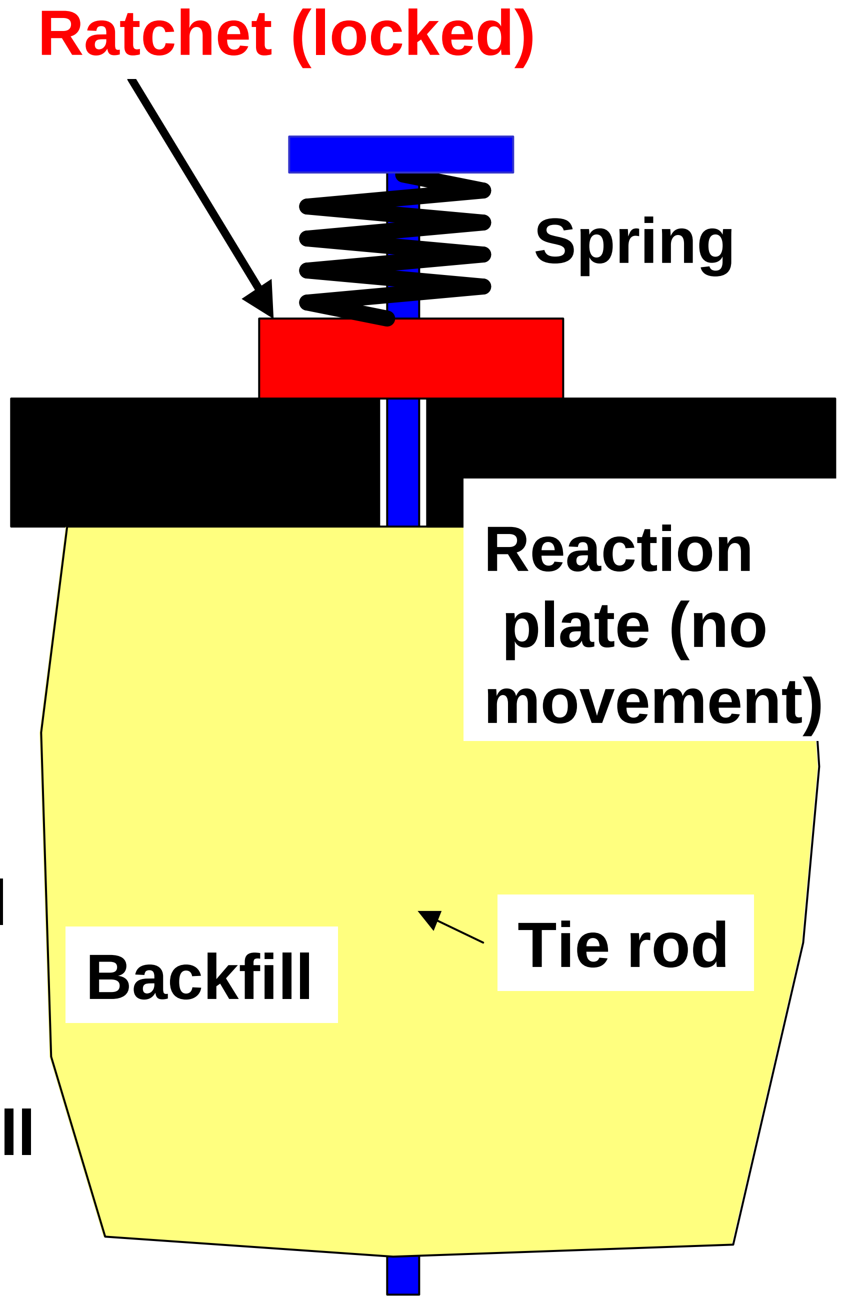
1) keeps the vertical stress constant
when the backfill height tends t
decrease (like preventing the occurrence of liquefaction by dissipating positive excess pore water pressure); and



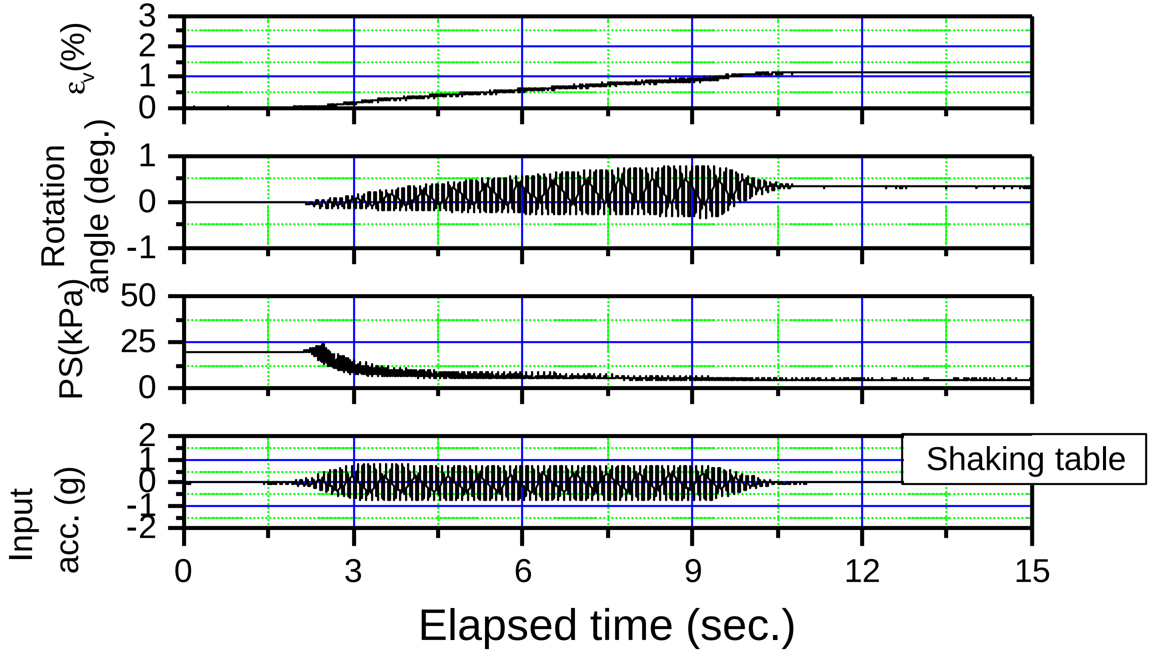
A ratchet system:

1) keeps the vertical stress constant
when the backfill height tends to decrease (like preventing the occurrence of liquefaction by dissipating positive excess pore water pressure); and

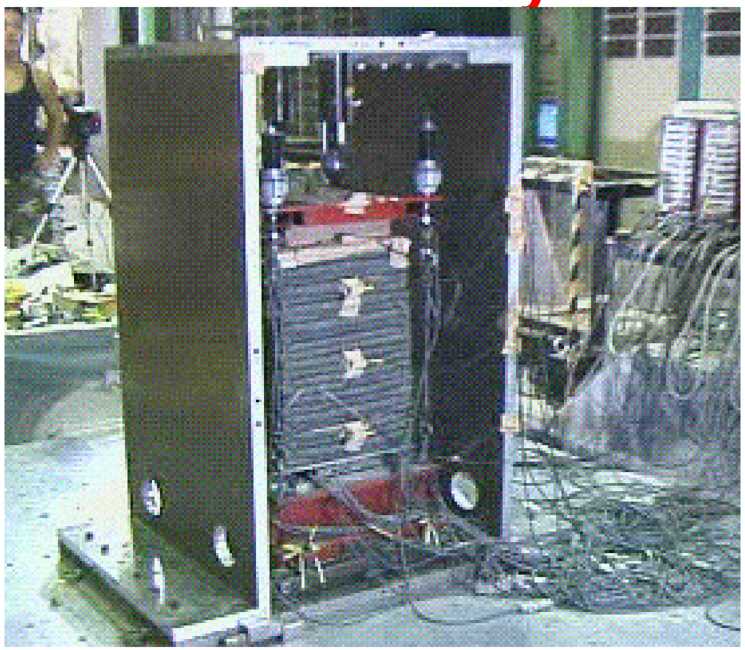
2) keeps the height of the backfill constant by increasing the effective vertical stress when the backfill height tends to increase.

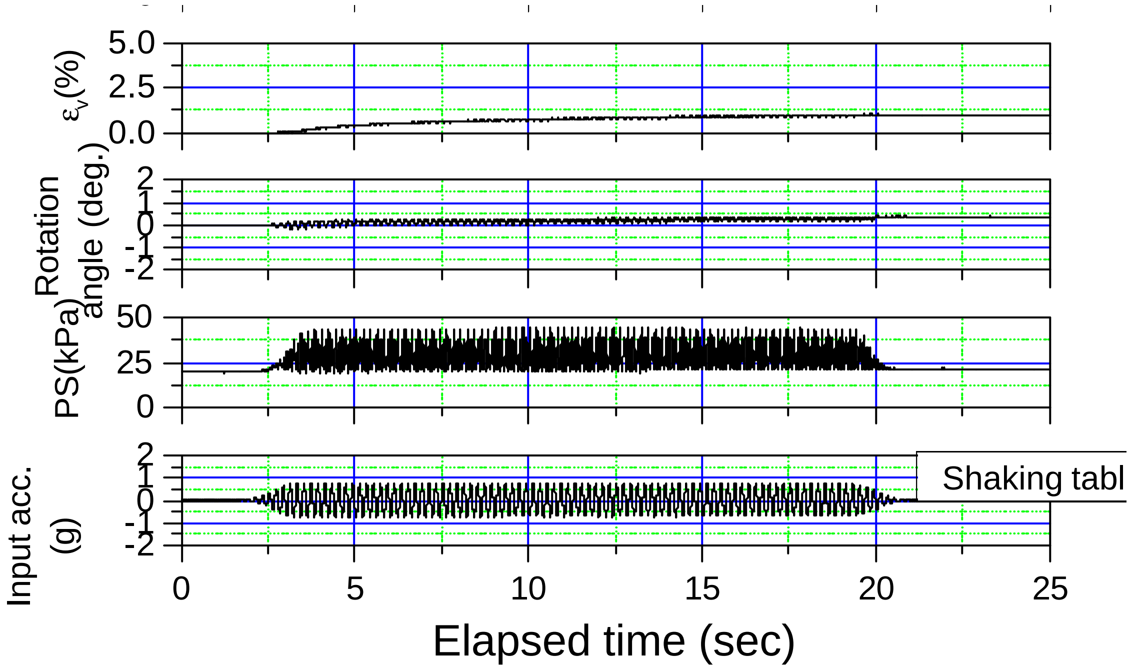


Without

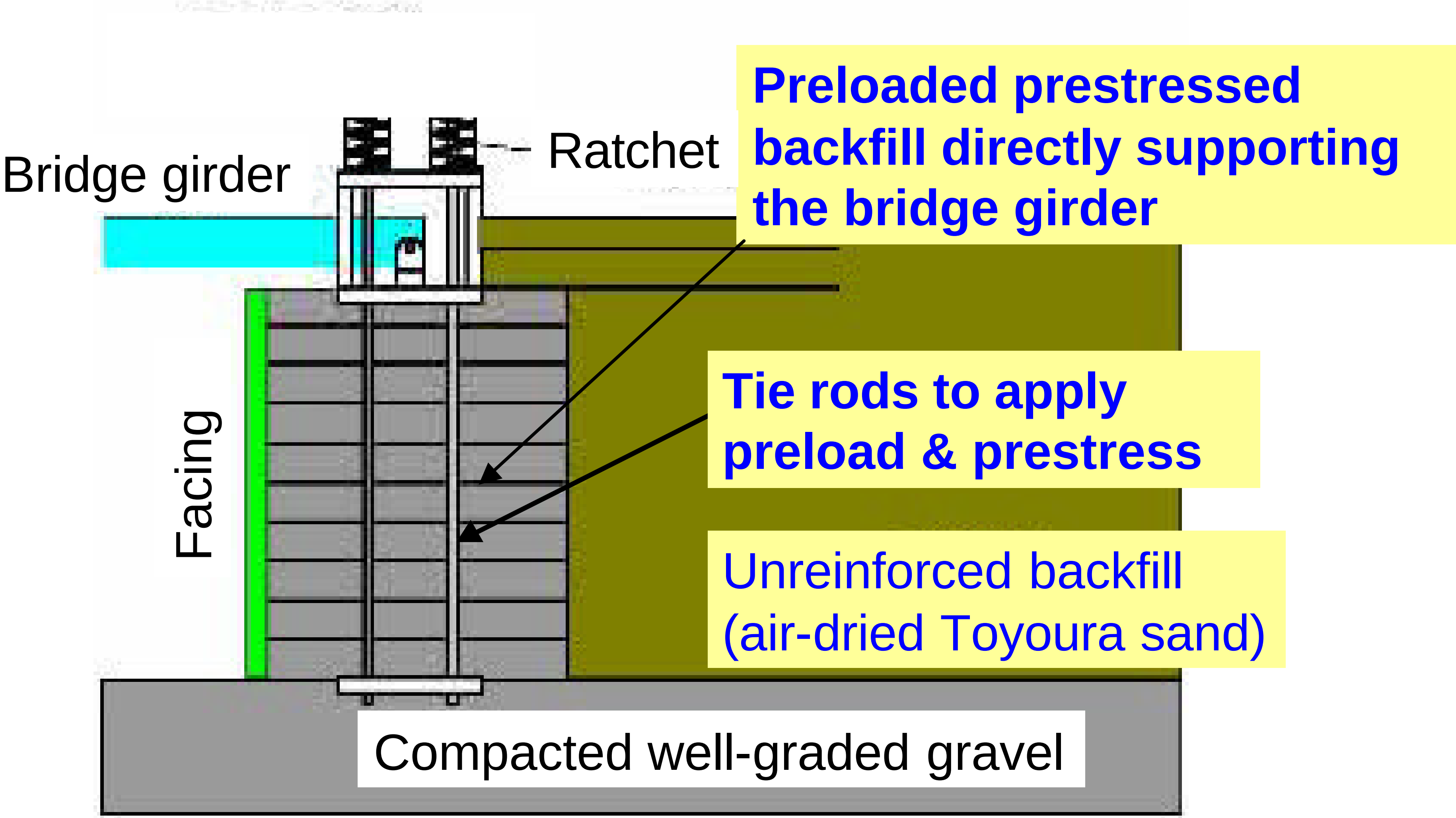


With a ratchet system



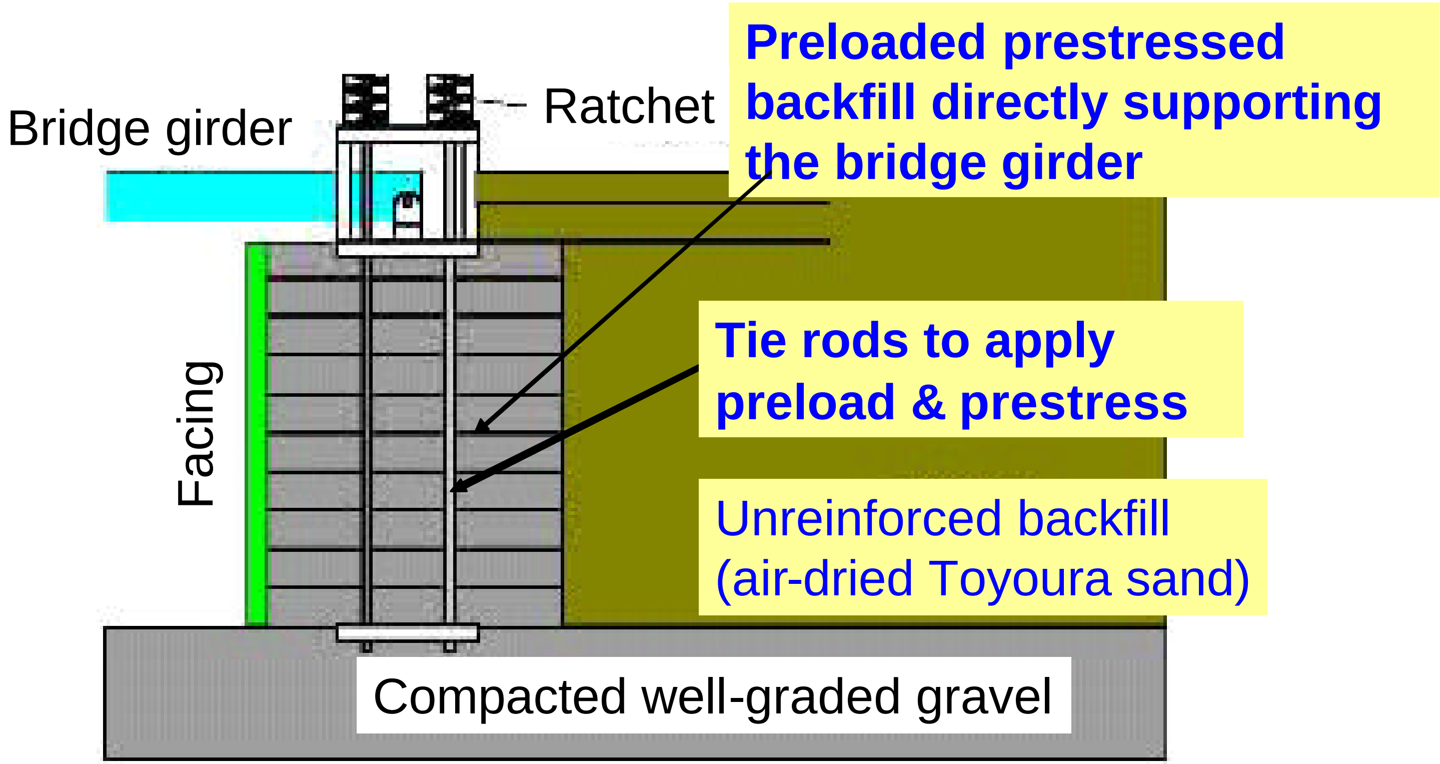


Recommendable new type bridge abutment, No. 1

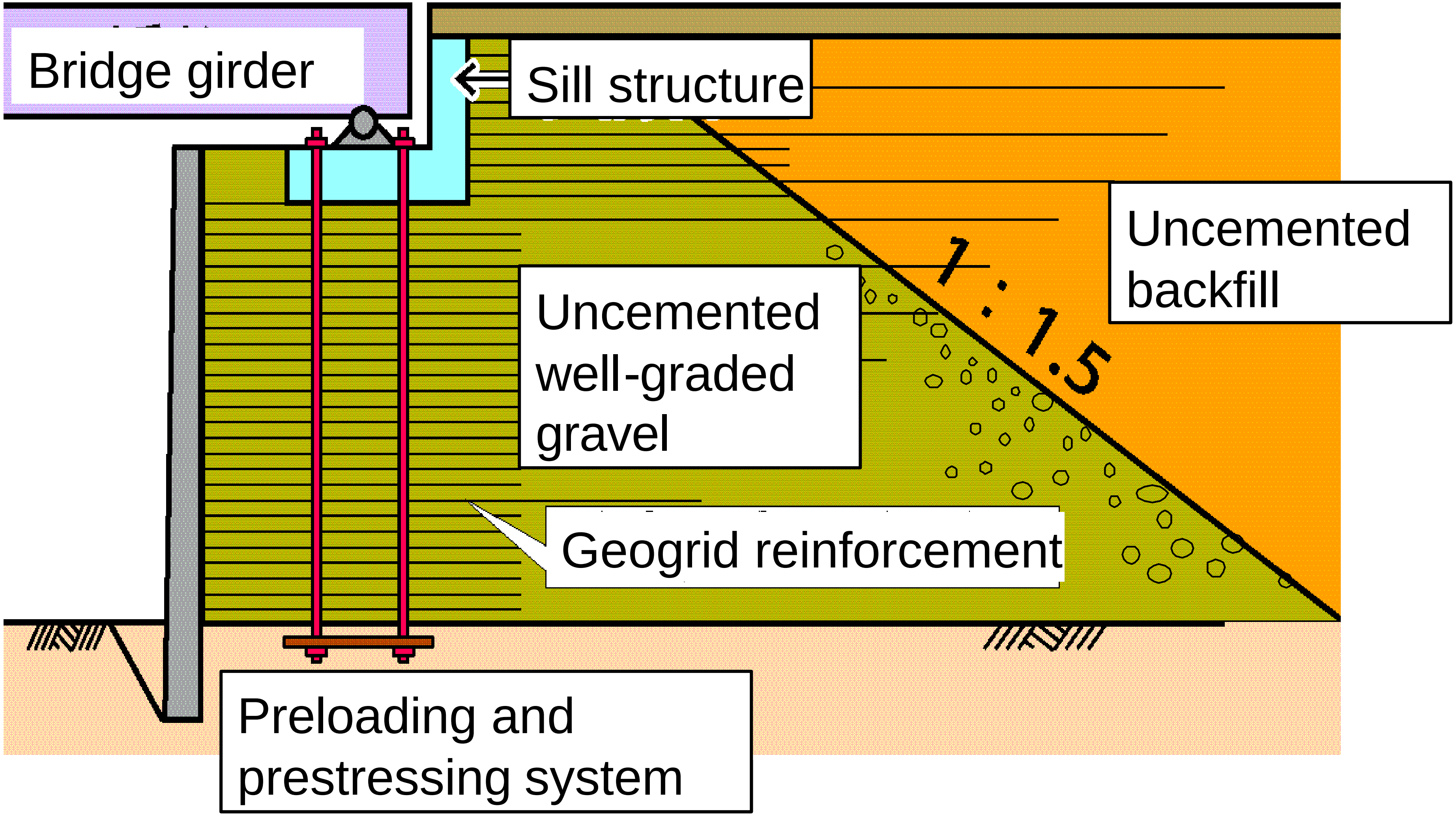


Sinusoidal 1,000 gals

Recommendable new type bridge abutment, No. 1

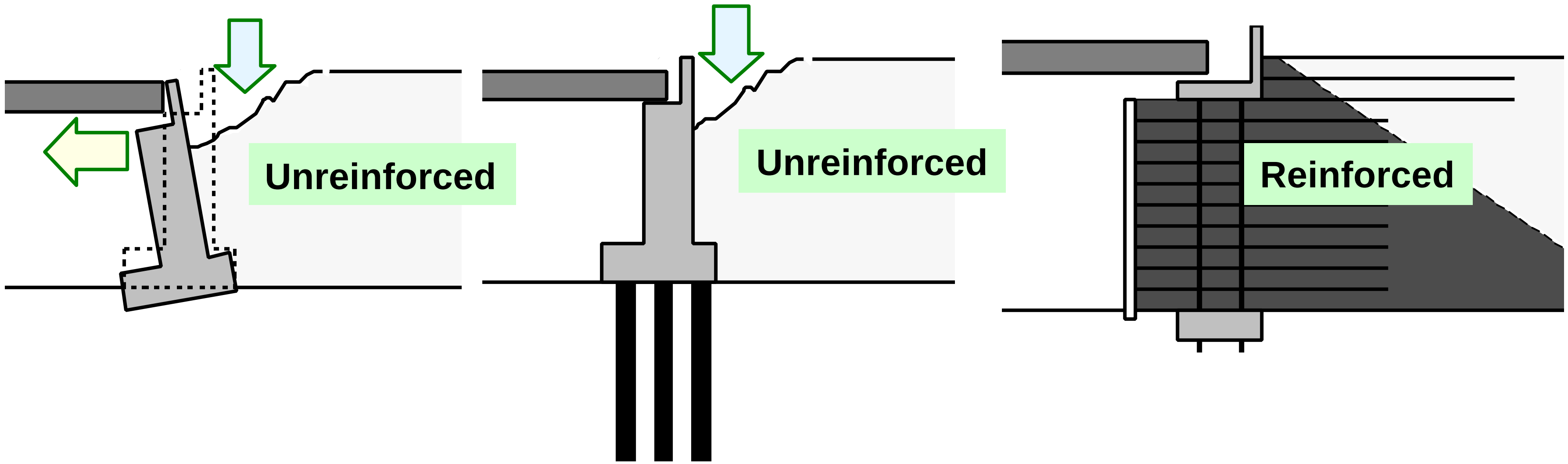


- ***A very high performance !***
- ***A very challenging bridge abutment type !***
- ***Perhaps, too new !***



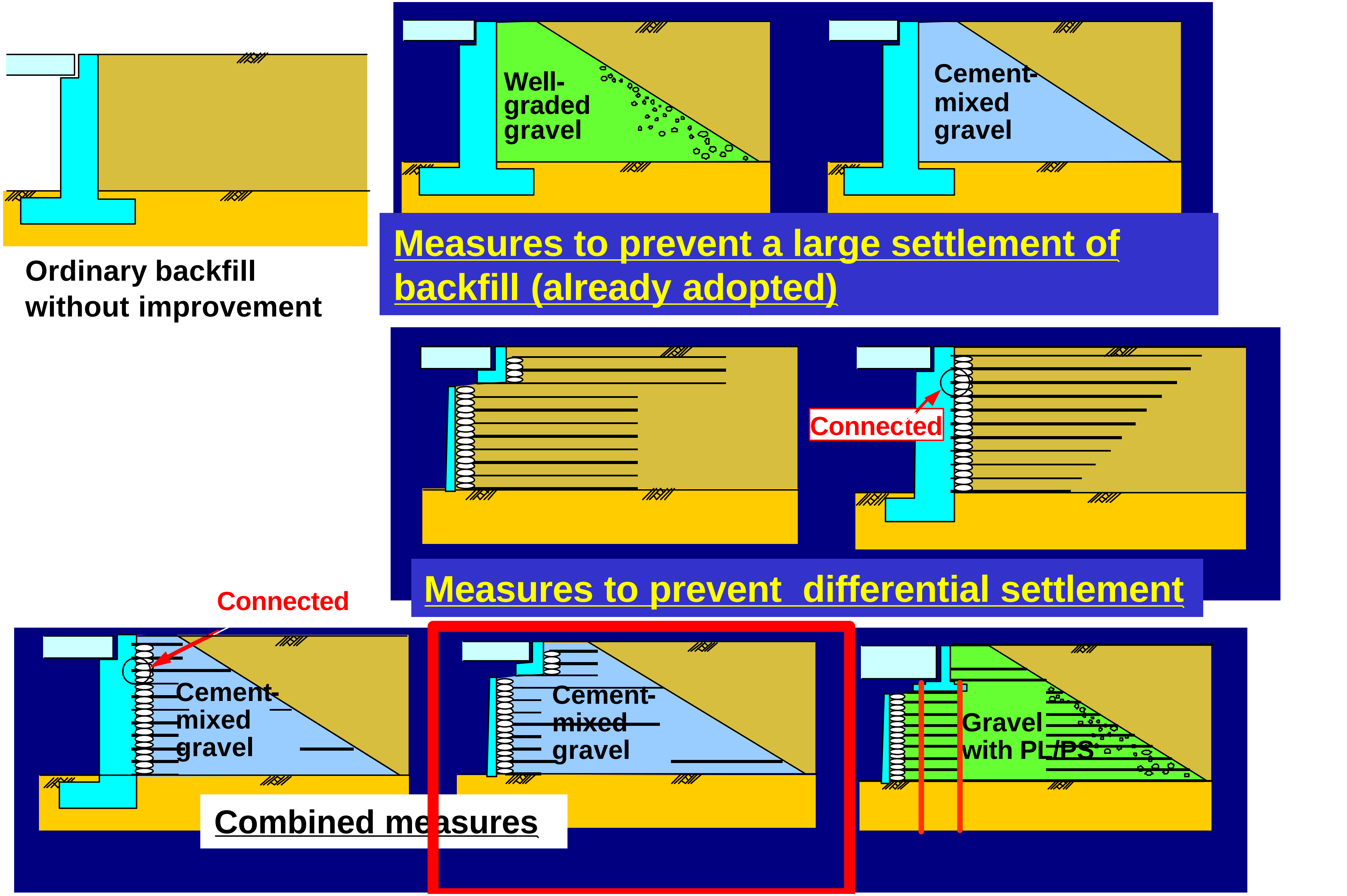
Typical prototype design of "*new type bridge abutment using preloaded and prestressed geogrid-reinforced backfill*"

Needs for highly aseimic and cost-effective bridge abutments

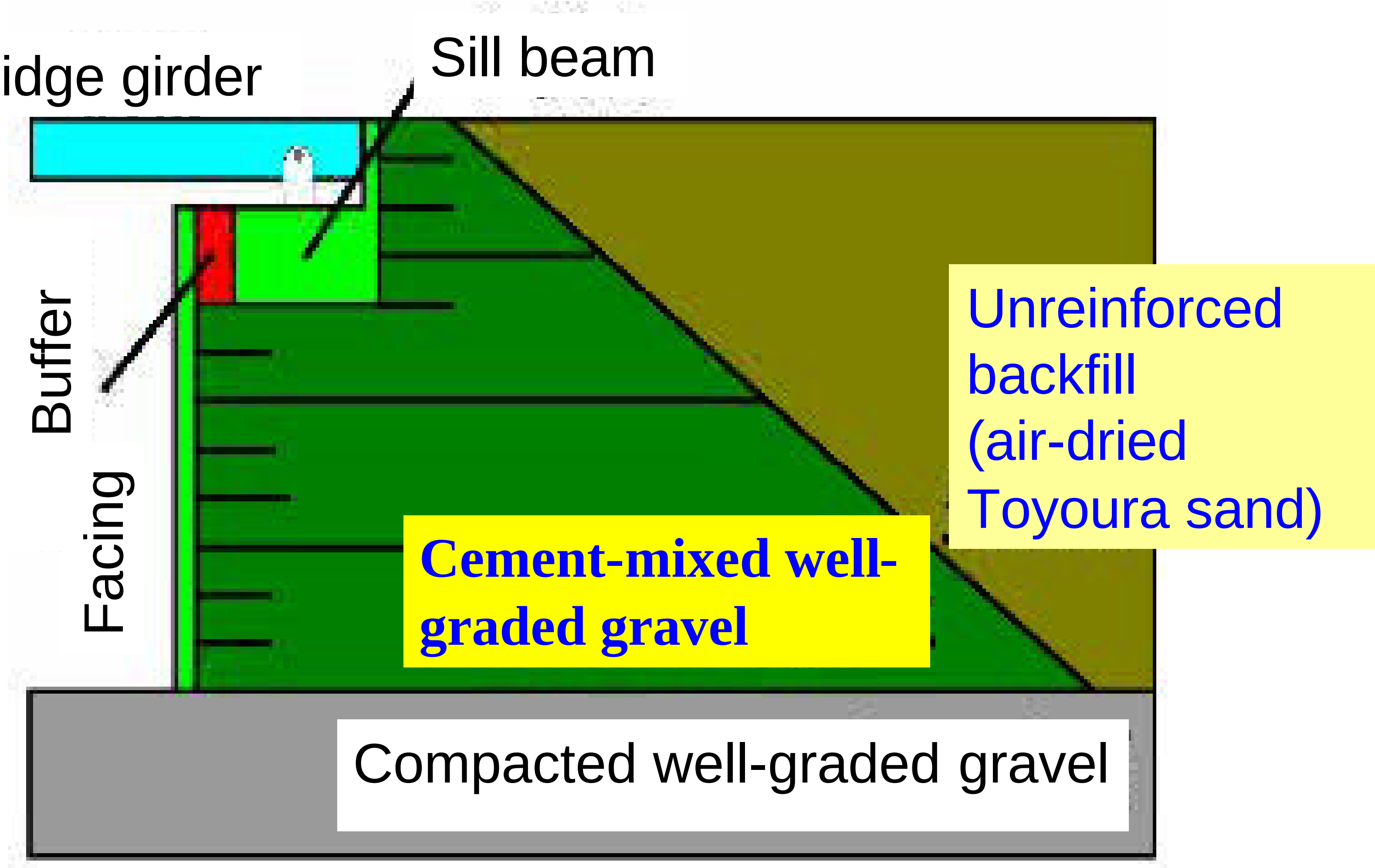


Conventional (no pile)	Conventional (piled)	Technical issues	PLPS GRS abutment
Low	High	Stability of abutment	High
Low	Low	Stability of backfill	High
Large	Not small	Settlement of backfill	Small
Low	High	Construction cost	Low

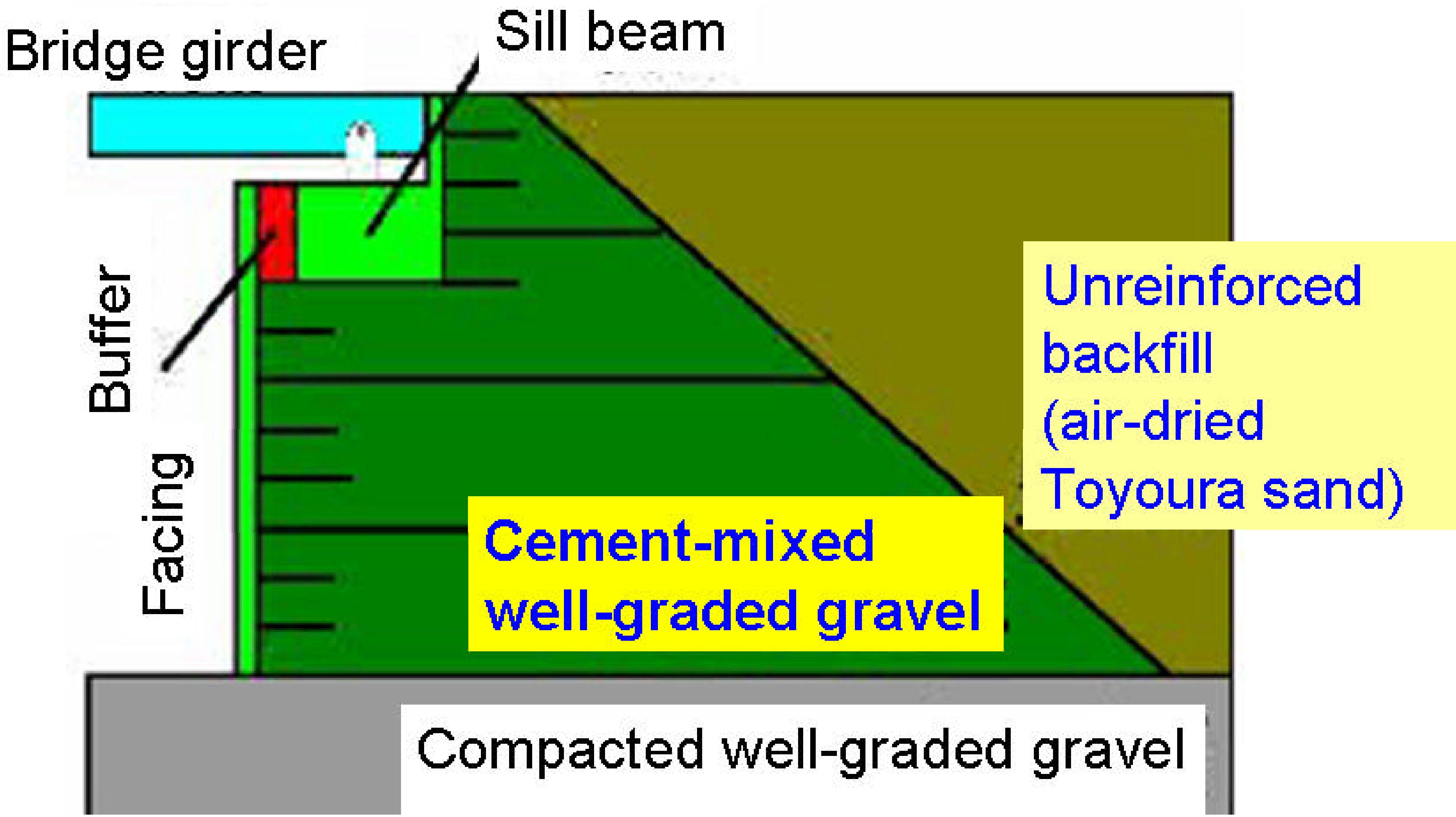
Shaking table tests to evaluate the possible solutions



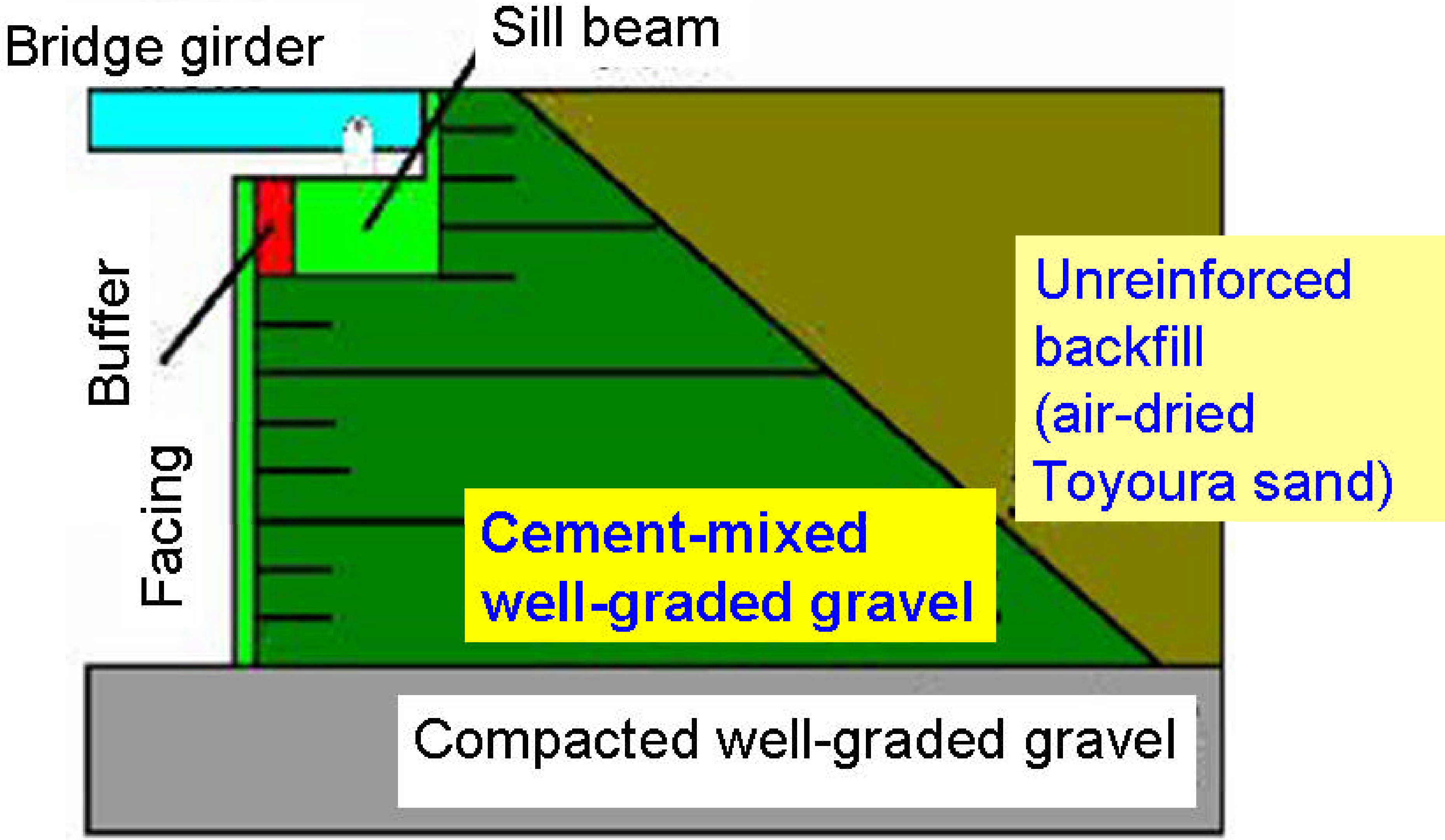
Recommendable new type bridge abutment, No. 2



Kobe earthquake load, 1,000 gals



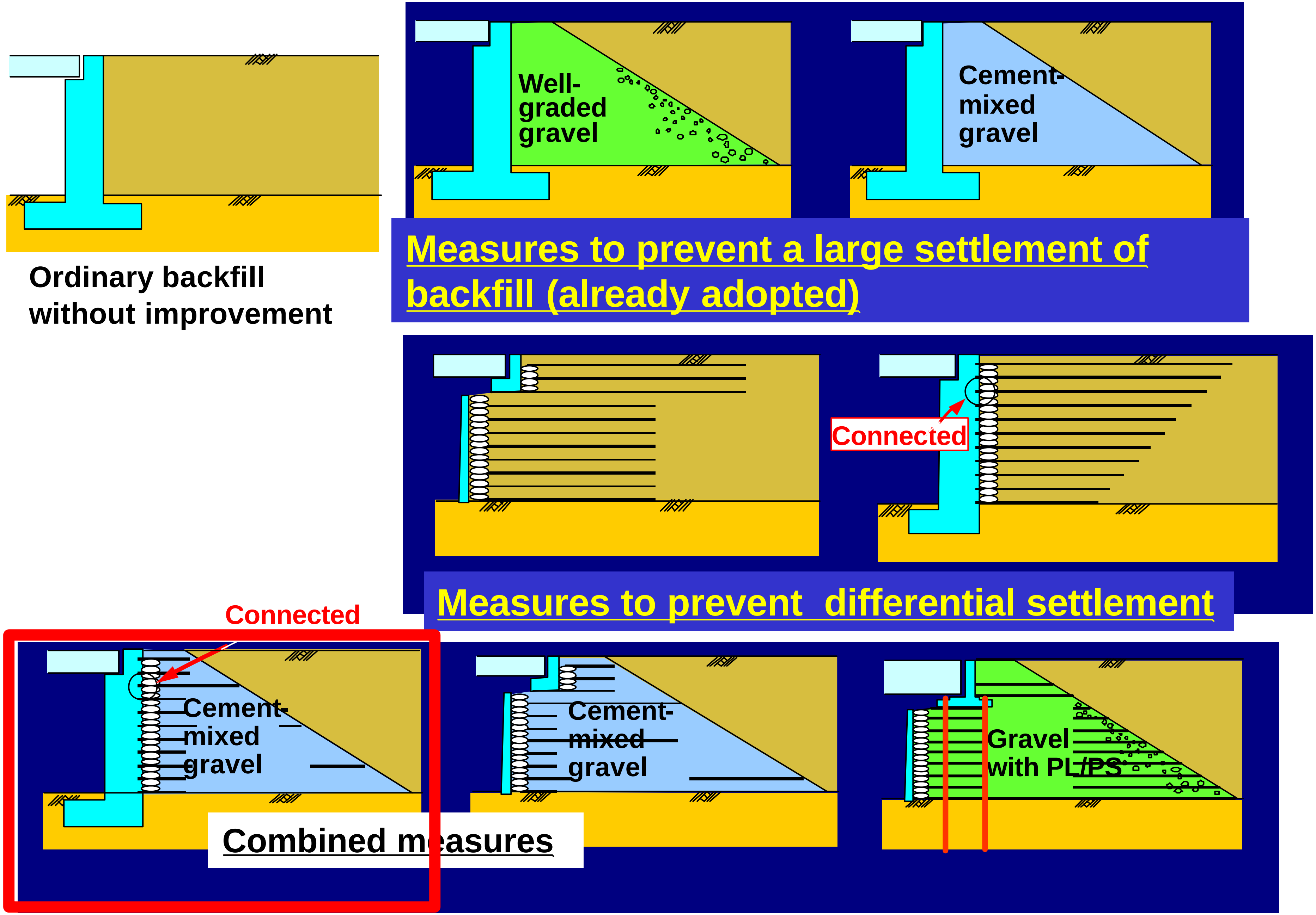
- *A very high performance !*
- *The backfill is more stable than the facing !*
- *The active earth pressure decreases during dynamic loading !*



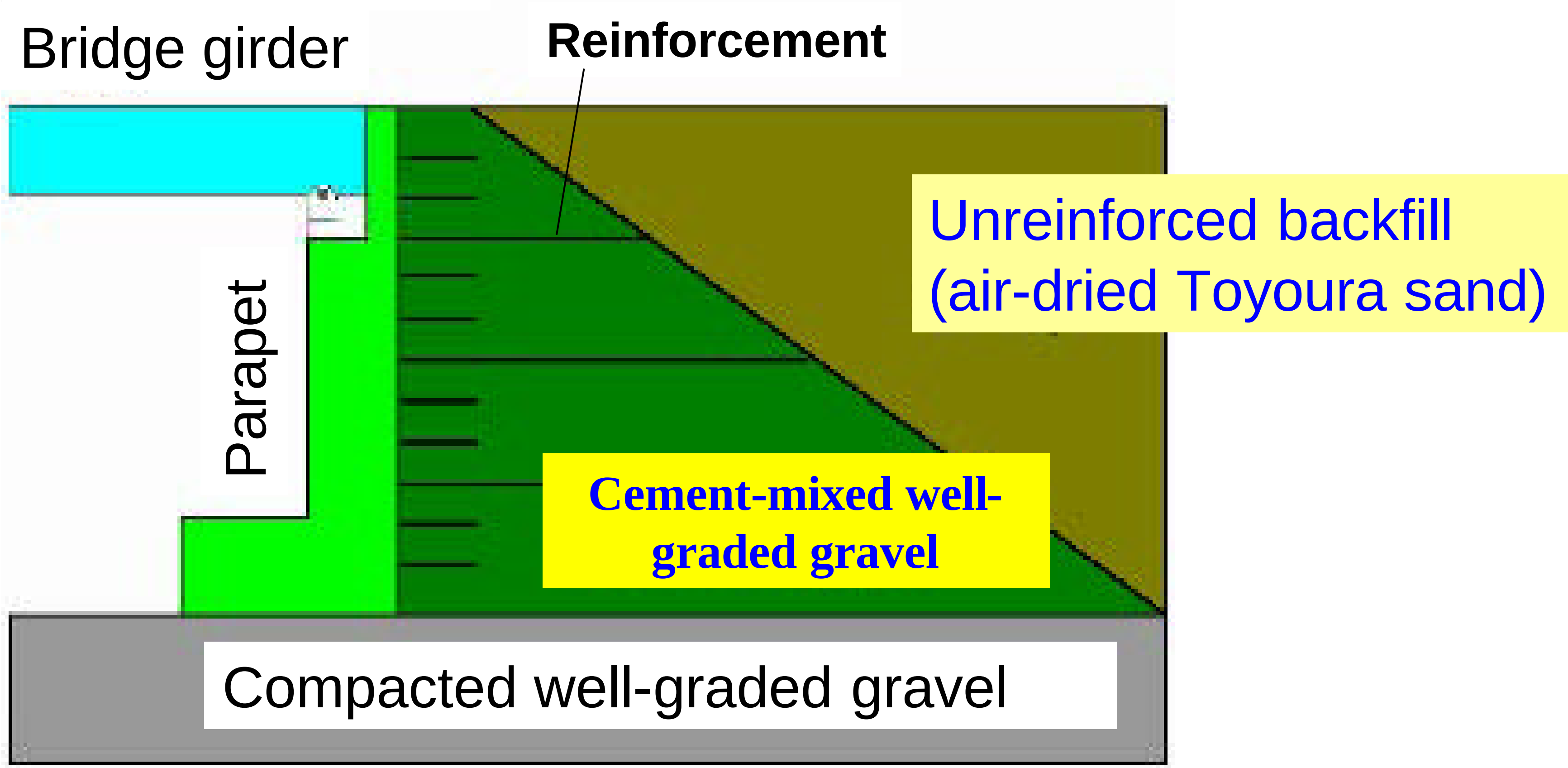
- A very challenging bridge abutment type !

- Perhaps, too new!

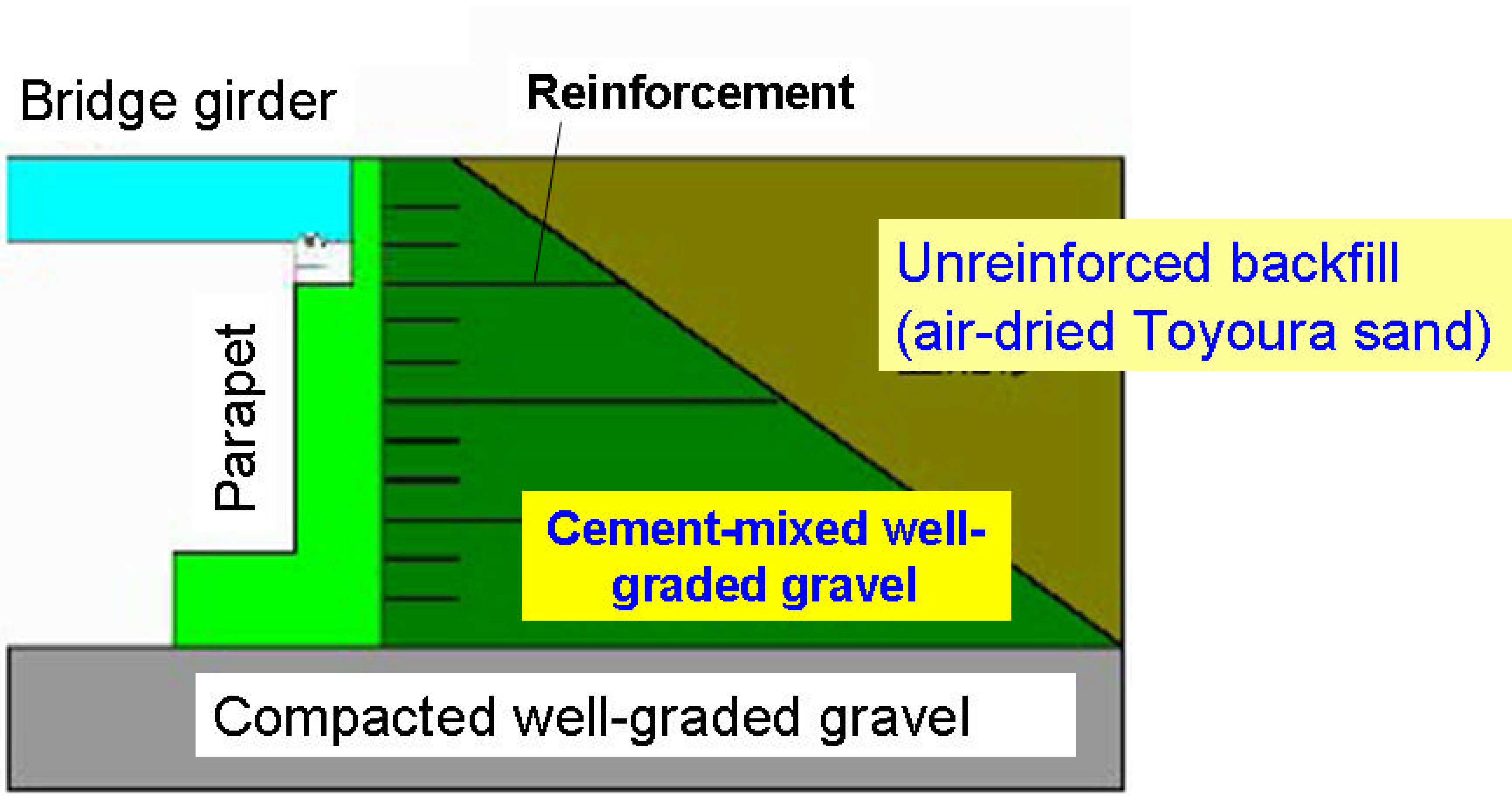
Shaking table tests to evaluate the possible solutions



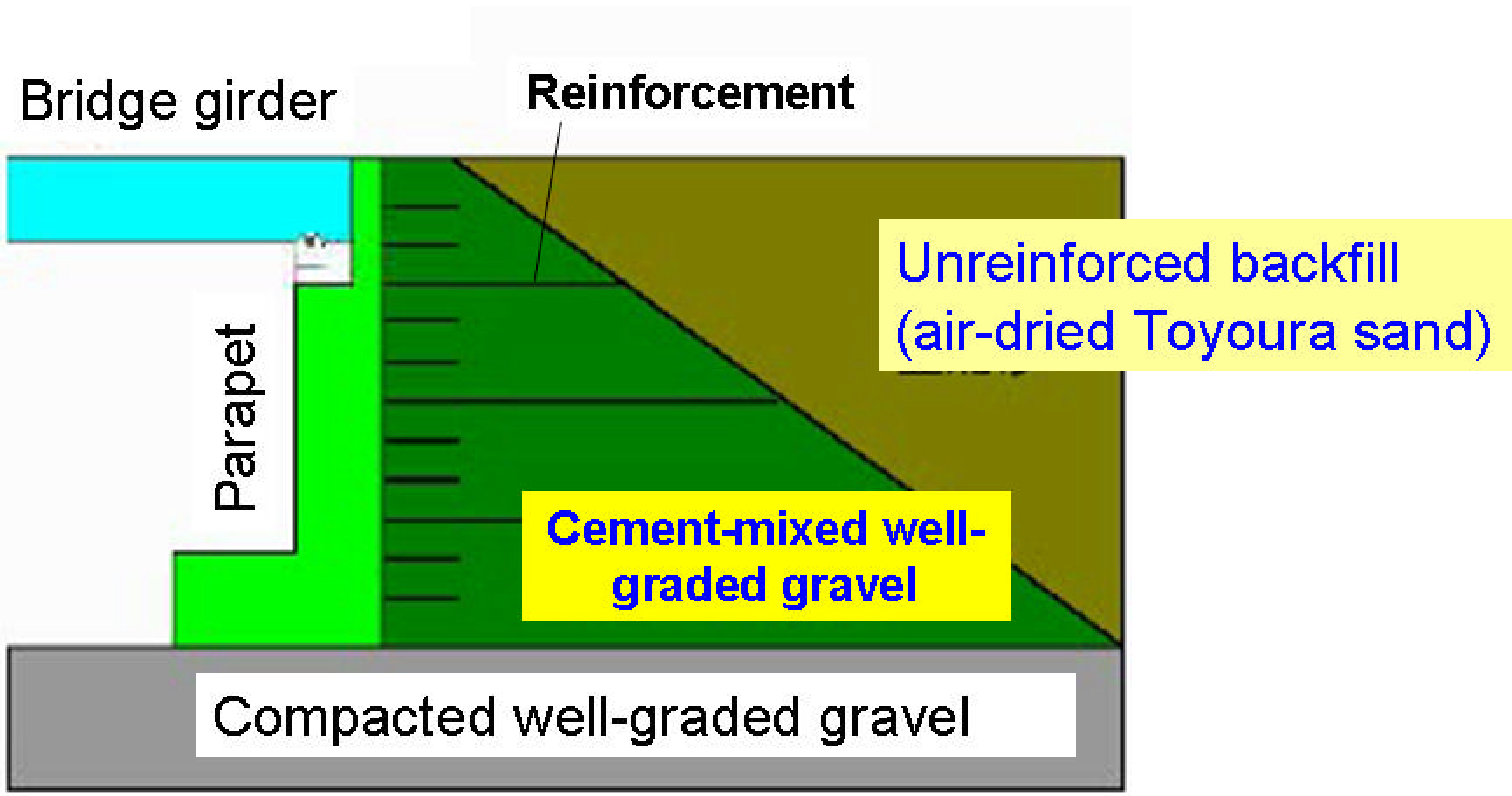
Recommendable new type bridge abutment, No. 3



Sinusoidal 1,000 gals



- ***A very high performance !***
- ***The backfill is more stable than the facing !***
- ***The active earth pressure decreases during dynamic loading !***

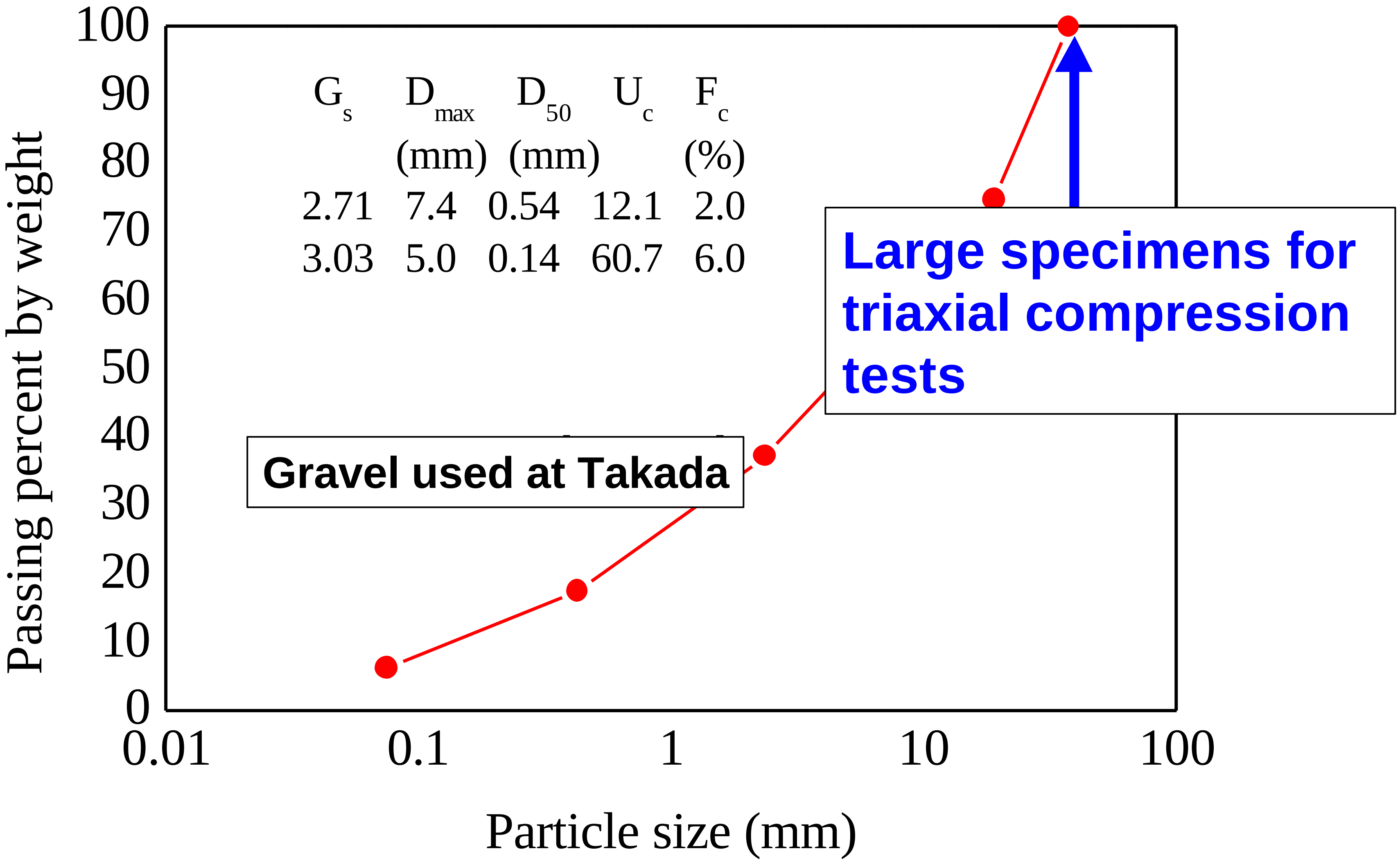


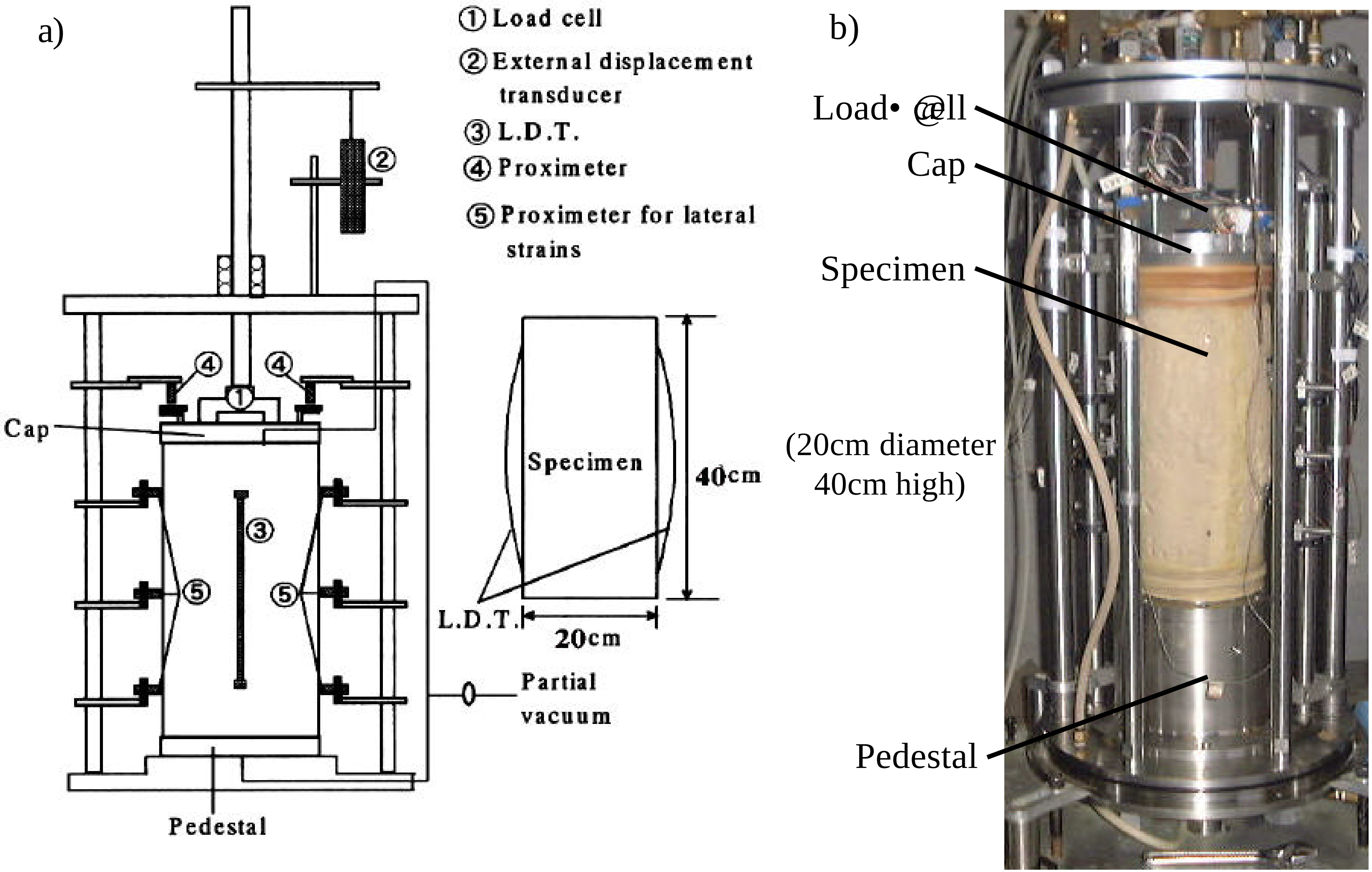
Presently, the most recommendable new type bridge abutment;

- has been accepted by railway engineers in Japan.

Contents:

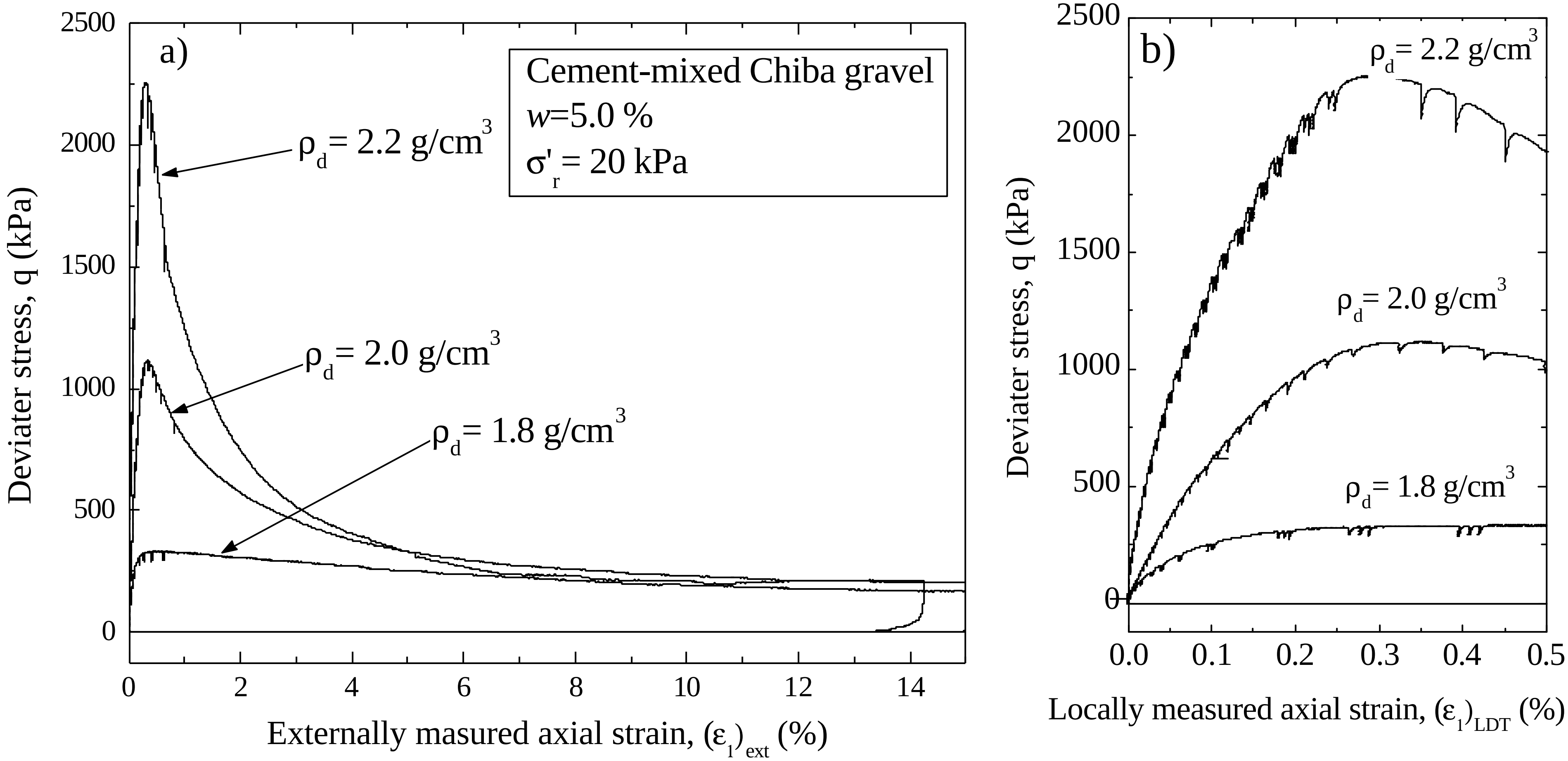
Use of well-graded gravel
to achieve a high compacted dry density



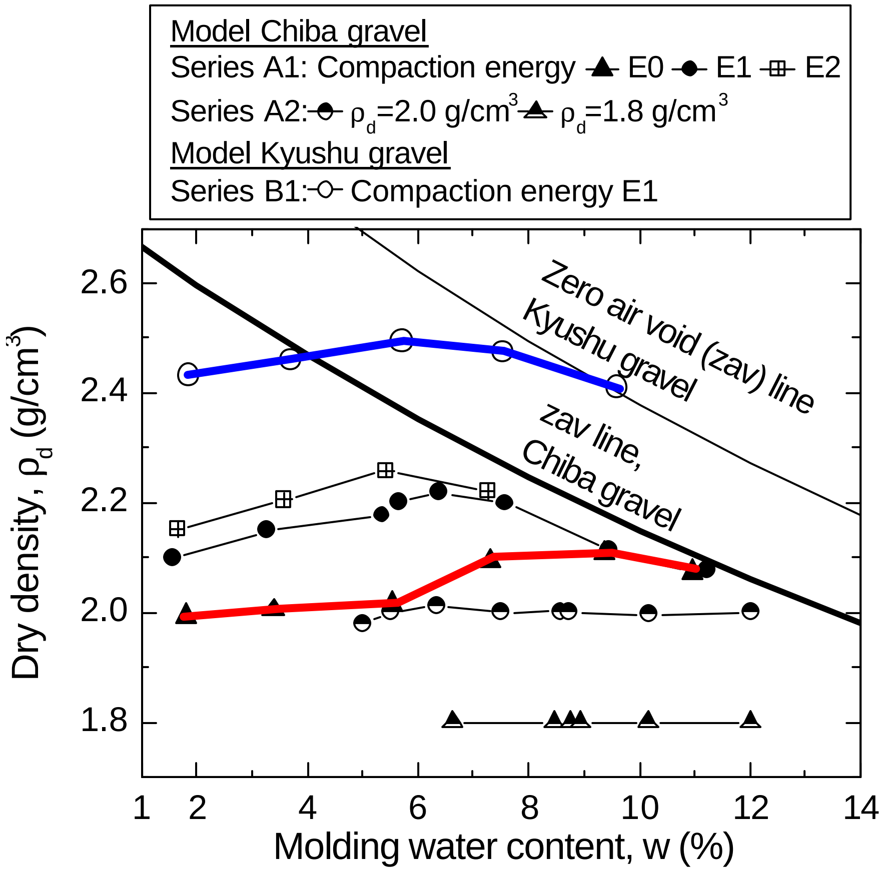


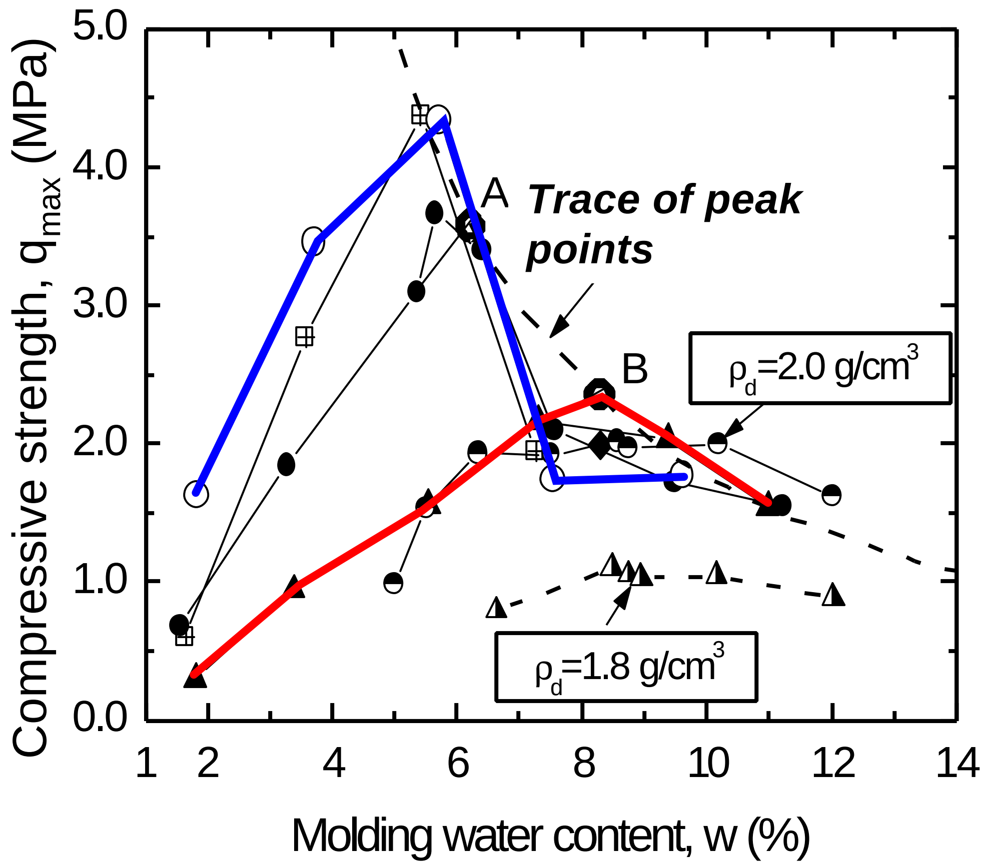
Large triaxial apparatus measuring locally the axial strains by using LDT (local deformation transducer)

Significant effects of compacted dry density (cement/gravel ratio by weight= 2.5 %)



A sharp peak of the compressive strength, q_{max}, at the optimum water content !





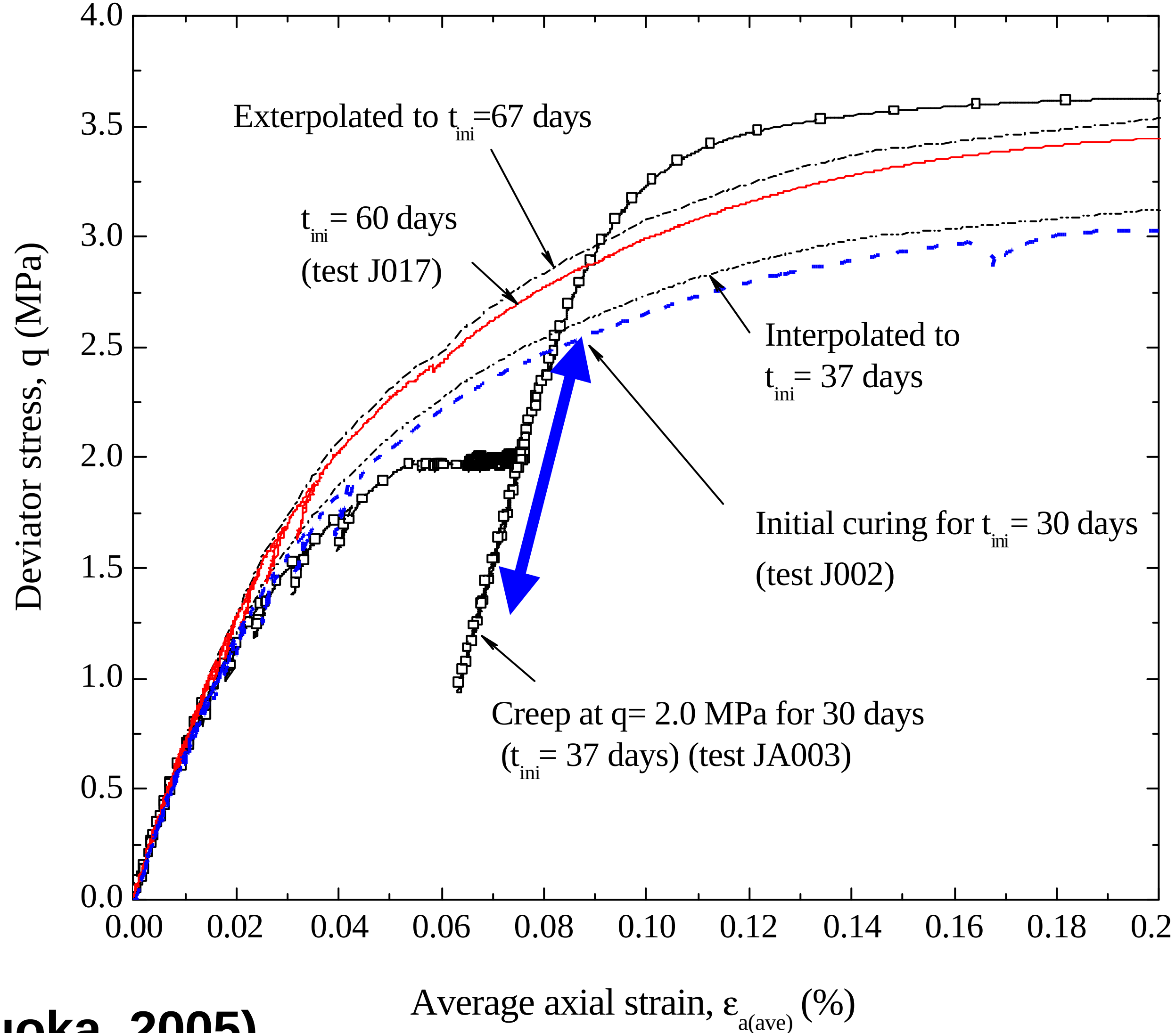
Laboratory stress-strain tests showed:

For a high performance and a high cost-effectiveness in the construction of cement-mixed gravel backfill;

1. **find the optimum water content for a given backfill type and a specified compaction energy; and**
2. **mix and compact cement-mixed gravel at the optimum water content.**

Another important fact:

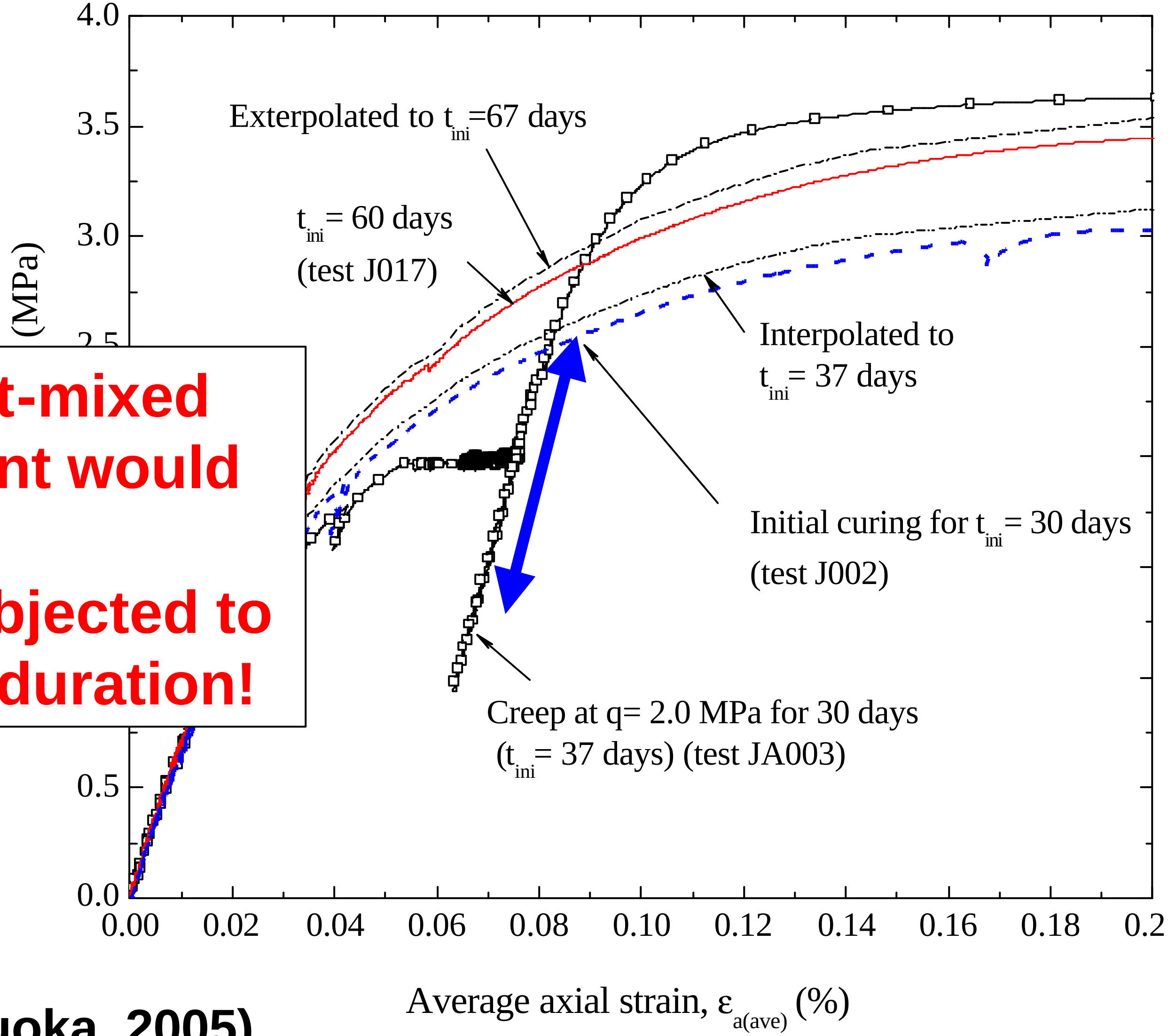
Cement-mixed soil exhibits nearly elastic behaviour after long-term ageing with shear stresses.



(Kongsukprasert & Tatsuoka, 2005)

Another important fact:

Cement-mixed soil exhibits nearly elastic behaviour after long-term ageing with shear stresses.



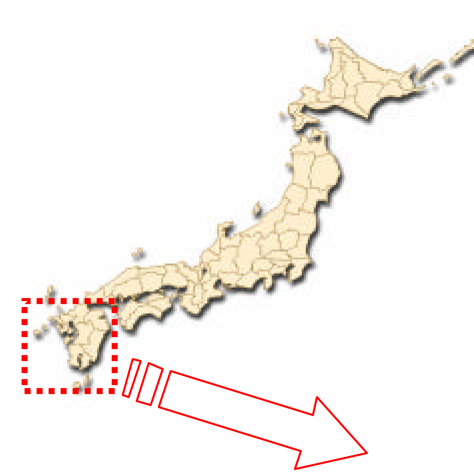
The backfill of cement-mixed gravel for the abutment would exhibit little residual deformation when subjected to traffic load for a long duration!

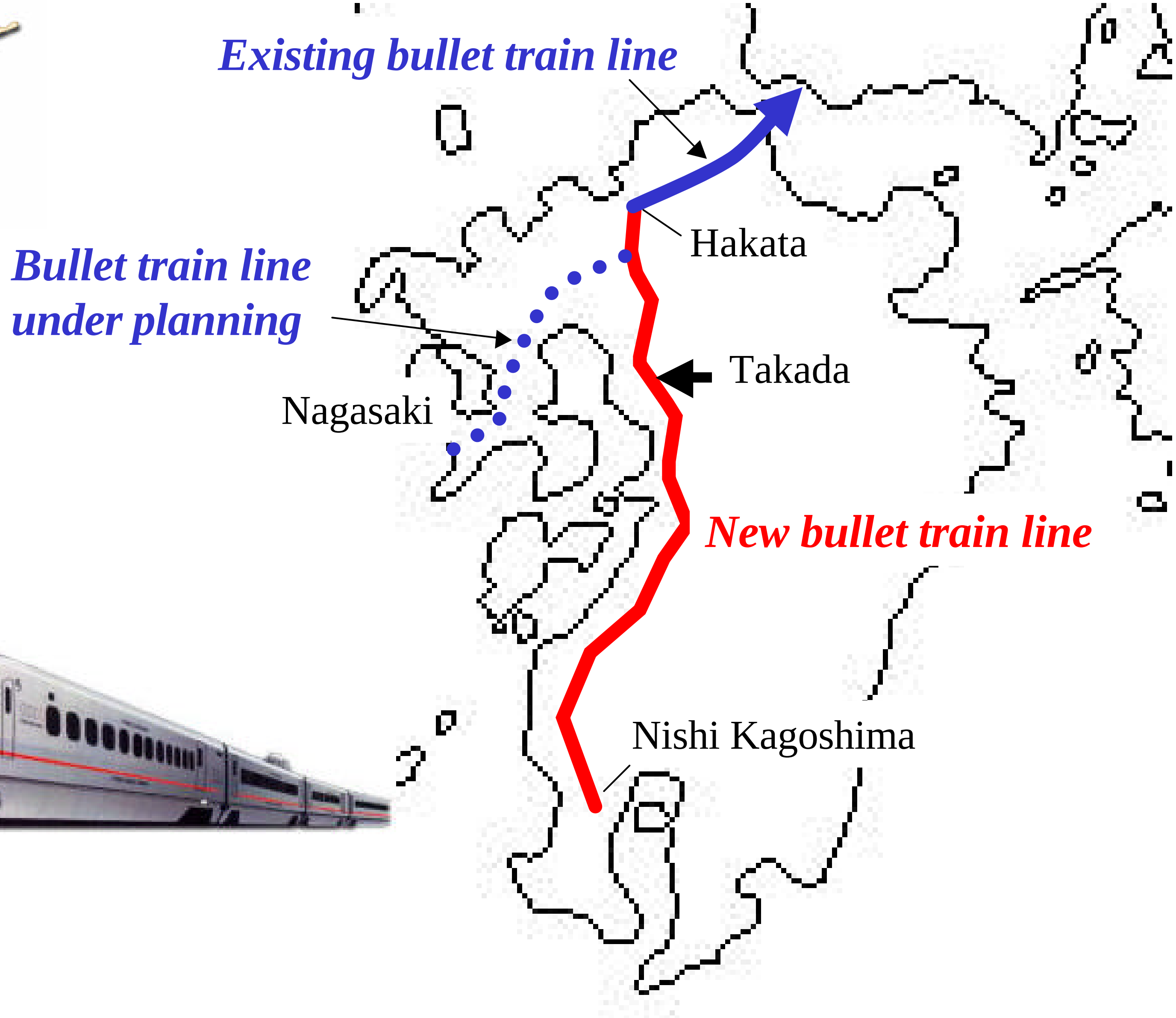
(Kongsukprasert & Tatsuoka, 2005)

Contents:

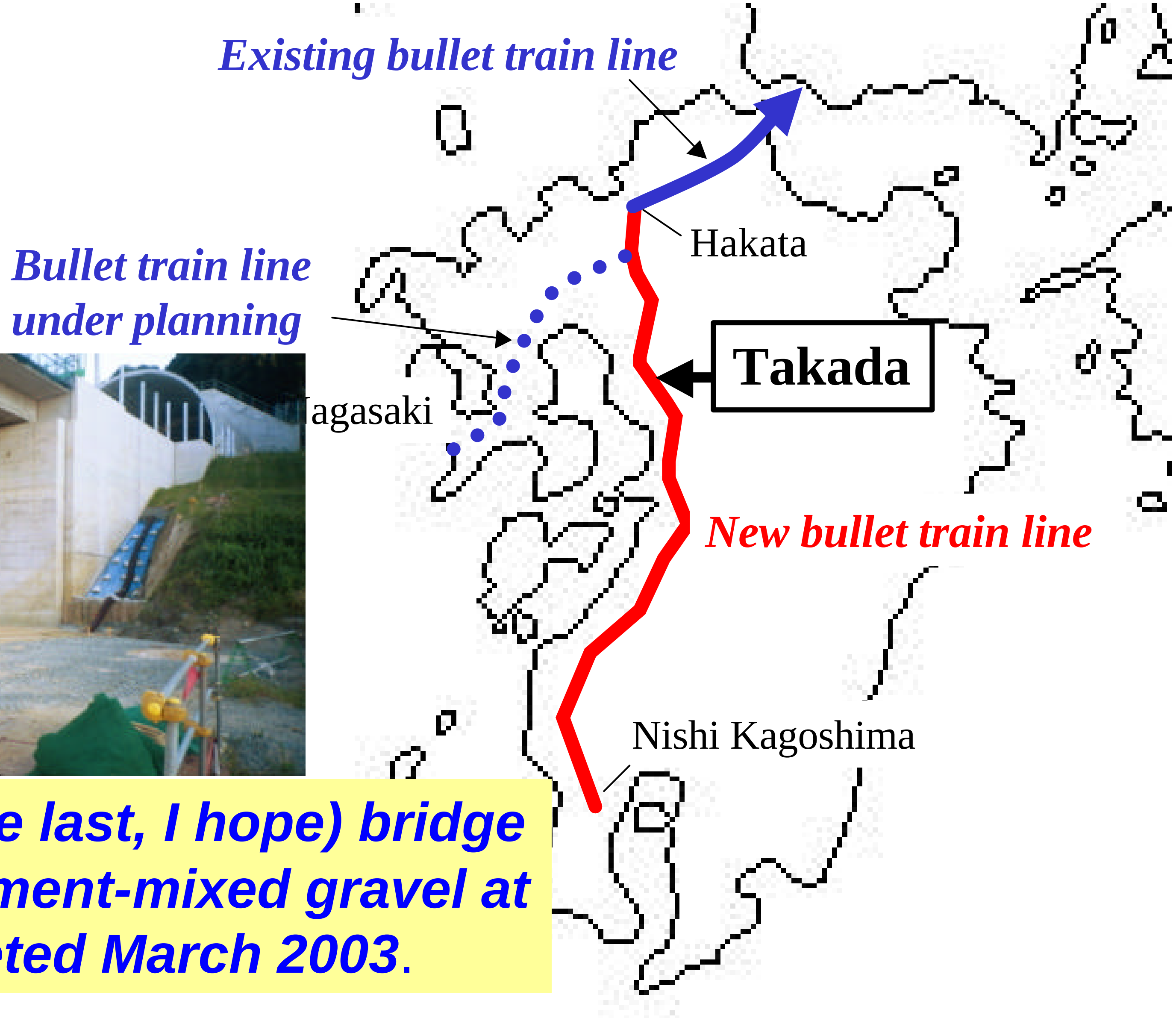
1. Background and research framework
2. Model shaking table tests
3. Stress-strain behaviour of cement-mixed gravely soil
4. **Design and construction of the new type bridge abutment**
5. Full-scale loading tests of the new type bridge abutment
6. Conclusions

A New Bullet Train Line in Kyushu Island





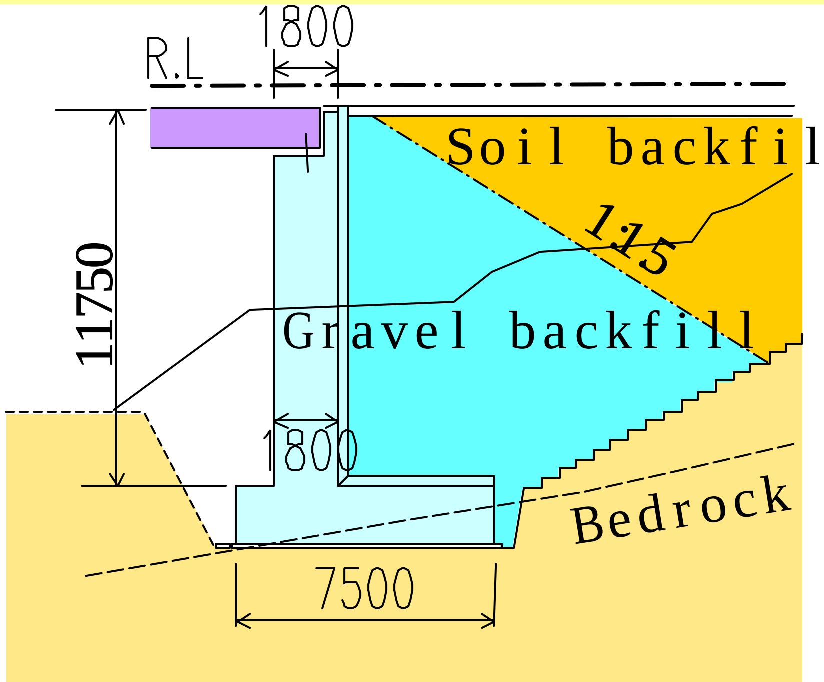
A New Bullet Train Line in Kyushu Island



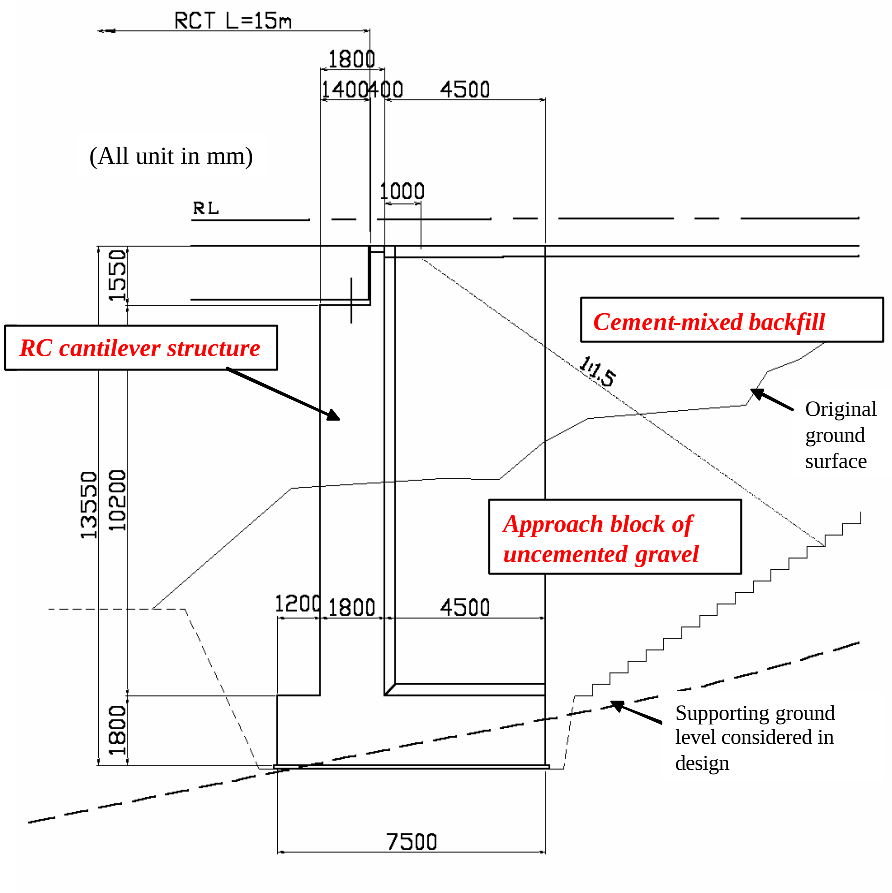
The first (not the last, I hope) bridge abutment of cement-mixed gravel at Takada, completed March 2003.

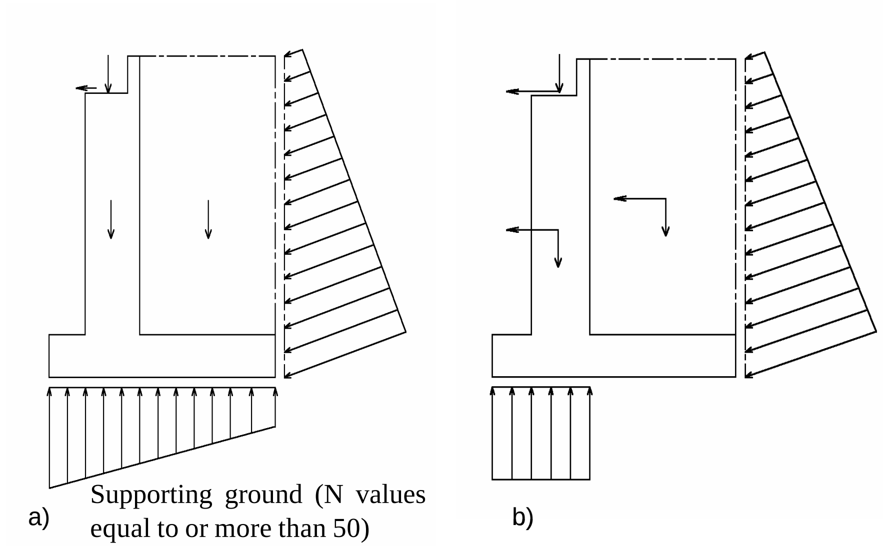
Conventional type versus **new type**

The conventional RC wall structure, supporting the backfill with the earth pressure from the backfill.



(only design)



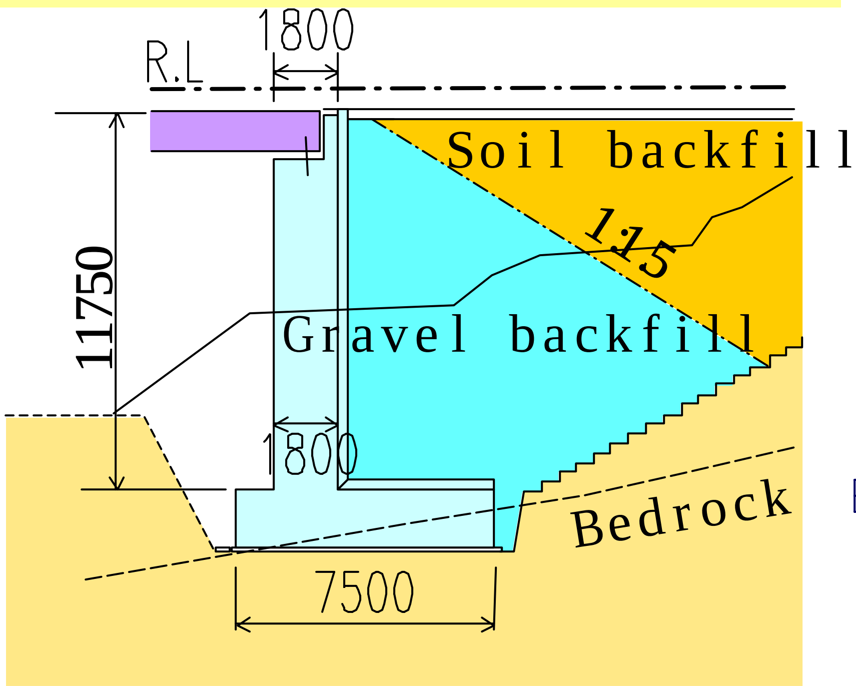


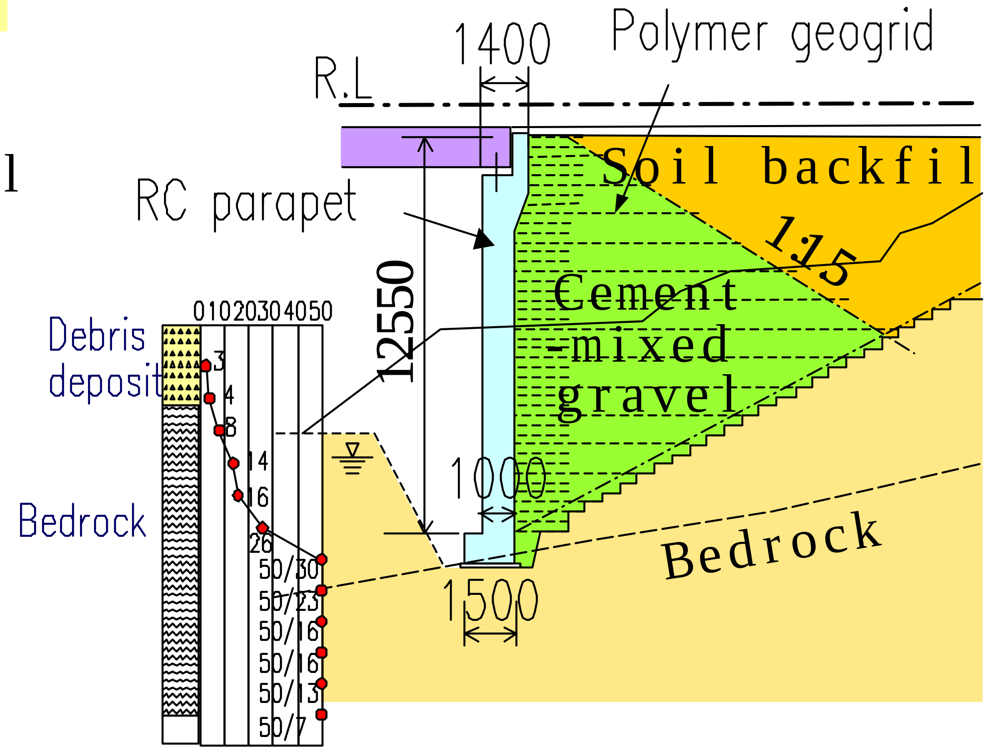
The conventional RC wall structure supports the backfill with the earth pressure from the backfill.

Conventional type versus new type

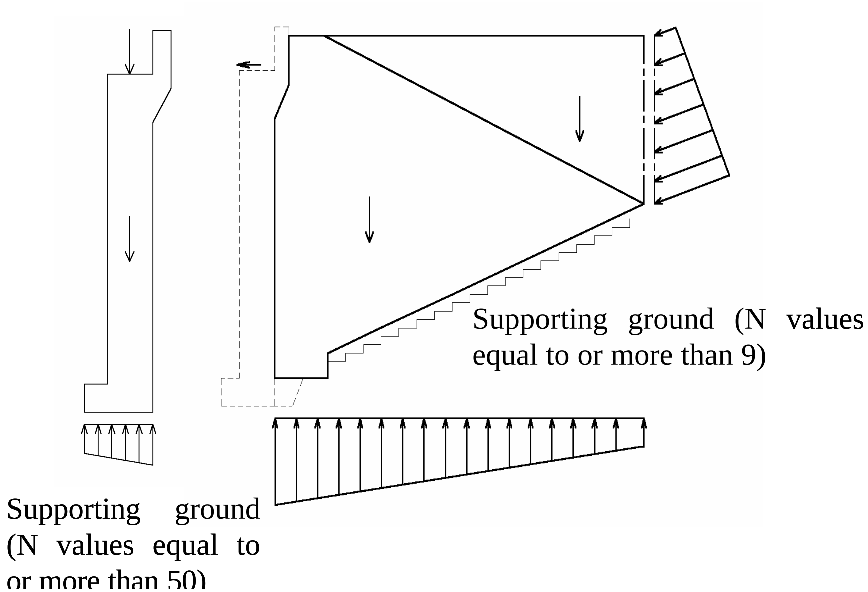
The conventional RC wall structure, supporting the backfill with the earth pressure from the backfill.

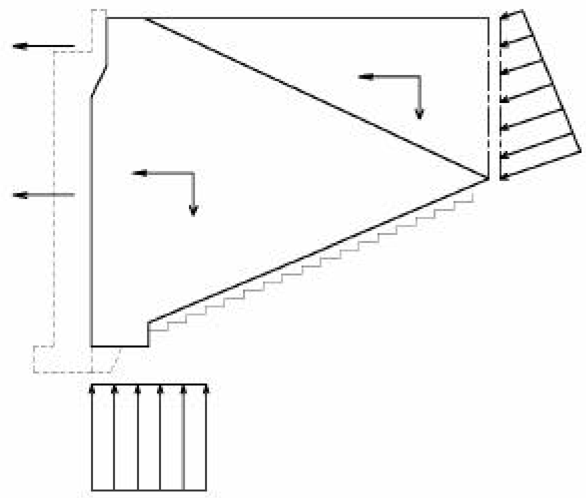
The backfill supporting the RC parapet without the earth pressure on the parapet.

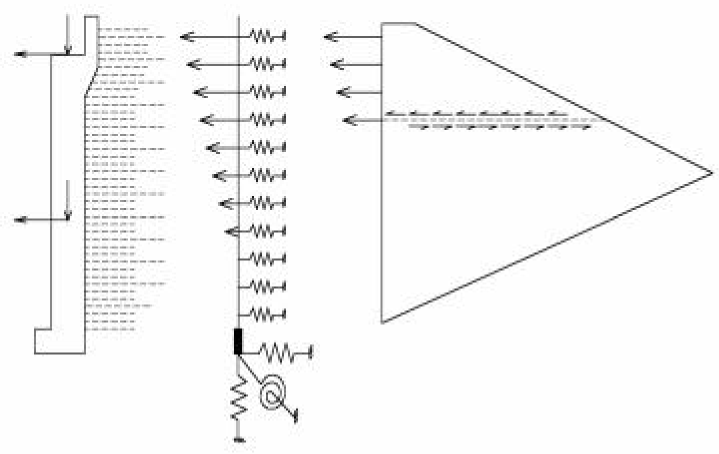




(only design) **(actually constructed)**



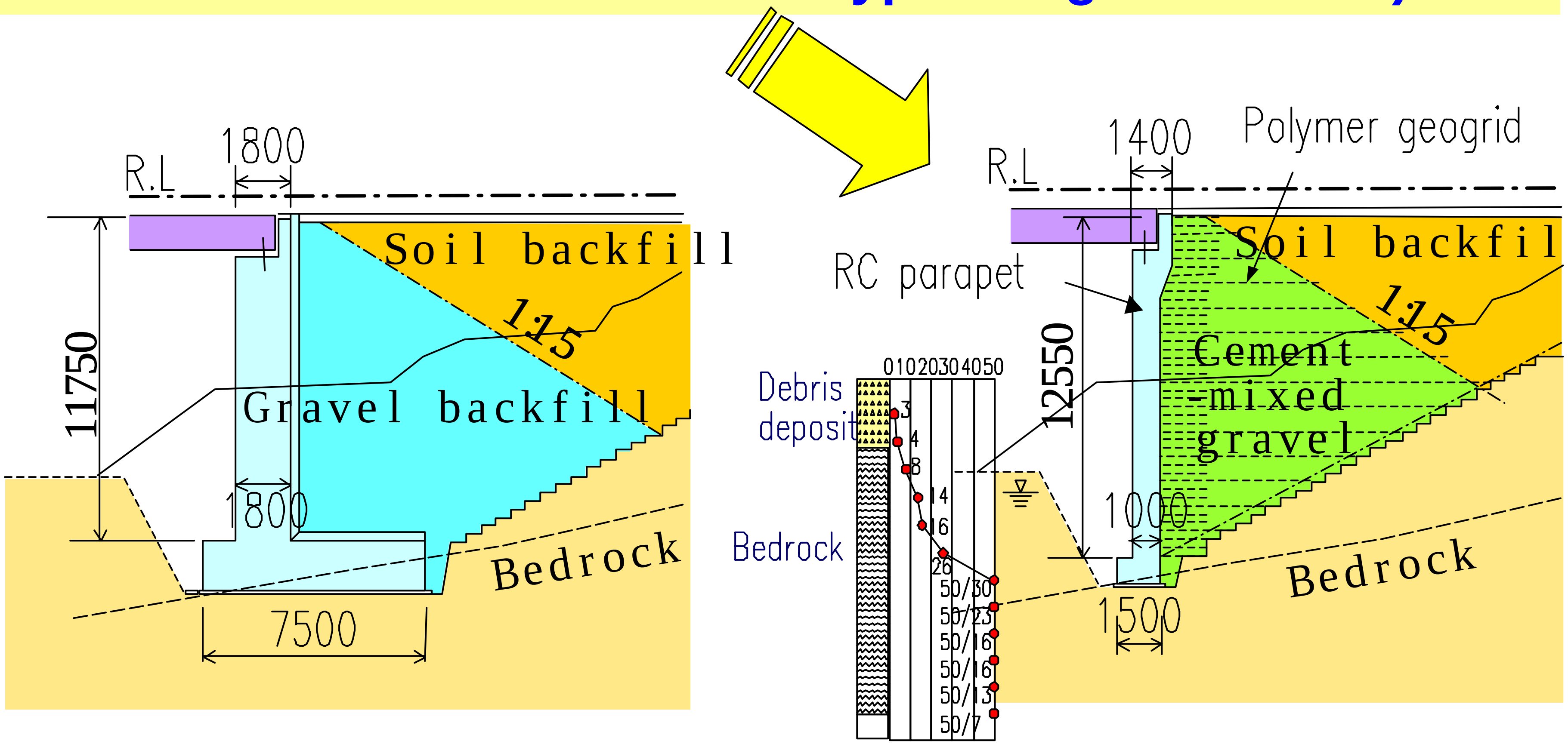




With the new type abutment, the backfill supports the RC parapet without activating the earth pressure on the back of the parapet.

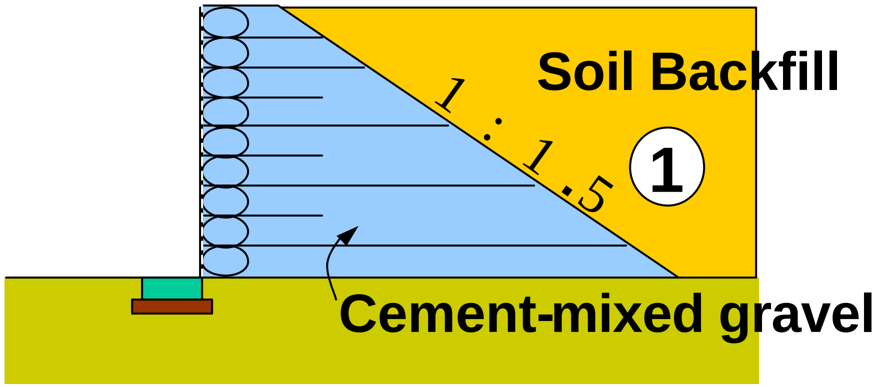
Conventional type versus **new type**

Cost reduction by 20 – 30 % (more if a pile foundation is constructed for a conventional type bridge abutment)



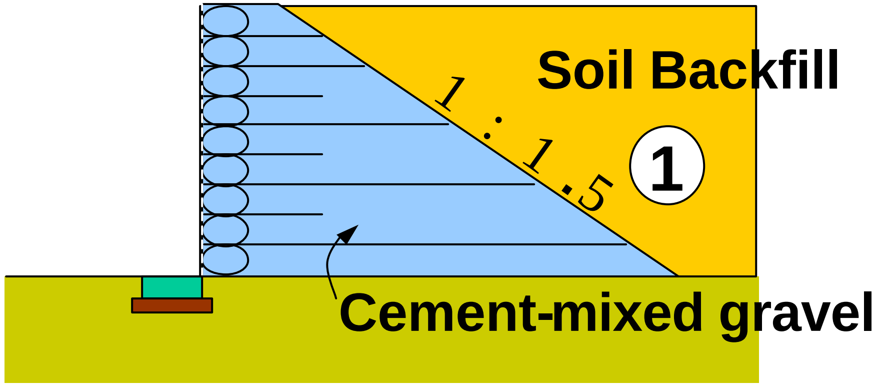
(only design) **(actually constructed)**

Staged construction procedure for the new type bridge abutment (1)



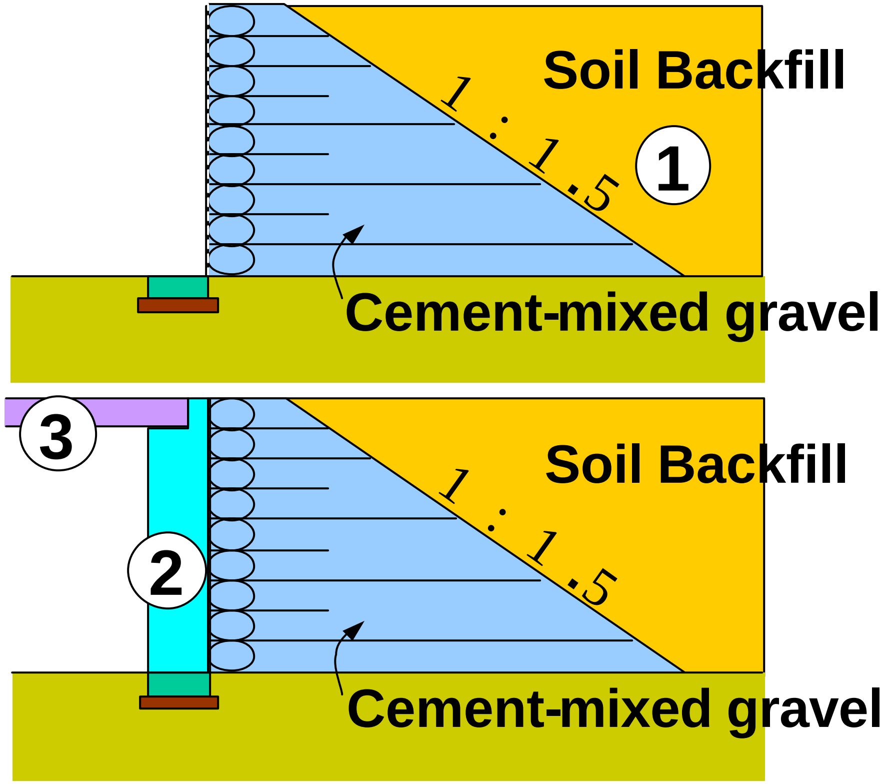
To avoid the damage to the connection between the reinforcement and the facing due to relative settlement of backfill during and after construction…….

Staged construction procedure for the new type bridge abutment (2)



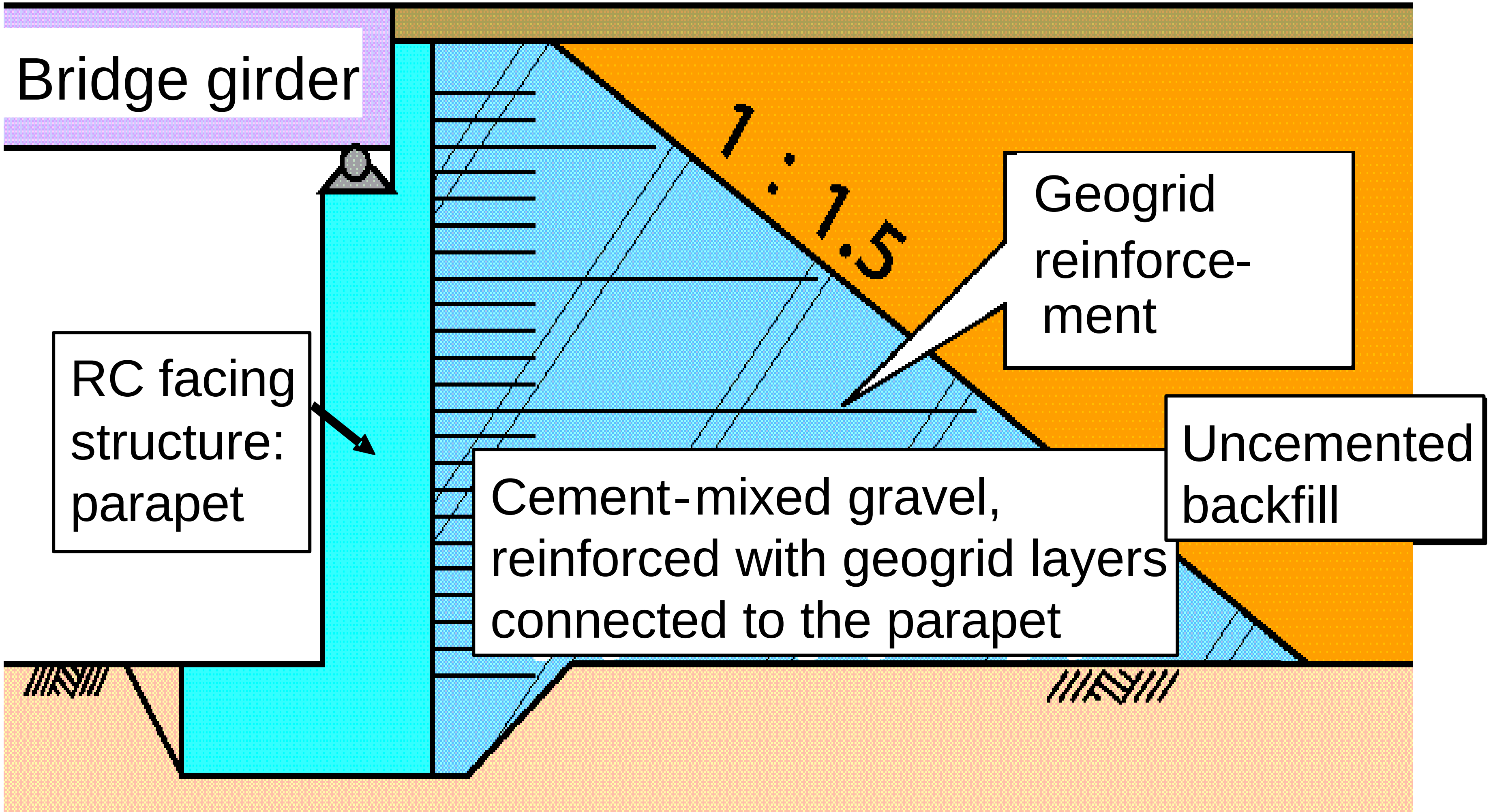
To avoid the damage to the connection between the reinforcement and the facing due to relative settlement of backfill during and after construction……..

Staged construction procedure for the new type bridge abutment (3)



The backfill supports the RC facing,

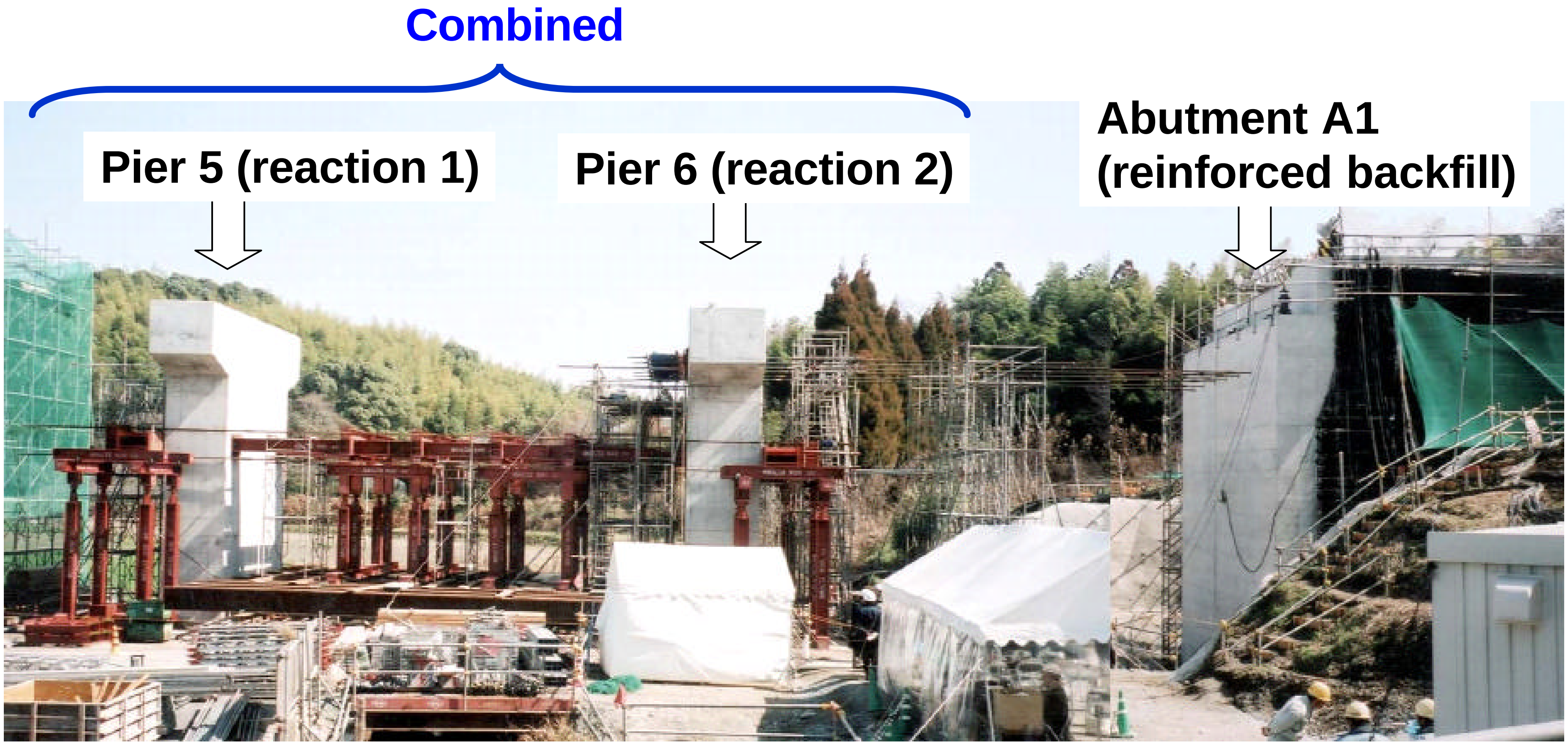
so no seismic earth pressure !



New type bridge abutment using backfill of cement-mixed gravel

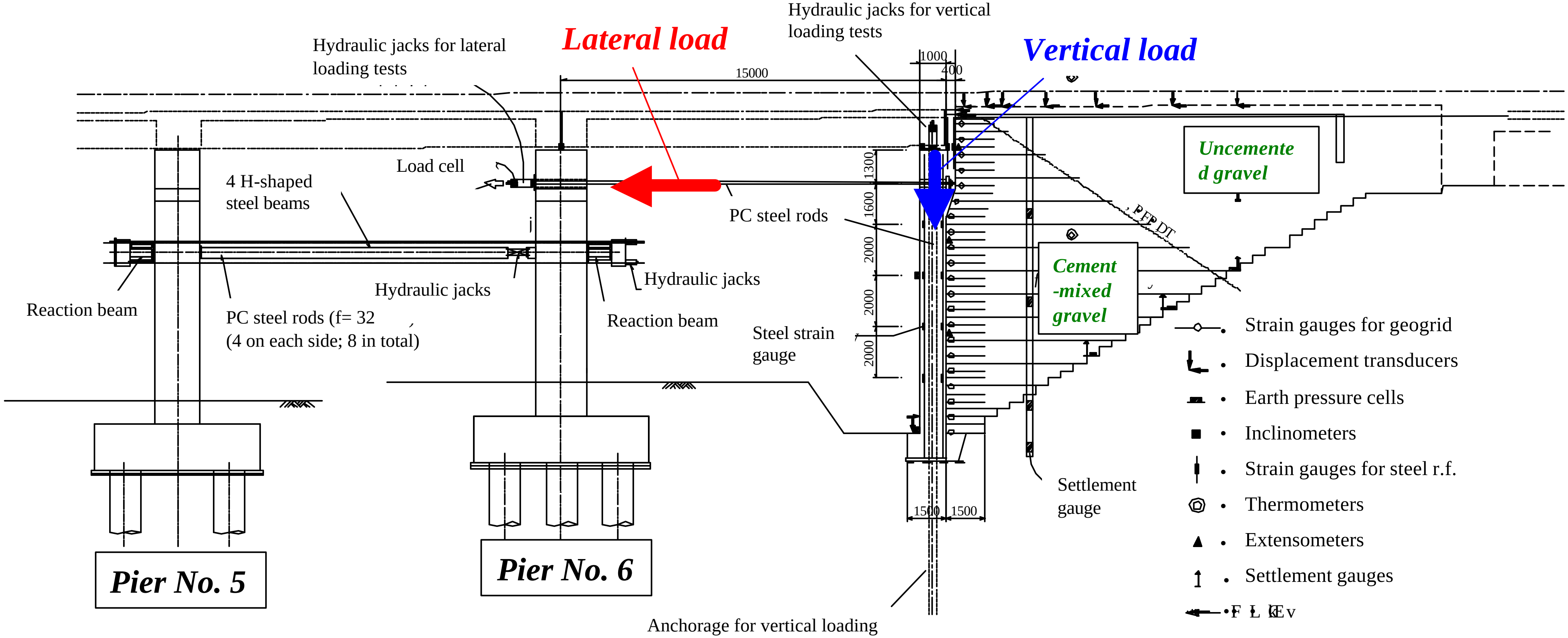
Contents:

Lateral loading test, 27 February 2003

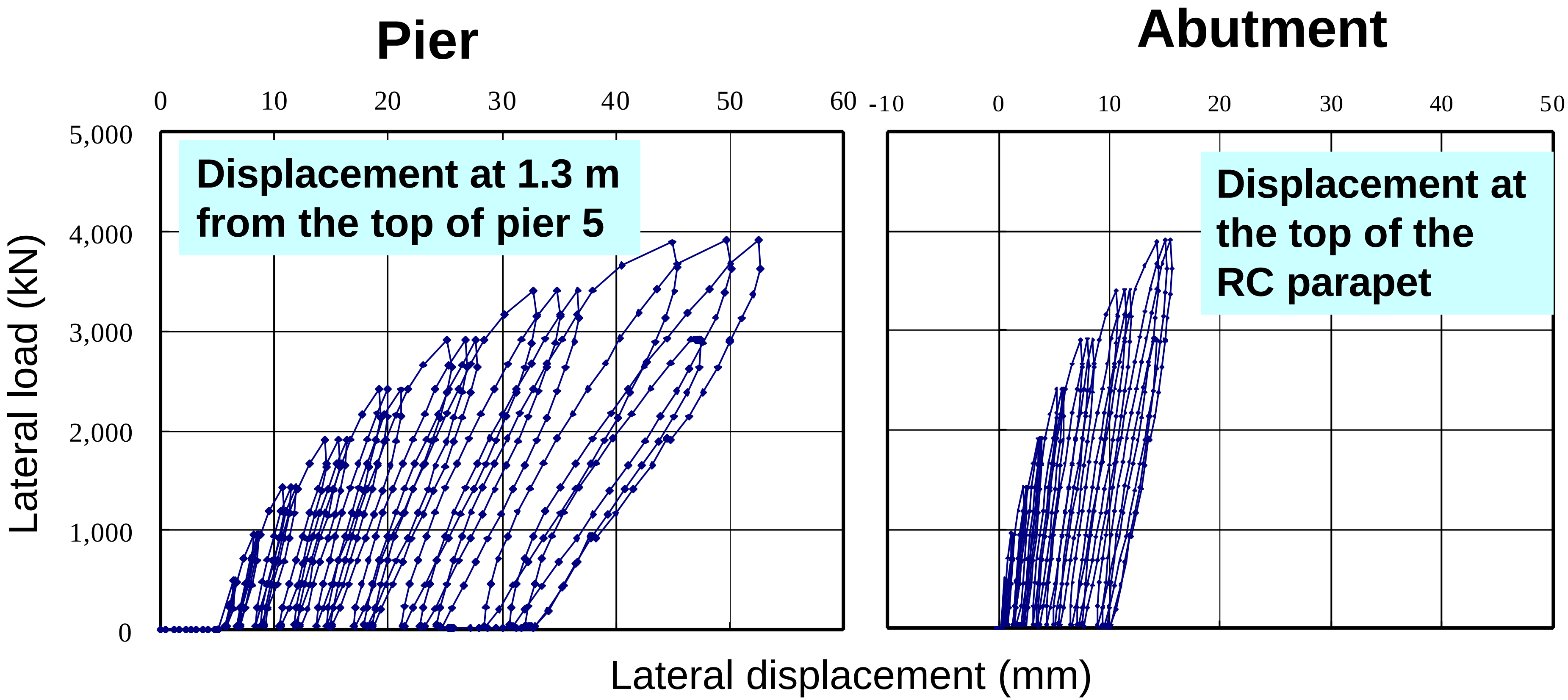


-Vertical loading test to ensure the vertical bearing capacity at the base of the parapet

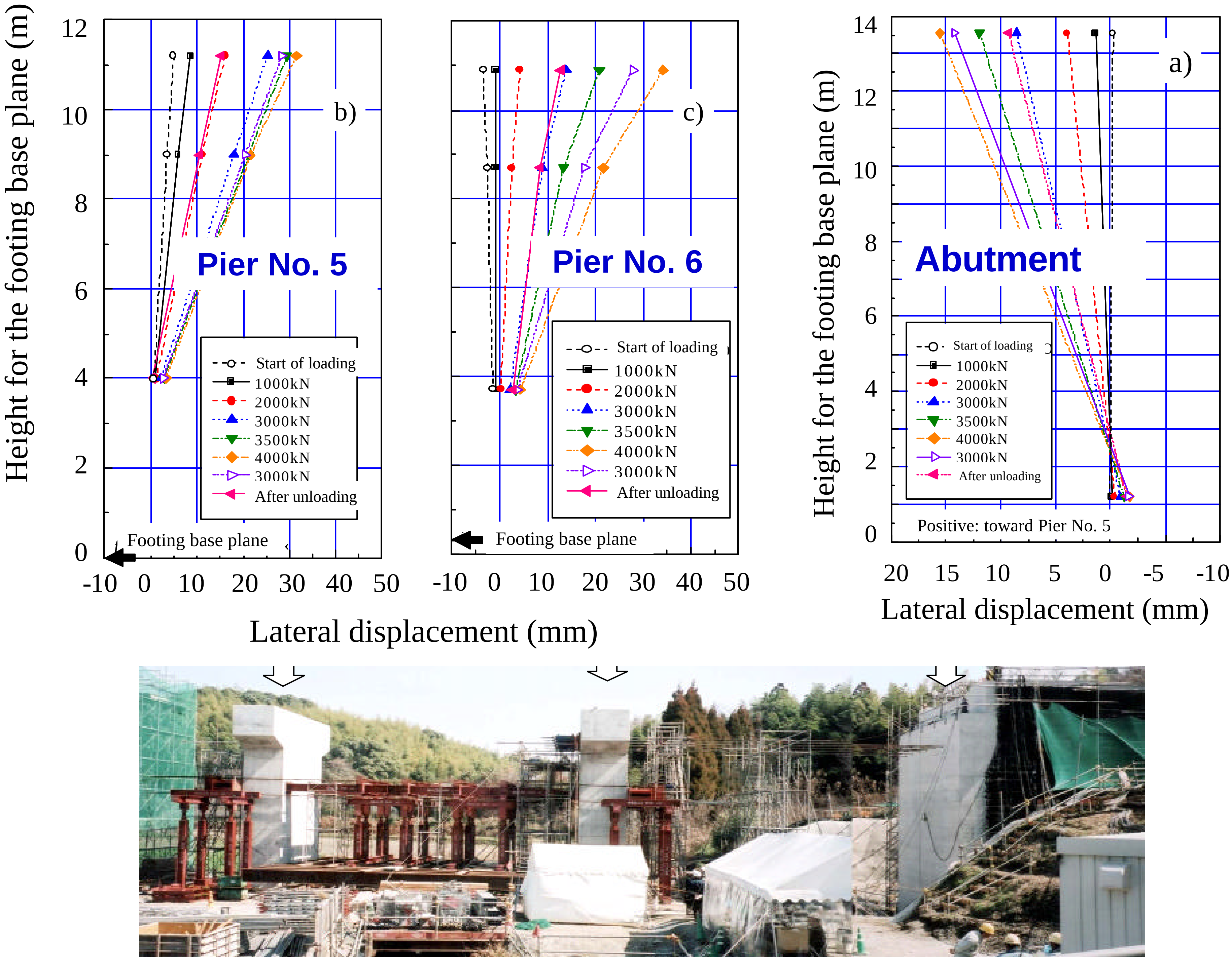
- Lateral loading test to ensure the connection strength



Lateral loading test
to ensure the connection strength

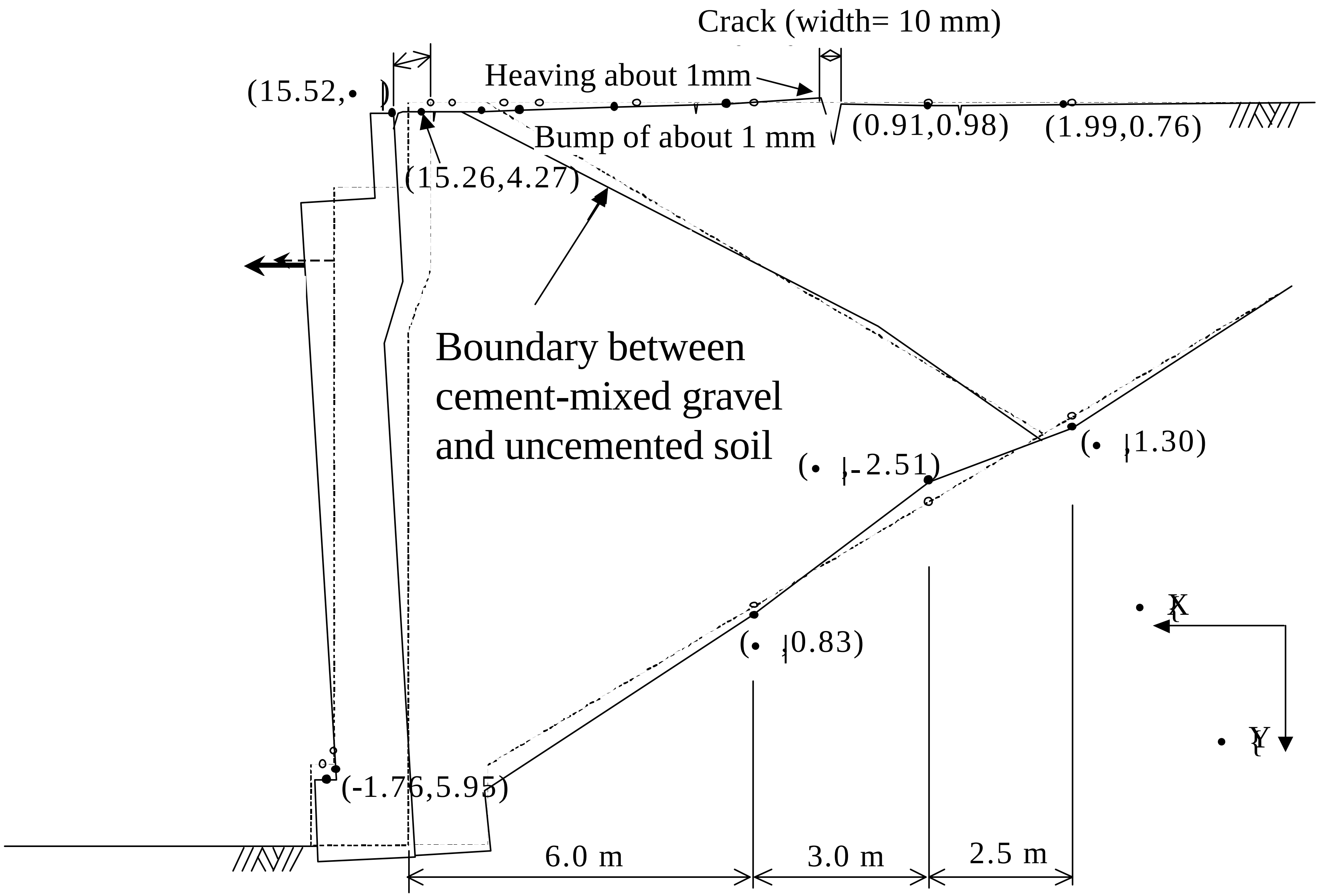


The bridge abutment was stiff enough and much more stable than two piers combined.

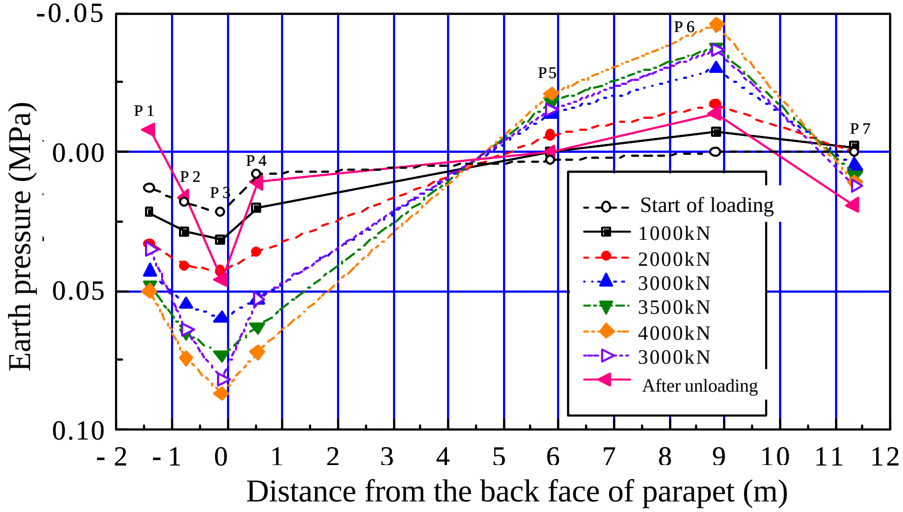

Height for the footing base plane (m)
b)
Pier No. 5
Start of loading
1000kN
2000kN
3000kN
3500kN
4000kN
3000kN
After unloading
Footing base plane
c)
Pier No. 6
Footing base plane
a)
Abutment
Positive: toward Pier No. 5
Lateral displacement (mm)

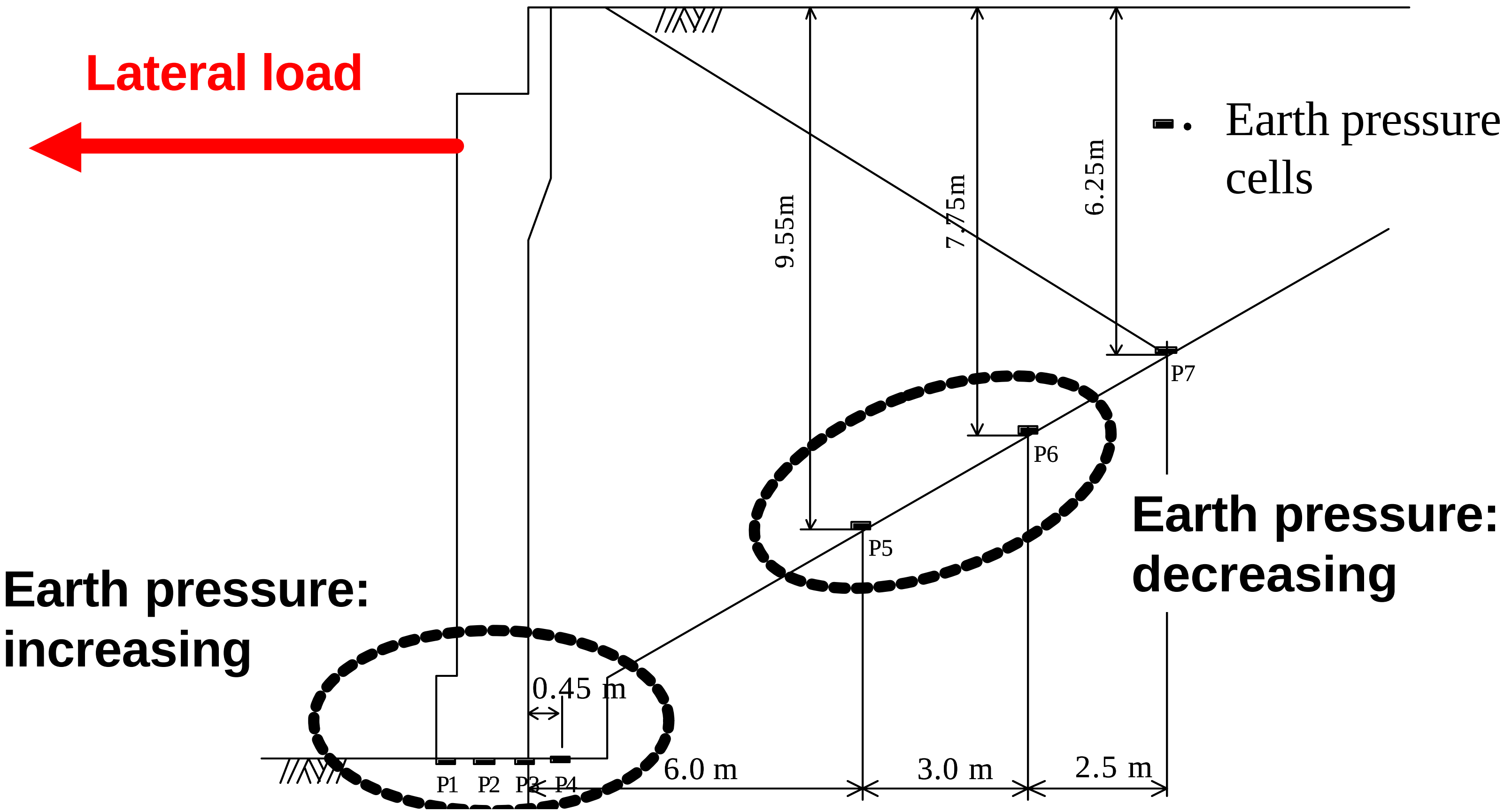
At the maximum lateral load, very small displacement and deformation of the abutment :

- *highly integrated behaviour !*



Earth pressure at the base of backfill of cement-mixed gravel also shows highly integrated behaviour of the backfill.





Summary -1:

A new cost-effective structural type has been proposed for bridge abutments requiring a high ultimate stability while allowing relatively small displacements; i.e., the backfill is geogrid-reinforced cement-mixed gravel, connected firmly to a RC facing structure (i.e., a parapet).

Because of a much higher stability of the backfill than a slender parapet, the backfill supports the parapet, rather than exerting active earth pressure to the parapet.

Summary -2:

The backfill of cement-mixed gravel was compacted at the optimum water content to maximize the compressive strength.

The RC parapet was staged-constructed after the backfill having a vertical wall face had been completed.

Summary-3:

Design and construction standard for bridge abutments having cement-mixed backfill **was published March 2004 to enhance the construction of new structural type bridge abutments at many other places.**

Summary of the talk today

1. Geosynthetic-reinforced retaining soil walls with a full-height rigid facing (GRS RW with a FR facing) is becoming popular to construct permanent important soil retaining structures.

2. GRS RW with a FR facing is much more stable, particularly more ductile, against dynamic load than gravity type RWs.

3. A new dynamic earth pressure theory accounting for strain softening and strain localization is proposed and used in practice.

4. The seismic stability of soil RWs on slope is particularly low. Remedy measures is proposed.

(to continue)

Summary (continued)

5. Three railway embankments that totally failed during 2004 Niigata-ken Chuetsu Earthquake were reconstructed to GRS-RWs with a FHR facing.

6. New type bridge abutments, GRS with PL&PS and cement-mixed backfill, were proposed.