NONLINEAR ANALYSES OF EMBANKMENT DAMS UNDER STRONG EARTHQUAKE

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2.2 *Kinematic hardening model*
3 SHAKING TABLE TESTS OF EMBANKMENT DAMS AND DYNAMIC ANALYSES
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3.2 *Dynamic analysis of embankment dam by simple constitutive model*
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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 tri-axial 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 rockfill 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

$$f = \alpha I_1 + \frac{\bar{\sigma}}{g(\theta_L)} - K = 0$$

$$K = \frac{6c\cos\phi}{\sqrt{3}(3-\sin\phi)}$$

$$\alpha = \frac{2\sin\phi}{\sqrt{3}(3-\sin\phi)}$$

$$\Phi = \alpha' I_1 + \bar{\sigma} - K = 0$$

$$\alpha' = \frac{2\sin\psi}{\sqrt{3}(3-\sin\psi)}$$

I_1 : primary invariant of stress

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

$$\frac{\sigma_1}{\sigma_3} = -K_{Rowe}\left(\frac{d\varepsilon_1^p}{d\varepsilon_3^p}\right)$$

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
p and R specify peak and residual

ELASTIC PROPERTIES

Shear modulus ***G*** *,* Bulk modulus **K** and damping ratio *h* are given by Hardin-Drnevich equation

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

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

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 IN ISOTROPIC HARDENING MODEL

Hardening regime $(\kappa \le \varepsilon_f)$

$$\alpha(\kappa) = \left(2\frac{\sqrt{\kappa\varepsilon_f}}{\kappa + \varepsilon_f} \right)^m \alpha_p$$

Softening regime $(\kappa > \varepsilon_f)$

$$\alpha(\kappa) = \alpha_R + (\alpha_p - \alpha_R)\exp\{-(\frac{\kappa - \varepsilon_f}{\varepsilon_r})^2\}$$

$$\alpha = \frac{2\sin\phi}{\sqrt{3}(3 - \sin\phi)} \qquad \alpha' = \frac{2\sin\psi}{\sqrt{3}(3 - \sin\psi)}$$

where m, ε_f and ε_r are material constants

PEAK FRICTION ANGLE

The peak friction angle (ϕ_p) is estimated from the empirical relations (by Tatsuoka)

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

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

e : Initial void ratio

δ : angle of σ_1 direction relative to horizontal bedding plane

RETURN-MAPPING ALGORITHM

A change in stresses can cause an associated change in the elastic strains even by

$$d\varepsilon^{e} = [\,\text{C}\,]^{-1}(\sigma_{B} - \sigma_{A})$$

[C] : Elastic matrix

As the total strain does not change during the relaxation process, the plastic strain change is balanced by an equal and opposite change in the elastic strains :

$$d\varepsilon_{ij} = d\varepsilon_{ij}^{e} + sd\,\varepsilon_{ij}^{p}$$

$$sd\varepsilon_{p} = -d\varepsilon^{e} = -[\,\text{C}\,]^{-1}(\sigma_{B} - \sigma_{A})$$

$s = F_{b} / F_{e}$

F_{e} is the area of the element

F_{b} is the area of a single shear band in each element

RETURN-MAPPING ALGORITHM(continued)

$$d\varepsilon^p = \lambda(\partial\Phi / \partial\sigma)$$

The plastic strain increments are proportional to the gradient of the plastic potential

$$\sigma_B = \sigma_A - S\lambda[D]^e(\partial\Phi / \partial\sigma)$$

Suitably relaxed stress must satisfy the yield function

$$f(\sigma_B, \kappa_B) = f\{(\sigma_A - S\lambda[D]^e(\partial\Phi / \partial\sigma)), (\kappa_A + \lambda)\} = 0$$

$$\lambda = f(\sigma_A, \kappa_A) / \{S(\partial f / \partial\sigma)[D]^e(\partial\Phi / \partial\sigma) - (\partial f / \partial\kappa)\}$$

EXPLICIT DYNAMIC (RELAXATION) METHOD

$$M_D a + Cv + P - P^{init} = F \quad \textbf{(1)}$$

$$C = \alpha M_D \quad \textbf{(2)}$$

$$q_{n+1} = \frac{1}{1+0.5\alpha\Delta t}\left[\frac{\Delta t^2}{M_D}(F - P + P^{init})^t + 2q_n - (1-0.5\Delta t)q_{n-1}\right] \quad \textbf{(3)}$$

M_D is the diagonalized mass matrix

C is the damping matrix

a is the acceleration vector

q_n is the displacement vector at time *n*,

P is the internal force vector

v is the velocity vector

Δt Is the time increment

α is the critical damping ratio

EXPLICIT DYNAMIC (RELAXATION) METHOD

$$\alpha = \sqrt[2]{\frac{xK\ x}{xM\ {}^{t}x}} \qquad (1)$$

$$\alpha = \sqrt[2]{\frac{q^{T}K^{lt}q}{q^{T}M_{D}q}} \qquad (2)$$

$$K^{lt} = \frac{P^{t} - P^{t-\Delta t}}{\Delta t\ {}^{t}v} \qquad (3)$$

$$\Delta t \leq \beta \frac{l}{V_c} \qquad (4)$$

$$\Delta t \leq \beta \sqrt{\frac{\rho(1+\nu)(1-2\nu)}{E(1-\nu)}} \qquad (5)$$

IMPLICIT-EXPLICIT DYNAMIC (RELAXATION) METHOD

$$\tilde{q}_{n+1} = q_n + \Delta t v_n + \Delta t^2 (1 - 2\beta) a_n / 2$$

$$\tilde{v}_{n+1} = v_n + \Delta t (1 - \gamma) a_n$$

v_n is the velocity vector; a_n is the acceleration vector at time n, and γ, β are constants

$$K^* = M / (\Delta t^2 \beta) + \gamma C_T / (\Delta t \beta) + K_T (\tilde{q}_{n+1})$$

$$K^* = M / (\Delta t^2 \beta)$$

$$K^* \Delta q = \Psi$$

Displacement is simultaneously solved from the explicit and implicit effective stiffness matrix using the Skyline solver

IMPLICIT-EXPLICIT DYNAMIC (RELAXATION) MRTHOD

The residual force is evaluated by the equation.

$$\Psi = f_{n+1} - Ma_{n+1} - p(\tilde{q}_{n+1}, \tilde{v}_{n+1})$$

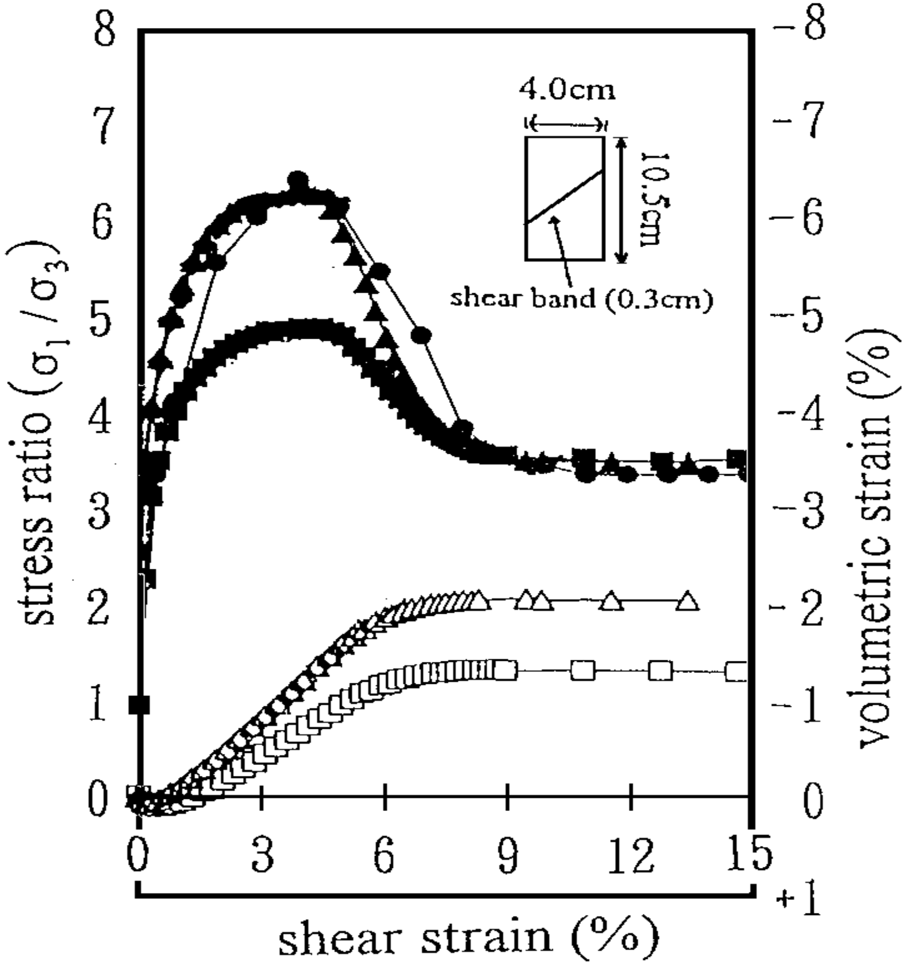
The displacement, velocity and acceleration of the next step are calculated by the following equations.

$$q_{n+1} = \tilde{q}_{n+1} + \Delta q$$

$$v_{n+1} = \tilde{v}_{n+1} = \Delta t \gamma \, a_{n+1}$$

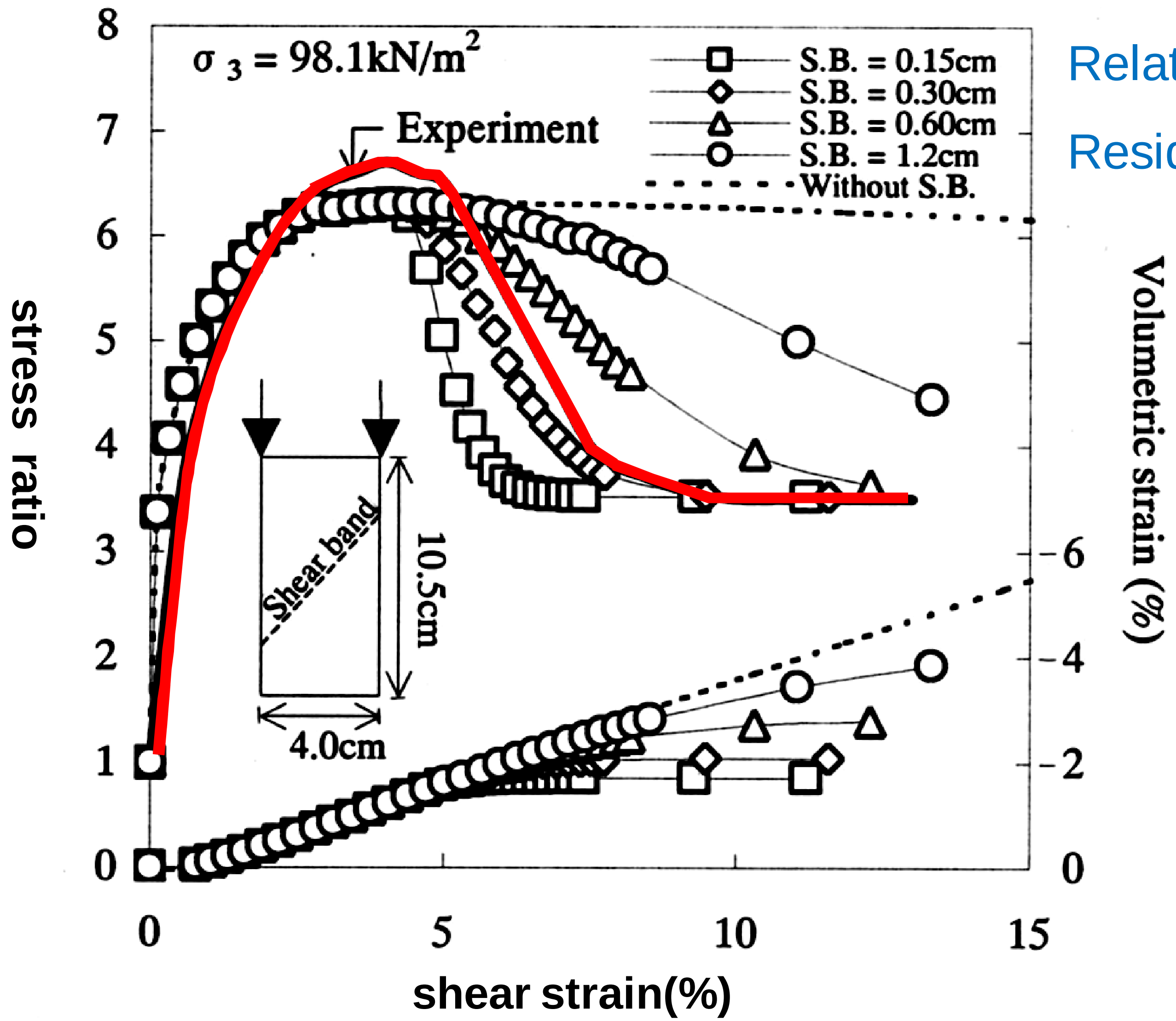
$$a_{n+1} = (q_{n+1} - \tilde{q}_{n+1}) / (\Delta t^2 \beta)$$

PLANE STRAIN TEST & FINITE ELEMENT ANALYSIS



	Stress ratio	Volumetric Strain
Experiment(σ_3=98.1kN/m²)	●	—
Analysis(σ_3=98.1kN/m²、S.B.=0.3cm)	▲	△
Analysis(σ_3=392.4kN/m²、S.B.=0.3cm)	■	□

PLANE STRAIN TEST & FINITE ELEMENT ANALYSIS USING 1 ELEMENT



Relative Density **: 85%**

Residual Friction Angle**: 34º**

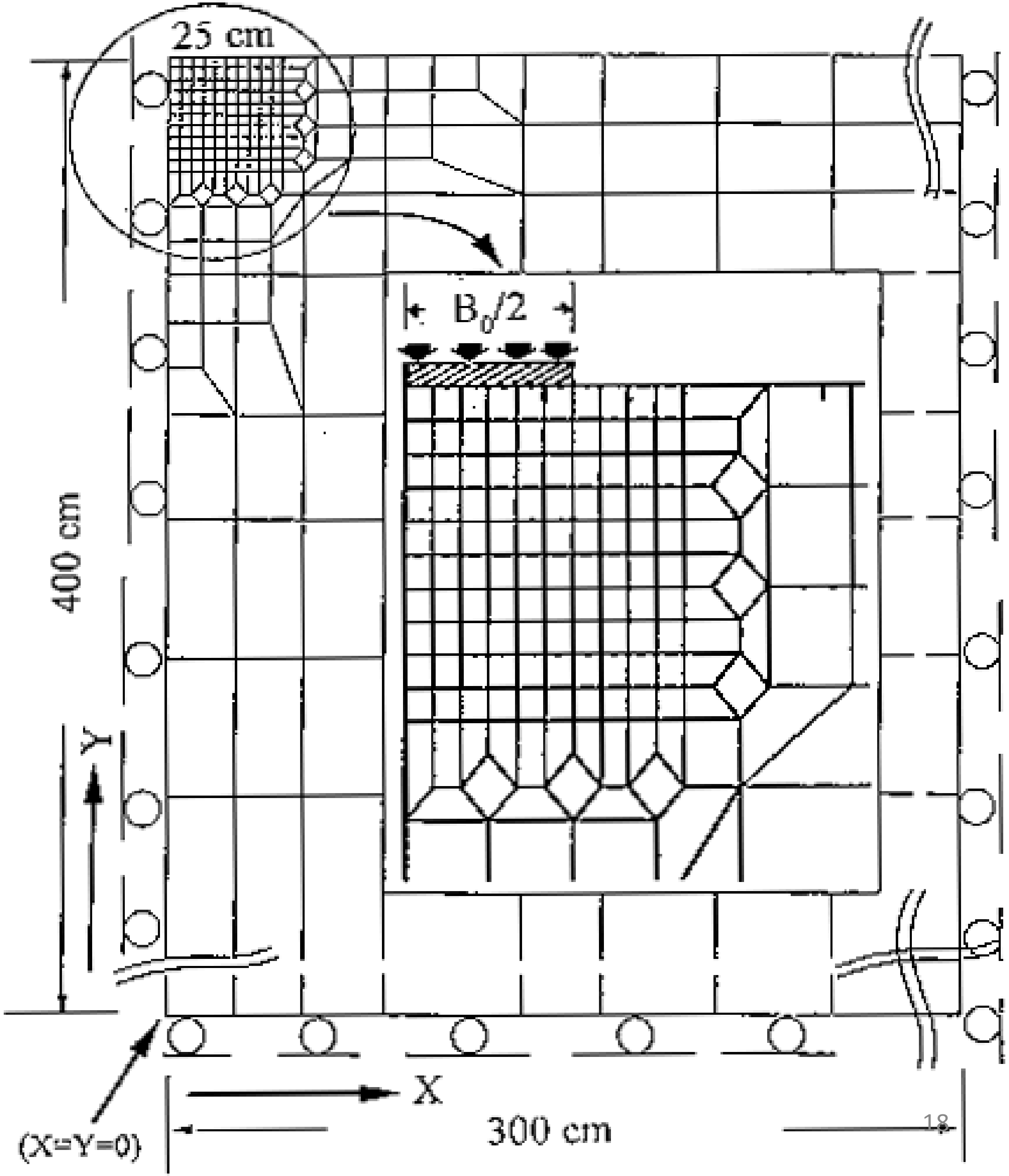
Thickness of Shear Zone is **0.3** cm

Plane Strain Test by Tatsuoka et. al

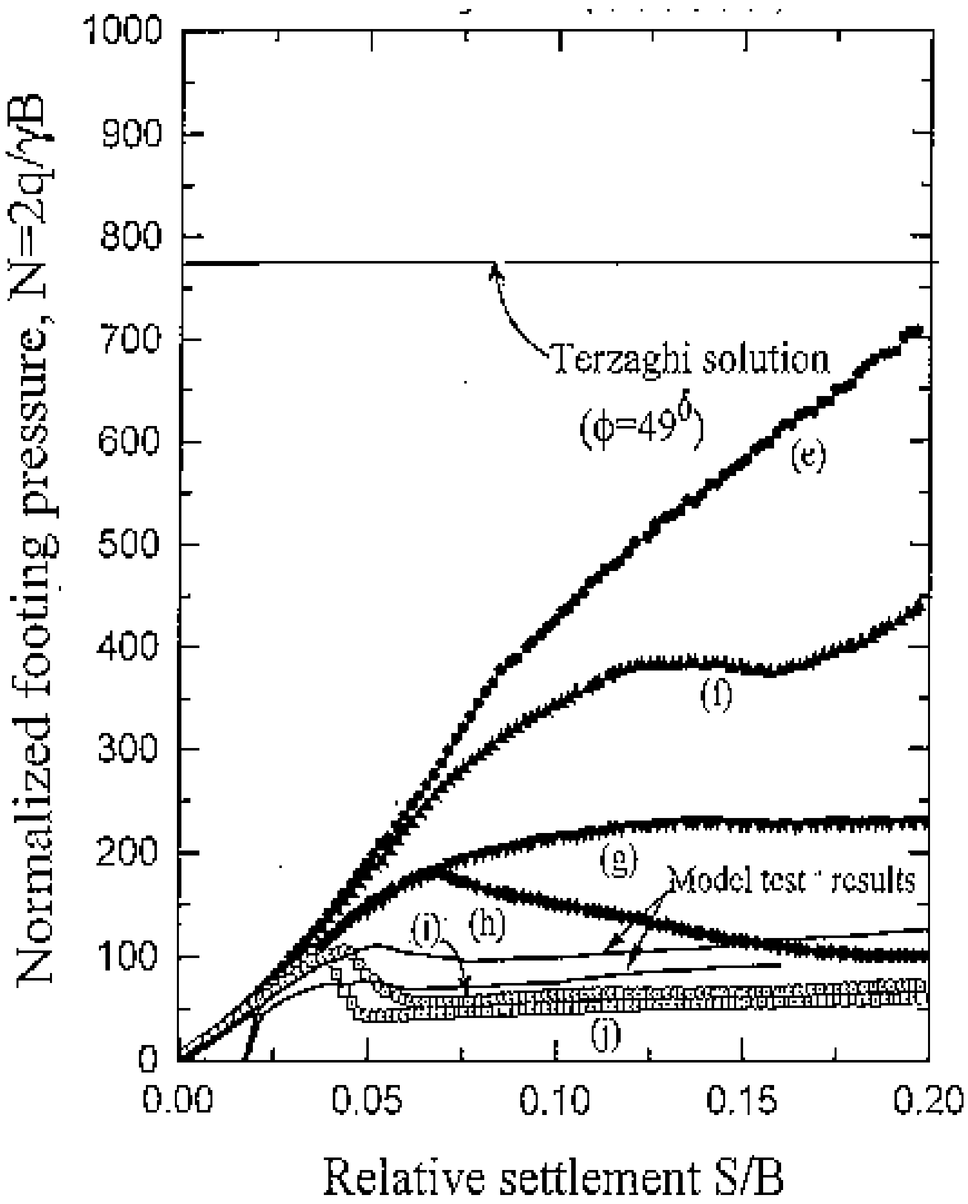
FINITE ELEMENT

MESH FOR FOOTING

Soils and Foundations Vol. 39, N0.4, 93-109, Aug. 1999

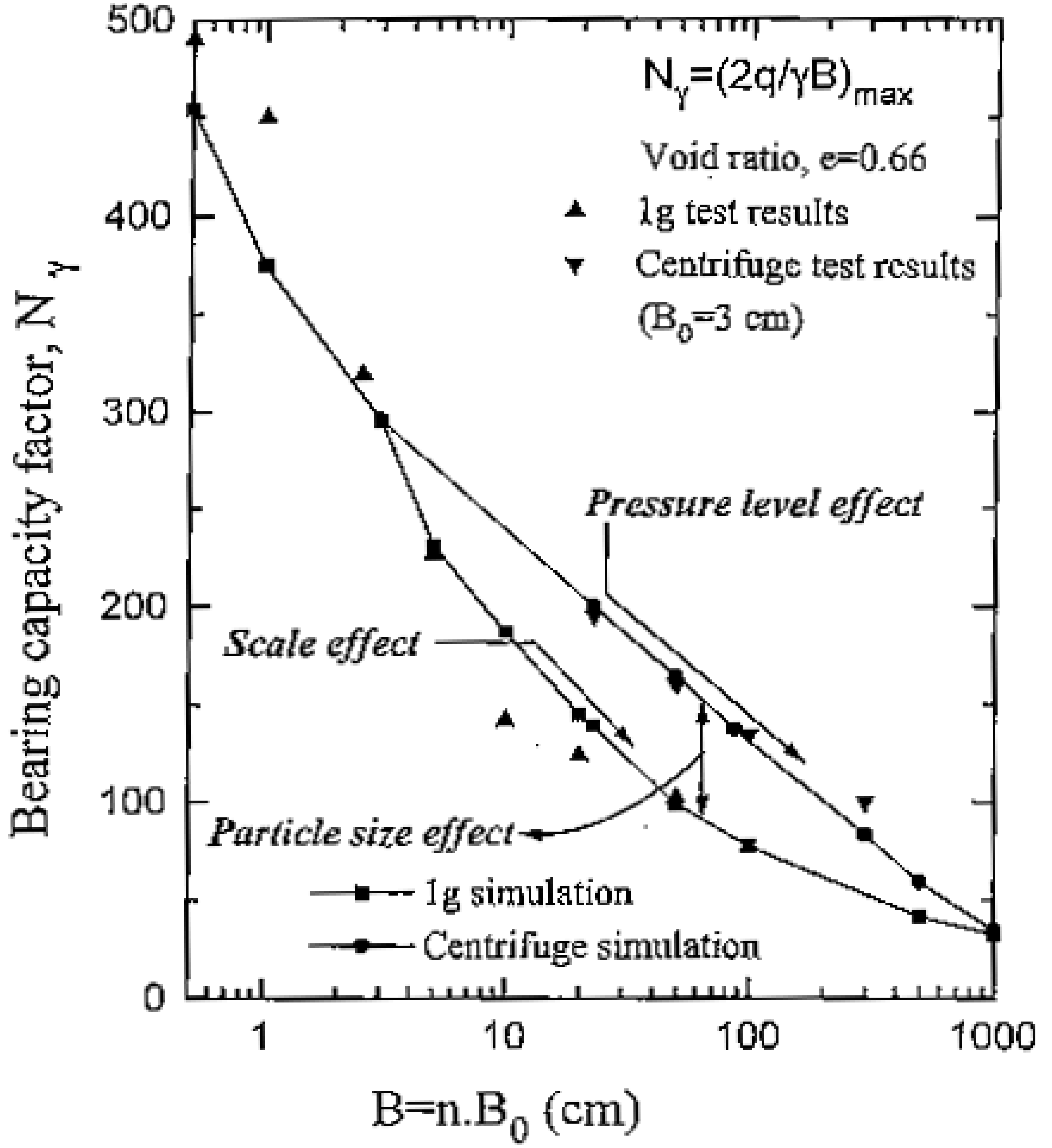


RELATIONSHIPS BETWEEN NRMALIZED FOOTING PRESSURE AND RELATIVE DISPLACEMENT



e: friction is assumed isotropic
f : pressure level dependency
g: anisotropc and pressure level
h: anisotropc, pressure level and strain softening
i: anisotropic, pressure level, strain softening and shear band

RELATIONSHIP BETWEEN BEARING CAPACITY AND WIDTH SCALE EFFECT



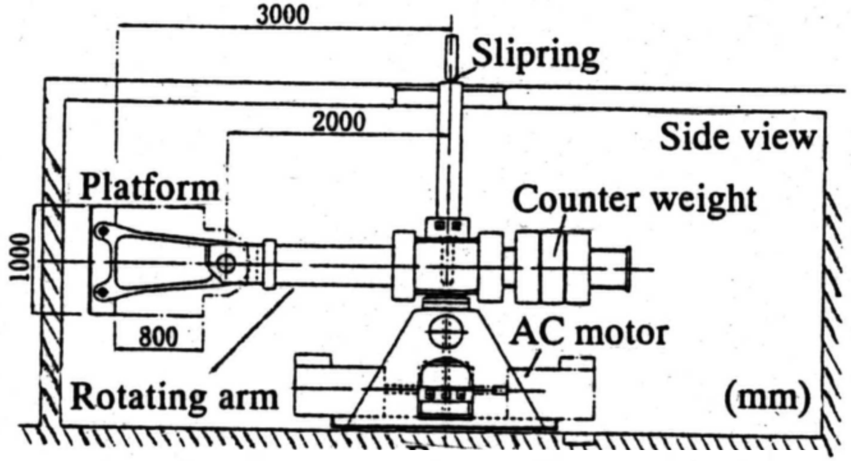
BEARING CAPACITY COEFFICIENT

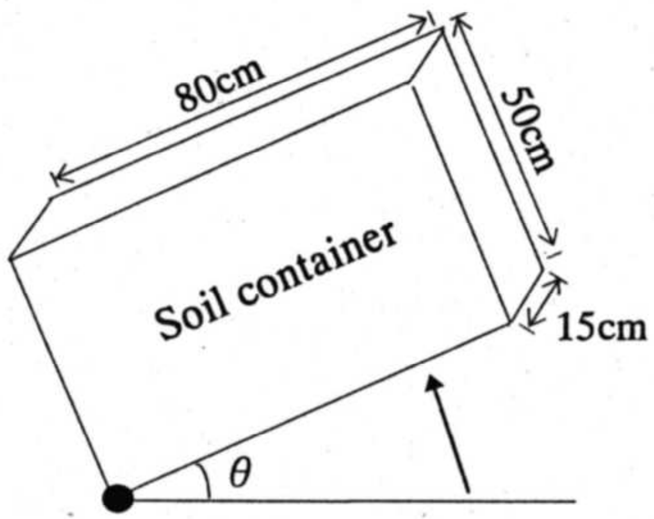
- FEM simulates closely the pre-peak behavior and peak load observed
- Simulates very well the bearing capacity in both types physical tests (1g and centrifuge condition)
- Can simulate not only the pressure level effect but also the particle size effect (shear band effect)

CENTRIFUGE STATIC TILTING MODEL TEST AND FINITE ELEMENT ANALYSIS

- The pseudo-static model test under centrifuge acceleration 50g was carried out
- The horizontal seismic force was applied by tilting the table
- The soil for this test was a mixture of Toyoura sand and kaolin

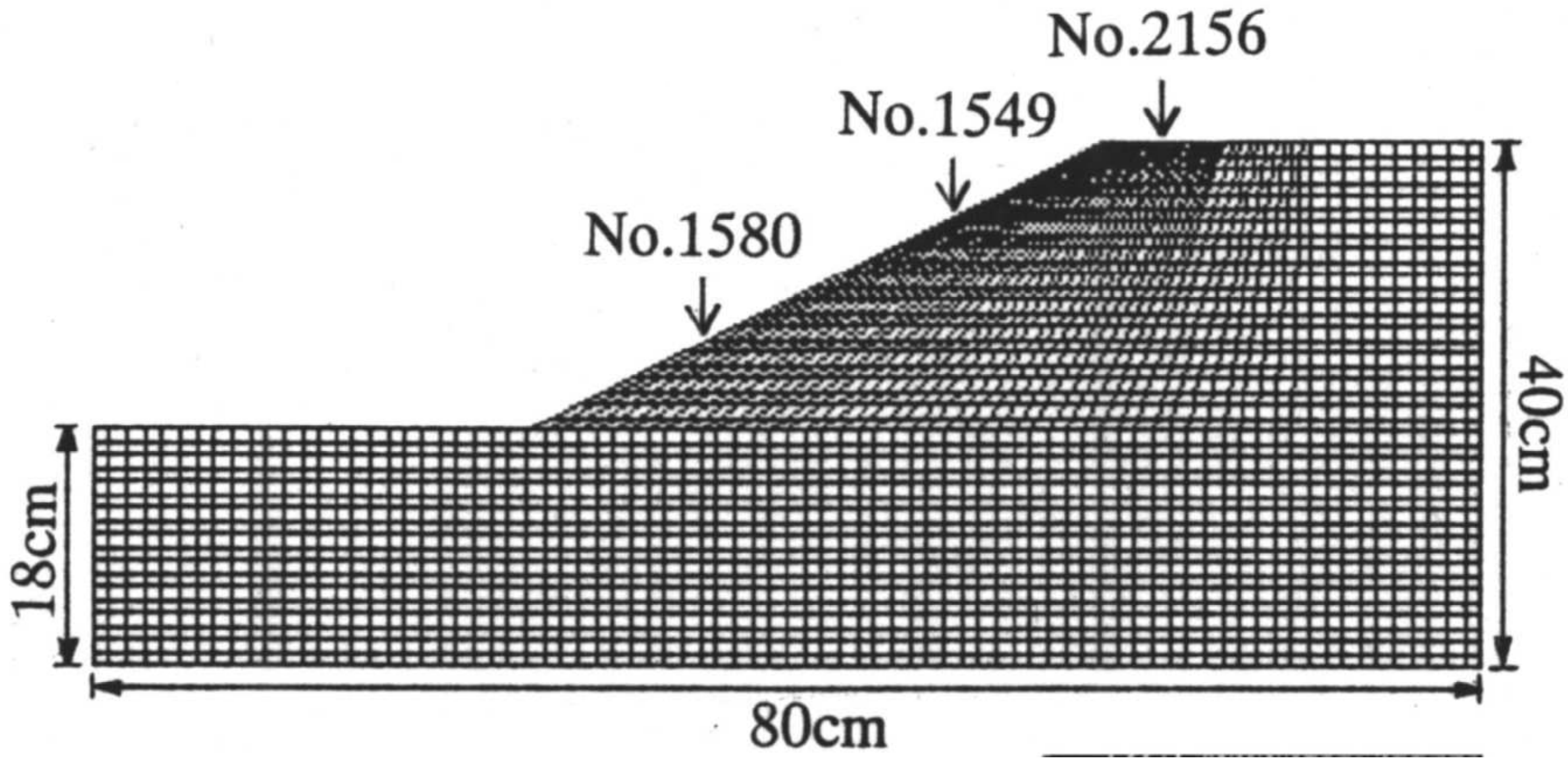
CENTRIFUGE TEST SIMULATER



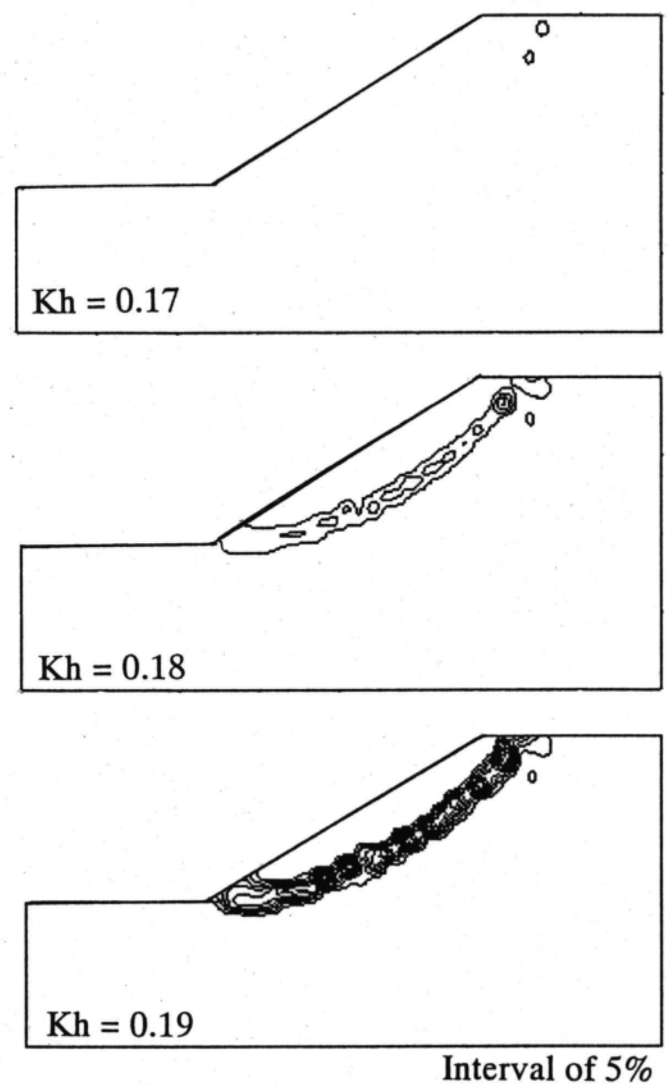


Tilting of soil container

FINITE ELEMENT MESH FOR CENTRIFUGE STATIC TILTING MODEL TEST



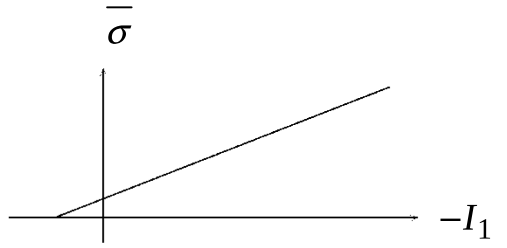
MAXIMUM SHEAR STRAINS BY FINITE ELEMENT ANALYSES



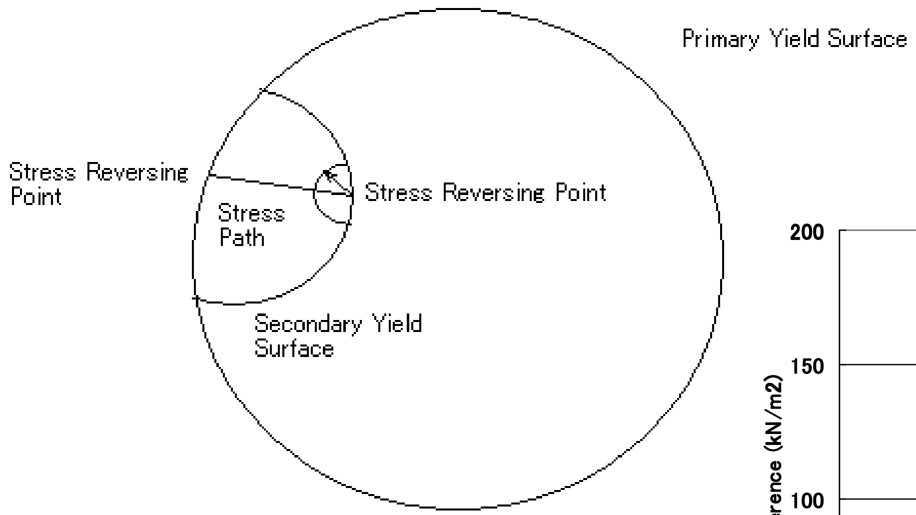
Kh: Seismic coefficient

These are considered to be a kind of push-over analysis

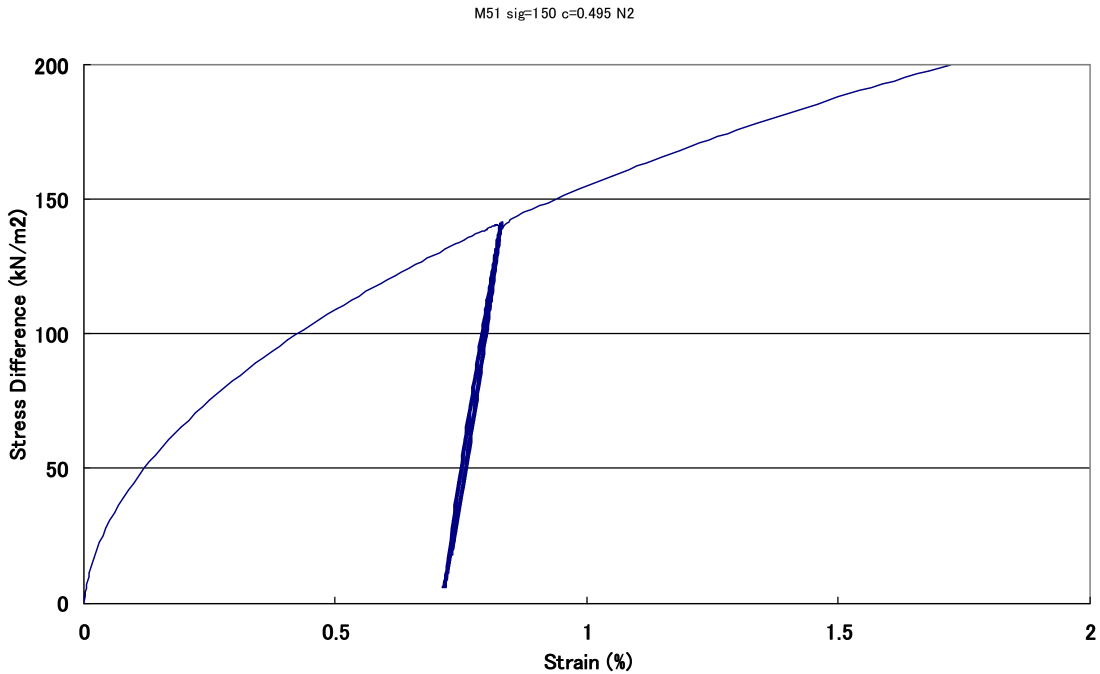
KINEMATIC HARDENING CONSTITUTIVE MODEL



$-I_1$ and $\overline{\sigma}$ relation



Mohr-Coulomb model takes pyramid shape in plane



Calculated stress strain relation of tri-axial test

KINEMATIC HARDENING MODEL WITHIN BOUNDING SURFACE

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

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

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

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

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

STRENGTH REDUCTION IN TOTAL STRESS ANALYSIS

- 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.

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 footing limit load analysis and pseudo-static slope analysis are good benchmarks regarding above remarks

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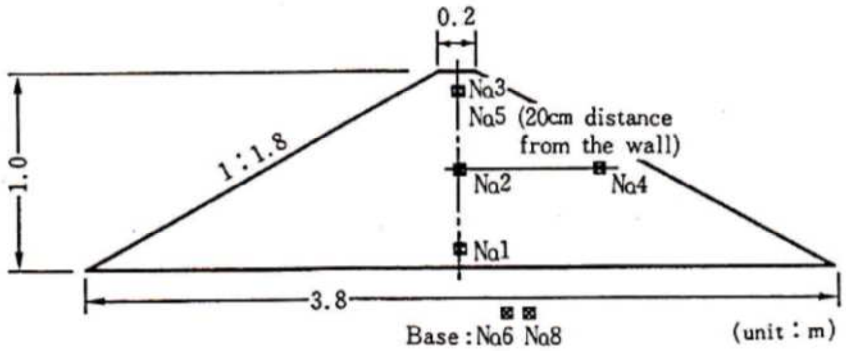
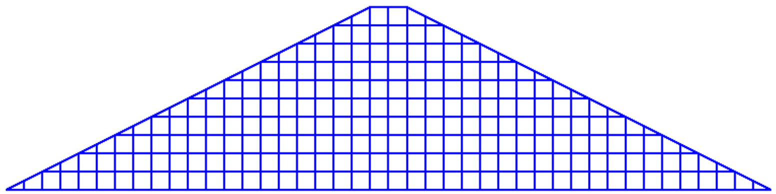
EMBANKMENT DAM MODEL AND LOCATION OF ACCELEROMETERS

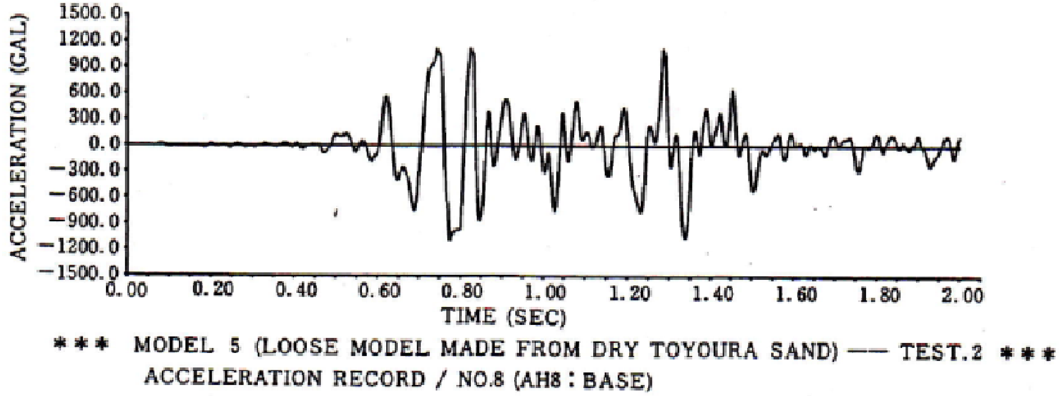


Embankment dam model and the location of accelerometers

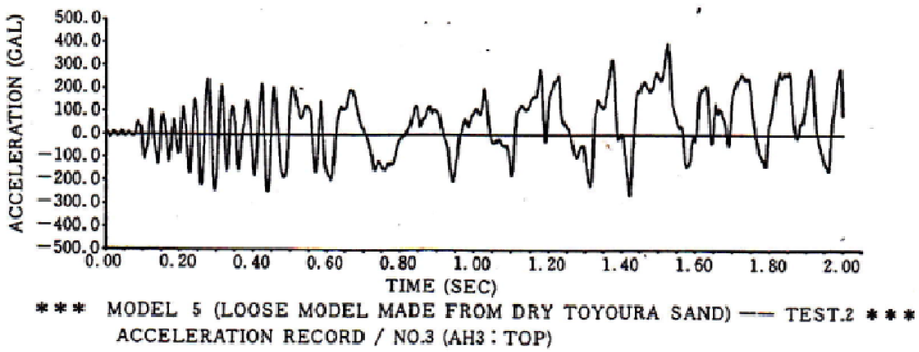


Finite element mesh used for the analysis

OBSERVED ACCELERATION AT THE BASE AND CREST OF DAM MODEL



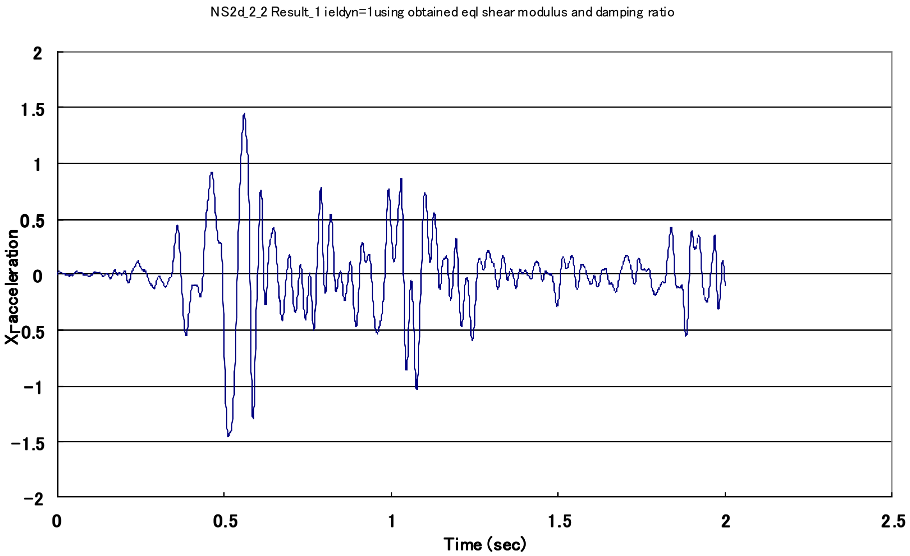
Input horizontal acceleration (observed at the base of shaking table)



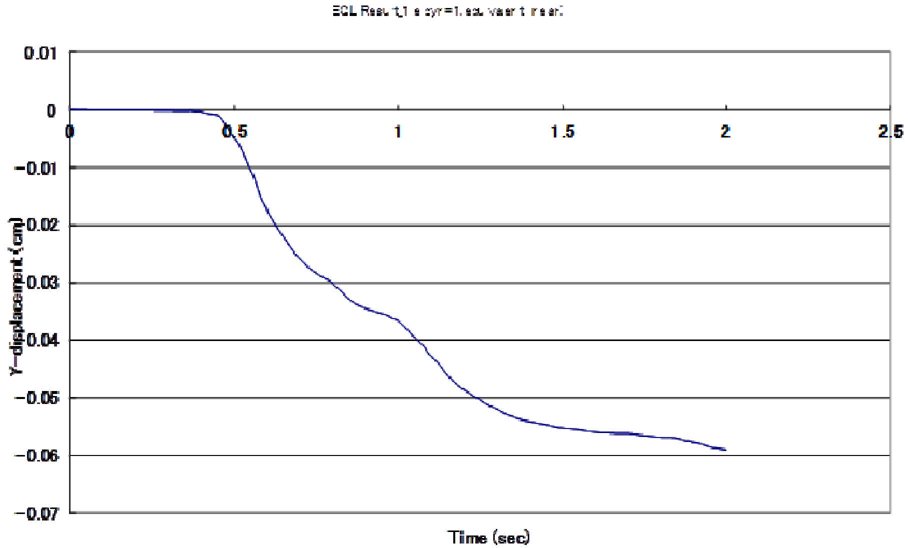
Observed acceleration at crest of the dam

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.



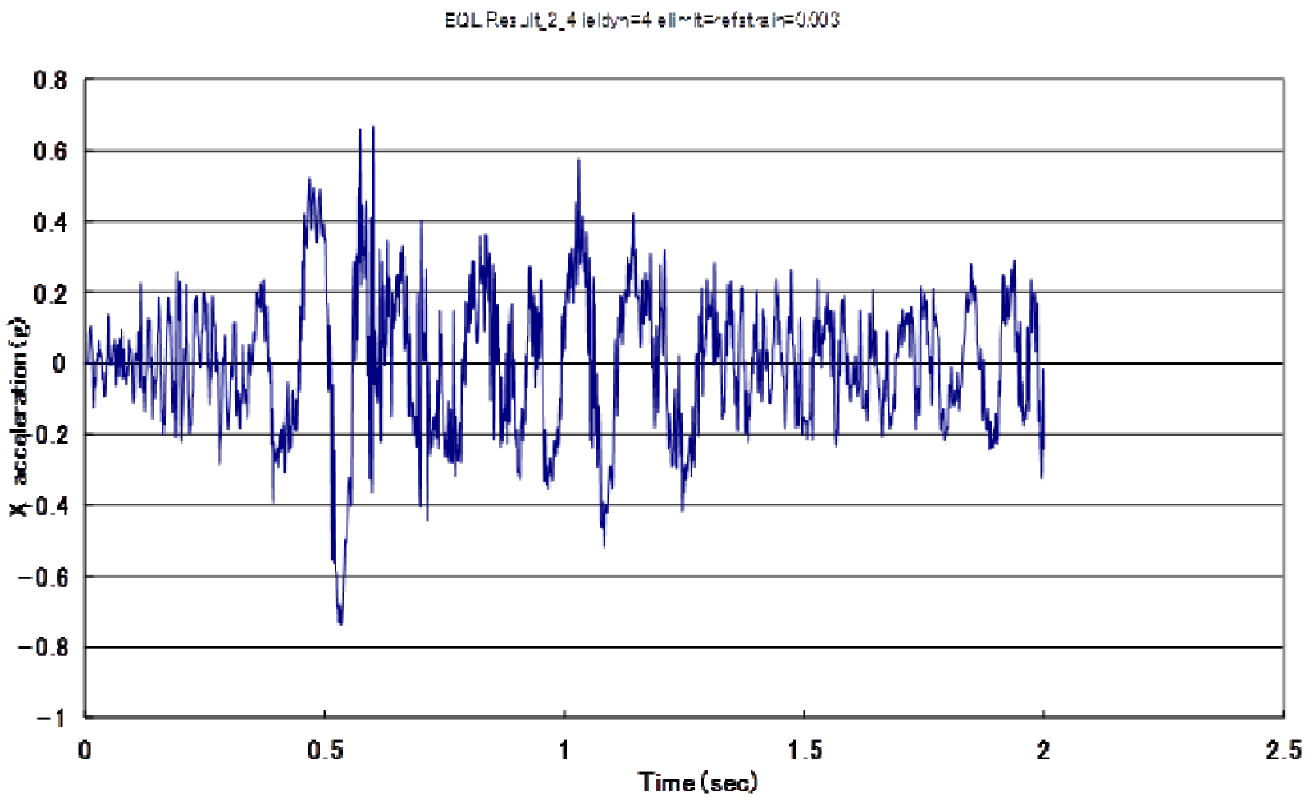
Computed horizontal acceleration at the center of dam crest



Computed settlement at the center of dam crest

COMPUTED ACCELERATION (2)

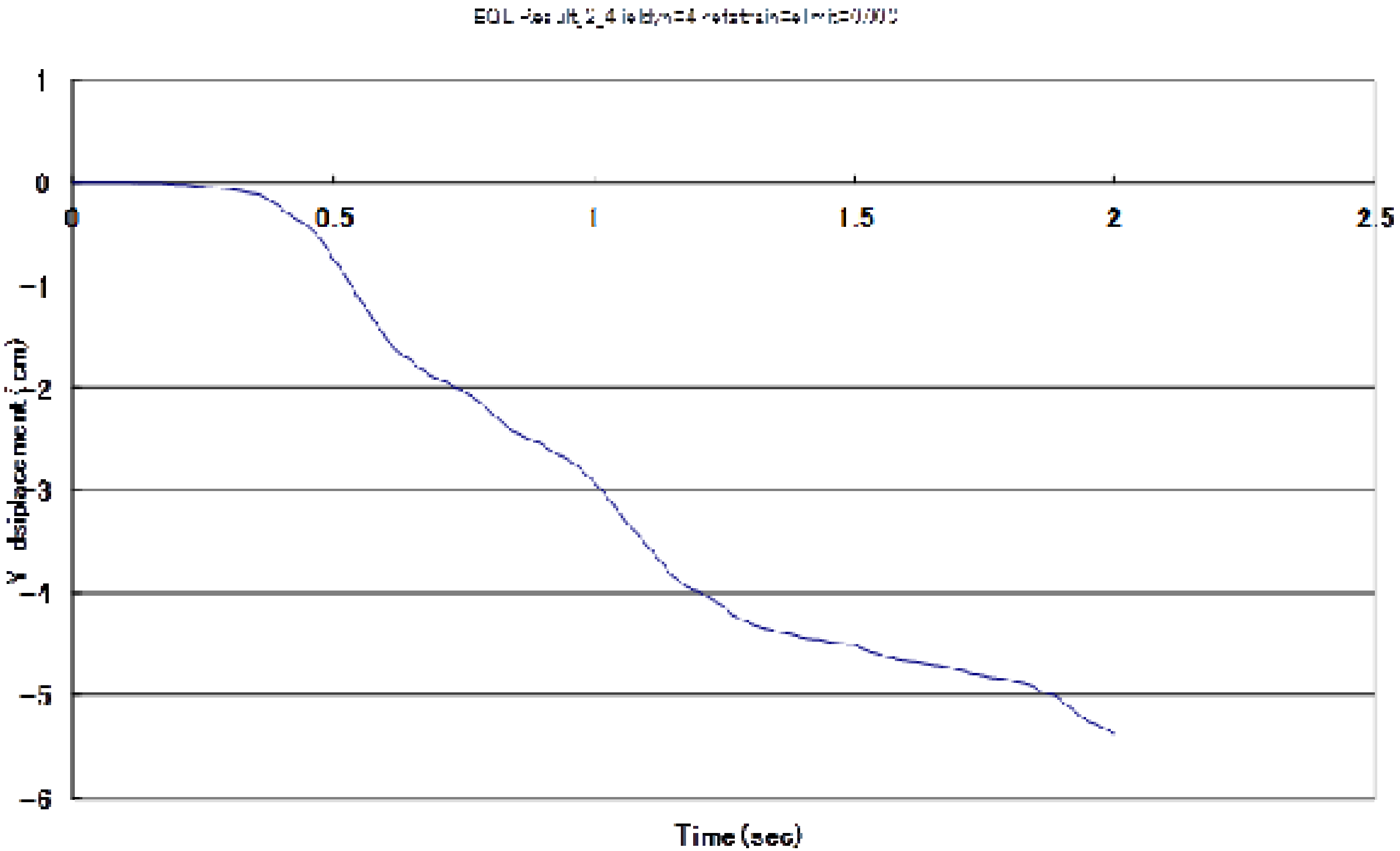
ELASTIC LIMIT: reference shear strain for Hardin-Drnevich equation



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

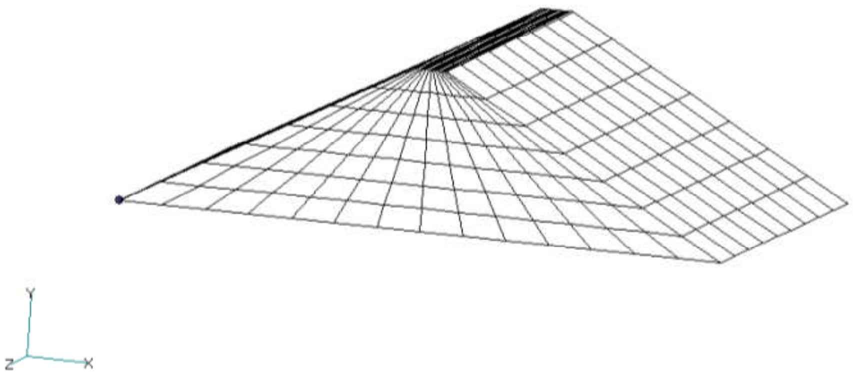
COMPUTED SETTLEMENT(2)

NONLINEAR ELASTIC LIMIT: reference shear strain for Hardin-Drnevich equation

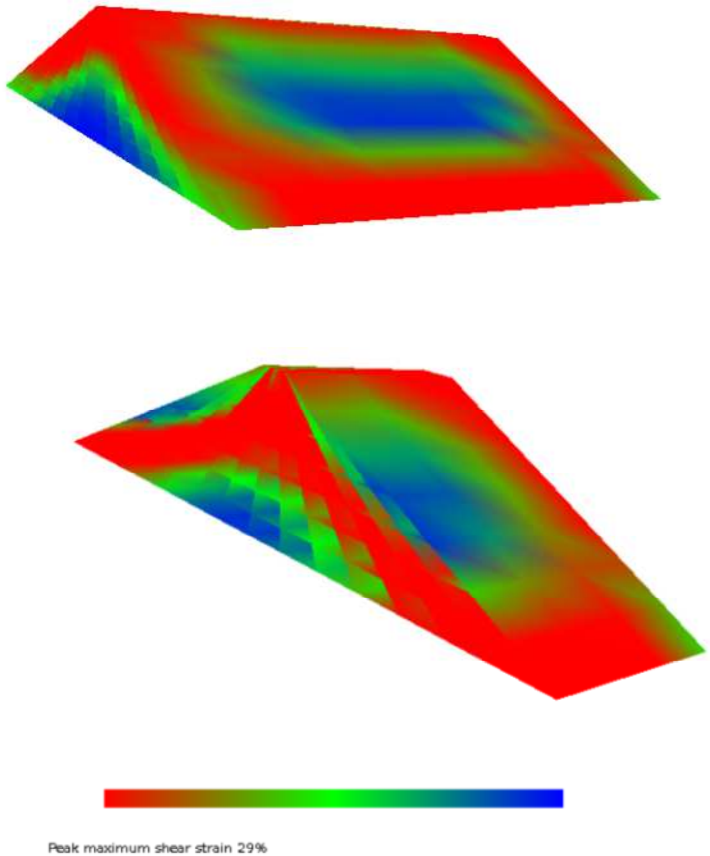


Computed settlement at the center of dam crest.
Rayleigh damping beta = 0, elastic limit = 0.0003

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%)

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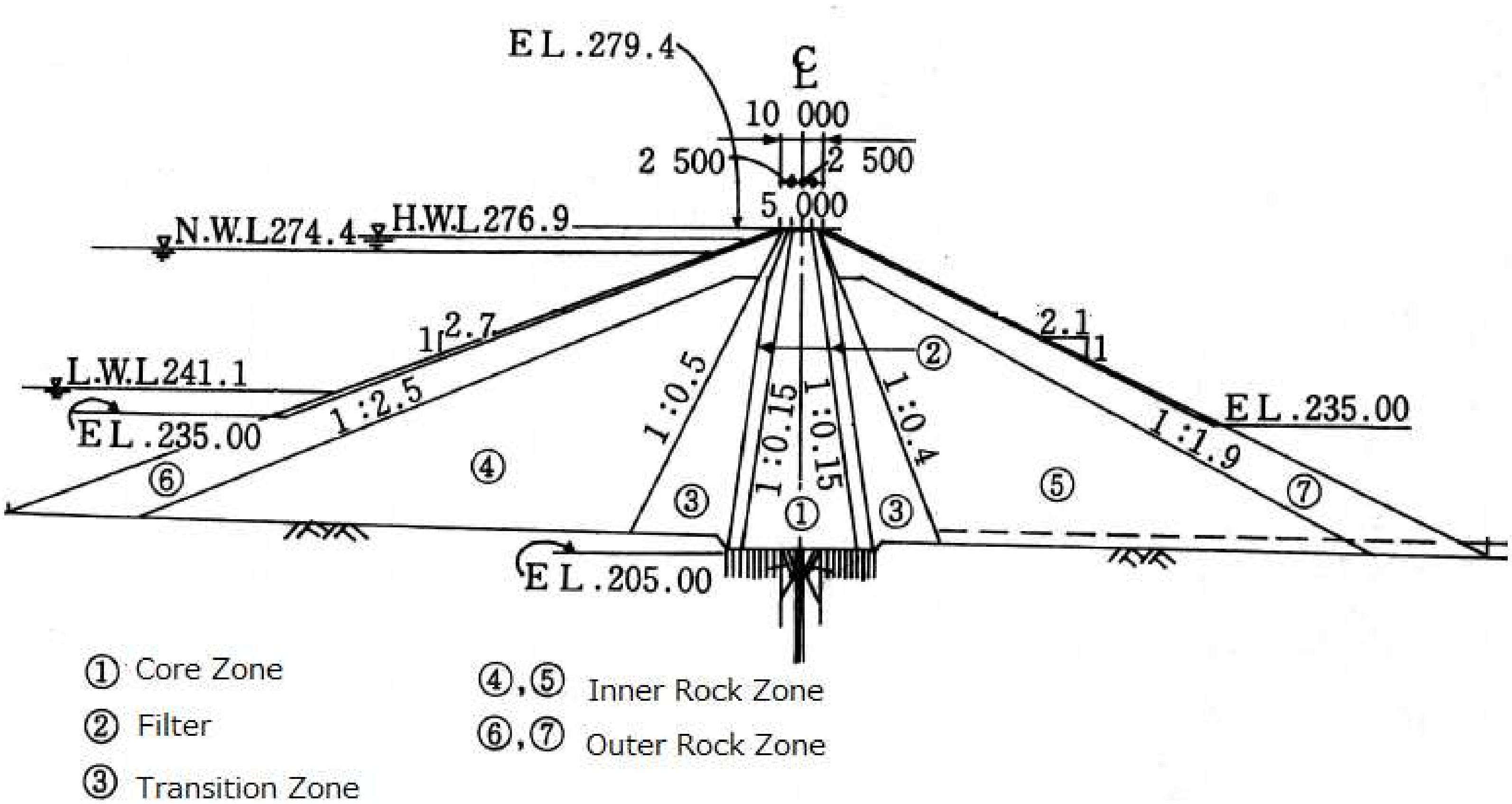
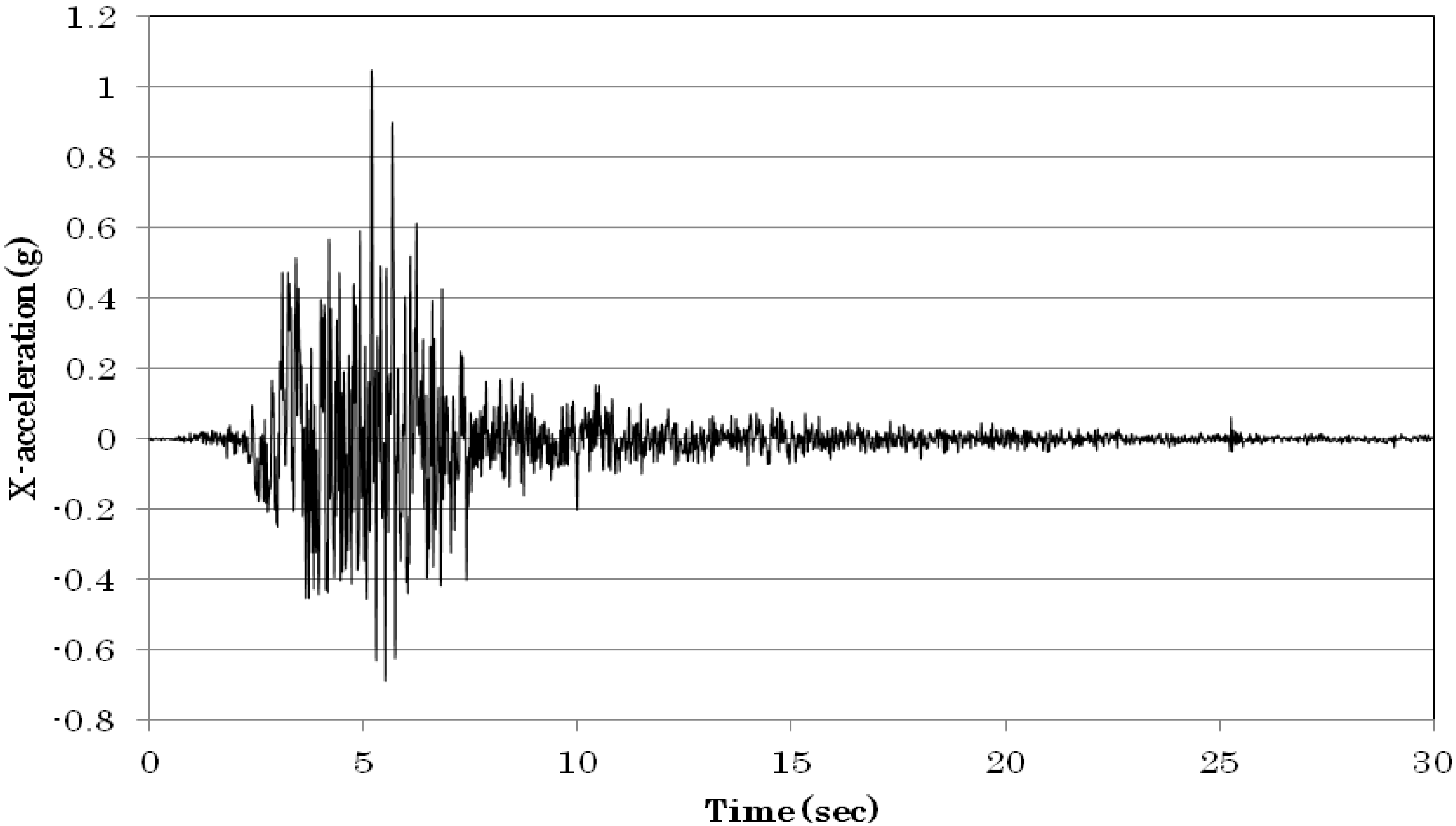
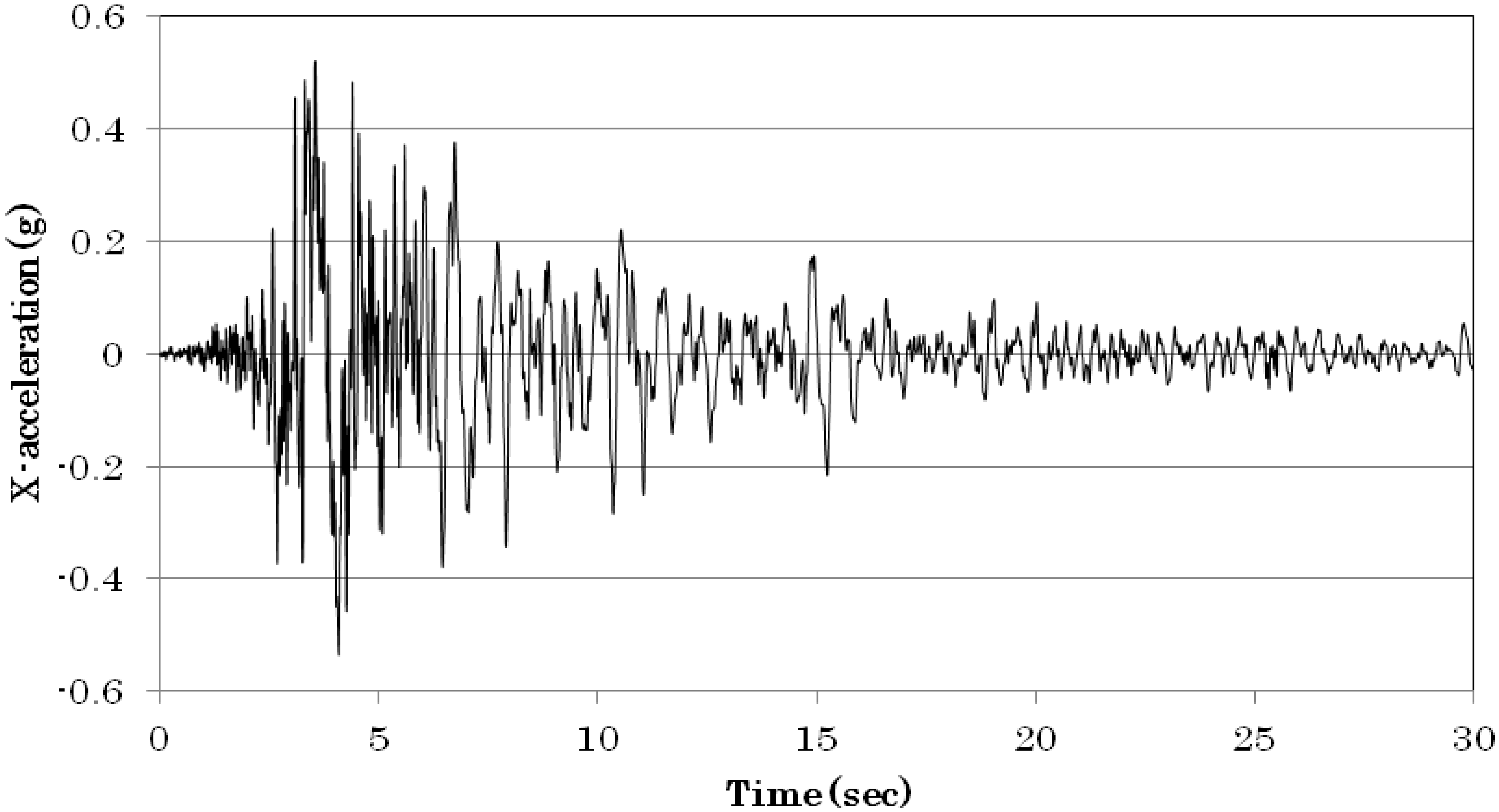
CROSS SECTION OF ARATOZAWA DAM



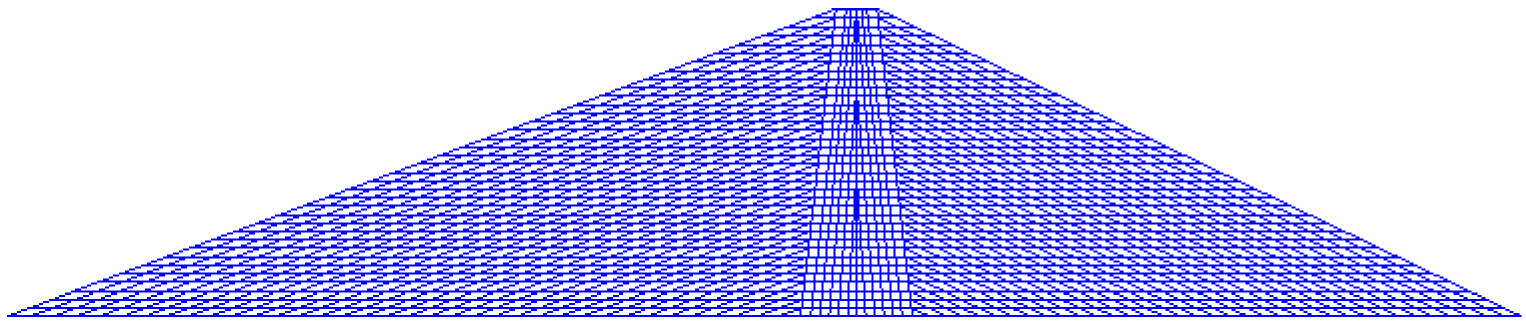
RCORDED ACCELERATION AT THE BASE OF ARATOZAWA DAM



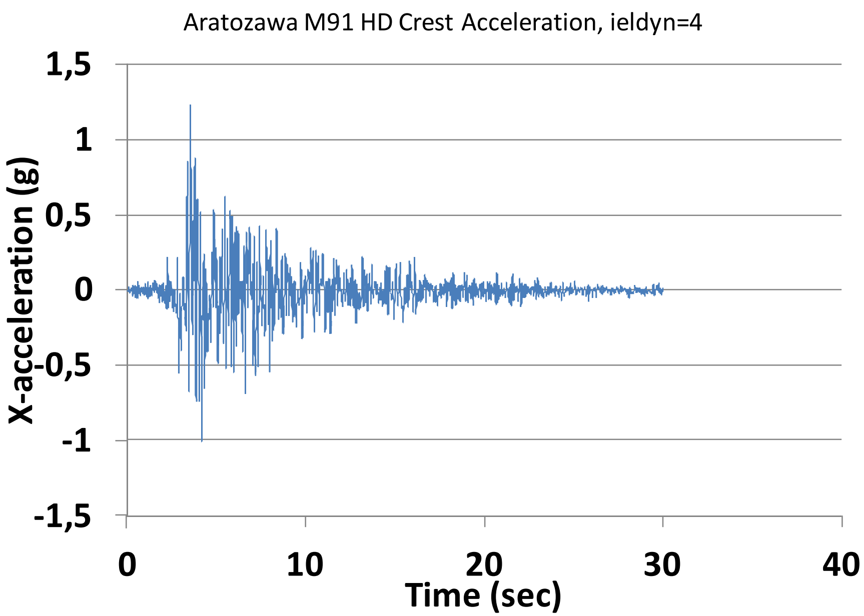
RCORDED ACCELERATION AT THE CREST OF ARATOZAWA DAM



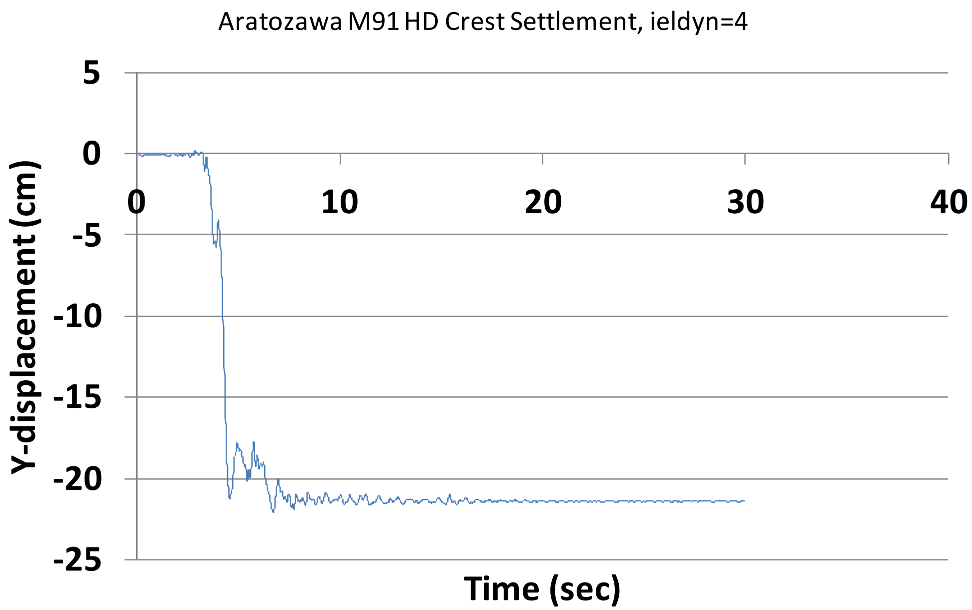
ARATOZAWA DAM
FINITE ELEMENT MODEL



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



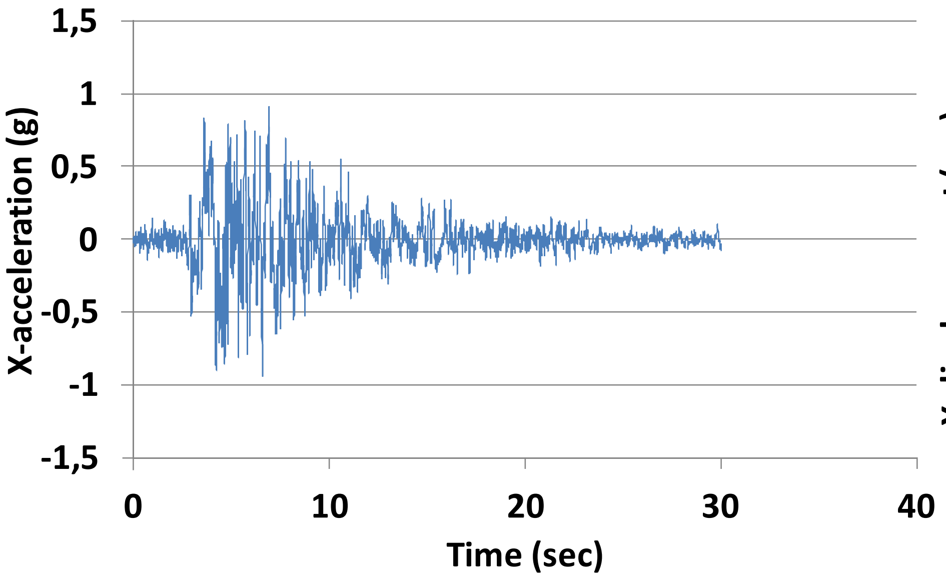
Computed horizontal acceleration at the crest of dam



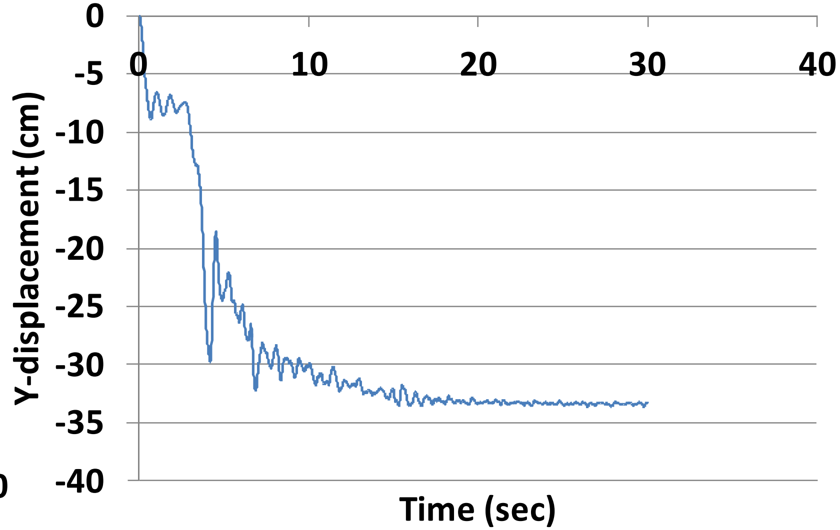
Computed settlement at the crest of dam

Simple strain softening constitutive model (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



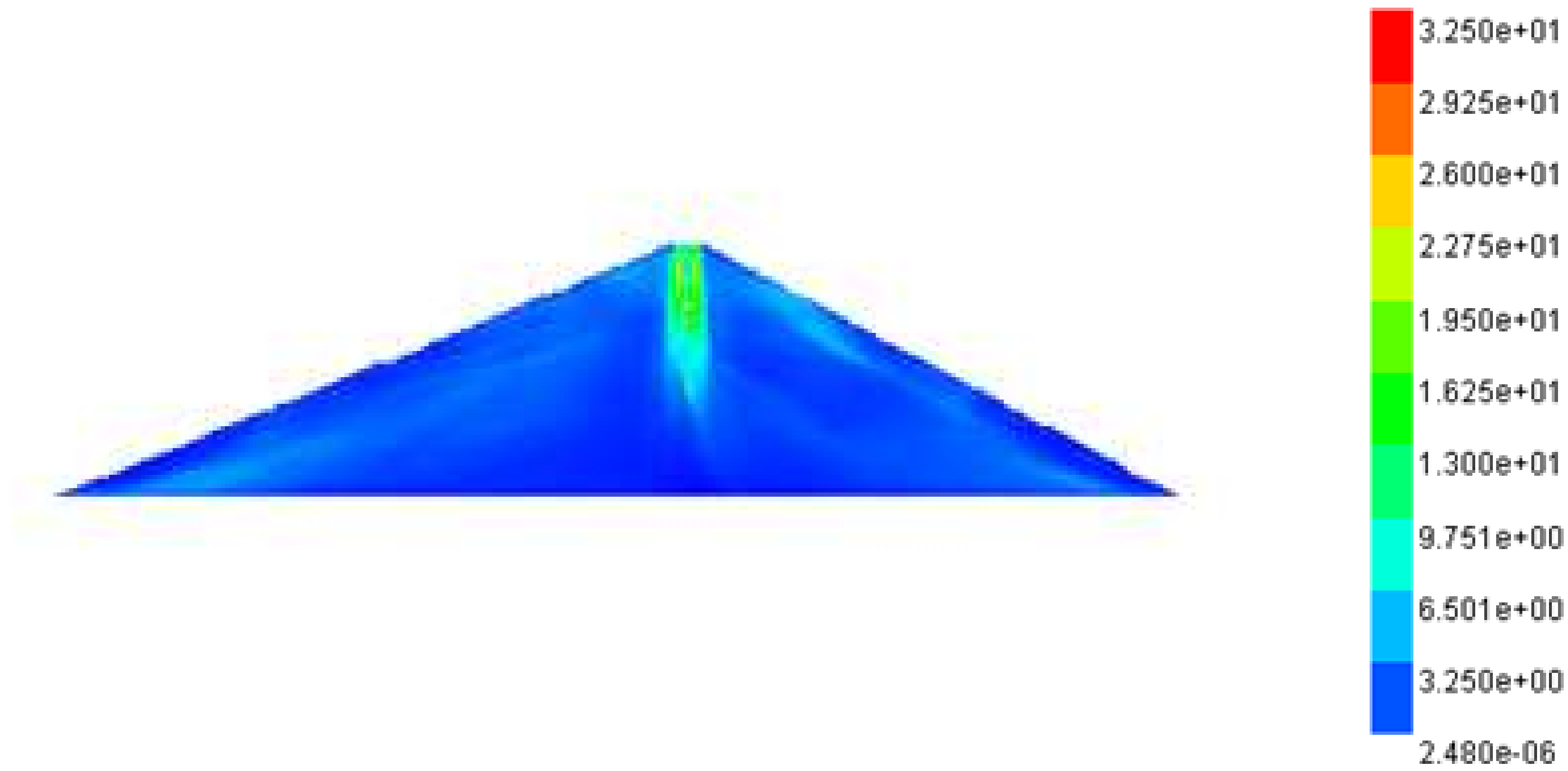
Computed settlement at the crest of dam

Kinematic hardening constitutive model

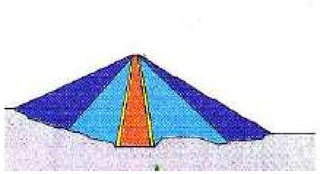
Effective stress analysis: Core zone is undrained, Rock zones are drained

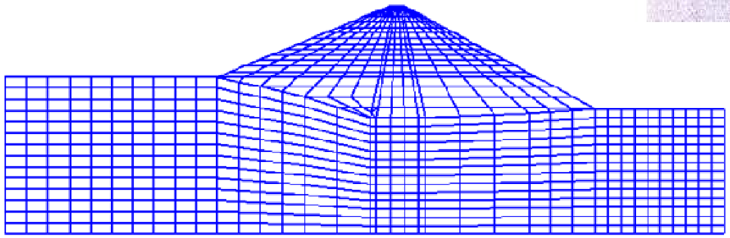
Elastic shear modulus and damping ratio : reference shear strain for Hardin-Drnevich equation

COMPUTED ARATOZAWA DAM MAXIMUM SHEAR STRAIN DISTRIBUTION (%)

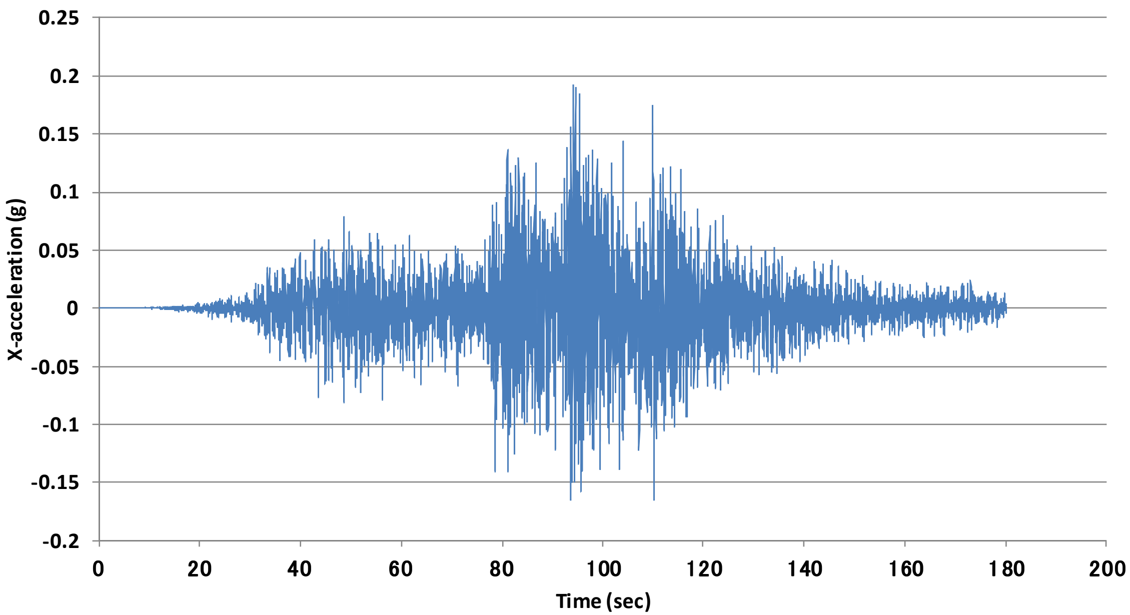


OOGAKI DAM DYNAMIC ANALYSIS BY SIMPLE ELASTO-PLASTIC MODEL





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



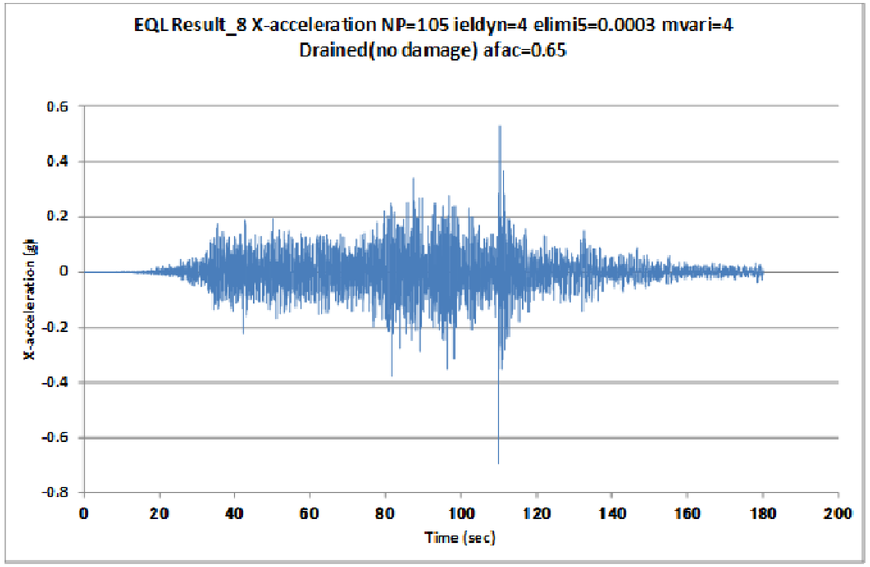
Estimated input acceleration at the base of the dam

COMPUTED RESULTS BY SIMPLE CONSTITUTIVE MODEL TRANSITION ZONE IS DRAINED

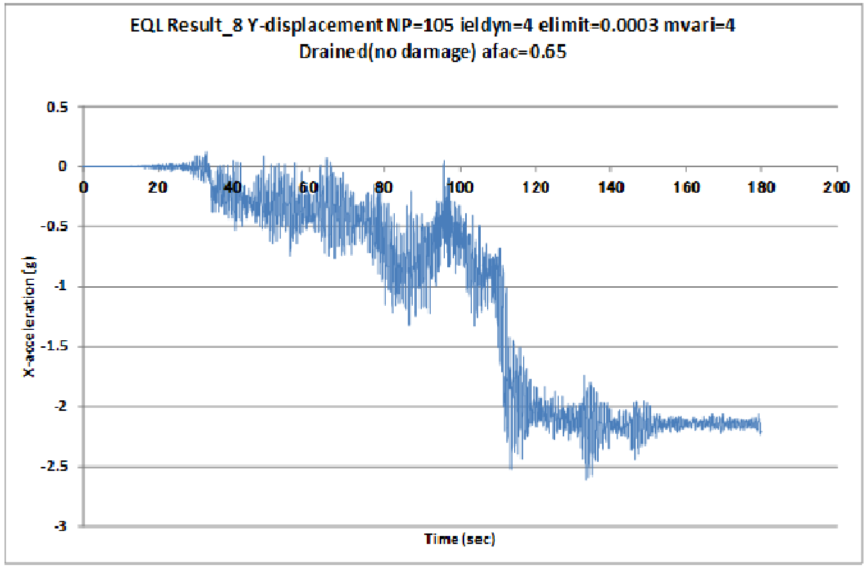
Core zone is undrained and another zones are completely drained

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



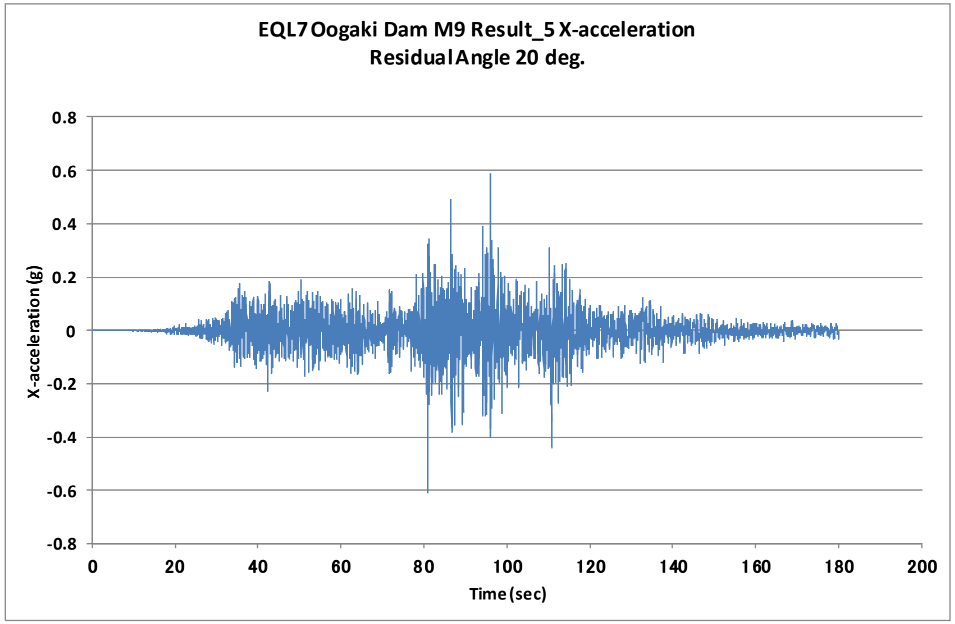
Computed crest settlement

COMPUTED RESULTS BY SIMPLE CONSTITUTIVE MODEL TRANSITION ZONE IS UNDRAINED

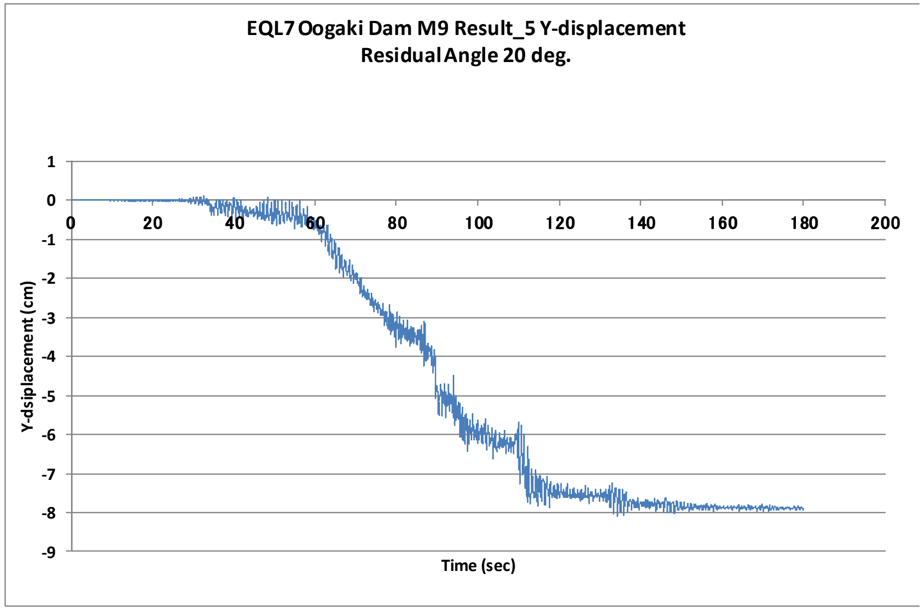
Core zone is undrained and another zones are completely drained

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.



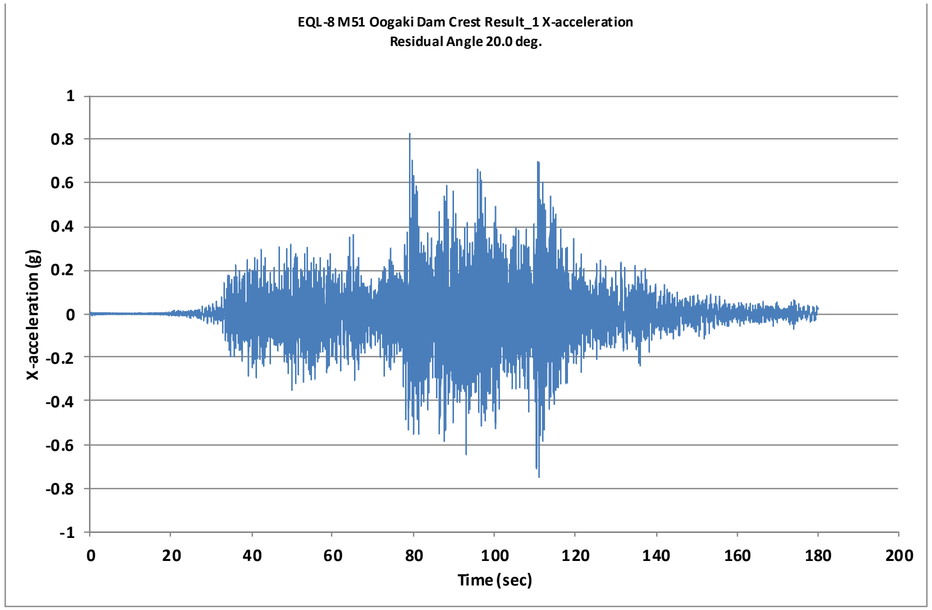
Computed crest acceleration



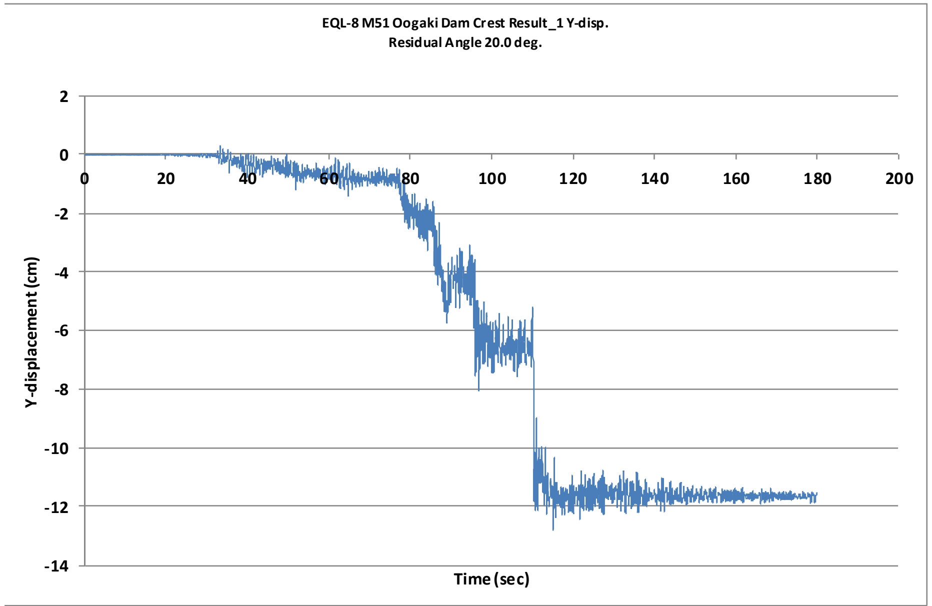
Computed crest settlement

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.



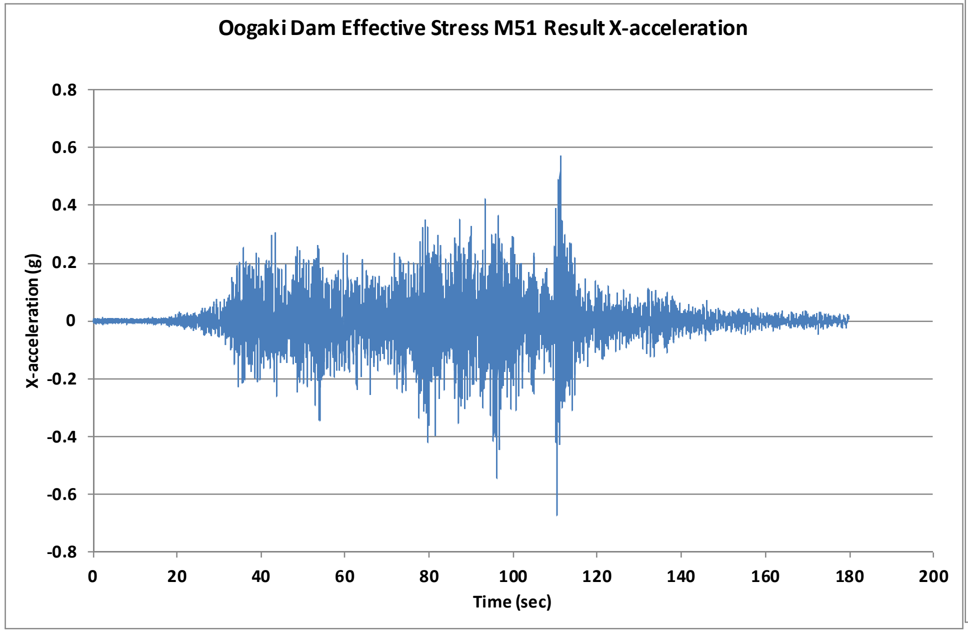
Computed crest acceleration by total stress analysis



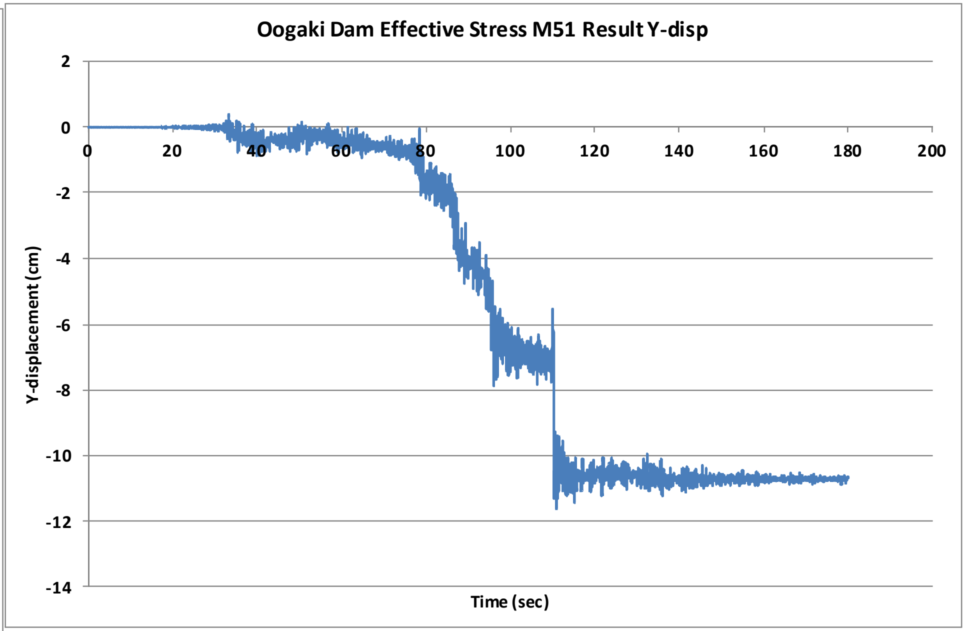
Computed crest settlement by total stress analysis

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.*

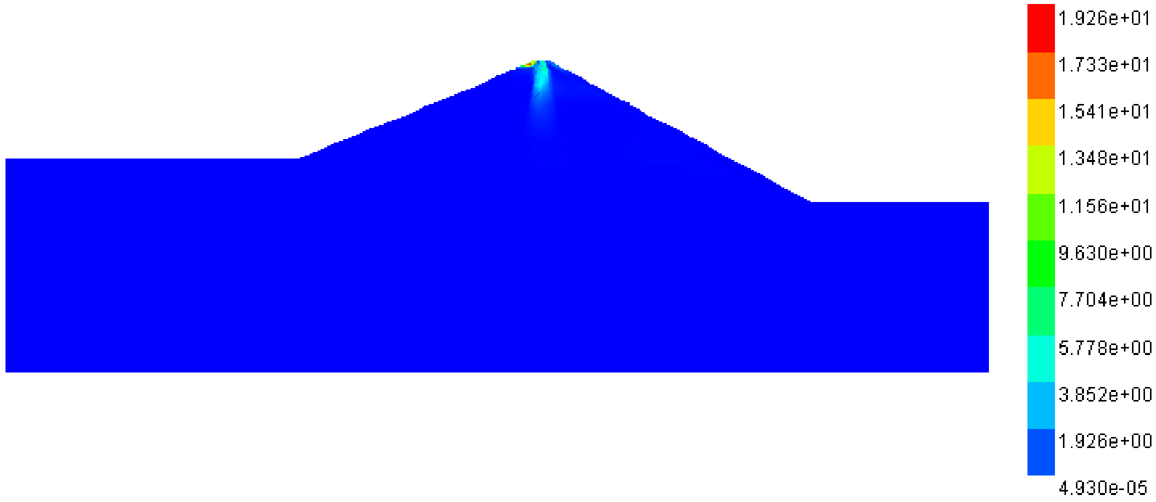


Computed crest acceleration by effective stress analysis



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.

A shear banding constitutive model incorporating a characteristics length of shear band is 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.

These computations are carried out by computer code **NONSOLAN**: Nonlinear Solid and Soil Analysis

THANK YOU FOR YOUR ATTENSION

DERIVATION OF STRESS-STRAIN RELATION

Total strain increment

$$d\boldsymbol{\varepsilon} = d\boldsymbol{\varepsilon}^{e} + s\lambda d\boldsymbol{\varepsilon}^{p}$$

$$d\boldsymbol{\varepsilon} = \mathbf{D}^{e-1} d\boldsymbol{\sigma} + s\lambda \frac{\partial \boldsymbol{\Phi}}{\partial \boldsymbol{\sigma}}$$

Multiply D^{e}

$$\mathbf{D}^{e} d\boldsymbol{\varepsilon} = d\boldsymbol{\sigma} + s\lambda \mathbf{D}^{e}\mathbf{b}$$

where $\left(\mathbf{b} = \frac{\partial \boldsymbol{\Phi}}{\partial \boldsymbol{\sigma}} \right)$

$$d\boldsymbol{\sigma} = \mathbf{D}^{e} d\boldsymbol{\varepsilon} - s\lambda \mathbf{D}^{e}\mathbf{b}$$

$s = F_b / F_e$ $\quad F_e$ is the area of the element

F_b is the area of a single shear band in each element

PARAMETER（λ）

$$d\boldsymbol{\varepsilon} = d\boldsymbol{\varepsilon}^e + s\lambda d\boldsymbol{\varepsilon}^p$$

Stress and strain must satisfy the yield condition

$$d\boldsymbol{\varepsilon} = \mathbf{D}^{e-1} d\boldsymbol{\sigma} + s\lambda \frac{\partial \boldsymbol{\Phi}}{\partial \boldsymbol{\sigma}}$$

Multiply both sides $\mathbf{a}^T \mathbf{D}^e$

$$\mathbf{a}^T \mathbf{D}^e d\boldsymbol{\varepsilon} = \mathbf{a}^T d\boldsymbol{\sigma} + s\lambda \mathbf{a}^T \mathbf{D}^e \mathbf{b}$$

$$= \left(\mathbf{A} + s\mathbf{a}^T \mathbf{D}^e \mathbf{b}\right)\lambda$$

$$\lambda = \frac{\mathbf{a}^T \mathbf{D}^e d\boldsymbol{\varepsilon}}{\mathbf{A} + s\mathbf{a}^T \mathbf{D}^e \mathbf{b}}$$

$$f(\sigma, \kappa) = 0$$

$df = 0$ （consistency condition）

$$\frac{\partial f}{\partial \boldsymbol{\sigma}} d\boldsymbol{\sigma} + \frac{\partial f}{\partial \boldsymbol{\kappa}} d\boldsymbol{\kappa} = 0$$

$$\mathbf{a}^T d\boldsymbol{\sigma} - \mathbf{A}\lambda = 0$$

$$d\boldsymbol{\sigma} = \mathbf{a}^{T-1} \mathbf{A}\lambda$$

$$\left(\mathbf{a}^T = \frac{\partial f}{\partial \boldsymbol{\sigma}}, \quad \mathbf{A} = -\frac{1}{\lambda}\frac{\partial f}{\partial \boldsymbol{\kappa}} d\boldsymbol{\kappa}\right)$$

STRESS-STRAIN RELATION

$$d\boldsymbol{\sigma} = \mathbf{D}^e d\boldsymbol{\varepsilon} - s\lambda \mathbf{D}^e \mathbf{b}$$

Substituting λ

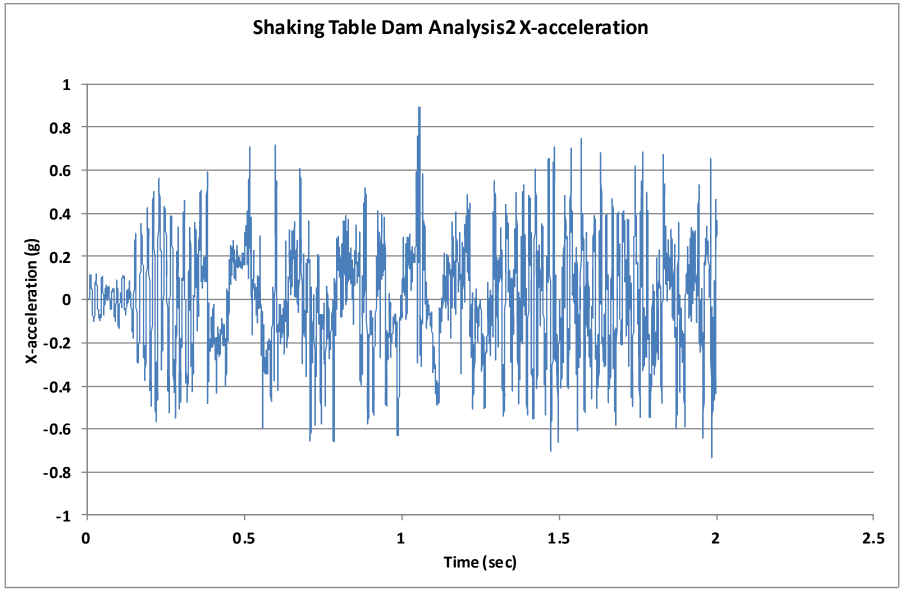
$$\left(\lambda = \frac{\mathbf{a}^T \mathbf{D}^e d\boldsymbol{\varepsilon}}{\mathbf{A} + s\mathbf{a}^T \mathbf{D}^e \mathbf{b}} \right)$$

We can obtain the next stress-strain relation

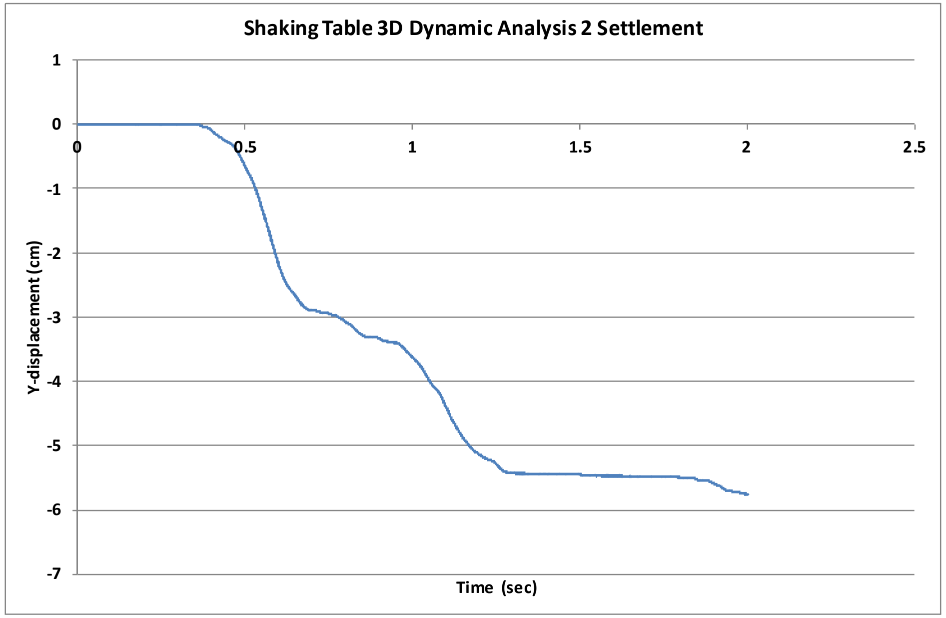
$$d\boldsymbol{\sigma} = \left(\mathbf{D}^e - \frac{s\mathbf{D}^e \mathbf{b}\mathbf{a}^T \mathbf{D}^e d\boldsymbol{\varepsilon}}{\mathbf{A} + s\mathbf{a}^T \mathbf{D}^e \mathbf{b}} \right) d\boldsymbol{\varepsilon}$$

$$\left(\mathbf{a}^T = \frac{\partial f}{\partial \boldsymbol{\sigma}} \quad , \quad \mathbf{b} = \frac{\partial \Phi}{\partial \boldsymbol{\sigma}} \quad , \quad \mathbf{A} = -\frac{1}{\lambda} \frac{\partial f}{\partial \boldsymbol{\kappa}} d\boldsymbol{\kappa} \right)$$

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

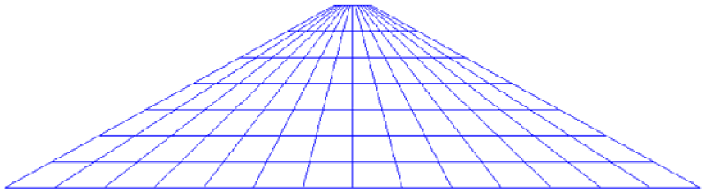


Computed horizontal acceleration at the crest of model dam

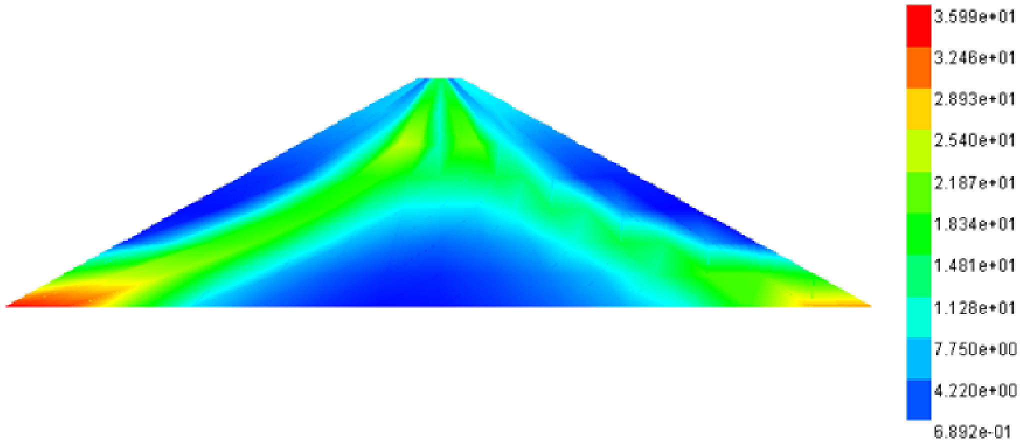


Computed settlement at the crest of model dam

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

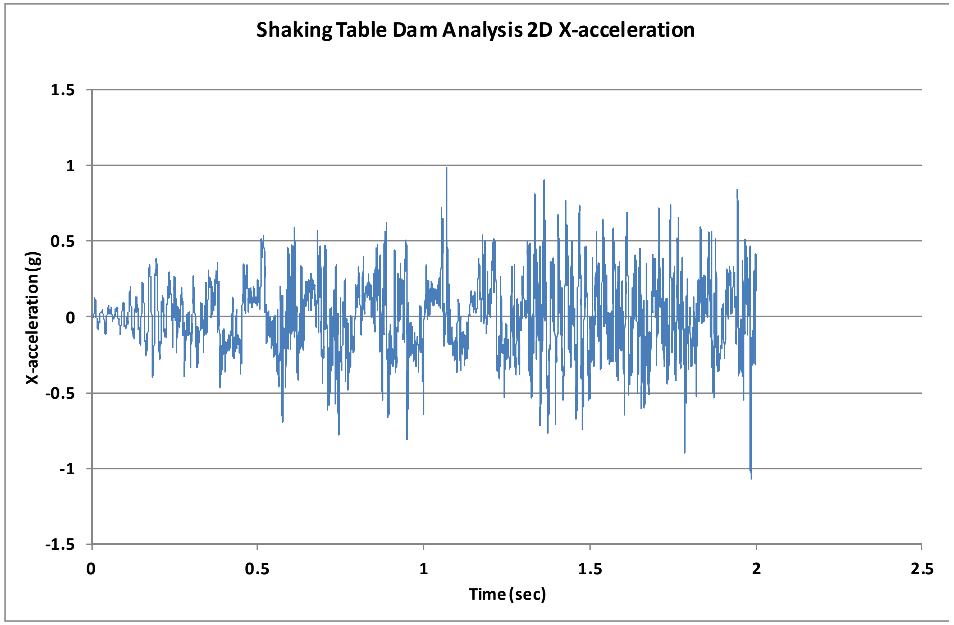


Two dimensional finite element mesh

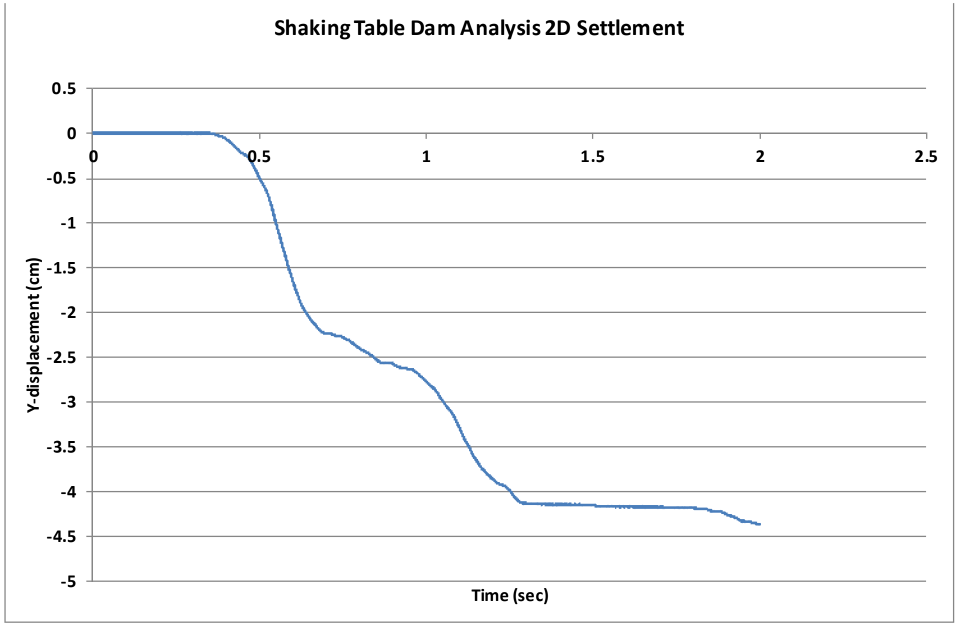


Computed maximum shear strain of model dam after shaking (peak strain 30%)

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

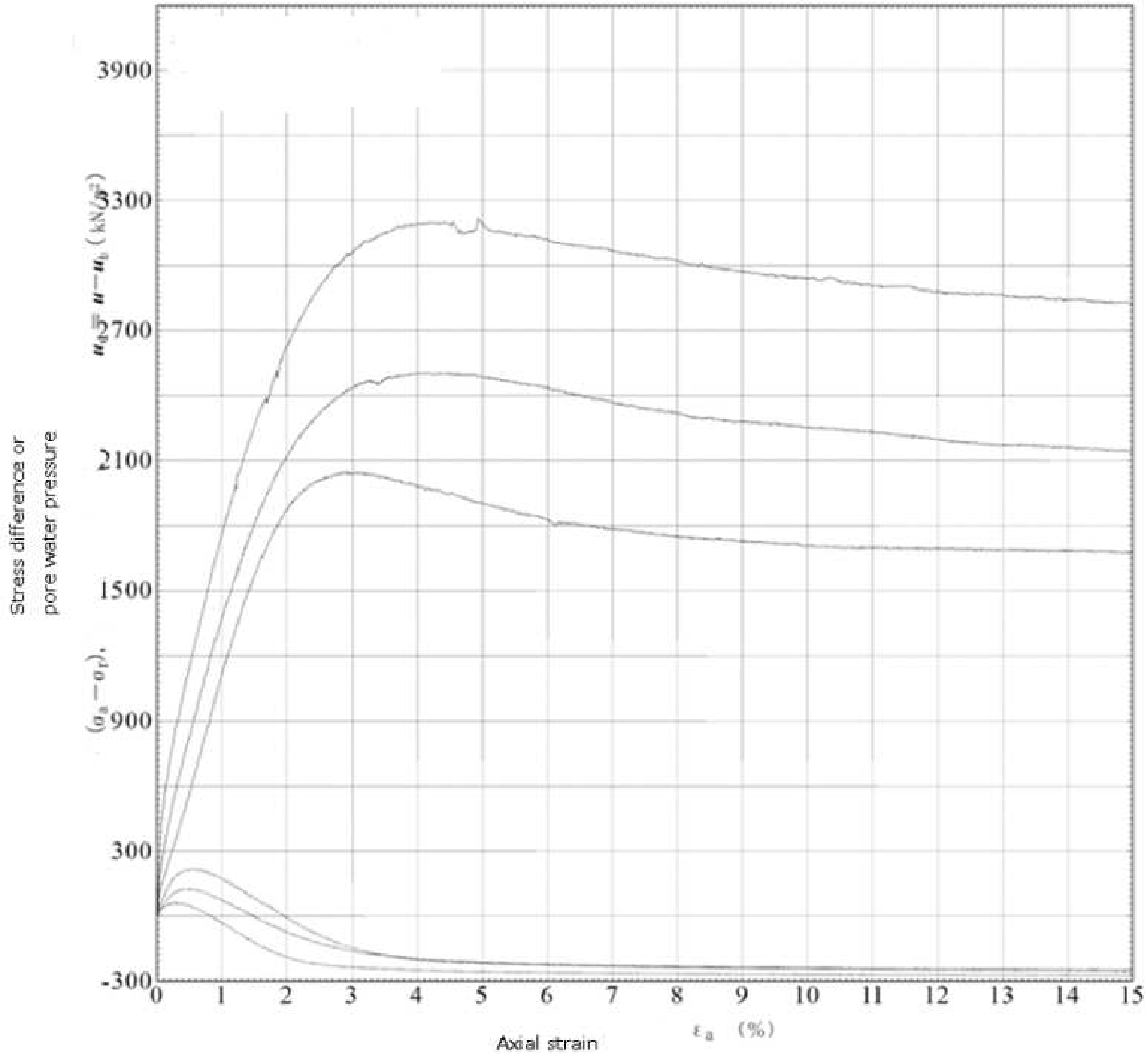


Computed horizontal acceleration at the crest of model dam



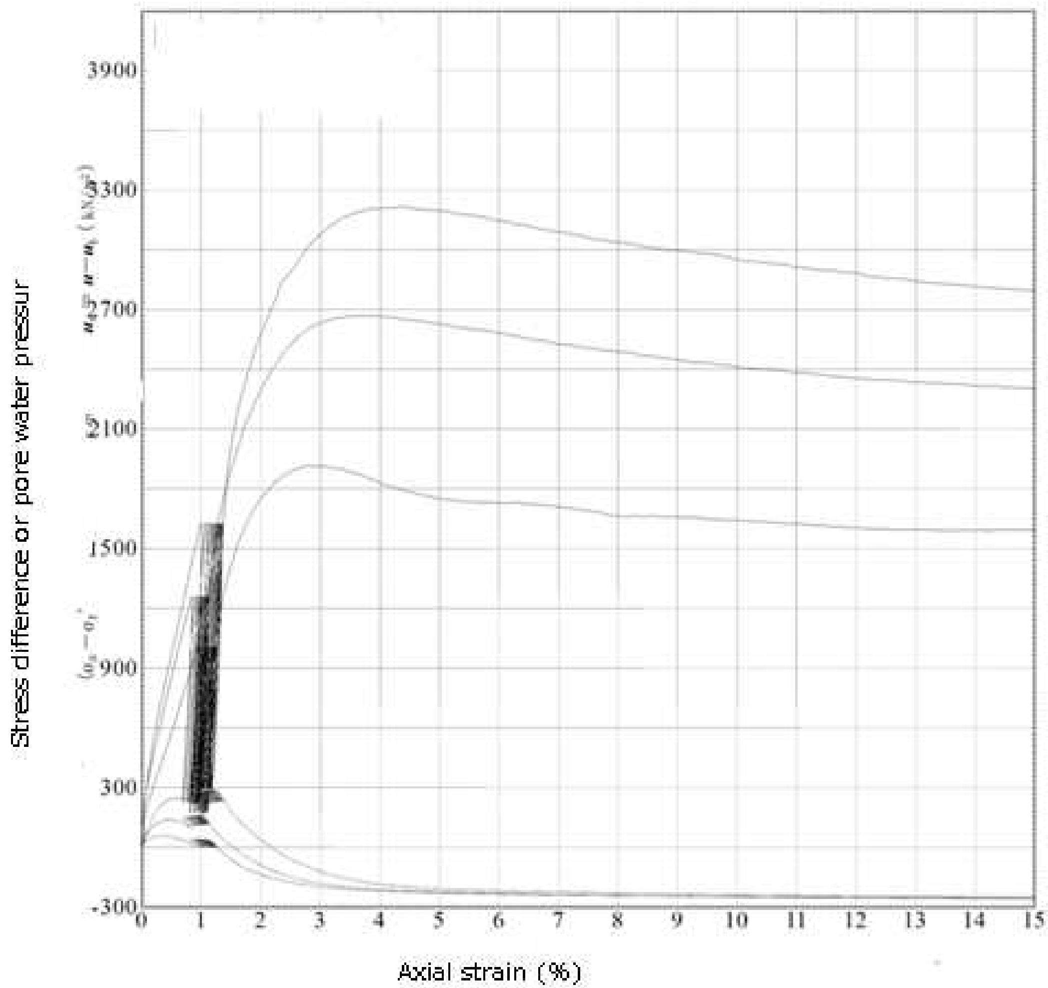
Computed settlement at the crest of model dam

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



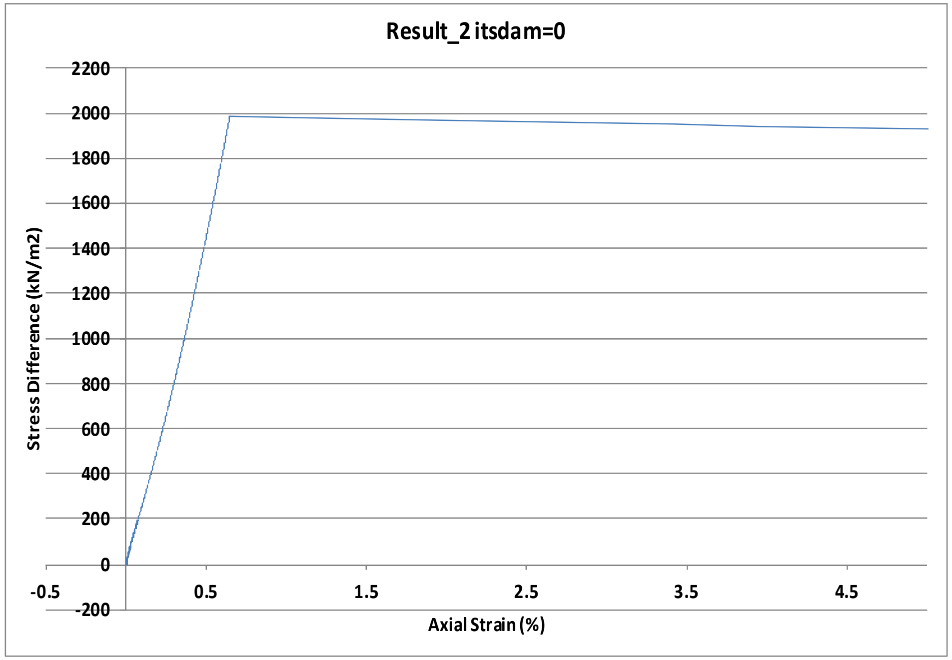
Confining pressure: 200,400,600(kN/m2) monotonic loading

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

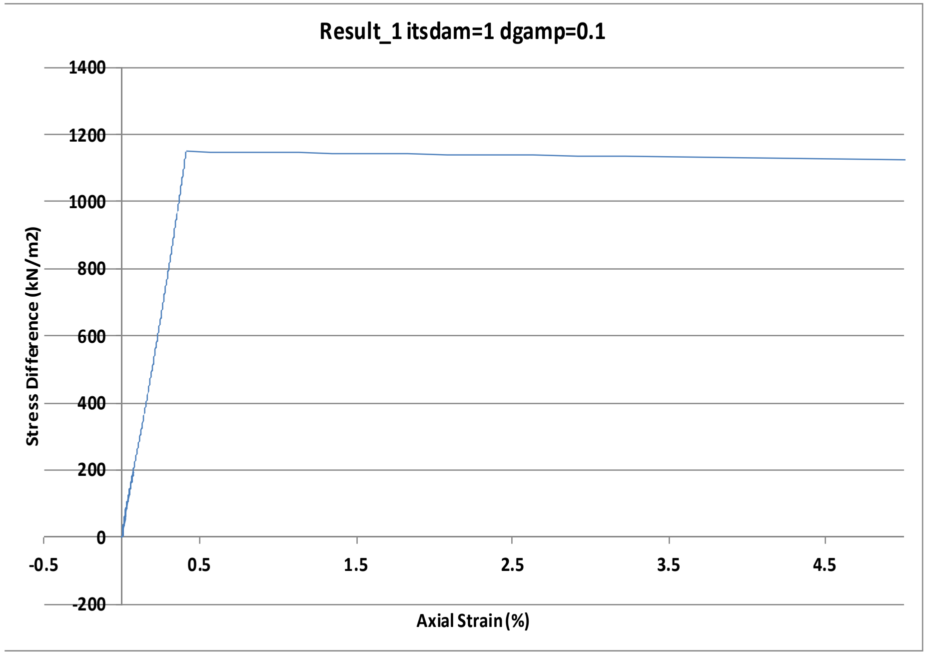


Confining pressure: 200,400,600(kN/m2) Cyclic and monotonic loading

COMPUTED STRESS-STRAIN RELATION BY SIMPLE CONSTITUTIVE MODEL



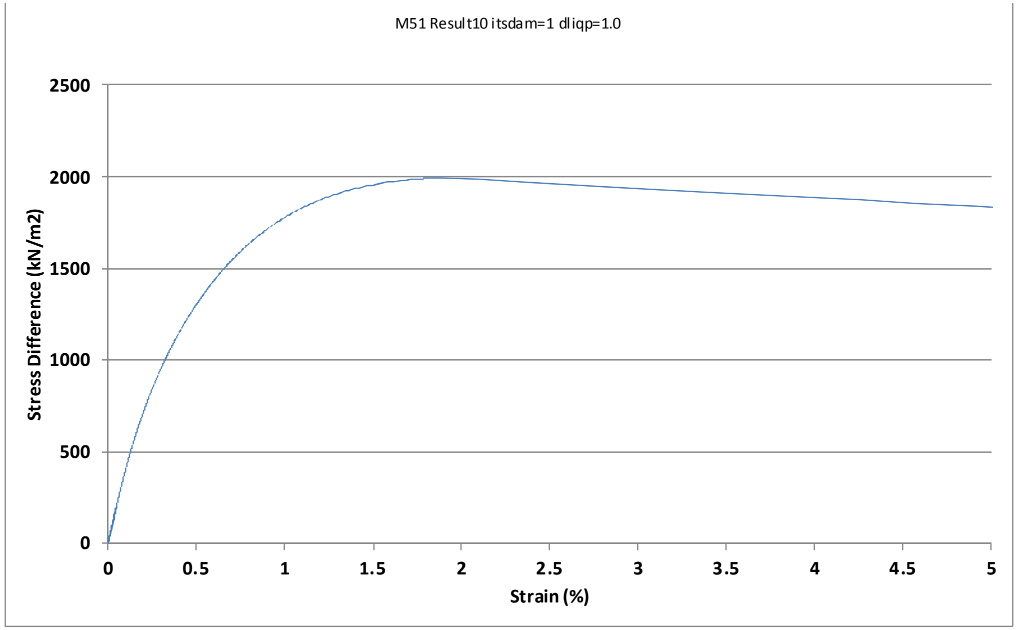
a)



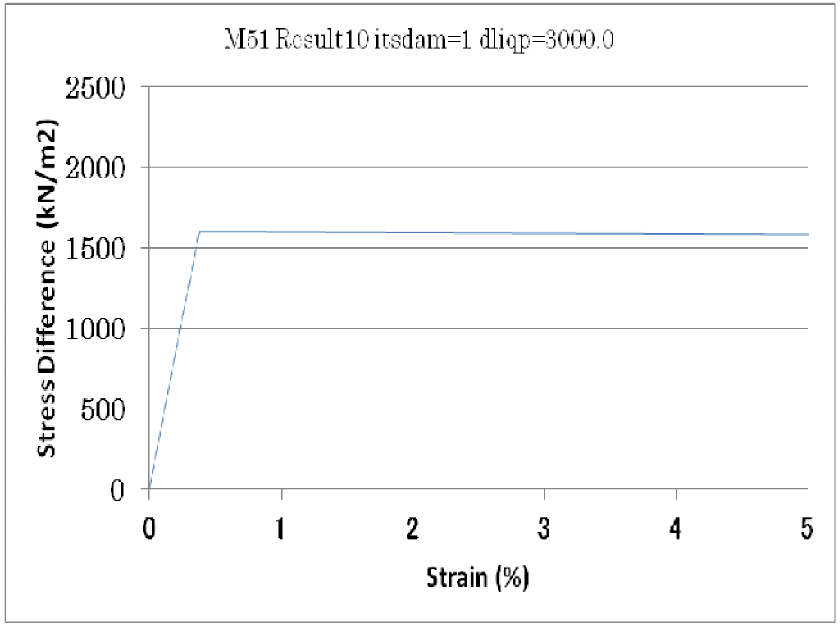
b)

Confining pressure is 200 (kN/m2) (applied cyclic load: 0 - 200kN/m2) $\phi_p = 36.6^\circ$, $\phi_r = 35.0^\circ$,
B = 0.7, C = 0.6, D = 0.7, cohesion = 354 *kPa*,
a) without factor of equivalent plastic parameter , b) factor is 0.1

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



a)



b)

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,
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