Destabilization of a caisson-type breakwater by scouring and seepage failure of the seabed due to a tsunami

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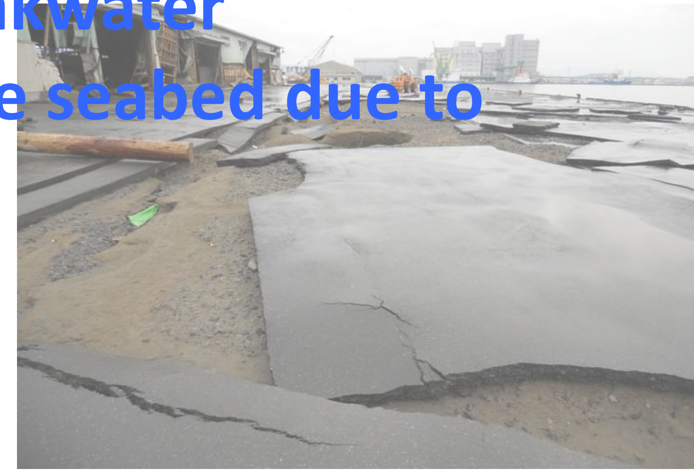
Topics

[Part.1] Introduction

[Part.2] Tsunami experiment using centrifuge model test

[Part.3] Numerical simulation using SPH method

[Part.4] Conclusion



Part. 1

Introduction

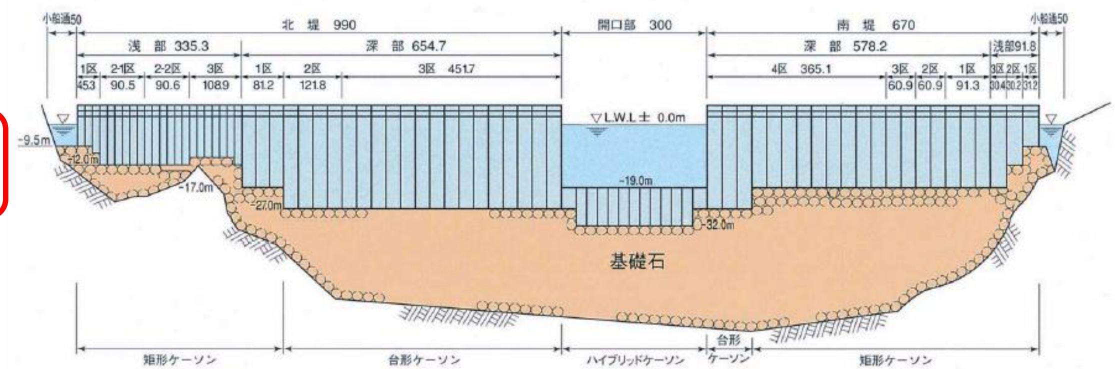
Introduction（The Great East Japan Earthquake）

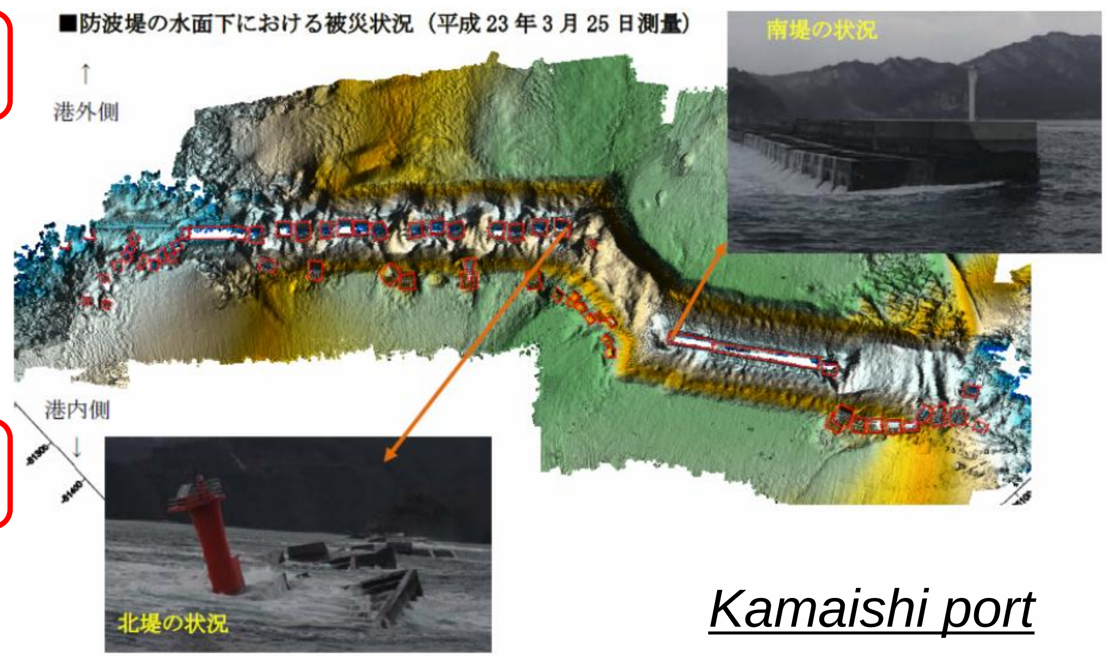
In 2011, tsunami hazard occurred in Japan (the Great East Japan earthquake).

In 26 minutes after earthquake

In 31 minutes after earthquake

In 46 minutes after earthquake





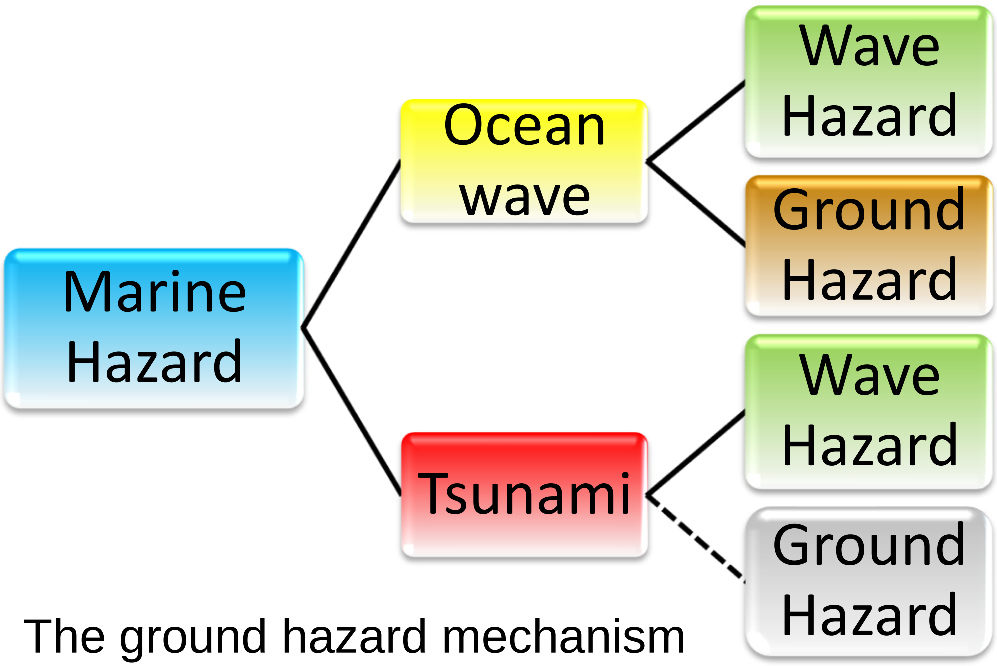
<u>*Kamaishi port*</u>

Ministry of Land, Infrastructure, Transport and Tourism

MAEDA Lab.

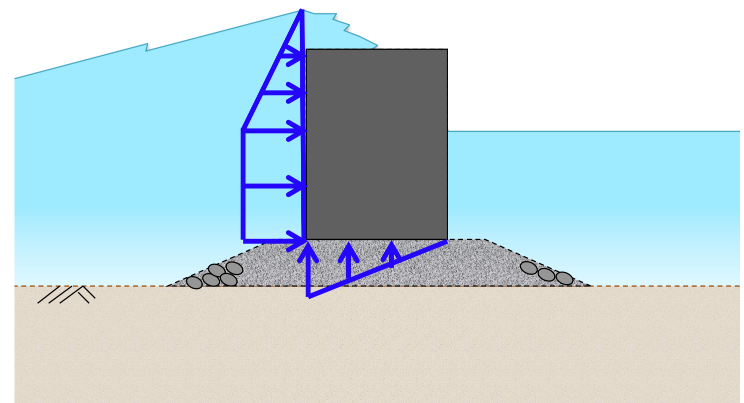
Technical Research Institute TOYO CONSTRUCTION Co., Ltd.

Introduction



The ground hazard mechanism by the tsunami is not understood.

A past tsunami research has been discussing damage of marine structure with the interaction of the tsunami and the structure.



Tanimoto (1994) et al. eq. : $P_S = 2.2\rho \cdot g \cdot a_i$

Clarification of damage mechanism
with interaction of the tsunami, seabed soil and structure

Started from 2009

Development of

Experimental methodology using centrifugal device

Numerical analysis using SPH method

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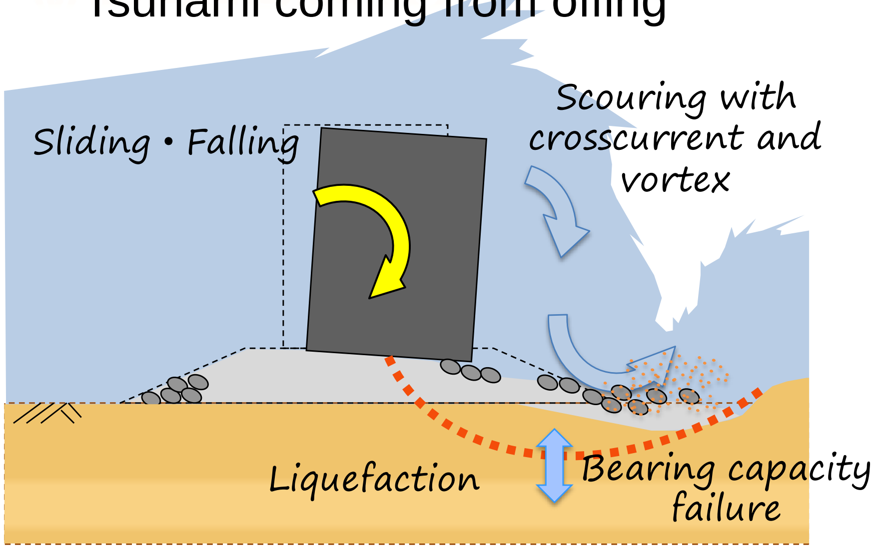
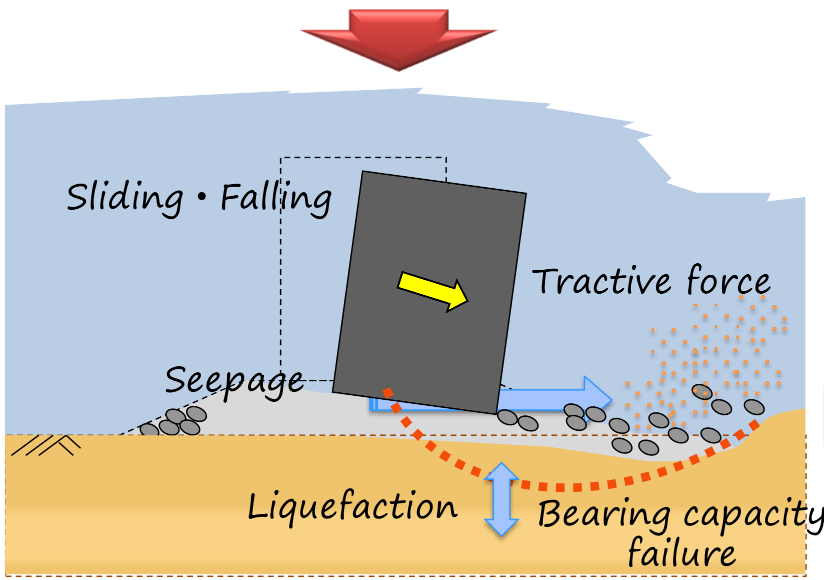
Introduction - Estimation of damage -

Anaseism

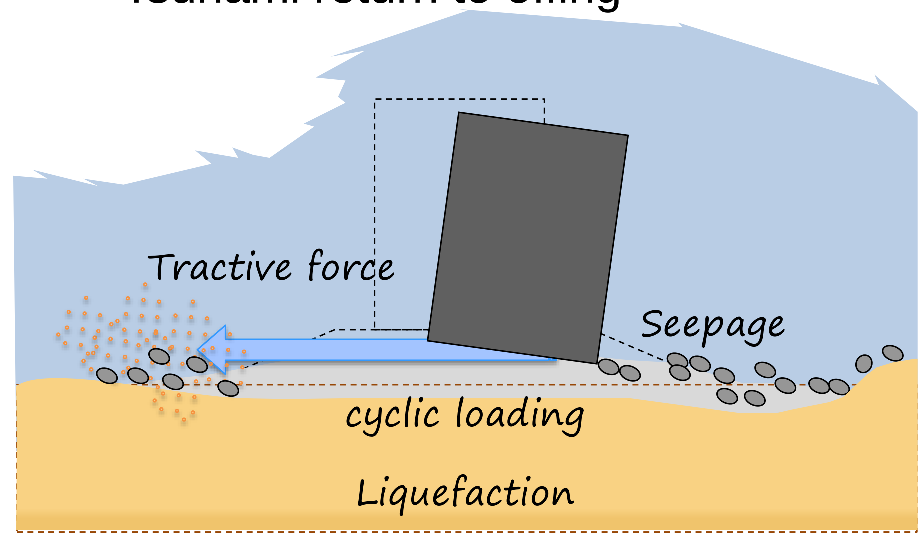
Tsunami coming from offing

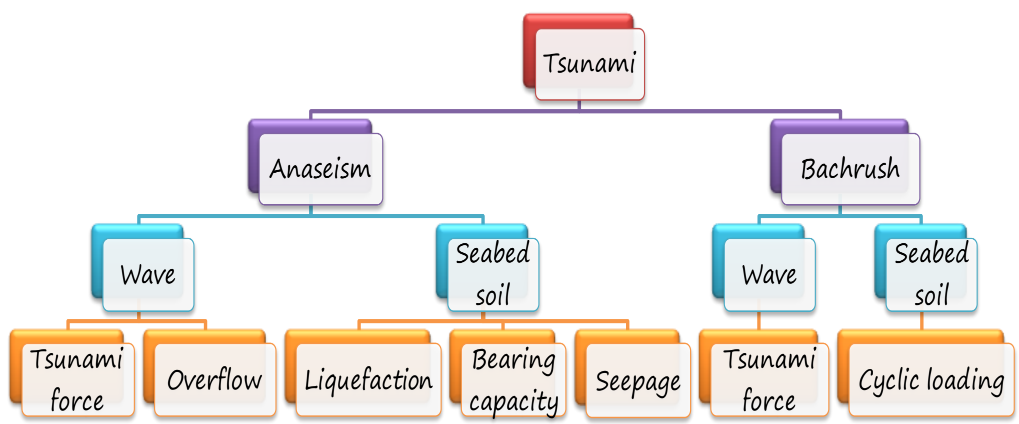




Backrush

Tsunami return to offing





Part. 2

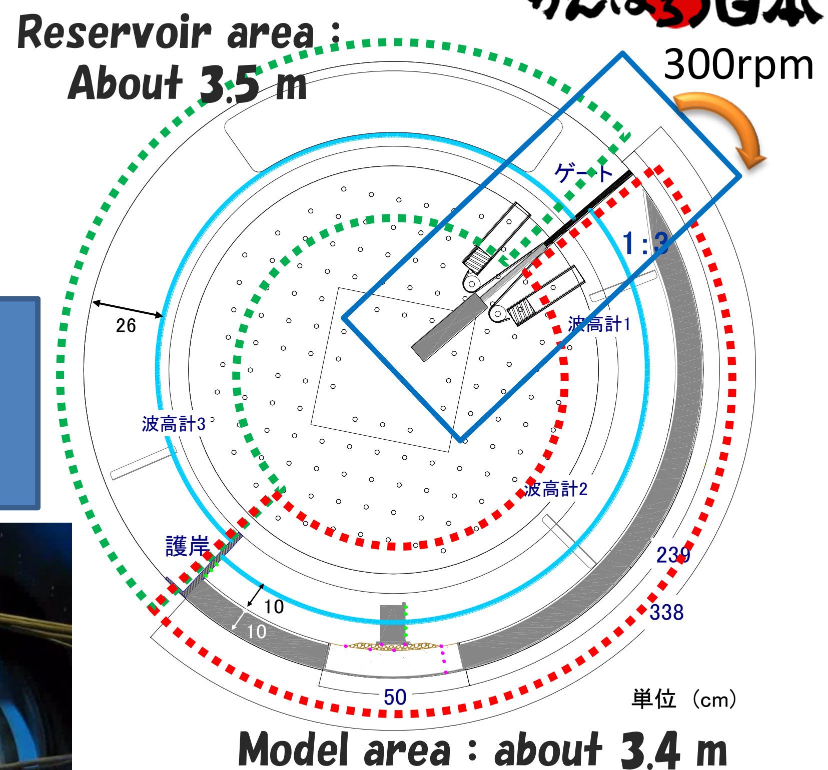
Tsunami experiment with drum-type centrifuge device

Model test device

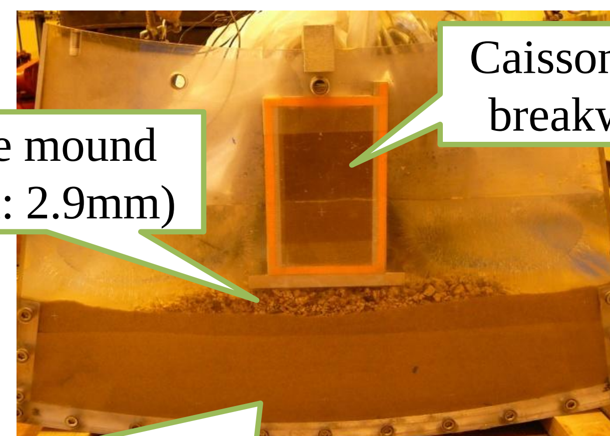
Drum-type centrifuge device

Toyo construction Co., Ltd.

The experiment study used a 2.2 m diameter drum-type centrifuge device. The tsunami was generated by dam break using gate.



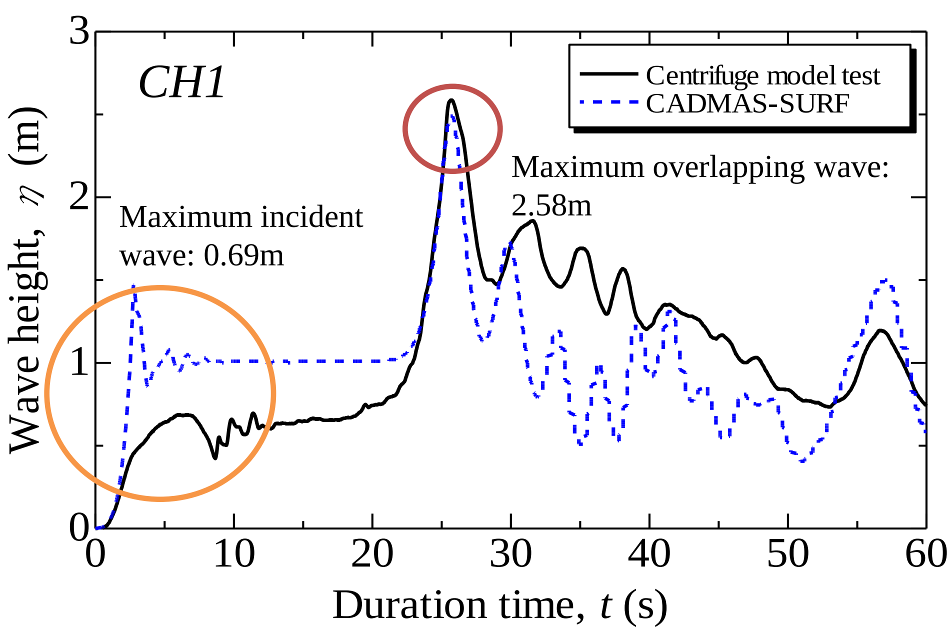
Maximum acc.	440G （600rpm）	
Dimensions	Model	Proto type （Maximum acc.）
Diameter	2.2 m	3041 m
Width	0.3 m	132 m
Depth	0.3 m	132 m （Ground 32 m)
Maximum force	3.7 ton	1628g-ton

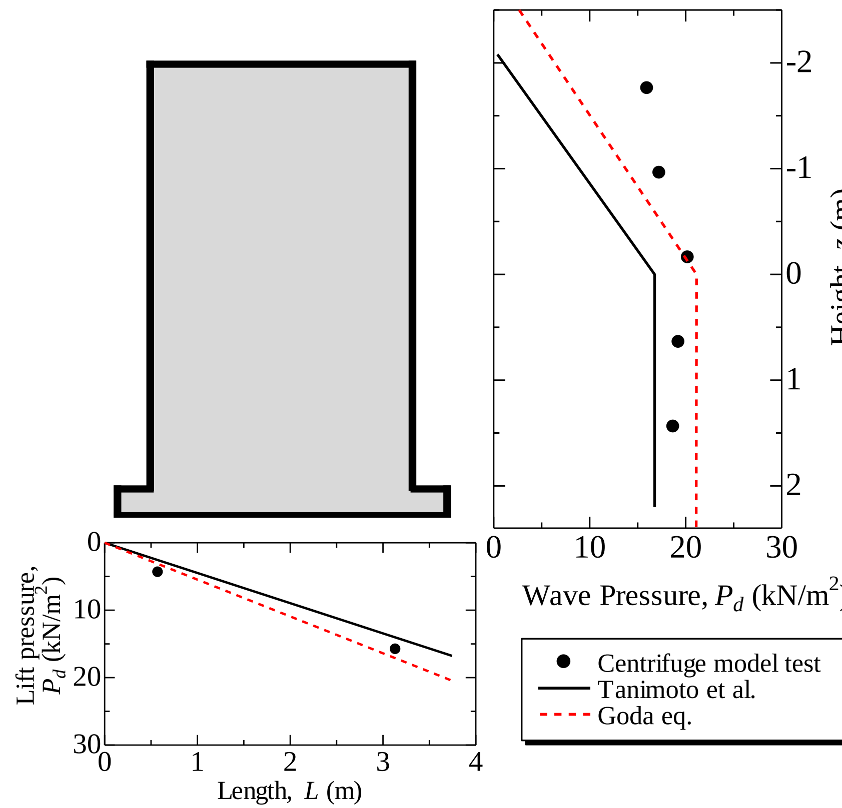


Tsunami force

Tsunami experiment in a 32 g field

Gage points	Maximum incident wave	Maximum overlapping wave
CH 1 near the gate	0.69 m	2.58 m
CH 2 near the breakwater	0.77 m	2.39 m



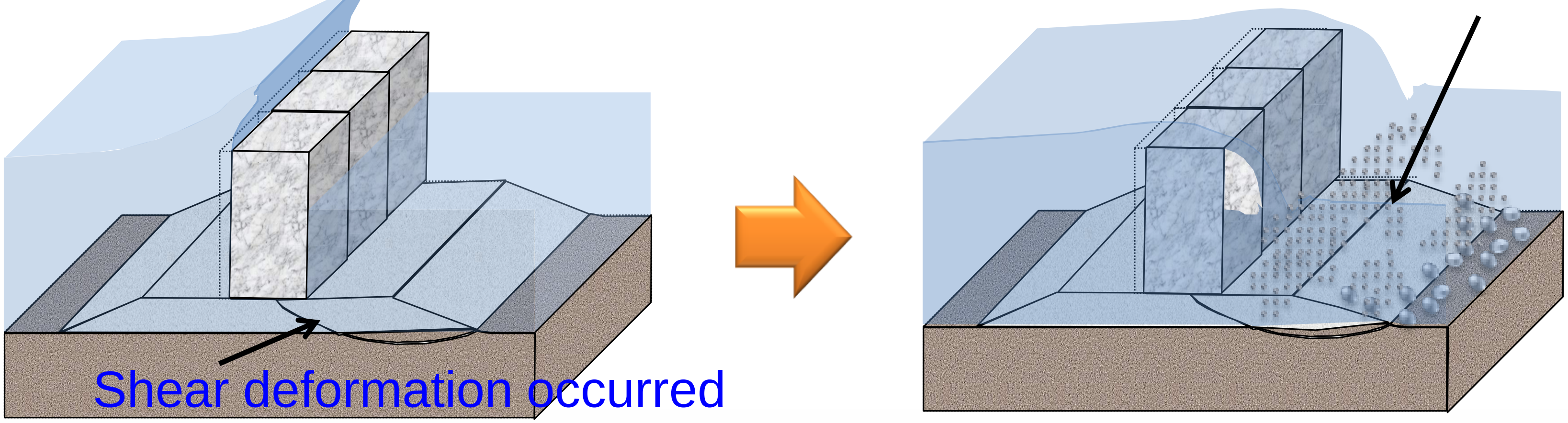


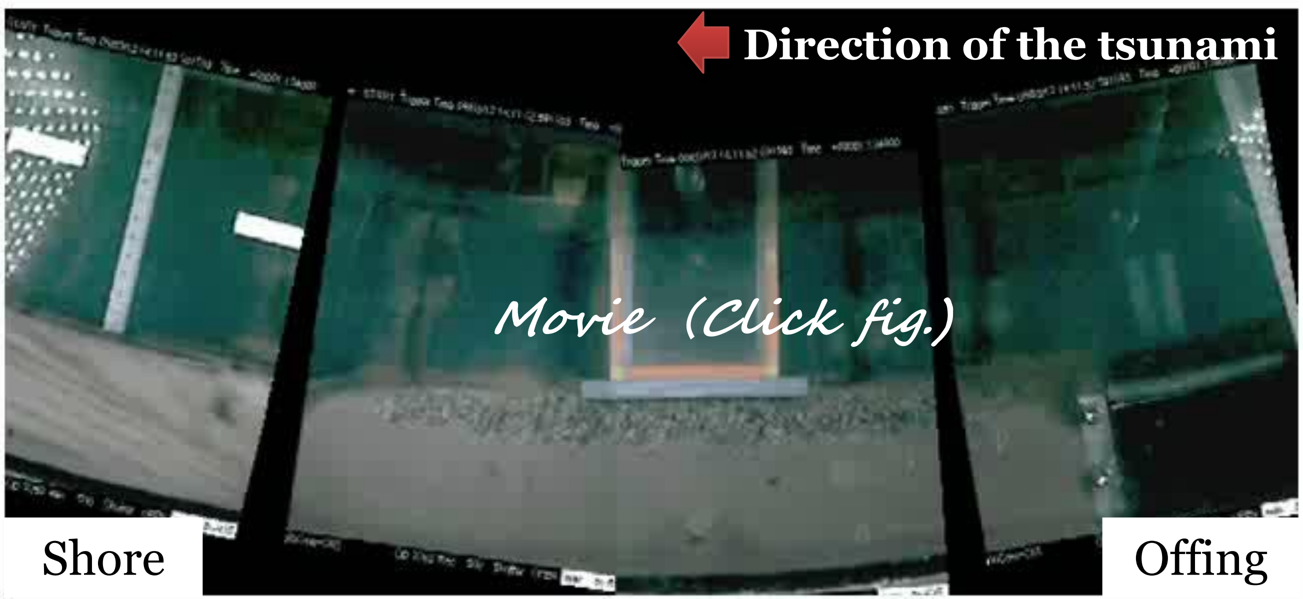
The experimental wave pressure were as large as than the results obtained using Tanimoto's equation or Goda equation.

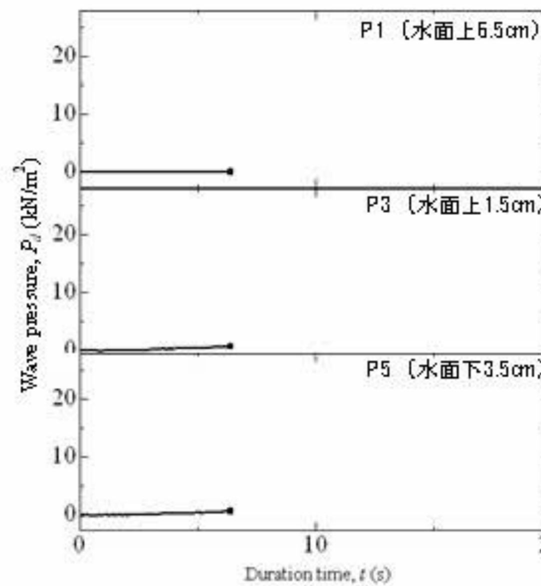
Deformation of breakwater, rubble mound and seabed soil

The breakwater was slided

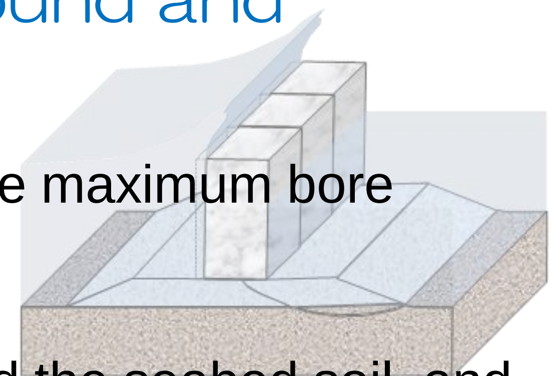
Rubble mound and seabed soil was scoured





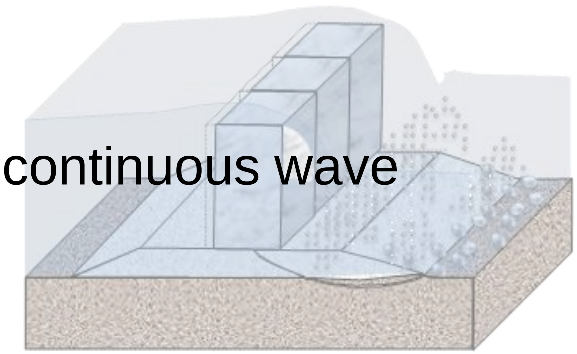


Deformation of breakwater, rubble mound and seabed soil

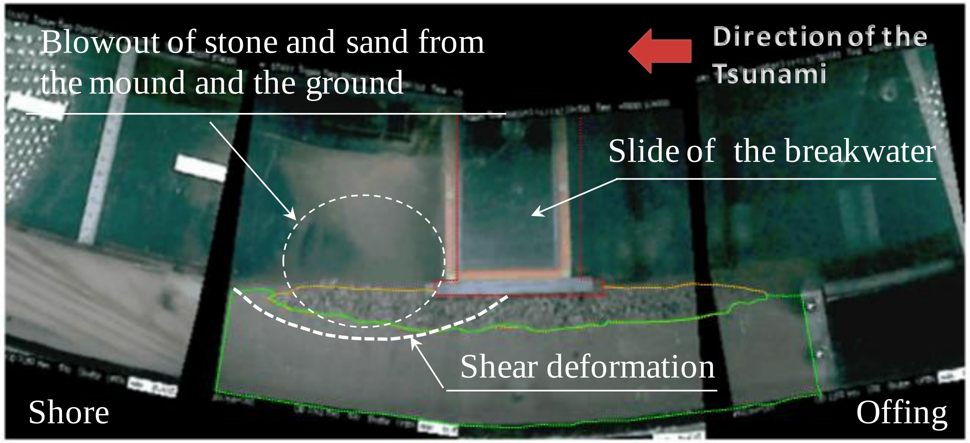


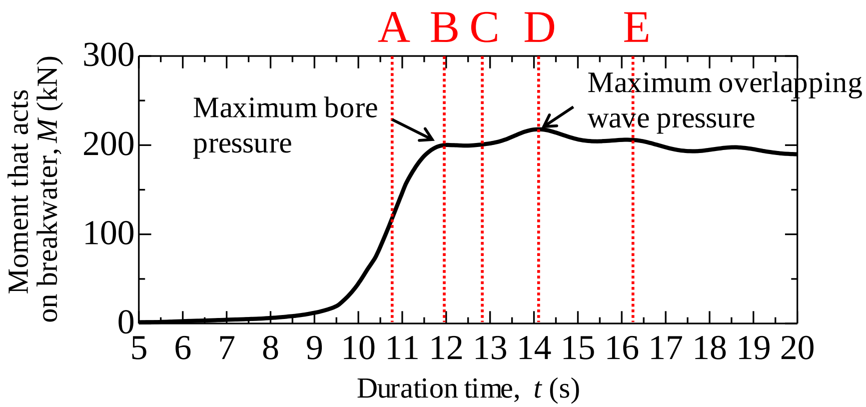
The breakwater was slided (points A and B) when the maximum bore pressure acted.

Shear deformation occurred in the rubble mound and the seabed soil, and decreased the bearing capacity.



Rubble mound and seabed soil was scoured while continuous wave pressures were acting (points B-E).

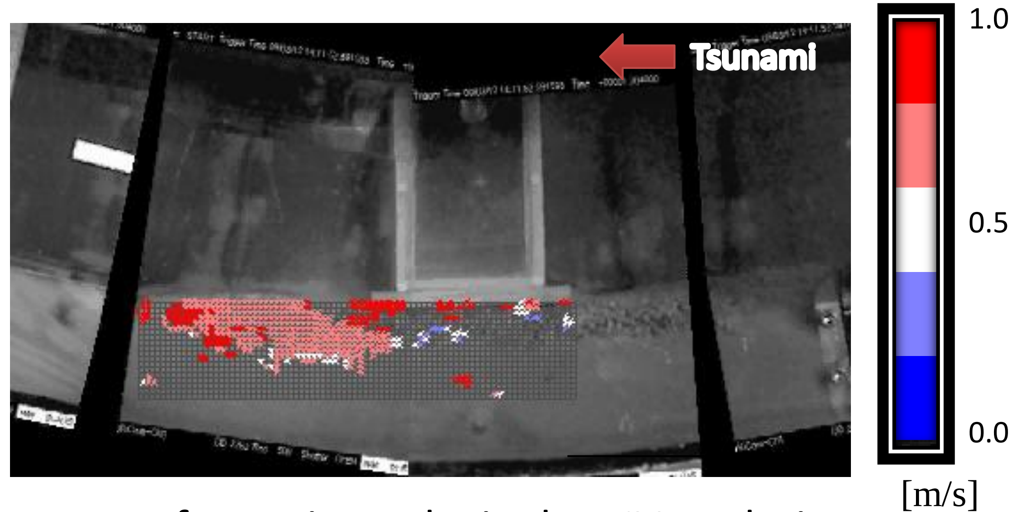




Deformation of rubble mound and seabed soil due to tsunami

Discussion of bearing capacity destruction

Shear deformation occurred in the rubble mound and the seabed ground with move of the breakwater.
And, the safety rate of circular slide was smaller than 1.0.



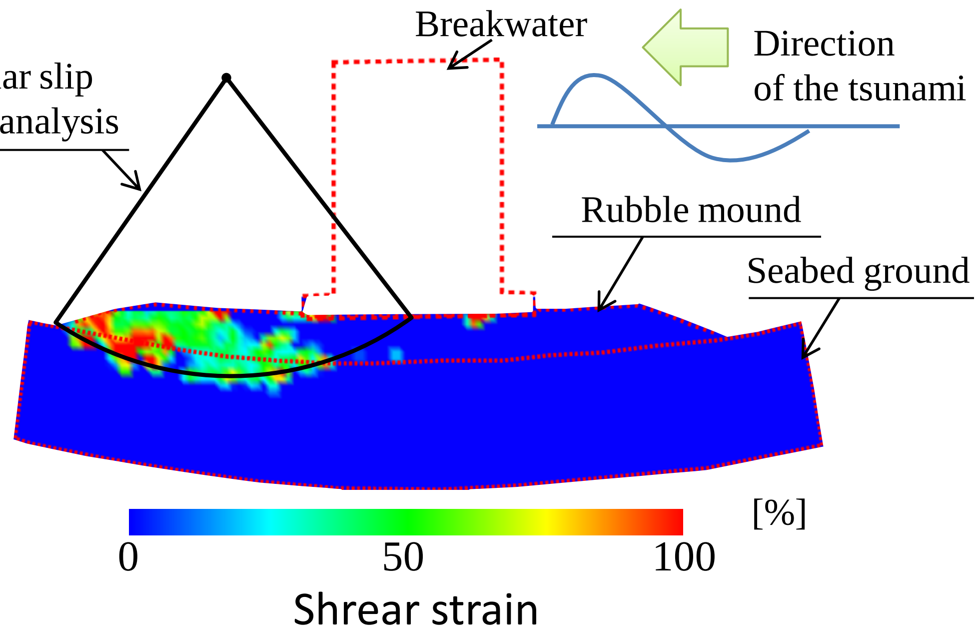
Deformation velocity by PIV analysis

Bishop method

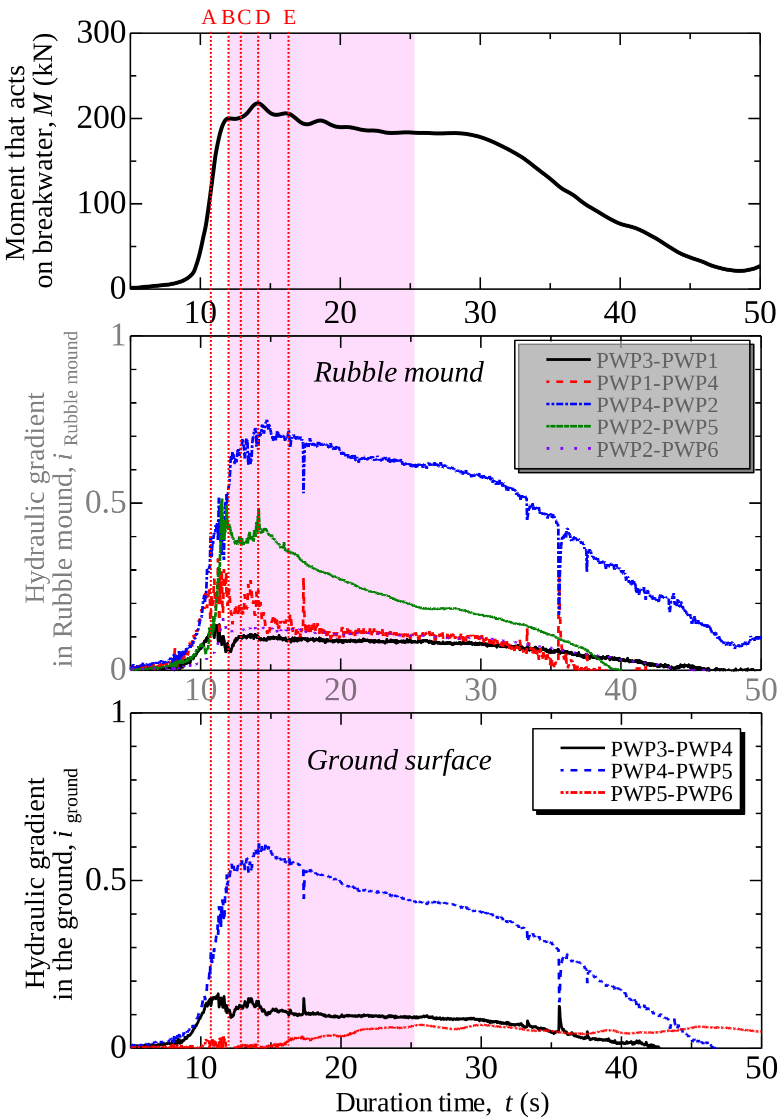
$$F_s = \frac{\sum\left\{\left[c's + (w'+q)\tan\phi'\right]\right\}\dfrac{\sec\theta}{1+\tan\theta\tan\phi'}}{\left\{\sum(w'+q)\sin\theta' + a P_h/R\right\}}$$

Safety rate of circular slide

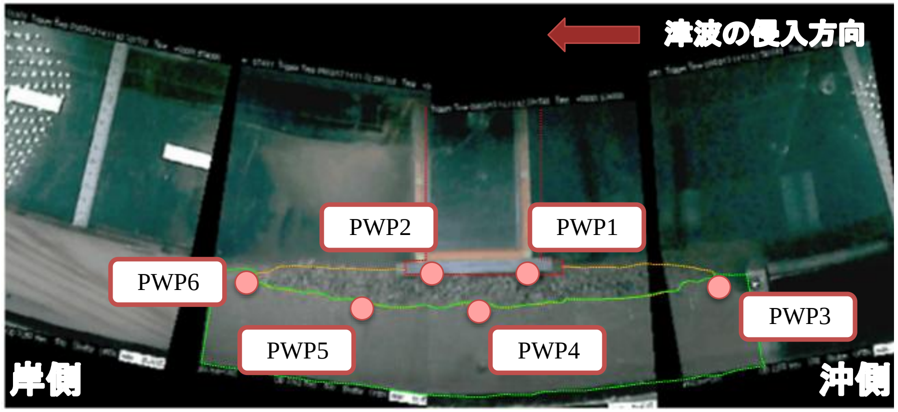
$$F_s = 0.91 \leq 1.0$$



Scouring and blowout with seepage flow



$$i_{cr} = \frac{G_S - 1}{1+e} = \frac{2.656-1}{1+0.8538} = 0.893$$



PWP2~PWP4 (into mound) : $i_{max} \fallingdotseq 0.8$

PWP2~PWP5 (into mound) : $i_{max} \fallingdotseq 0.5$

PWP 4 ~PWP 5 (near the ground surface) : $i_{max} \fallingdotseq 0.65$

We calculated the hydraulic gradient using the measured pore water pressure.

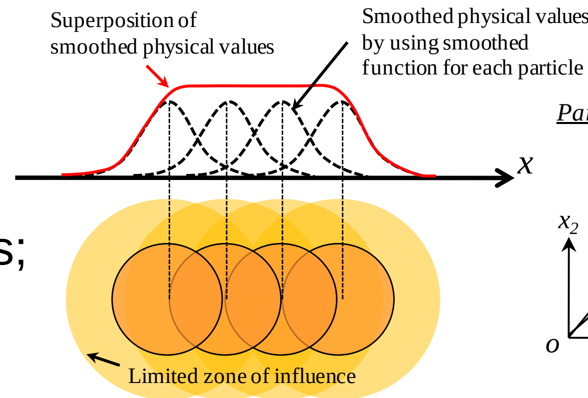
The results, the hydraulic gradient increased from the center of the breakwater bottom toward the shore while the continuous wave pressure was acting.

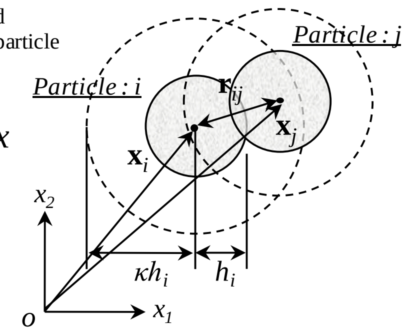
Part. 3

Numerical simulation using SPH method

Smoothed Particle Hydrodynamics

$$< f(\mathbf{x}) >= \int W(\mathbf{x}-\mathbf{x}', h) f(\mathbf{x}') d\mathbf{x}'$$





The feature of the SPH method is as follows;

- Mesh free
- Lagrangian method
- Initial modeling is easy.

Soil-fluid coupling

Soil-fluid coupling in the SPH method calculate fluid phase and solid phase, and the obtained results are overlapping by Darcy's low.

$$\mathbf{f}^{sf} = n^2 \frac{\rho_f g}{k} (\mathbf{v}^s - \mathbf{v}^f)$$

n : Porosity

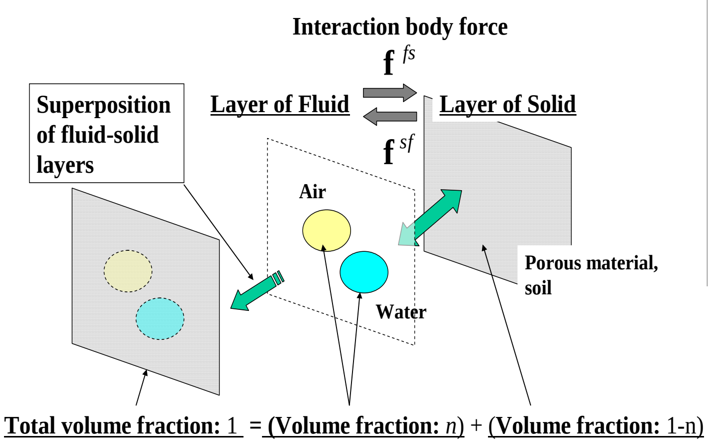
k : Permeability

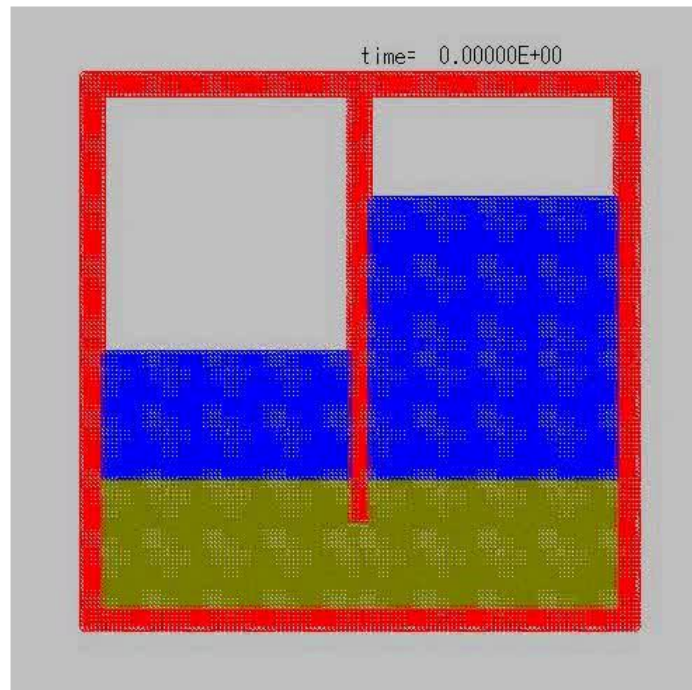
ρ_f : Density of fluid

g : Acceleration of gracity

v^s : Velocity of solid

v^f : Velocity of fluid

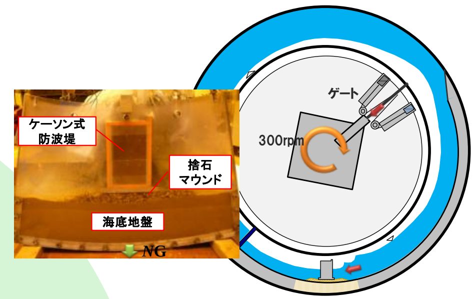


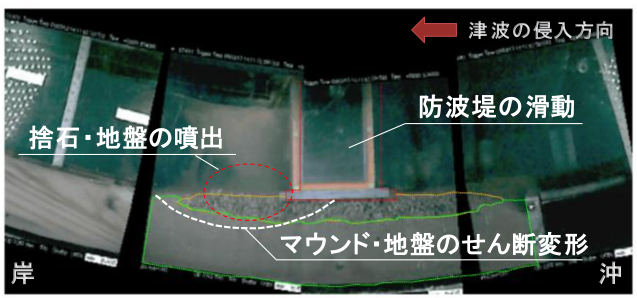


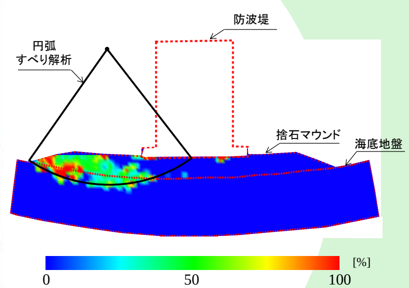
Seepage around sheet pile
(K. Maeda, M. Sakai (2004))

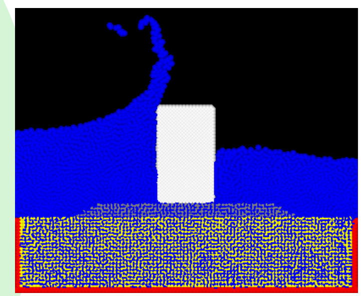
Comparison between experimental result and numerical analysis result

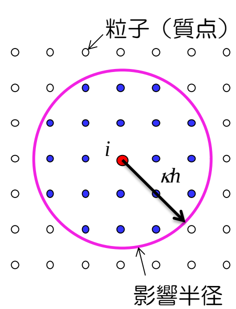
Tsunami experiment
using centrifuge device

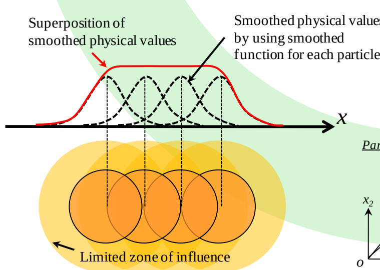


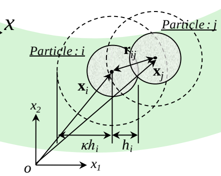












Numerical Analysis
using SPH method

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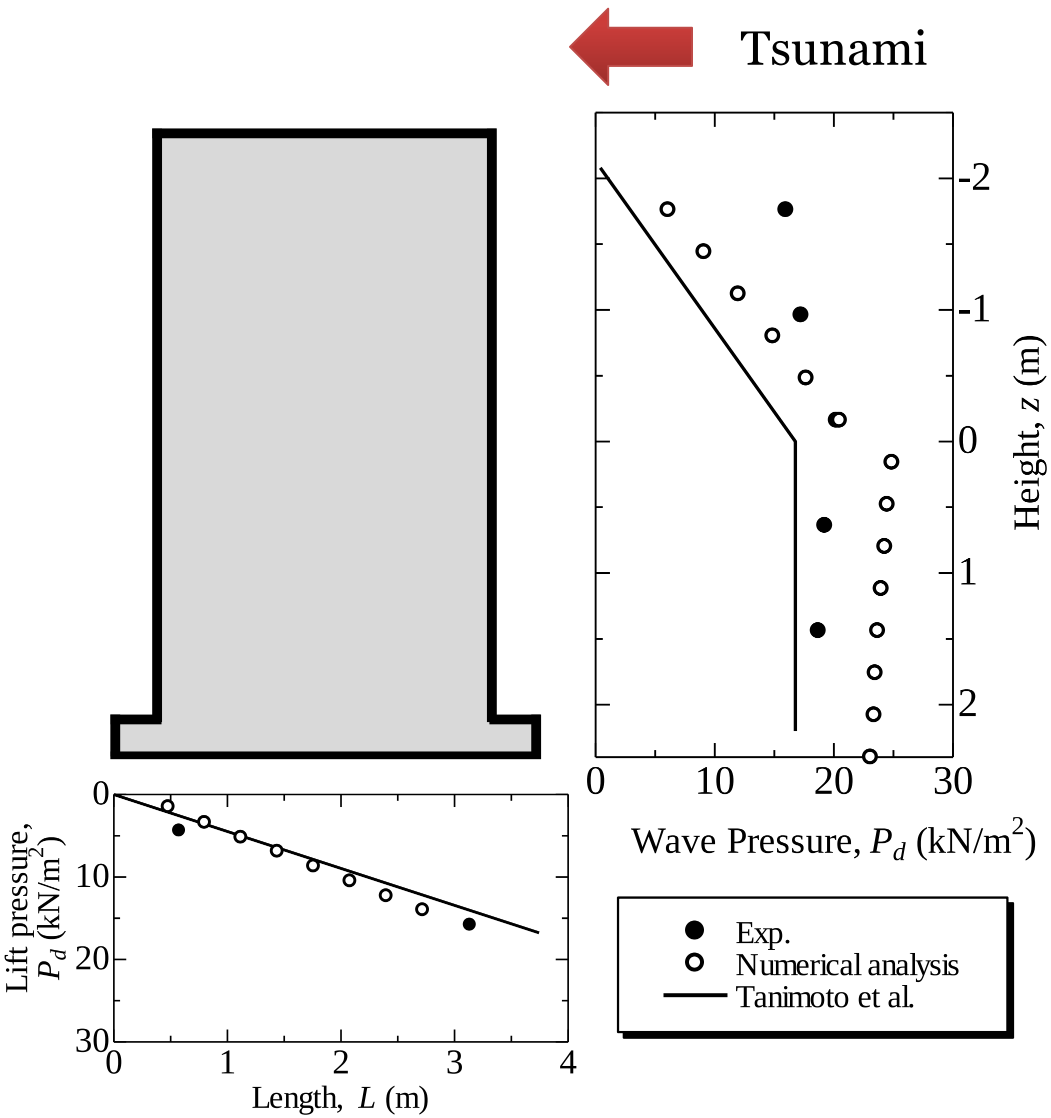
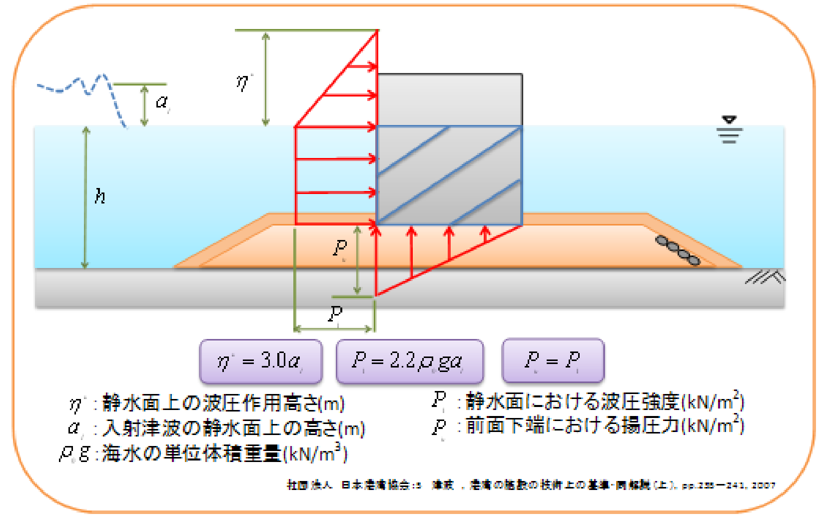
Wave pressure that acts on breakwater



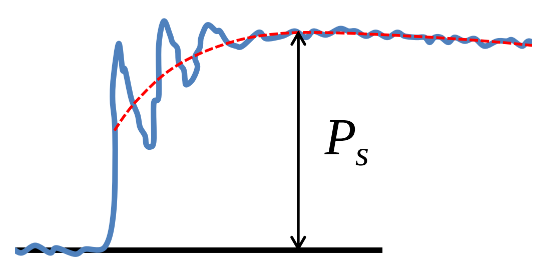
Standard in technology of facilities in harbors



Tanimoto(1994) et al. eq.

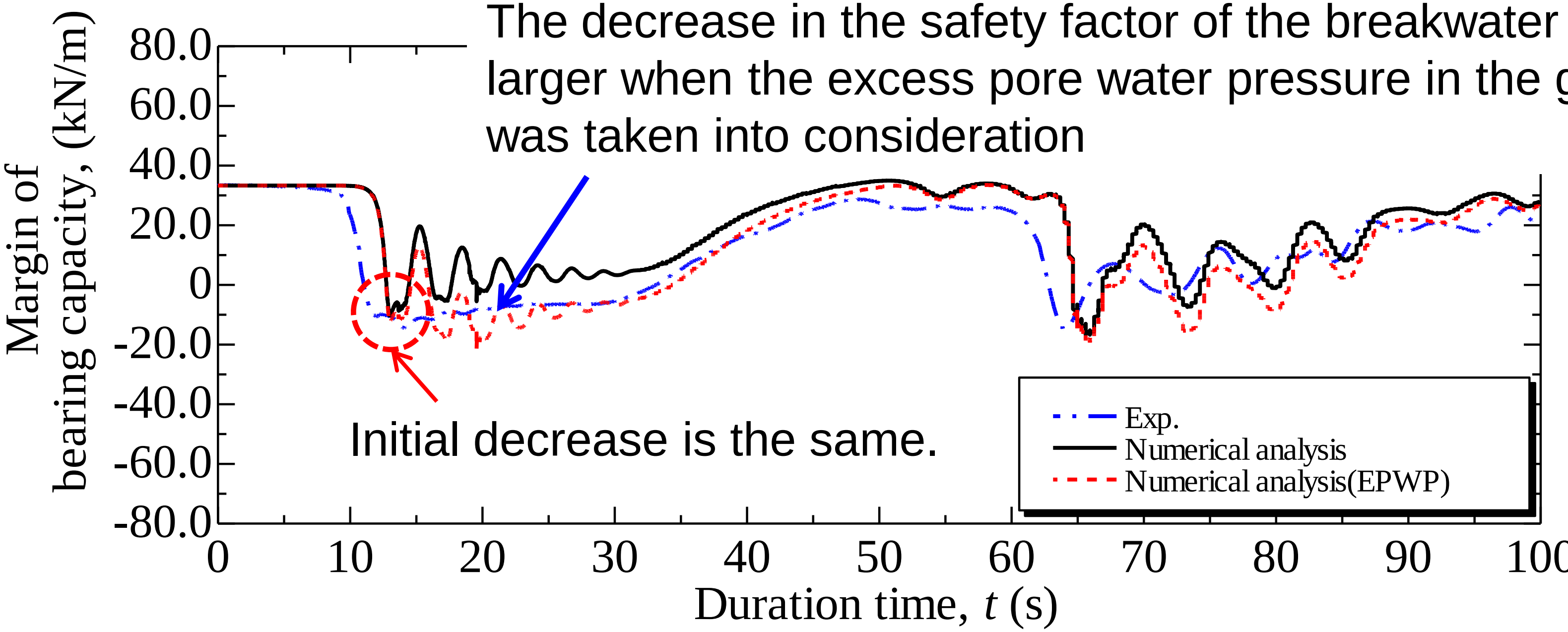
$$P_S = 2.2\rho \cdot g \cdot a_i$$

第1波襲来時



The wave pressure of numerical analysis were as large as than experiment result and tanimoto's equation.

Margin of bearing capacity



Margin of bearing capacity

$$F_s = \sum \{[c's + (w'+q)\tan\phi']\} \frac{\sec\theta}{1+\tan\theta\tan\phi'} - \{\sum (w'+q)\sin\theta' + a P_h / R\}$$

Margin of bearing capacity＝

Bearing capacity strength － Tsunami force

－ Bearing capacity decrease in breakwater due to increase of excess pore water pressure in the ground

Comparison between experimental result and numerical analysis result

Seepage flow into rubble mound and seabed soil

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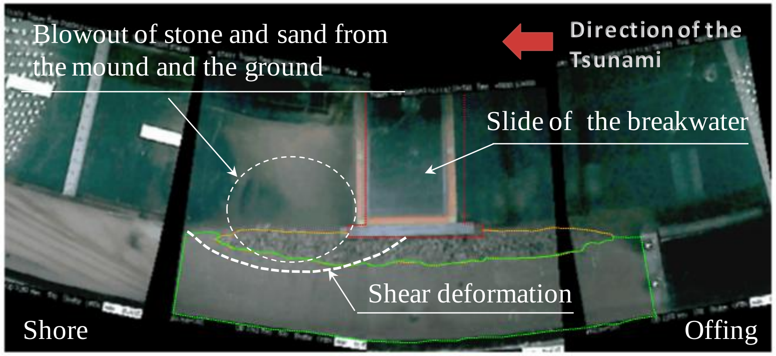
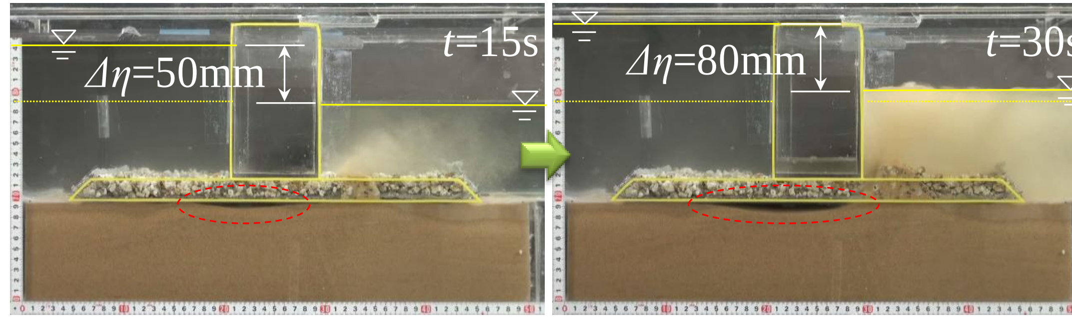
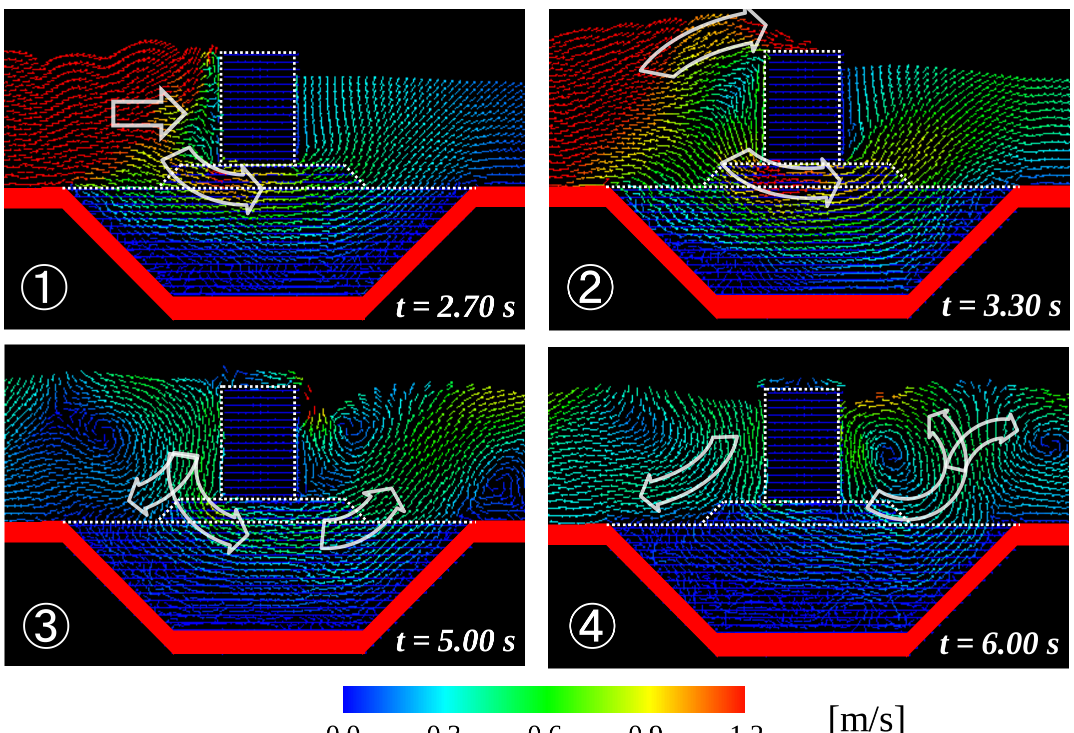


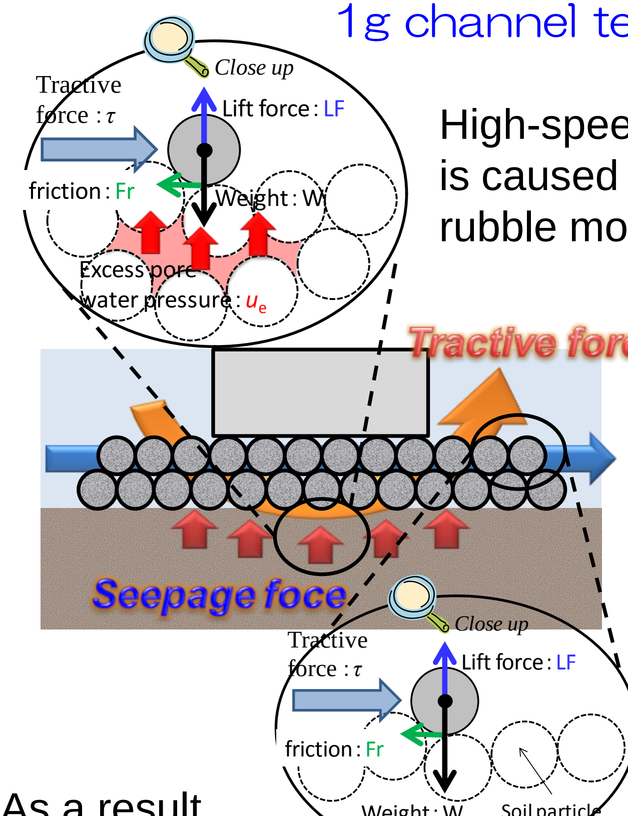
Centrifuge model test



1g channel test



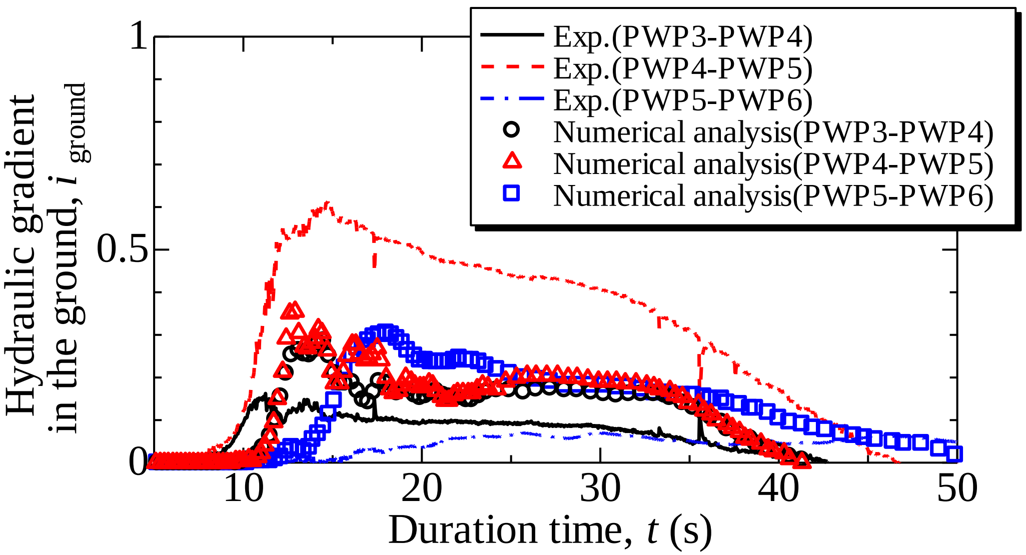
result of SPH analysis（flow velocity vector）

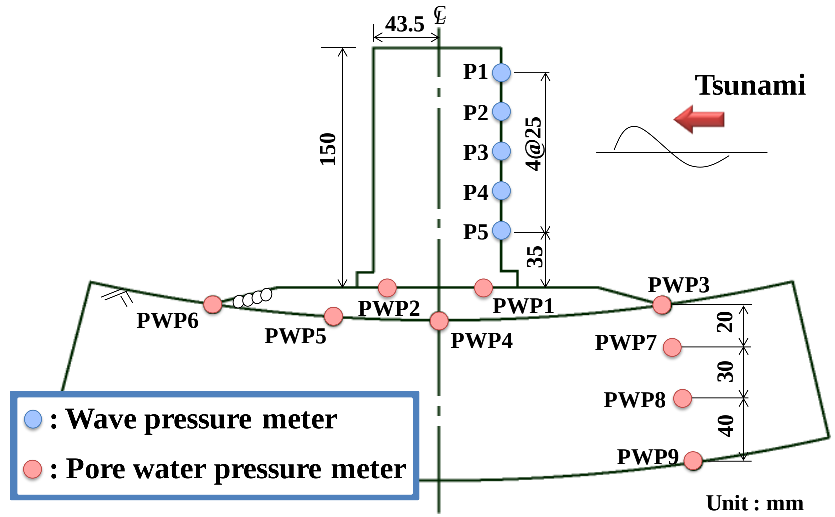


High-speed flow is caused in the rubble mound.

As a result, seepage flow was generated on the seabed soil surface.

Hydraulic gradient into seabed soil





$$i_{ij} = \frac{\left(\dfrac{P_i - P_j}{\rho^f g}\right) + \left(H_{ei} - H_{ej}\right)}{\Delta_{ij}}$$

ρ^f : Density of fluid

g : Acceleration of gravity

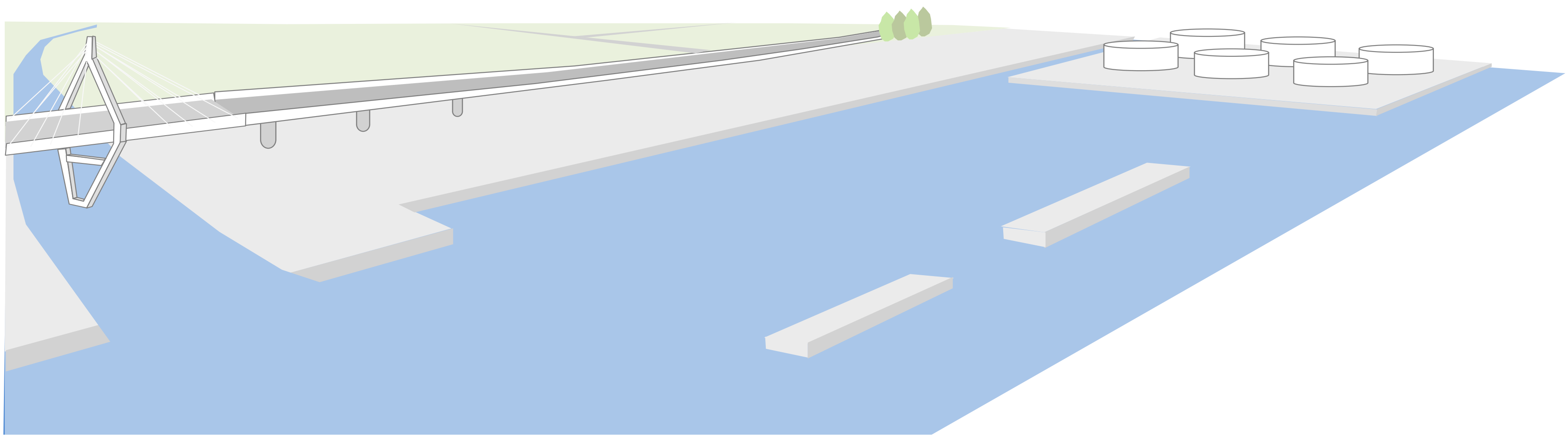
P_i : Total head of measurement point i

H_{ei} : Elevation head of measurement point i

Δ_{ij} : Distance of measurement point i and j

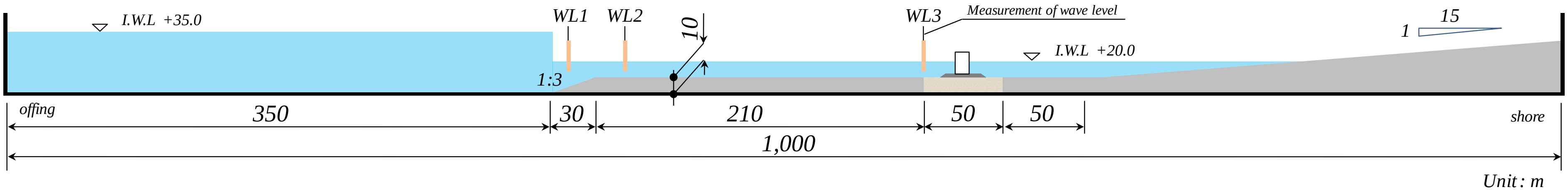
In the outcome of an experiment and the analytical result, the value is different. Behavior looks like.

Tsunami simulation
in virtual coastal area

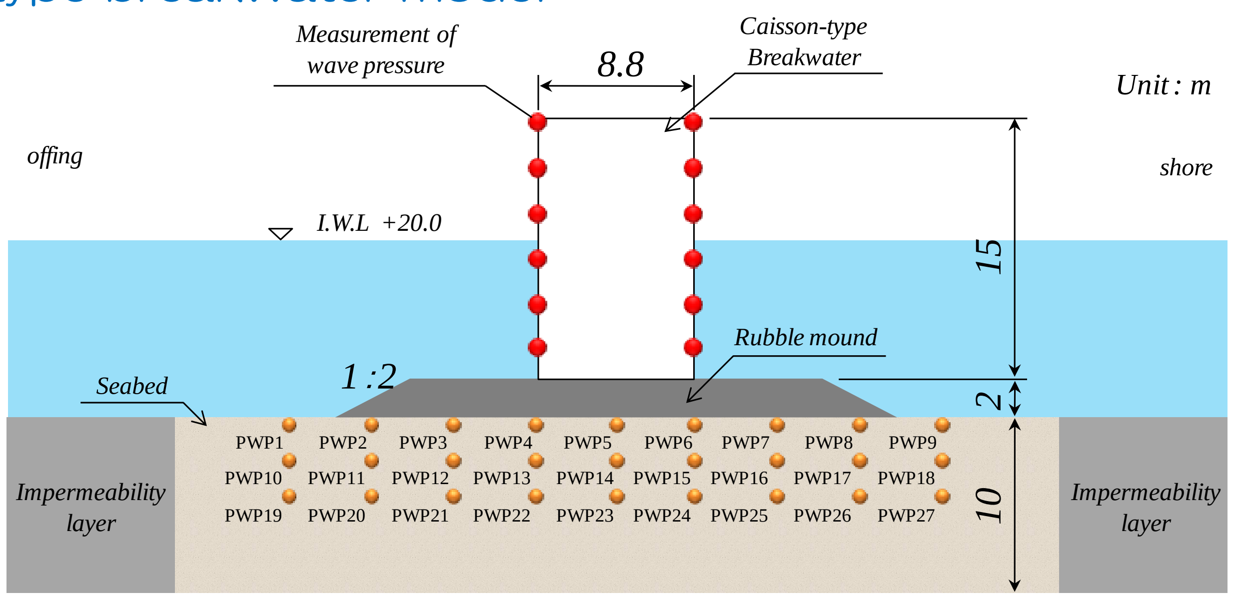


Tsunami simulation in virtual coastal area

Marine model



Caisson-type breakwater model



Stability of breakwater due to tsunami was investigated using a standard model of coastal area. A tsunami was generated by dam break. The permeability coefficients of the rubble mound and the seabed soil were set as 1×10^{-2} m/s and 2×10^{-5} m/s.

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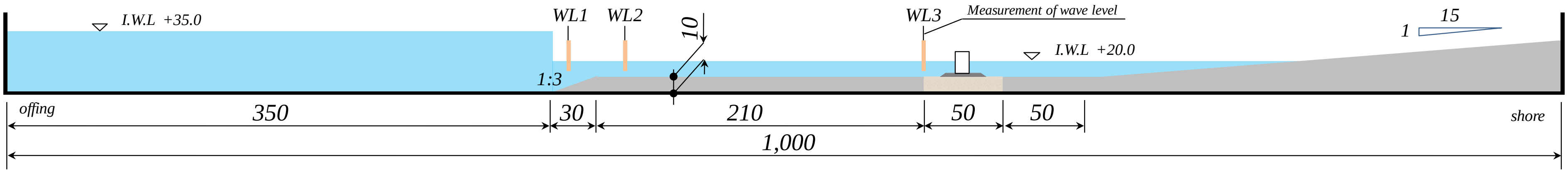
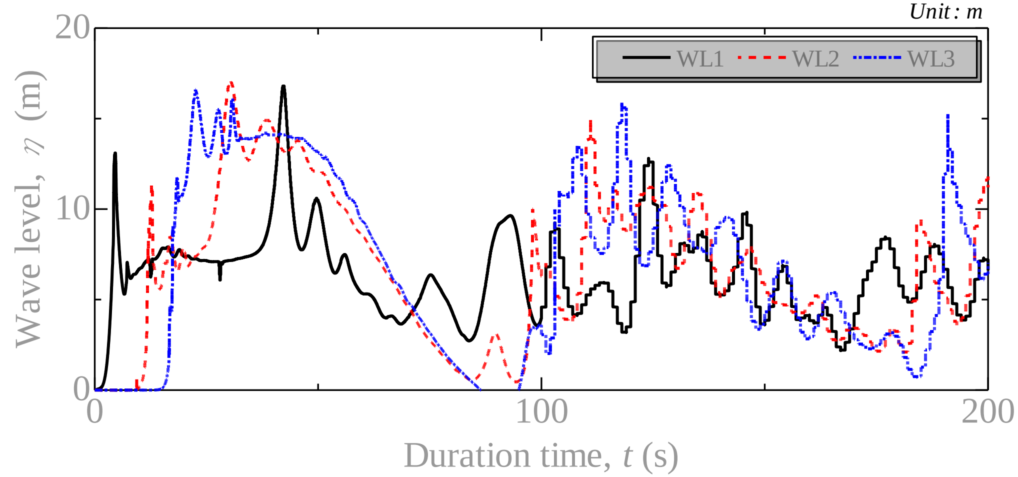
Tsunami simulation in virtual coastal area



	Height of incident wave	Height of overlapping wave
WL1	7.17 *m*	16.8 *m*
WL2		17.1 *m*
WL3	–	16.6 *m*

Wave period: About 70 [s]



Soliton wave was confirmed for the first time in middle Japan Sea Earthquake in 1983.

The feature of soliton wave is as follows;

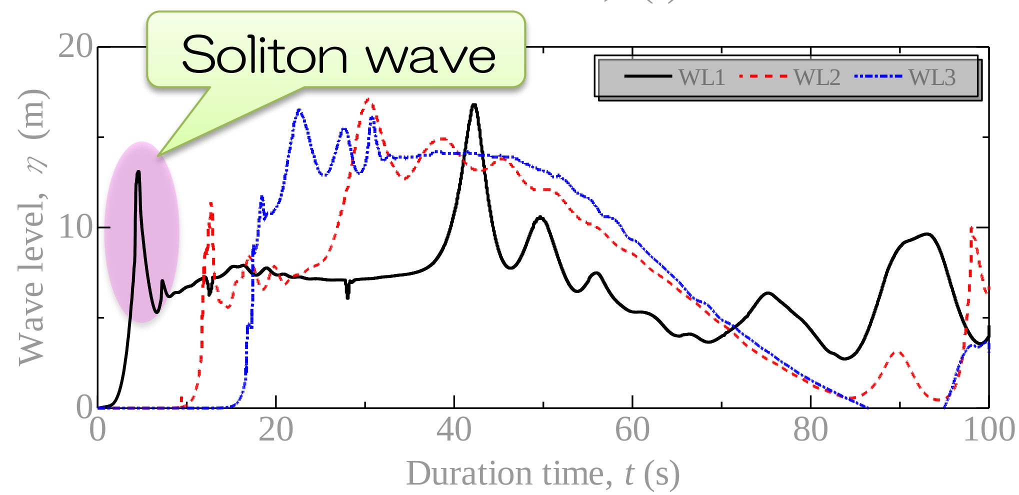
Wave for about ten a few seconds of cycle

↓

Breaking wave

↓

wave force is very large



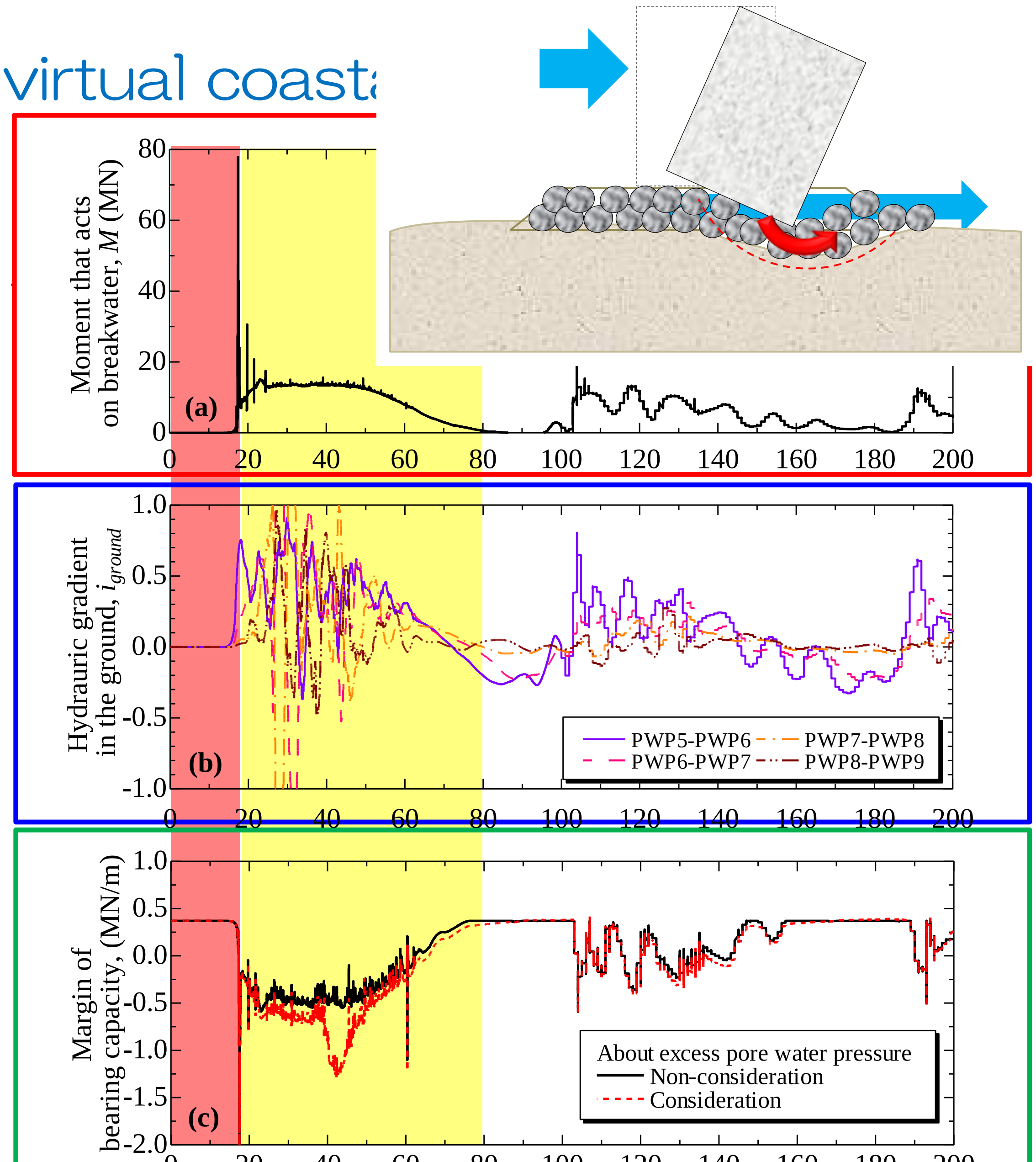
Tsunami simulation in virtual coasta

Stability of the breakwater against anaseism

The breakwater is moved due to the action of the initial impulsive wave force.

Shear deformation occurred in the rubble mound and the seabed soil and it receive seepage force, which decreased the stability. Especially, the decrease in the safety factor of the breakwater was larger when the excess pore water pressure in the ground was taken into consideration

The breakwater will be large deformation.

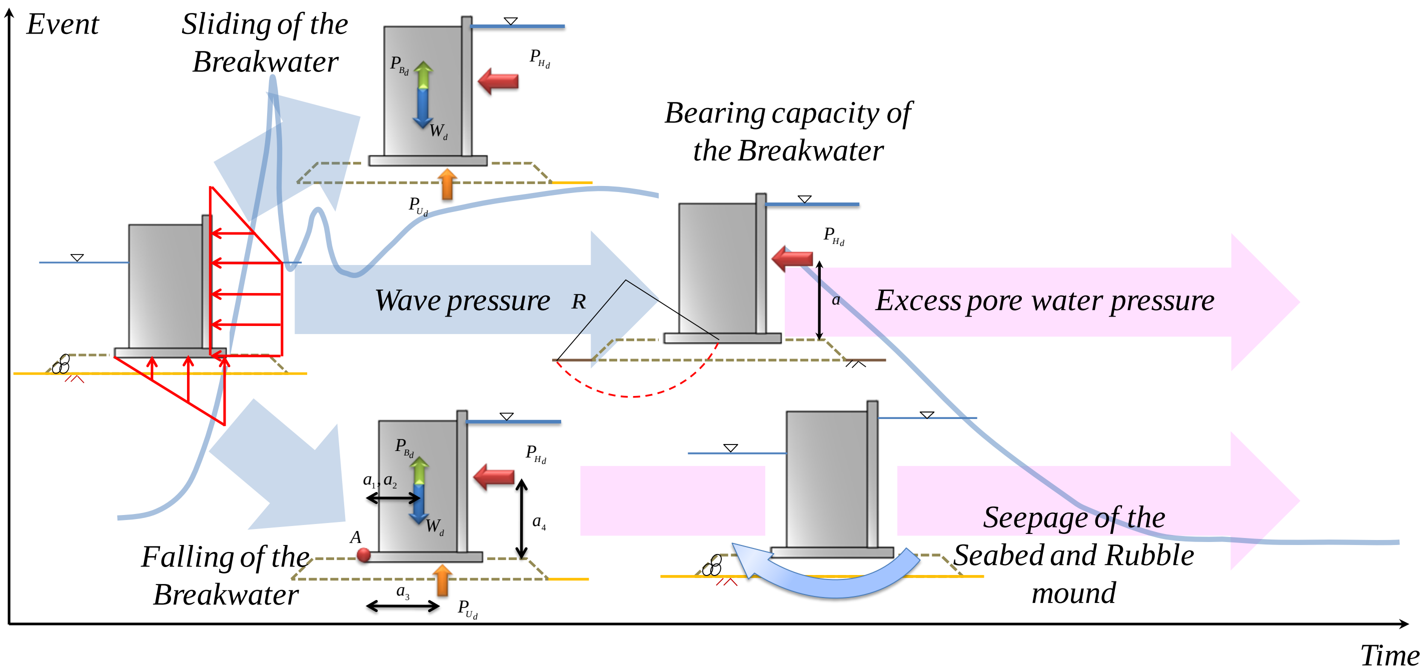


Margin of bearing capacity = **Bearing capacity strength** − **Tsunami force** − **Bearing capacity decrease in breakwater due to increase of excess pore water pressure in the ground**

Part. 4

Conclusion

Tsunami - Seabed soil - Breakwater interaction



Damage mechanism of breakwater by tsunami

The breakwater slid due to the action of the initial impulsive wave force.

Shear deformation occurred in the rubble mound and the seabed soil, which decreased the bearing capacity.

The hydraulic gradient increased in rubble mound and seabed soil at the shore side under breakwater due to seepage flow with the continuous wave pressure.

The bearing capacity of breakwater decreased due to degradation of the ground caused due to the increment of excess pore water pressure in the seabed soil.

Thank you
for your kind attention

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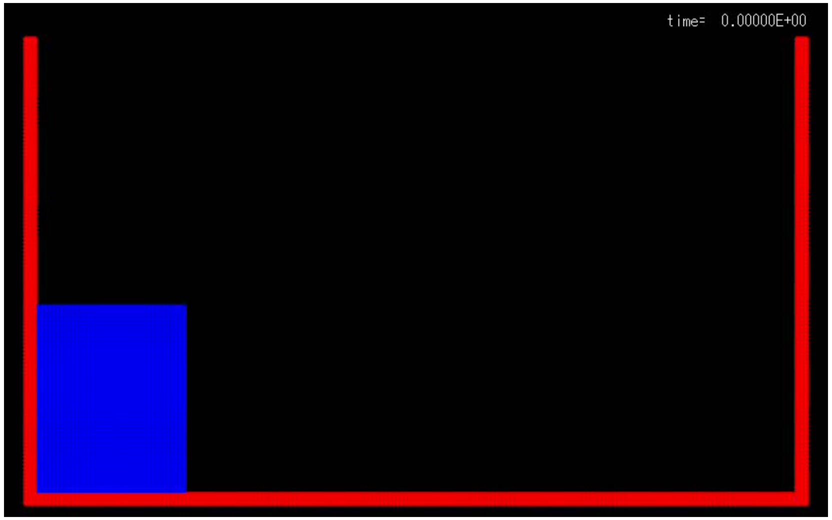
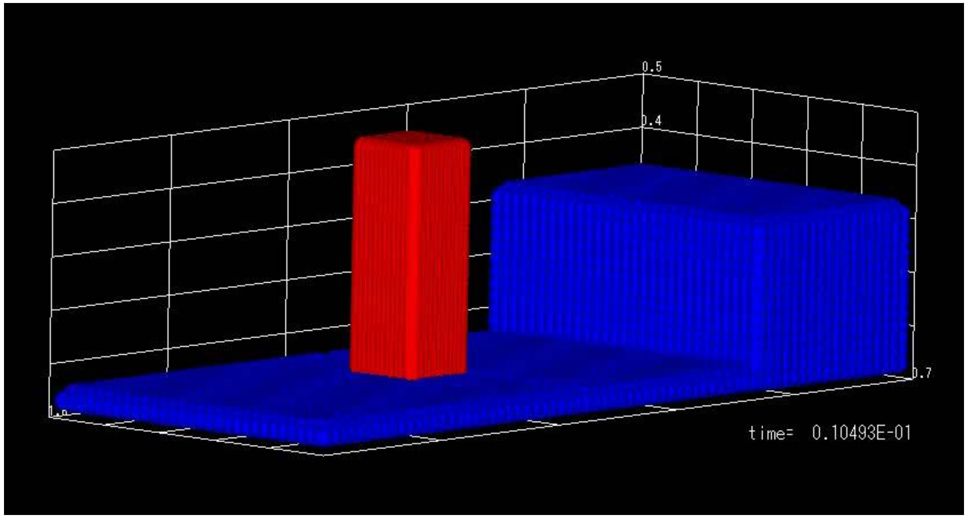
Fluid, Fluid-Solid coupling

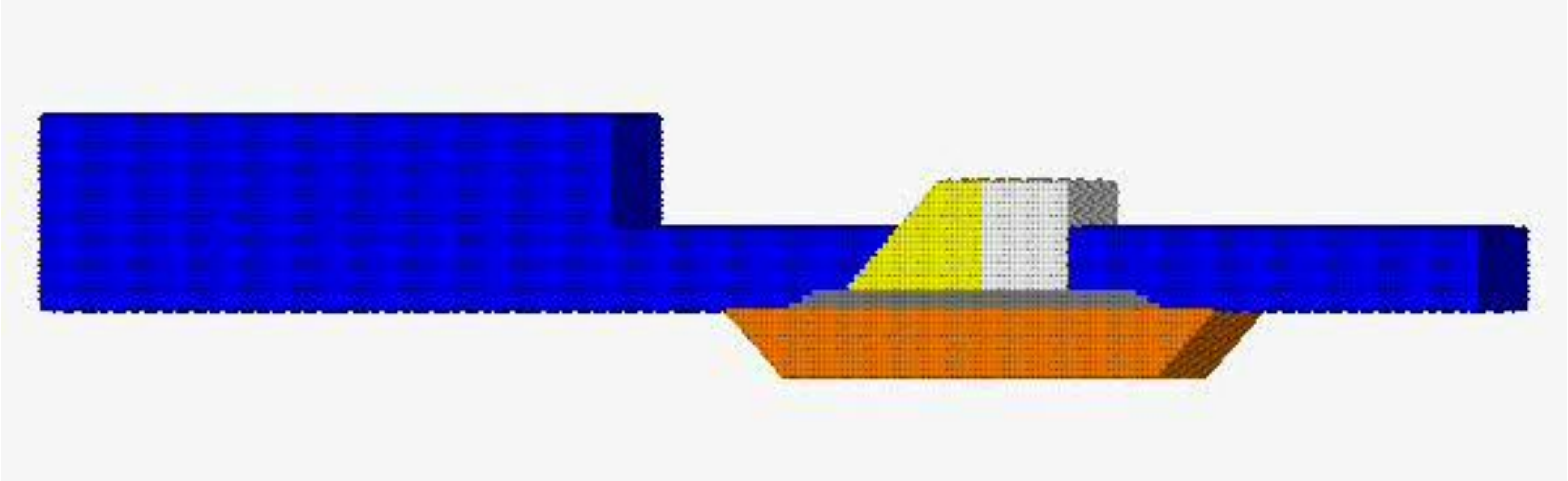
Movie (Click fig.)



2D-Dambreak



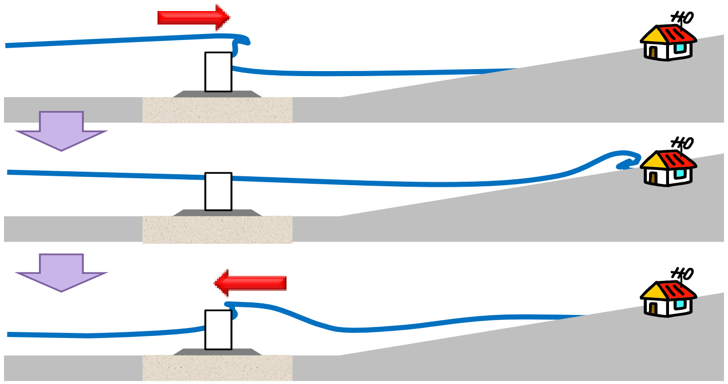
3D-Dambreak

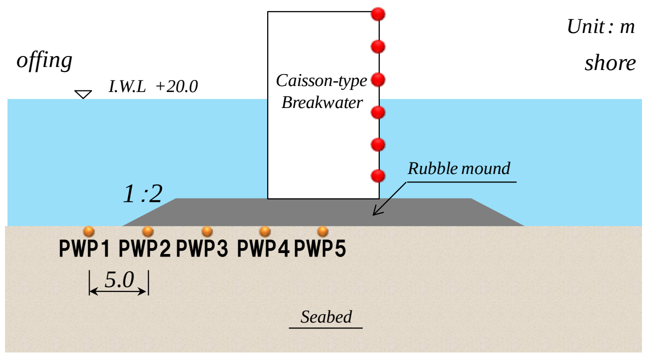


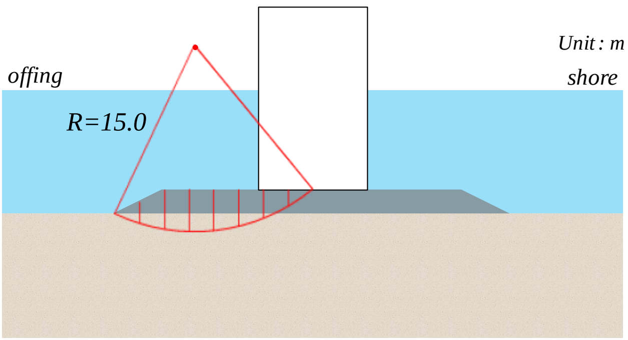
Tsunami hazard simulation (Hachinohe port)

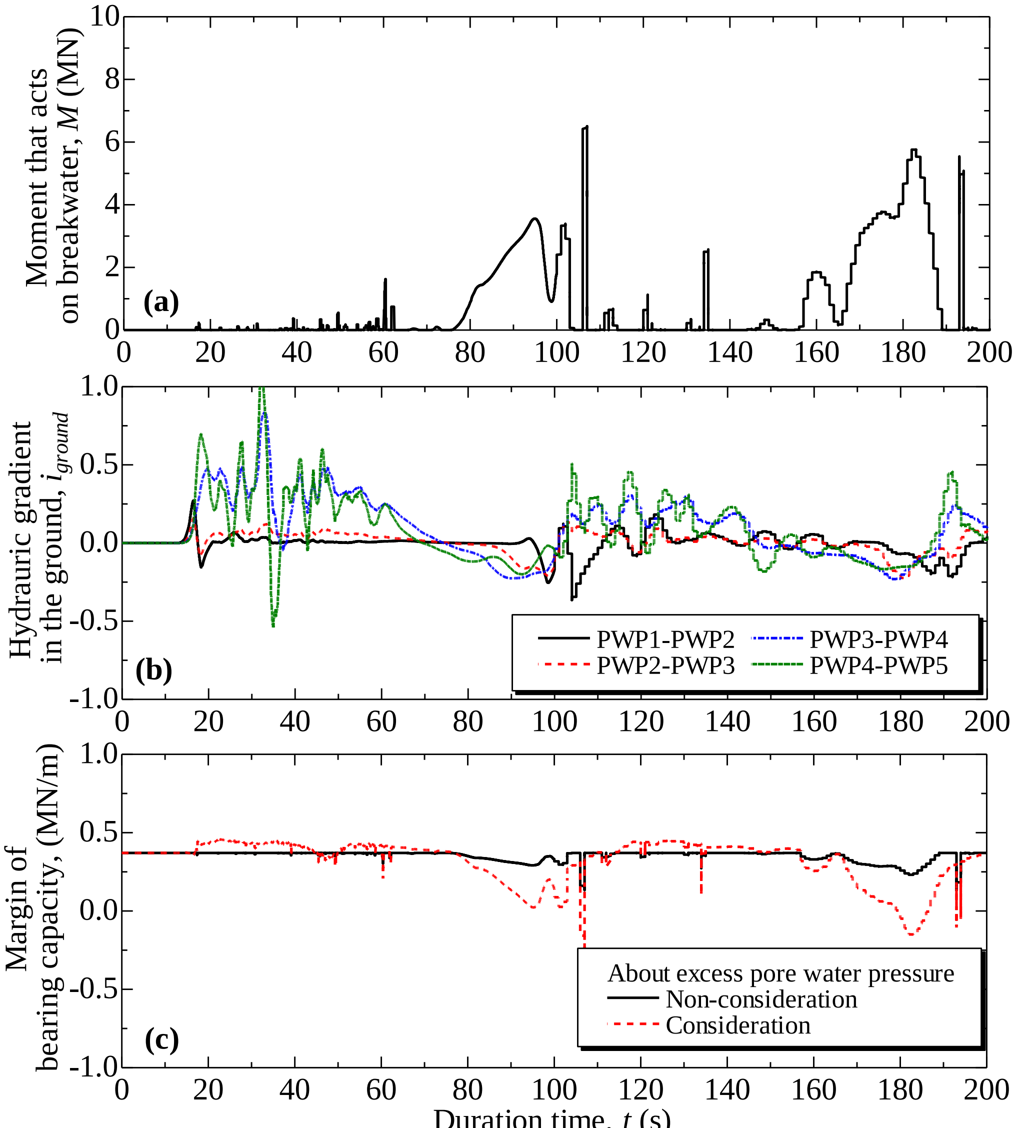
Tsunami simulation in virtual coastal area

Stability of the breakwater against backrush



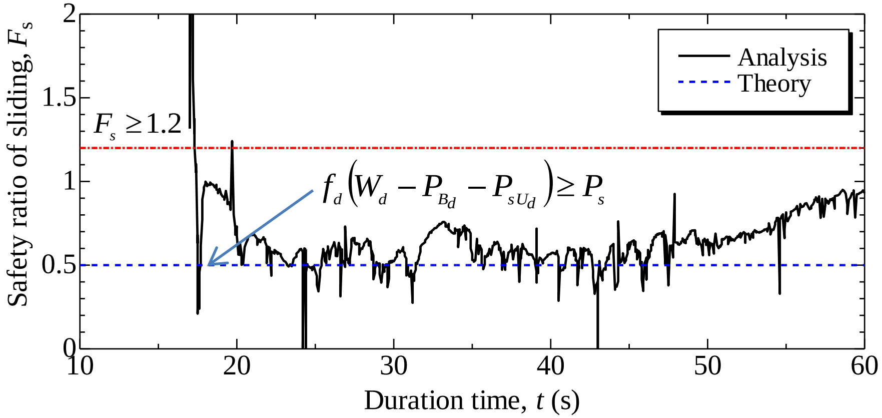




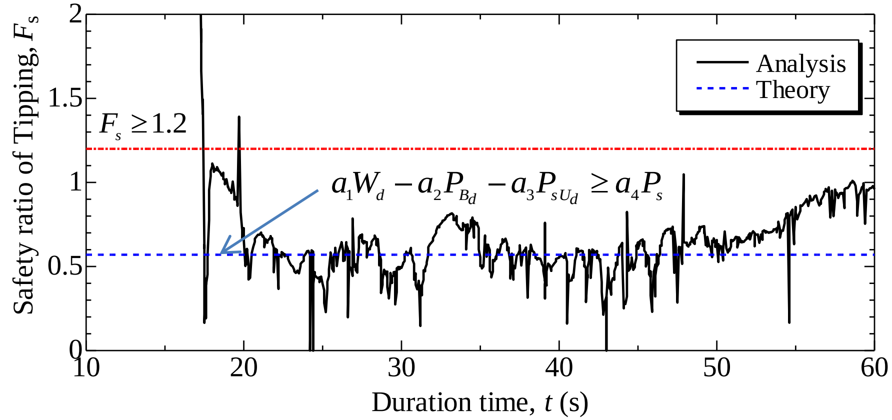


Margin of bearing capacity = Bearing capacity strength − Tsunami force − Bearing capacity decrease in breakwater due to increase of excess pore water pressure in the ground

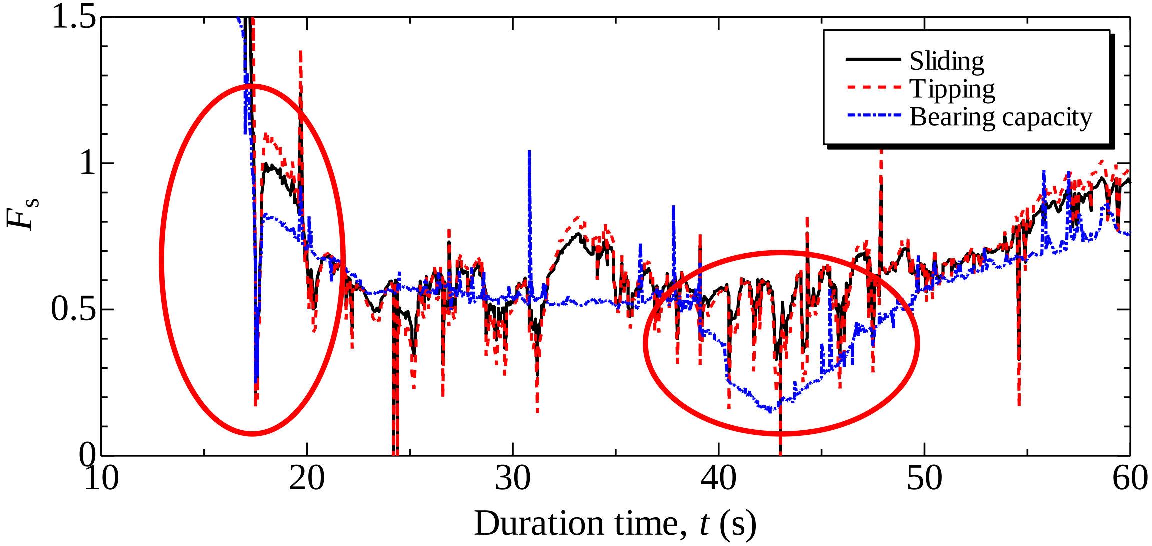
Tsunami simulation in virtual coastal area

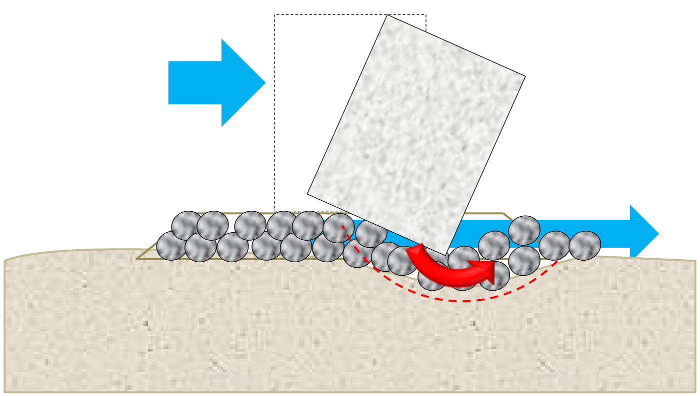


Safety ratio of sliding



Safety ratio of tipping





Safety ratio of sliding, tipping and bearing capacity

八戸港における津波来襲時の様子

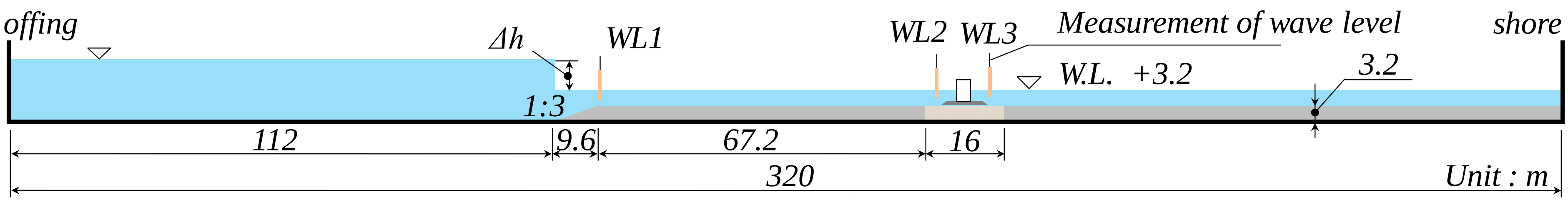
津波越流力による
防波堤背後地盤の不安定化

出典: You Tube

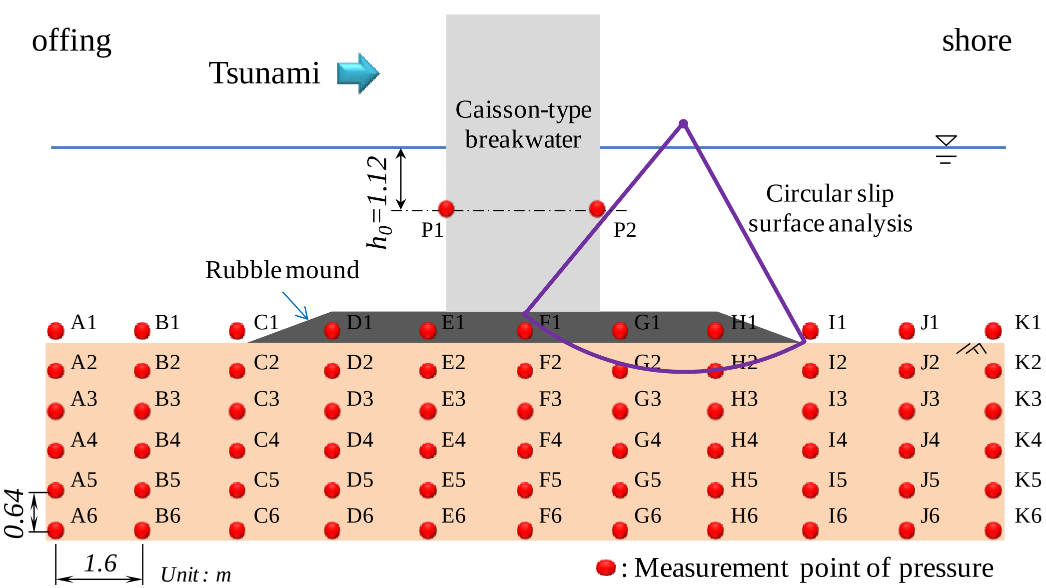
津波流動場を想定した海岸域のモデル化

がんばろう日本

Case	Offing site water level	shore site water level	Δh
Non-overflow	7.36 (m)	3.20 (m) + 3.20(m): Seabed	+ 0.96 (m)
overflow	12.80(m)	3.20 (m) + 3.20(m): Seabed	+ 6.40 (m)



全 域



混成堤モデル

防波堤：不動剛体構造物

捨石マウンド：不動透過性構造物
透水係数$k_m = 1.0 \times 10^{-2}$m/s
（Dupuit－Forchheimer則）

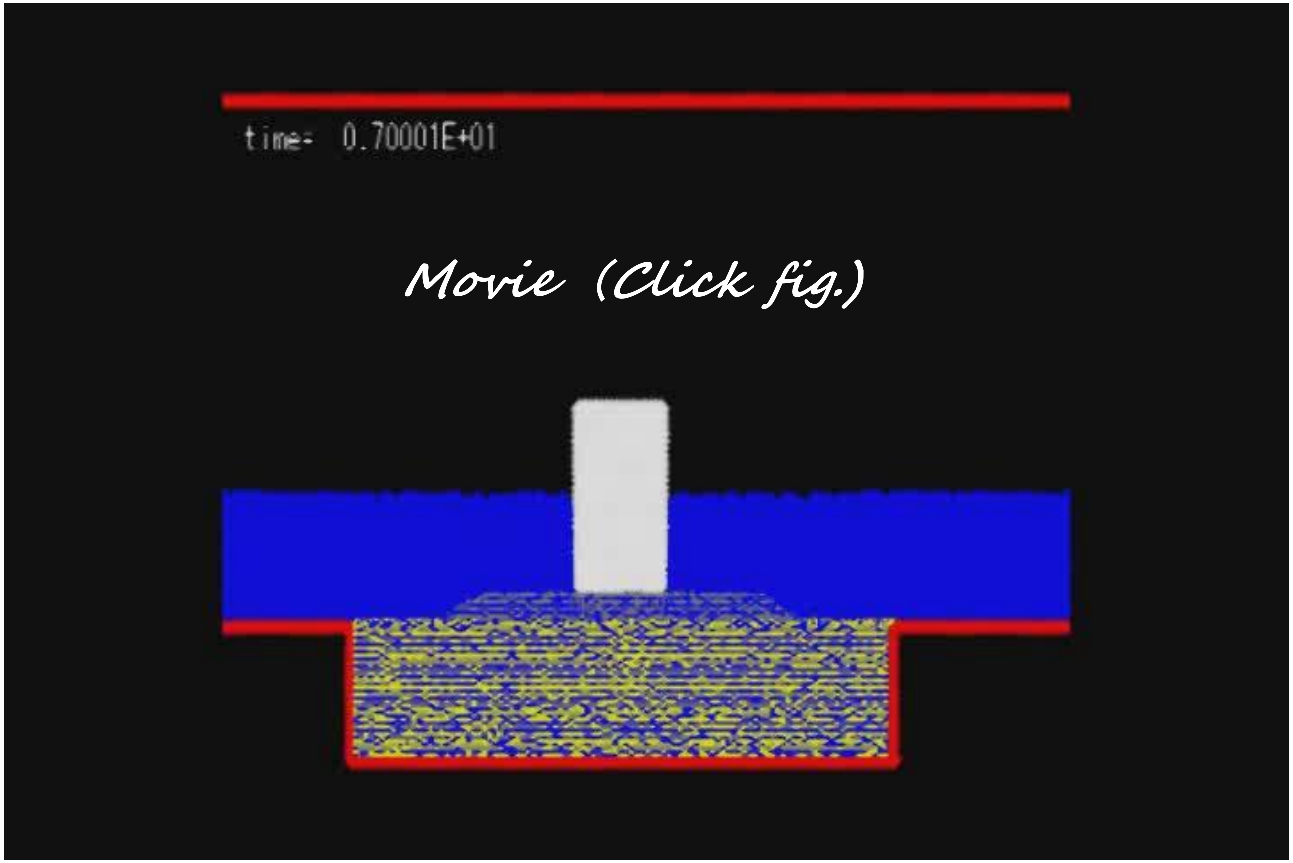
海底地盤：不動透過性構造物
透水係数$k_s = 2.0 \times 10^{-5}$m/s
（Darcy則）

海底床：不透水性（境界）

初期粒子間距離：0.16m

粒子数：10万個程度

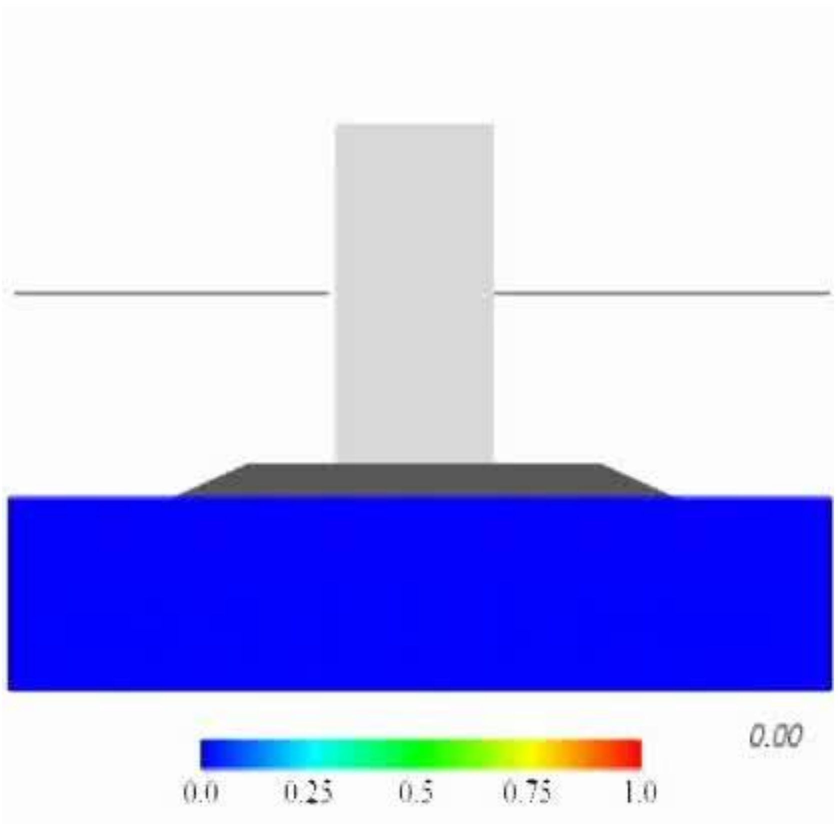
防波堤越流時の津波挙動



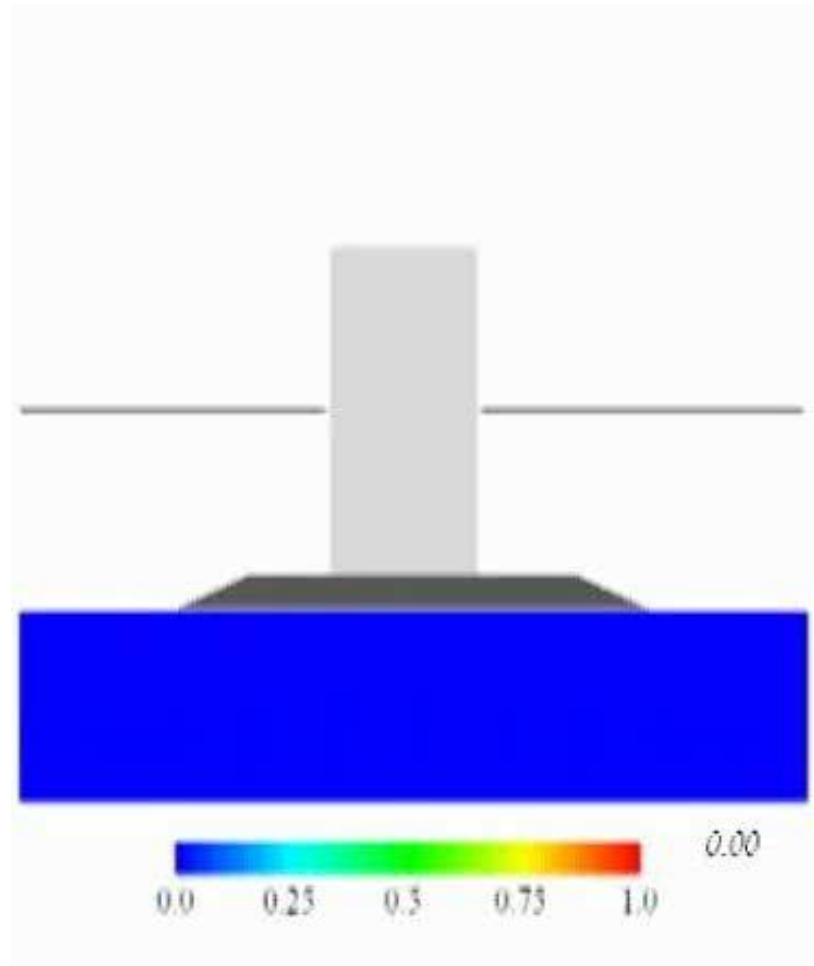
越流あり：Δh=+6.4mにおける解析結果

越流後の落下水塊による地盤内応力変化

-動水勾配の経時変化に着目した検討-



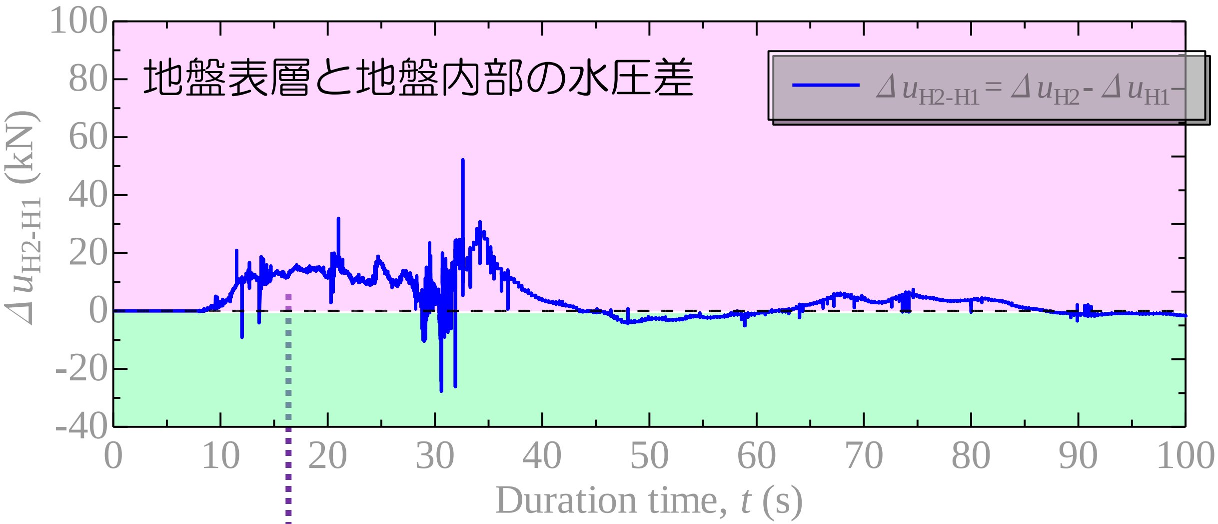
越流なし

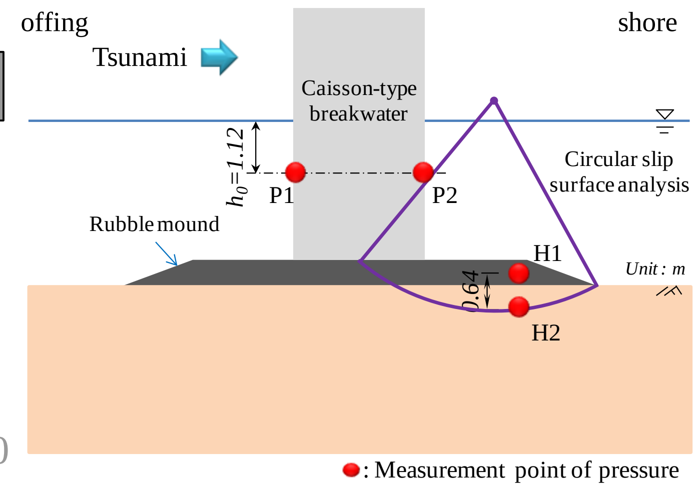


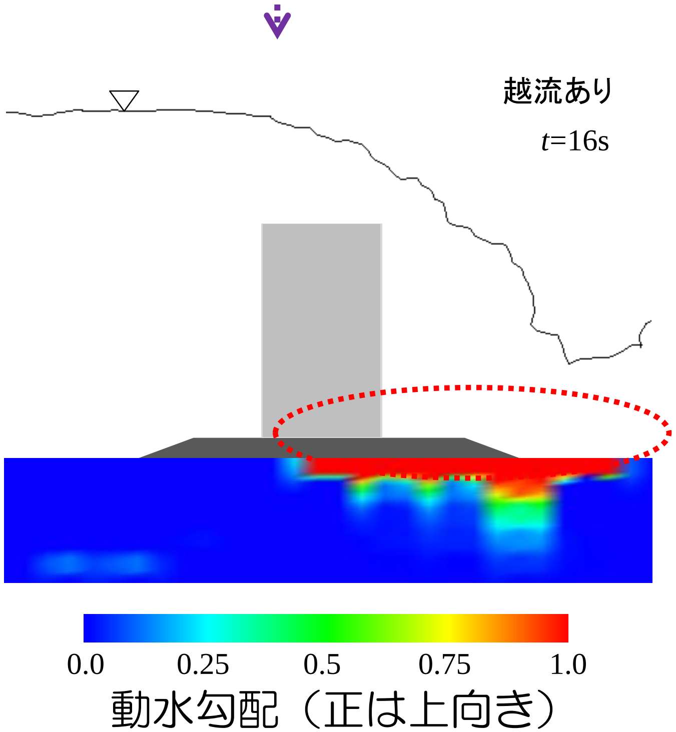
越流あり

がんばろう日本

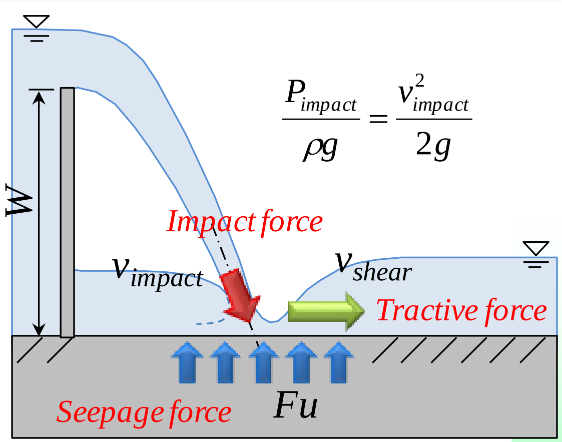
防波堤背後地盤に作用する落下水塊の影響





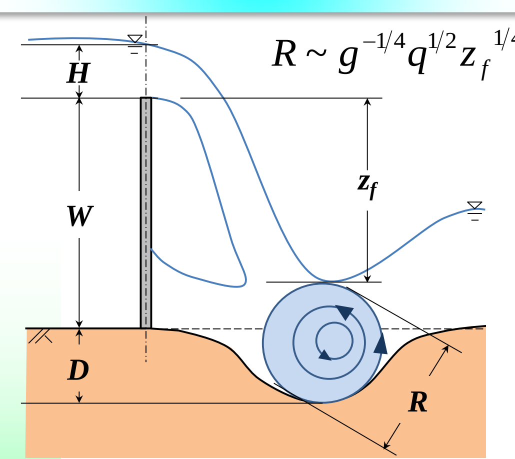


越流水塊による急速載荷



土木学会刊(1999), 水理公式集　平成11年度版.

乱流・渦に伴う乱れ

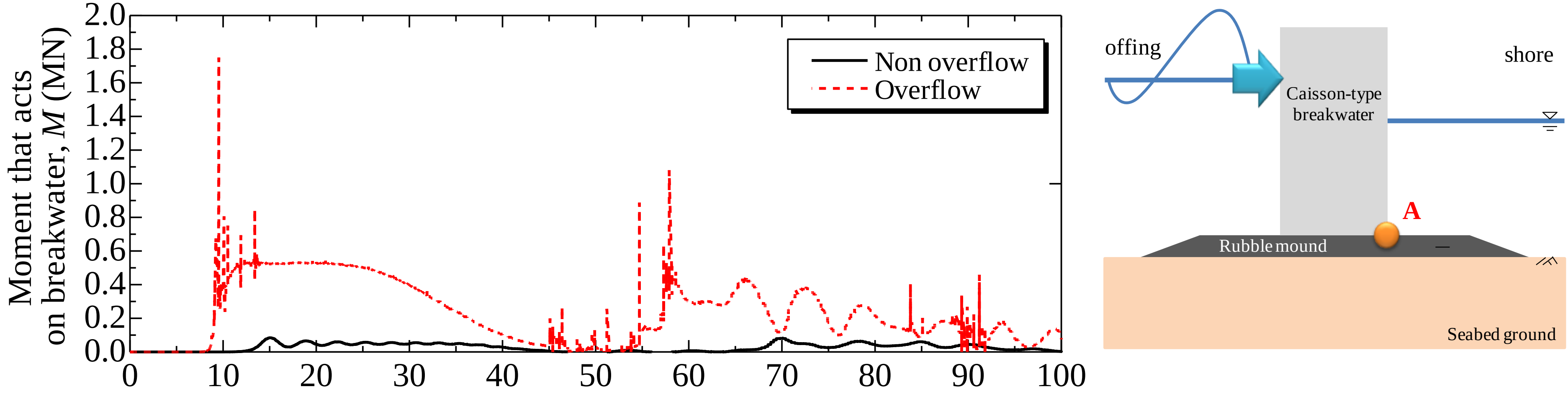


野口 他(1997): 津波遡上による護岸越流および前面洗掘の大規模模型実験，海工論，第44巻，pp.296-300

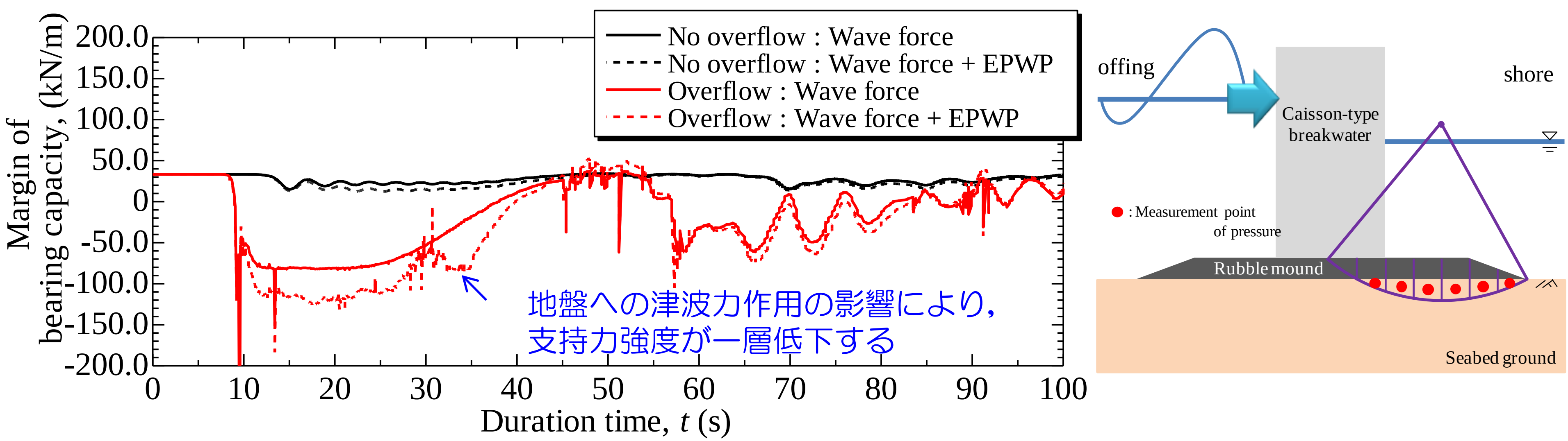
過剰間隙水圧，浸透力による土粒子の浮遊（液状化）

がんばろう日本

地盤の剛性低下を考慮した支持力破壊の検討



(a) 防波堤に作用する総モーメント力

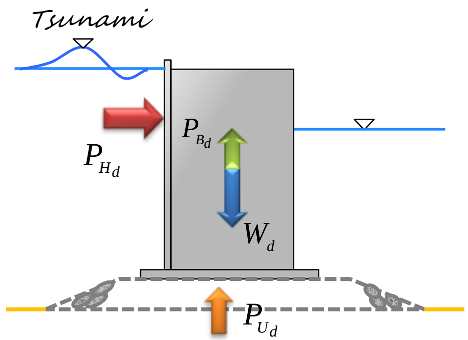


(b) 支持力に対する安全率(地盤内過剰間隙水圧の考慮)

防波堤背後の水圧変動に伴う 防波堤の滑動に対する安全性低下

滑動に対する安全率

$$F_S=\frac{f_d\left(W_d-P_{B_d}-P_{U_d}\right)}{P_{H_d}}$$



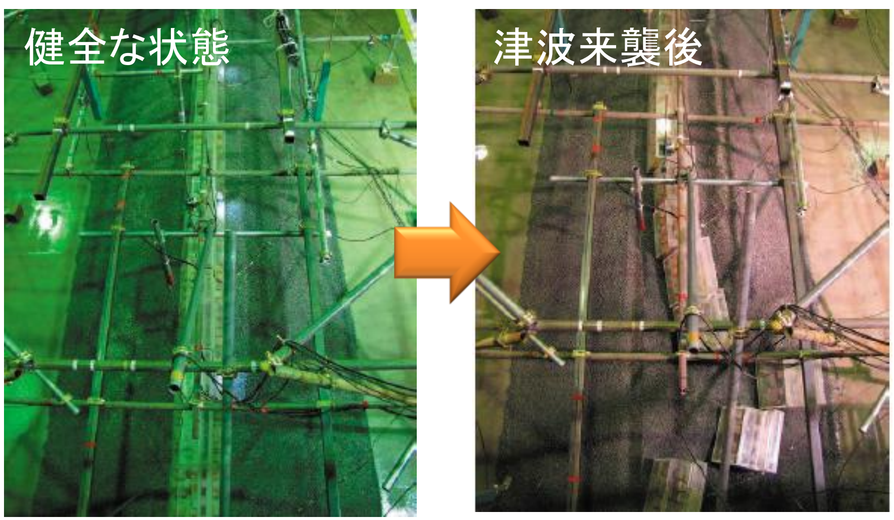
W_d ：堤体重量（kN/m）　　P_{H_d} ：水平波力（kN/m）

P_{B_d} ：浮力（kN/m）　　a_i ：力の作用するアーム長(m)

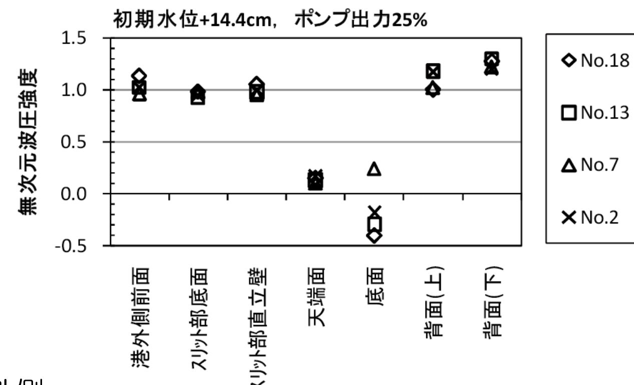
P_{U_d} ：揚圧力（kN/m）　　f_d ：壁体底面と基礎との摩擦（=0.6）

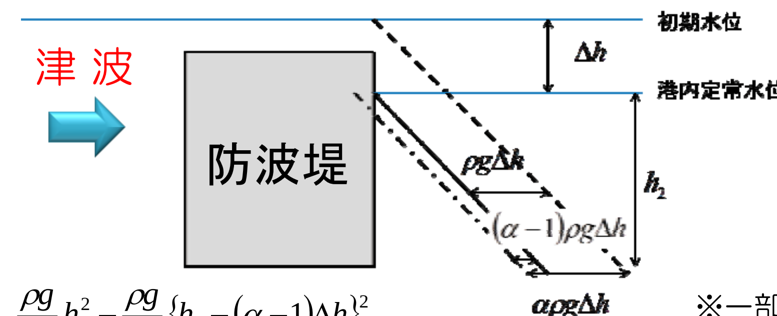
※（社）日本港湾協会：港湾の施設の技術上の基準・同解説（上）・（下），2007

有川ら（2012）による釜石湾口防波堤の被災検討



有川太郎・佐藤昌治・下迫健一郎・富田孝史・辰巳大介・廉慶善・高橋研也(2012): 釜石湾口防波堤の津波による被災メカニズムの検討－水理特性を中心とした第一報－, 港空研資料, No.1251.

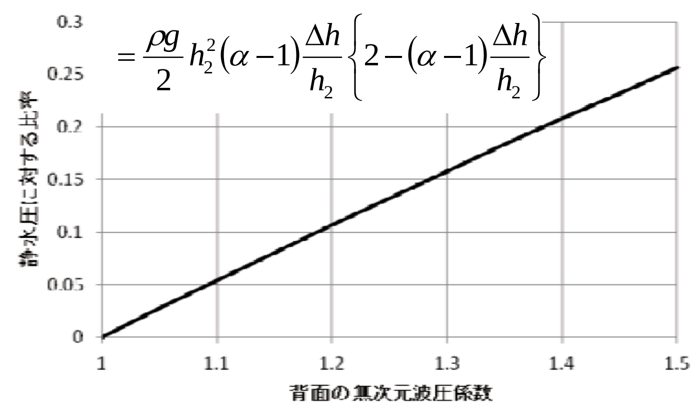




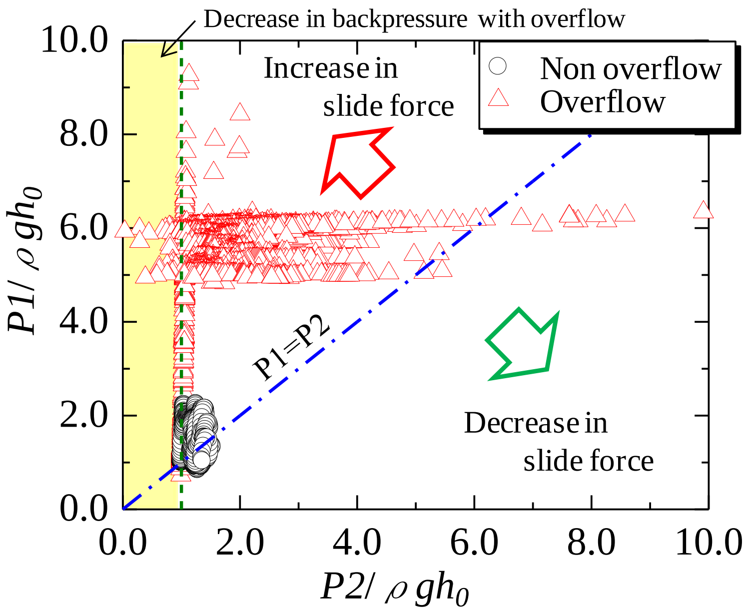
※一部加筆

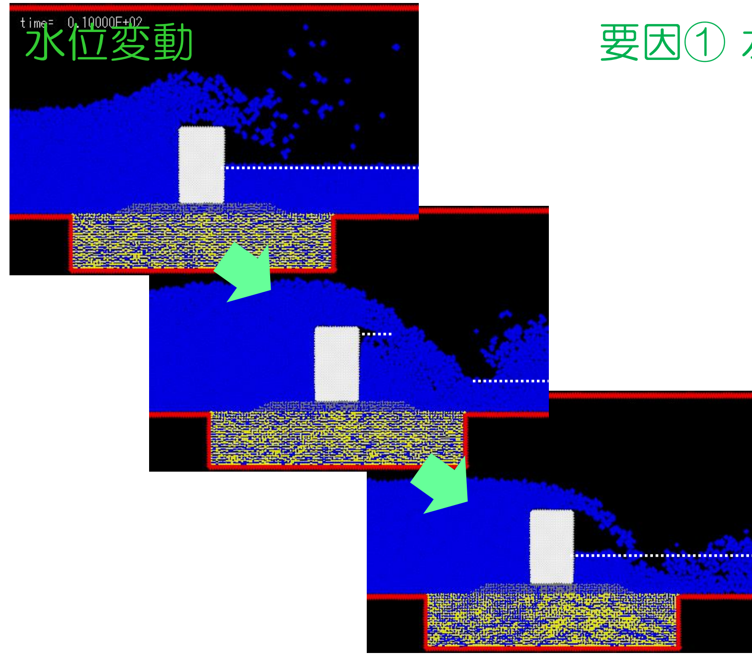
$$\frac{\rho g}{2}h_2^2-\frac{\rho g}{2}\left\{h_2-(\alpha-1)\Delta h\right\}^2$$

$$=\frac{\rho g}{2}h_2^2(\alpha-1)\frac{\Delta h}{h_2}\left\{2-(\alpha-1)\frac{\Delta h}{h_2}\right\}$$

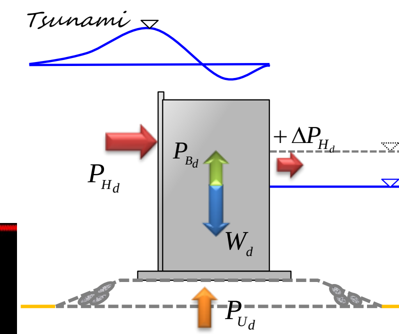


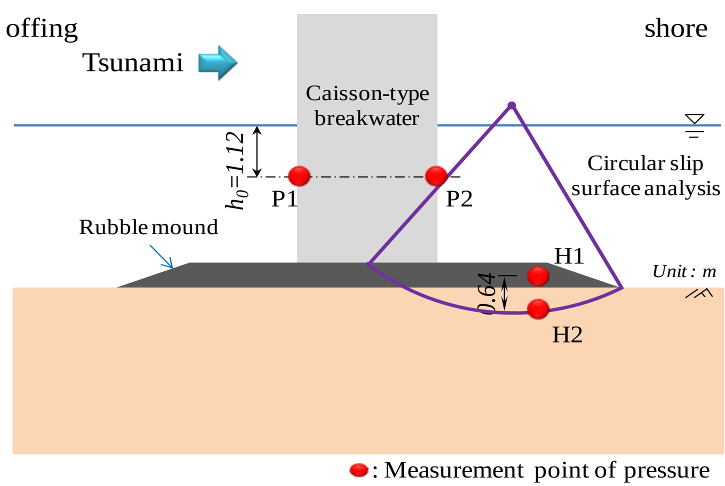
防波堤背後の水圧変動に伴う
防波堤の滑動に対する安全性低下

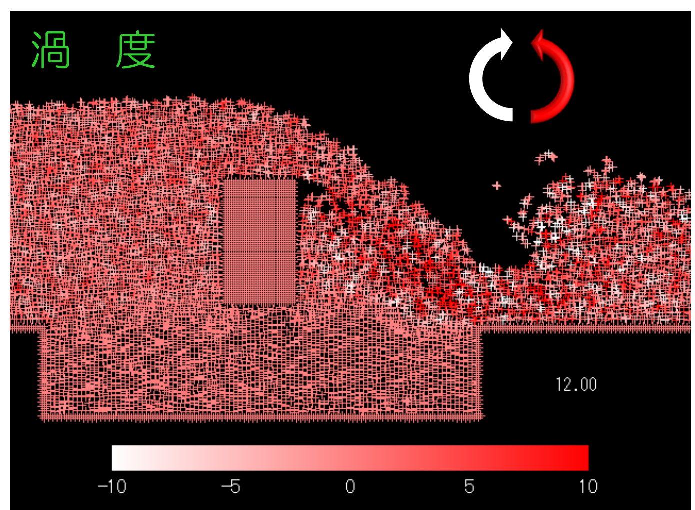




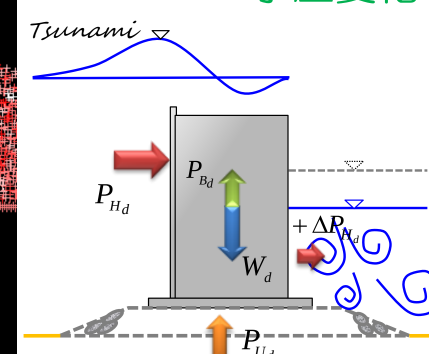
要因① 水位変動に伴う
水圧変化





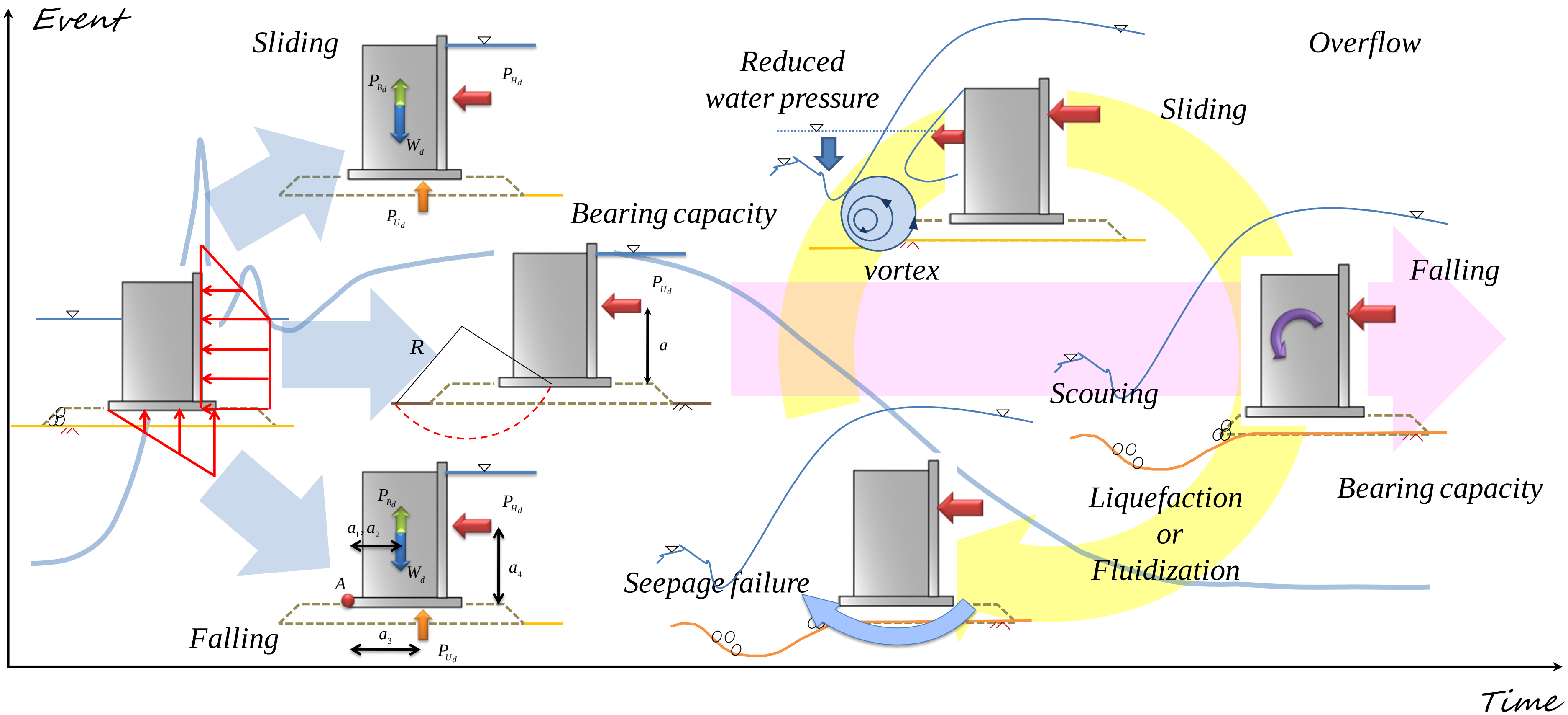


要因② 渦等伴う
水圧変化



がんばろう日本

結　言



越流による落下水塊による背後地盤への影響と防波堤の不安定化

落下水塊の衝突力とその後のせん断流

乱流・渦に伴う乱れと圧力低下

地盤の洗掘・局所的液状化を誘発し、支持力強度を低下させる

滑動に対する安全性を低下させ、支持力破壊と相まって防波堤が移動・転倒

今後に対策として

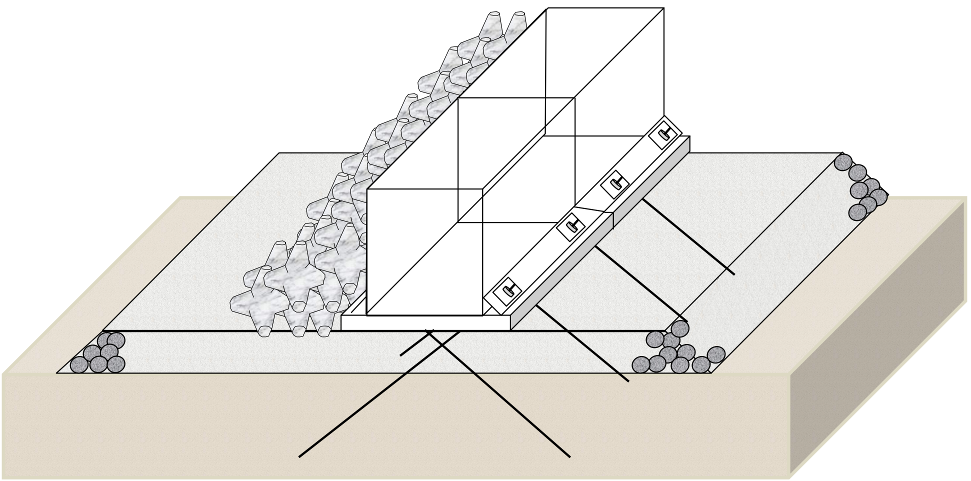
何度襲来する津波や引き波に対して

粘り強い構造形式

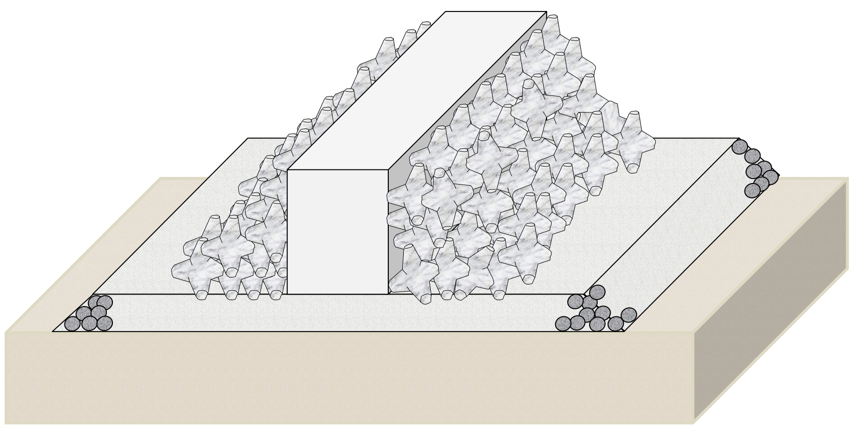
我が国において

重力式防波堤

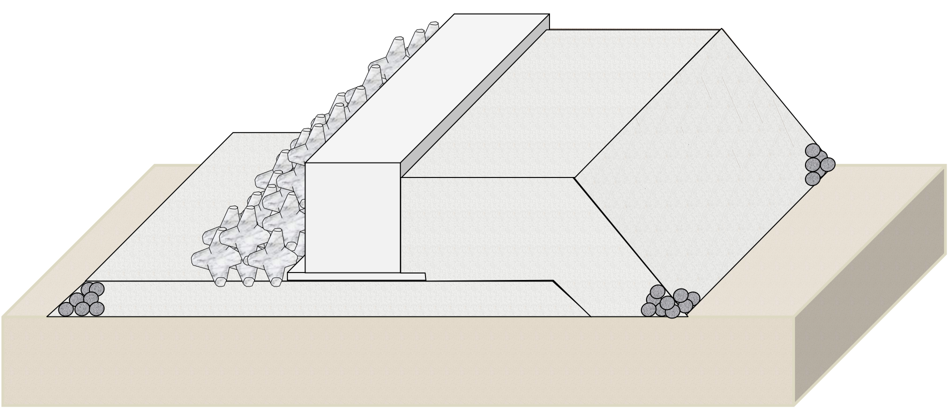
が多い



アンカーを用いた防波堤の固定による耐波強化



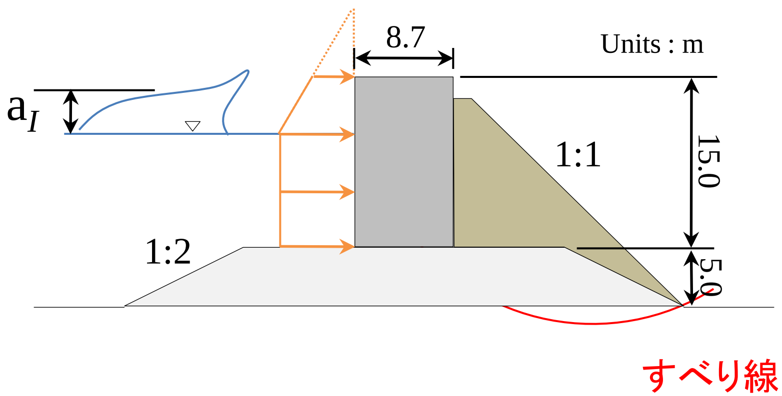
消波ブロックによる耐波強化



裏込カウンターによる耐波強化

がんばろう日本

支持力補強（裏込め）



	防波堤	捨石マウンド，割石	海底地盤
水中単位体積重量 γ'【kN/m³】	12.2	9.8	8.7
粘着力 c_D【kN/m²】	—	0	0
内部摩擦角 ϕ_D 【°】	—	35	45

補強なし：　$F_s = 1.08$

補強あり：　$F_s = 1.55$

安全率の上昇

滑動・転倒に対する効果も考えられる。

裏込め効果は十分期待できる

津波外力

：谷本ら(1984)

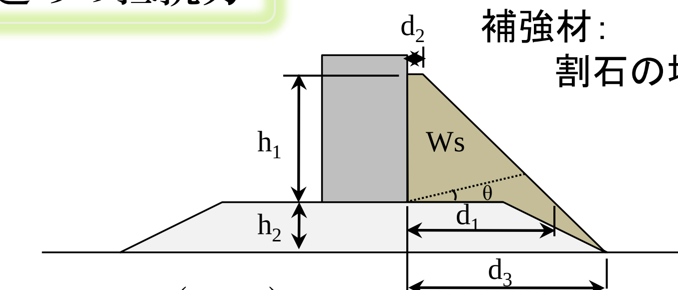
a_I=5.0m　　$P_p = 2.4\rho g a_I$

支持力に対する安全率

$$F_s = \frac{\sum\left\{\left[c_d S + \left(W'_d + q_d\right)\tan\phi_d\right]\frac{\sec\theta}{1+\tan\theta\tan\phi_d}\right\}}{\sum\left\{\left(W_d + q_d\right)\sin\theta + aP_{Hd}/R\right\}}$$

裏込めの抵抗力

補強材：
割石の場合



$$R = W_s \cdot \tan(\theta + \phi)$$

$$W_s = \frac{1}{2}\gamma'\left[\left(d_2 + d_4\right)\left(h_1 - d_4\tan\theta\right) + d_4 \cdot \left(d_4\tan\theta\right)\right]$$

W_s ：最上層の被覆層を除いた滑り面より上の割石の水中重量

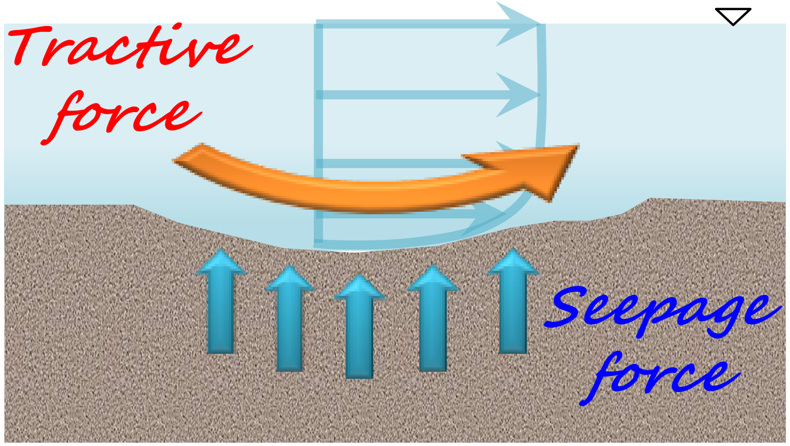
θ ：滑り面傾斜角（度）

ϕ ： $\tan^{-1}\mu_2$　　　μ_2 ：割石と割石の摩擦係数（=0.8）

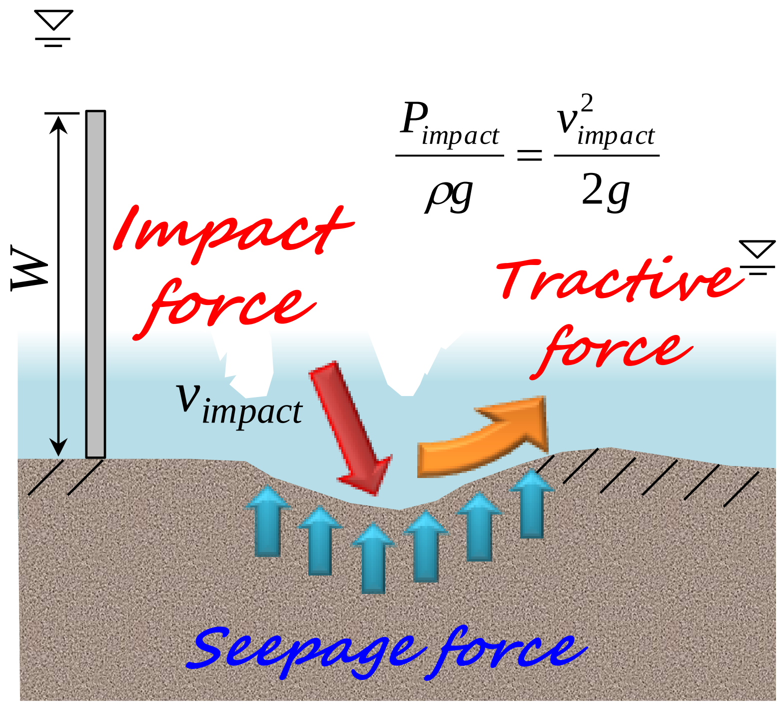
赤塚雄三,竹田英章,蓮見隆：混成堤の堤体背後に設置したコンクリート方塊あるいは割石の滑動抵抗,第22回海岸工学講演会論文集,Vol.2,pp.421-425,1975.

菊池喜昭,新舎博,河村健輔，江口信也：裏込めを有するケーソン式混成堤の安定性の検討,土木学会論文集C（地圏工学）,Vol.67, No.4, pp.474-487,2011.

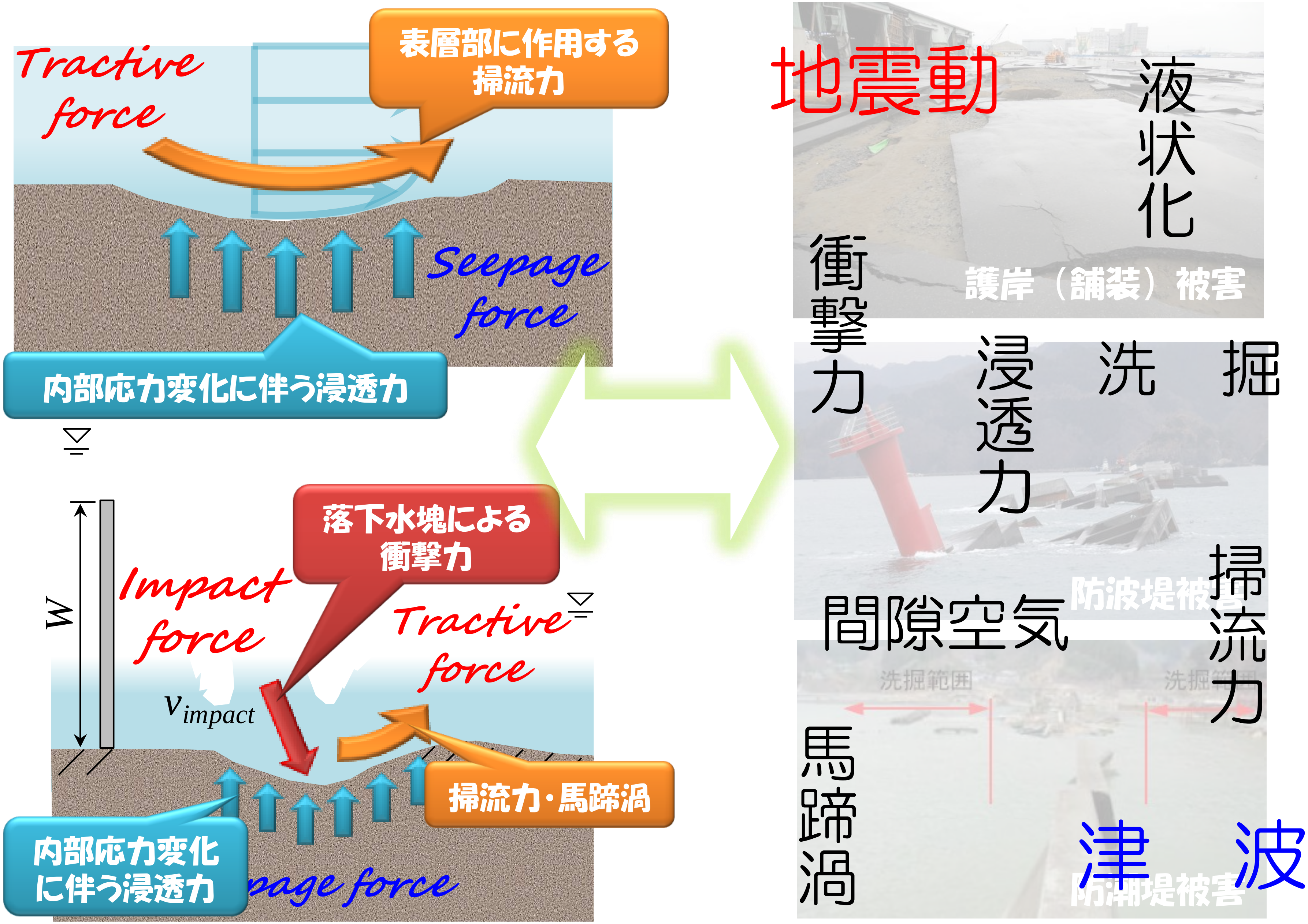
Scouring seabed surface

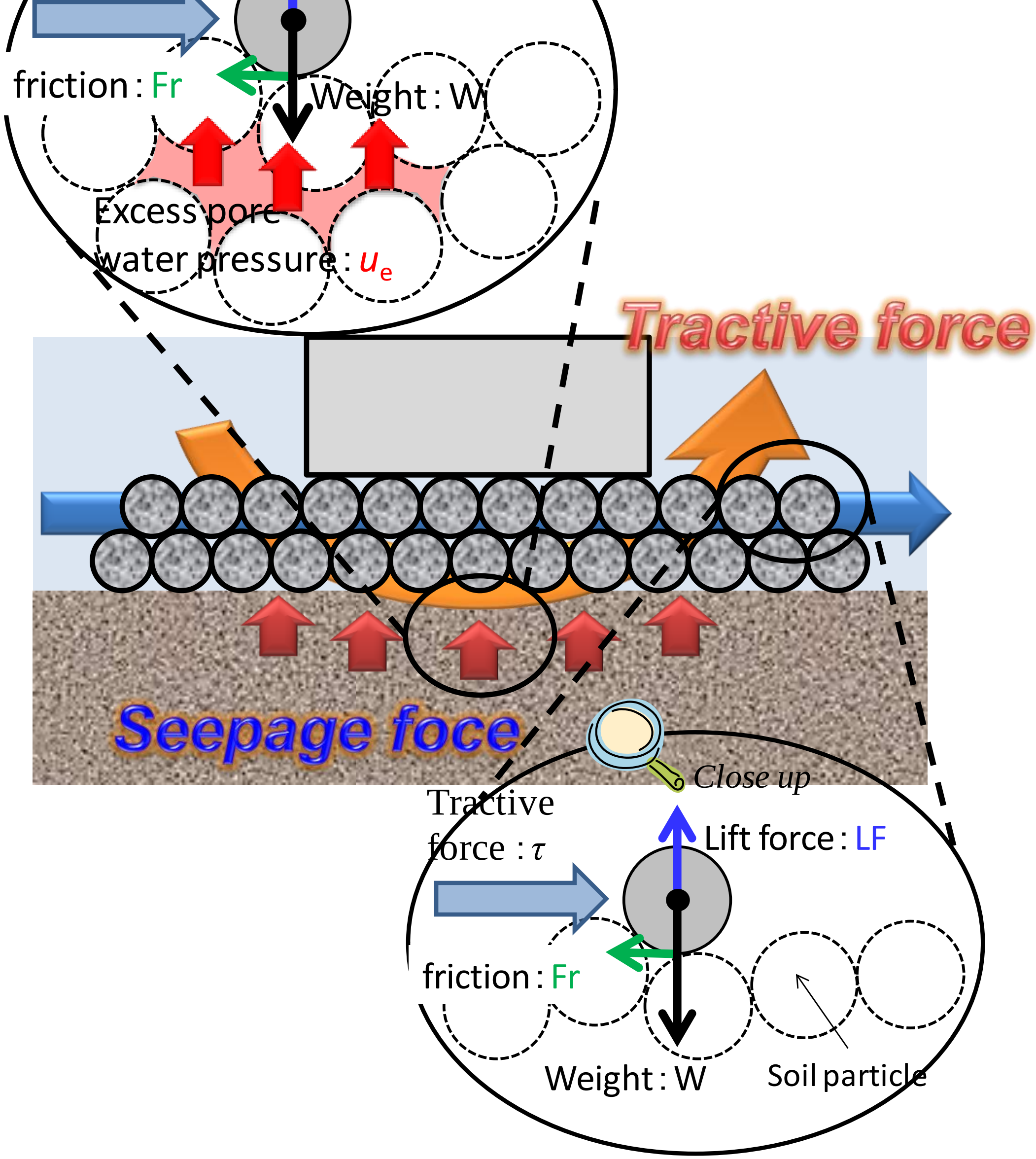


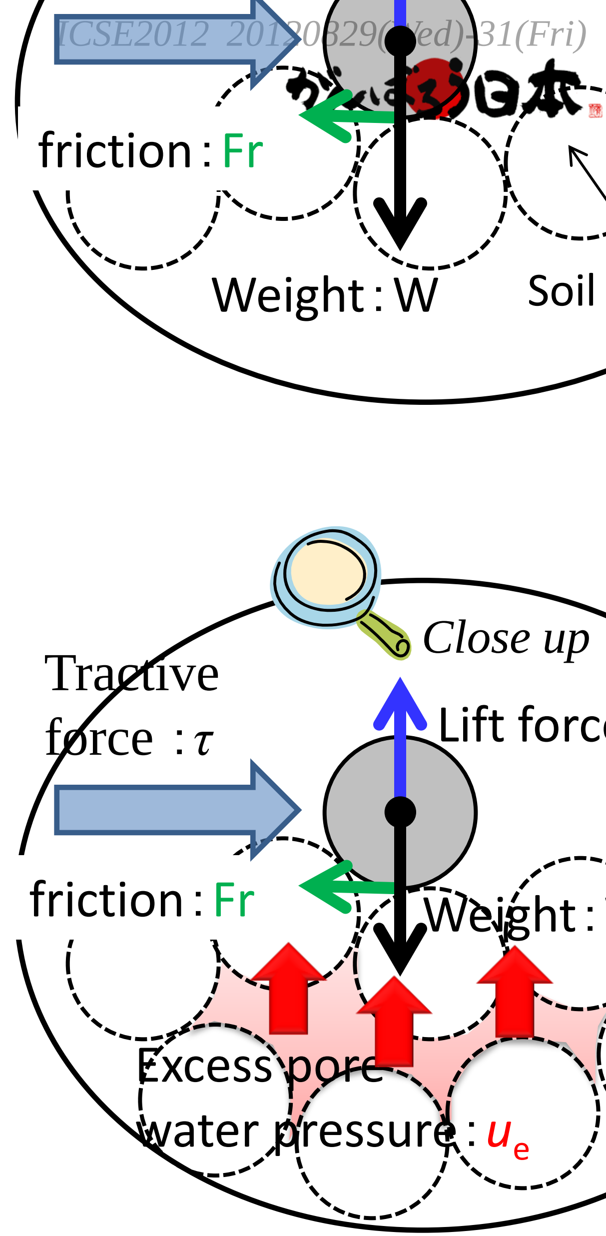
Overflow



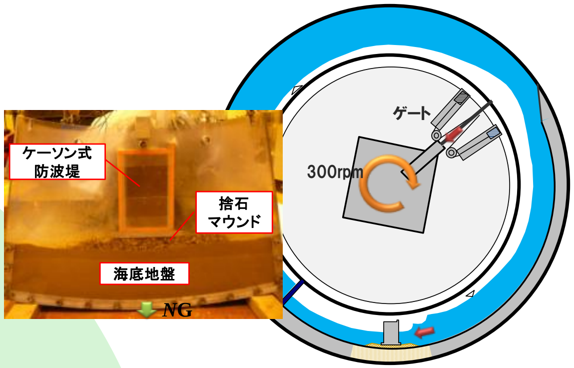
結言 ～其の2：地盤工学における新たな課題～

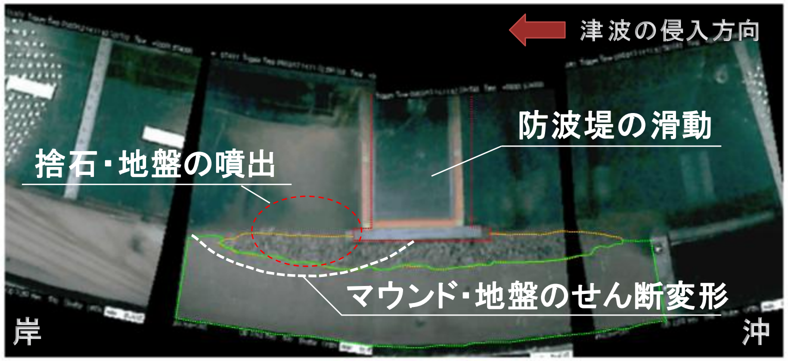


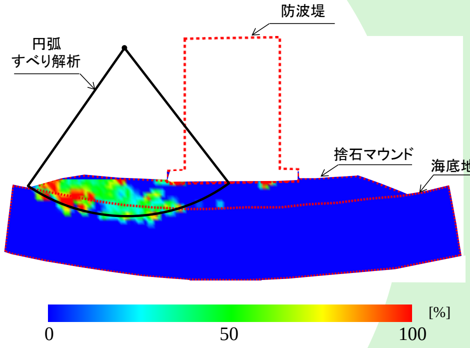

friction : Fr
Weight : W
Excess pore
water pressure : u_e
Tractive force
Seepage foce
Close up
Tractive
force : τ
Lift force : LF
friction : Fr
Weight : W
Soil particle

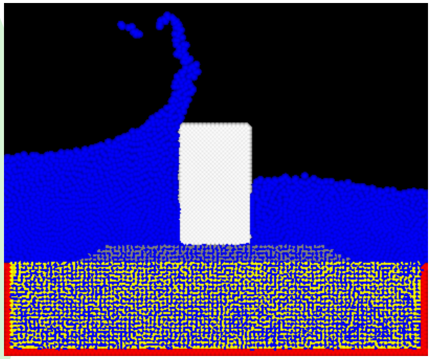

friction : Fr
Weight : W
Soil
Close up
Tractive
force : τ
Lift forc
friction : Fr
Weight :
Excess pore
water pressure : u_e

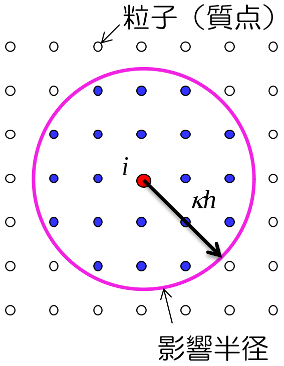
Tsunami experiment using centrifuge device

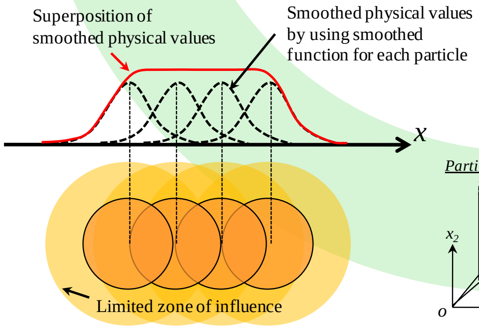


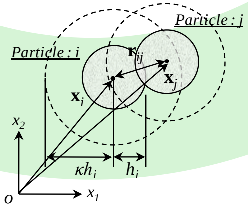












Numerical Analysis using SPH method