Evaluation of the compaction effects on the seismic stability of earth-fill dam – a case history -

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Abstract: The main and auxiliary Fujinuma dams, Fukushima Prefecture, Japan, totally collapsed by the 2011 Off the Pacific Coast of Tohoku Earthquake (M9.0). The collapse was primarily due to poor compaction of the dam body while triggered by a prolonged strong seismic motion. The collapse mechanism of the main dam consisted of multiple sliding failures in the upstream and downstream slopes, which caused an overflow that eroded very fast the dam body, resulting in a total breach. This failure mechanism is successfully captured by numerical analysis consisting of a modified Newmark slip displacement method and a non-linear FEM, both taking into account the continuous deterioration of undrained stress-strain behaviour of the dam materials caused by undrained irregular cyclic excitation during seismic loading.

The collapsed earth-fill dams were restored to new ones by April 2017, designed to be very highly against the seismic loads by which the old dams collapsed based on seismic stability evaluated by the numerical analysis methods that successfully simulate the collapse of the old main dam. Relevant fill materials for the core and shell were prepared and carefully compacted controlling not only the dry density and water content but also the degree of saturation towards the optimum degree of saturation. It was confirmed that the fills could be compacted on average to a degree of compaction by Standard Proctor $(D_c)_{1Ec}$ equal to nearly 100 %, which is substantially higher than 82 - 88 % with the old dams. A high seismic stability of the completed new main dam was confirmed by numerical analysis based on the stress-strain behaviour of the fill materials compacted to $(D_c)_{1Ec}$= 100 %.

A rather strong aftershock (M7.3) took place in the same region in Feb. 13, 2021, by which a maximum horizontal acceleration= 102 gals was recorded immediately below the toe of the downstream slope of the new main dam. No damage was reported to the new main and auxiliary dams. The seismic response analysis of the main dam using the recorded earthquake motion showed a good agreement with the horizontal acceleration with a maximum of about 300 gals observed at the dam crest. Besides, stability analysis showed only very small residual deformation with no slip displacement. These results validate the analysis method. The stability analysis using the drained shear strengths of the dam materials compacted to $(D_c)_{1Ec}$= 90 % following the previous seismic design code for agricultural earth-fill dams in Japan noticeably over-estimated the observed residual displacements. This result is on the safe side whereas it unduly underestimates the actual high stability of the new dams.

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Contents

1. **Collapse of Fujinuma dam by the 2011 Great East Japan Earthquake and its reconstruction**

2. Numerical analysis methods for stability and deformation of collapsed and restored dams

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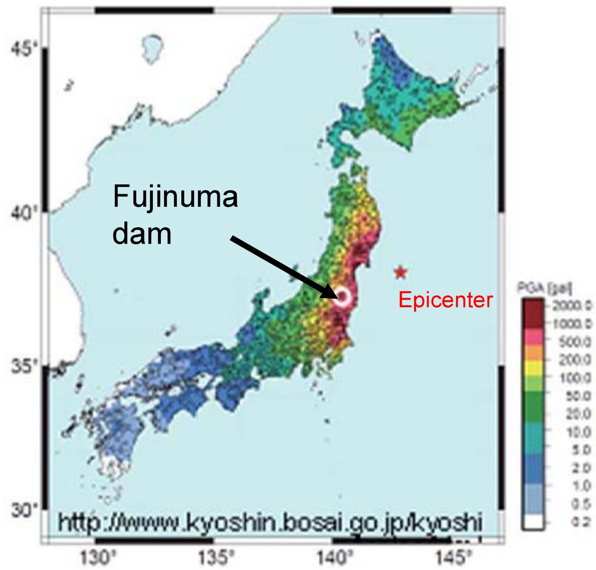
4. Conclusions

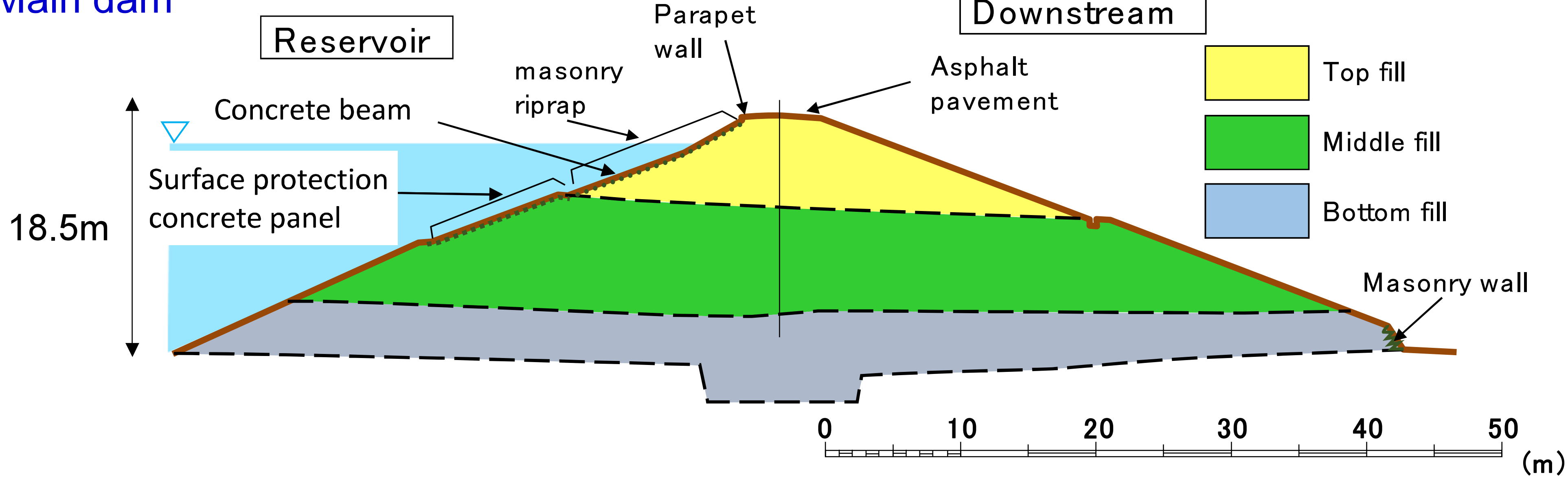
Collapse of Fujinuma dams, Fukushima Prefecture, Japan

- Irrigation earth-fill dams constructed from April 1937; interrupted during WW II & completed October 1949. (main dam: H=18.5m; L=133.2 m; no central core),
- Collapsed by the 2011 Great East Japan EQ.
- Among about 3,000 irrigation dams & dykes in Fukushima Pref., ≈750 were damaged (25%) while Fujinuma dams & 2 others fully collapsed.
- Over 200,000 irrigation dams in Japan.



Locations of the epicenter of the 2011 Great East Japan E.Q. (**M9.0**) & Fujinuma dam (approx.250km)

Main dam



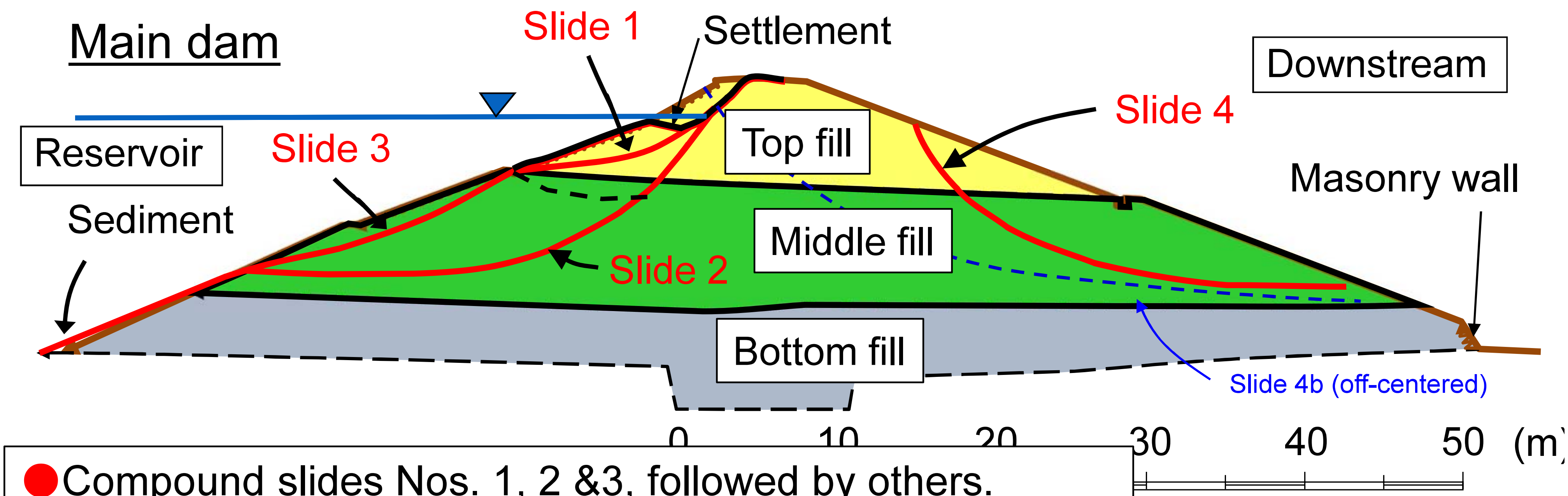
Main dam:
sliding failure resulted in over-topping flow that eroded the fill very fast.

11 March 2011, PM 15：11, immediately after the 2011 Great East Japan Earthquake (PM 14:46:18, 11 March 2011)

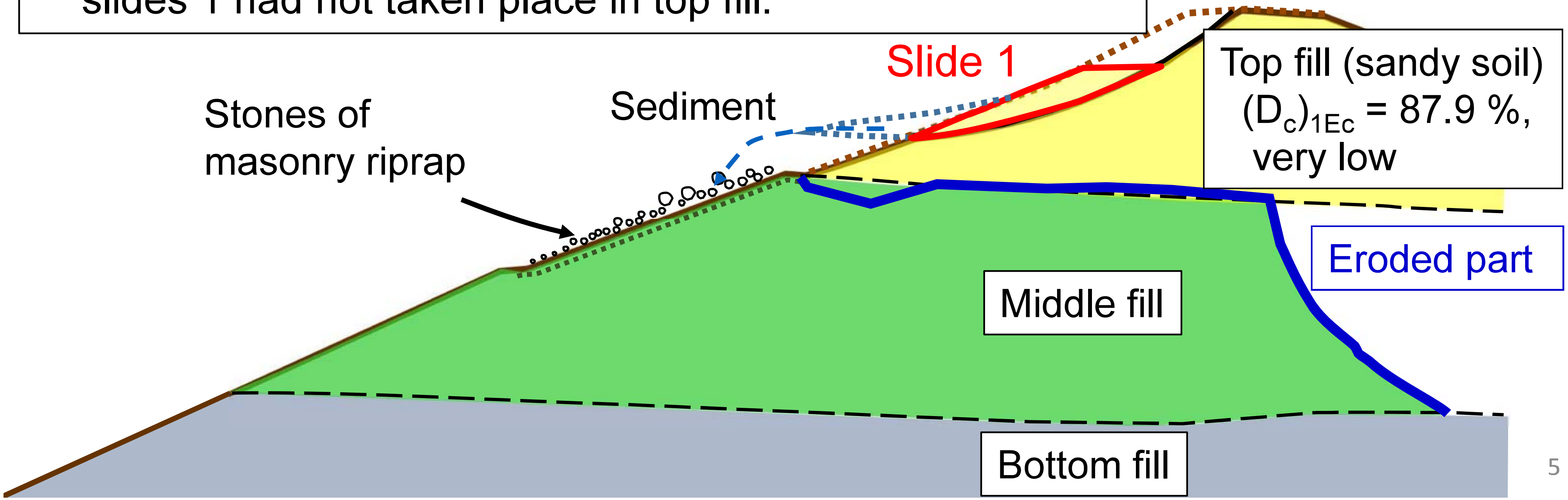
Around one hour later

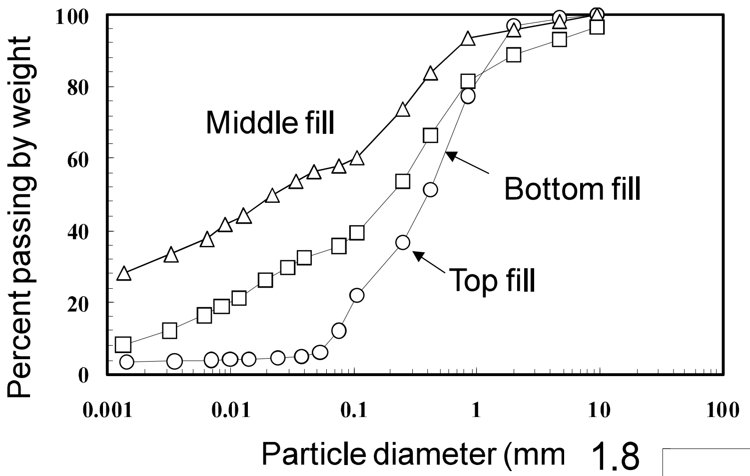
(courtesy of Prof. Tanaka, T.)

Tanaka, Tatsuoka & Mohri(2012), Earthquake-induced failure of Fujinuma Dam, *Proc. Int. Symp. On Dams for a Changing World*, 24th Congress ICOLD, Kyoto, 6.47-6.52 (2012).



- Compound slides Nos. 1, 2 &3, followed by others.
- Slide 1 in top fill was multiple, which resulted in over-topping flow, fast erosion then breaching of the dam.
- Over topping flow may not have taken place if multiple slides 1 had not taken place in top fill.

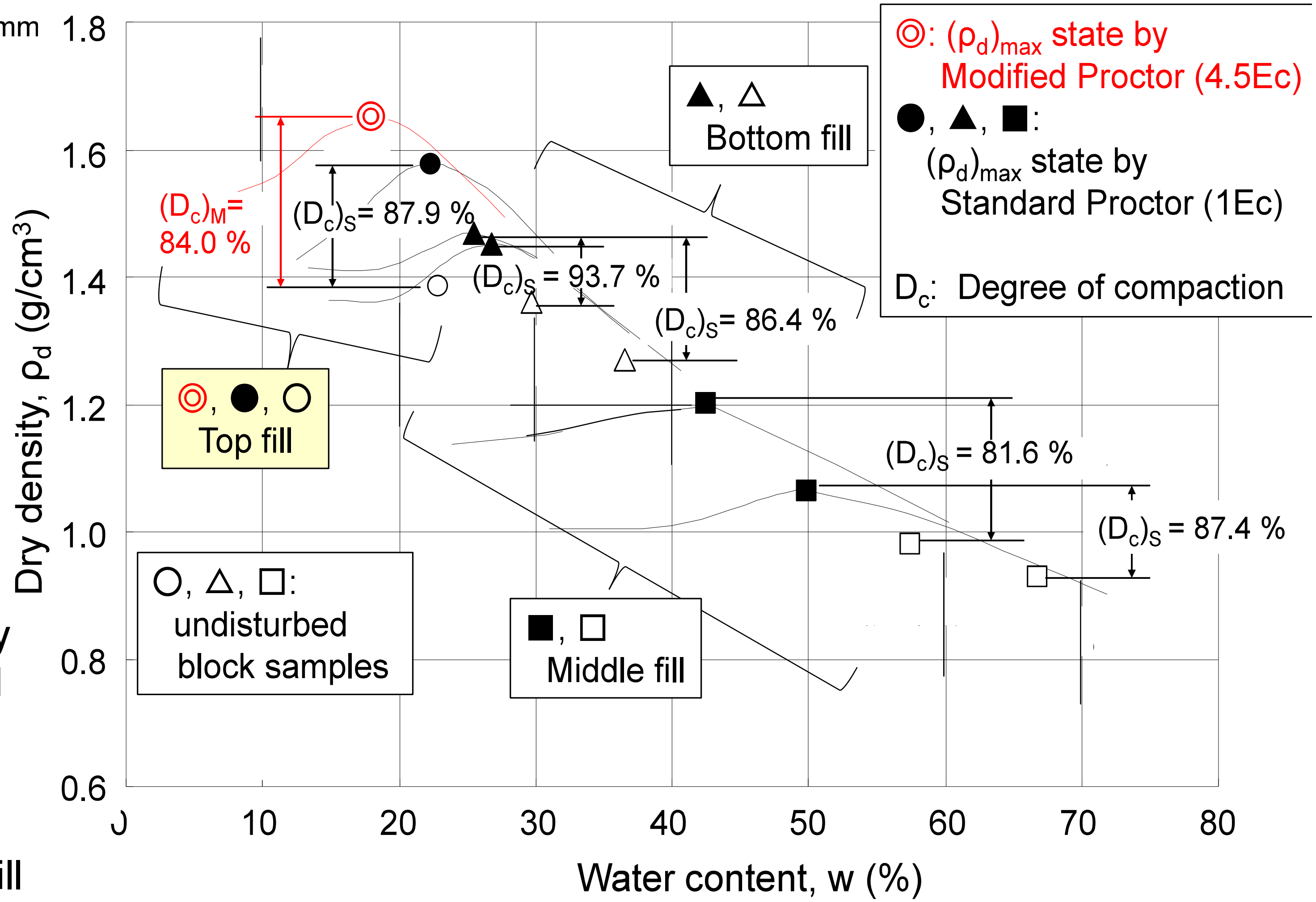




Main causes of collapse :
Particularly poor compaction:
very low $(D_c)_{1Ec}$ values
Top fill: sandy soil compacted to
$(D_c)_{1Ec}$= 87.9%

a) the use of **sandy soil**, which is highly liquefiable when loose & saturated; highly permeable; and highly erodible;

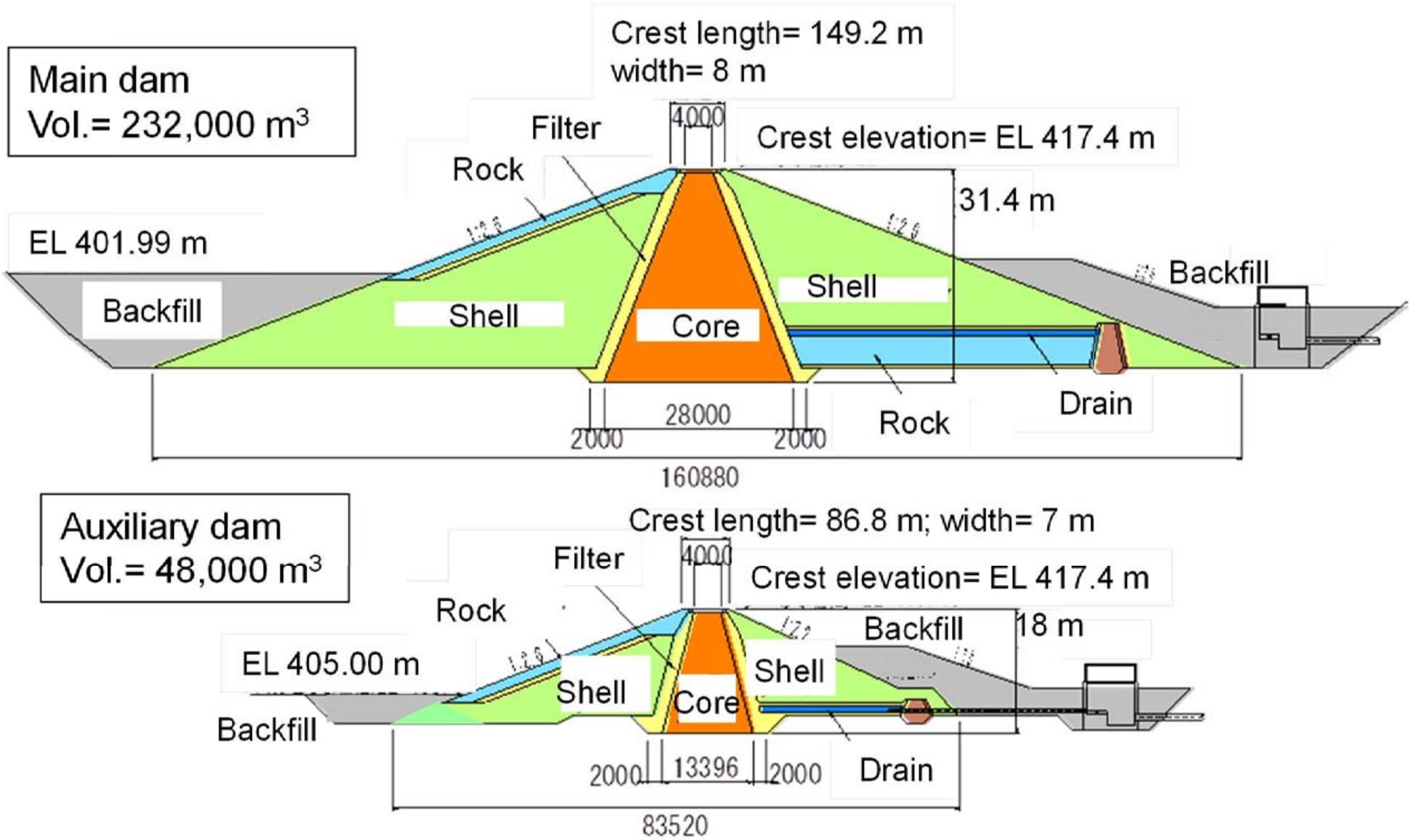
b) very low $(D_c)_{1Ec}$.
⇒ **low undrained strength, fast & large decrease by cyclic undrained loading**
→ start of failure unusually from the top fill, followed by larger failure in top & middle fills
→ over-topping flow
→ fast erosion of the top fill
→ breaching of the dam



Reconstruction of Fujinuma dams (2014 - 2017)

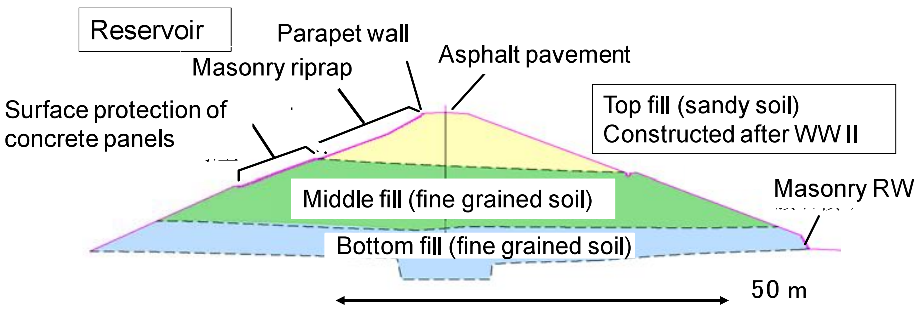
- The new dams were designed to be much more stable than the collapsed old dams, being sufficiently stable against the severe seismic load by which the old dams fully collapsed.

⇒ a modern dam structure; the use of better fill materials; and better compaction by controlling not only dry density & water content but also the degree of saturation.

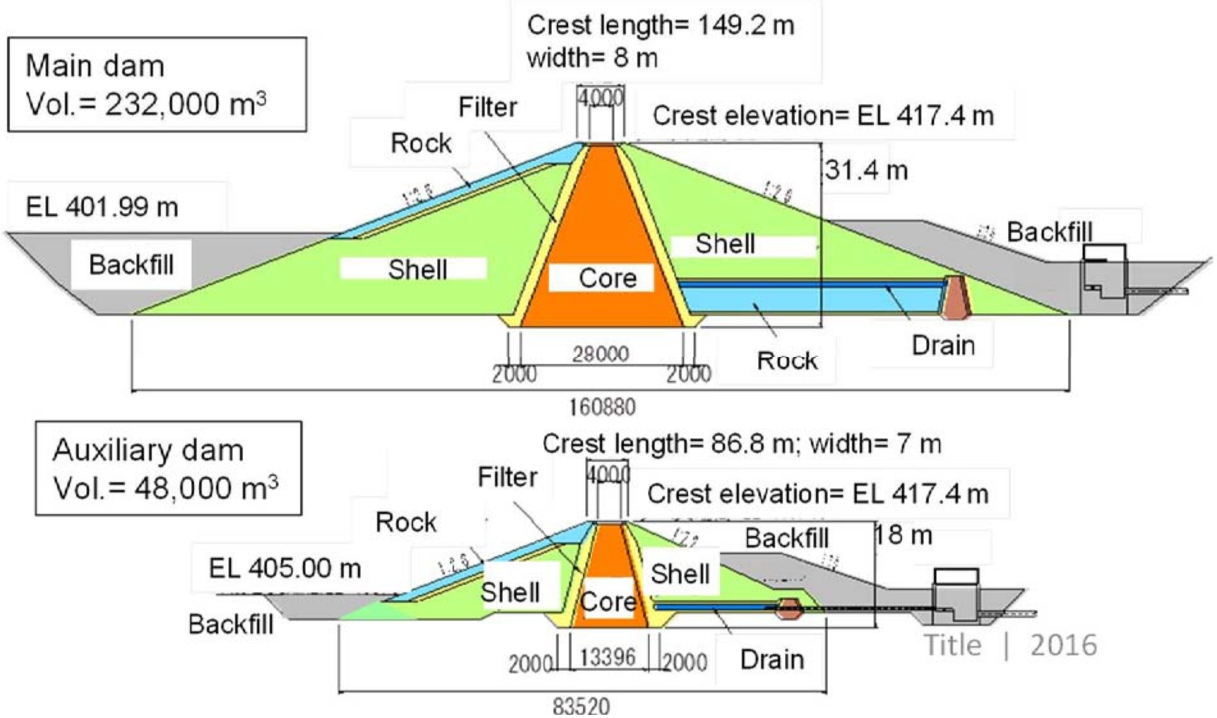


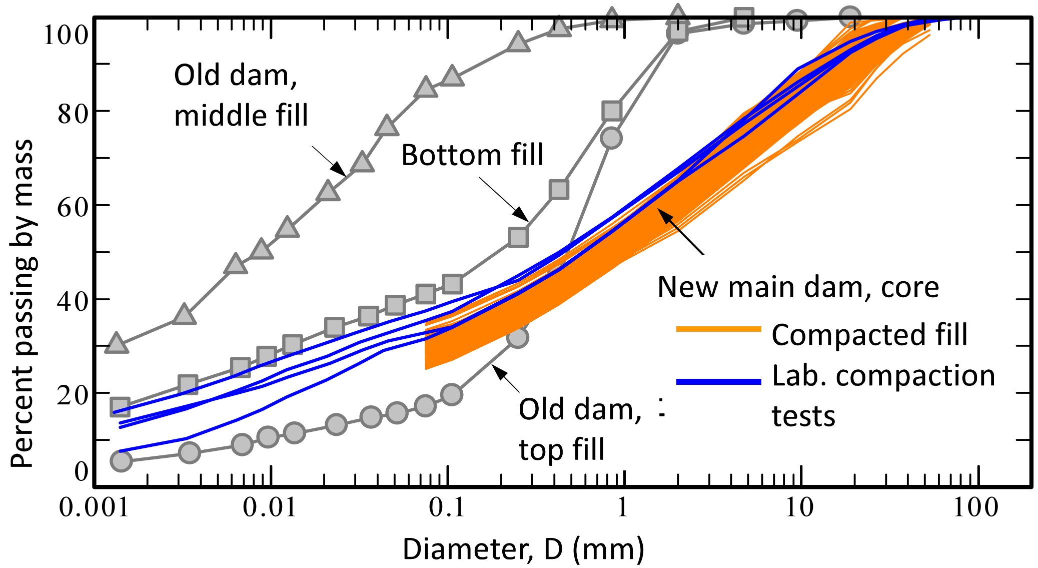
Adequate dam structure; and better fill materials for New Fujinuma dams

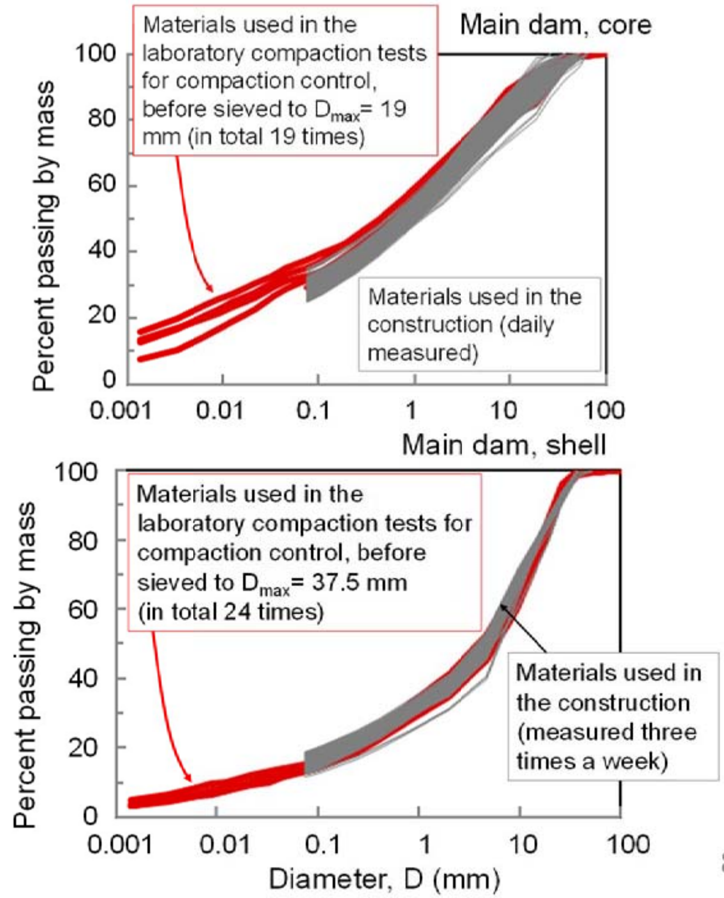
Old main Fujinuma dam



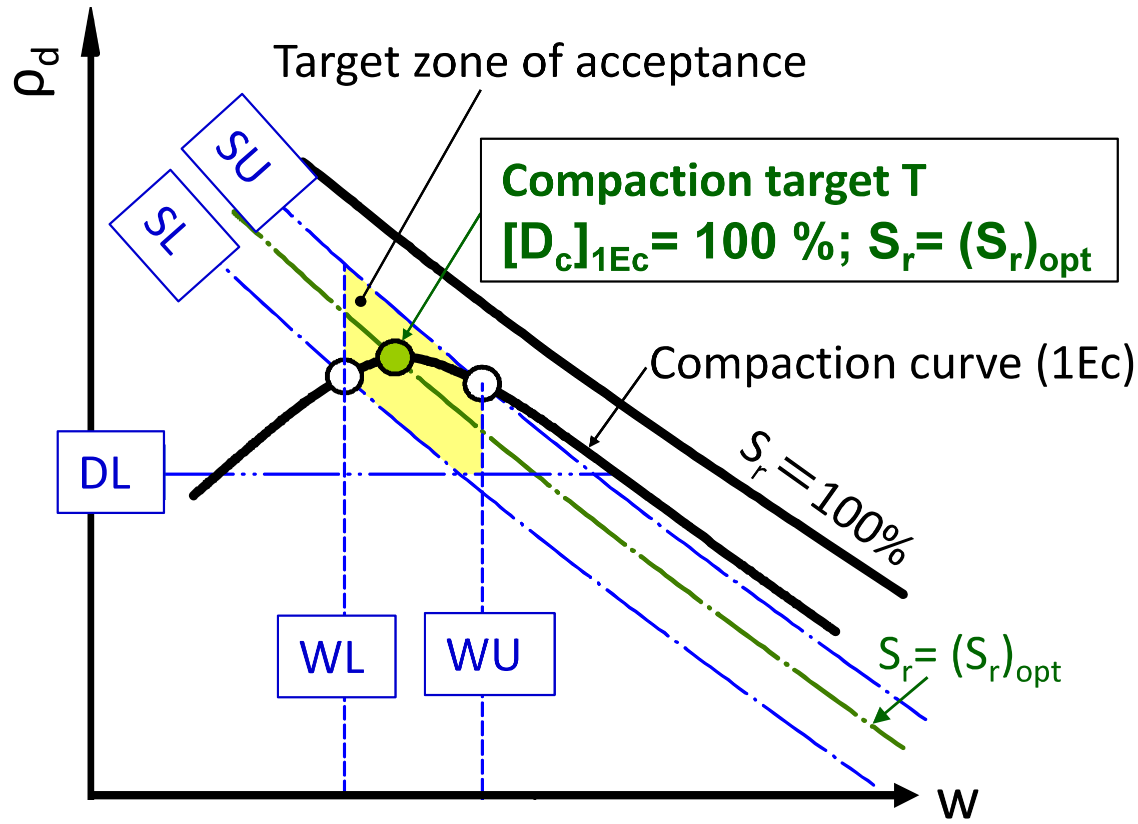
New Fujinuma dams







Specifications of the control of ρ_d, w & S_r for the reconstruction of Fujinuma dam



Material / Control boundary	Core (Lift= 25 cm) (N= 8)	Random (Shell) (Lift= 25 cm) (N= 8)
WL: LB for w	$[w_{opt}]_{1Ec}$ - 1.0%	$[w_{opt}]_{1Ec}$ - 1.0%
WU: UB for w	$[w_{opt}]_{1Ec}$ + 2.0%	$[w_{opt}]_{1Ec}$ + 1.0%
DL: LB for ρ_d	95 % of $[D_c]_{1Ec}$	95 % of $[D_c]_{1Ec}$
SL: LB for S_r	$(S_r)_{op}$ −5%	$(S_r)_{opt}$ −15%
SU: UB for S_r	$(S_r)_{opt}$ +5%	$(S_r)_{opt}$ +6%

N= number of pass by 20 ton-class vibratory tamping roller

Shell material is a mixture of the fill material of the old dam and a borrowed crushed gravel.

Conventional design/construction code for irrigation dams (effective at the time of the design and construction of new Fujinuma dam):

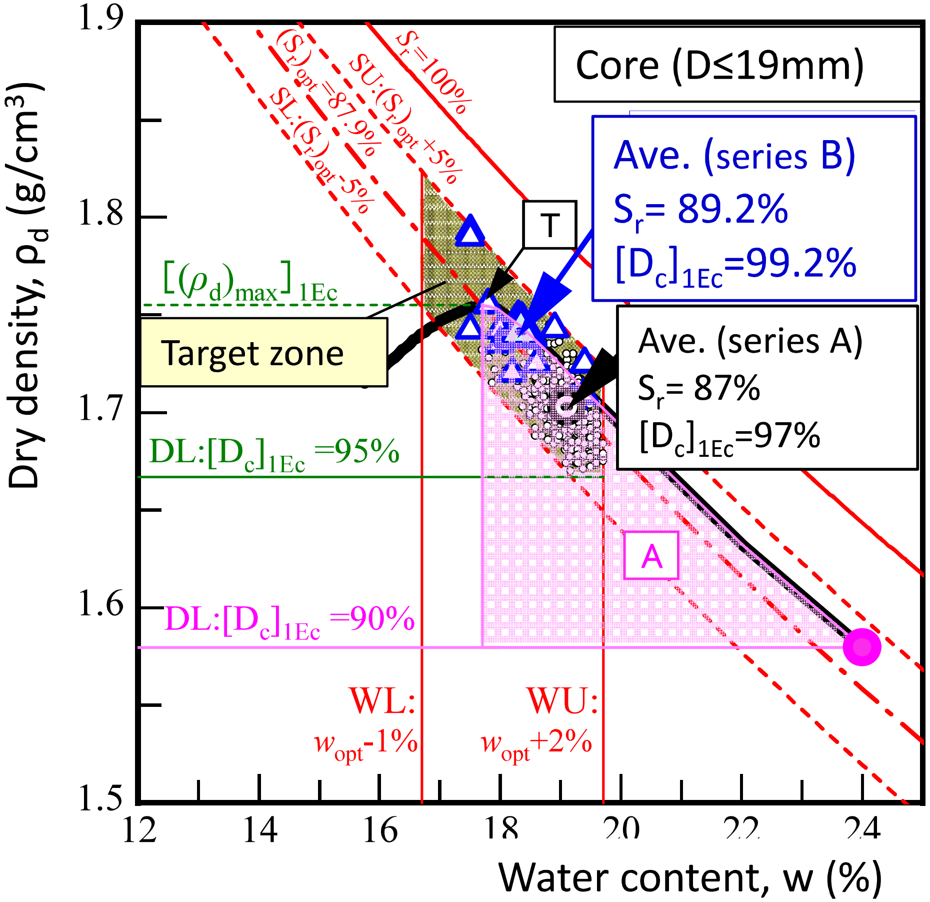
A: Target zone of acceptance, allowable lower bound of $[D_c]_{1Ec}$= 90 %

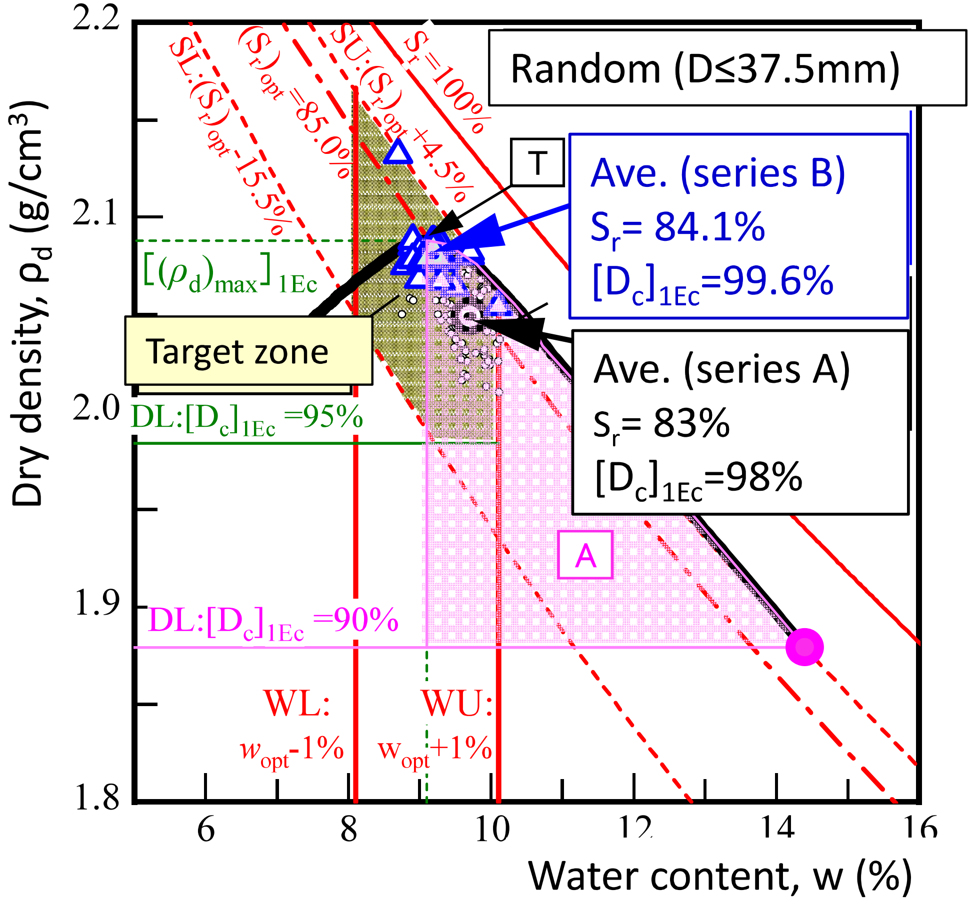
●: Compacted state where the design strength/stiffness is determined.

⇒ leading to ineffective compaction at too high w values

In the construction of new Fujinuma dam, by the new compaction control, satisfactory compacted states at higher $[D_c]_{1Ec}$ & lower *w* around target T.

At target T, $[D_c]_{1Ec}$= 100% & S_r= $(S_r)_{opt}$ are realized cost-effectively.





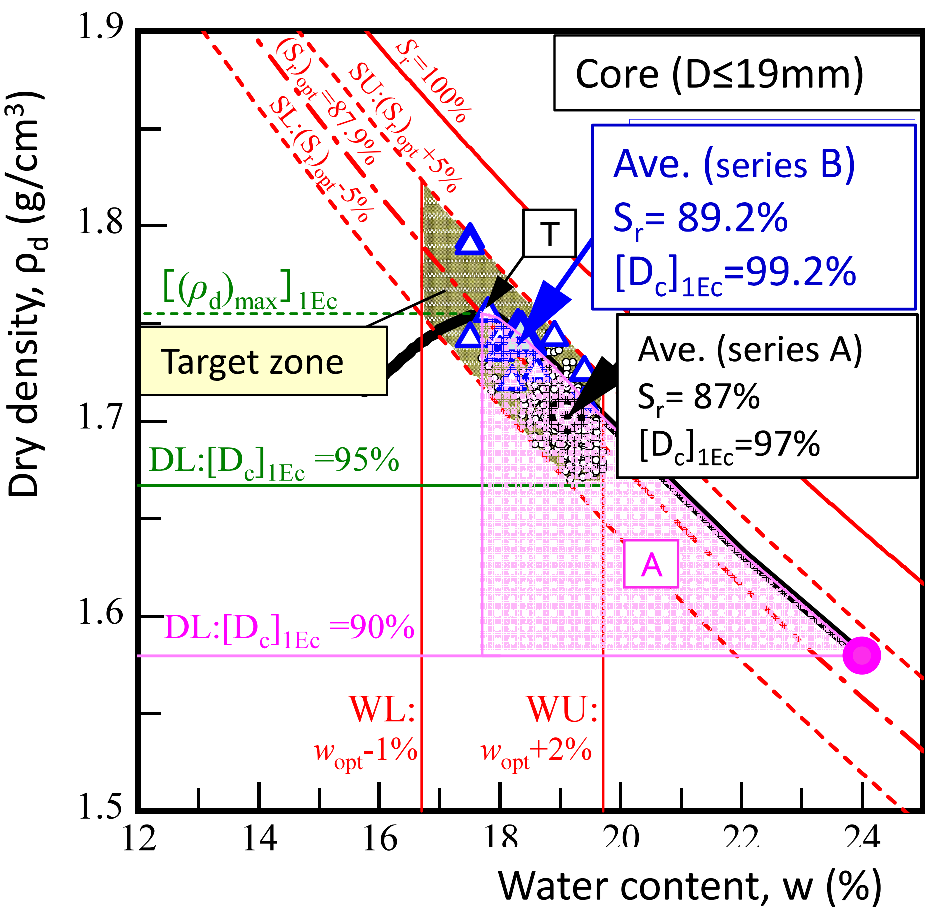
Conventional design/construction code for irrigation dams (effective at the time of the design and construction of new Fujinuma dam):

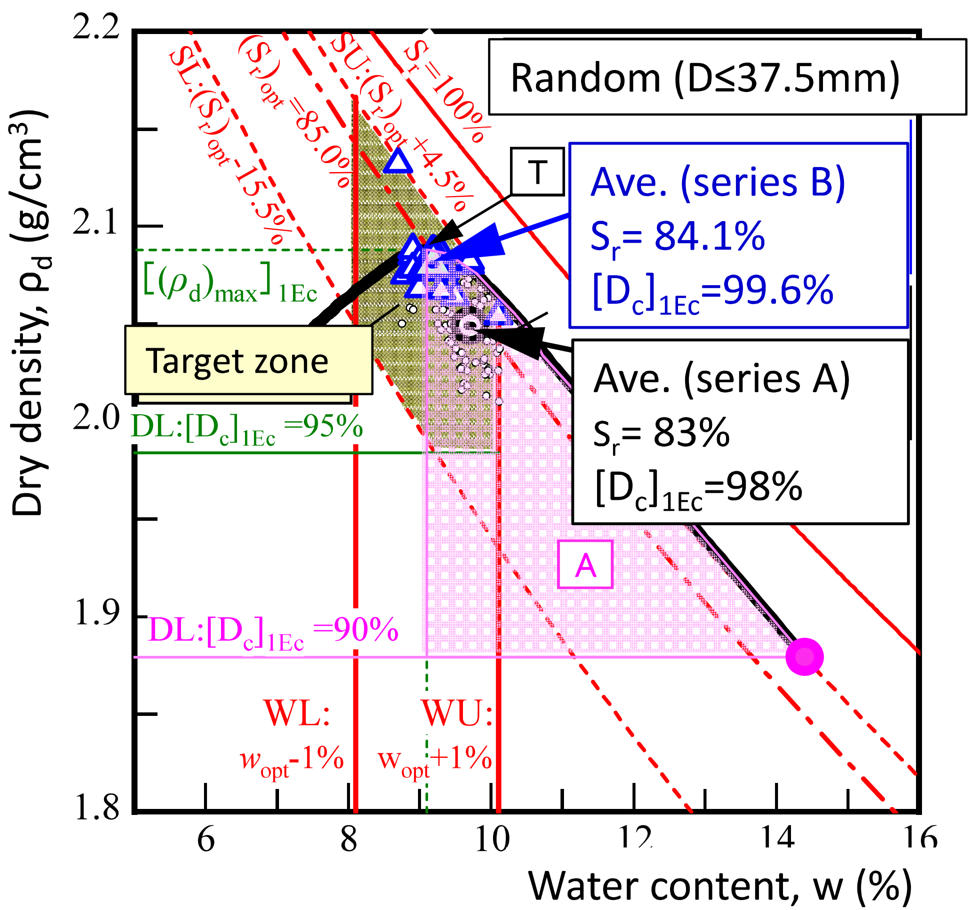
A: Target zone of acceptance, allowable lower bound of $[D_c]_{1Ec}$= 90 %

●: Compacted state where the design strength/stiffness is determined.

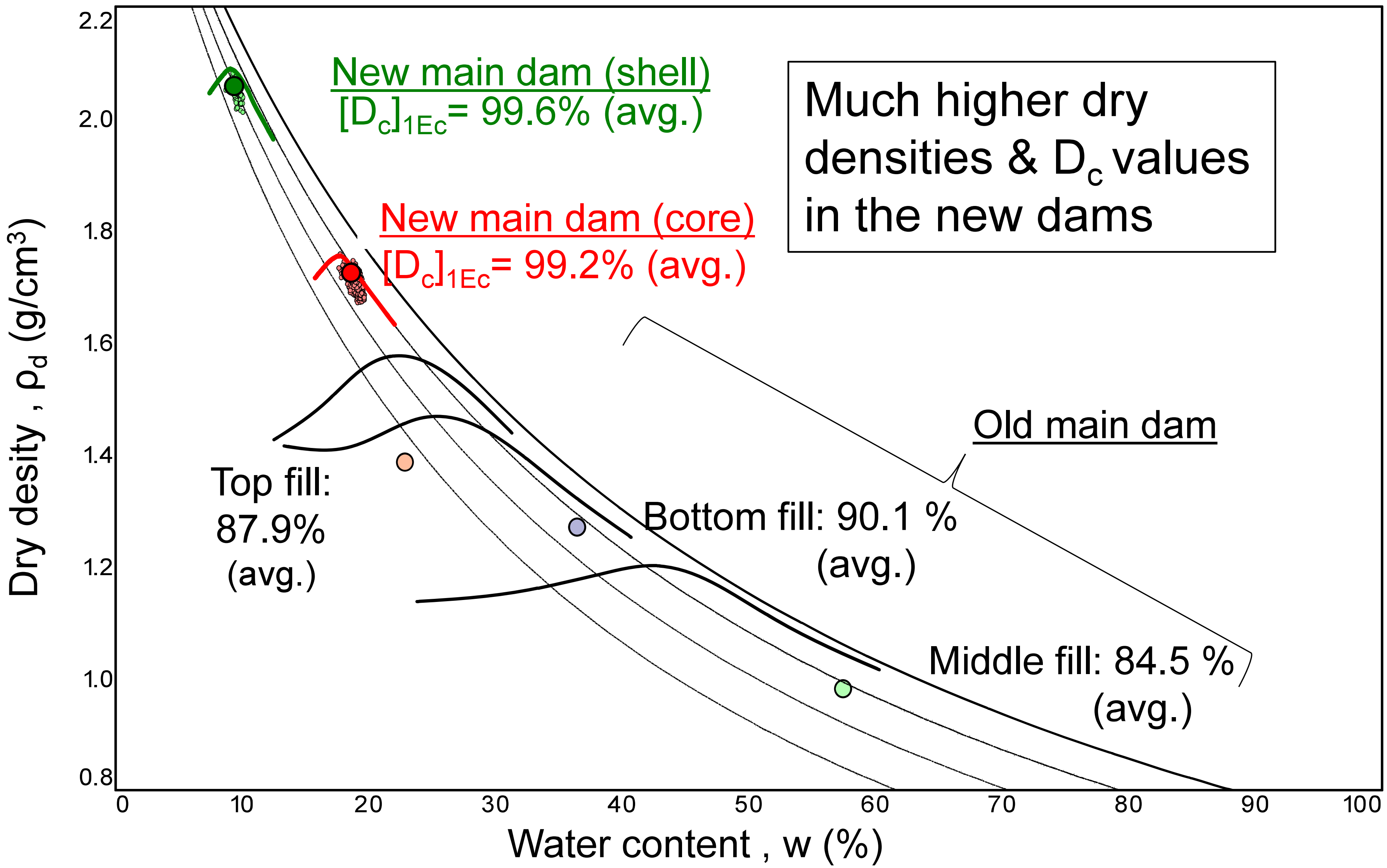
⇒ leading to ineffective compaction at too high w values

⇒ Subsequently to this project, the design/construction code was revised introducing the structure of this new compaction control.





Comparison of compacted states between the old main dam and the new main dam



Tatsuoka & Miura (2019): Compacted states and physical properties of soil controlled by the degree of saturation during compaction, *this conference*

Contents

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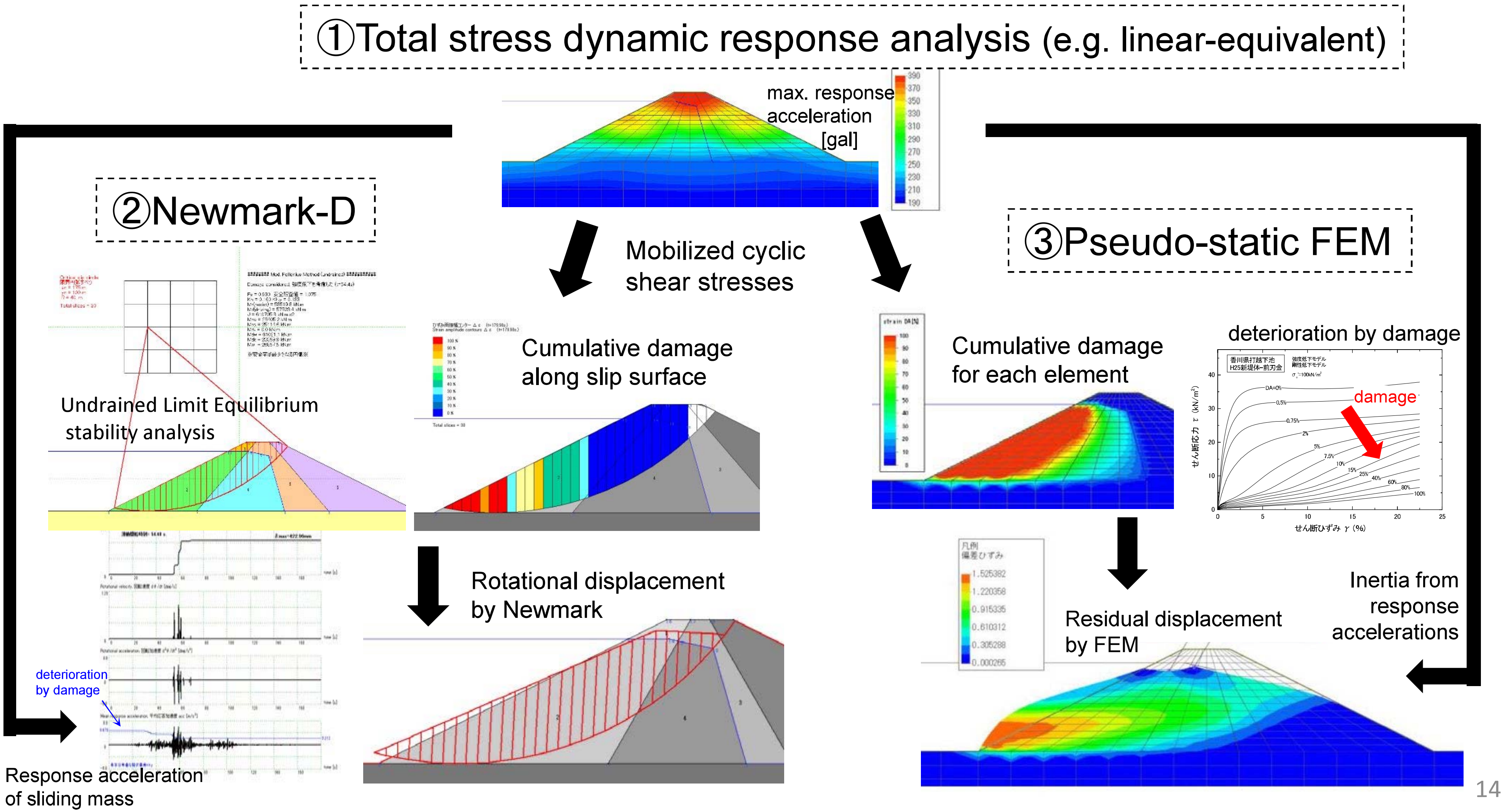
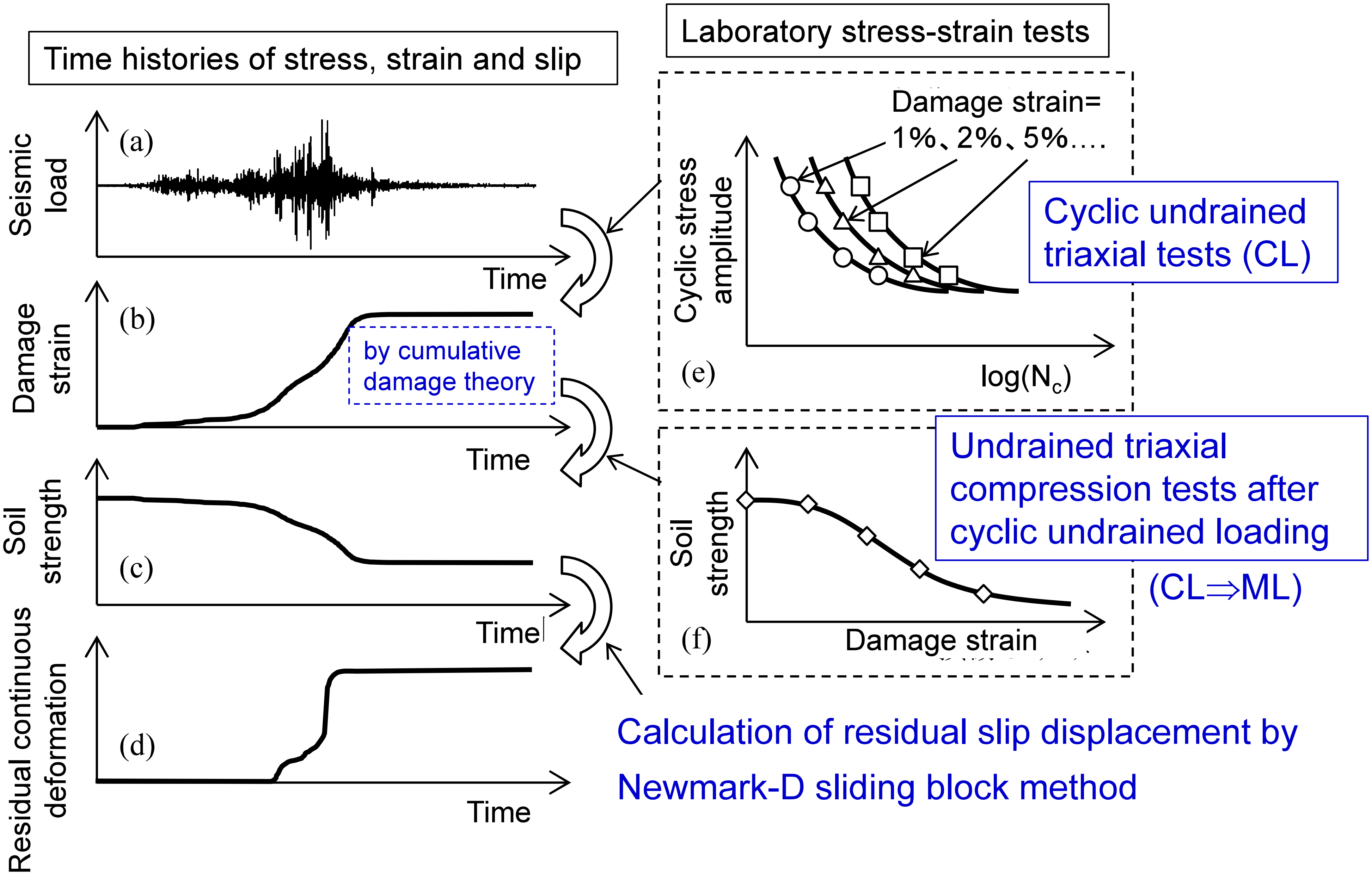
4. Conclusions

1. Tatsuoka et al. (2018): Soil properties and seismic stability of old and new Fujinuma dams, *Proc. Int. Symp. on Qualification of dynamic analyses of dams and their equipment, Saint-Malo, France* (eds. Fry, J.-J. & Matsumoto, N.), 119-170 (2018)
2. Duttine et al. (2018): A new simplified seismic stability analysis taking into account degradation of soil undrained stress – strain properties and effects of compaction, *Proc. Int. Sym. on Qualification of dynamic analyses of dams and their equipment, Saint-Malo, France* (eds. Fry, J.-J. & Matsumoto, N.), 215-234 (2018)

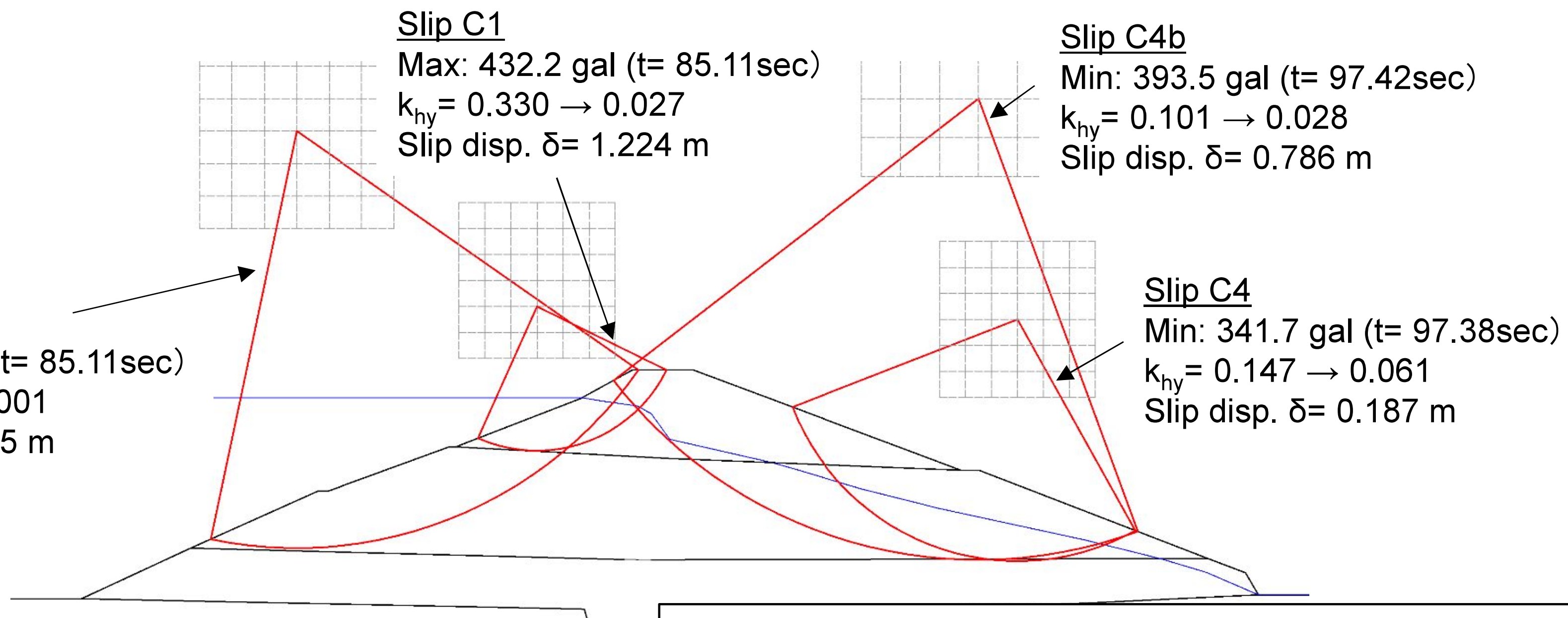
⇨ ① 2D FEM total stress dynamic response analysis (deterioration of soil strength ignored) →response acceleration & stresses (conservatively evaluated)
② Newmark-D sliding block analysis (taking into account deterioration of strength by UCL) →residual deformation by sliding
③ 2D FEM pseudo-static analysis (taking into account deterioration of non-linear stiffness & strength by UCL) →residual deformation excluding the one by sliding



2) Slip displacement analysis by Newmark-D method



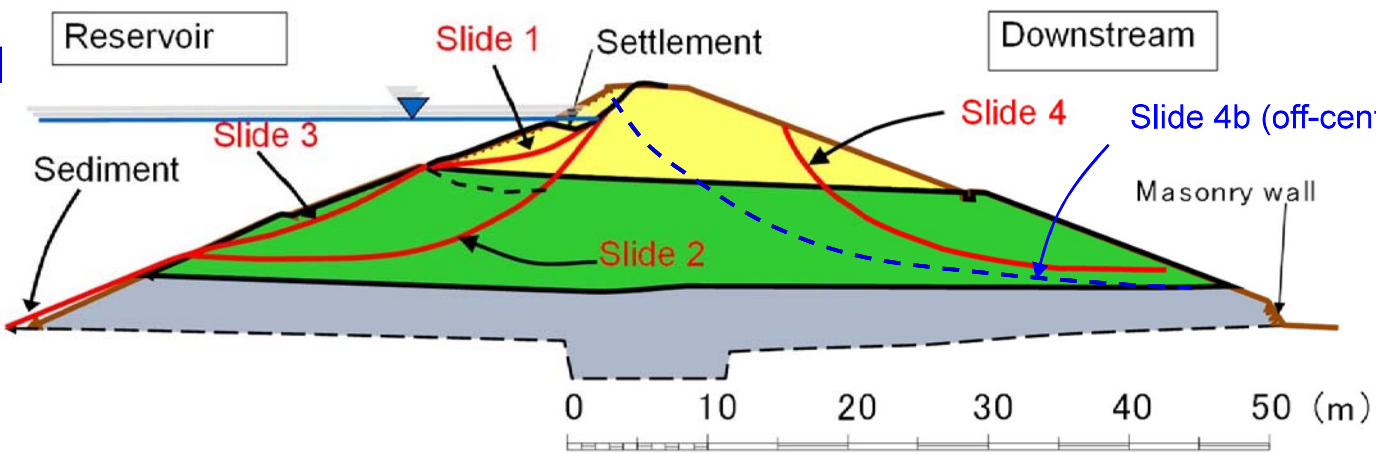
RESULTS: Critical slip circles exhibiting large slip for OLD DAM



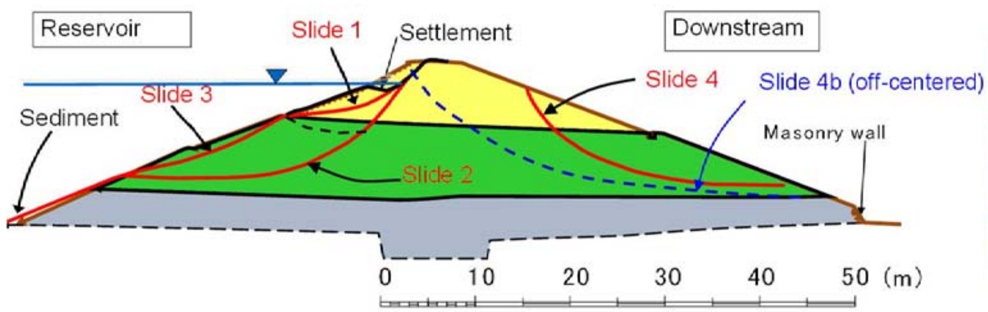
k_{hy}: yield horizontal seismic coefficient, the value of k_h by which the global safety factor F_s becomes unity.

Max: maximum horizontal response acceleration unfavorable to upstream
Min: maximum horizontal response acceleration unfavorable to downstream

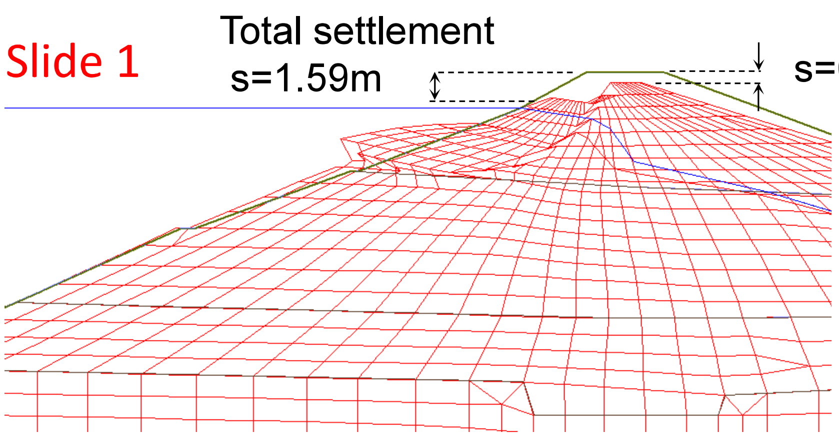
Very similar to the actual slip planes, 1, 2, 3 & 4

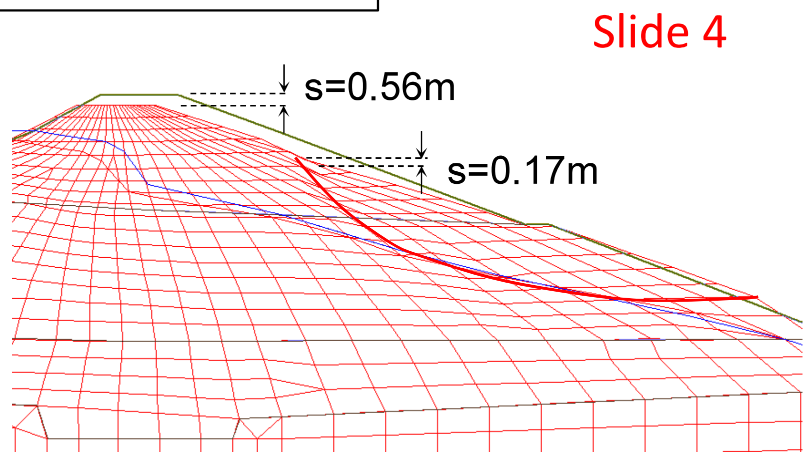


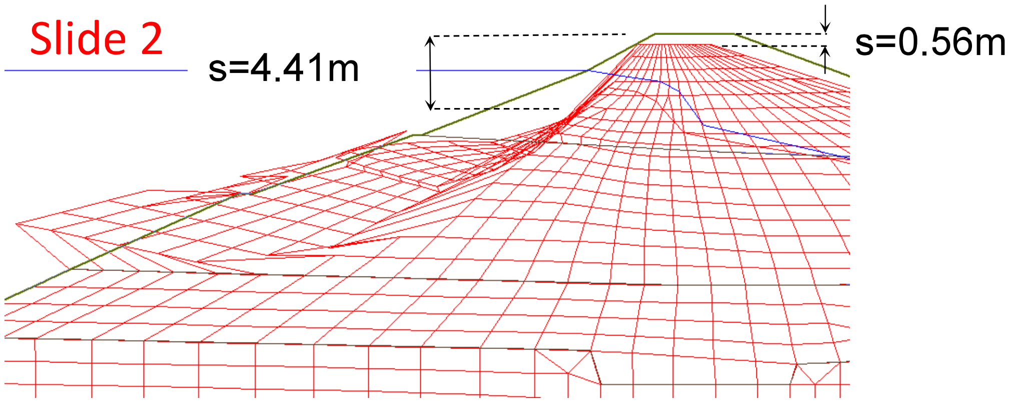
Actual slip plane

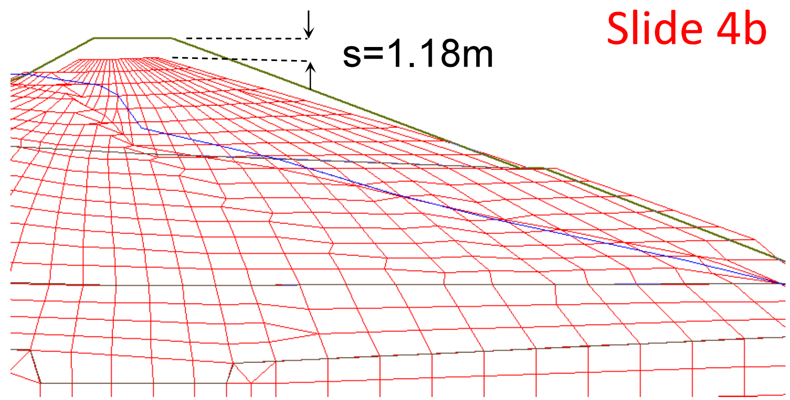


Combined displacements by Newmark-D + 2D pseudo-static FEM

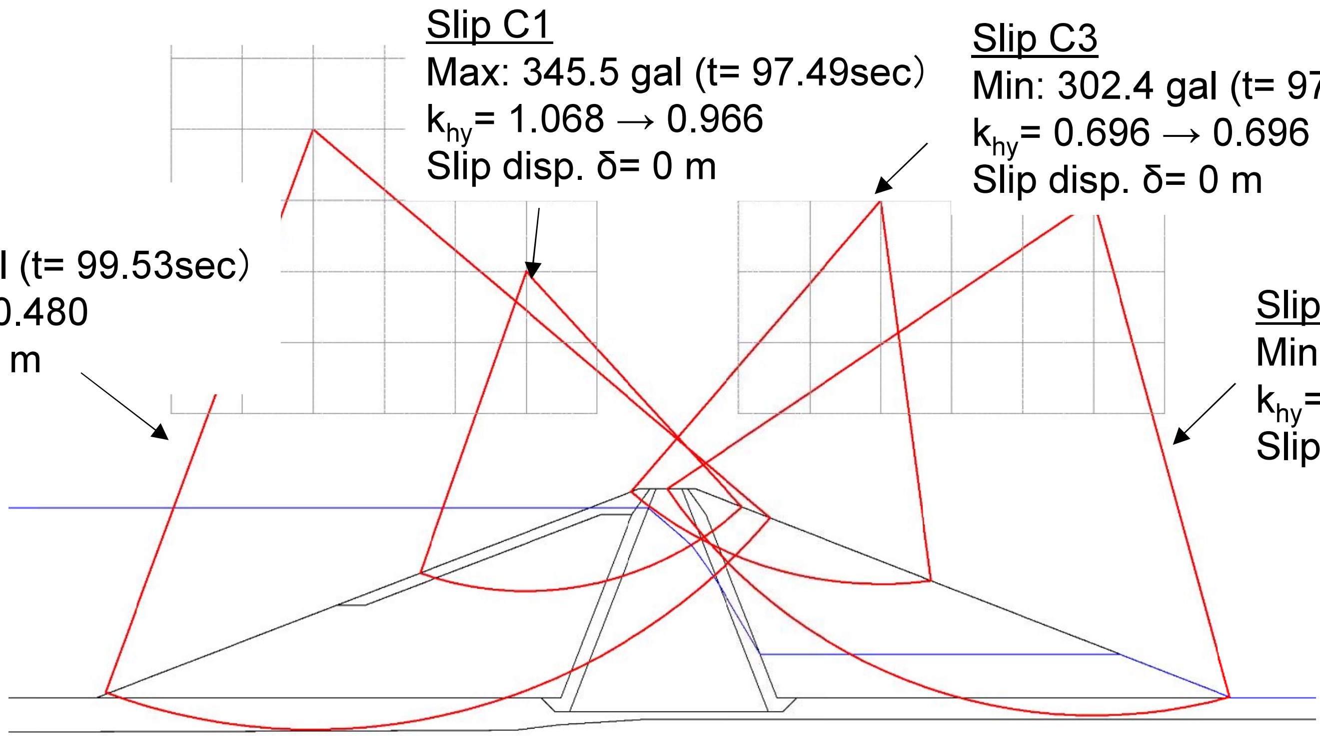








Critical slip circles exhibiting smallest k_{hy} values : NEW DAM



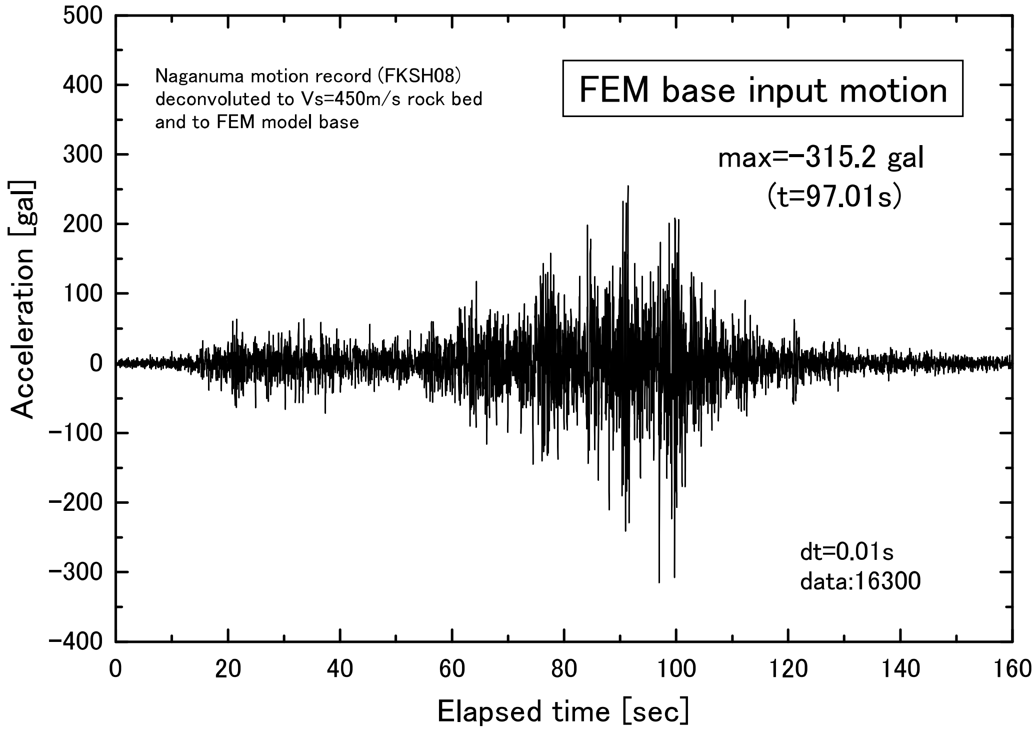
k_{hy}: yield horizontal seismic coefficient,
the value of k_h by which the global safety factor F_s becomes unity.

Slip C1: k_{hy} decreases from a high initial value (1.068) toward a still high final value (0.966).

Slip C2: k_{hy} decreases from a high initial value (0.502) toward a still high final value (0.480)

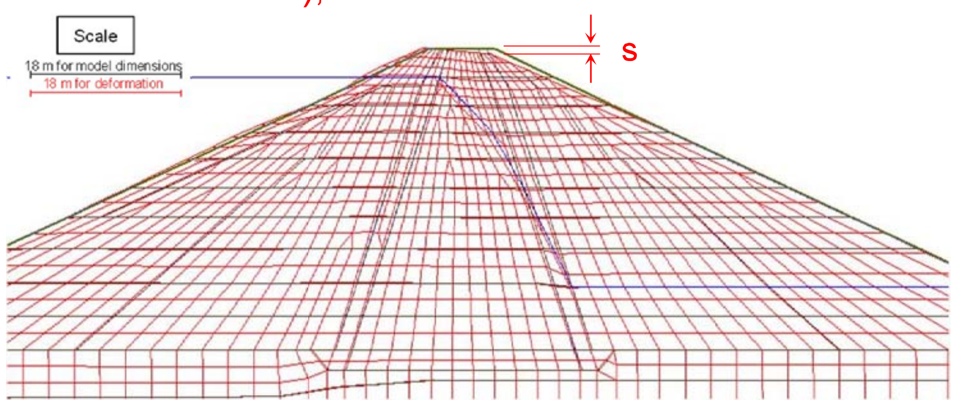
→ Zero sliding displacements along slips C1 to C4

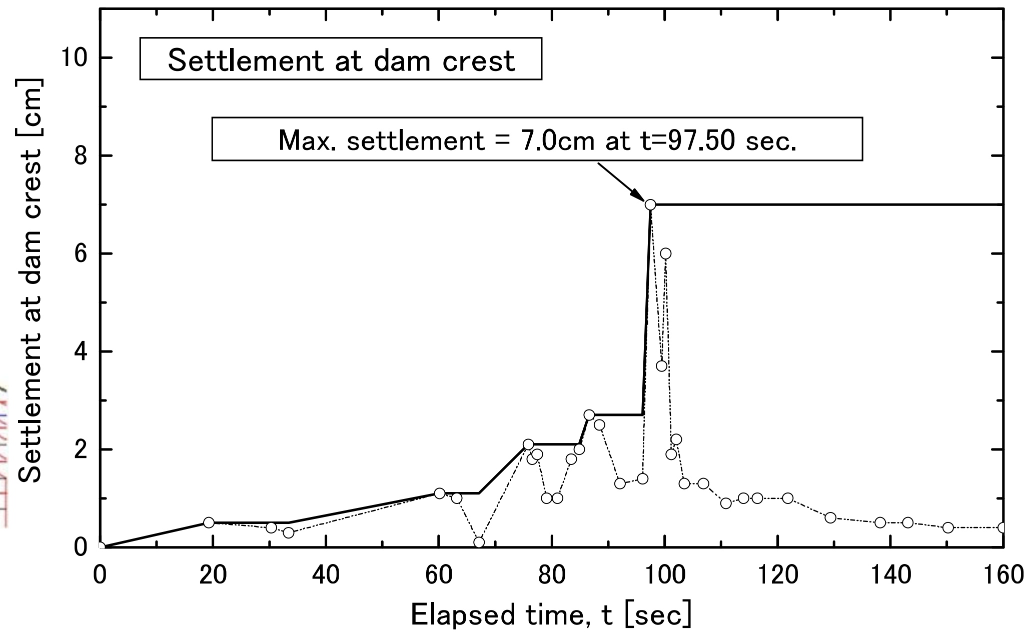
Analysis of residual continuous deformation by a series of pseudo-static non-linear FEM analysis: new main dam



A very small settlement at the dam crest

s= 0.07m : total settlement at crest by continuous deformation (pseudo-static FEM), s/H= 0.2%



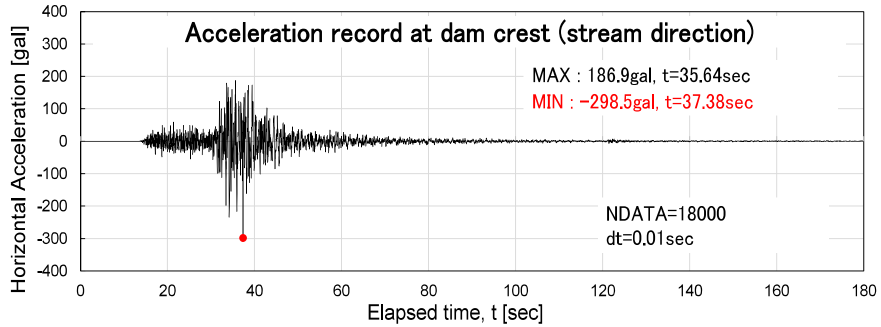


Contents

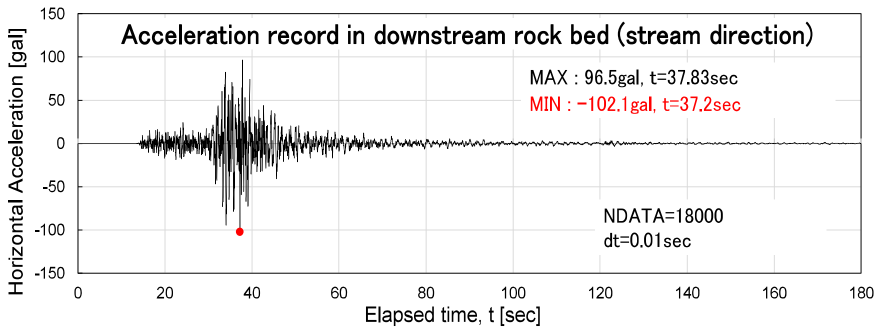
Fukushima Prefecture Offshore Earthquake (M7.3) on Feb. 13, 2021, one of the major aftershocks of the 2011 Great East Japan Earthquake.

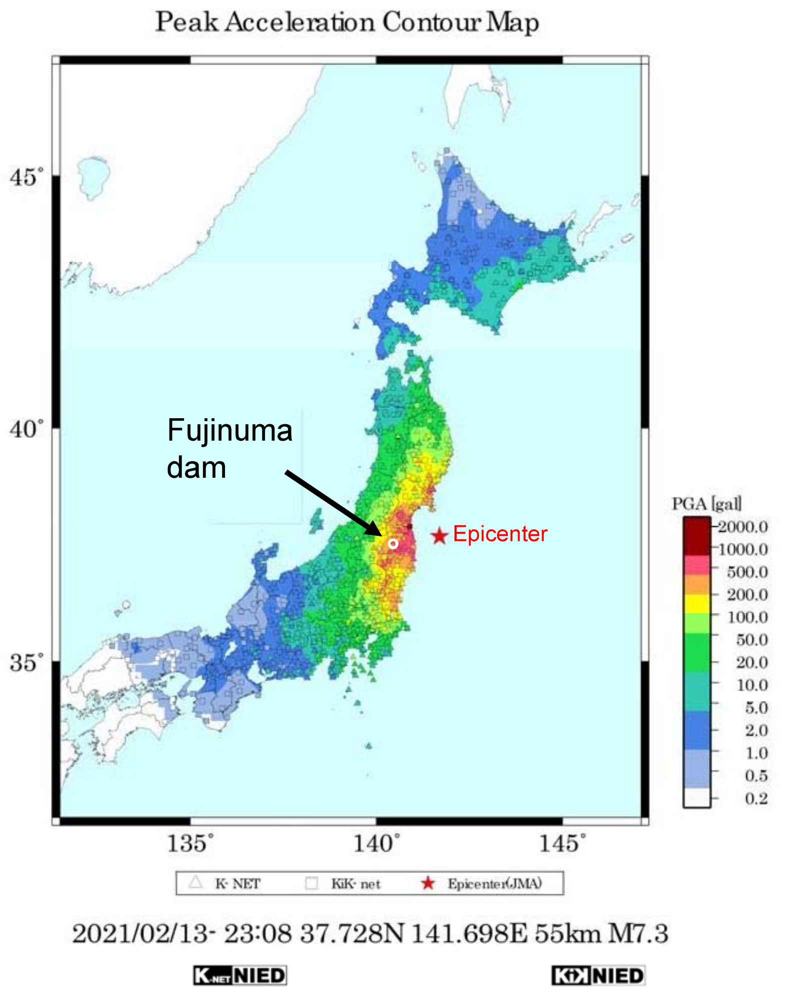
・Distance to epicenter: 148.5km , JA intensity:4.8

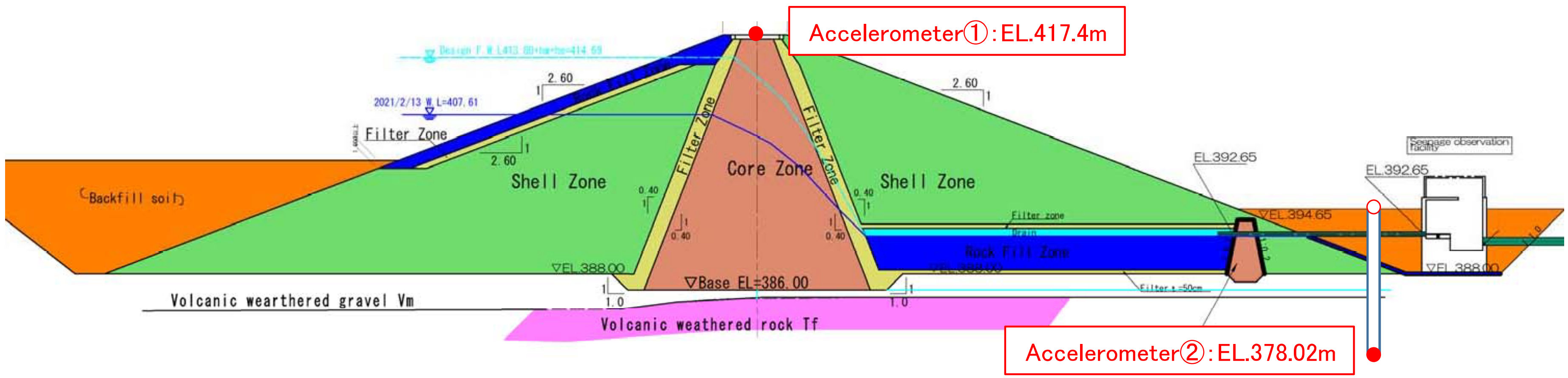
①



②

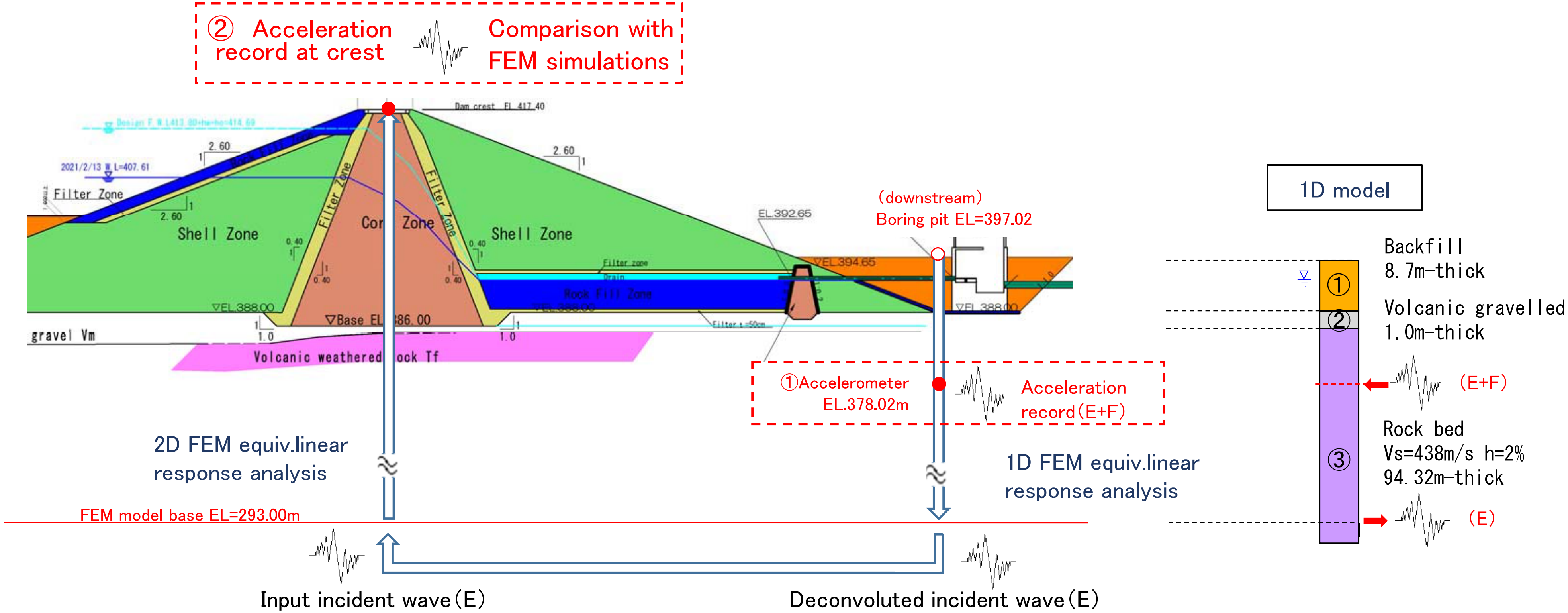


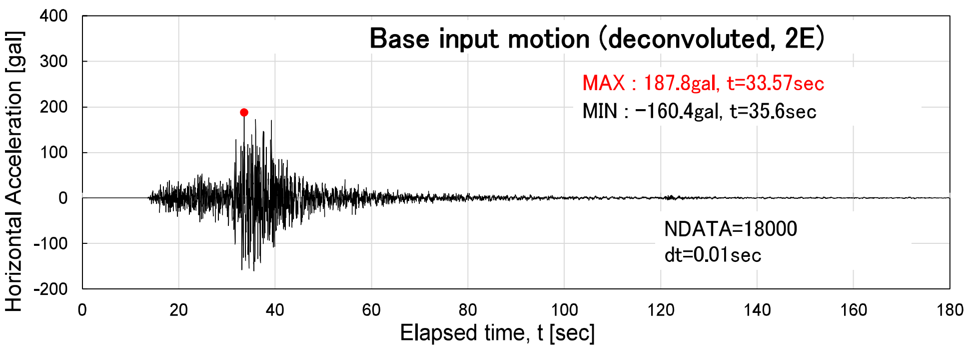


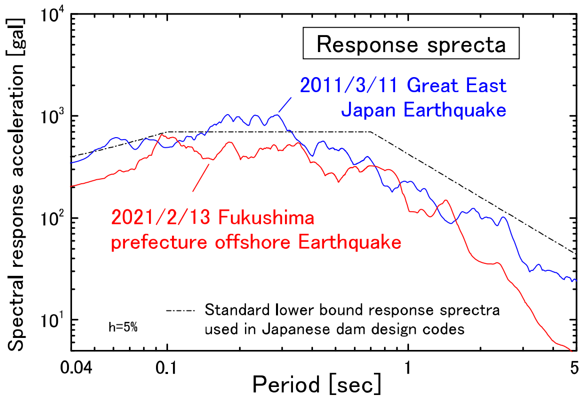


Base input motion used in the 2D FEM seismic response analysis

・Acceleration record in the rock bed deconvoluted to FEM model base by 1D equiv.-linear response analysis

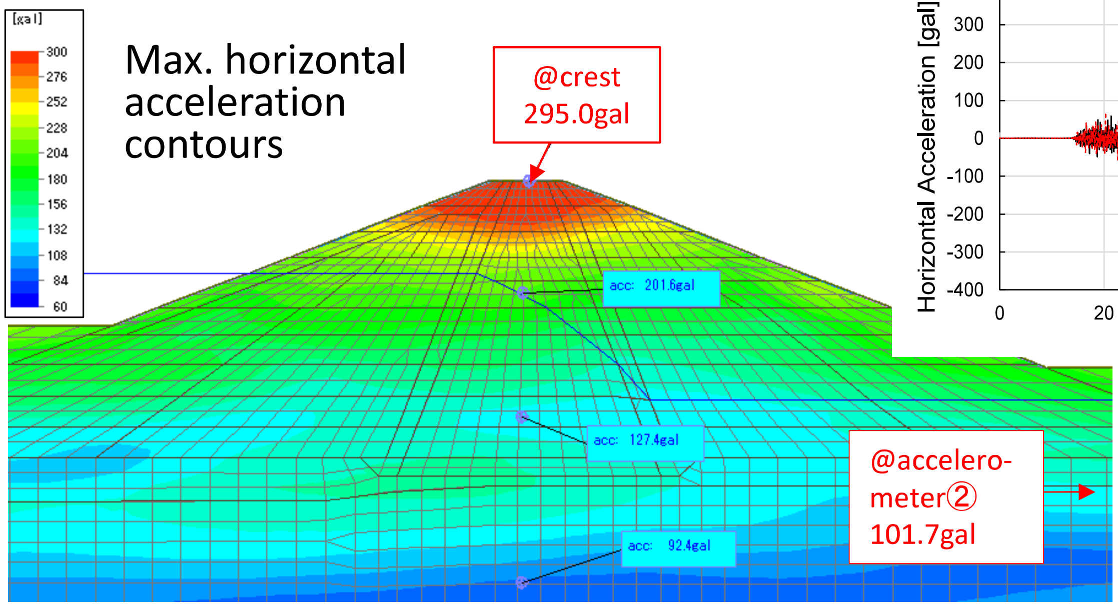


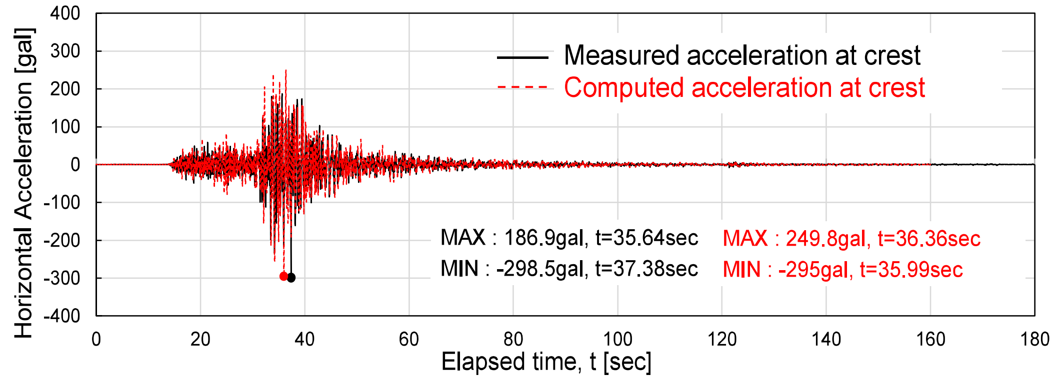


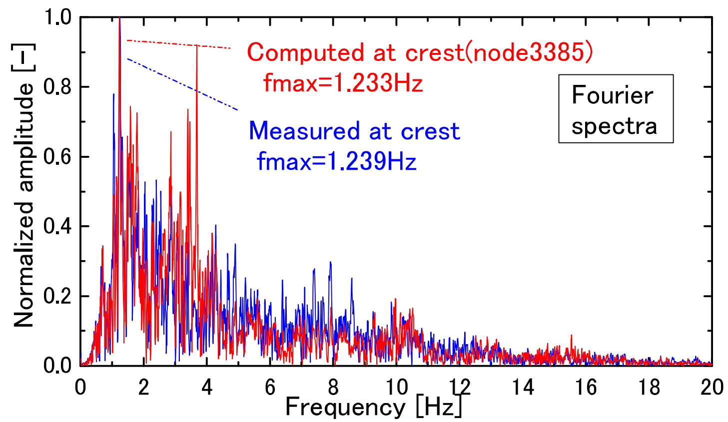


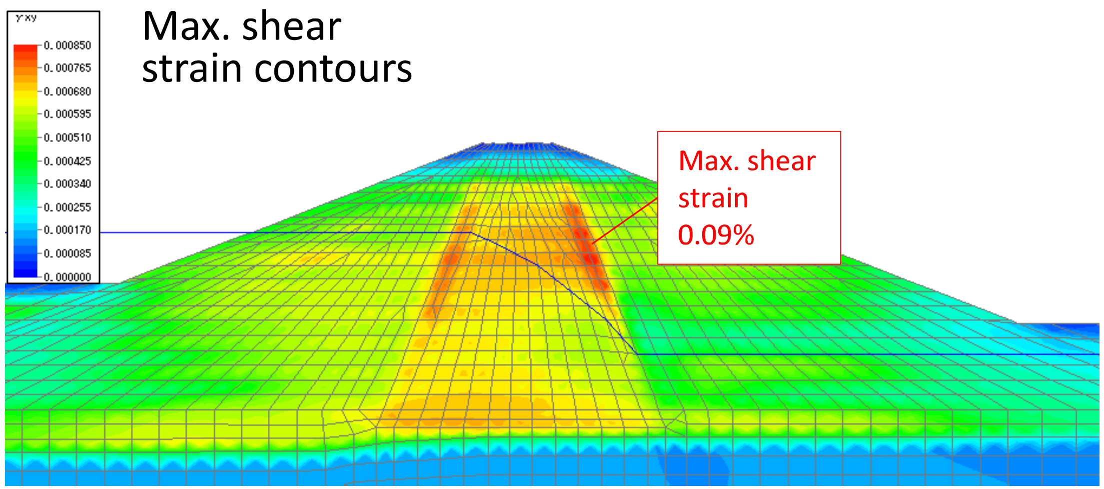
Results of 2D FEM dynamic response analysis

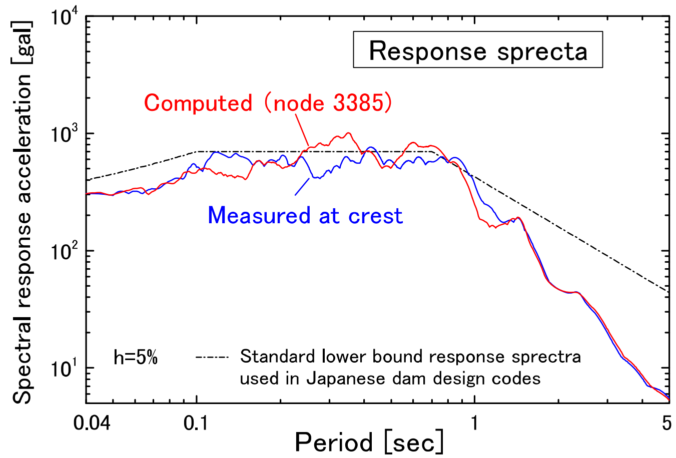
・Computed acceleration time history in good agreement with records at crest





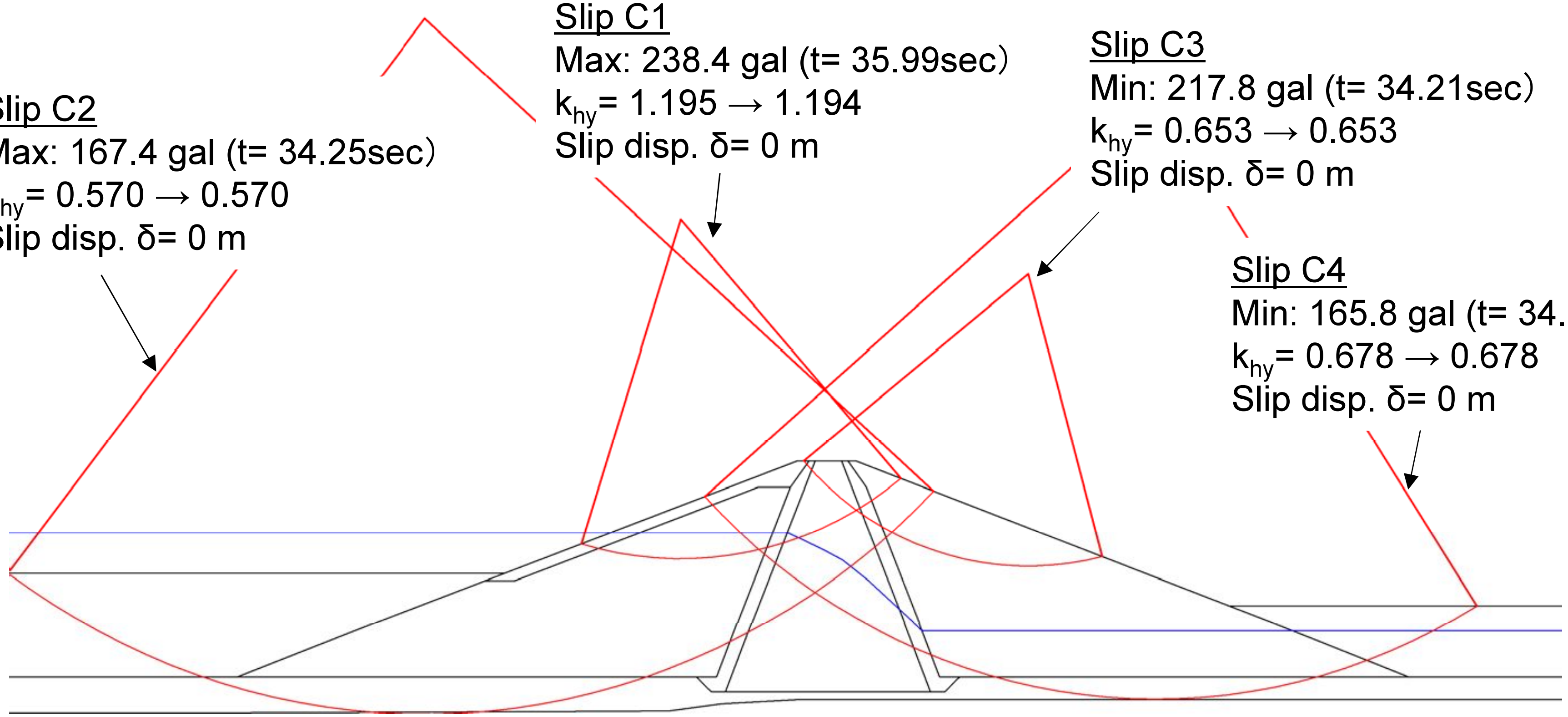






Newmark-D (based on undrained shear strength of saturated soil that deteriorates during seismic loading) :

Critical slip circles exhibiting smallest k_{hy} values



k_{hy}: yield horizontal seismic coefficient,
the value of k_h by which the global safety factor F_s becomes unity.

→ High initial yielding seismic coefficients for slips C1 to C4 with little or no degradation

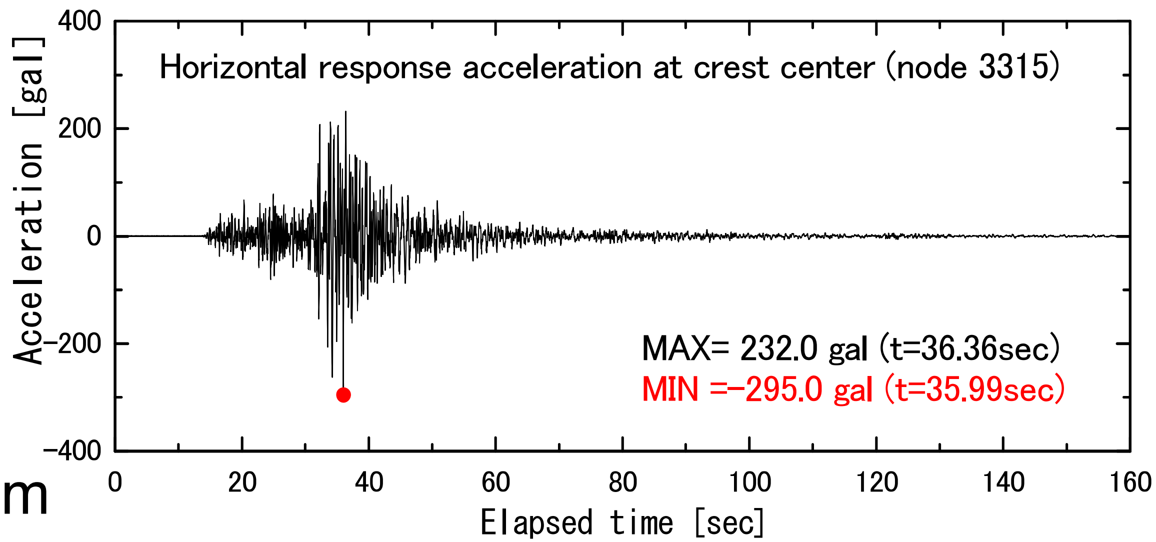
→ Zero sliding displacements along slips C1 to C4

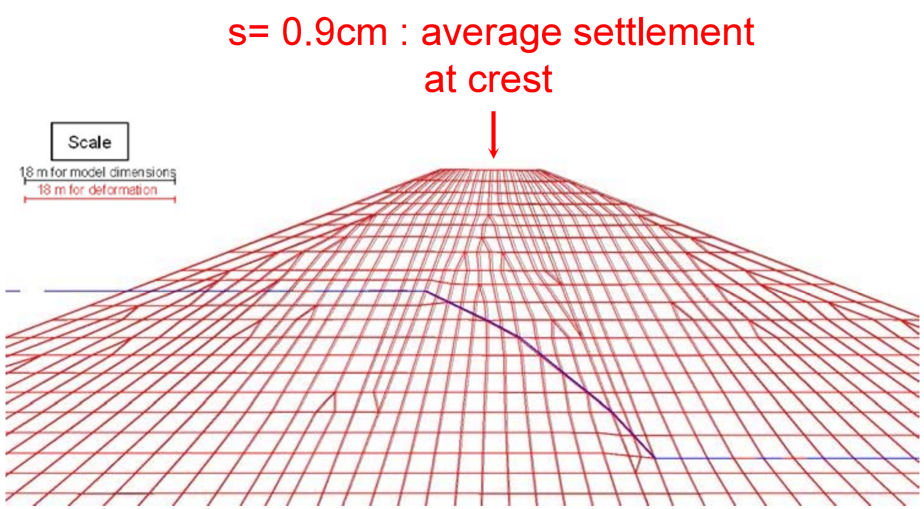
Analysis of residual continuous deformation by pseudo-static non-linear FEM analysis (based on deteriorating undrained properties)

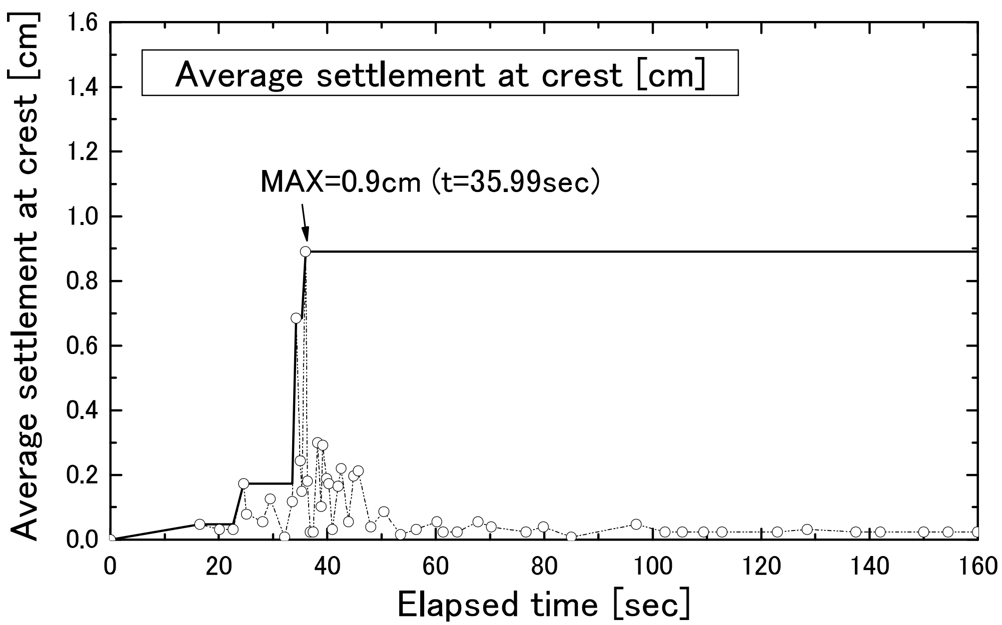
Fukushima Prefecture Offshore Earthquake (M7.3)

- Measurement: no discernible residual displacement at the dam crest

- Analysis: only a very small settlement computed at the dam crest



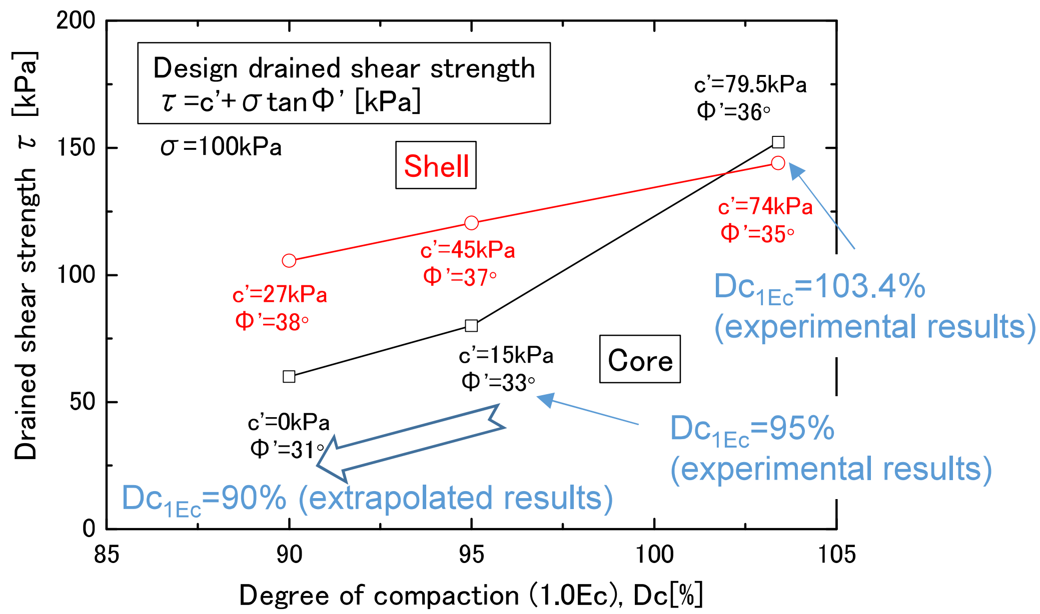




Seismic stability analysis using design drained shear strengths

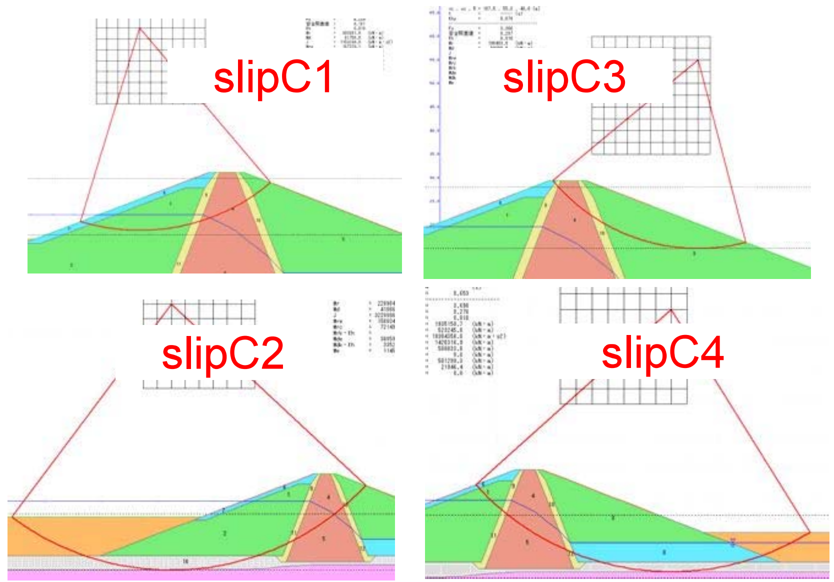
- Drained shear strengths of the dam materials compacted to
  - a) Dc_{1Ec}=95% (current design code for agricultural earth-fill dams in Japan) and estimated for
  - b) Dc_{1Ec}=90% (previous design code) by extrapolating existing data

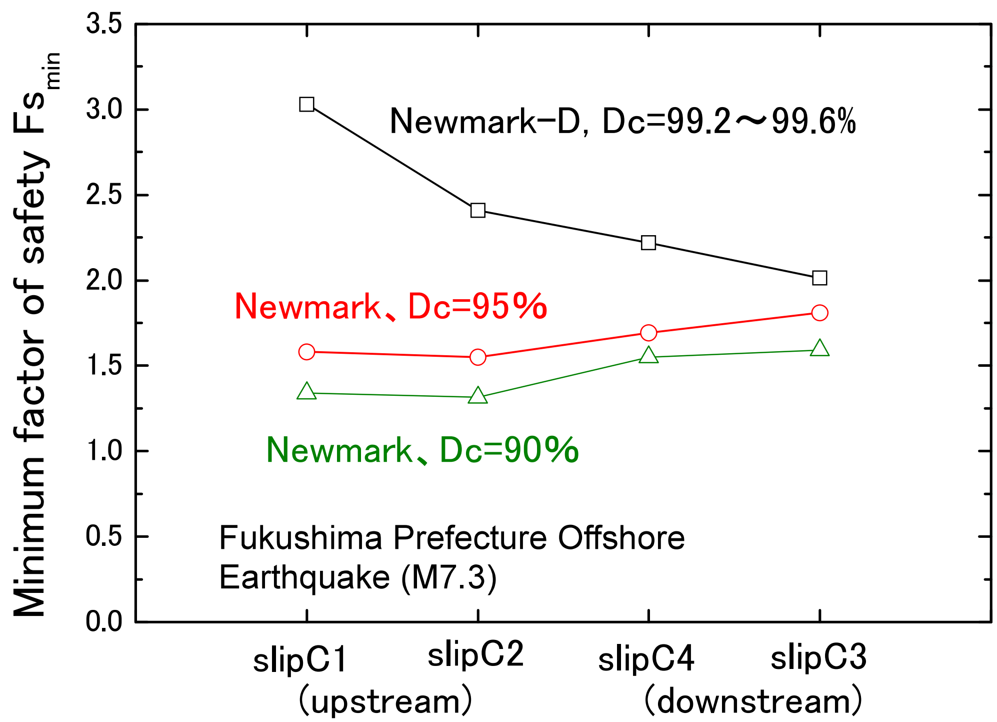
⇒ The dam stability is unduly under-estimated, in particular in case b), unlike Newmark-D method



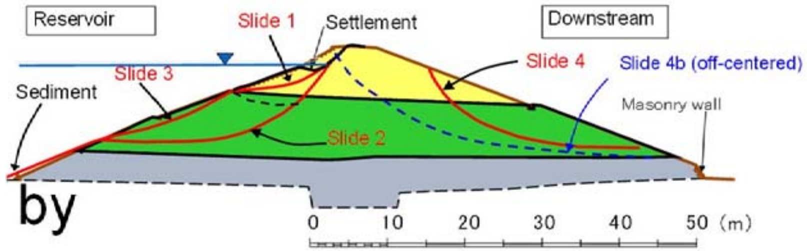
- Factor of safety Fs for slips showing the minimum Fs during earthquake

*slip location slightly differ for each Dc





CONCLUDING REMARKS - 1



Main & auxiliary Fujinuma dams collapsed by the 2011 Great East Japan Earthquake. They were reconstructed to much more stable ones (Spring 2017).

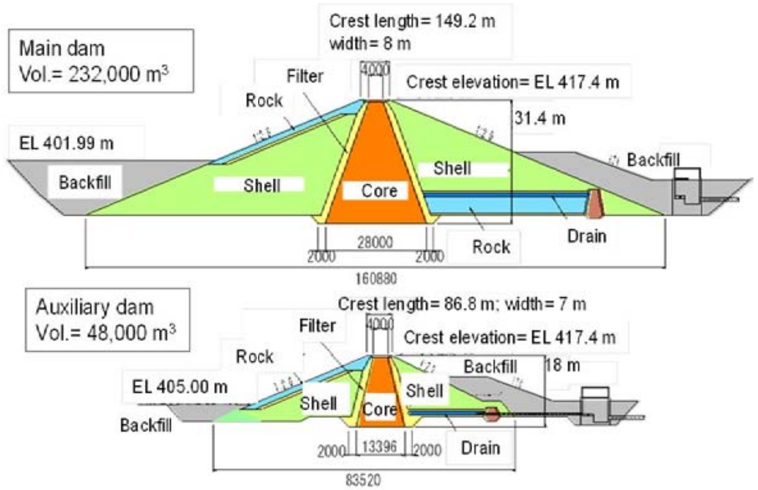
Collapsed old dams:

a) the compacted state was generally very poor;

b) the top fill consisted of **poorly compacted sandy soil**, with a possible significant reduction of undrained shear strength due to cyclic undrained loading during a prolonged strong seismic motion and fast erosion by over-topping flow.

New dams:

relevant fill materials were selected; and soil was compacted by monitoring and controlling water content, dry density and **the degree of saturation** of compacted soil".



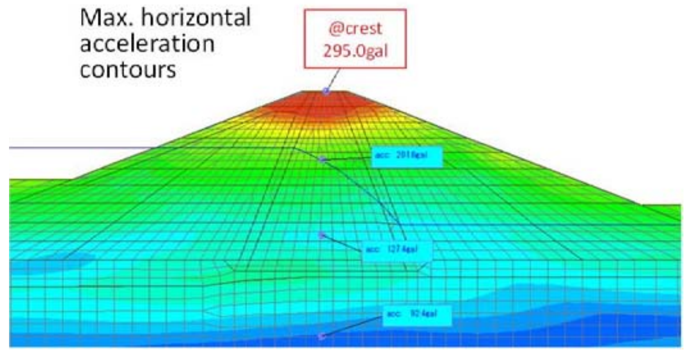
CONCLUDING REMARKS - 2

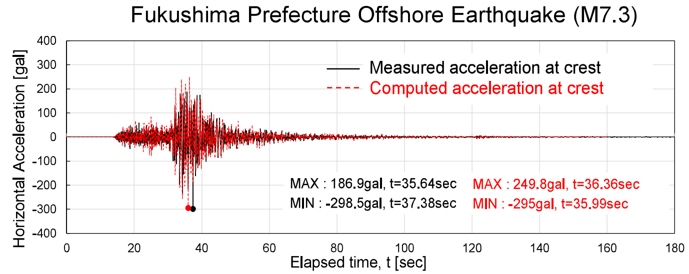
A rather strong aftershock (M7.3) took place in Feb. 13, 2021. The same set of analysis based on relevant undrained TC test results that could successfully simulate the collapse of the old dam was used.

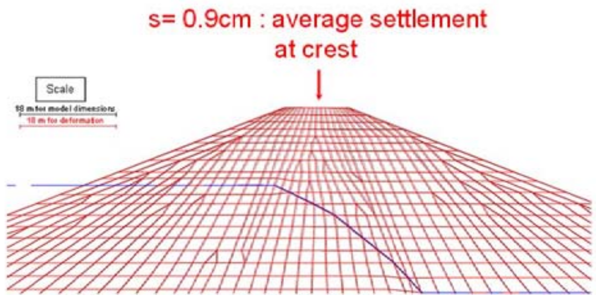
Seismic response analysis shows good agreement with the recorded accelerations. The stability and deformation analysis simulated very well observed very small residual deformation, further validating the analysis methods.

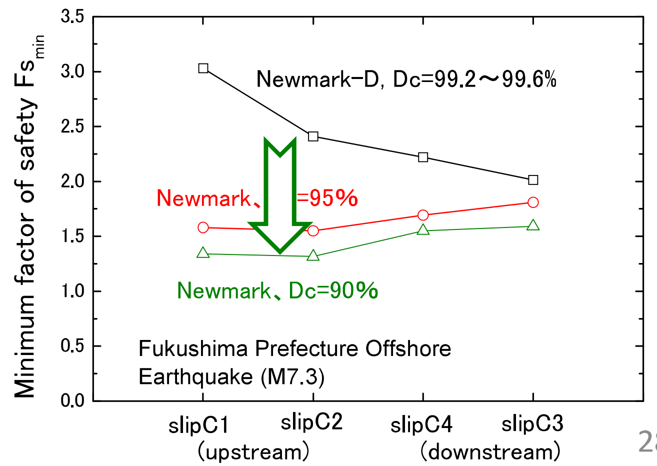
Stability analysis using the drained shear strengths of the dam materials estimated for a degree of compaction $(Dc)_{1Ec}$=90% (previous seismic design code for agricultural earth-fill dams in Japan) is too conservative (largely underestimating the factor of safety against sliding) compared to the analysis using the relevant undrained shear strengths, validated above.

Thank you for your attention!









FOR DISCUSSION

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東京大学 THE UNIVERSITY OF TOKYO

TOKYO UNIVERSITY OF SCIENCE since 1881

JARUS Think Globally, Act Locally.

茨城大学 Ibaraki University

NTC 総合建設コンサルタント NTCコンサルタンツ株式会社

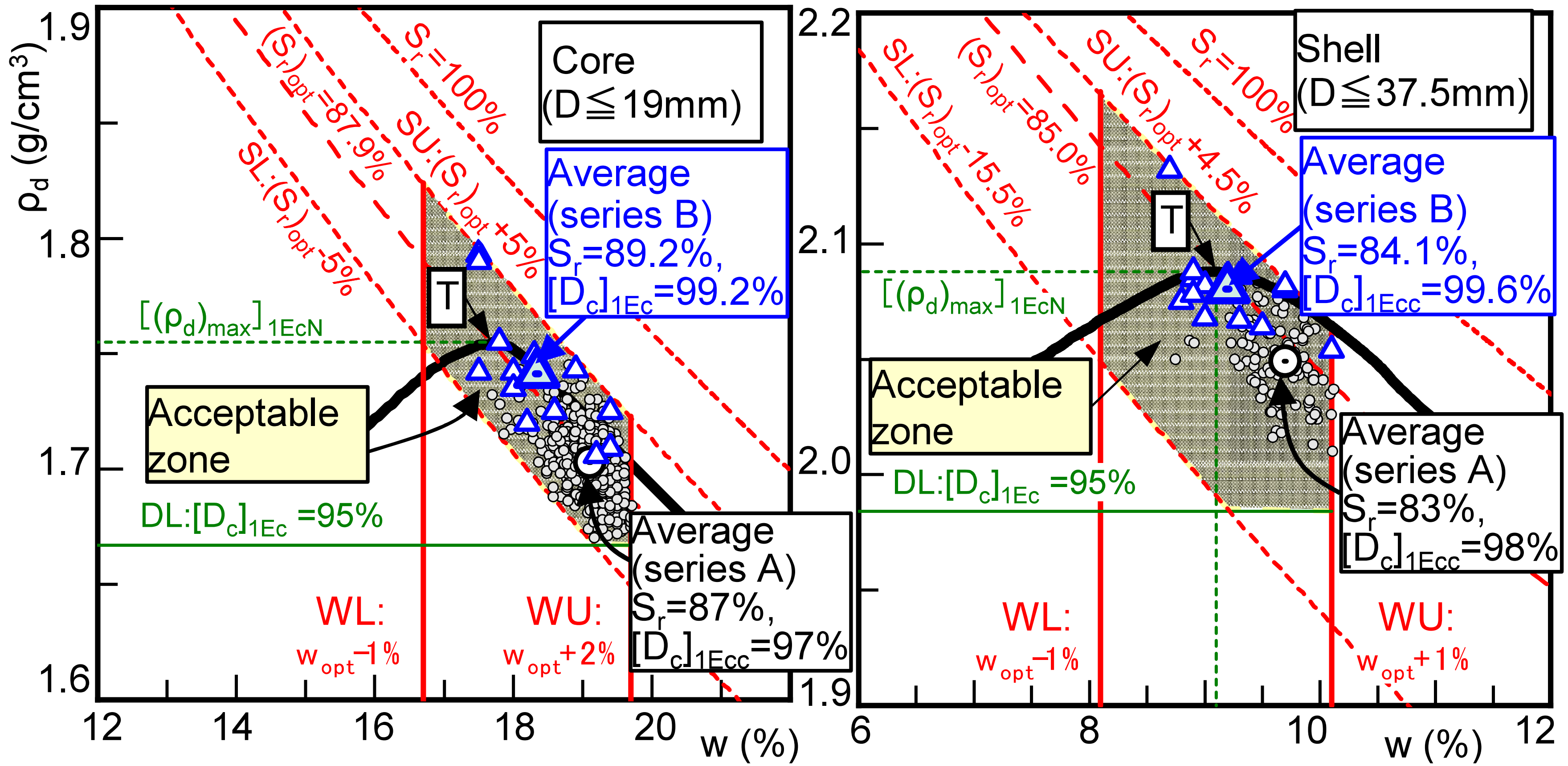
福島県 Fukushima Prefecture

○ Series A (performed routinely during construction): Laboratory compaction tests used the representative sample of each batch of 3,000 m^3

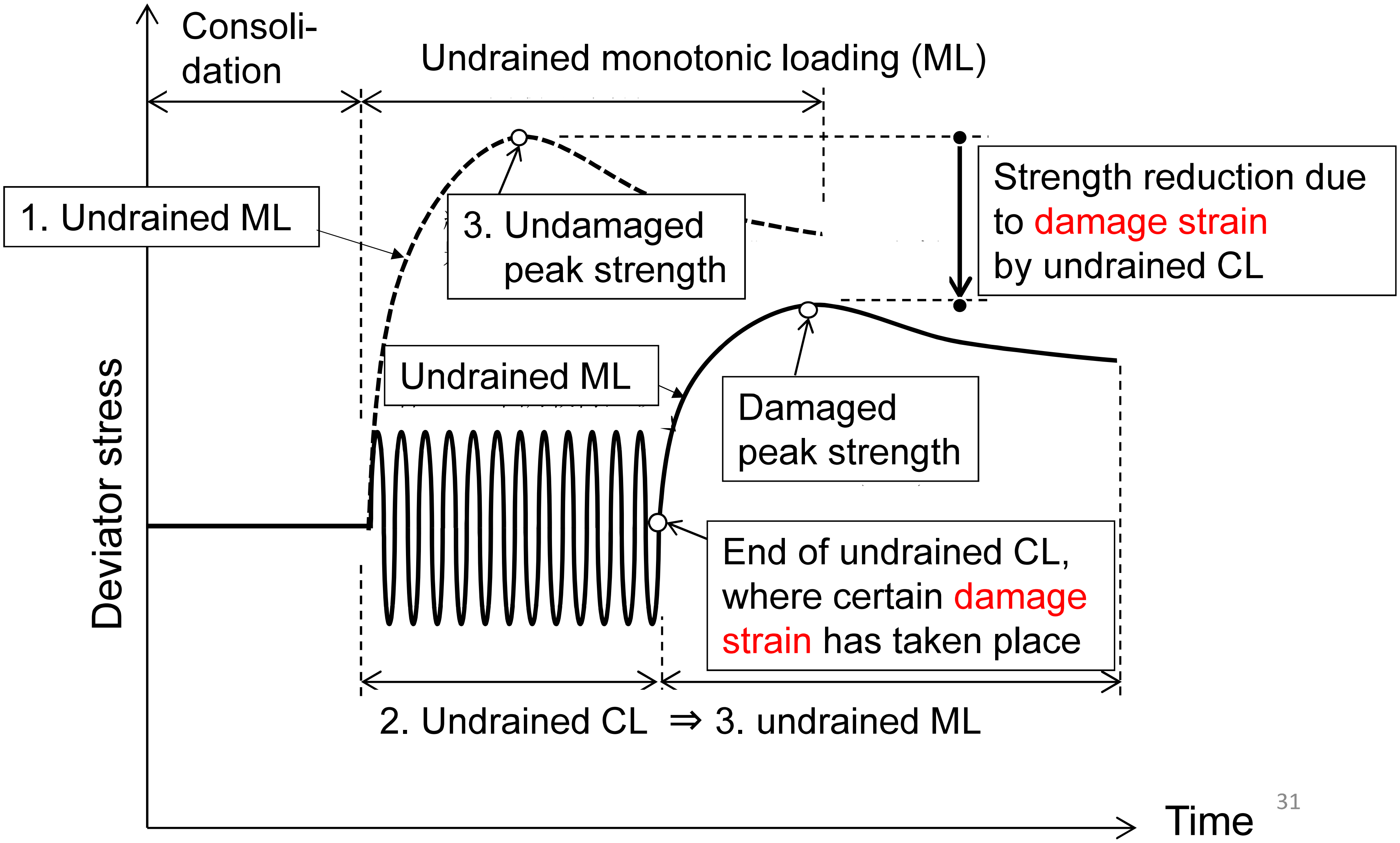
⇒ All the data points are located in the acceptable zone, but they are biased to wetter states, due to biased sampling for laboratory compaction tests.

△ Series B (performed at selected spots to accurately evaluate the compacted states): Lab. compaction tests used the samples retrieved from the spots where field values of ρ_d & w were measured. So, the results are much more reliable than series A.

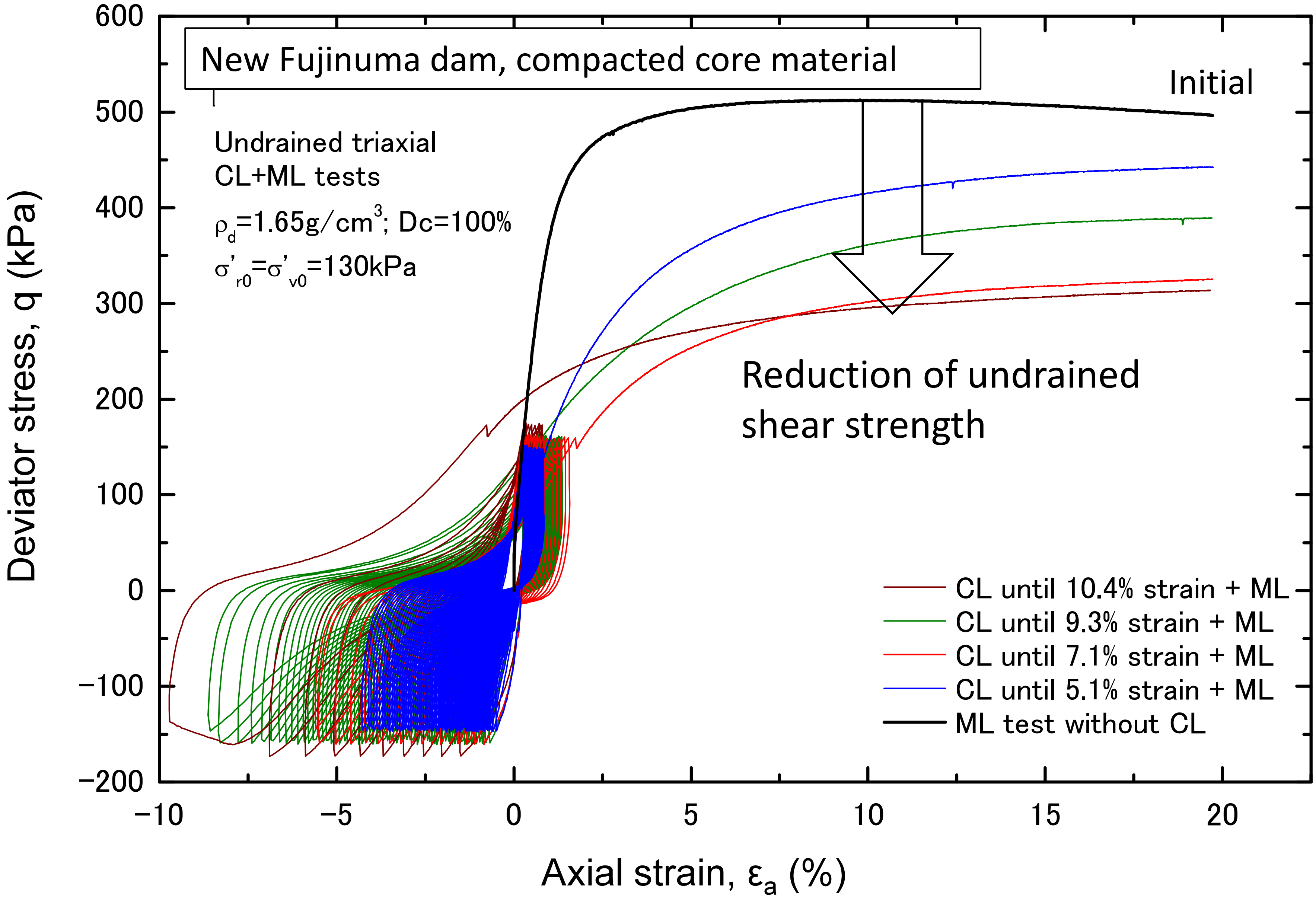
⇒ The compacted states are located very close to target T (as expected)



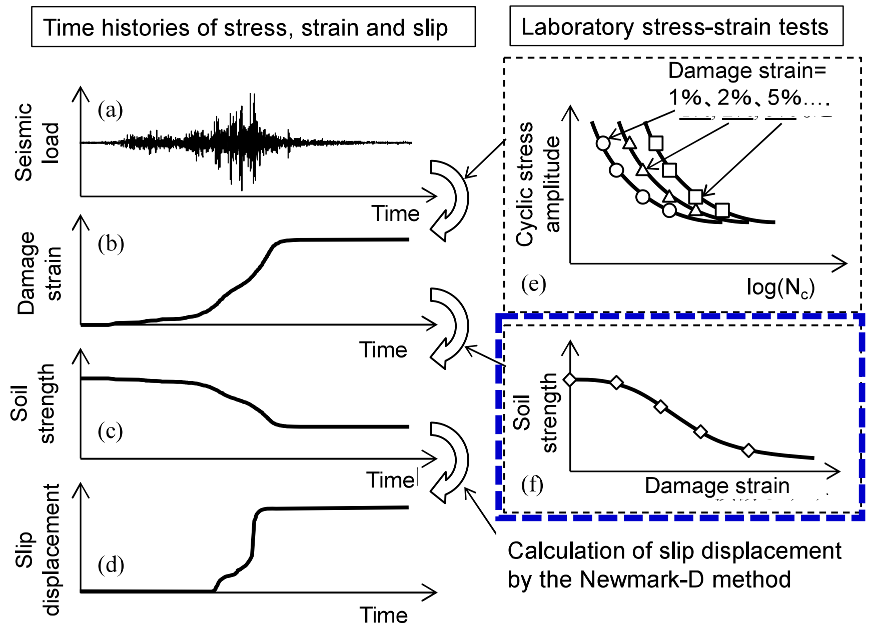
Laboratory stress-strain test to evaluate the reduction of undrained shear strength by damage strain that has taken place during preceding cyclic undrained loading

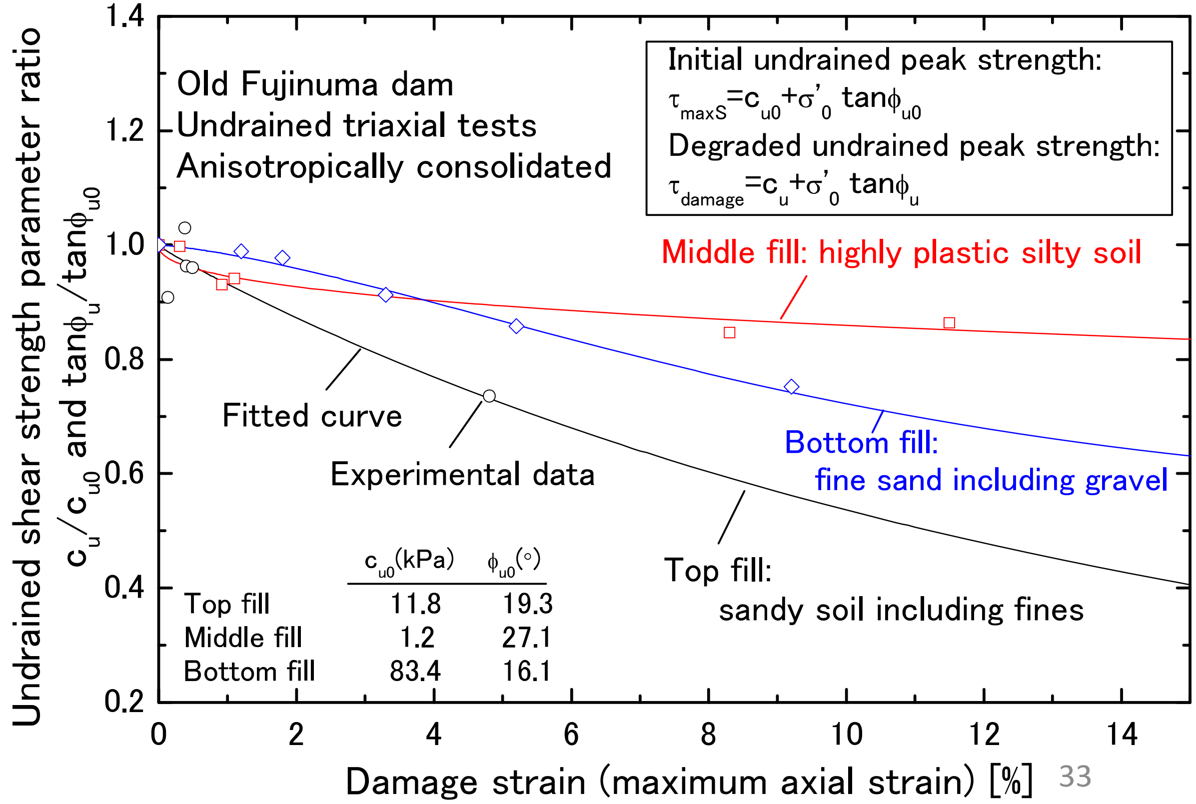


Reduction of undrained shear strength by damage strain that has taken place during preceding cyclic undrained loading

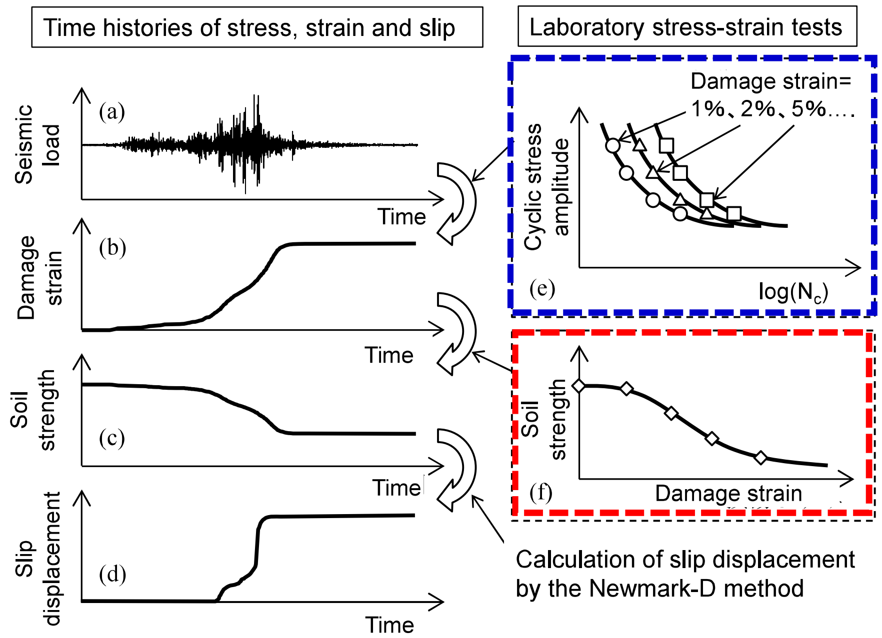


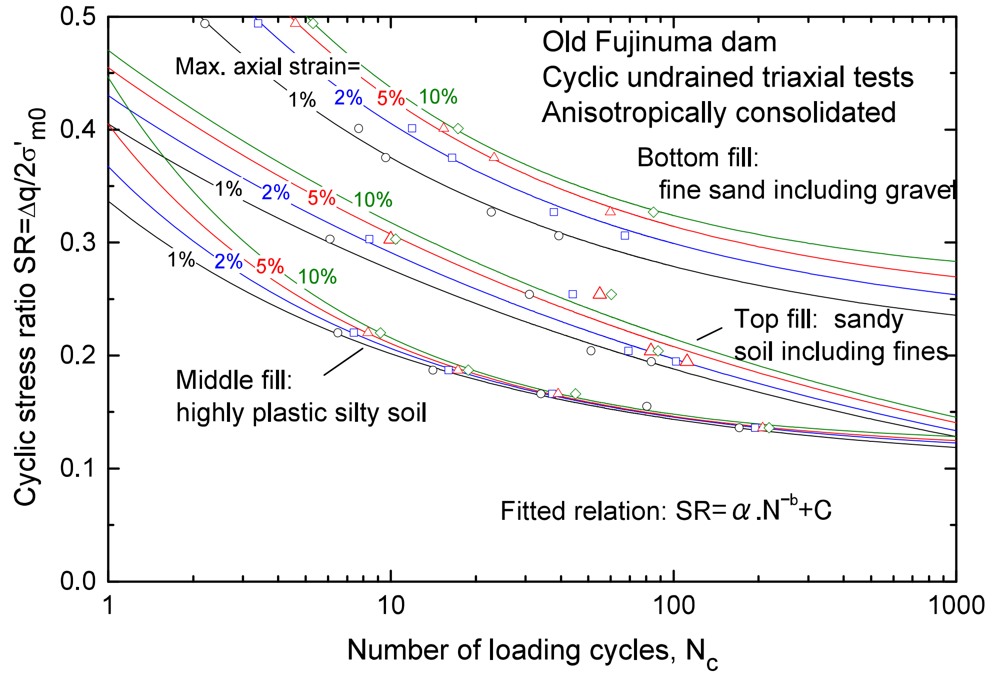
Reduction of undrained shear strength by damage strain that has taken place during preceding cyclic undrained loading, old Fujinuma dam

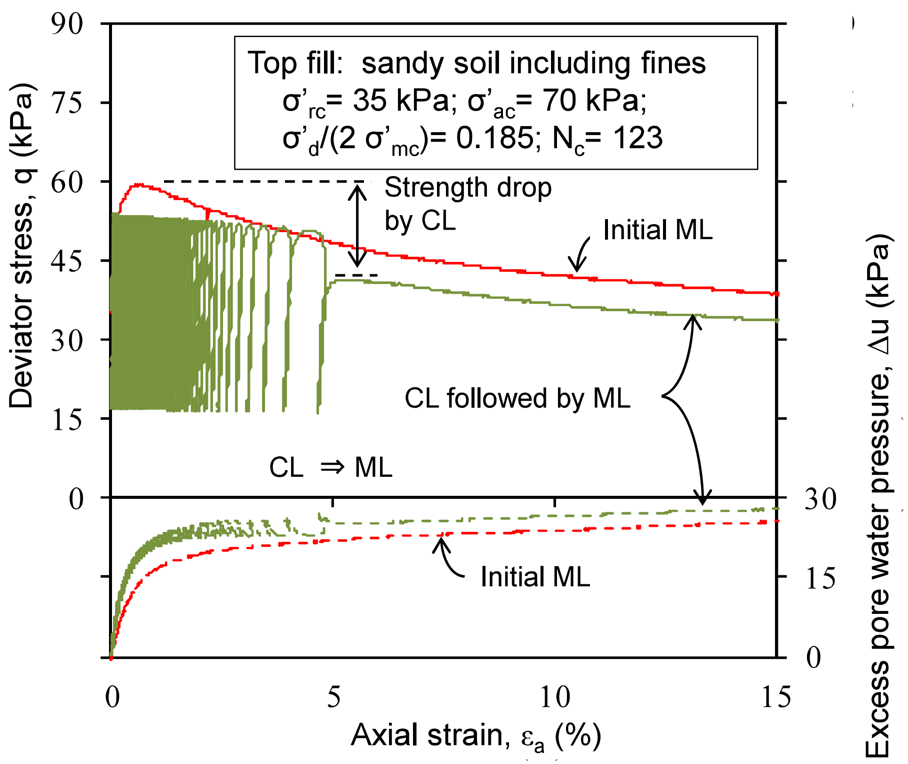




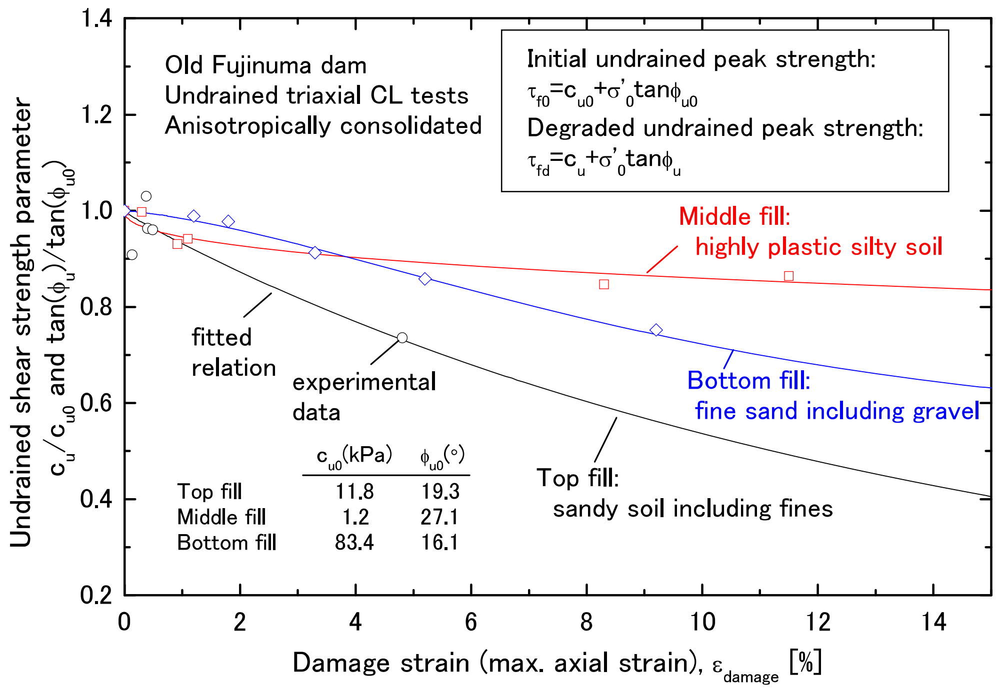
	c_{u0}(kPa)	ϕ_{u0}(°)
Top fill	11.8	19.3
Middle fill	1.2	27.1
Bottom fill	83.4	16.1

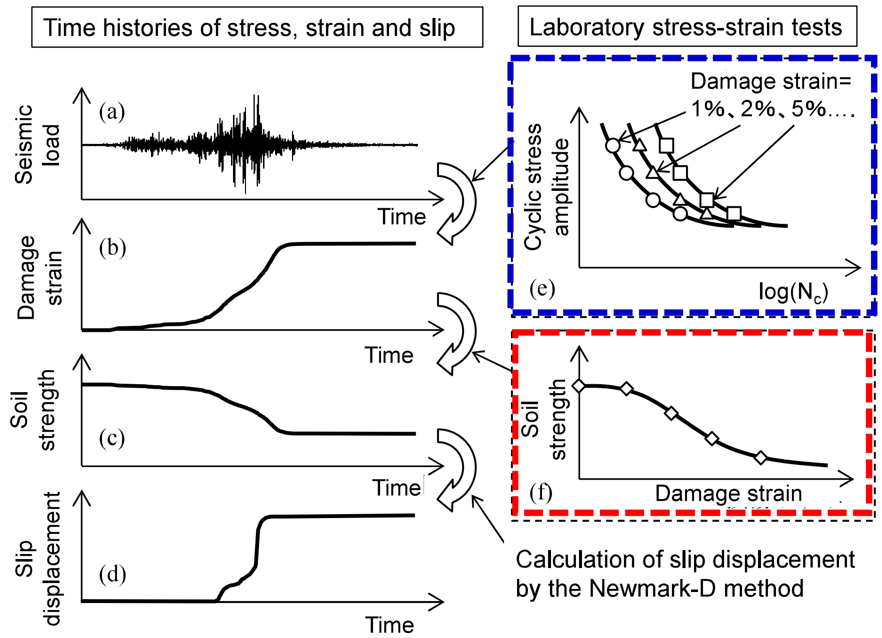


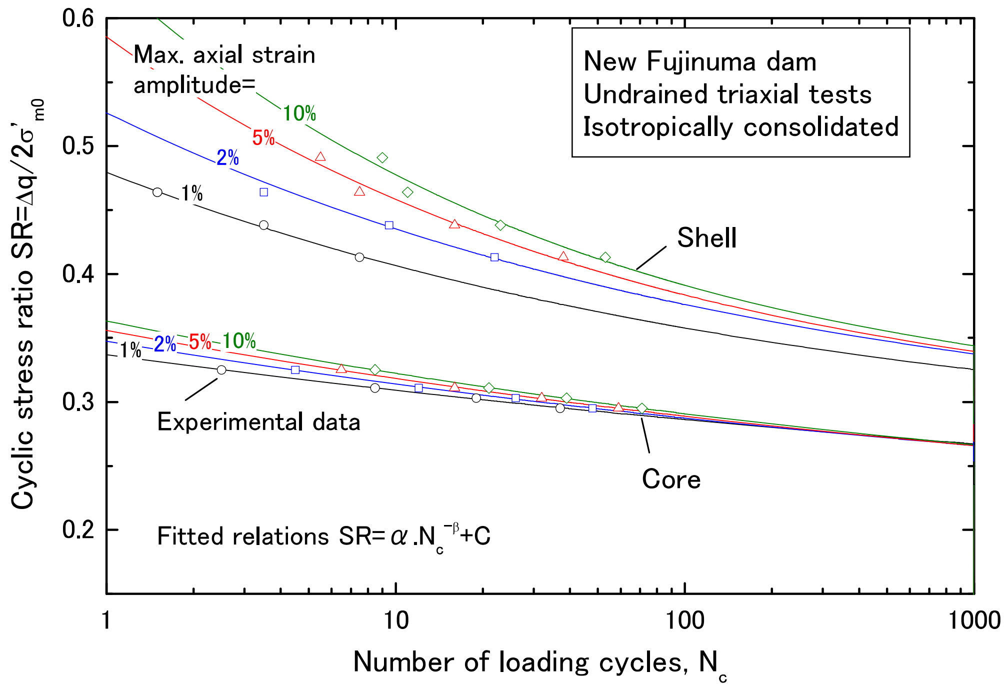




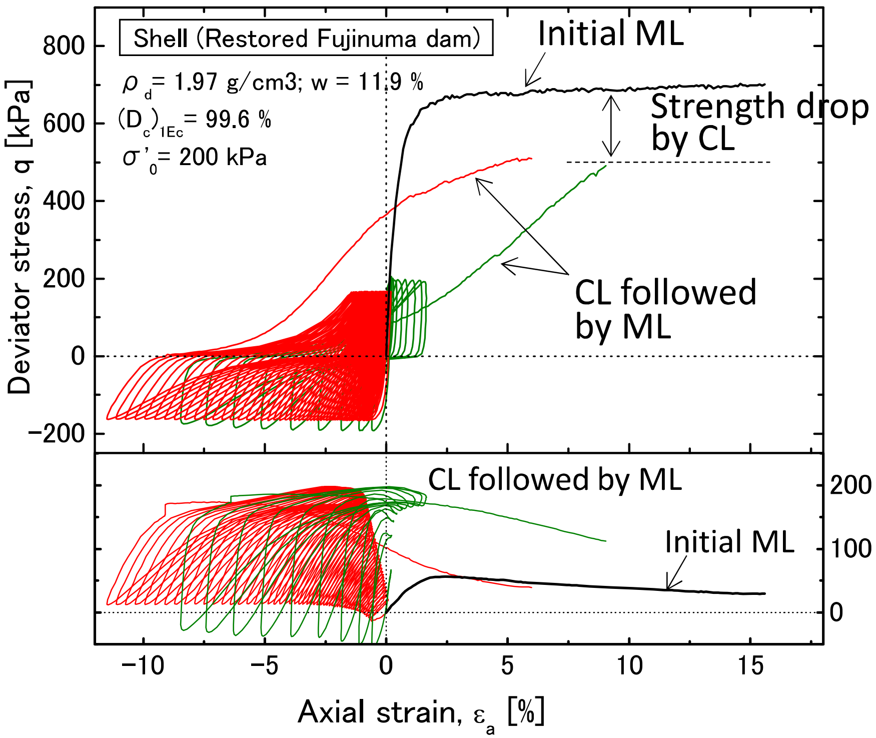
Reduction of undrained shear strength by preceding cyclic undrained loading : OLD DAM

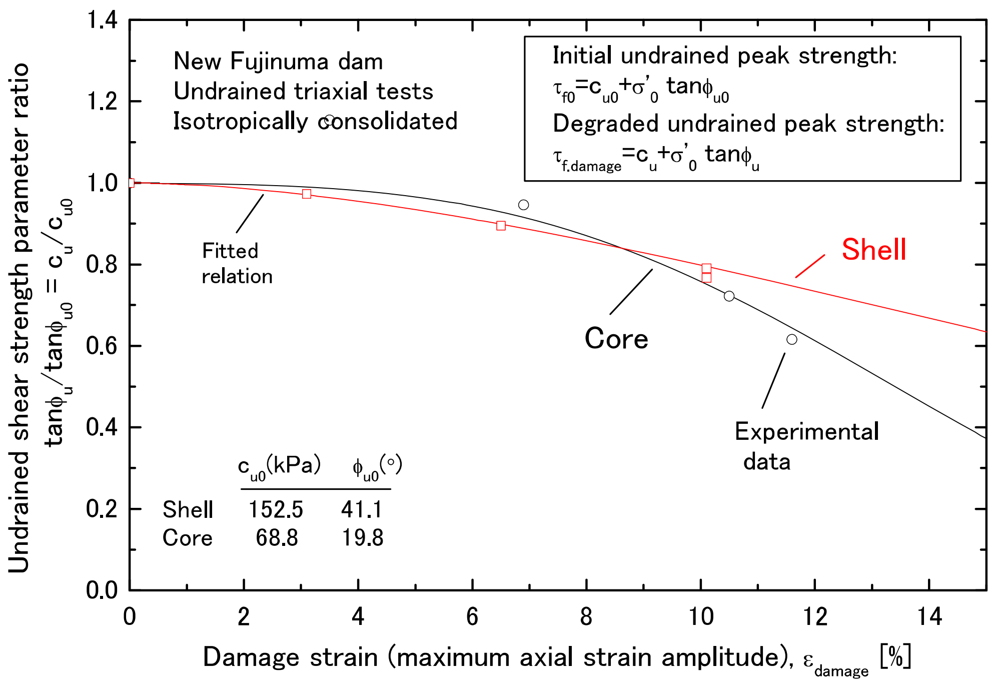




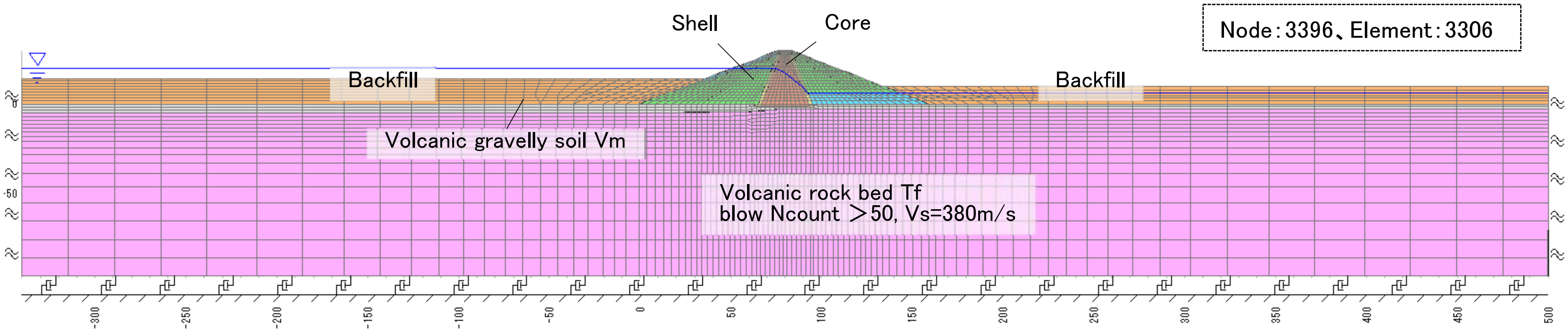


Reduction of undrained shear strength by preceding cyclic undrained loading : NEW DAM





2D FEM seismic response analysis (equivalent linear, frequency domain)



Material properties based on laboratory test results on samples retrieved from site after restoration

Layer	Basic properties			Dynamic properties						
	blow count N	wet density γt (kN/m3)	sat. density γsat (kN/m3)	Swave vel. Vs (m/s)	Pwave vel. Vp (m/s)	Shear modulus G0 (kN/m2)	Shear modulus G0 stres dependency A (kN/m²)	Shear modulus G0 stres dependency B	Poisson ratio νd	Non linear properties (G/G0・h～γ)
Shell	–	21.60	22.00	–	–	–	11,232	0.623	0.450	from undrained cyclic triaxial tests
Core	–	20.10	20.30	–	–	–	12,526	0.515	0.450	from undrained cyclic triaxial tests
Rock	–	20.50	23.00	–	–	–	11,232	0.623	0.450	same as shell
Filter	–	19.90	22.30	–	–	–	12,526	0.515	0.450	same as core
Bakfill	–	21.60	22.00	–	–	–	11,232	0.623	0.450	same as shell
Volcanic gravel Vm	19	17.00	17.40	287	1156	150,000	–	–	0.441	PWRI(alluvial sand)
Rock bed Tf	>50	19.60	19.60	438	1480	380,000	–	–	0.449	linear, damping h=2%

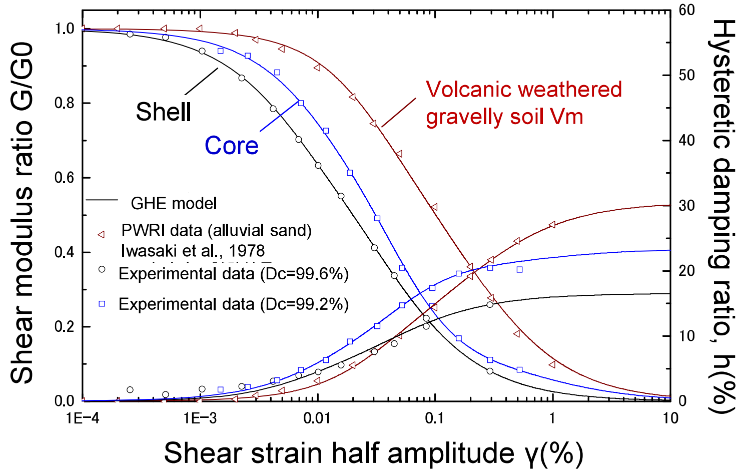
Note) $G_0 = A \times \sigma'^{B}_{m}$

GHE model (General Hyperbolic Equation, Tatsuoka&Shibuya,1991)

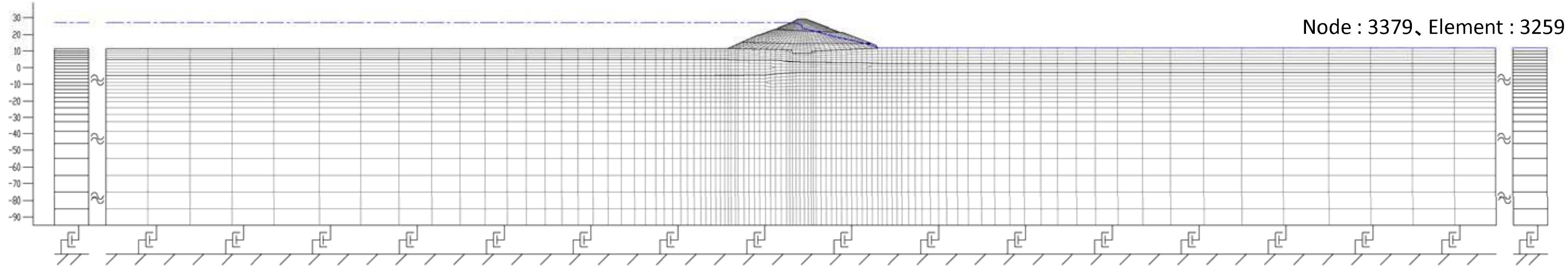
$$\frac{G}{G_{max}} = \frac{1}{1/C_1 + x/C_2} \qquad C_1(x) = \frac{C_1(0)+C_1(\infty)}{2} + \frac{C_1(0)-C_1(\infty)}{2}\cdot\cos\left\{\frac{\pi}{\alpha/x+1}\right\}$$

$$h = h_{max}\cdot\left(1-\frac{G}{G_{max}}\right)^{\beta_1} \qquad C_2(x) = \frac{C_2(0)+C_2(\infty)}{2} + \frac{C_2(0)-C_2(\infty)}{2}\cdot\cos\left\{\frac{\pi}{\beta/x+1}\right\}$$

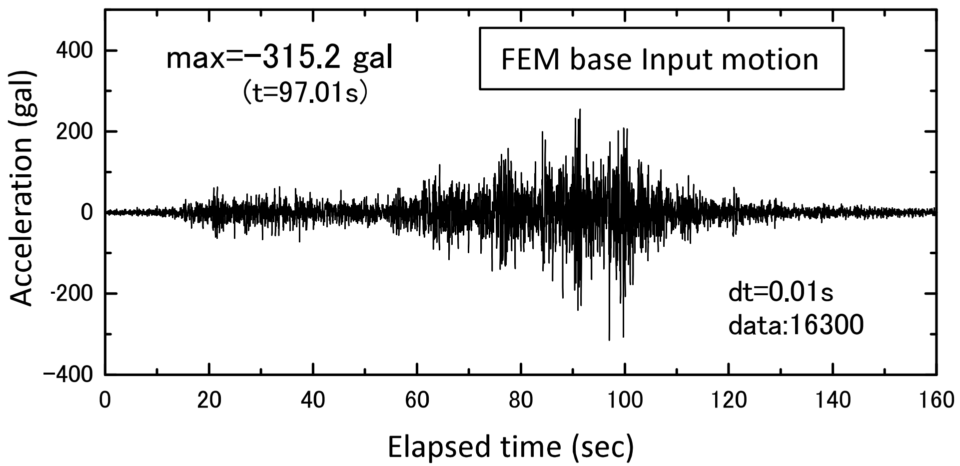
*Tatsuoka,F. and Shibuya, S.: "Deformation characteristics of soils and rocks from field and laboratory tests", Proc. 9th Asian Regional Conference on Soil Mechanics and Foundation Eng.,Bangkok, Vol.II, pp101-170, 1991.

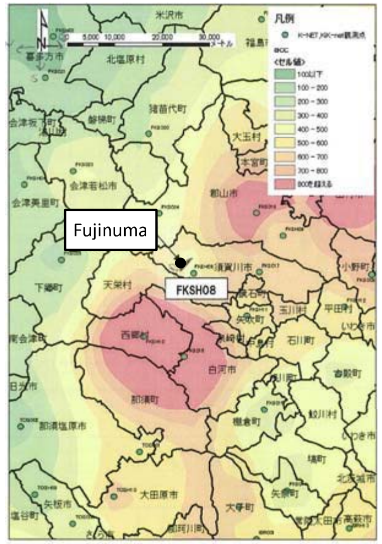


⇨ 2DFEM seismic response analysis (equivalent-linear)

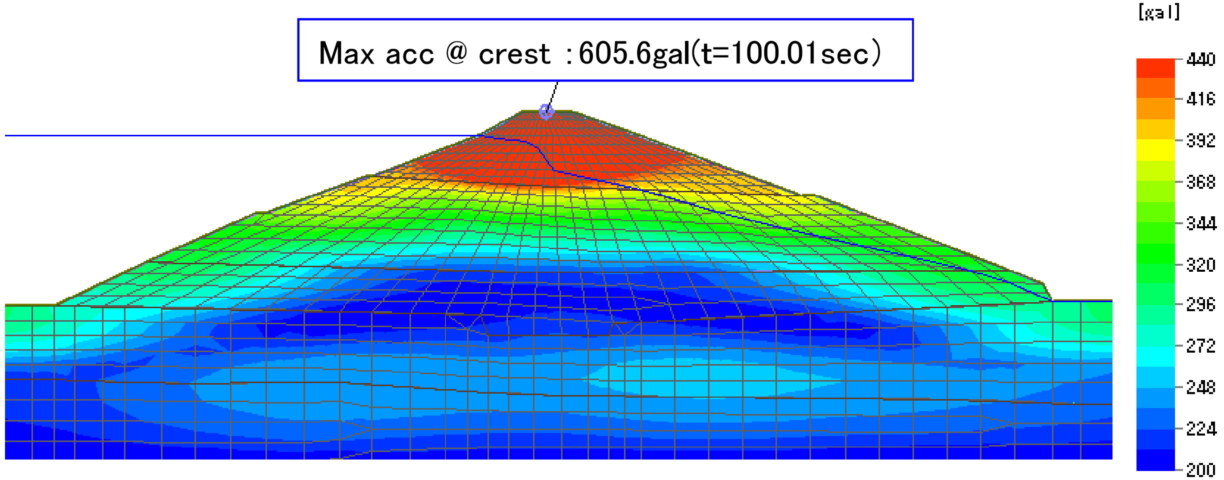


Kik-net Naganuma station (FKSH08)

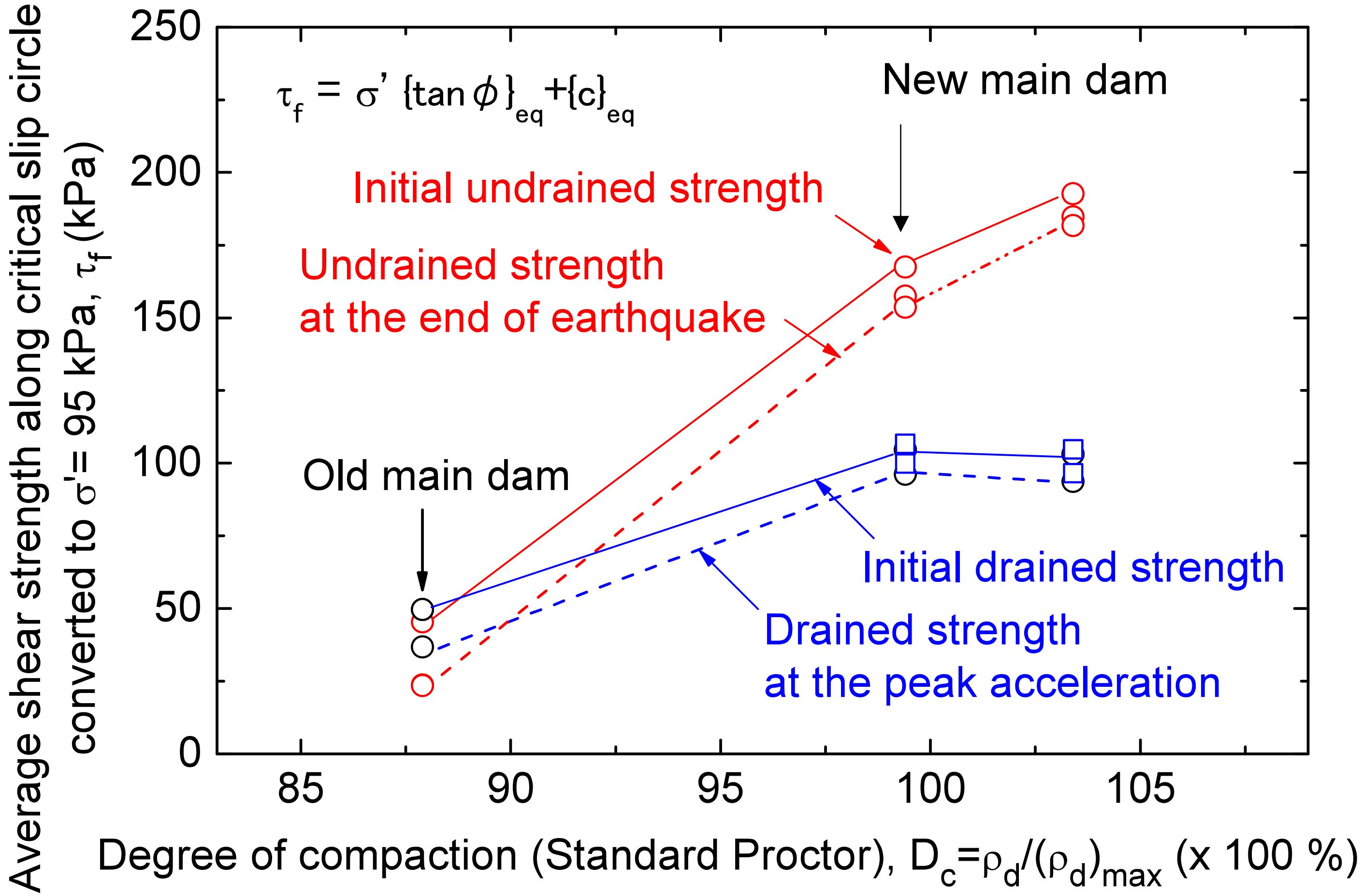


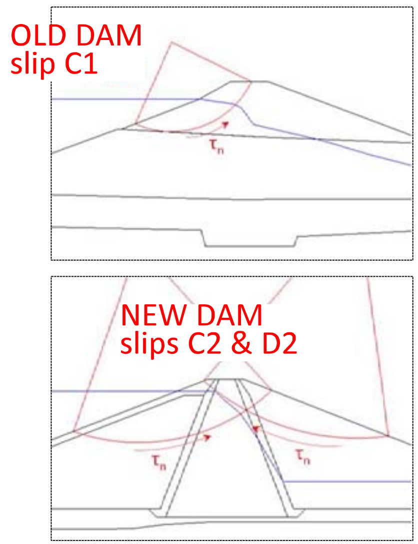


Ground input motion measured at Naganuma
↓
deconvoluted to rock bed (Vs=450m/s)



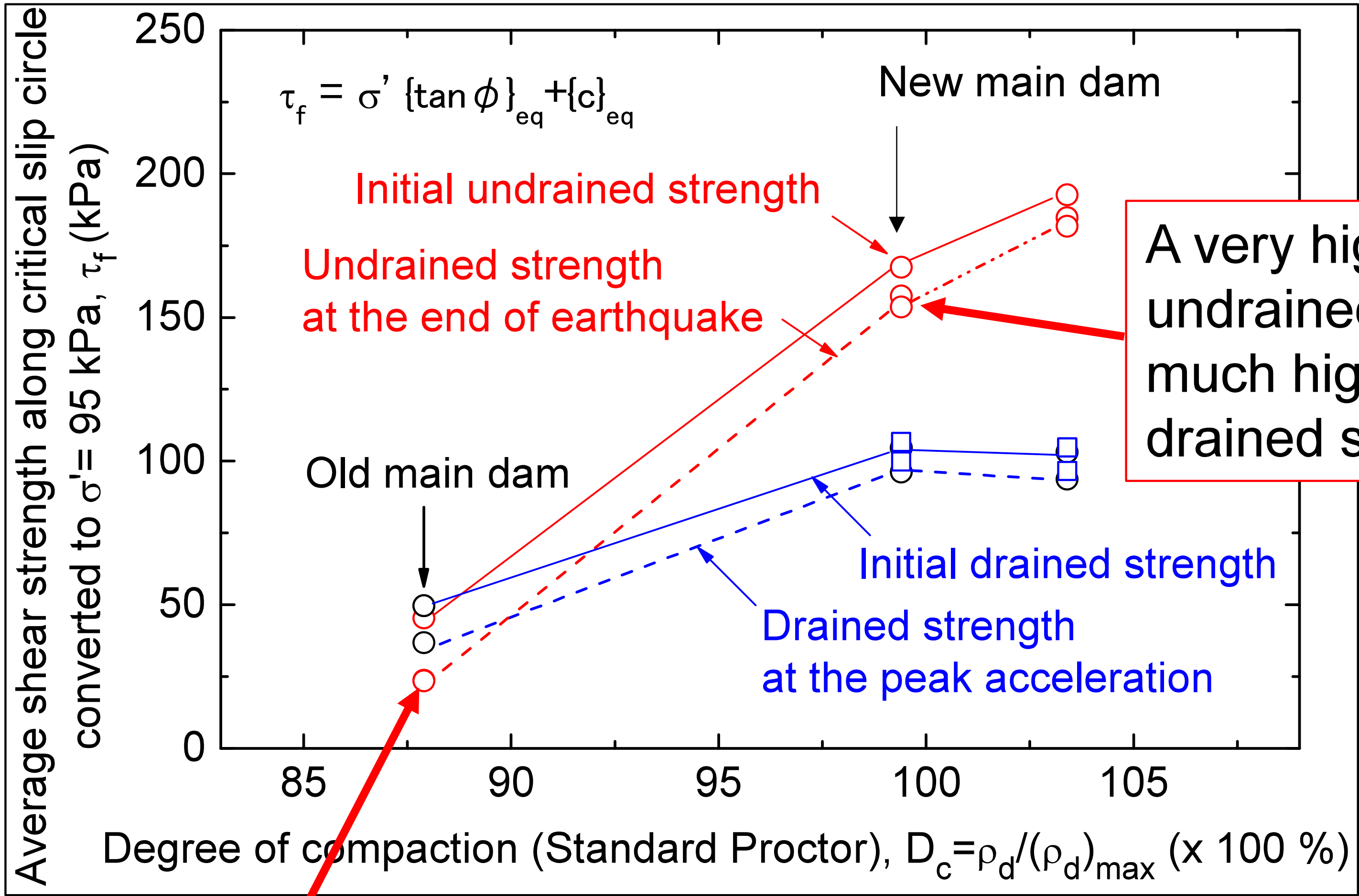
Average shear strength vs. D_c relation in the top zone of dam



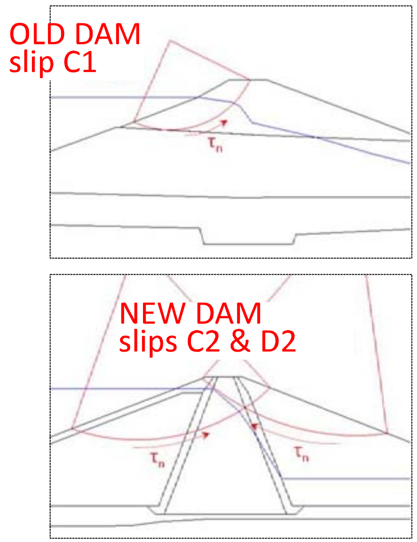


During an earthquake, the undrained shear strength decreases by cyclic undrained loading, while the drained shear strength decreases by a decrease in the normal stress along the slip circle by seismic inertia.

Average shear strength vs. D_c relation in the top zone of dam



A very high ultimate undrained shear strength, much higher than the drained shear strength



A very low ultimate undrained shear strength, much lower than the drained shear strength