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Qualification of dynamic analyses of dams and their equipments
and of probabilistic assessment seismic hazard in Europe
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Session :

ELASTO-PLASTIC DYNAMIC RESPONSE OF FILL-TYPE DAMS: TOTAL STRESS AND EFFECTIVE STRESS ANALYSES

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OUTLINE

INTRODUCTION

Simple strain softening material model for soil is used with the features of non-associated flow characteristics, post-peak strain softening, and strain-localization into a shear band. Then a kinematic hardening model considering the cumulative deformation by cyclic loading is developed based on the soil model of isotropic strain-hardening-softening property.

Total stress elasto-plastic constitutive model is rather simple and robust for application to a dynamic response analysis of fill-type dams. A cumulative damage concept for simple elasto-plastic model is effective by using the results of cyclic triaxial tests of saturated soils.

Dynamic progressive failure analysis of a small dry sand dam on shaking table is carried out. The computed acceleration and displacement at the crest of model dam is compared to the measured one. The computation of real fill-type dam is also carried out by total stress elasto-plastic model and effective stress constitutive model by taking into account the pore water build-up.

YIELD & PLASTIC POTENTIAL FUNCTION

The yield function (f) and the plastic potential function ($\boldsymbol{\Phi}$) are given by

$$f = \alpha I_1 + \frac{\overline{\sigma}}{g(\theta_L)} - K = 0 \qquad K = \frac{6c\cos\phi}{\sqrt{3}(3-\sin\phi)}$$

$$\Phi = \alpha' I_1 + \overline{\sigma} - K = 0 \qquad \alpha = \frac{2\sin\phi}{\sqrt{3}(3-\sin\phi)}$$

I_1 : primary invariant of stress

$\overline{\sigma}$: second invariant of deviatoric stress

YIELD FUNCTION

In case of Mohr-Coulomb model, $g(\theta_L)$ takes the following form

$$g(\theta) = \frac{3 - \sin\phi}{2\sqrt{3}\cos\theta - 2\sin\theta\sin\phi}$$

ϕ : mobilized friction angle

θ : Lode angle (third invariant of deviatoric stress)

SIMPLE STRAIN SOFTENING CONSTITUTIVE MODEL

Frictional softening is given by next function

$$\alpha(\kappa) = \alpha_p + \frac{\alpha_1 \kappa}{B + \kappa} \qquad \alpha_1 = -(\alpha_p - \alpha_R)$$

Cohesion softening is given by next function.

$$K(\kappa) = K_p + \frac{K_1 \kappa}{D + \kappa} \qquad K_1 = -(K_p - K_R)$$

Dilatancy is reduced by next equation

$$\alpha'(\kappa) = \alpha'_p (1 - \frac{\kappa}{C + \kappa})$$

κ is plastic parameter, *B,C,D* are constants for softening function

ELASTIC PROPERTIES

Shear modulus *G* and damping ratio *h* are given by Hardin-Drnevich quation

$$G = \frac{G_0}{1 + \dfrac{\gamma}{\gamma_r}} \qquad h = \frac{\dfrac{\gamma}{\gamma_r}}{1 + \dfrac{\gamma}{\gamma_r}} h_{\max}$$

$$G_0 = G_E \frac{(2.17 - e)^2}{1 + e} \sigma_m^{0.4} \qquad K = \frac{2(1+\nu)}{3(1-2\nu)} G$$

γ is shear strain, γ_r is reference shear strain, ν is Poisson's ratio, *e* is void ratio and , G_E , $h_{\max}$ are empirical constants

FRICTIONAL HARDENING-SOFTENING FUNCTIONS

$$\alpha(\kappa) = \left(\frac{2\kappa^l \varepsilon_f^l}{\kappa^{2l} + \varepsilon_f^{2l}} \right)^m \alpha_p \quad (\kappa \le \varepsilon_f)$$:hardening-regime

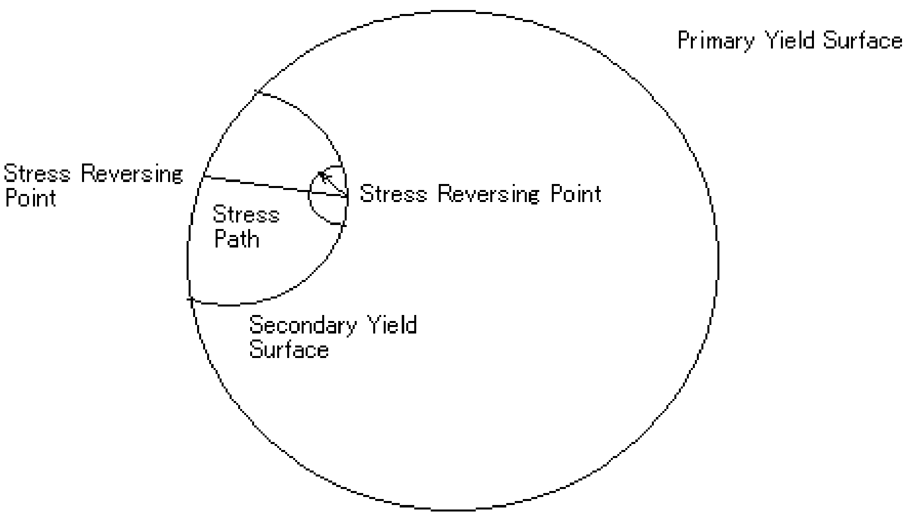
$$\alpha(\kappa) = \alpha_r + (\alpha_p - \alpha_r)\exp\left\{ -\left(\frac{\kappa - \varepsilon_f}{\varepsilon_r} \right)^2 \right\} \quad (\kappa \ge \varepsilon_f)$$: softening-regime

$$\phi_p(\deg) = \left\{ 59.47(1.5 - e) - 10(1 - e)\log\left\{ \frac{\sigma_3}{(\sigma_3)_0} \right\} \right\} g_R(\delta)$$: Tatsuoka et al.

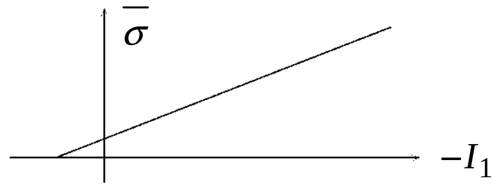
$$(\sigma_3)_0 = 4(1 - e)p_a (p_a = 98kPa)$$

$$d\varepsilon_{ij} = d\varepsilon_{ij}^e + sd\varepsilon_{ij}^p \qquad s = F_b / F_e$$: Shear band effect

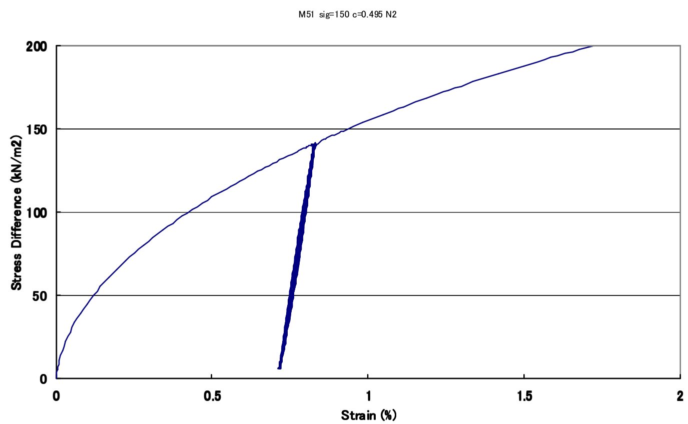
KINEMATIC HARDENING CONSTITUTIVE MODEL ON π PLANE



Mohr-Coulomb model takes pyramid shape



Upper part of $-I_1$ and $\overline{\sigma}$ relation



Calculated stress strain relation of triaxial test

KINEMATIC HARDENING MODEL WITHIN BOUNDING SURFACE

$$\alpha_{iy}(\kappa') = \left(\frac{2\kappa'^{l}(\varepsilon_f / a_f)^l}{\kappa'^{2l} + (\varepsilon_f / a_f)^{2l}} \right)^m \alpha_p$$

$$\alpha(\kappa') = \alpha_{iy}(\kappa')(1 + e^n),\ e = \eta / R$$

$$\alpha_{id}(\kappa') = \alpha - \alpha_p(e - 1.0) \qquad e \geq 1.0$$

$$\alpha_{id}(\kappa') = \alpha \qquad e < 1.0$$

$$\sin\psi' = \frac{3\sqrt{3}\alpha_{id}}{2 + \sqrt{3}\alpha_{id}}$$

SOLUTION OF DYNAMIC ELASTO-PLASTIC FINITE ELEMENT ANALYSIS

Element type

Very few element types can avoid the shear locking and dilatancy locking

One point integration of 4 nodes iso-parametric element with hour-glass control (8 nodes in three dimension element)

15 nodes triangular element (PLAXIS)

Nonlinear solution method to avoid the accumulation of error

Dynamic equilibrium iteration is absolutely necessary (implicit method)

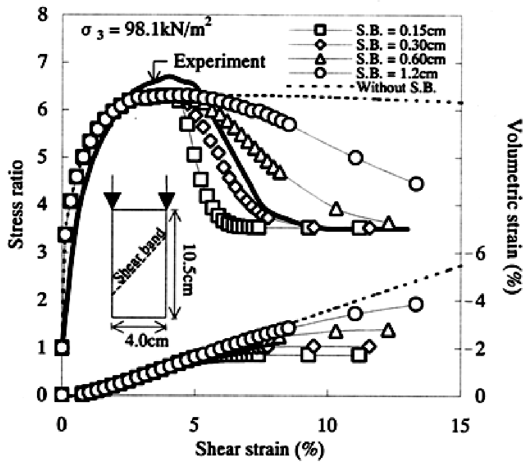
Return mapping method by explicit method is effective

Strain softening with shear banding

Objectivity of analysis (mesh independancy) : incorporating a characteristics length of shear band in the material modeling based on physical experimental observations

Simple check

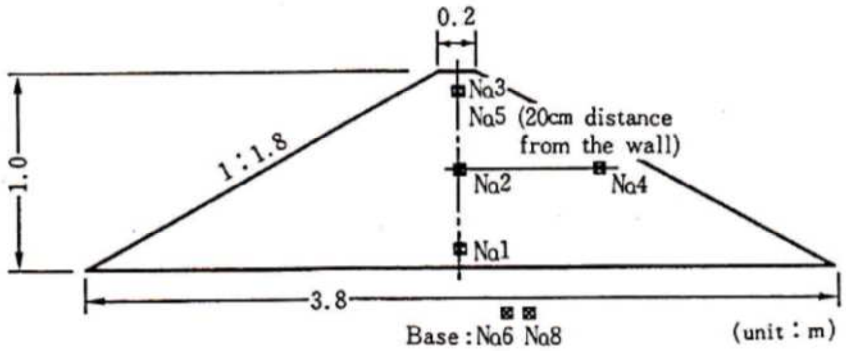
Static 2D footing limit load analysis is a good benchmark regarding above remarks



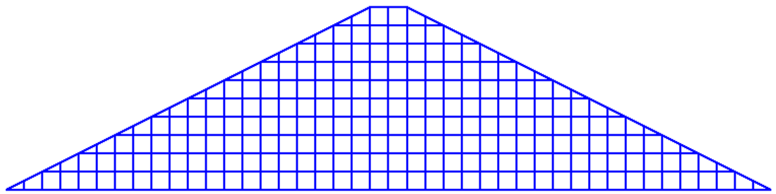
OUTLINE

1 INTRODUCTION
2 MATERIAL MODEL FOR CYCLIC BEHAVIOR OF SOIL
2.1 *Elasto-plastic model*
2.2 *Kinematic hardening model*
3 SHAKING TABLE TESTS OF EMBANKMENT DAMS AND DYNAMIC ANALYSES
3.1 *Outline of a model dam experiment*
3.2 *Dynamic analysis of embankment dam by simple constitutive model*
3.3 *Comparison of two and three dimensional dynamic analysis*
4 TOTAL STRESS ELASTO-PLASTIC DYNAMIC ANALYSIS
4.1 *Consolidated-undrained cyclic triaxial test*
5 OOGAKI DAM ELASTO-PLASTIC DYNAMIC ANALYSIS
5.1 *Oogaki Dam dynamic analysis by simple elasto-plastic model*
5.2 *Oogaki dam dynmaic analysis by kinematic hardening elasto-plastic model*
6. SUMMARY

EMBANKMENT DAM MODEL AND LOCATION OF ACCELEROMETERS

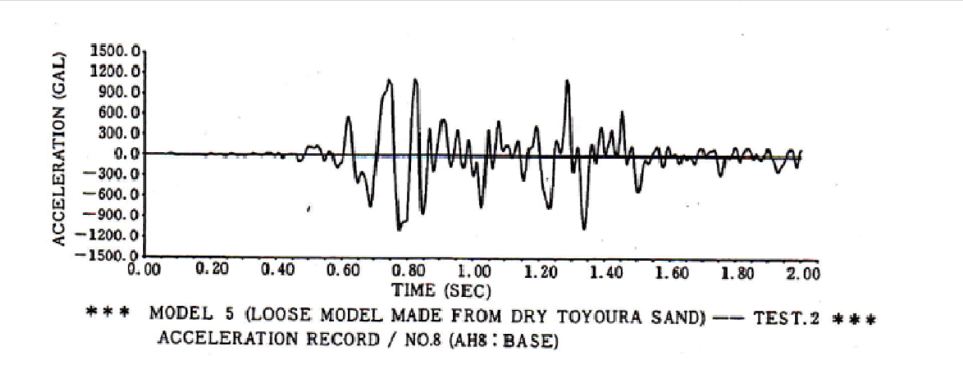


Embankment dam model and the location of accelerometers

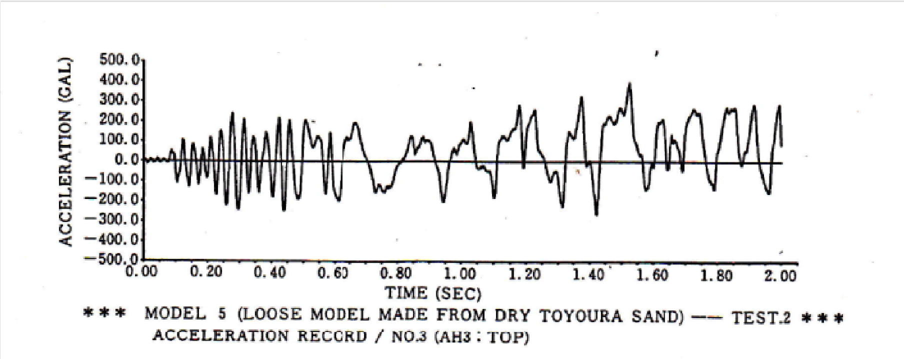


Finite element mesh used for the analysis

OBSERVED ACCELERATION AT THE BASE AND CREST OF DAM MODEL



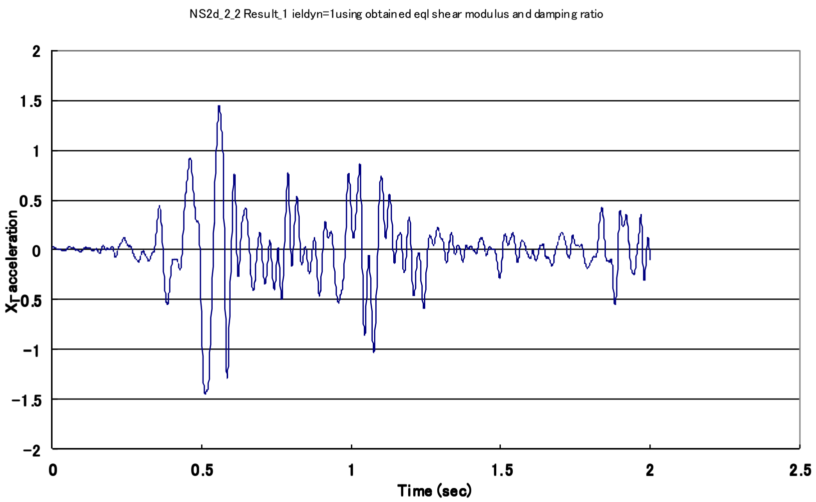
Input horizontal acceleration (observed at the base of shaking table)



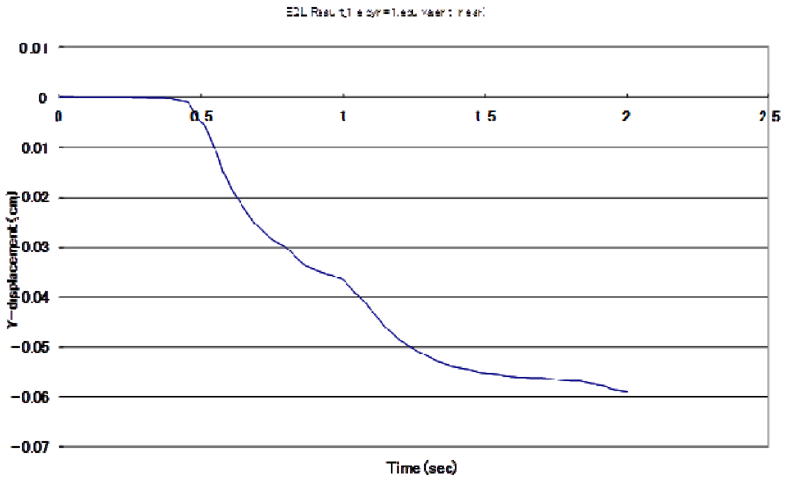
Observed acceleration at crest of the dam

COMPUTED ACCELERATION AND SETTLEMENT (1)

Shear modulus and damping ratio are estimated by applying the equivalent linear method dry density =0.0014kg/cm^3, ϕ_P = 35°, ϕ_R = 34°, G_E =1200.0, h_{max}= 0.25.

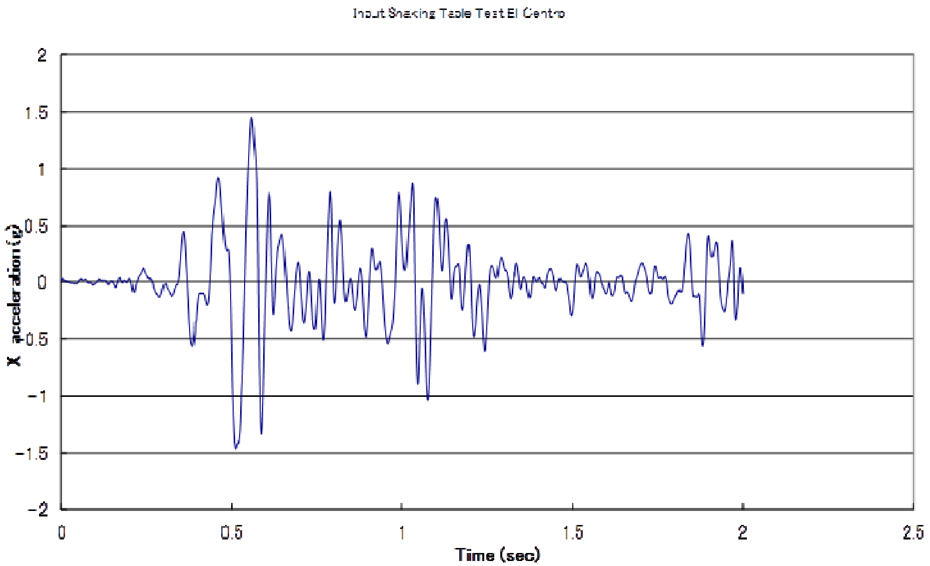


Computed horizontal acceleration at the center of dam crest

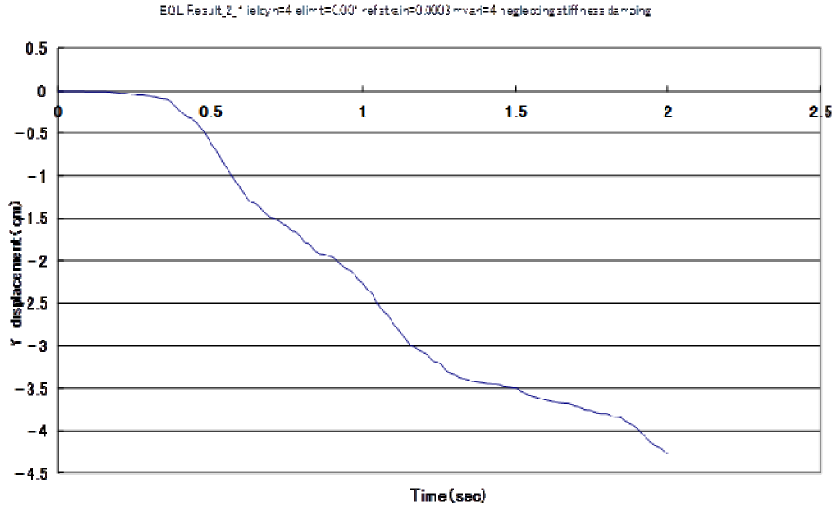


Computed settlement at the center of dam crest

COMPUTED ACCELERATION AND SETTLEMENT (2)
ELASTIC LIMIT=0.001



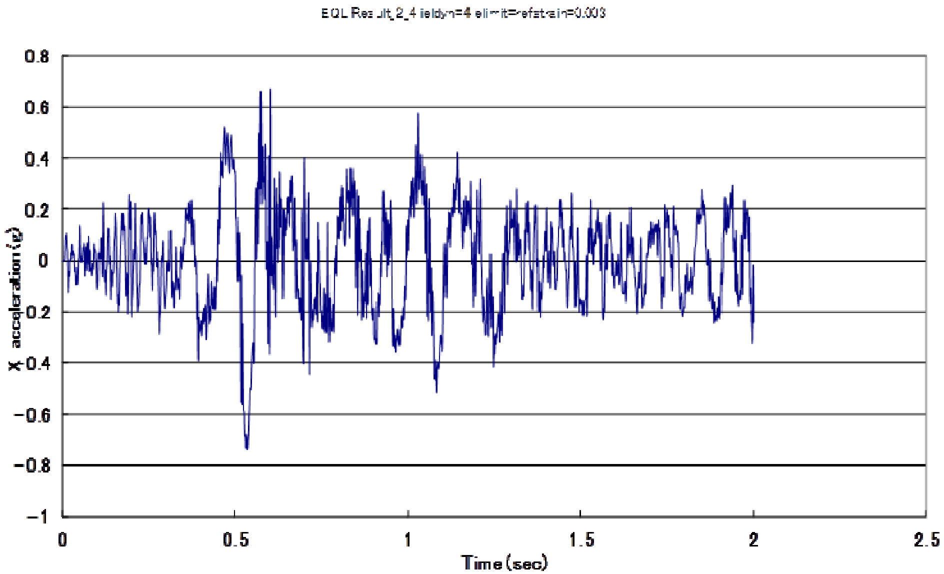
Computed acceleration at the center of dam crest (Rayleigh damping beta = 0)



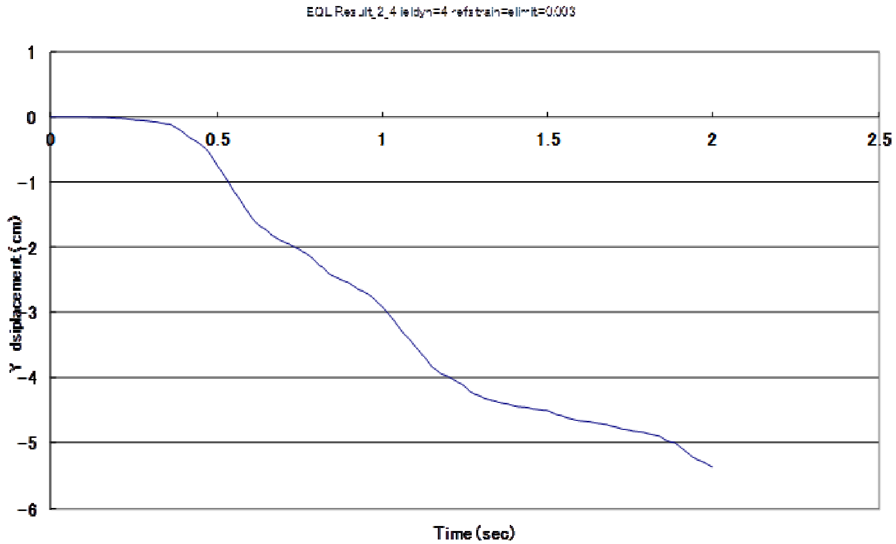
Computed settlement at the center of dam crest (Rayleigh damping beta = 0)

COMPUTED ACCELERATION AND SETTLEMENT (3)

ELASTIC LIMIT: reference shear strain for Hardin-Drnevich equation

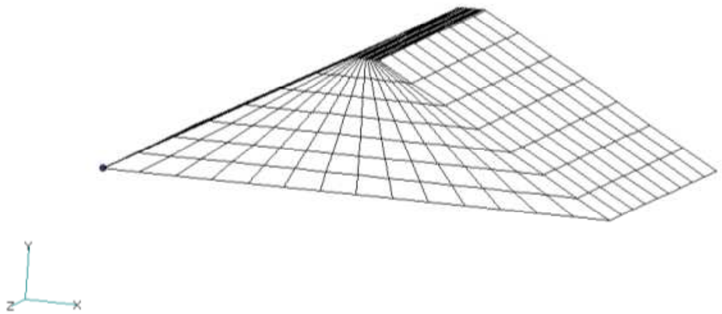


Computed acceleration at the center of dam crest.
Rayleigh damping beta = 0, elastic limit = 0.0003

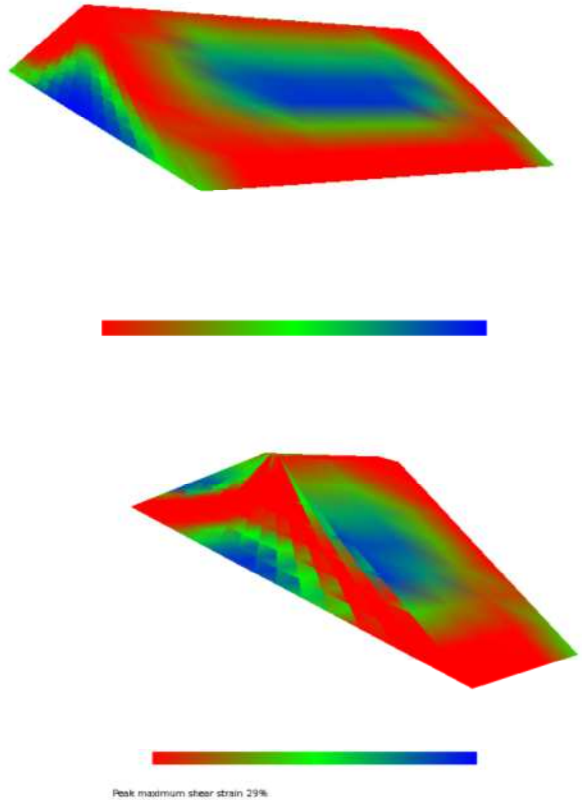


Computed settlement at the center of dam crest.
Rayleigh damping beta = 0, elastic limit = 0.0003

THREE DIMENSIONAL FINITE ELEMENT ANALYSIS OF MODEL DAM ON SHAKING TABLE TEST

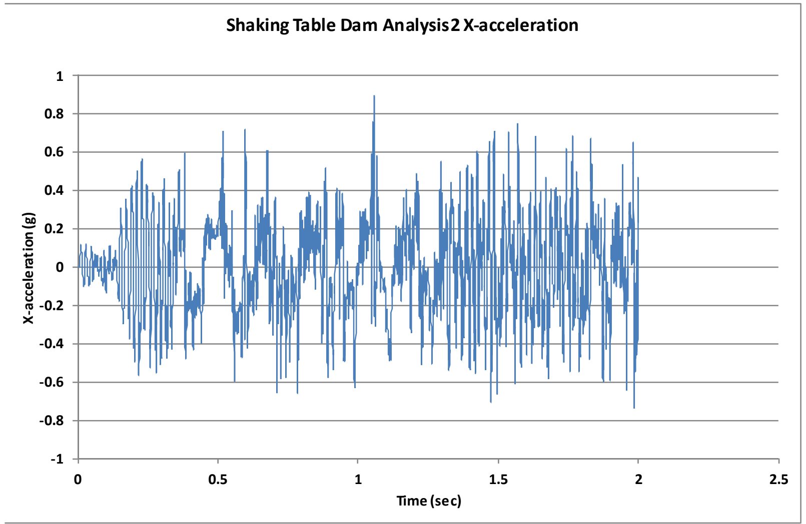


Three dimensional finite element mesh

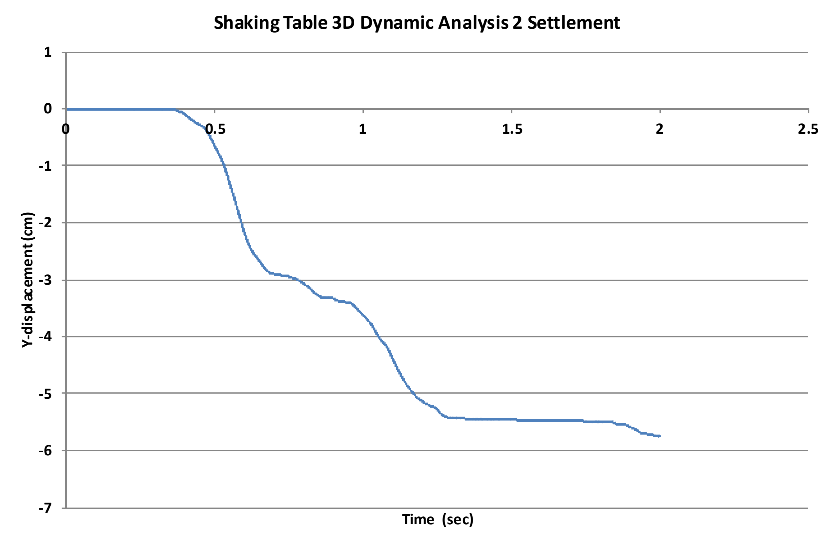


Computed maximum shear strain of model dam after shaking (peak strain 30%)

COMPUTED HORIZONTAL ACCELERATION AND SETTLEMENT AT CREST OF MODEL DAM THREE DIMENSIONAL ANALYSIS

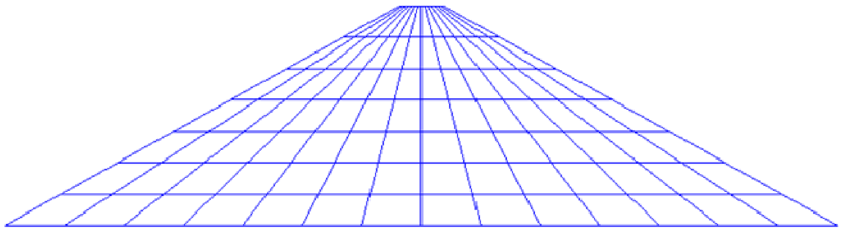


Computed horizontal acceleration at the crest of model dam

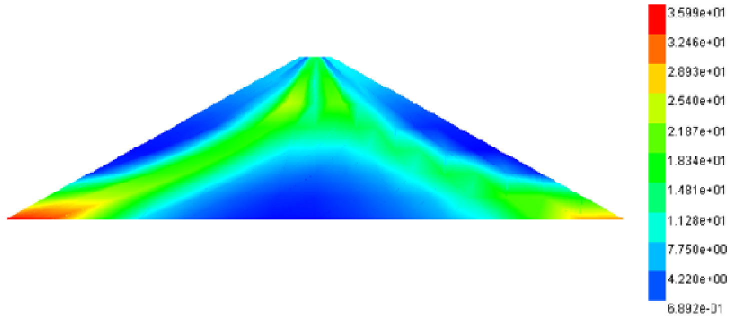


Computed settlement at the crest of model dam

TWO DIMENSIONAL FINITE ELEMENT ANALYSIS OF MODEL DAM ON SHAKING TABLE TEST

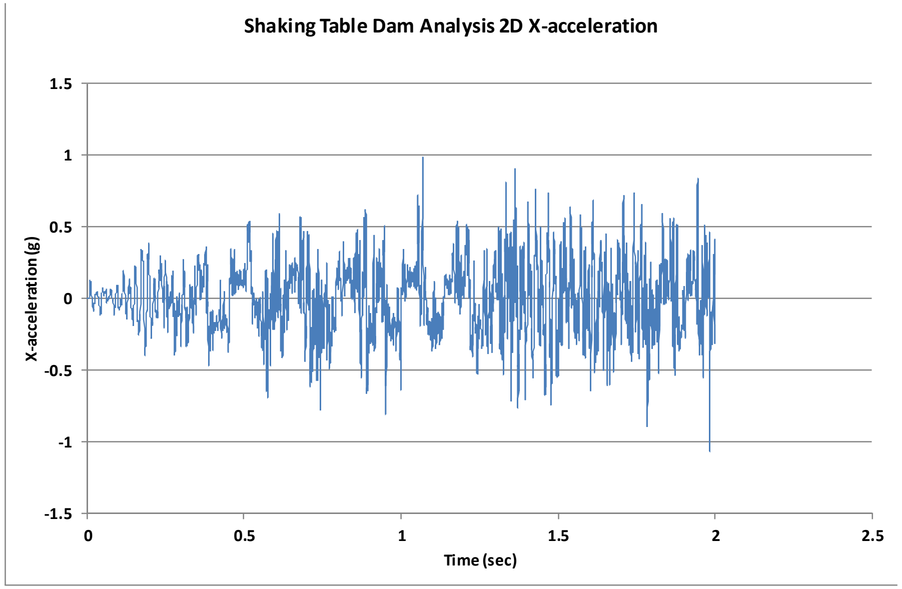


Two dimensional finite element mesh

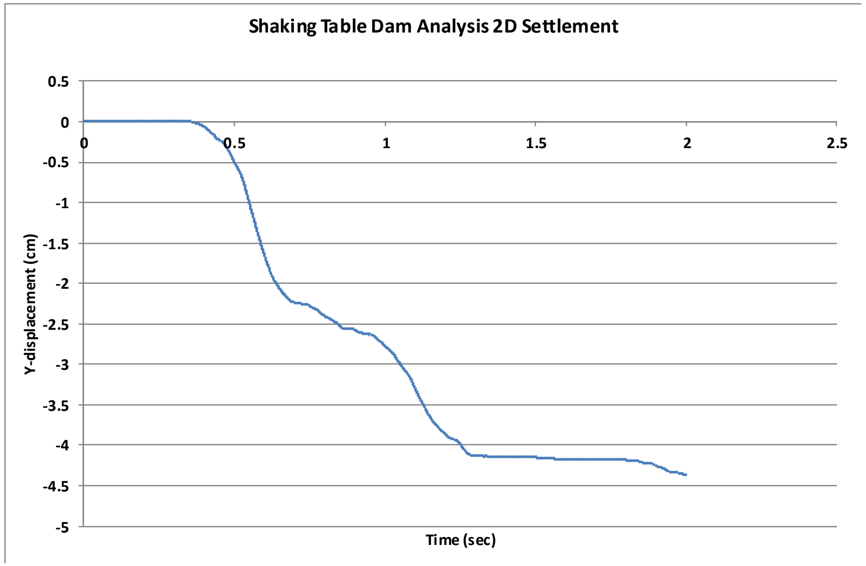


Computed maximum shear strain of model dam after shaking (peak strain 30%)

COMPUTED HORIZONTAL ACCELERATION AND SETTKEMENT AT CREST OF MODEL DAM TWO DIMENSIONAL ANALYSIS

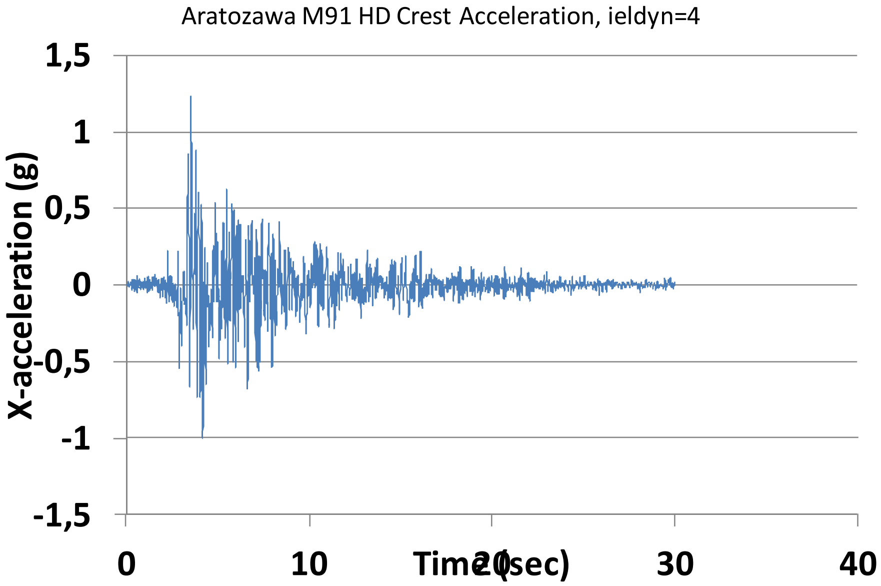


Computed horizontal acceleration at the crest of model dam

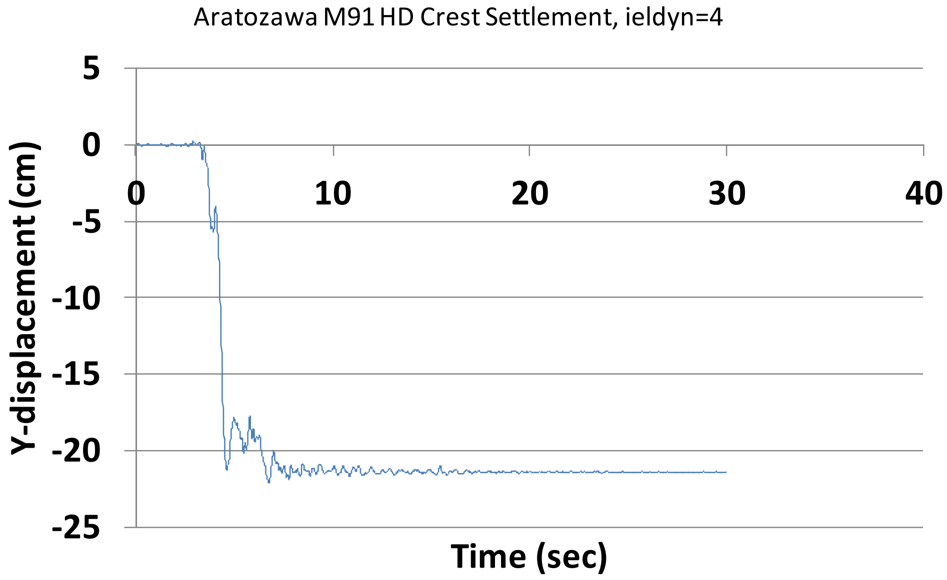


Computed settlement at the crest of model dam

COMPUTED HORIZONTAL ACCELERATION AND SETTLEMENT AT CREST OF ARATOZAWA DAM TWO DIMENSIONAL ANALYSIS



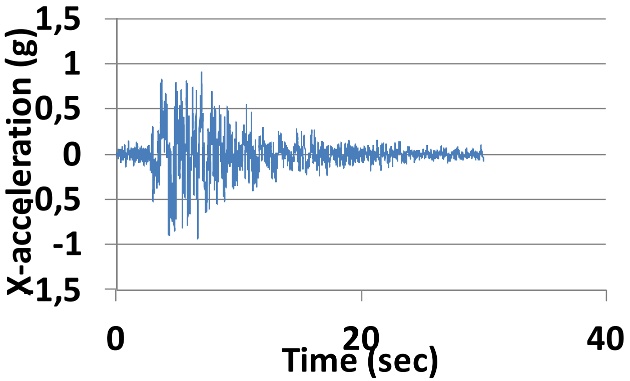
Computed horizontal acceleration at the crest of dam



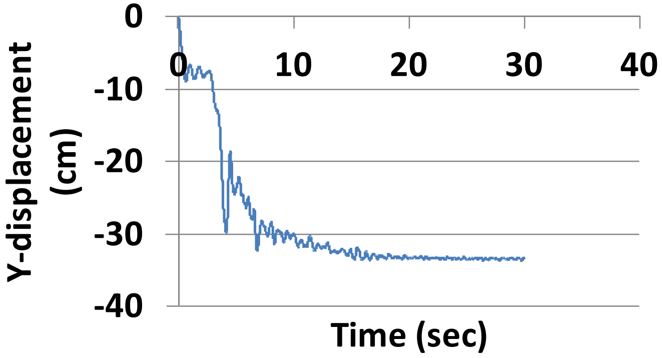
Computed settlement at the crest of dam

Simple strain softening constitutive model (only peak strength, residual strength, shear band thickness & softening rate are needed)
Elastic limit for shear modulus and damping ratio : reference shear strain for Hardin-Drnevich equation

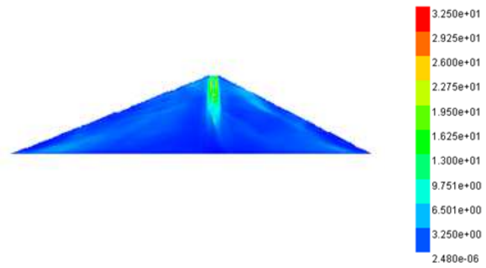
COMPUTED HORIZONTAL ACCELERATION AND SETTLEMENT AT CREST OF ARATOZAWA DAM TWO DIMENSIONAL ANALYSIS



Computed horizontal acceleration at the crest of dam



Computed settlement at the crest of dam



Maximum shear strain distribution (%)

Kinematic hardening constitutive model

Effective stress analysis: Core zone is undrained, Rock zones are drained

Elastic shear modulus and damping ratio * 0.2 : reference shear strain for Hardin-Drnevich equation

OUTLINE

CNSOLIDATED-UNDRAINED CYCLIC TRIAXIAL TEST OF TRANSITION ZONE MATERIAL

- The advantage of the total stress analysis is that it is simple and numerically stable. The strength reduction in total stress is due to the plasticity or damage from a viewpoint of effective stress, cumulative shear strain that is similar to equivalent plastic parameter can be calculated in the elastic state.

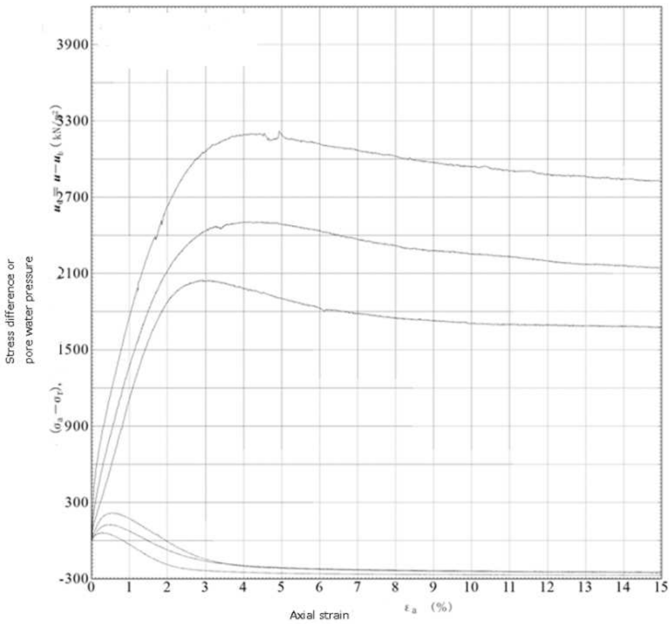
  The integral of shear strain increments can be given by next equation.

  $$\bar{\varepsilon} = \int d\bar{\varepsilon} \qquad d\bar{\varepsilon} = (de_x^2 + de_y^2 + de_z^2) + 2de_{xy}^2$$

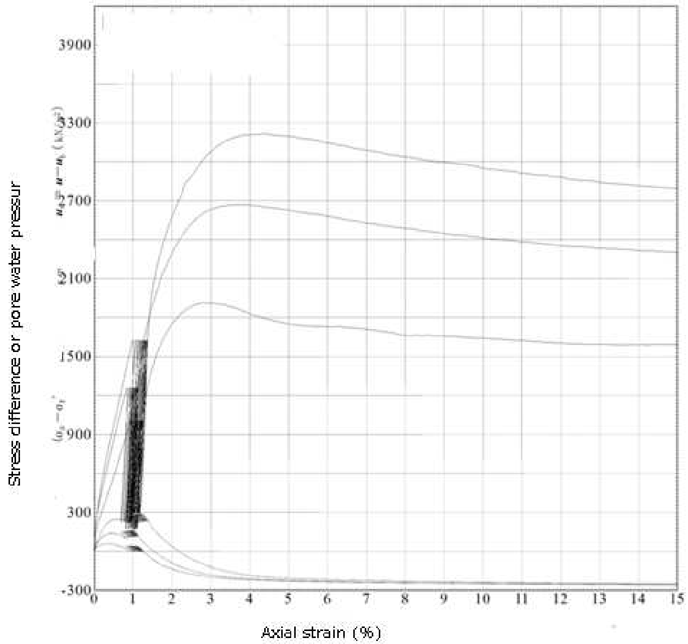
  where $de_x, de_y, de_z, de_{xy}$ are deviatoric components of strain.

- By applying empirical factor to this value by using the cyclic tri-axial test result, we can estimate the reduction of strength in total stress analysis. This process is similar to the damage concept of "simplified method by Tatsuoka et al.".

CONSOLIDATED UNDRAINED STRESS-STRAIN RELATION OOGAKI DAM ROCK MATERIAL (TRANSITION ZONE)



a)



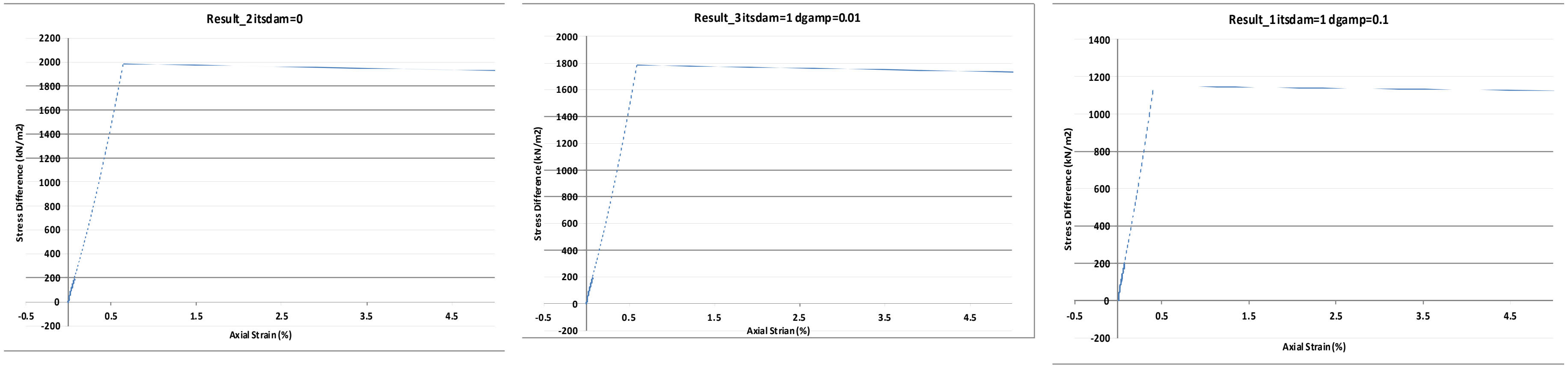
b)

Confining pressure: 200,400,600(kN/m2)

a)Monotonic loading

b)Cyclic and monotonic loading

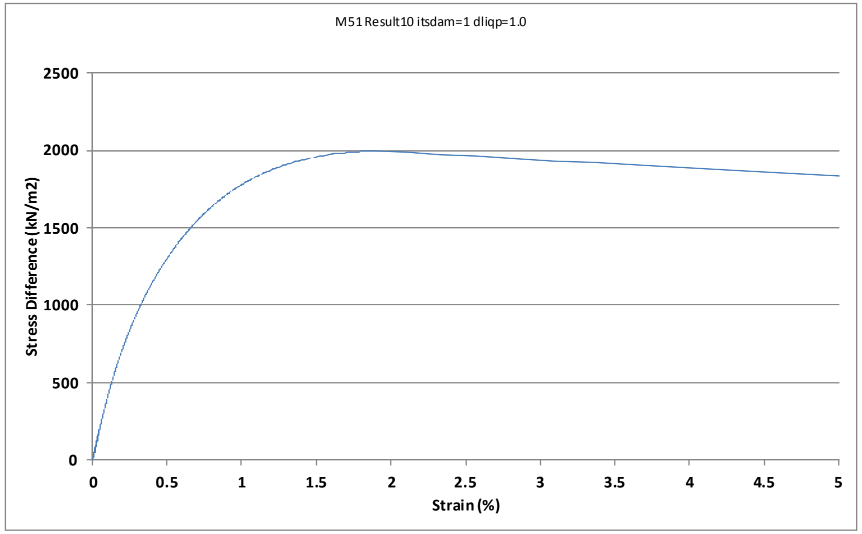
COMPUTED STRESS-STRAIN RELATION BY SIMPLE CONSTITUTIVE MODEL



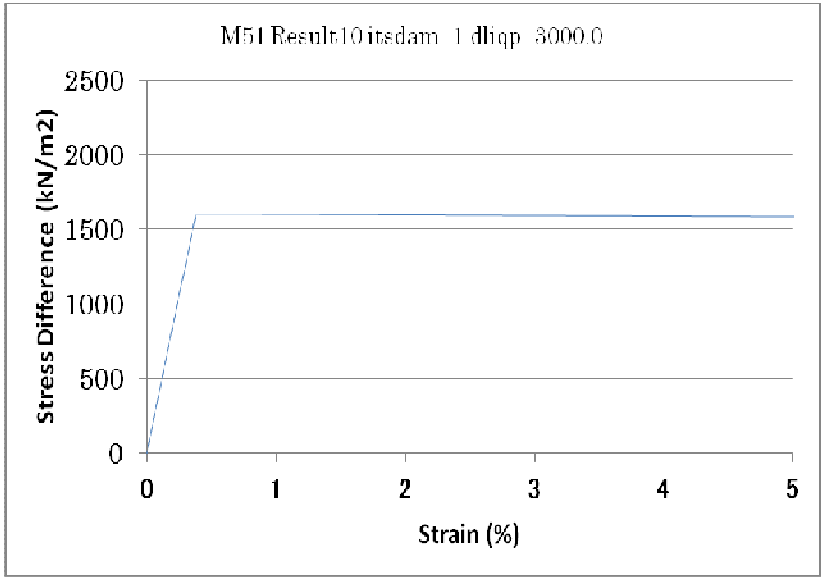
a) b) c)

Confining pressure is 200 (kN/m2) (applied cyclic load: 0 - 200kN/m2) ϕ_p = 36.6˚, ϕ_r = 35.0˚,
B = 0.7, *C* = 0.6, *D* = 0.7, cohesion = 354 *kPa*,
a) without factor of equivalent plastic parameter , b) factor is 0.001, c) factor is 0.1

COMPUTED STRESS-STRAIN RELATION BY KINEMATIC HARDENING ELASTO-PLASTIC MODEL



a)



b)

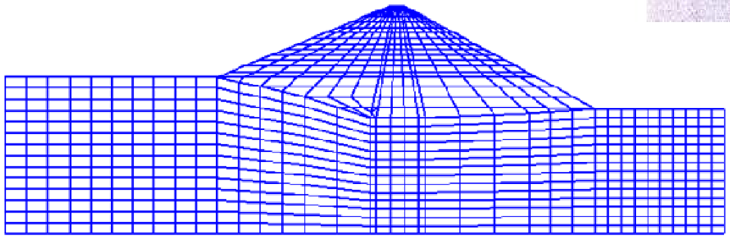
$\phi_p = 36.6°$, $\phi_r = 20.0°$, $\varepsilon_f = 0.03$, $\varepsilon_r = 0.6$, $a_f = 5.0$, m =1, l = 0.5, n = 1.0, c = 374 kPa ,

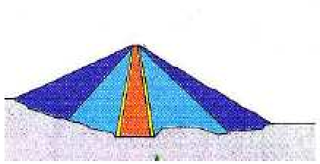
factor for plastic parameter κ' : a) = 1.0 , b) = 3000.0

In case b) stress-strain behavior is just similar to simple elasto-plastic strain softening model

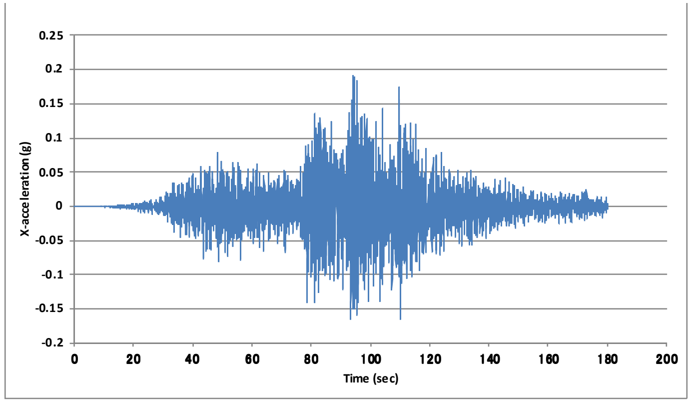
OUTLINE

OOGAKI DAM DYNAMIC ANALYSIS BY SIMPLE ELASTO-PLASTIC MODEL





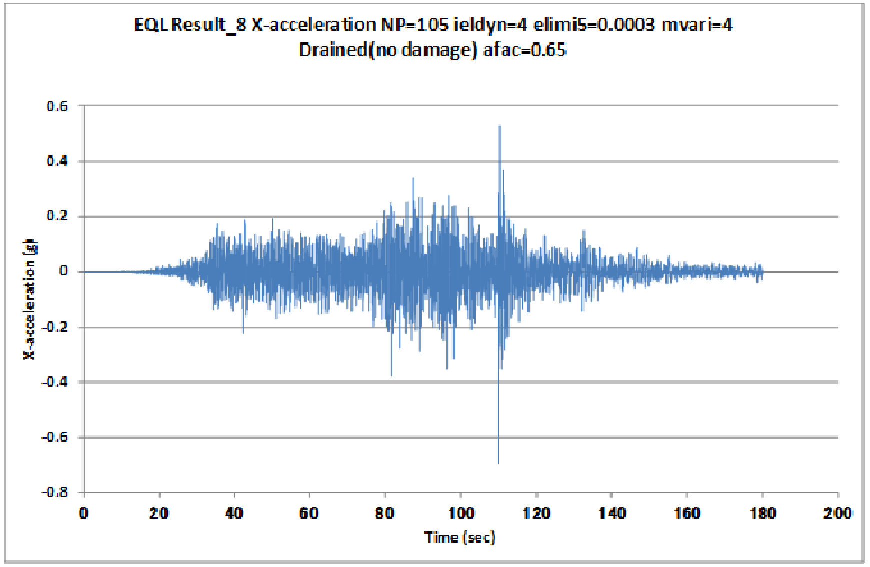
Oogaki Dam finite element mesh. Height of this dam is 85.5 m. Blue shows transition zone



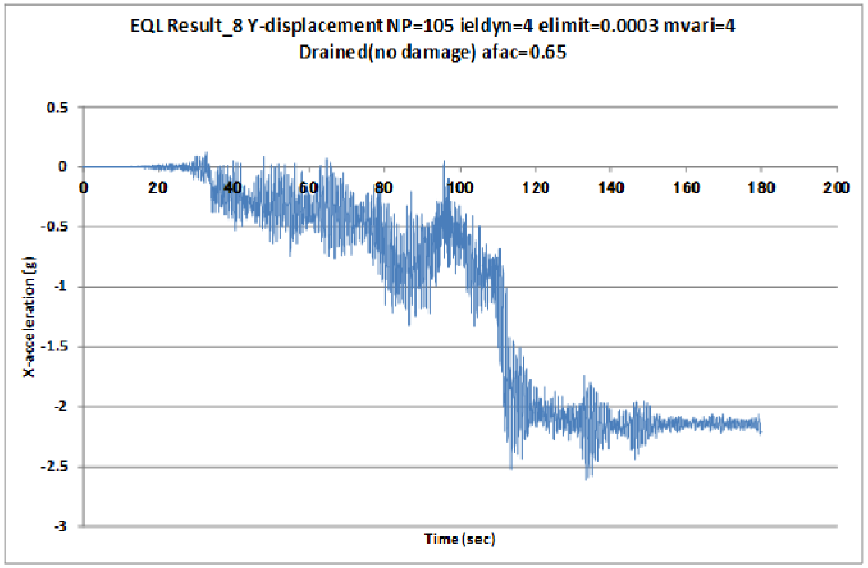
Estimated input acceleration at the base of the dam

COMPUTED RESULTS BY SIMPLE CONSTITUTIVE MODEL TRANSITION ZONE IS DRAINED

Core zone is undrained and another zones are completely drained
Transition zone material properties:
ϕ_p = 42.1˚, ϕ_r = 34.0˚, B = 0.7, *C* = 0.6, *D* = 0.7, cohesion = 76 *kPa*, shear band thickness = 4 *cm*, G_E =1200.0, Rayleigh damping alpha = 0, elastic limit = 0.0003.



Computed crest acceleration



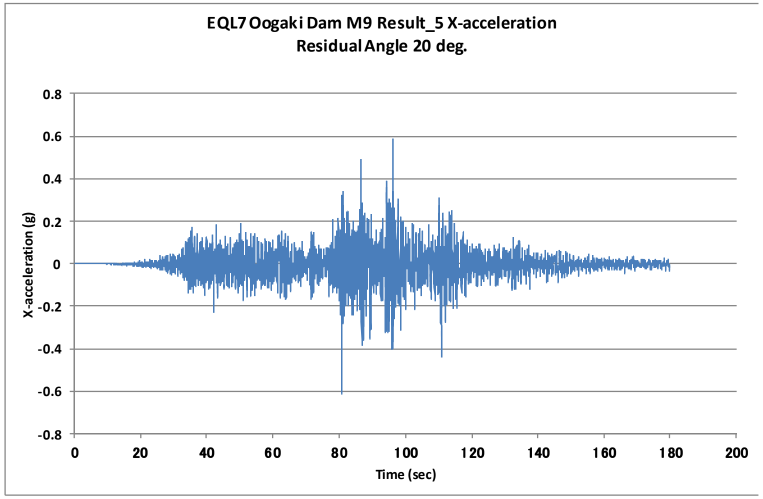
Computed crest settlement

COMPUTED RESULTS BY SIMPLE CONSTITUTIVE MODEL TRANSITION ZONE IS UNDRAINED

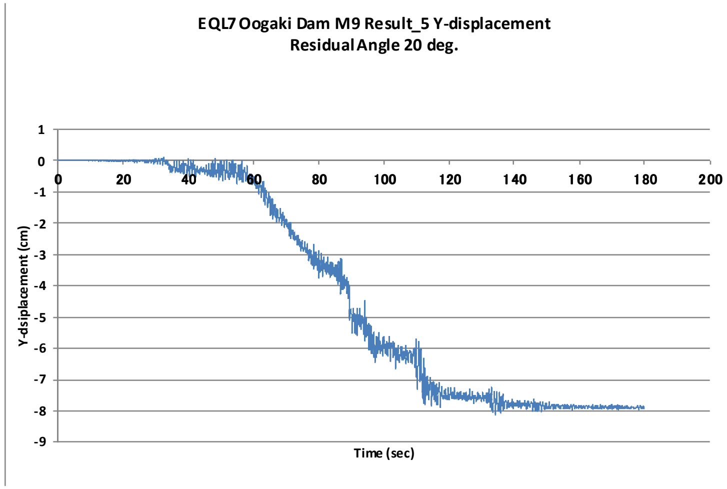
Core zone is undrained and another zones are completely drained

Transition zone material properties:

ϕ_p = 36.6˚, ϕ_r = 20.0˚, B = 0.7, *C* = 0.6, *D* = 0.7, cohesion = 353 *kPa*, shear band thickness = 4 *cm*, G_E =1200.0, Rayleigh damping alpha = 0, elastic limit = 0.0003.



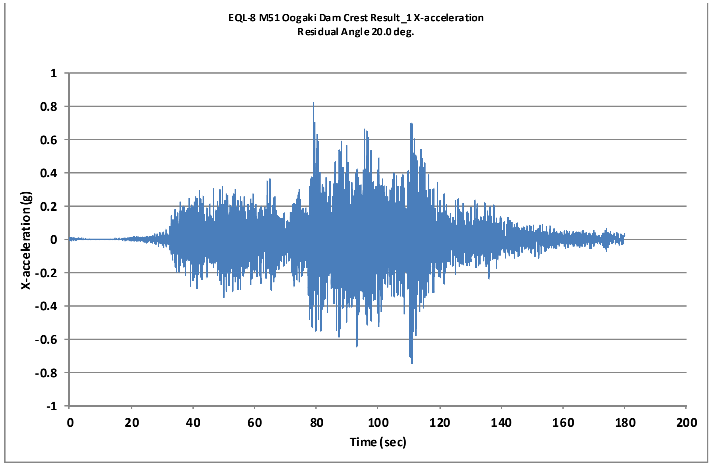
Computed crest acceleration



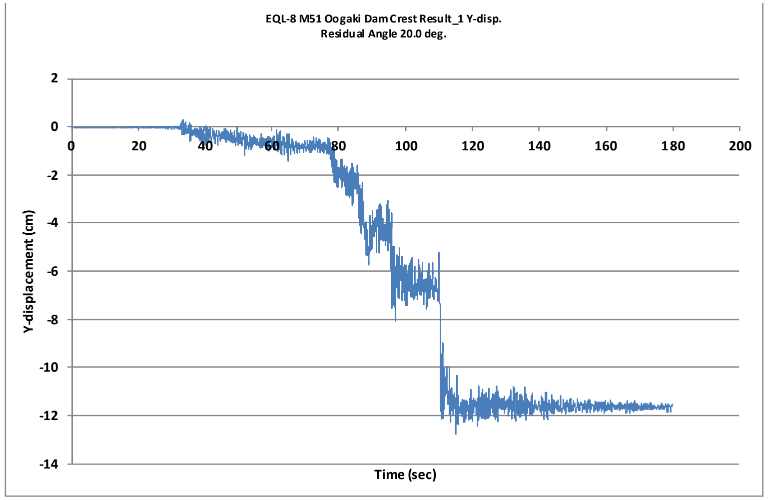
Computed crest settlement

DYNAMIC RESPONSE ANALYSIS BY KINEMATIC HARDENING ELASTO-PLASTIC MODEL (TOTAL STRESS ANALYSIS)

c = 374 *kPa*, ϕ_p = 36.6°, ϕ_r = 20.0°, ε_f = 0.03, ε_r = 0.6, a_f = 5.0, m = 1, l = 0.5, n = 1.0, Thickness of shear zone = 4 *cm.* The factor of plastic parameter is 3000.0.



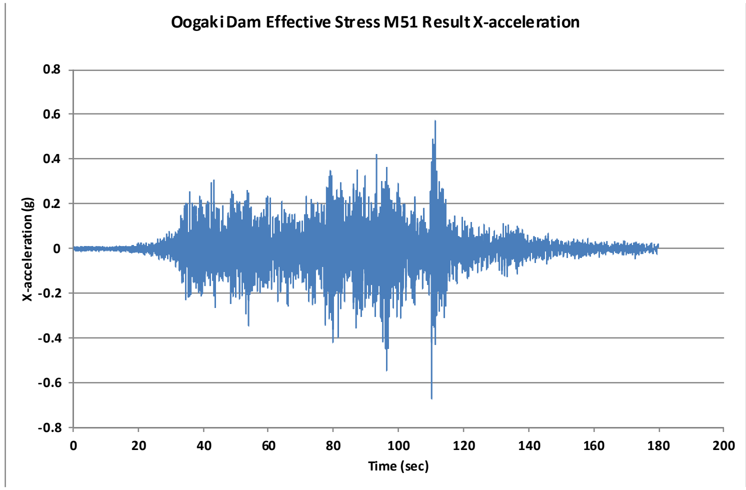
Computed crest acceleration by total stress analysis



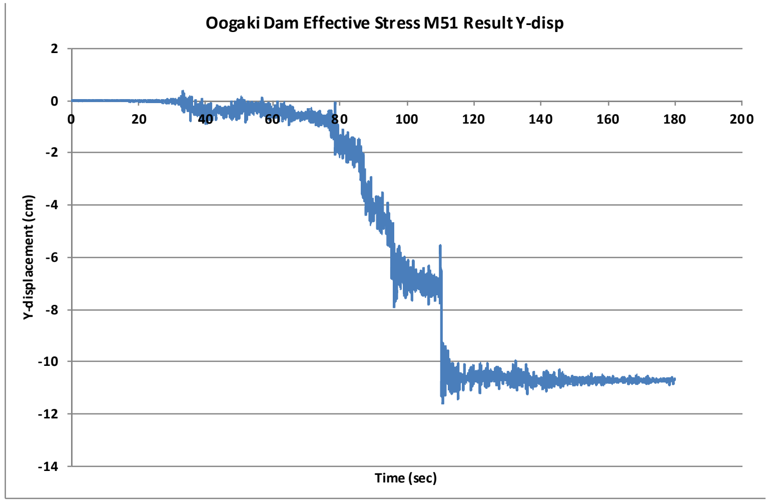
Computed crest settlement by total stress analysis

DYNAMIC RESPONSE ANALYSIS BY KINEMATIC HARDENING ELASTO-PLASTIC MODEL (EFFECTIVE STRESS ANALYSIS)

c = 11 *kPa*, ϕ_p = 42.0° , ϕ_r = 34.0° , ε_f = 0.03, ε_r = 0.6, a_f = 5.0,
m = 0.5, *n* = 1.0, Thickness of shear zone = 4 *cm.*

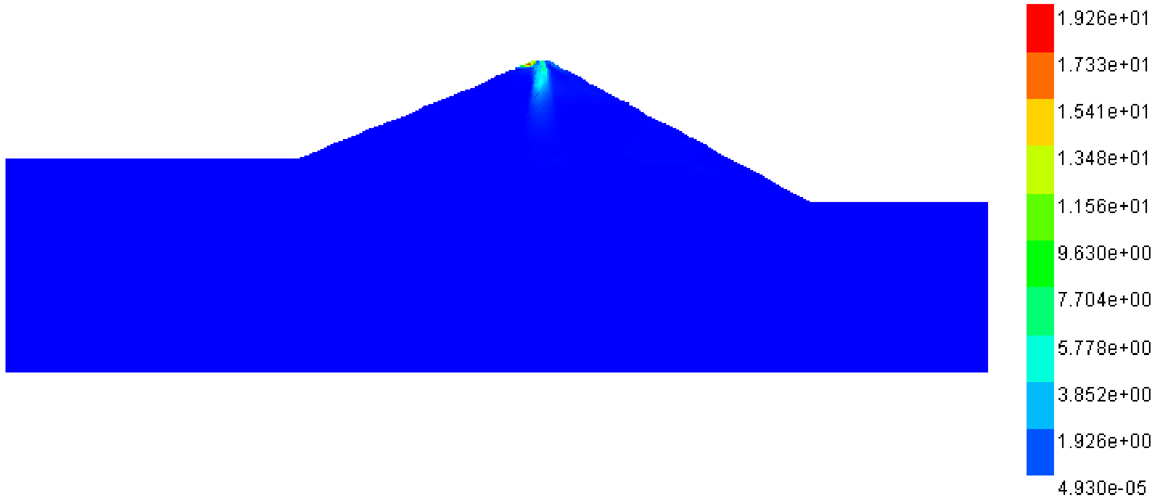


Computed crest acceleration by effective stress analysis



Computed crest settlement by effective stress analysis

COMPUTED MAXIMUM SHEAR STRAIN (%) BY KINEMATIC HARDENING MODEL EFFECTIVE STRESS ANALYSIS



SUMMARY

A dynamic progressive failure analysis of a small embankment dam using dry sand on shaking table is carried out. The acceleration simulating El Centro earthquake is applied to the base of shaking table. The computed acceleration at the crest of model dam is compared to the observed one and the computed displacement is also verified by the observed displacement. Then comparison of two and three dimensional analyses of dry sand embankment dam are also carried out.

A cumulative damage concept for simple elasto-plastic strain softening model is effective by using the results of cyclic triaxial tests of saturated soils. A shear banding constitutive model incorporating a characteristics length of shear band is also necessary. Both a simple strain softening constitutive model and a kinematic hardening model are also applicable to total stress dynamic response analyses by applying incompressible condition in case of saturated soils.

The computation of real fill-type dam is also carried out by total stress elasto-plastic constitutive model and effective stress constitutive model by taking into account the pore water build-up.


THANK YOU FOR
YOUR ATTENTION