DYNAMIC RESPONSE APPROACH AND METHODOLOGY

Traditional seismic stability procedures

vs

coupled effective-stress approach.

Traditional seismic stability procedures:

- Empirical and laboratory corrections and simplified procedure to evaluate the potential for liquefactions of embankment and foundation soils-SPT, CPT or Vs based methods.
- Limit equilibrium stability analyses to evaluate post-earthquake stability.
- Newmark-type estimates of permanent deformation.

What Traditional Seismic Stability Assessment Give you and Can't Give you

- DO:
  - State-of-practice estimates of the potential for occurrence or non-occurrence of liquefaction during and at the end of strong earthquake shaking.
- DON'T:
  - model the progressive changes in the soil's state during earthquake shaking,
  - the potential for buildup of pore water pressure,
  - The occurrence of liquefaction,
  - The resulting permanent deformations during and after the earthquake

Coupled Effective-Stress Analysis of this Webinar

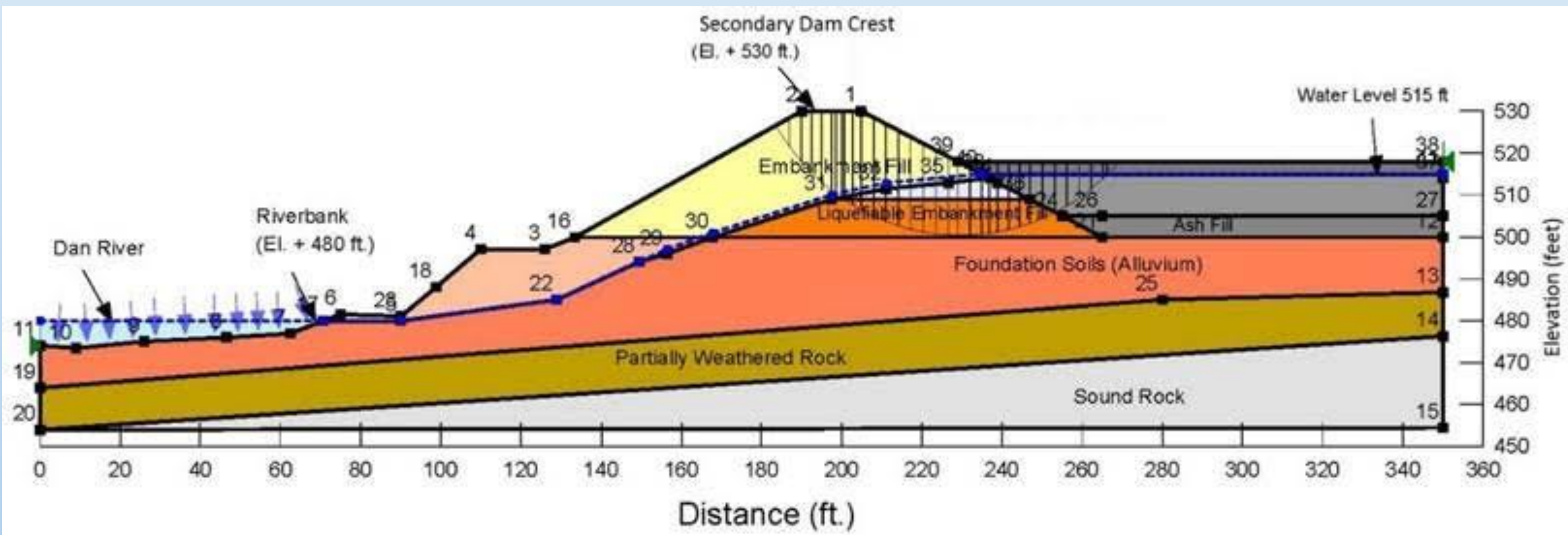
- To estimate the performance of the embankment during and after earthquakes
  - model the progressive changes in the soil's state during earthquake shaking,
  - the pore water pressure build up,
  - If liquefaction occurs or not,
  - The resulting permanent deformations during and after the earthquake

Problem Descriptions

- The pond is located along a river. Liquefaction screening results indicate that zones within the pond embankment are potentially susceptible to liquefaction based on the estimated seismicity for the design seismic event with a 2,475 year return period. The susceptible zones are composed of embankment fill at depths ranging from 15 to 30 feet below the ground surface.

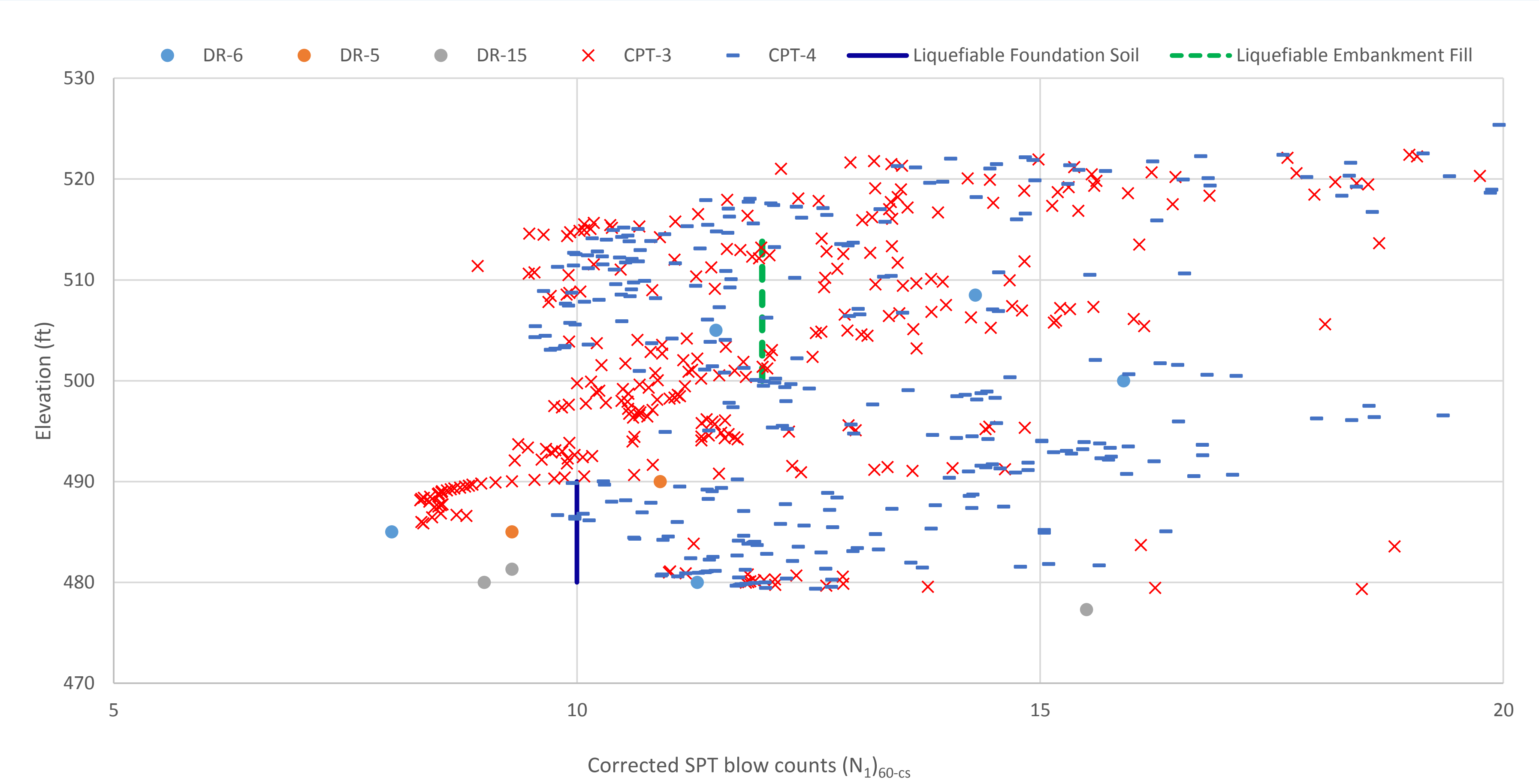
- Preliminary post-earthquake limit equilibrium slope stability analyses based on the results of the screening level liquefaction analysis suggest that the part of the pond Dam do not meet the required slope stability factors of safety. To bring more insight into the liquefaction potential of the site materials and seismic stability of the embankment, more sophisticated nonlinear dynamic analyses are performed herein under the design level earthquake shaking.

Representative Section From Slop Stability Analysis with Liquefiable Embankment Layer.



GTS NX 2017

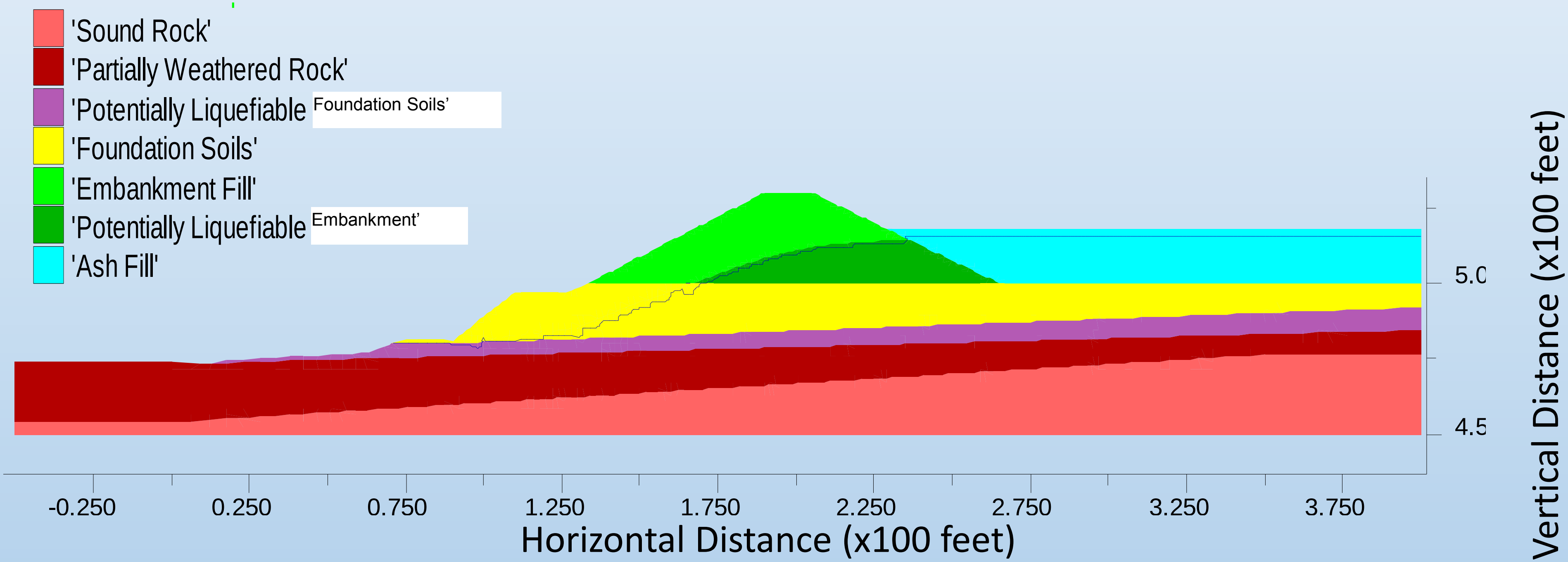
Relevant Soil Exploration Data



MIDAS

Potentially Liquefiable Soil Layers based on (N1)60:

– a liquefiable layer in the weathered rock was added and the slope of the weathered rock was revised



Steps of Coupled Effective-Stress Analysis

- Evaluation of initial static stresses
- Establishing phreatic surface using water table or through seepage analysis
- Switching nonlinear soil constitutive model to the liquefiable layers
- Obtaining earthquake input motion through site specific hazard analysis or building code
- Seismic runs and result processing

This Webinar Does not cover details of following

- Static initial stress state was established from a construction stage and phreatic surface was input as a water table based prior anlysis.
- Three Earthquake records was obtained through a site specific hazard analysis but only one is used in this demonstration

Focus on Liquefaction

- Liquefaction: Liquefaction occurs when effective stresses become or close to zero due to generation of excess pore water pressure.
- For civil or geotechnical engineers, when we talk liquefaction, we mainly are talking about saturated cohesionless soil under short term loading such as earthquake when there is no time for the excess pore pressure to dissipate.

Consequences of liquefaction

- liquefied soil softens and loss its shear strength so potential large deformation could occur.
- For embankment of impoundment, when large deformation occurs due to liquefaction, dam could fail or lose its functionality.
- For structures, the foundation bearing capacity could be reduced to an extent to cause detrimental effects to the structure such as differential/large settlements, cracking, etc.

Key for Modeling Liquefaction

- The single most important task in liquefaction modeling is to capture the excess pore pressure reasonably accurate by the chosen soil constitutive model.
- Great effort has been spent in this area for many years by academia and engineers. The available models are UBCSand, URS Model, PM4Sand (UC Davis), WangCS (Amec Foster Wheeler) among others.
- The first two are models modified from the Mohr-Coulomb model and the last two are models developed using bounding surface plasticity theory. UBCSand is the first and only one available for Midas users at this time.

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1. UBC Sand Model

- An effective stress model for predicting liquefaction behavior of sand under seismic loading.
- GTSNX Liquefaction Model is extended to a full 3D implementation of the modified UBCSAND model using **implicit method.**

Nonlinear Elastic:

- Exponential function per effective pressure

$$G^e = K_G^e p_{ref}\left(\frac{p' + p_t}{p_{ref}}\right)^{ne}$$

Plasticity / Shear

- Yield function : Mohr – Coulomb
- Flow rule : Menetrey-Willam (non-associated)
- Hardening behavior : Hyperbolic hardening

$$\Delta \sin\phi_m = \frac{G^p}{p'}\Delta\kappa_s = K_G^p\left(\frac{p'}{p_{ref}}\right)^{np-1}\left\{1-\left(\frac{\sin\phi_m}{\sin\phi_p}\right)R_f\right\}^2\Delta\kappa_s$$

$$\Delta\kappa_s = \left|\Delta\varepsilon_1^p - \Delta\varepsilon_3^p\right|$$

Plasticity / Compression (cap)

- Yield function : Modified Mohr-Coulomb Cap

$$f_2 = (p+\Delta p)^2 + \alpha\left(\frac{q}{R_2(\theta)}\right)^2 - p_c^2 = 0$$

- Flow rule : Same with yield function (Associated flow)
- Hardening behavior : Hardening of allowable compression per volumetric strain

$$\Delta p_c = K_B^p p_{ref}\left(\frac{p'}{p_{ref}}\right)^{mp}\Delta\varepsilon_v^p$$

Plasticity / Pressure cut-off

- Yield function & Flow rule

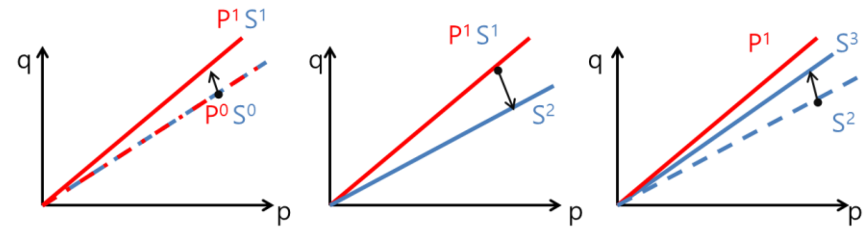
$$f_{pr} = p_{cut} - p'$$

- No Hardening behavior

Cyclic loading behavior

- Consider Shear, Plasticity function for primary and secondary yield surface respectively ➔ Check difference of hardening behavior
- Primary yield surface: In case that the current stress ratio (or mobilized friction angle) reach to the critical (MAX) state of the material
- Secondary yield surface: In case that the current stress ratio is smaller than the critical (MAX) state of the material according to the unloading/reloading conditions
- Secondary hardening (Soil Densification)

$$\Delta \sin\phi_m = K_{G,2}^p\left(\frac{p'}{p_{ref}}\right)^{np-1}\left\{1-\left(\frac{\sin\phi_m}{\sin\phi_p}\right)R_f\right\}^2\Delta\kappa_s,\quad K_{G,2}^p = K_G^p\left(4+\frac{n-1}{2}\right)F_{dens}$$



[Primary hardening] [Elastic unloading] [Secondary hardening]

UBC Sand Model Parameters

- Additional parameters to simulate liquefaction
- Estimation of each parameter using Standard Penetration Test (SPT) - **$((N_1)_{60}$: Equivalent SPT blow count for clean sand.**

Parameter	Description	Reference
Pref	Reference Pressure	In-situ horizontal stress at mid-level of soil layer
Elastic (Power Law)		
K_G^e	Elastic shear modulus number	Dimensionless
ne	Elastic shear modulus exponent	Dimensionless
Plastic / Shear		
ϕ_p	Peak Friction Angle	Failure parameter as in MC model
ϕ_{cv}	Constant Volume Friction Angle	-
C	Cohesion	Failure parameter as in MC model
K_G^p	Plastic shear modulus number	Dimensionless
np	Plastic shear modulus exponent	Dimensionless
R_f	Failure ratio (qf / qa)	0.7~0.98 (< 1), decreases with increasing relative density
F_{post}	Post Liquefaction Calibration Factor	Residual shear modulus
F_{dens}	Soil Densification Calibration Factor	Cyclic Behavior
Advanced parameters		
Pcut	Plastic/Pressure Cutoff (Tensile Strength)	-
K_B^p	Cap Bulk Modulus Number	-
mp	Plastic Cap Modulus Exponent	-
OCR	Over Consolidation Ratio	Normal stress / Pre-overburden pressure

$$K_G^e = 21.7 \times 20.0 \times (N_1)_{60}^{0.333}$$

$$30^0 < \phi_{cv} < 34^0$$

$$\nu = 0.0163$$

$$K_G^p = K_G^e (N_1)_{60}^{2} \times 0.003 + 100.0$$

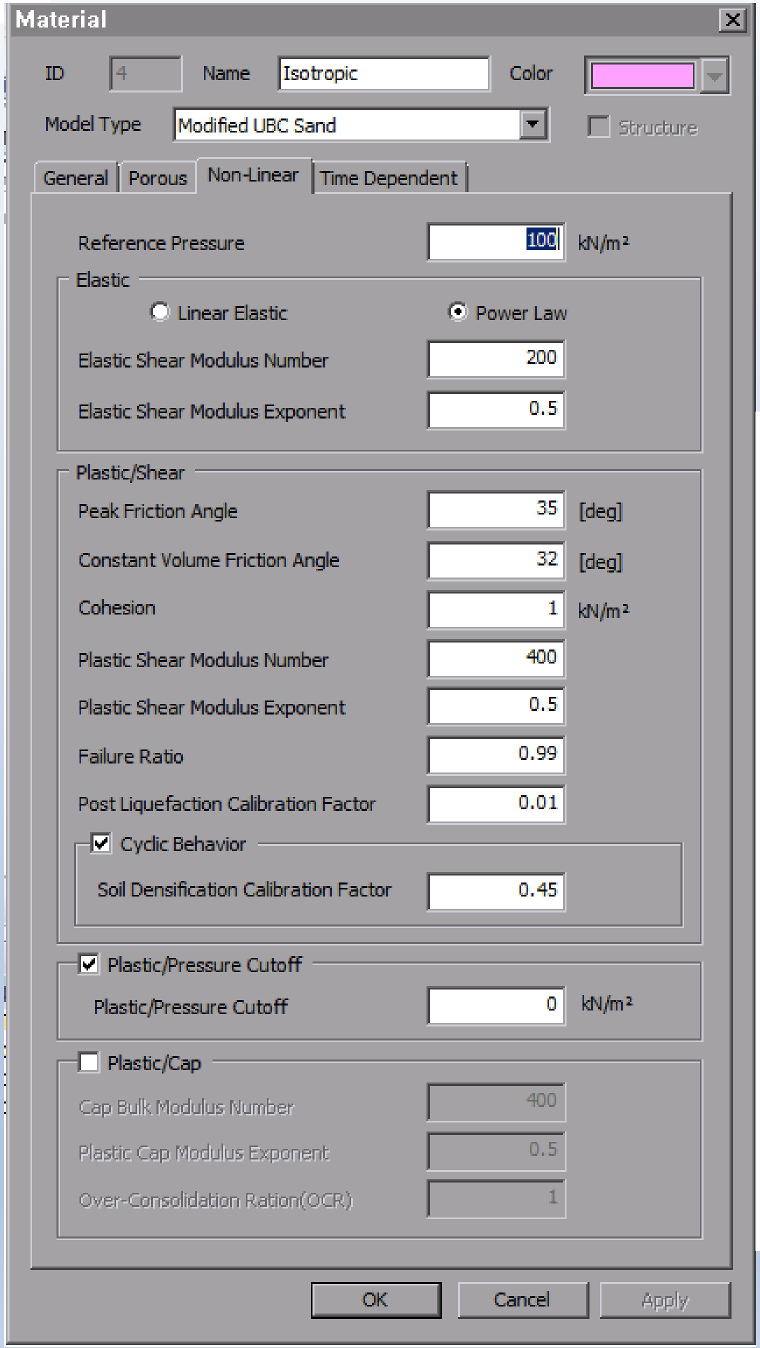
$$ne = 0.5$$

$$np = 0.4$$

$$\phi_p = \begin{cases} \phi_{cv} + (N_1)_{60} / 10.0 & ((N_1)_{60} < 15.0) \\ \phi_{cv} + (N_1)_{60} / 10.0 + \max\left(0.0, \frac{(N_1)_{60} - 15}{5}\right) & ((N_1)_{60} \geq 15.0) \end{cases}$$

$$R_f = 1.1 \times (N_1)_{60}^{-0.15}$$

[Parameters and Equations for Calibration]



Input Parameter

p_{ref}	Reference pressure
K_G^e	Elastic shear modulus number
ne	Elastic shear modulus exponent
ϕ_p	Peak friction angle
ϕ_{cv}	Constant volume friction angle
c	Cohesion
K_G^p	Plastic shear modulus number
np	Plastic shear modulus exponent
R_f	Failure ratio
F_{post}	Post liquefaction calibration factor
F_{dens}	Soil densification calibration factor
p_{cut}	Pressure cut-off
K_B^p	Plastic bulk modulus number
mp	Plastic bulk modulus exponent
OCR	Over consolidation ratio

Modified UBCSAND _ GTSNX

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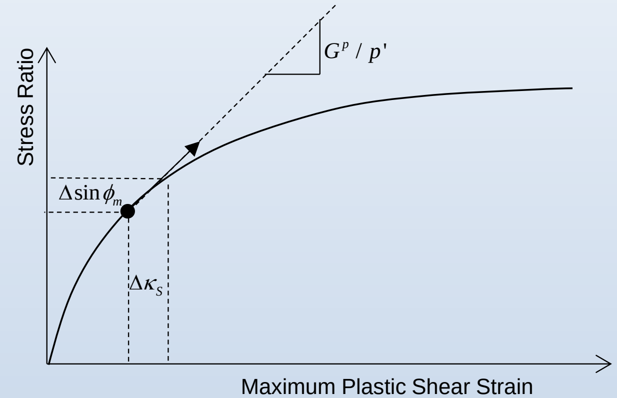
- Nonlinear Elastic
  - Exponential function per effective pressure

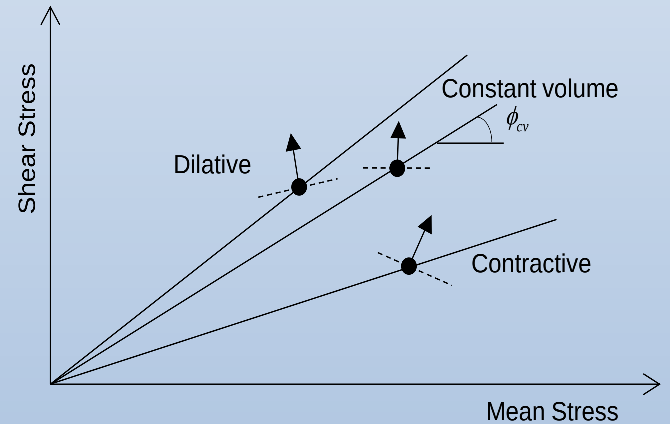
$$G^e = K_G^e p_{ref} \left(\frac{p' + p_t}{p_{ref}} \right)^{ne}$$



- Plasticity/Shear
  - Yield Function: Mohr-Coulomb
  - Flow Rule: Menetrey-Willam (non-associated)
  - Hardening behavior : Hyperbolic Hardening

$$\Delta \sin\phi_m = \frac{G^p}{p'} \Delta\kappa_s = K_G^p \left(\frac{p'}{p_{ref}} \right)^{np-1} \left\{ 1 - \left(\frac{\sin\phi_m}{\sin\phi_p} \right) R_f \right\}^2 \Delta\kappa_s$$

$$\Delta\kappa_s = \left| \Delta\varepsilon_1^p - \Delta\varepsilon_3^p \right|$$



$$\sin\psi_m = \sin\phi_m - \sin\phi_{cv}$$

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- Plasticity/Compression (cap)
  - Yield Function: Modified Mohr-Coulomb Cap

$$f_2 = (p + \Delta p)^2 + \alpha \left(\frac{q}{R_2(\theta)} \right)^2 - p_c^2 = 0$$

  - Flow Rule: Same with Yield Function (Associated flow)
  - Hardening Behavior: Hardening of allowable compression per volumetric strain

$$\Delta p_c = K_B^p p_{ref} \left(\frac{p'}{p_{ref}} \right)^{mp} \Delta \varepsilon_v^p$$

- Plasticity/Pressure cut-off
  - Yield Function & Flow Rule:

$$f_{pr} = p_{cut} - p'$$

  - No Hardening Behavior

- Cyclic loading behavior
  - Consider Shear, Plasticity function for primary and secondary yield surface respectively ➔ Check difference of hardening behavior
  - Primary yield surface: In case that the current stress ratio (or mobilized friction angle) reach to the critical (MAX) state of the material
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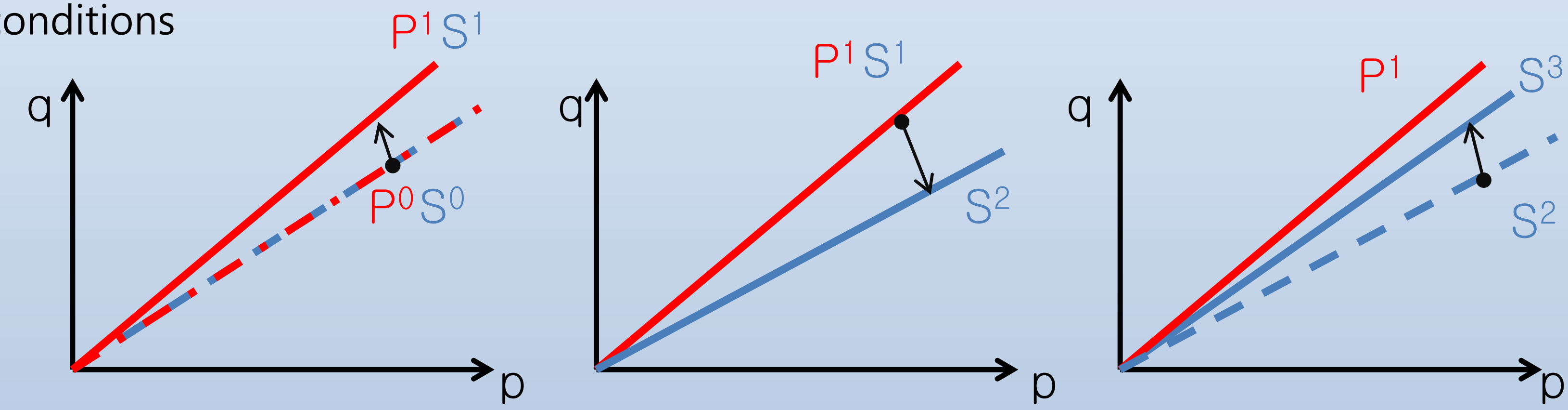


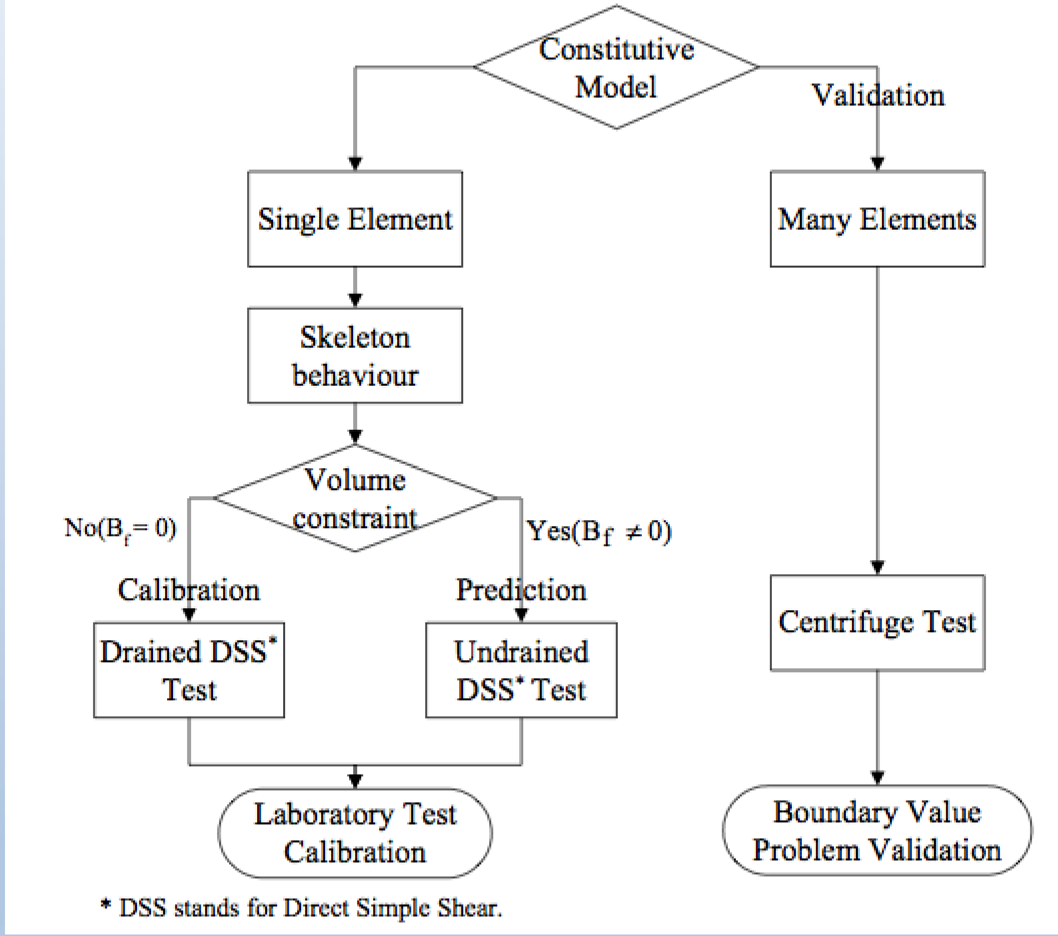
Primary hardening Elastic unloading Secondary hardening

<Secondary hardening (Soil densification)>

$$\Delta \sin\phi_m = K^p_{G,2}\left(\frac{p'}{p_{ref}}\right)^{np-1}\left\{1-\left(\frac{\sin\phi_m}{\sin\phi_p}\right)R_f\right\}^2 \Delta\kappa_s, \qquad K^p_{G,2}=K^p_G\left(4+\frac{n-1}{2}\right)F_{dens}$$

Model Calibration

- Lab test
  - Monotonic and cyclic drained Direct Simple Shear (DSS) test (skeleton response)
  - Constant volume DSS test (undrained test)
  - Single element test (3D or 2D), calibration



Model Calibration – initial estimates

- Standard Penetration Test (SPT), Calibration (Beaty, Byrne)
  - Clean sand equivalent SPT blow count measurement: $(N_1)_{60}$

$$K_G^e = 21.7 \times 20.0 \times (N_1)_{60}^{\ 0.333}$$

$$30^0 < \phi_{cv} < 34^0$$

$$\nu = 0.0163$$

$$ne = 0.5$$

$$K_G^p = K_G^e (N_1)_{60}^{\ 2} \times 0.003 + 100.0$$

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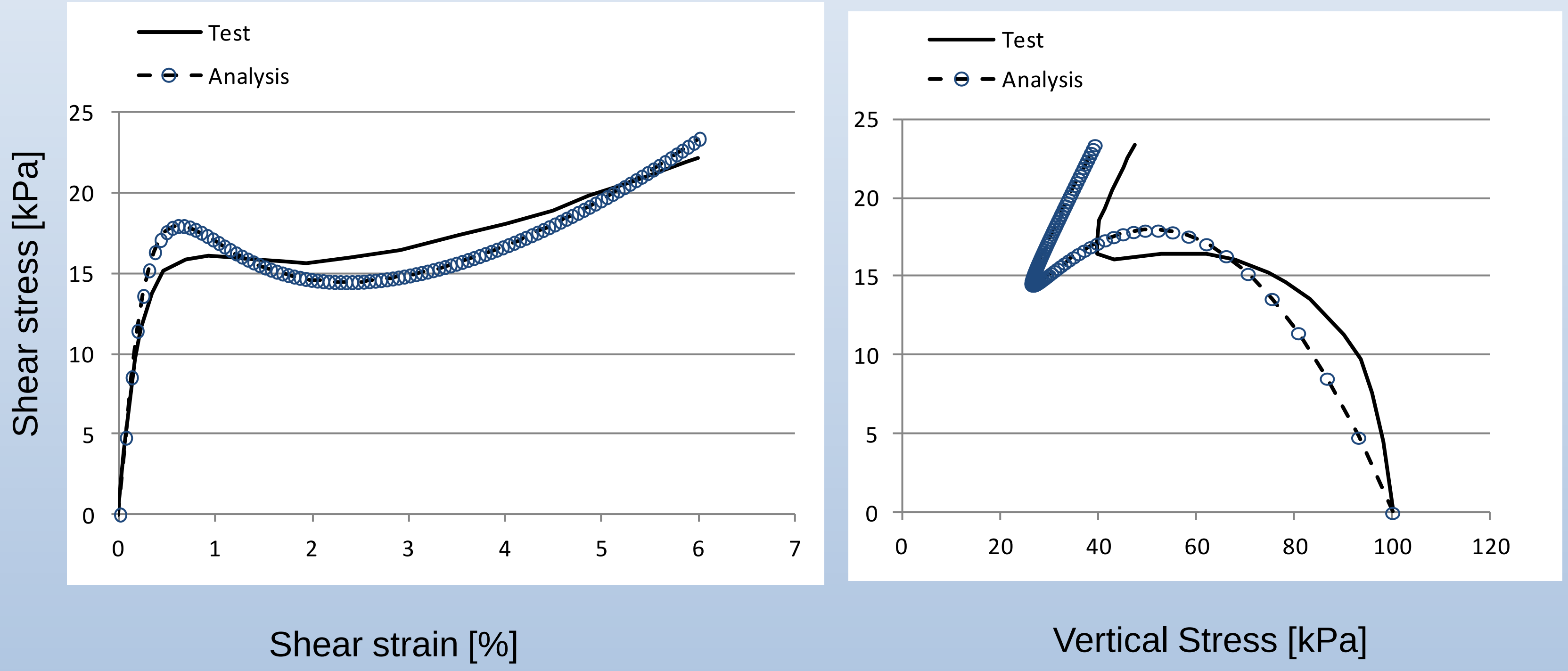
$$\phi_p = \begin{cases} \phi_{cv} + (N_1)_{60} / 10.0 & ((N_1)_{60} < 15.0) \\ \phi_{cv} + (N_1)_{60} / 10.0 + \max\left(0.0, \dfrac{(N_1)_{60} - 15}{5}\right) & ((N_1)_{60} \geq 15.0) \end{cases}$$

$$R_f = 1.1 \times (N_1)_{60}^{\ -0.15}$$

Undrained DSS (Monotonic)



Undrained DSS (Cyclic)

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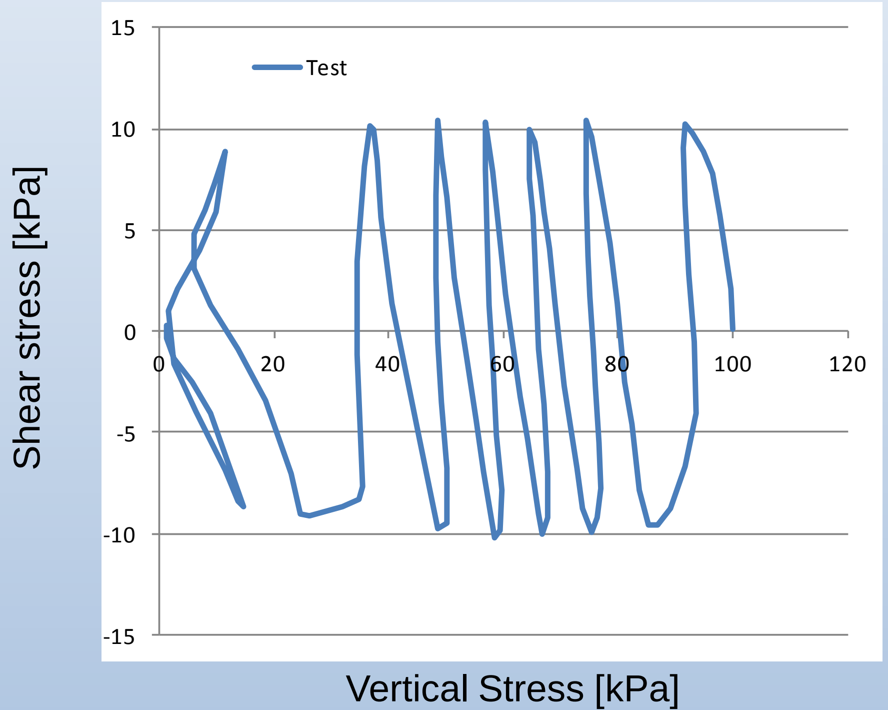

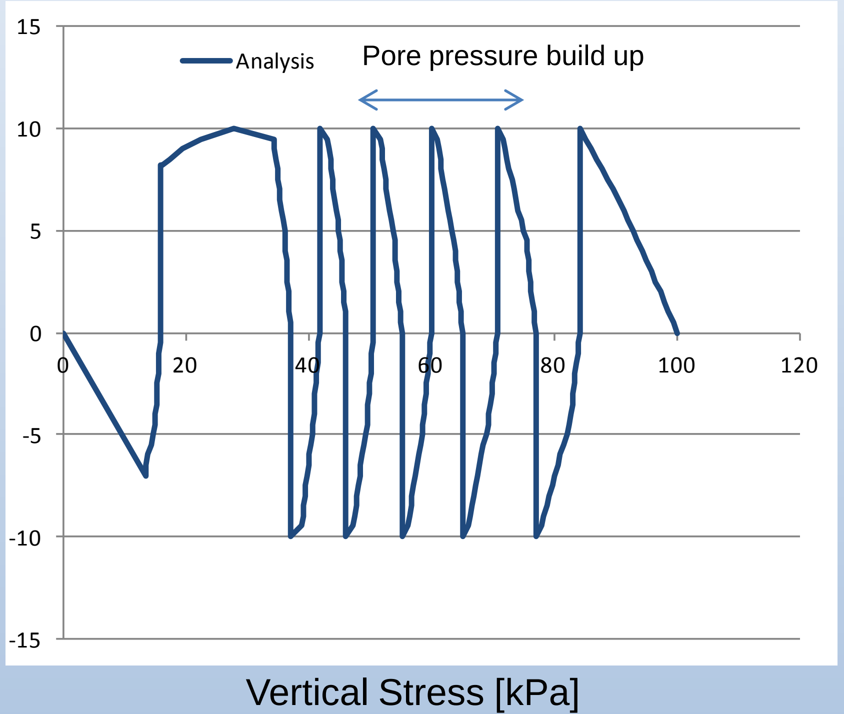

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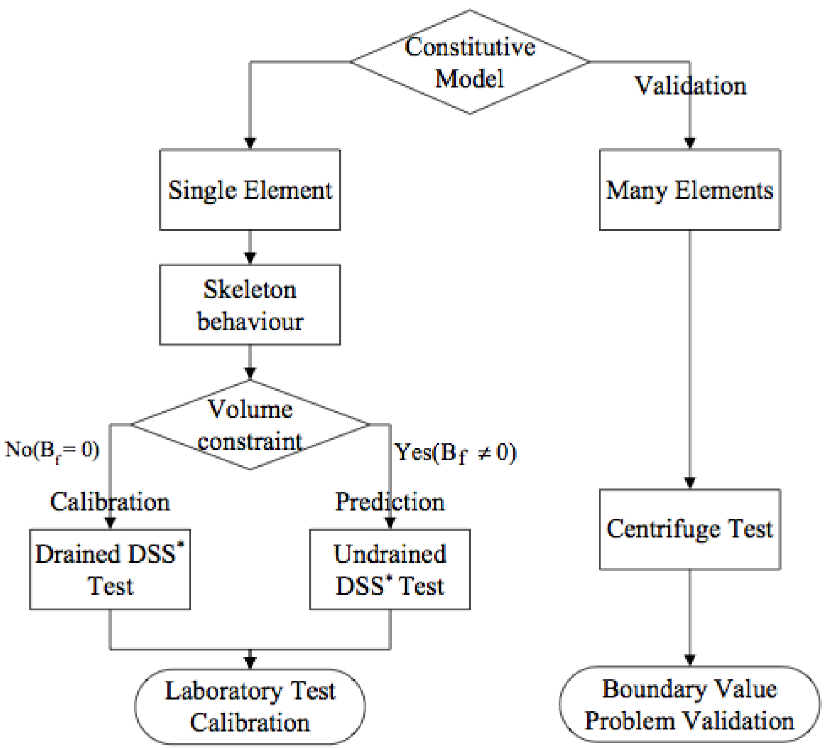
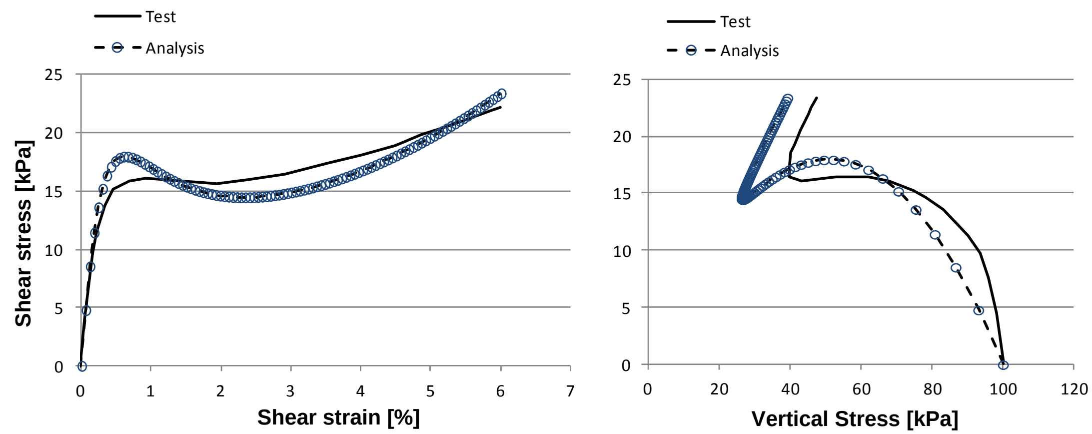
UBC SAND _ Model Calibration_Summary

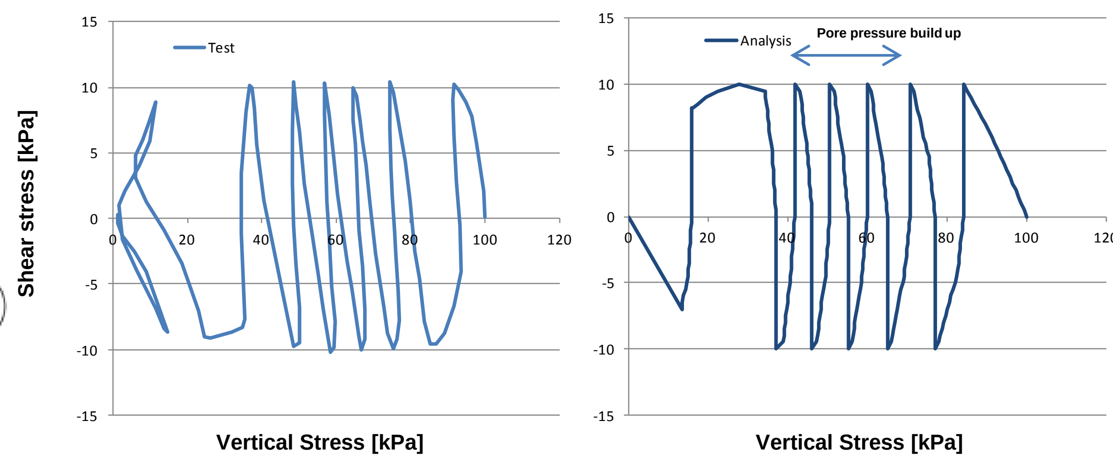
- Monotonic and cyclic drained Direct Simple Shear (DSS) test (skeleton response).
- Constant volume DSS test (undrained test)



* DSS stands for Direct Simple Shear.

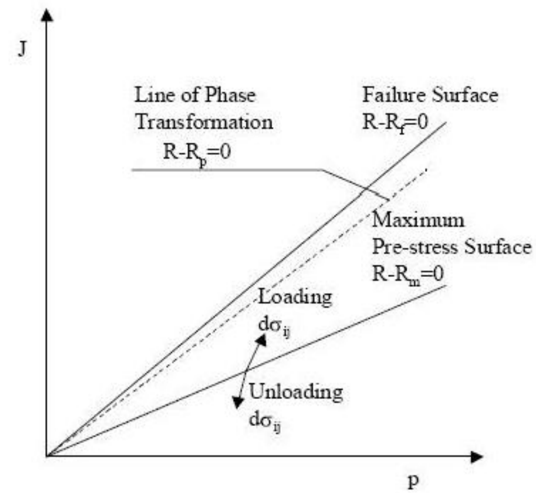


[Undrained DSS (Monotonic)]

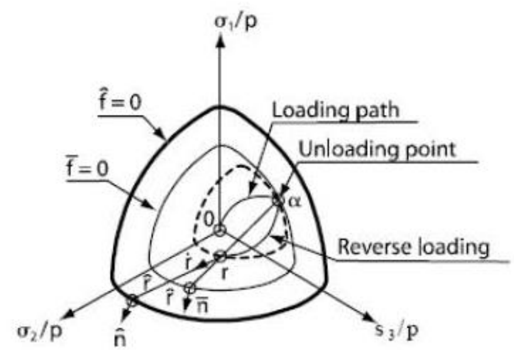


[Undrained DSS (Cyclic)]

Bounding Surface Plasticity Wang Model



(a) Surfaces and stress variables in J-p plane



(b) Surfaces and stress variables in p = constant plane

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Wang Critical State Model

Wang captures most of cohesionless soil behaviors under complex loading such as cyclic. Two simple observations (the UBCSand-Slide 14 can't capture): pore pressure build-up during unloading phase; and dilation when loading beyond the phase transformation line.

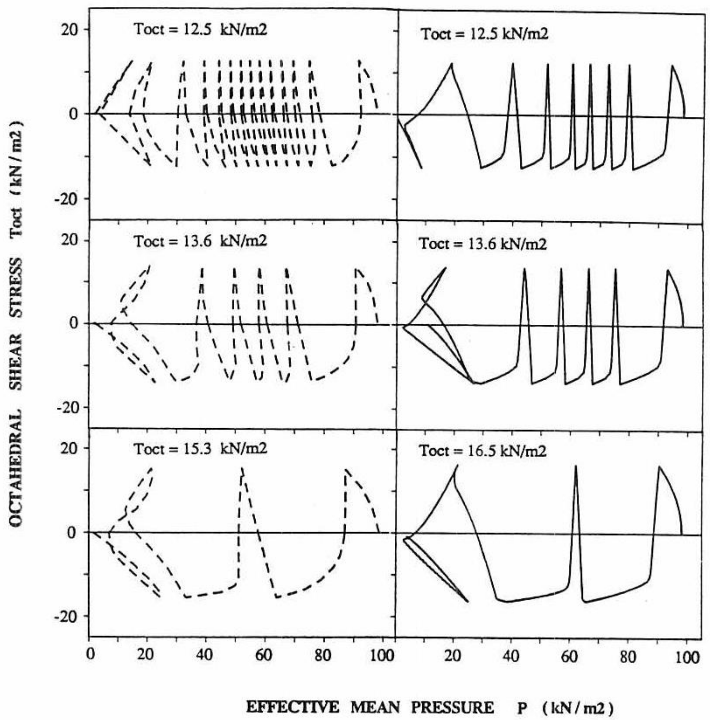

Figure 5.11 Effective stress of undrained cyclic path *ZC-ZE* ($T_{oct}=\tau_{oct}\cos\theta$)

____ Model calibration

------ Test results from Yamada et al.(1983).

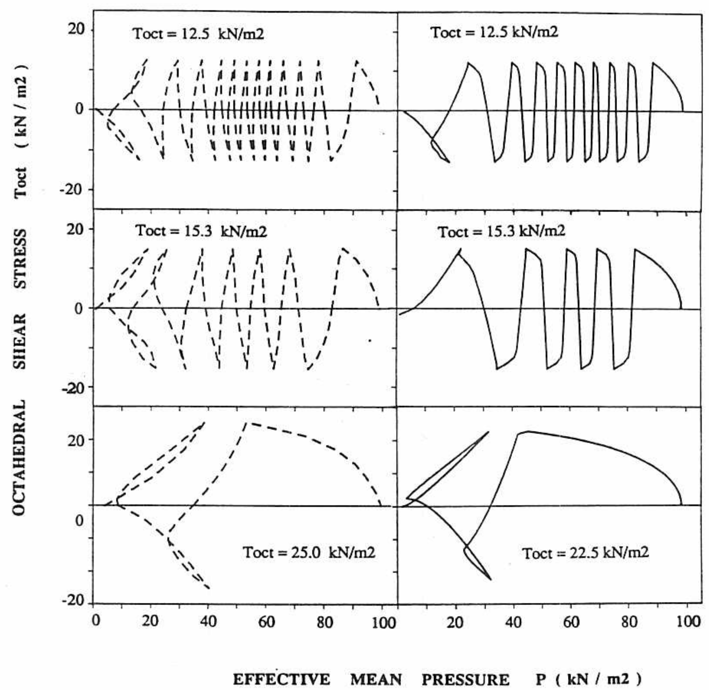

Figure 5.12 Effective stress of undrained cyclic path *90-270* ($T_{oct}=\tau_{oct}\sin\theta$)

____ Model calibration

------ Test results from Yamada et al.(1983).

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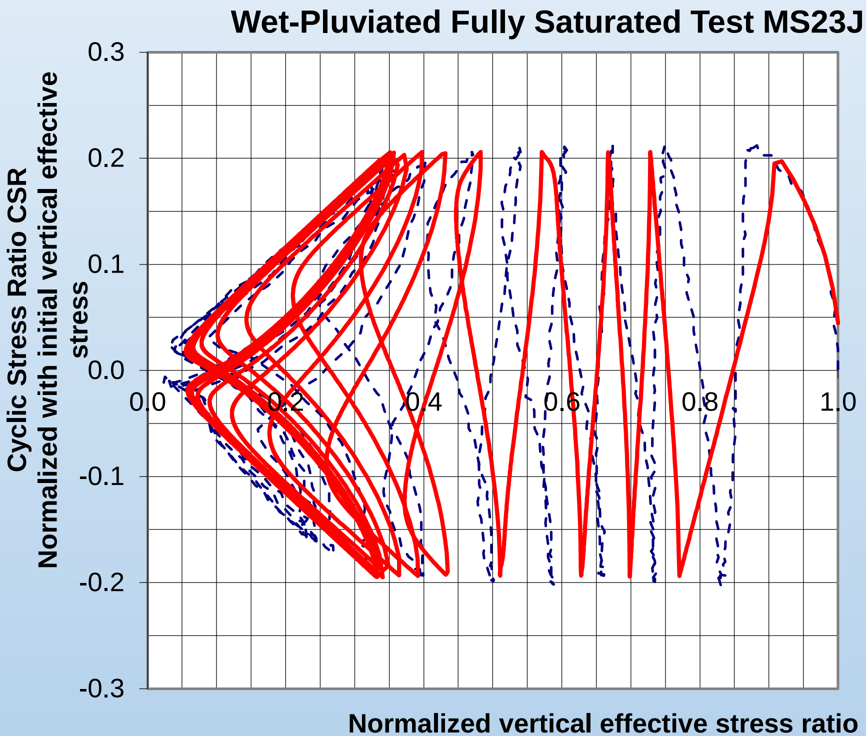
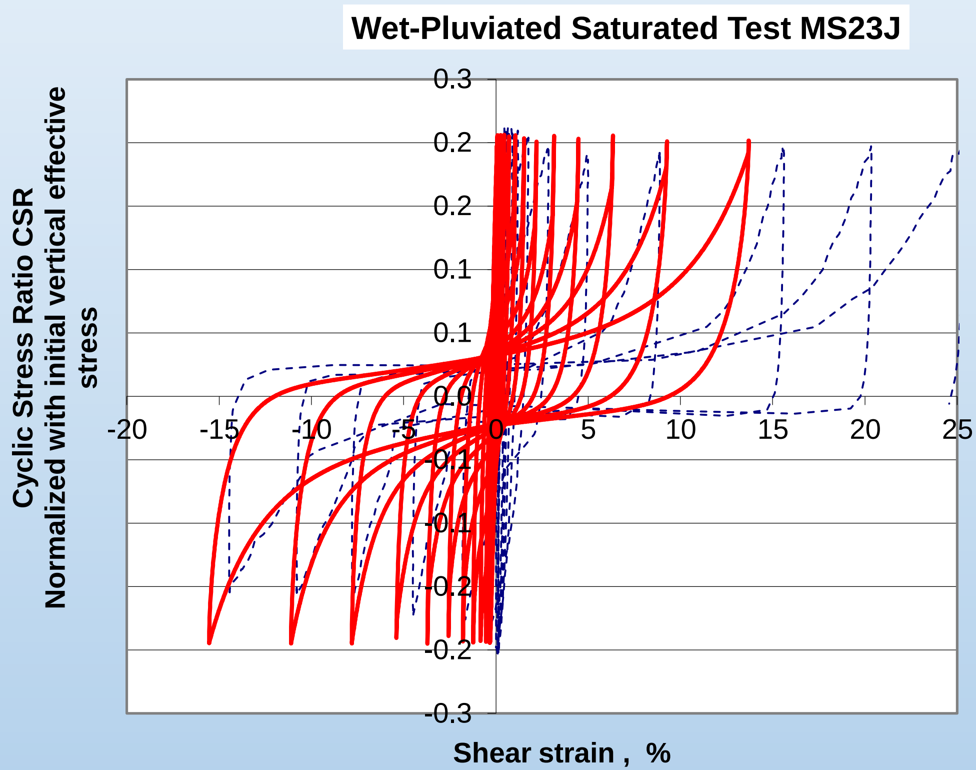
More Model comparisons with Dynamic Simple Shear (J. Wu and R. Seed, 2003)





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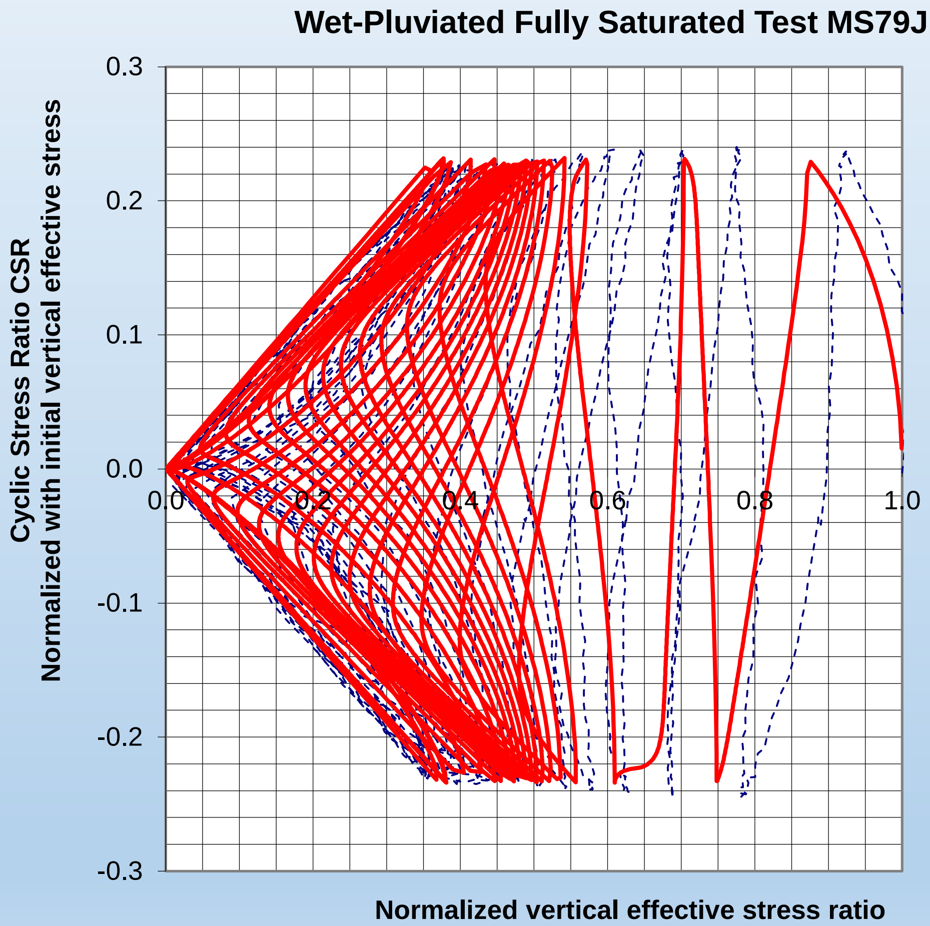
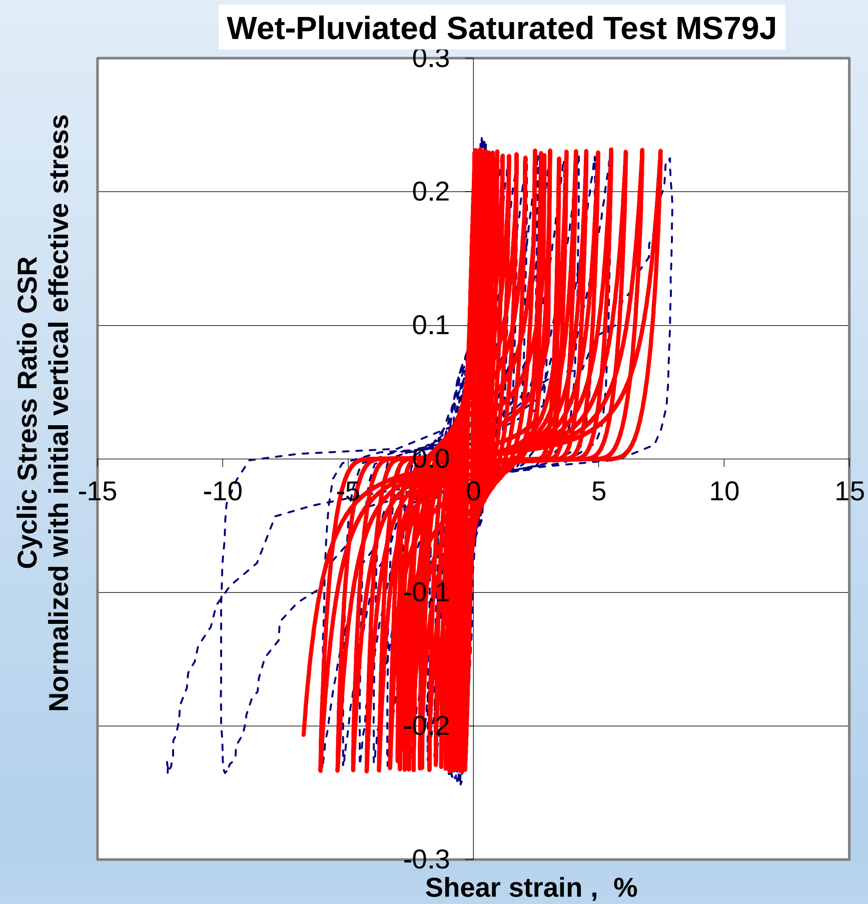


Wet-Pluviated Fully Saturated Test MS79J
Cyclic Stress Ratio CSR
Normalized with initial vertical effective stress
0.3
0.2
0.1
0.0
-0.1
-0.2
-0.3
0.0
0.2
0.4
0.6
0.8
1.0
Normalized vertical effective stress ratio



Wet-Pluviated Saturated Test MS79J
Cyclic Stress Ratio CSR
Normalized with initial vertical effective stress
0.3
0.2
0.1
0.0
-0.1
-0.2
-0.3
-15
-10
-5
0
5
10
15
Shear strain , %

Wang Model Parameters

Table 1 Model Parameters of Sand (Dr=45% or 60%)

Type	Dr (%)	ϕ' degree	Go	hr	kr	d	Poisson ratio	gamma	ita	Rp/Rf	Void ratio e[1]
1	45	28	181	0.15	0.2	4	0.33	15	7	0.50	0.69
2	60	28	242	0.15	0.2	6	0.33	15	20	0.50	0.73

Notes:

(1) Void ratio e=0.541 + Dr*0.314 where assuming emax=0.855 and emin=0.541

Continue – Wang model parameters

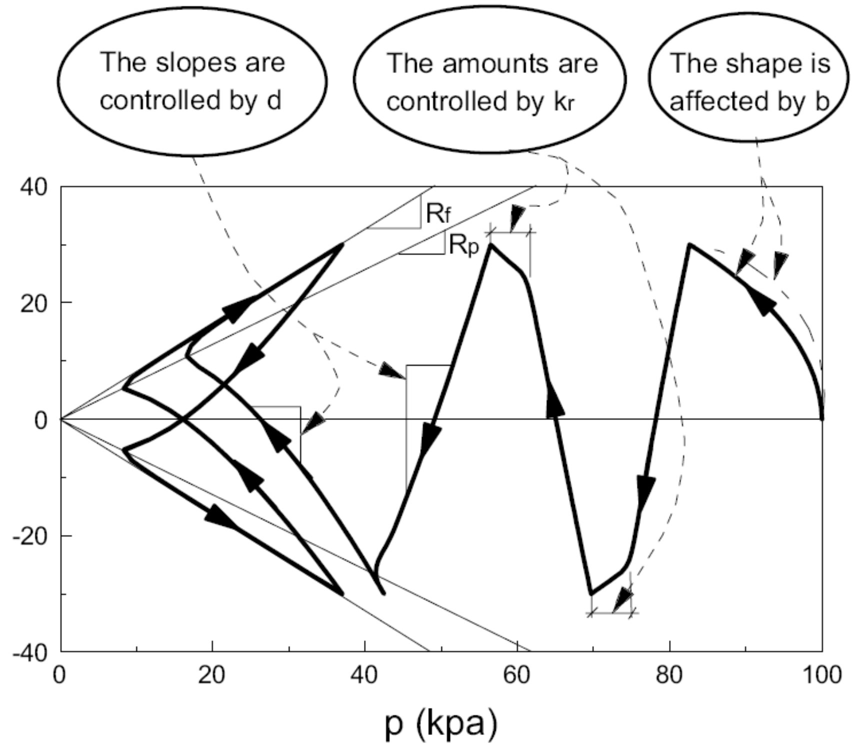
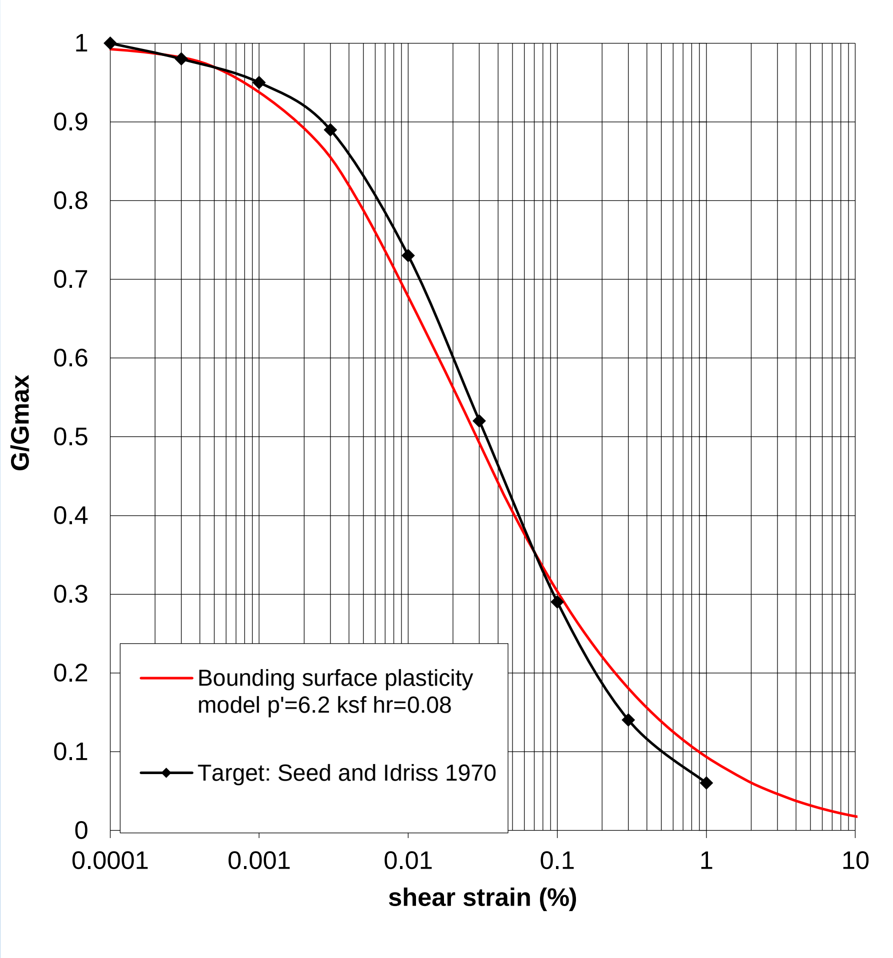
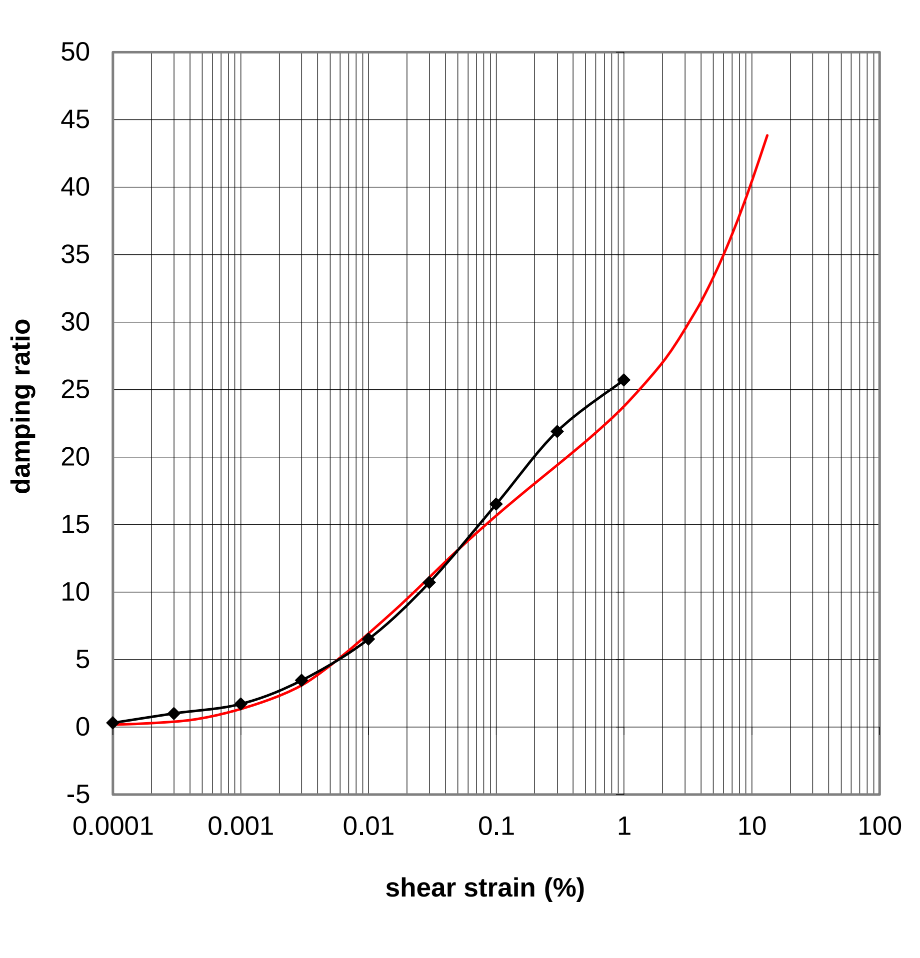


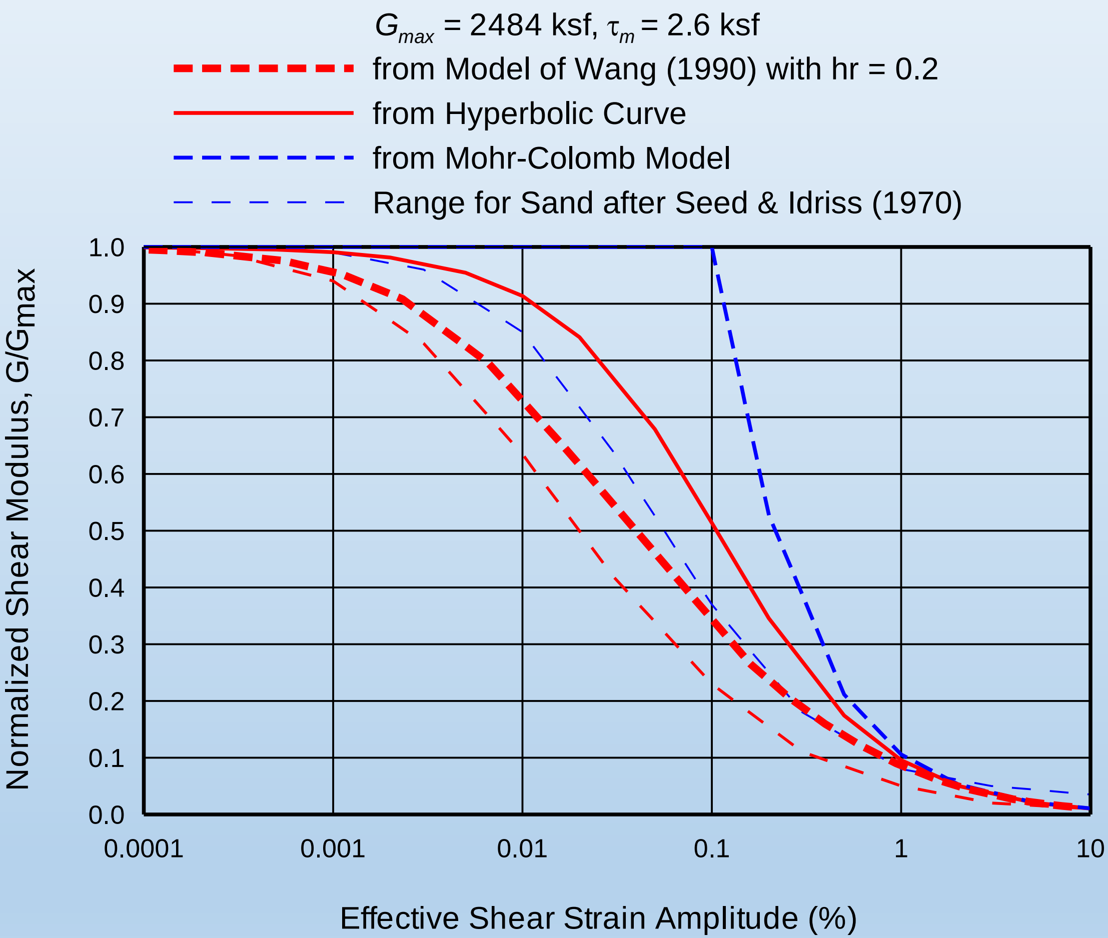
Fig. A2.4 Influences of d, k_r, b, and R_p

Continue - Model Parameters hr calibrated from G/Gmax and damping curves





Continue – Model Parameter Go that defined the Initial (maximum) Elastic Modulus



$$d\gamma = d\gamma^e + d\gamma^p$$

$$d\gamma^e = \frac{1}{G} d\tau$$

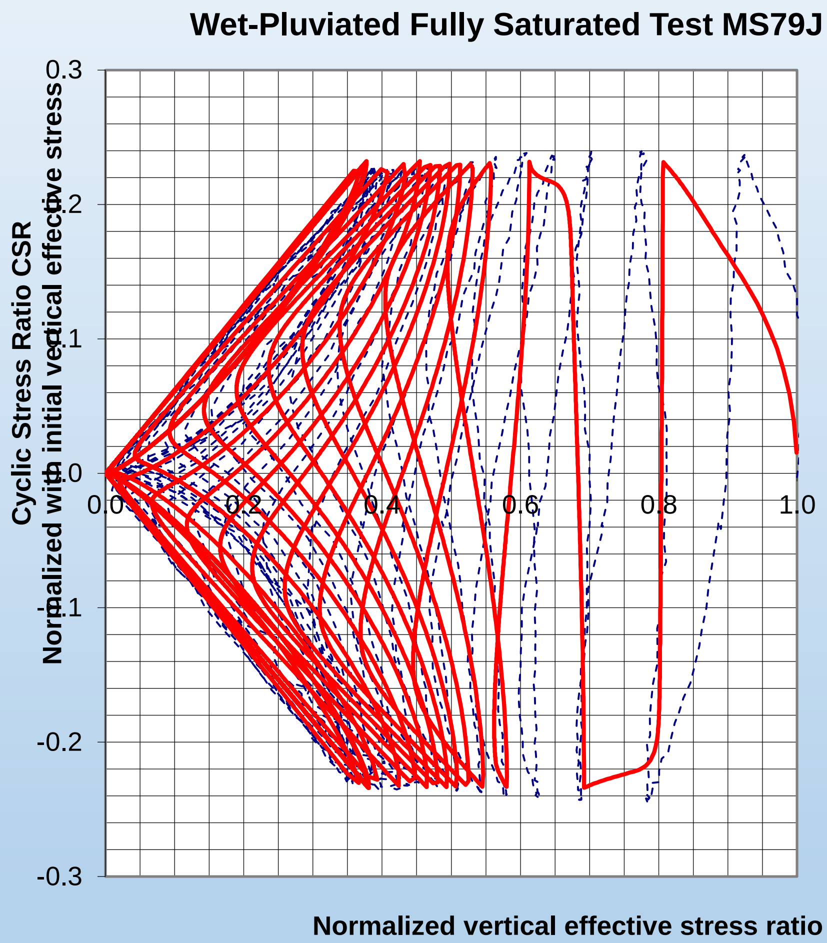
$$d\gamma^p = \frac{1}{H} d\tau$$

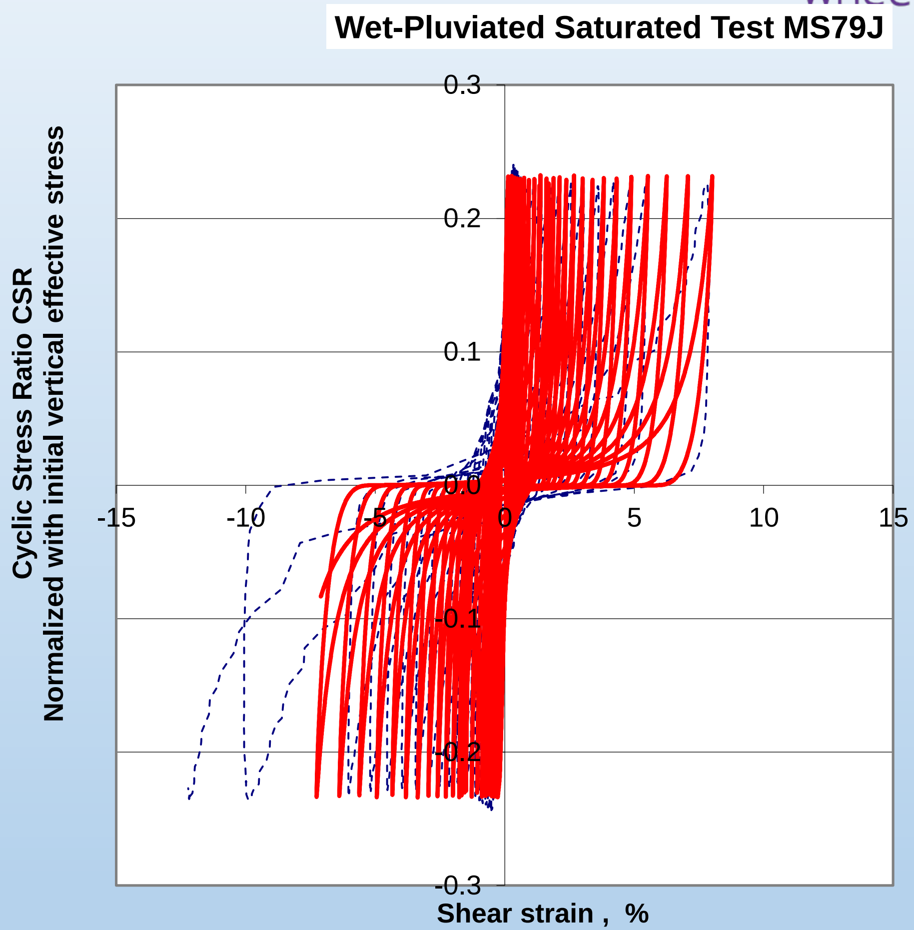
$$H = h_r G\left(\frac{\tau_f - \tau}{\tau}\right)$$

$$G = G_e(e) p_a \sqrt{p / p_a}$$

$$d\gamma = \frac{1}{G}\left(1 + \frac{\tau}{h_r(\tau_f - \tau)}\right) d\tau$$

Continue – last 2 Model Parameters gamma and ita, which control the post-liquefaction strain accumulations, i.e. stress path stabilized (left figure) but strain continue accumulating (right figure)





Pre-Existing Shear Stress, k_α effect

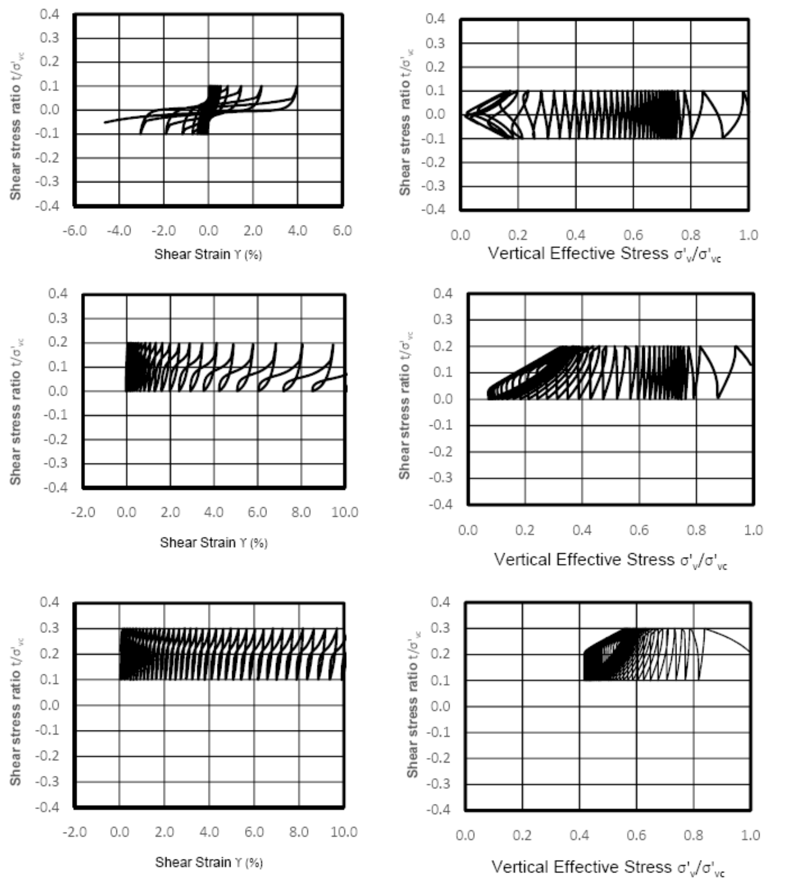


Figure 6.1. Undrained cyclic DSS loading response for d=4 (loose sand) with vertical effective consolidation stress of 1 atm and with initial static shear stress ratios of 0.0, 0.1, and 0.2.

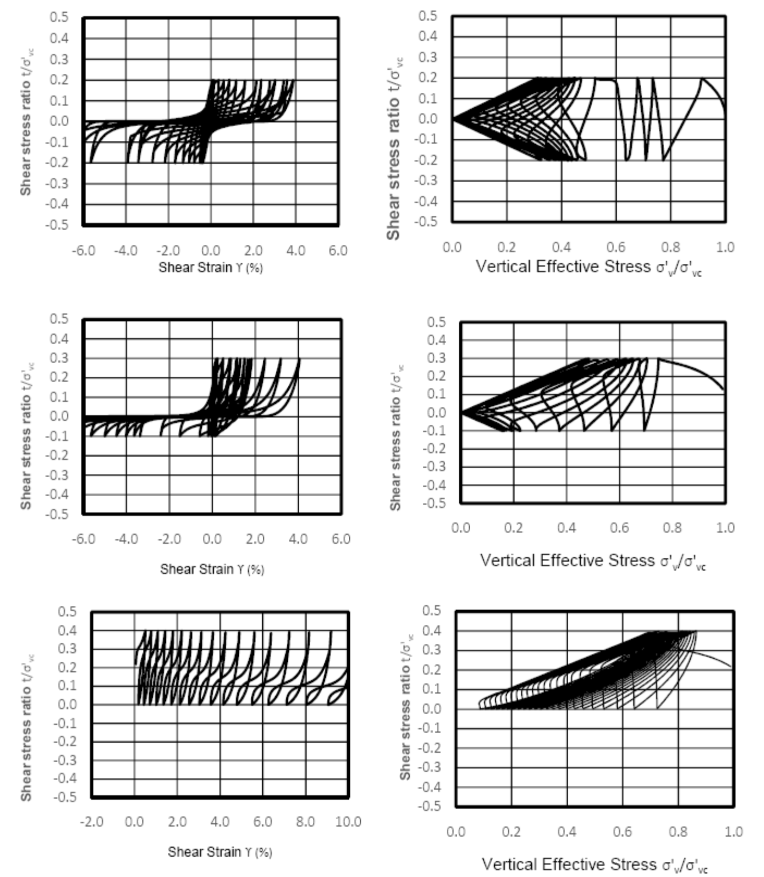


Figure 6.2. Undrained cyclic DSS loading response for d=6 (median dense sand) with vertical effective consolidation stress of 1 atm and with initial static shear stress ratios of 0.0, 0.1, and 0.2.

UBC Sand Post Processing – Liquefiable Area or Pore Pressure Ratio or Ru

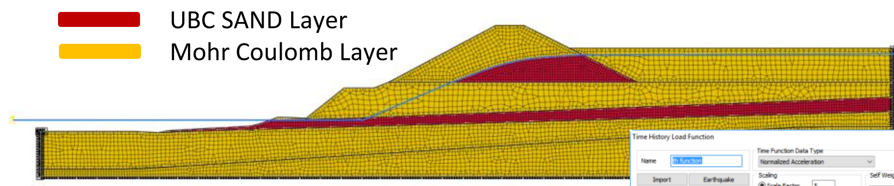
- Specific results which can check the liquefiable area directly
- Two types of results are available to measure the possibility of liquefaction.

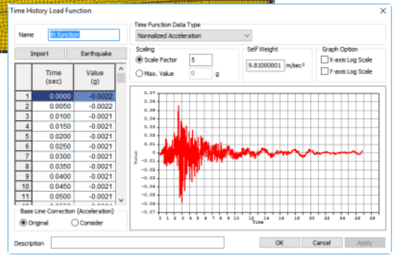
Pore Pressure Ratio (PPR)

- The ratio of excessive pore pressure change and the initial effective pressure

$$PPR = -\frac{\Delta p_w}{p'_{init}} = \frac{p'_{init} - p'_{current}}{p'_{init}}$$

Δp_w	Excessive Pore Pressure Change
p'_{init}	Initial Effective Pressure
$p'_{current}$	Current Effective Pressure



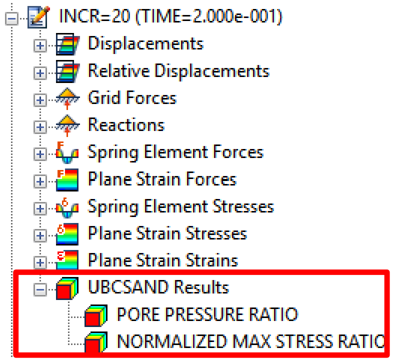


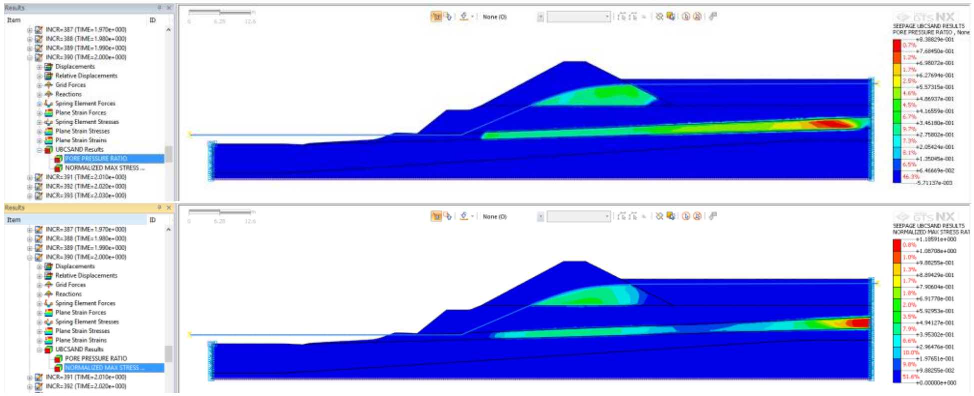
Normalized Max Stress Ratio

- The ratio of mobilized friction angle and the peak friction angle
- When the Max stress ratio is reached, the mobilized friction angle is close to the peak friction angle, liquefaction is triggered (1 = Liquefaction)

$$\max\left(\frac{\sin\phi_m}{\sin\phi_p}\right)$$

ϕ_m	Mobilized Friction Angle
ϕ_p	Peak Friction Angle





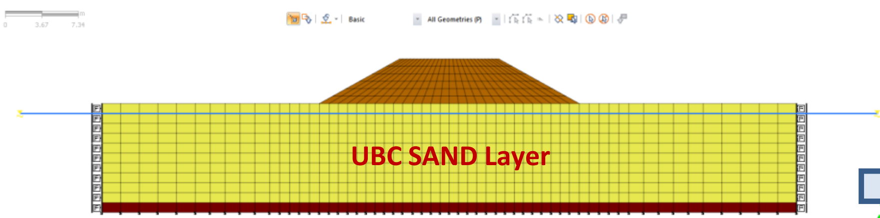
[Nonlinear Time History Analysis under the earthquake]

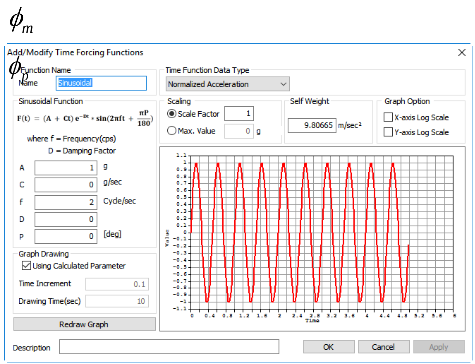
UBC Sand Normalized Max Stress Ratio

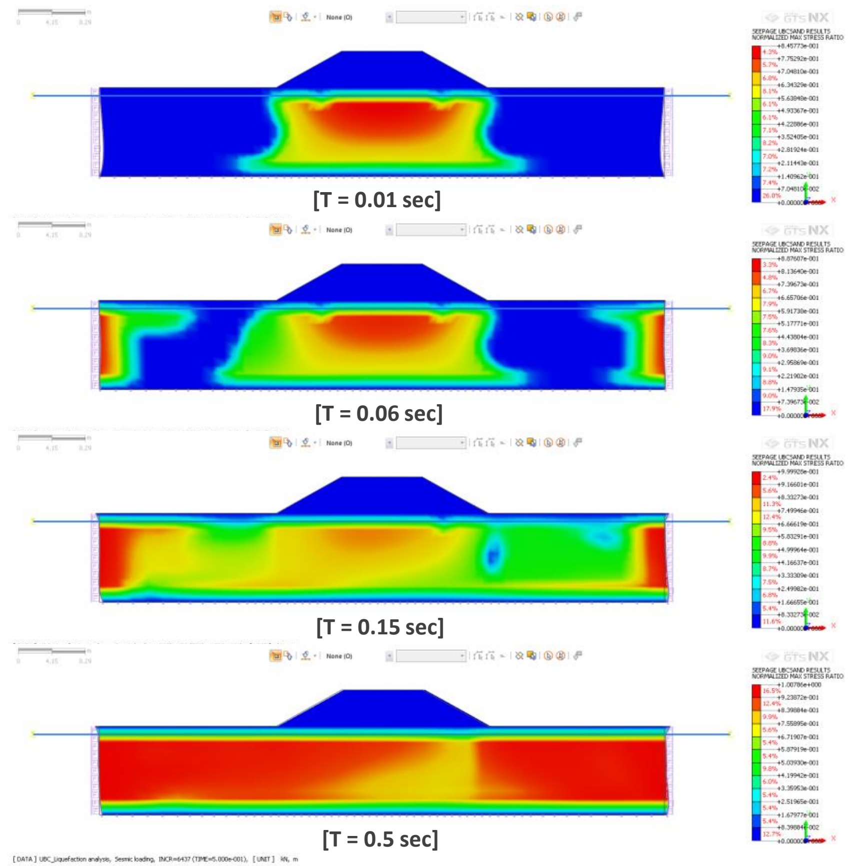
- ***Normalized Max Stress Ratio***
- When the Max possible stress ratio is reached, liquefaction is triggered and K_G^p is reduced as

$$K_G{}^P = K_G{}^P{}_0 * fac_{pos}$$

- , where fac_{pos} is a user defined Post Liquefaction Calibration Factor







[Nonlinear Time History Analysis under the cyclic loading]

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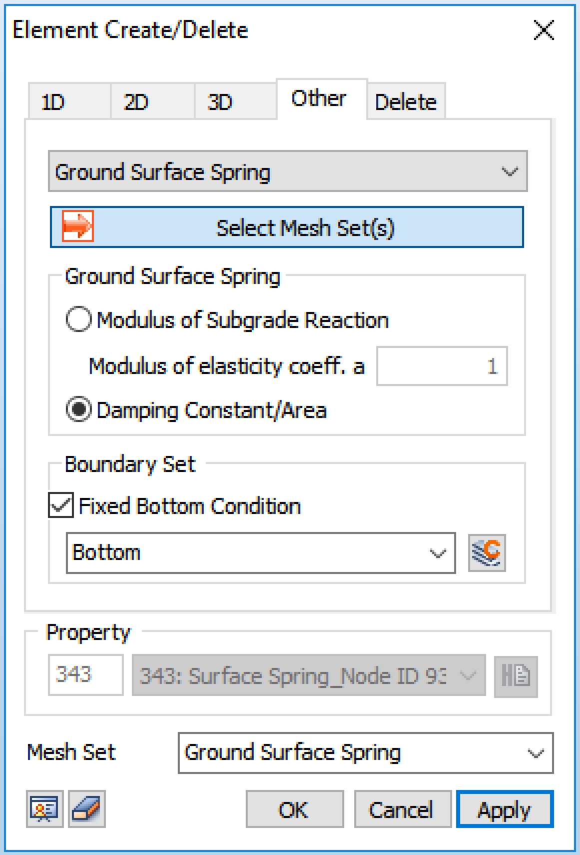
Analysis options

(Large deformation, Excessive Pore Water Pressure, Post-Processing)

1. Boundary Conditions for the damping effect

1) Damper
2) Free Field (Infinite boundary, Absorbent boundary)

1) Damper (Mesh > Element > Create > Other > Ground Surface Spring)



- The viscous boundary element required as a model boundary condition for time history analysis.
- The viscous boundary element can be created from the following steps.

1. Compute Cp, Cs : Cp, Cs can be calculated using the equation below.

$$C_p = \rho \cdot A \cdot \sqrt{\frac{\lambda + 2G}{\rho}} = W \cdot A \cdot \sqrt{\frac{\lambda + 2G}{W \cdot 9.81}} = c_p \cdot A$$

$$C_s = \rho \cdot A \cdot \sqrt{\frac{G}{\rho}} = W \cdot A \cdot \sqrt{\frac{G}{W \cdot 9.81}} = c_s \cdot A$$

$$\lambda = \frac{\nu \cdot E}{(1+\nu)(1-2\nu)} \qquad G = \frac{E}{2(1+\nu)}$$

λ : Bulk modulus, G : Shear modulus, E : Elastic modulus, ν : Poisson's ratio, A : Cross-section area

2. The cross-section area is automatically considered until the surface spring is created, so only the Cp, Cs needs to be computed. When creating the viscous boundary element automatically, the spring is automatically created by considering the element area (effective length*unit width) as shown below. Input the Cp value for the normal direction coefficient at the point of spring creation and input the Cs value for the parallel direction.

1. Boundary Conditions for the damping effect

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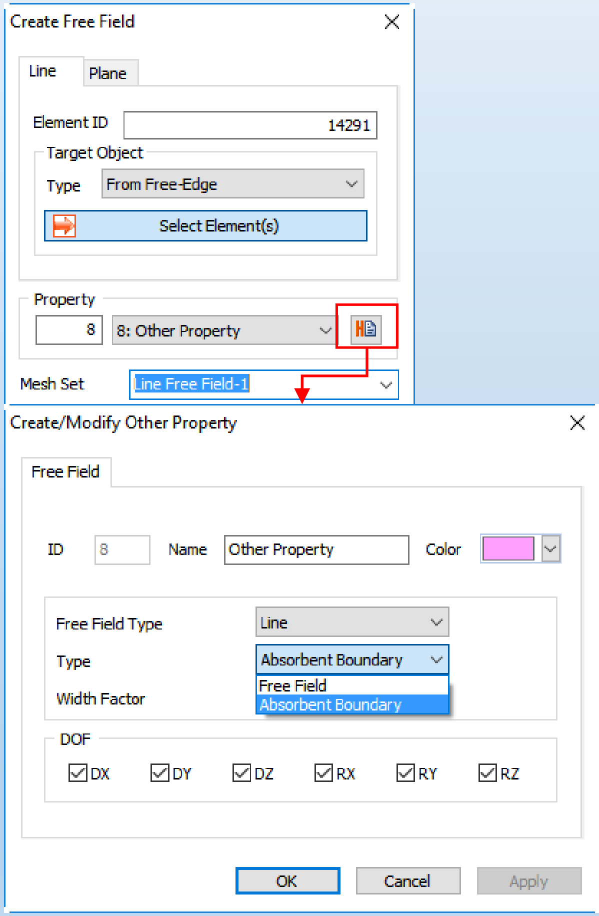
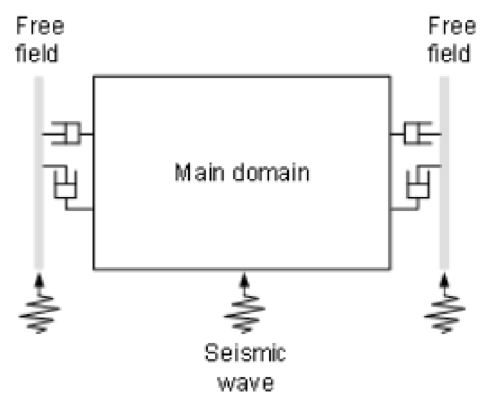
1) Damper
2) Free Field (Infinite boundary, Absorbent boundary)

2) Free Field (Mesh > Element > Free Field)

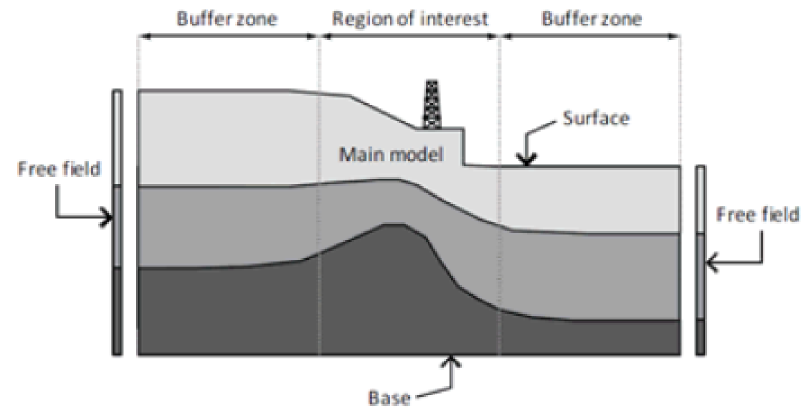


For the seismic analysis, users need to model infinite ground to eliminate the boundary effect caused by reflection wave. Since it is not possible to model infinite ground, users can apply Free Field Element at the boundary.

Absorbent Boundary : Enable to eliminate reflection wave at the ground boundary
Width Factor (Penalty Parameter) : In order to minimize the size effect of the model, users have to input more than 10^4, This value will be multiplied by model width (In case of 2D, this is plain strain thickness (unit width).



[Schematic Overview of Free Field Element]



[Free field effect (O), Absorb reflection (O)]

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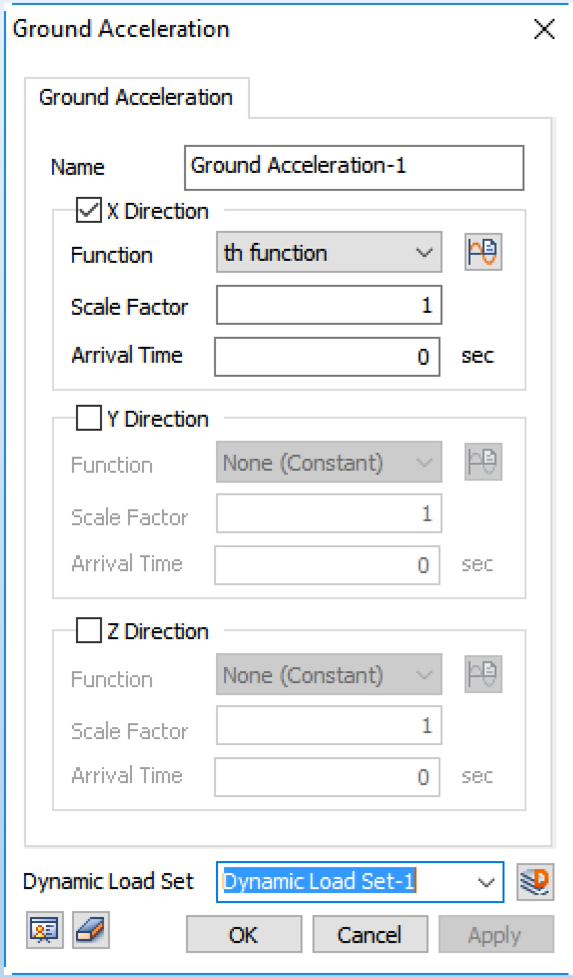
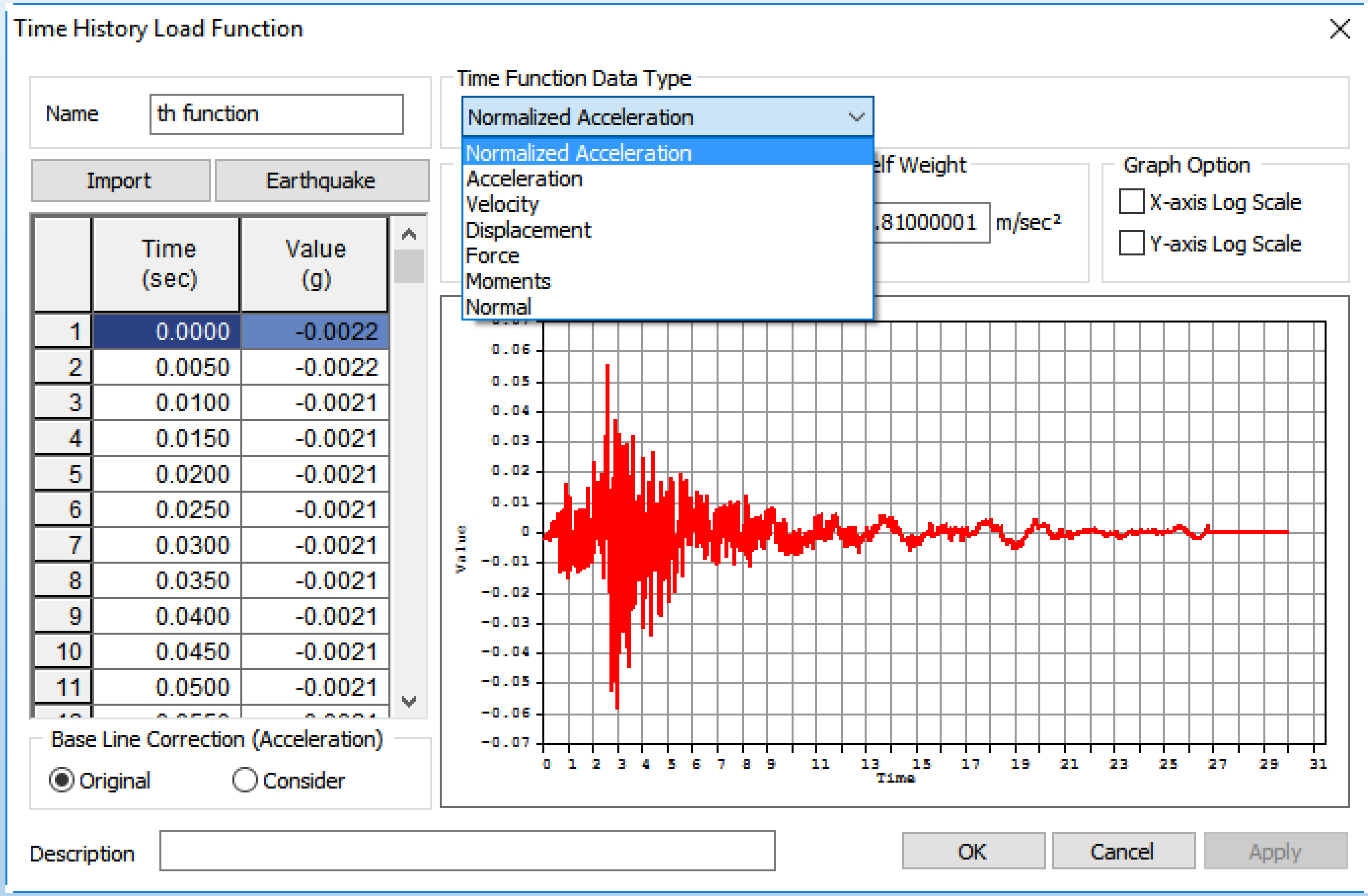
1) Ground Acceleration (Dynamic Analysis > Load > Ground Acceleration)





MIDAS

2. Dynamic Load

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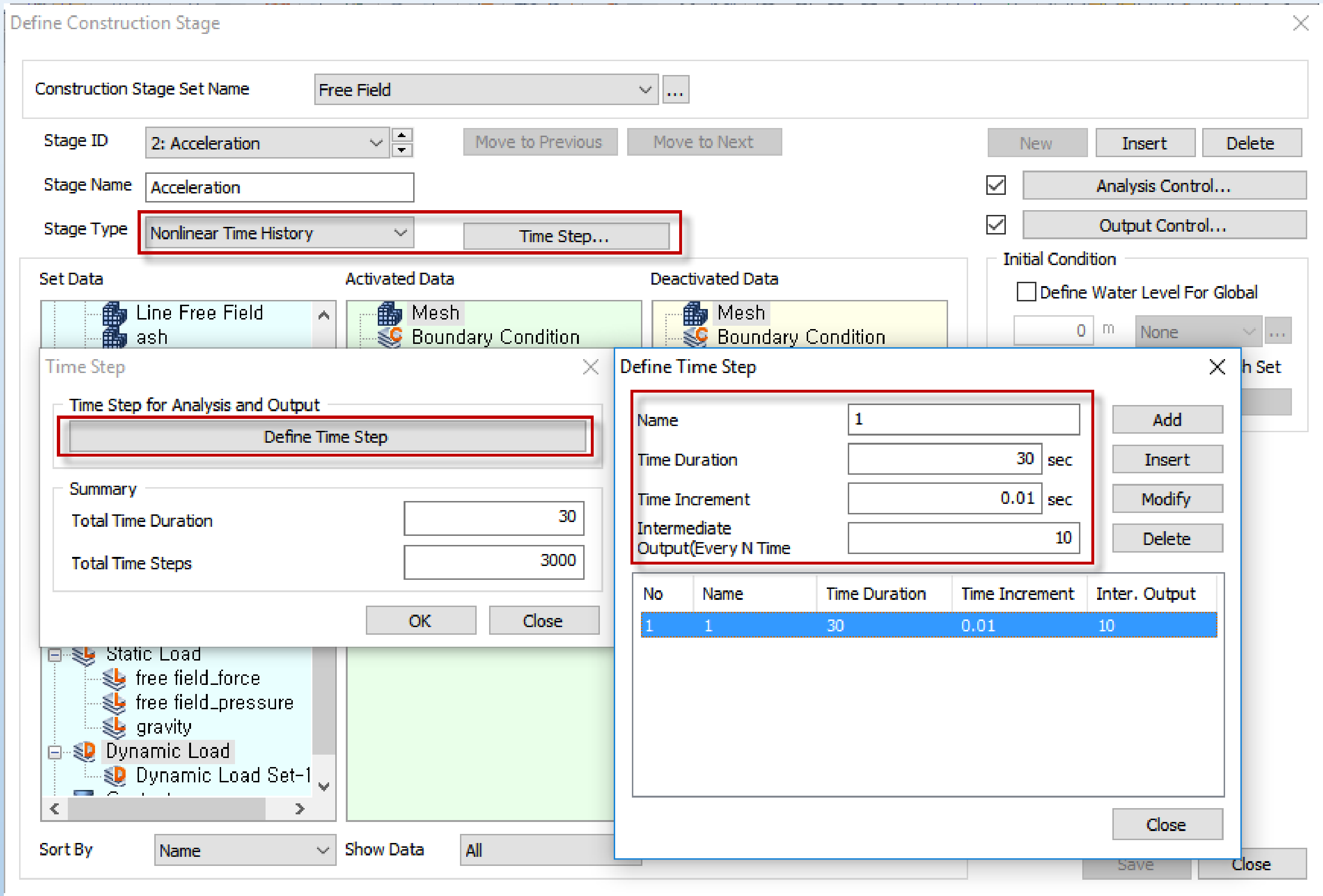
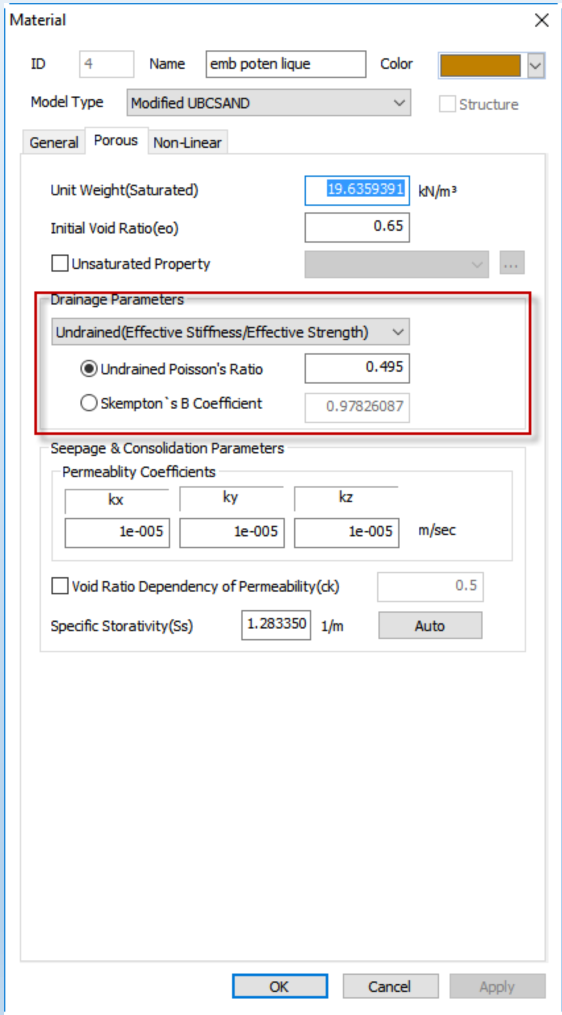
2) Analysis Time : Define Time Step for the analysis and the output

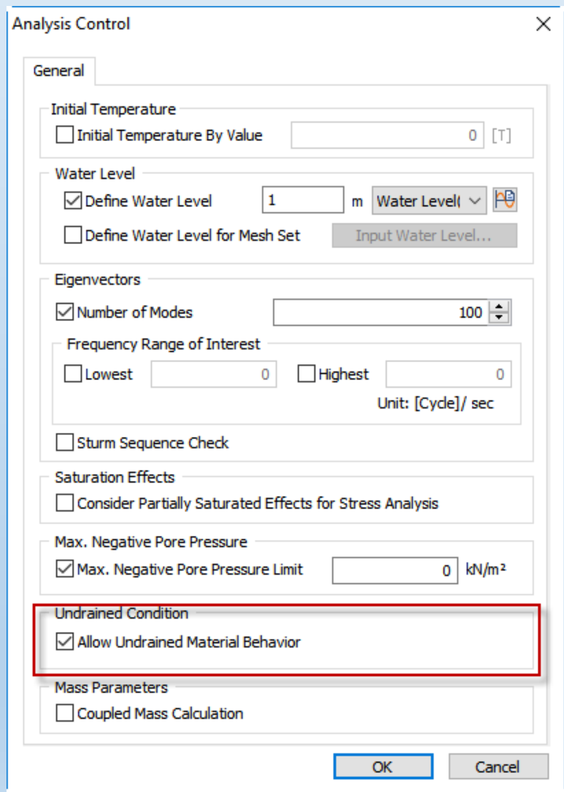


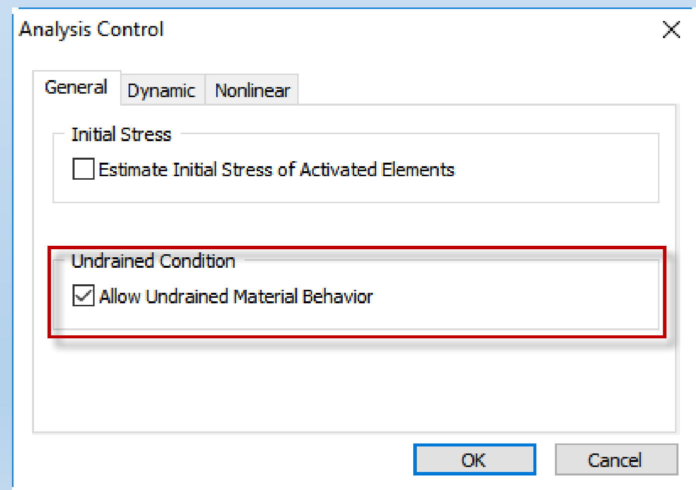
3. Analysis Options

1) Undrained Condition : Allow Undrained Material Behavior to check the generated excessive pore water pressure under short term loading like earthquake



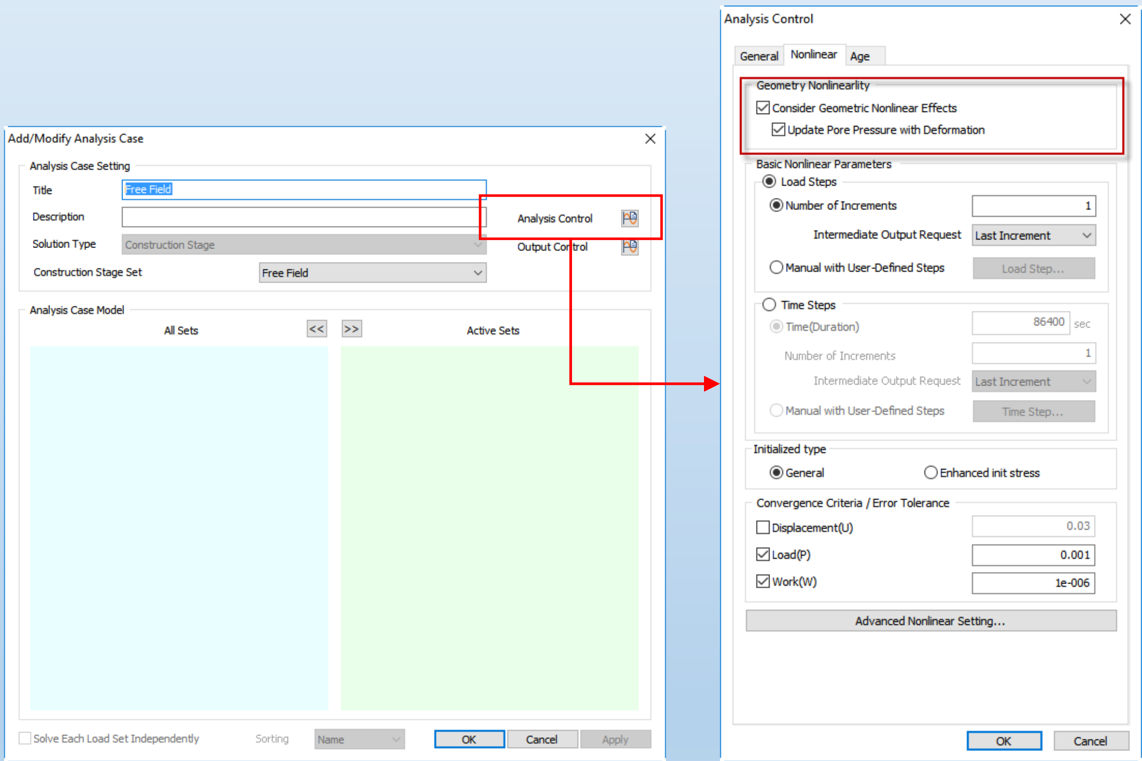
- Material > Porous > Drainage Parameters
- Analysis Control > Undrained Condition (for each analysis case or for each stage)

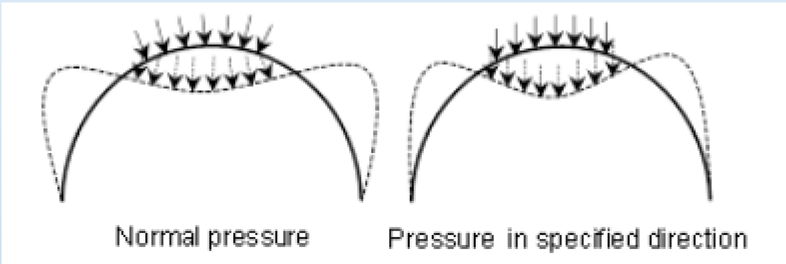




2) Geometry Nonlinearlity

- Consider Geometric Nonlinear Effects to simulate large deformation
(Analysis Case > Analysis Control > Nonlinear)
- Analysis can take into account load nonlinearity which is reflecting the effects of follower loads, where the load direction changes with the deformation. Depending on the deformed shape, the pore water pressure can be updated automatically.

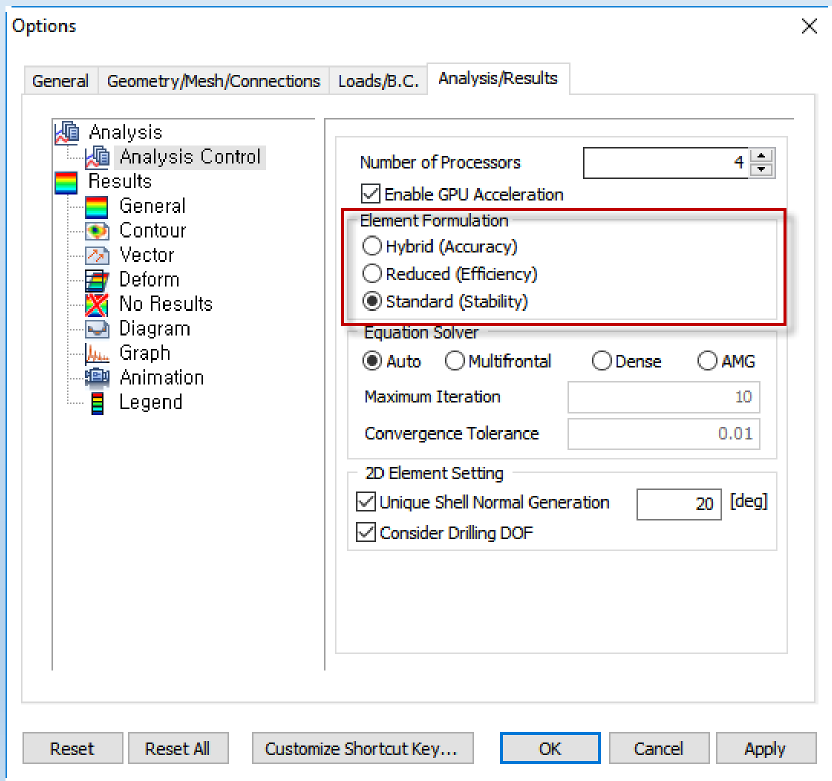




3. Analysis Options

3) Element Formulation (Analysis > Tools > Options > Analysis/Results > Element Formulation)

- Hybrid (Default setting)
- Standard (for large deformation, geometric nonlinear option)



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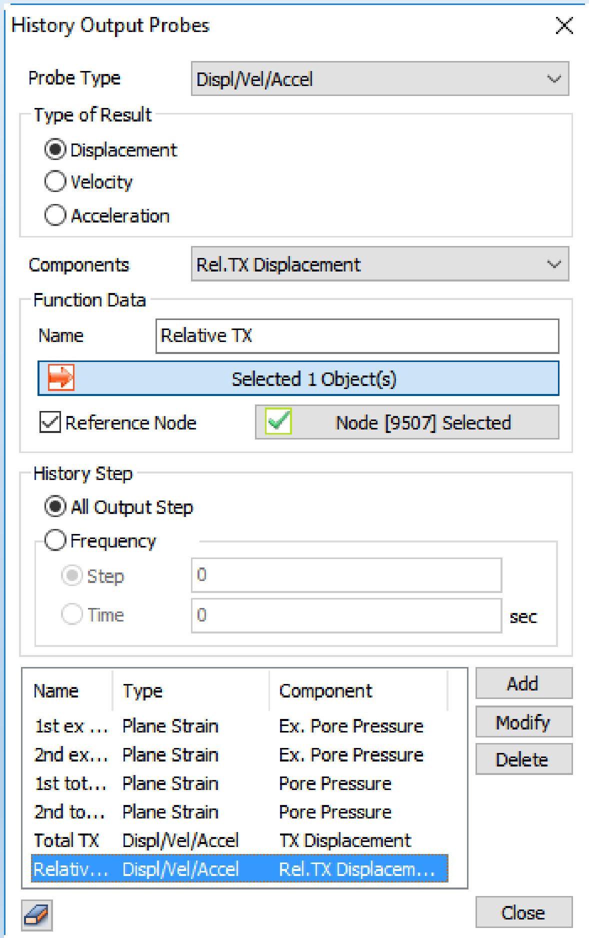
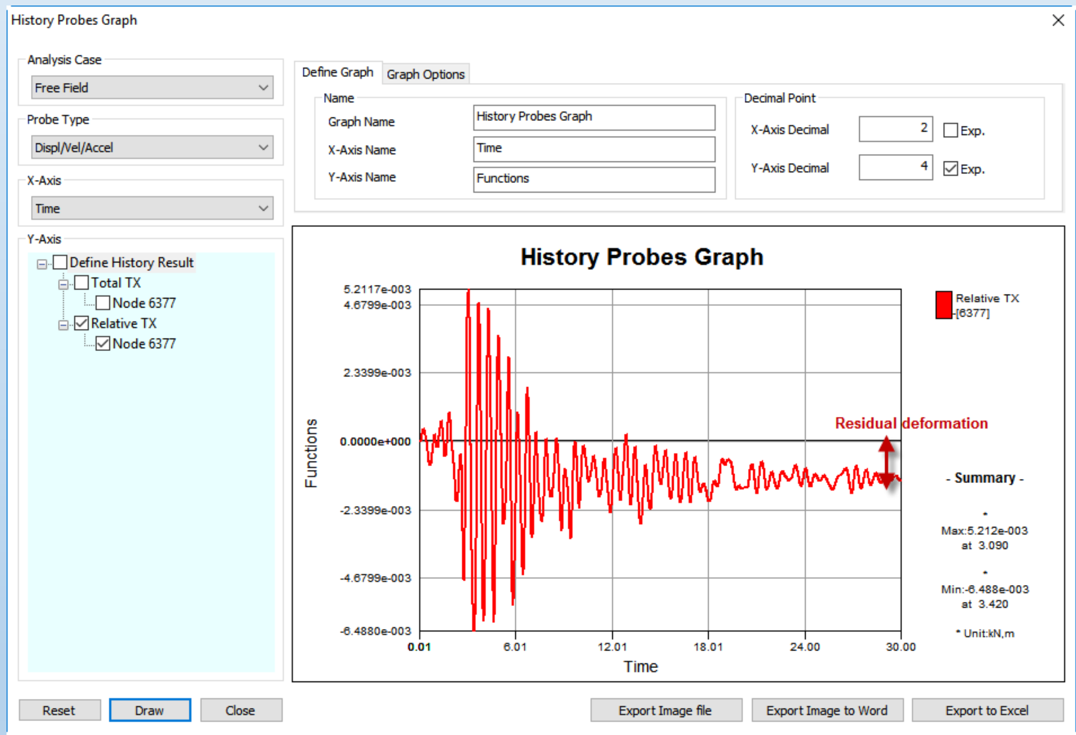
1) History Output Probes
(Analysis > History > History Output Probes & Result > Special Post > History > Graph)

- Output option which can check the result with time at the specified node or element such as total or relative displacement and the excessive pore water pressure with time





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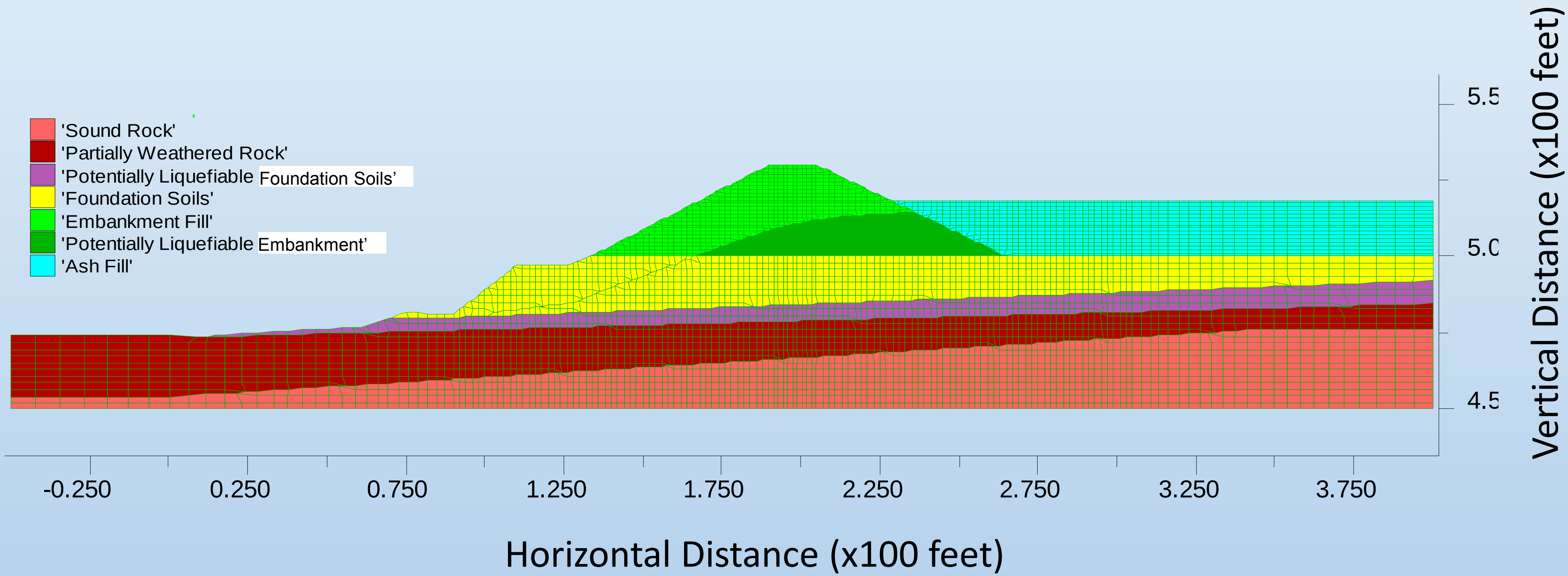
Case Study

(Tailings Pond Embankment Seismic Stability – Excess Pore Pressure and Permanent Displacement using FLAC and GTSNX)

1. The FLAC Model.

One of the difficulty of using
FLAC is the creating of the most appropriate grid for
the problem such as more zone density at the
highly deformed location of the liquefiable layer.



Constitutive Models in FLAC

- FLAC built in – Finn model
- User defined dlls: UBCSand, Wang model, PM4Sand among others.
- The one used for this project: Wang bounding surface plasticity model

Table 3 Material Properties for the nonlinear bounding surface plasticity models [1]

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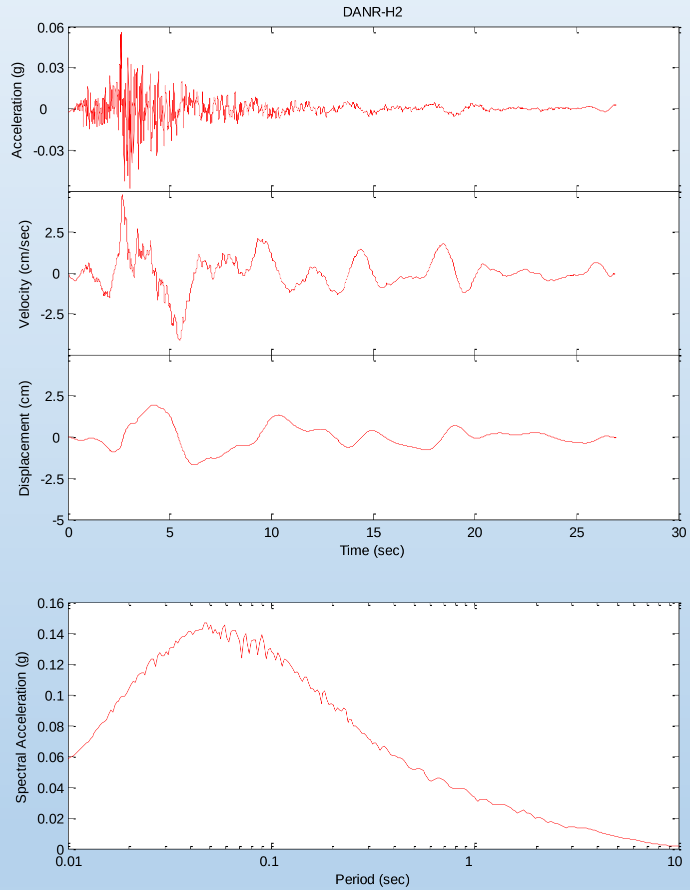
Wang Model Parameters: Most of them have clear physical meaning and can be calibrated from routine lab and/or field tests

Material Description	Friction Angle ϕ (deg)	G_0	h_r	d	kr	fp	bc	ein	Poisson
Potentially Liquefiable (i.e., Saturated) Embankment Fill	31	380	0.25	2.5	0.5	0.75	2	0.65	0.4
Potentially Liquefiable Foundation Soils (Alluvium)	34	150	0.08	1.2	0.5	0.75	2	0.78	0.4

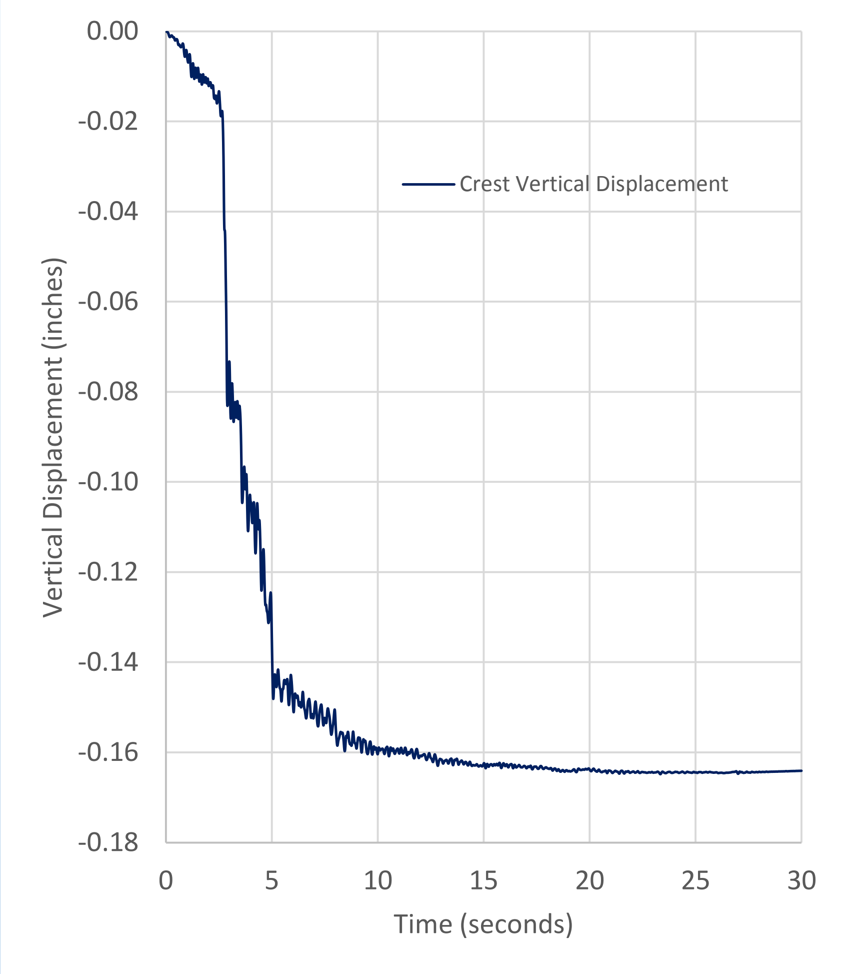
Input Earthquake Record:

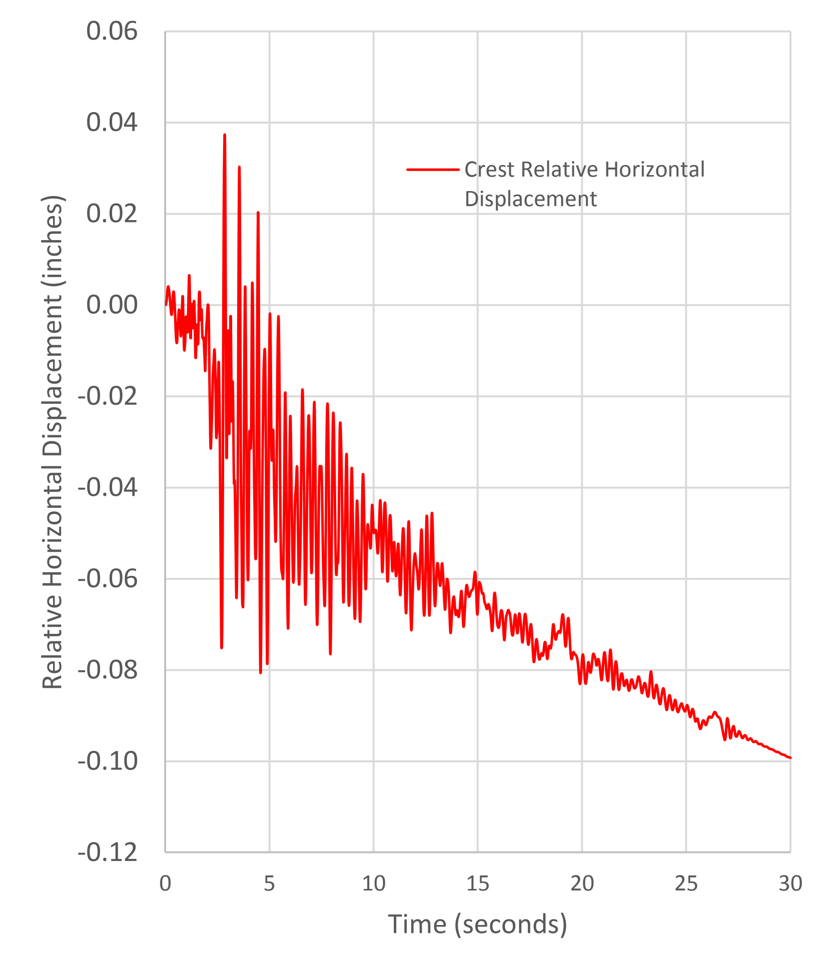
PGA=0.055g. Three earthquake records were obtained from a site specific hazard analysis but for this presentation only the short duration record is used.



Dam Crest Displacement Histories

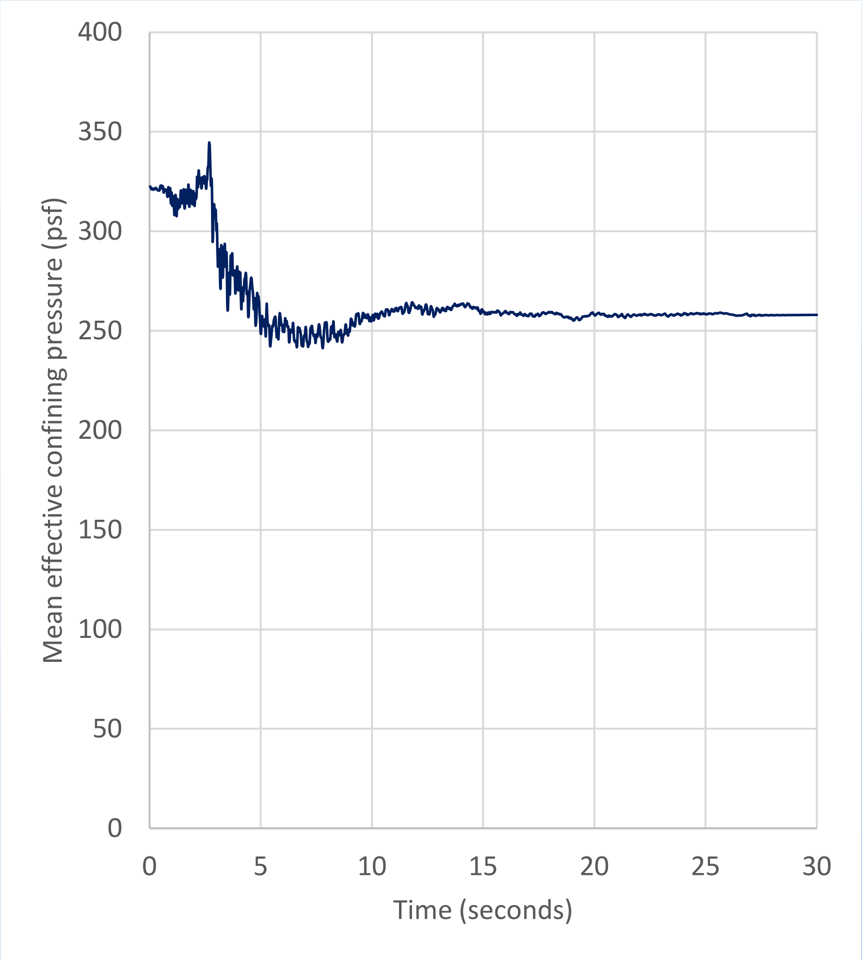
Negative sign for the horizontal direction indicates toward the downstream side; and negative sign for the vertical direction indicates settlement.

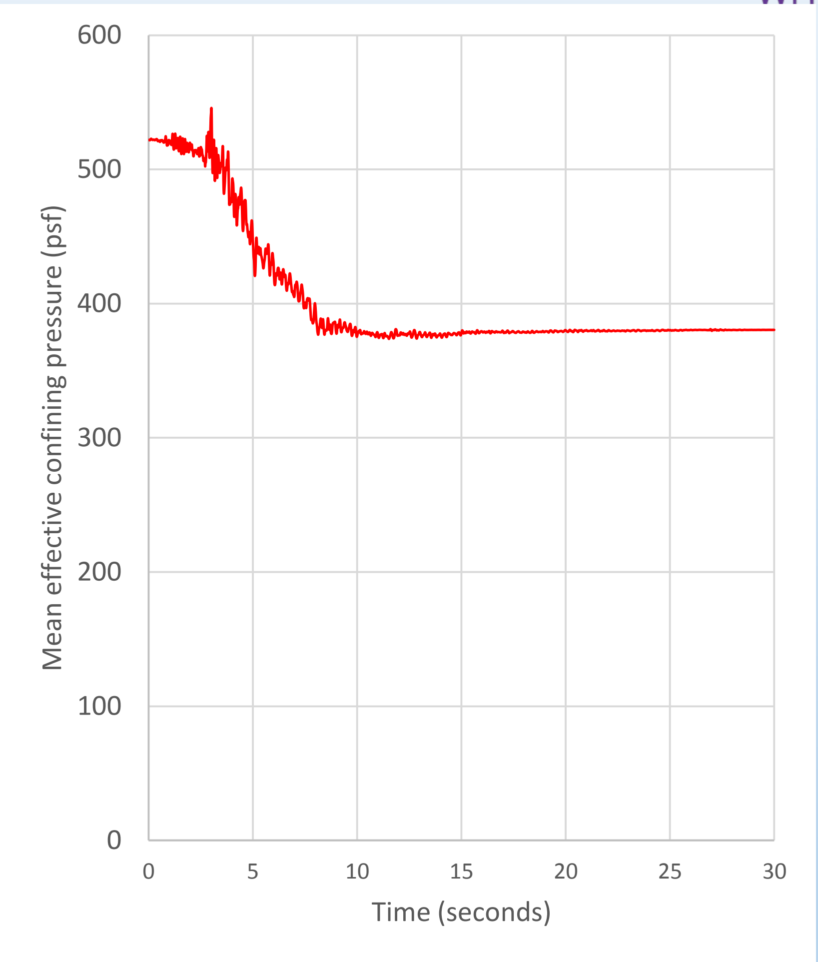




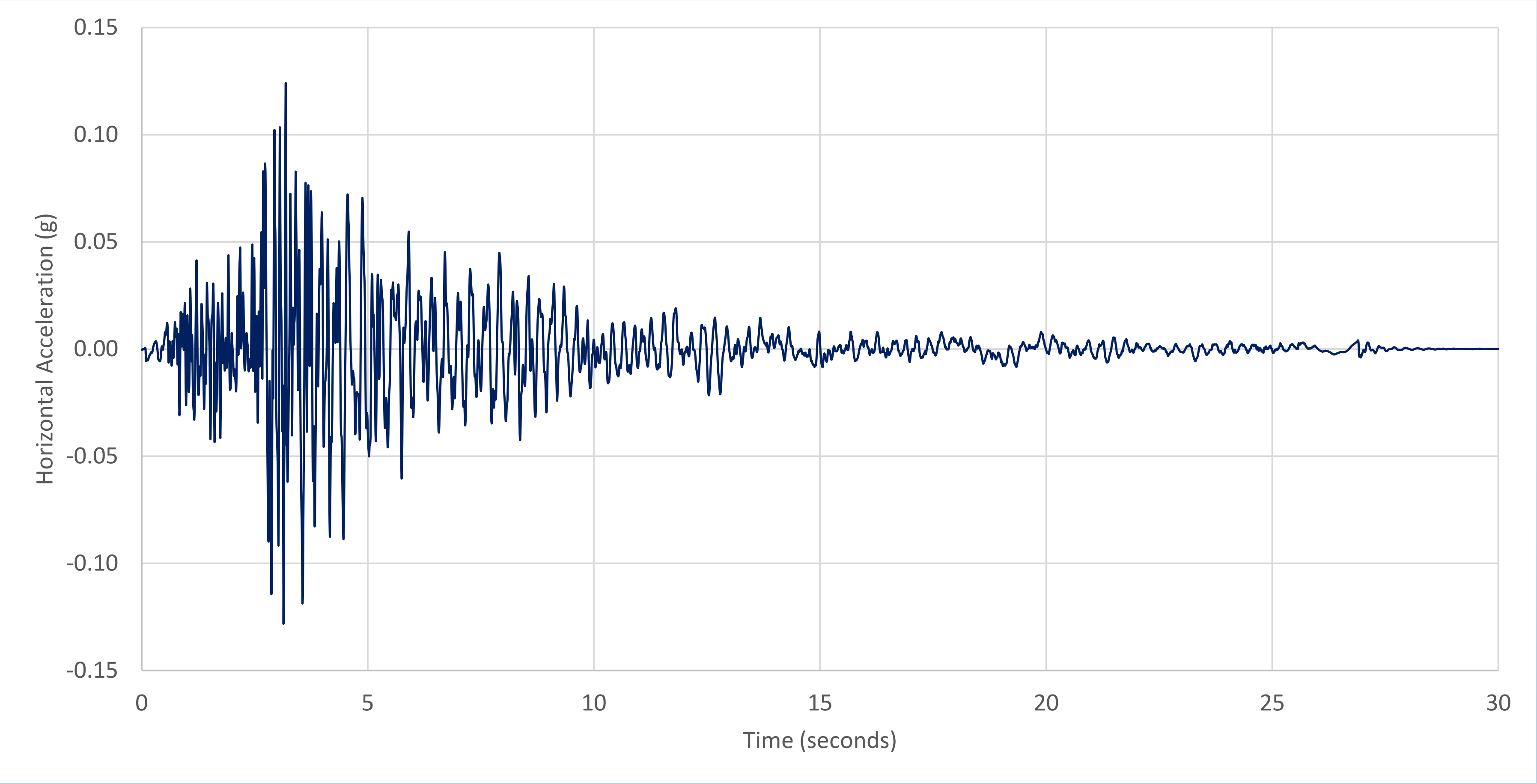
Mean Effective Confining Pressure P Histories: P reduced some but not close to liquefaction when P should be close to zero (0).

Left figure is for a zone near the downstream toe in the liquefiable foundation layer; and right figure is for a zone in the liquefiable embankment about 30 ft horizontally from the Upstream crest edge and 20 ft below the crest.





Dam Crest Horizontal Acceleration:
the input acceleration (slide 32) was amplified more than two times



Result Summary

- Dam embankment experience some shake but with very limit permanent displacements
- Some excess pore pressure generated in the liquefiable foundation and embankment layers but not enough to cause liquefaction
- This result seems reasonable as the input motion has a pga of 0.055g only