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LS-DYNA Training

*MAT_HYSTERETIC_SOIL (*MAT_079)

Richard Sturt, Kirk Ellison, Ben Shao

May 2022

BC hydro

LS-DYNA Training - *MAT_HYSTERETIC_SOIL

- Section 1 – Theory, capabilities and limitations
- Section 2 – Validation and benchmarks
- Section 3 – Project examples
- Section 4 – Other soil models
- Q&A

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LS-DYNA *MAT_HYSTERETIC_SOIL (*MAT_079)

Session 1 - Theory, capabilities and limitations

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MAT_079: theory, capabilities and limitations

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Agenda

- Overview of LS-DYNA soil models
- Shear response
- Volumetric response
- Pressure-dependence of shear response
- Advanced capabilities
- Modelling advice
  - ELFORM, etc
  - Overview of multi-stage analysis
  - Overview of pore pressure analysis

*MAT_079: Theory, capabilities and limitations

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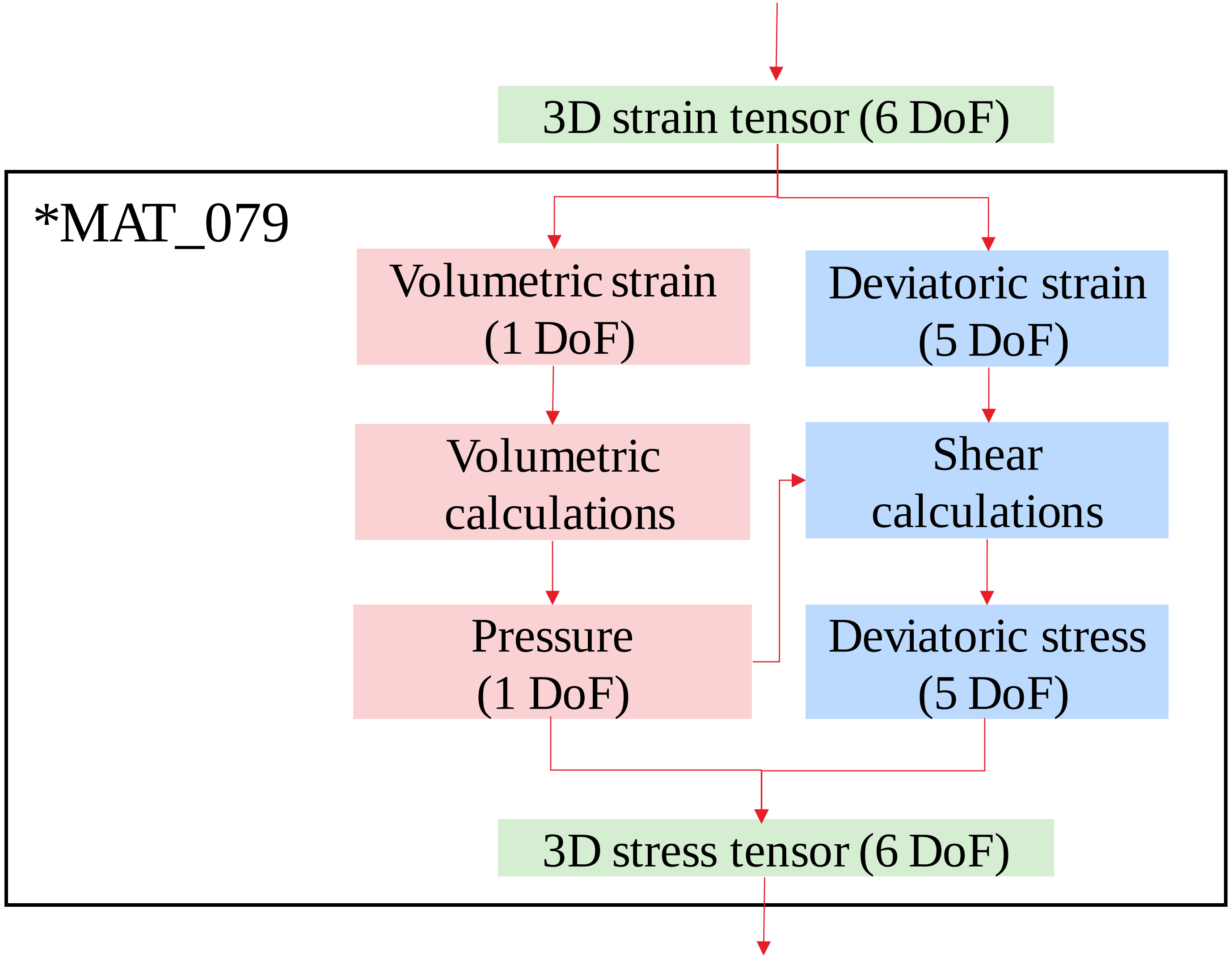
Overview

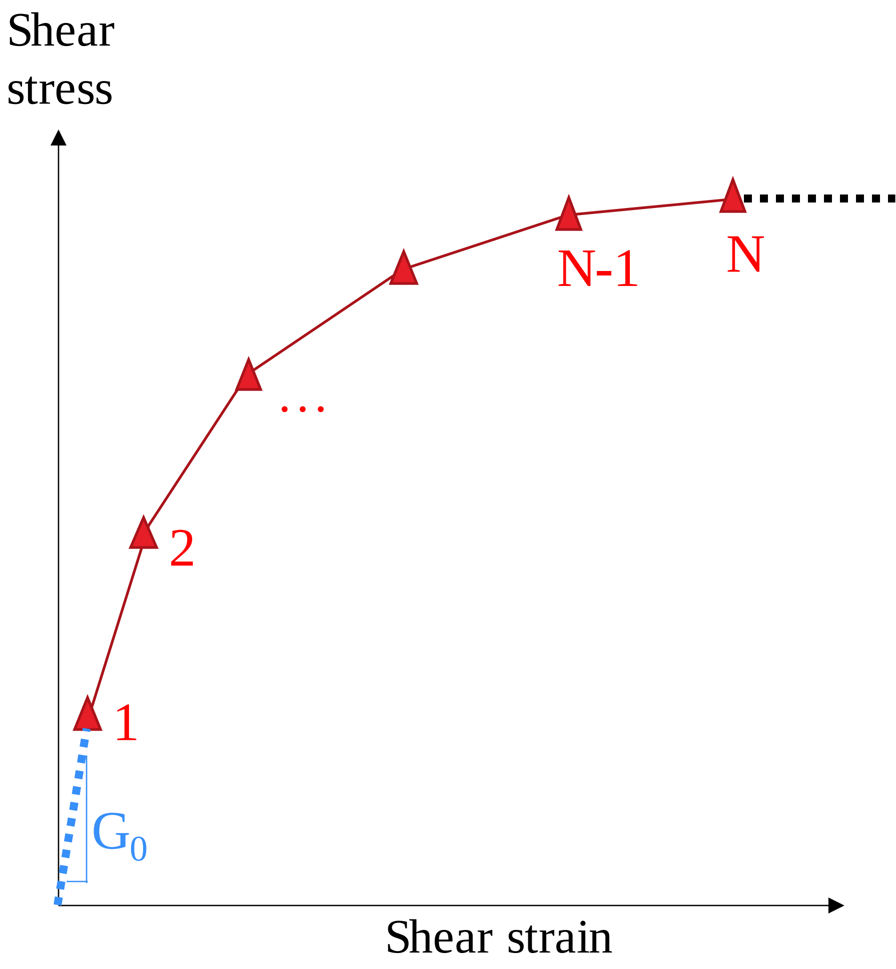


Shear response

Shear response: monotonic

Shear stress-strain curve

- Monotonic shear stress-strain curve (*backbone curve*) is provided by the user.
- Curve applies at a *reference pressure* e.g. 100kPa
- Points 1 through N in ascending order of shear strain
- Gradient must not increase
- Small-strain elastic shear modulus G_0 is defined by gradient from origin to point 1
- Response after last point N is perfectly-plastic
- Watch-it: strain is NOT in percent!



Shear response: monotonic

Input of stress-strain curve

- LCID is the shear stress vs strain curve
  - Maximum 10 points
  - From R14: 20 points
- SFLC scales the stress axis
- GAM1-GAM5 and TAU1-TAU5 are obsolete: same function as LCID but only 5 points.
- RP is the reference pressure at which the curve applies

Card 1	1	2	3	4	5	6	7	8
Variable	MID	R0	K0	P0	B	A0	A1	A2
Type	A	F	F	F	F	F	F	F

Card 2	1	2	3	4	5	6	7	8
Variable	DF	RP	LCID	SFLC	DIL_A	DIL_B	DIL_C	DIL_D
Type	F	F	F	F	F	F	F	F

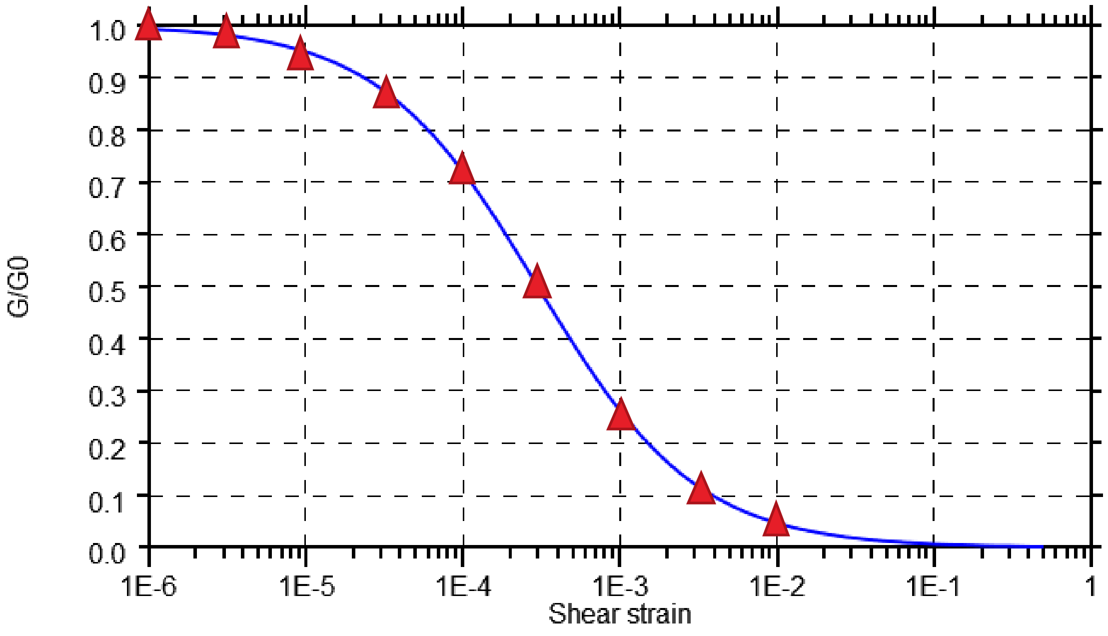
Card 3	1	2	3	4	5	6	7	8
Variable	GAM1	GAM2	GAM3	GAM4	GAM5	LCD	LCSR	PINIT
Type	F	F	F	F	F	I	I	I

Card 4	1	2	3	4	5	6	7	8
Variable	TAU1	TAU2	TAU3	TAU4	TAU5	FLAG5		
Type	F	F	F	F	F			

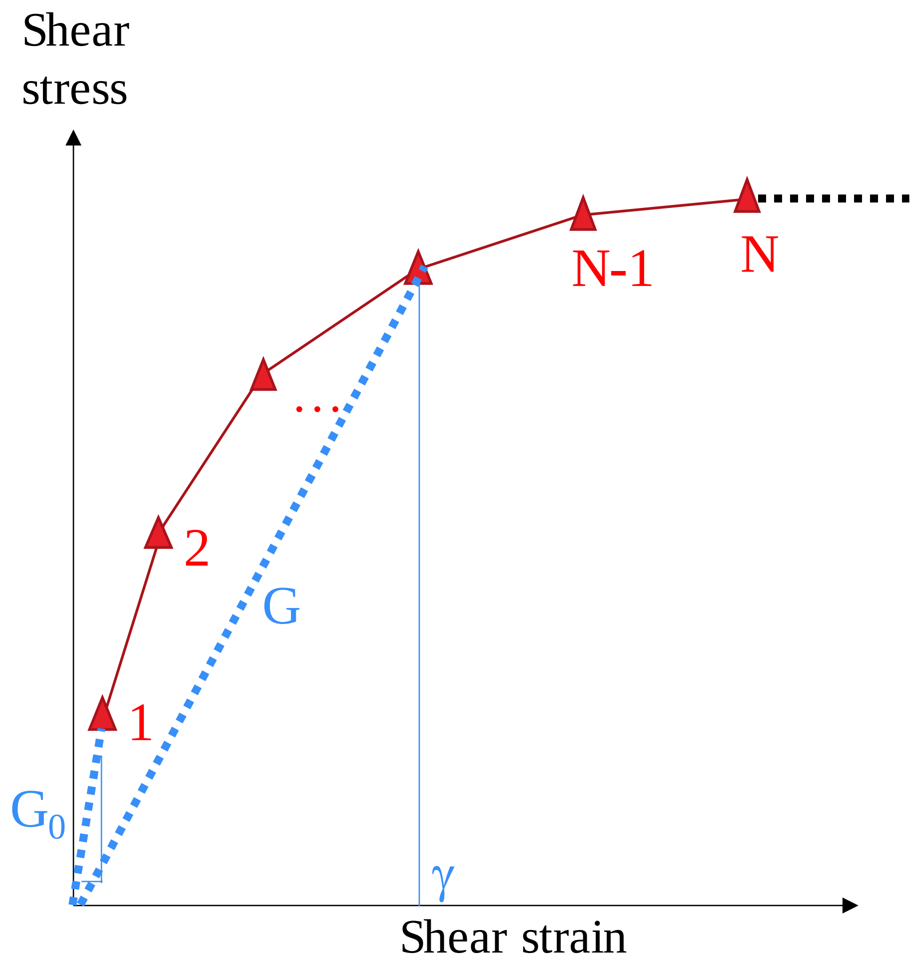
Shear response: monotonic

Shear modulus degradation: converting to shear stress-strain

- Soil mechanics data is often provided in terms of secant shear modulus degradation: G/G_0 vs γ
- Calculate shear stress from G_0, G/G_0 and γ as follows:



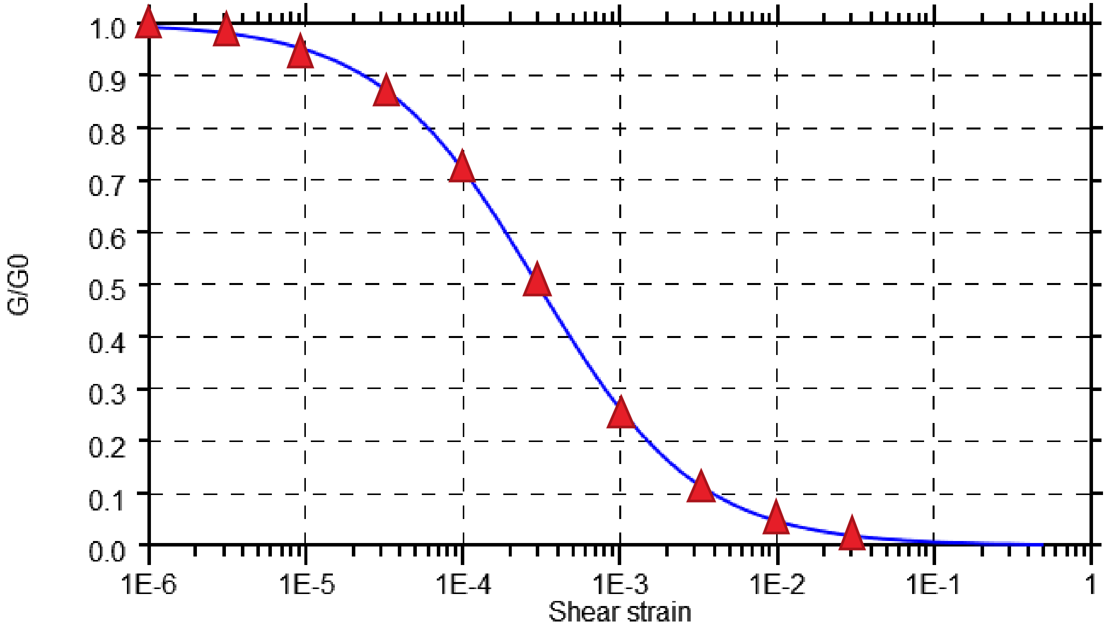
$$\tau(\gamma) = \left({G}/{G_0}(\gamma)\right) G_0 . \gamma$$



Shear response: monotonic

10-point limit: selecting the ten points optimally

- Start with points spread out evenly on the **log-scale** strain axis
- Consider removing points from low-strain end of curve and adding points at high-strain end, or in strain range where maximum response is expected.



Note 1: in LS-DYNA R14 the limit is raised to 20 points

Note 2: interpolation between stress-strain points is linear

Shear strain	Shear stress
1.00e-6	…
Remove? → 3.16e-6	…
1.00e-5	…
3.16e-5	…
1.00e-4	…
3.16e-4	…
1.00e-3	…
3.16e-3	…
1.00e-2	…
3.16e-2	…

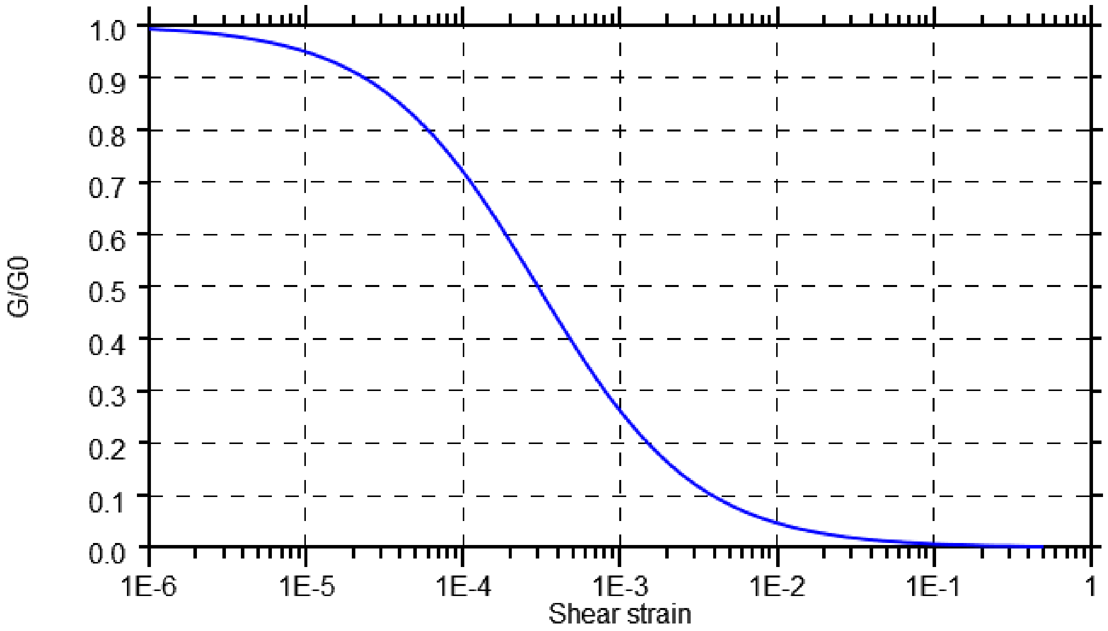
Add? →

Shear response: monotonic

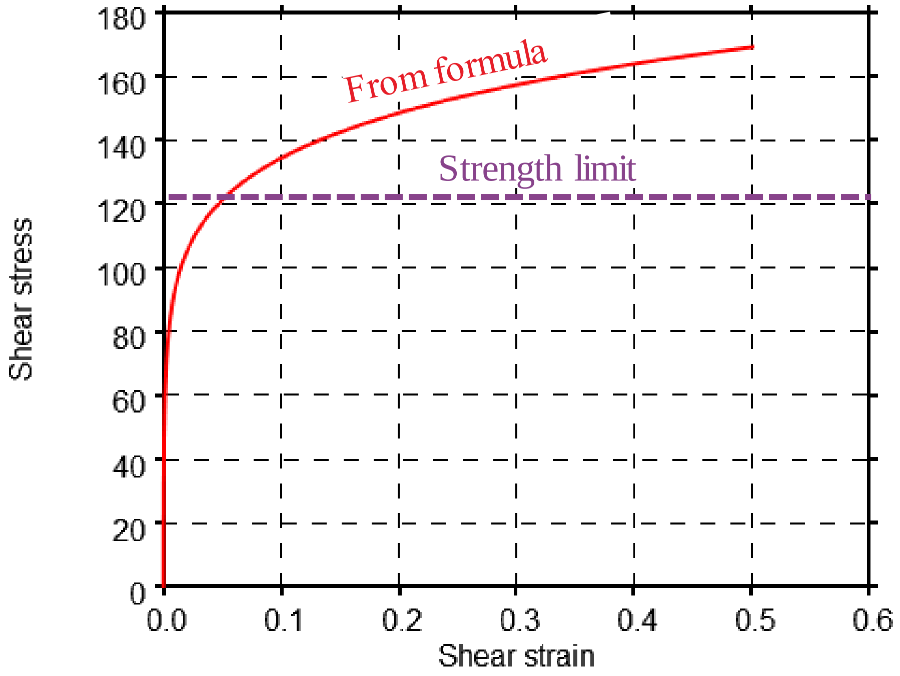
Shear modulus degradation: converting to shear stress-strain

- When the G/G_0 curve comes from a formula, e.g. Darendeli: $$\frac{G}{G_0} = \frac{1}{1 + \left(\frac{\gamma}{\gamma_r}\right)^a}$$

- ... beware of anomalies at the high-strain end of the curve, e.g. exceeding strength limit.



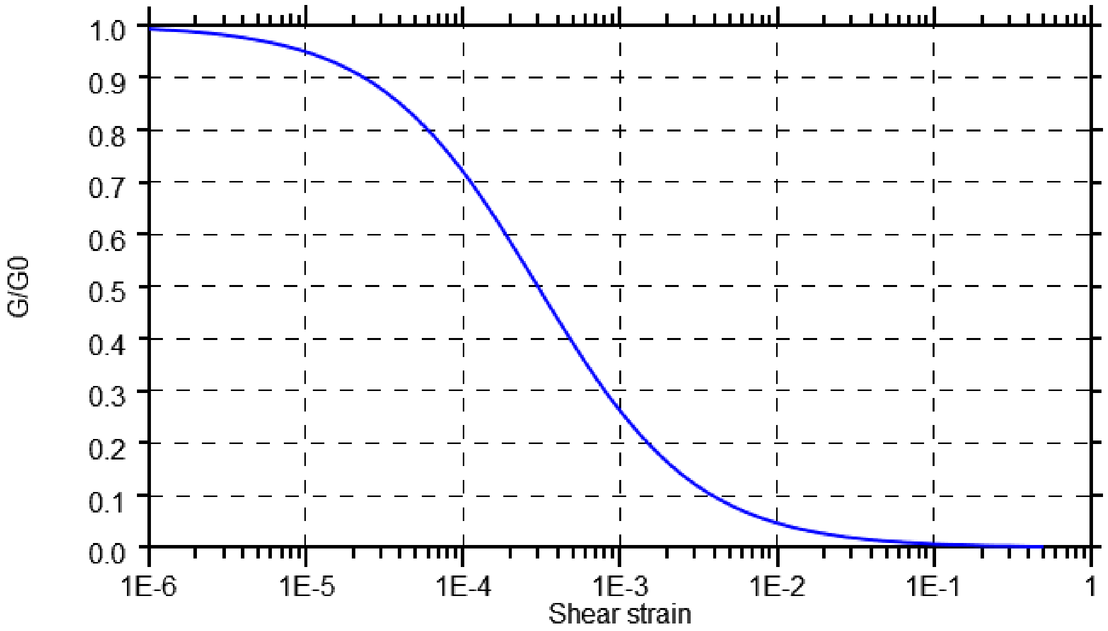
$$\tau(\gamma) = \left({}^{G}/_{G_0}(\gamma) \right) G_0 . \gamma$$



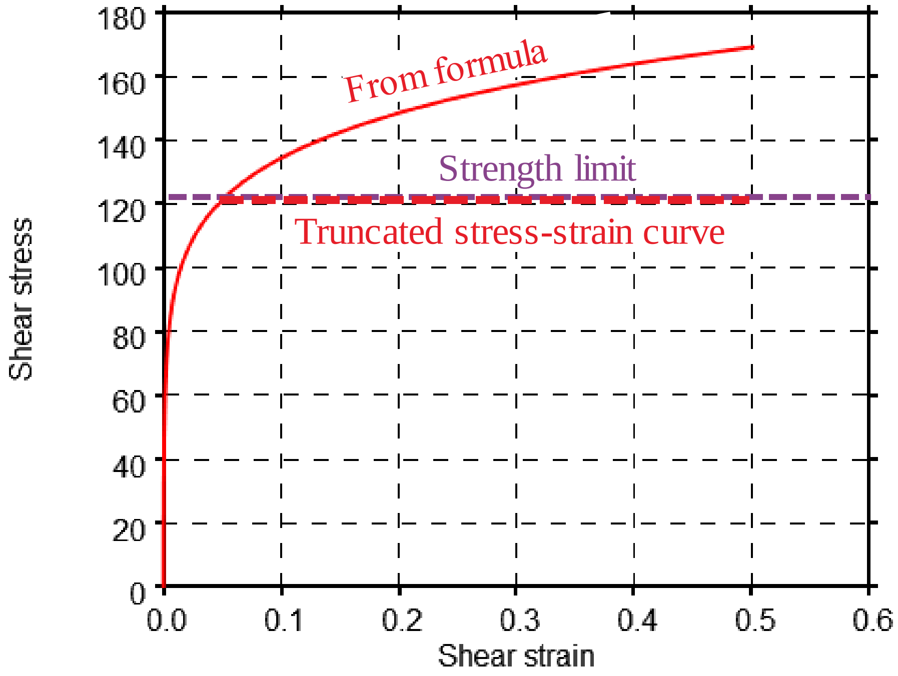
Shear response: monotonic

Shear modulus degradation: converting to shear stress-strain

- Do not just truncate the stress-strain curve at the strength limit
- Avoid abrupt change of slope of stress-strain curve



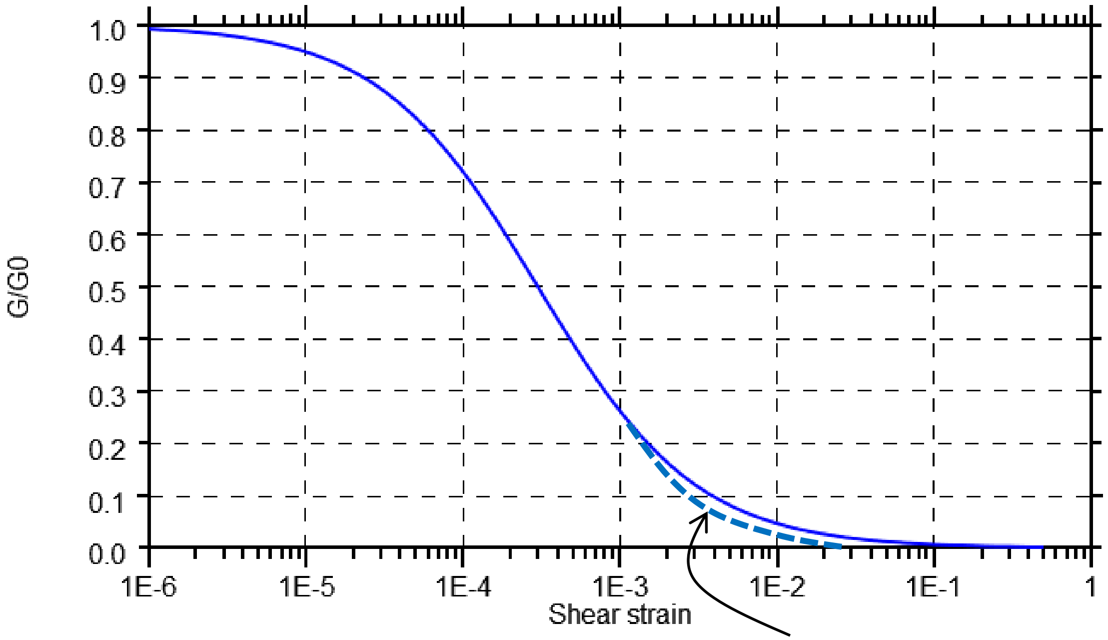
$$\tau(\gamma) = \left({}^{G}\!/_{G_0}(\gamma) \right) G_0 . \gamma$$



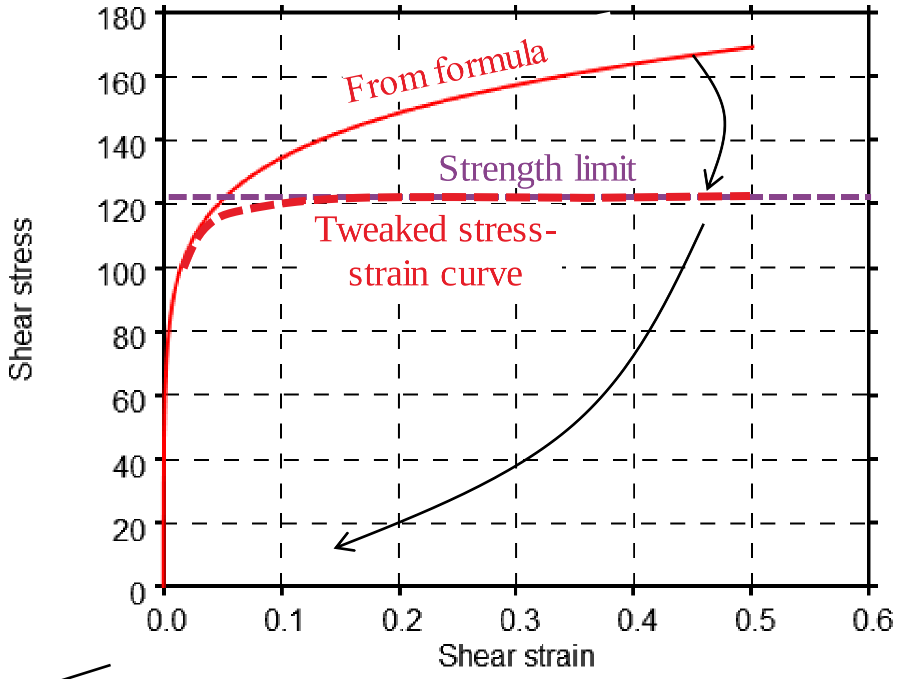
Shear response: monotonic

Shear modulus degradation: converting to shear stress-strain

- Tweak the stress-strain curve to satisfy strtength limit with a smooth curve ...
- ... And back-calculate G/G0 to check the effect of the tweaking on the degradation curve.



$$\tau(\gamma) = \left({}^{G}/_{G_0}(\gamma)\right) G_0 . \gamma$$



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Shear response: monotonic

Use of SFLC

SFLC scales the stress axis of the curve LCID.

Method A:

- SFLC=1.0, LCID is the stress-strain curve
- G_0 is the gradient to point 1 of LCID

Method B:

- SFLC= G_0, y-axis of LCID = stress/G_0 = $\gamma G/G_0$
- E.g. first point on curve = (1.e-6, 1.e-6)
- Same LCID can be re-used for soils with different G_0, provided that degradation curve is the same.
- Makes the G_0 more visible in the input data.

Card 1	1	2	3	4	5	6	7	8
Variable	MID	R0	K0	P0	B	A0	A1	A2
Type	A	F	F	F	F	F	F	F

Card 2	1	2	3	4	5	6	7	8
Variable	DF	RP	LCID	SFLC	DIL_A	DIL_B	DIL_C	DIL_D
Type	F	F	F	F	F	F	F	F

Card 3	1	2	3	4	5	6	7	8
Variable	GAM1	GAM2	GAM3	GAM4	GAM5	LCD	LCSR	PINIT
Type	F	F	F	F	F	I	I	I

Card 4	1	2	3	4	5	6	7	8
Variable	TAU1	TAU2	TAU3	TAU4	TAU5	FLAG5		
Type	F	F	F	F	F			

Shear response: theory

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Shear stress and shear strain definitions

- See notes in User Manual…
- How to reduce 3D stress and strain tensors to single values of "shear stress" and "shear strain"?
- Need a method that works for any axis system and can be calculated for any stress path
  – *Equivalent shear stress* τ_{eq}
  – *Equivalent shear strain* γ_{eq}

From user manual:

Definition of shear strain and shear stress:

Different definitions of "shear strain" and "shear stress" are possible when applied to three-dimensional stress states. MAT_079 uses the following definitions:

- Input shear stress is treated by the material model as,

$$\tau_{eq} \quad 0.5 \times \text{Von Mises Stress} = \sqrt{(3\sigma'_i : \sigma'_i/8)}. \quad = 0.5q \text{ in p'-q terminology}$$

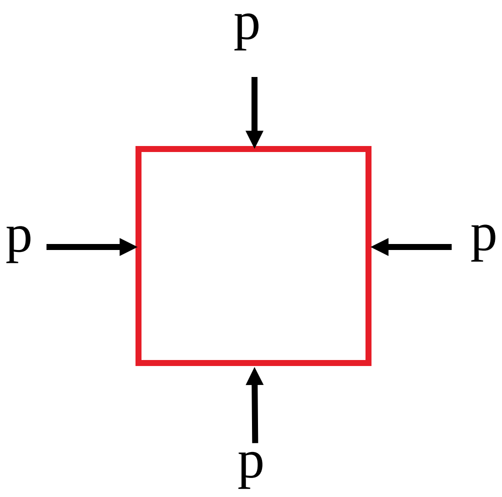
- Input shear strain is treated by the material model as

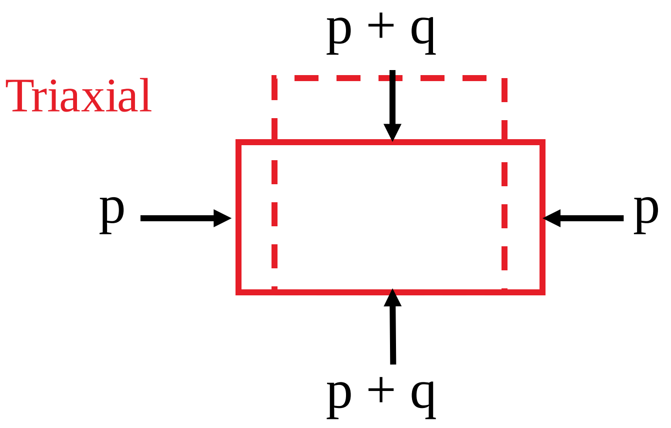
$$\gamma_{eq} \quad 1.5 \times \text{Von Mises Strain} = \sqrt{(3\varepsilon'_i : \varepsilon'_i/2)}.$$

For a particular stress or strain state (defined by the relationship between the three principal stresses or strains), a scaling factor may be needed in order to convert between the definitions given above and the shear stress or strain that an engineer would expect. The MAT_079 definitions of shear stress and shear strain are derived from triaxial testing in which one principal stress is applied, while the other two principal stresses are equal to a confining stress which is held constant, meaning principal stresses and strains have the form (a, b, b). If instead the user wishes the input curve to represent a test in which a pure shear strain is applied over a hydrostatic pressure, such as a shear-box text, then we recommend scaling both the x-axis and the y-axis of the curve by $\boxed{0.866}$. This factor assumes principal stresses of the form $(p + t, p - t, p)$ where t is the applied shear stress, and similarly for the principal strains.

Shear response: theory

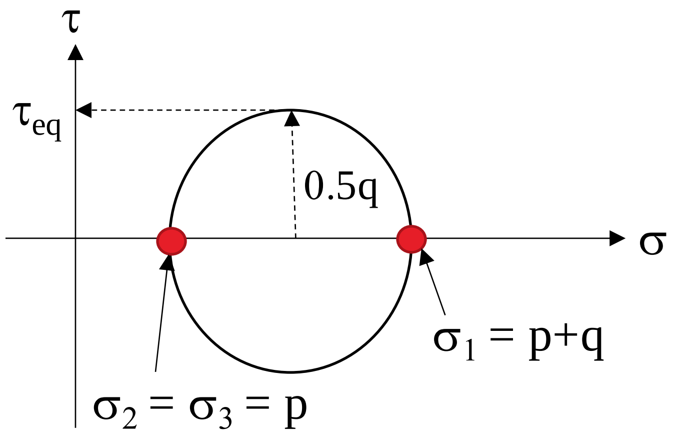
Shear stress and shear strain definitions





Principal stresses $(\sigma_1, \sigma_2, \sigma_3) = (p+q, p, p)$
Principal deviatoric stresses $(\sigma_1, \sigma_2, \sigma_3) = (2q/3, -q/3, -q/3)$
$\sigma_{\text{Von Mises}} = q$

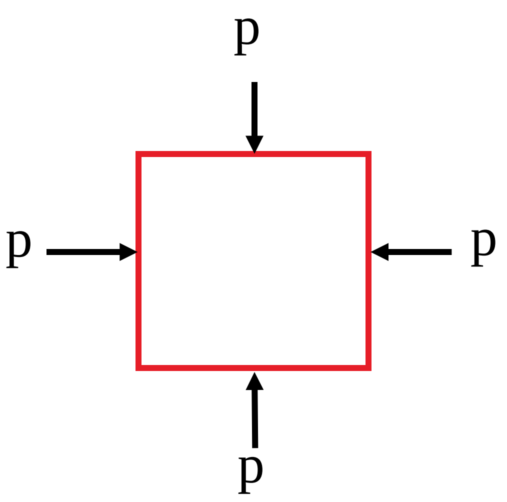
$$\boldsymbol{\tau_{eq}} = 0.5\sigma_{\text{Von Mises}} = \mathbf{0.5q}$$

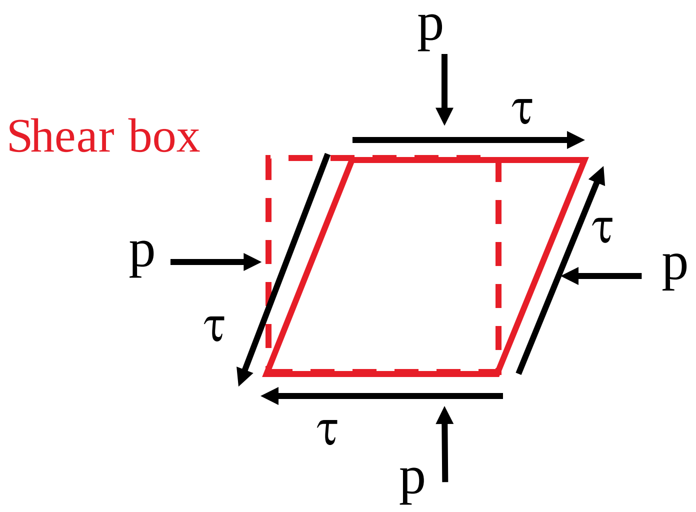


Conclusion: if we want to use a stress-strain curve from a triaxial test, the stress axis of LCID should be 0.5q

Shear response: theory

Shear stress and shear strain definitions



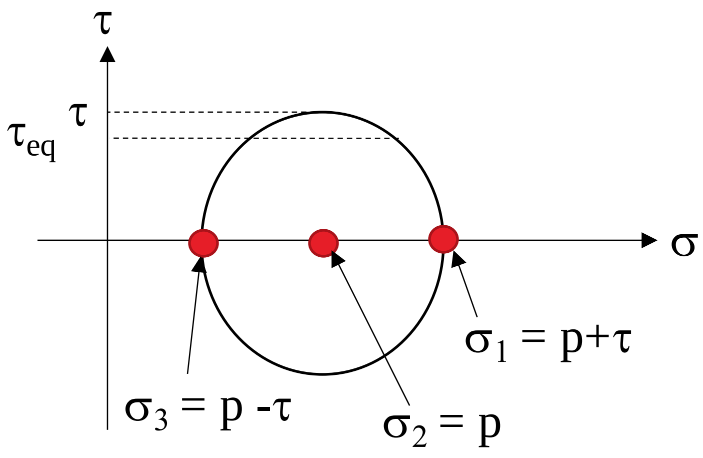


Principal stresses $(\sigma_1, \sigma_2, \sigma_3) = (p+\tau, p, p-\tau)$

Principal deviatoric stresses $(\sigma_1, \sigma_2, \sigma_3) = (\tau, 0, -\tau)$

$\sigma_{Von\ Mises} = \sqrt{3}\tau$

$\boldsymbol{\tau_{eq}} = 0.5\sigma_{Von\ Mises} = \mathbf{0.5\sqrt{3}\tau = 0.866\tau}$ **NOT = τ**



Conclusion: if we want to use a stress-strain curve from a shear box test, the stress axis of LCID should be 0.866 times the measured shear stress.

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Shear response: theory

Shear stress and shear strain definitions

- If using shear stress-strain curves calculated from G/G0 curves, these will be for a stress path like the shear box example, so we normally apply a 0.866 factor to the shear stress axis of LCID
- Working through the math for γ_{eq} in the same way shows that the x-axis of LCID also needs to be scaled by 0.866.
- We apply these factors via SFA, SFO on *DEFINE_CURVE.

*DEFINE_CURVE_{OPTION}

0.866 0.866

Card 1	1	2	3	4	5	6	7	8
Variable	LCID	SIDR	SFA	SFO	OFFA	OFFO	DATTYP	LCINT
Type	A	I	F	F	F	F	I	I
Default	none	0	1.	1.	0.	0.	0	0

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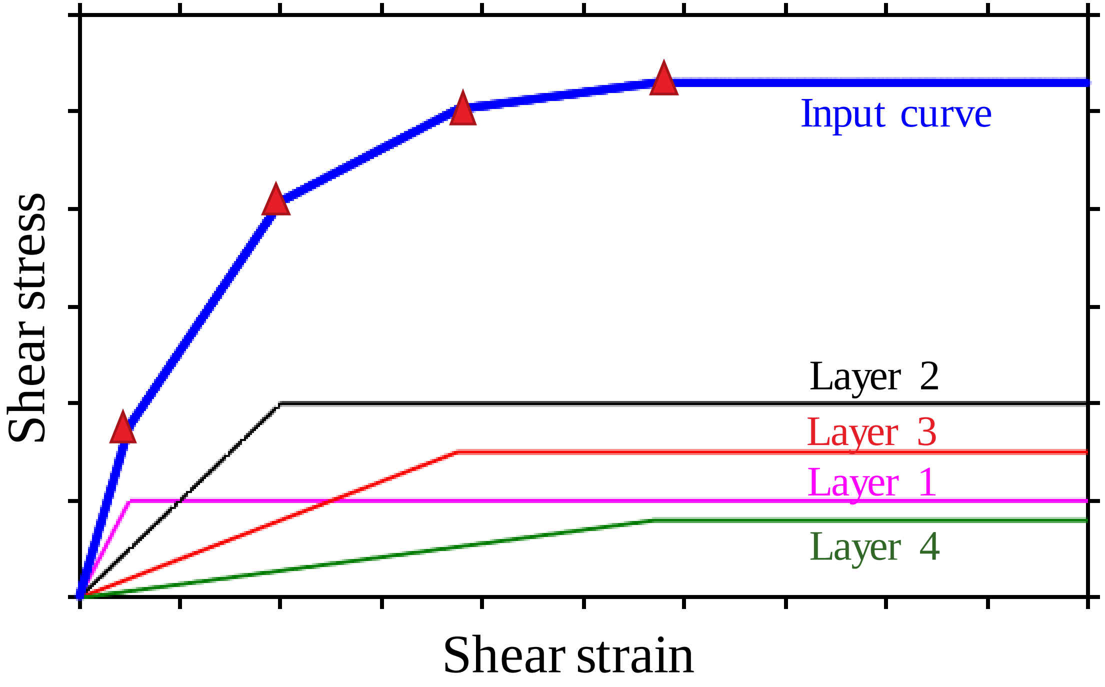
Shear response: theory

Nested layers

- LS-DYNA uses the input shear stress-strain curve to calculate material properties for up to 10 layers (one layer per point on LCID).
- Each layer is elastic-perfectly-plastic with its own shear modulus and yield stress (like a simple metal material)
- Each layer responds to the 3D deviatoric strain tensor and calculates a 3D deviatoric stress tensor.
- The layers act in parallel, i.e. the layer deviatoric stress tensors are summed to give the overall deviatoric stress tensor: like 10 different metals occupying the same element
- Response is independent of axis system and shearing direction: the intended shear stress-strain curve is not limited to certain shearing directions or planes.



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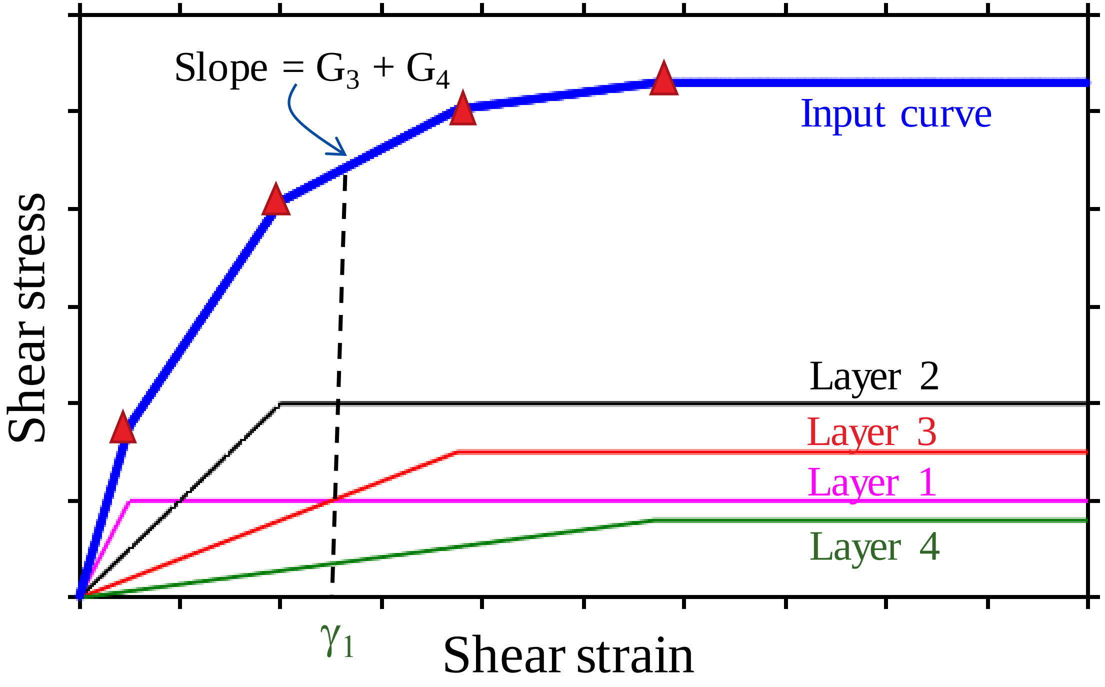
Shear response: theory

Nested layers

- At each point on the input curve there is a reduction of gradient, achieved in practice because one layer yields.
- The slope of the input curve at any given shear strain equals the sum of the elastic stiffnesses of any layers that are still elastic at that shear strain



Example: at shear strain γ_1, layers 1 and 2 are in the plastic regime while layers 3 and 4 are in the elastic regime.

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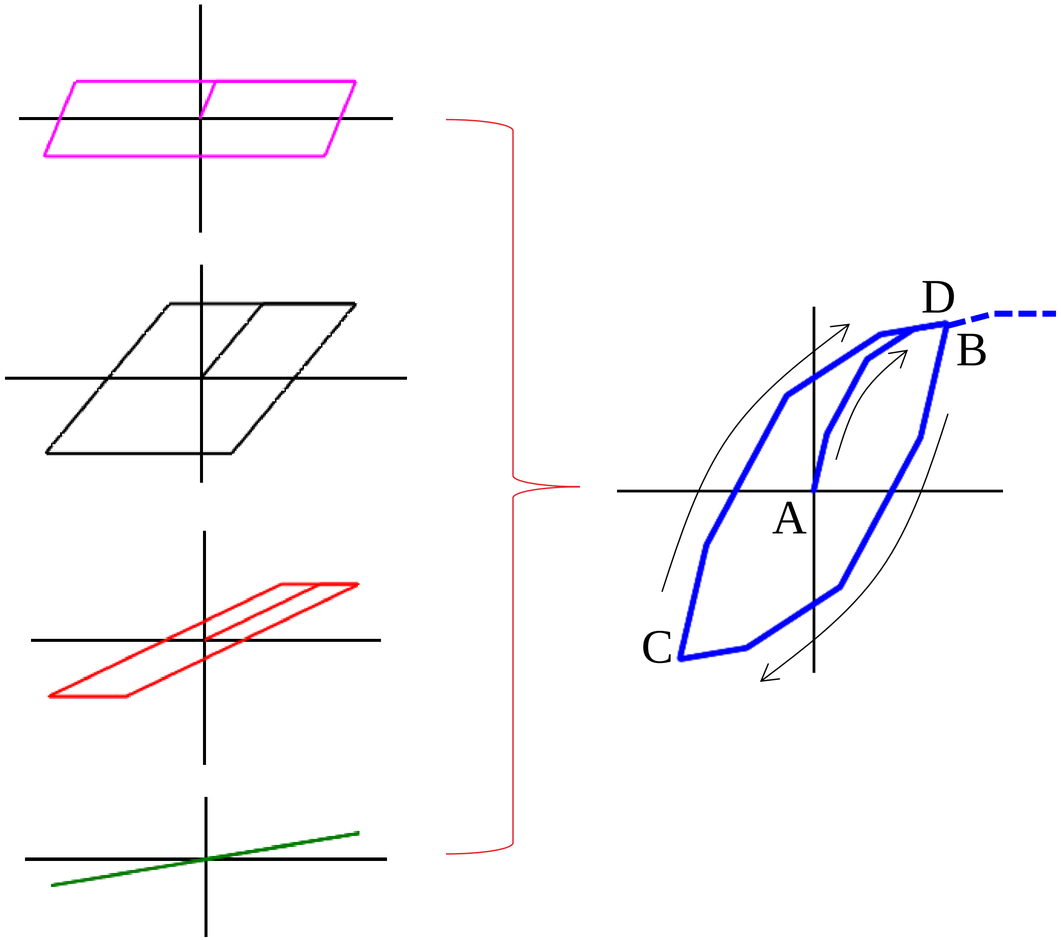
Shear response: hysteresis

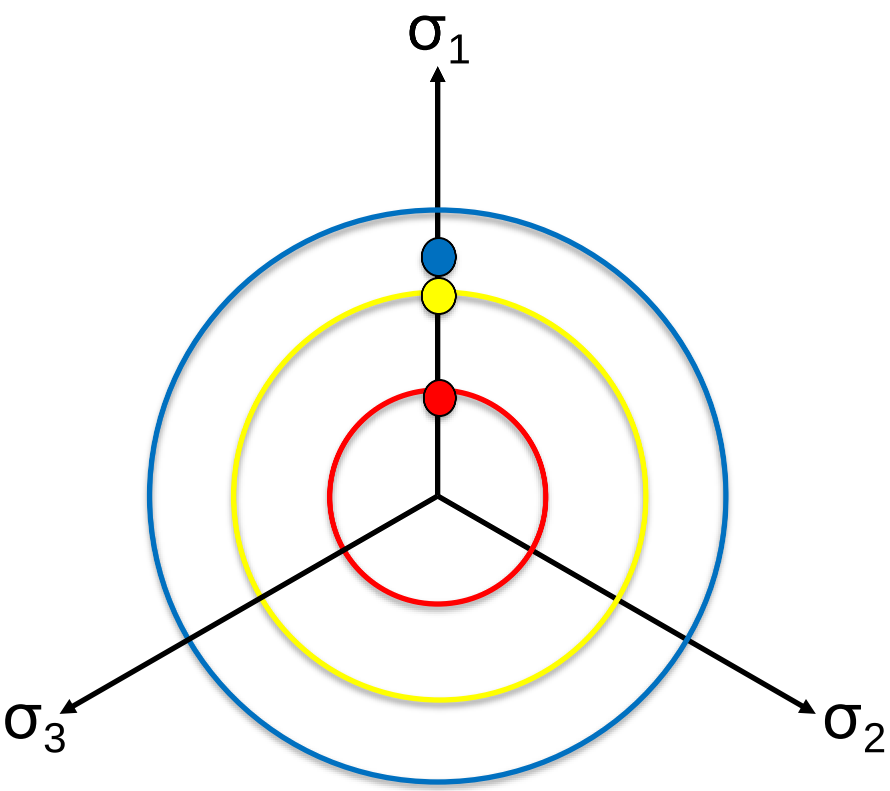
Nested layers

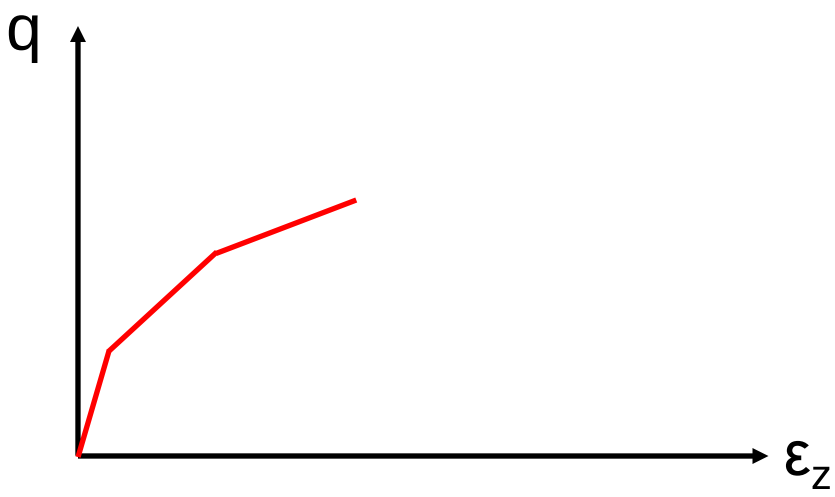
- During hysteresis, each layer responds according to its own elasto-plastic properties – yielding, elastic unload/reload, etc.
- The overall stress-strain response is the sum of the responses of the layers.
- A characteristic of the hysteresis response is that the curve shapes BC and CD are identical to AB but scaled by 2 on both axes.
- The response follows *Masing's Rule*



Shear response: hysteresis

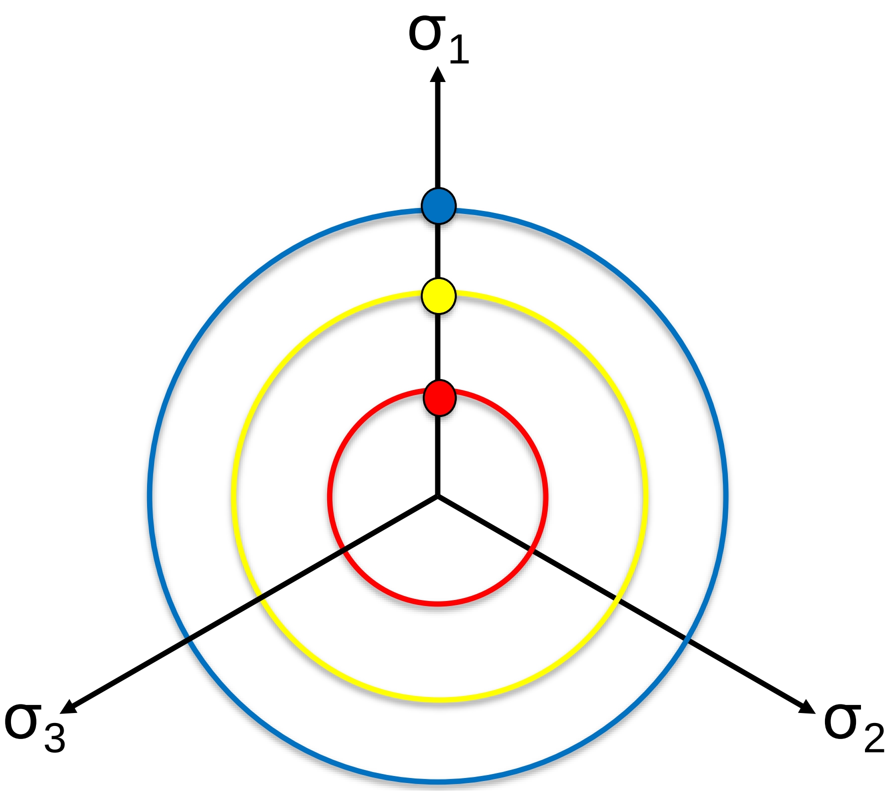
Nested layers

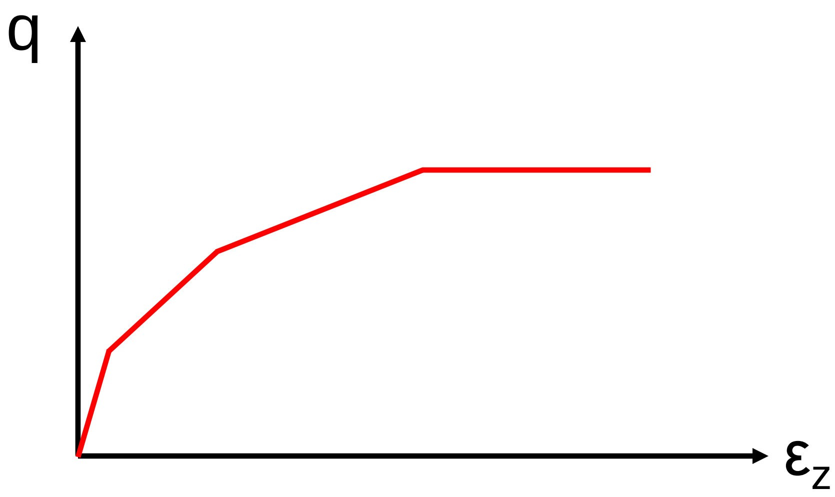




Shear response: hysteresis

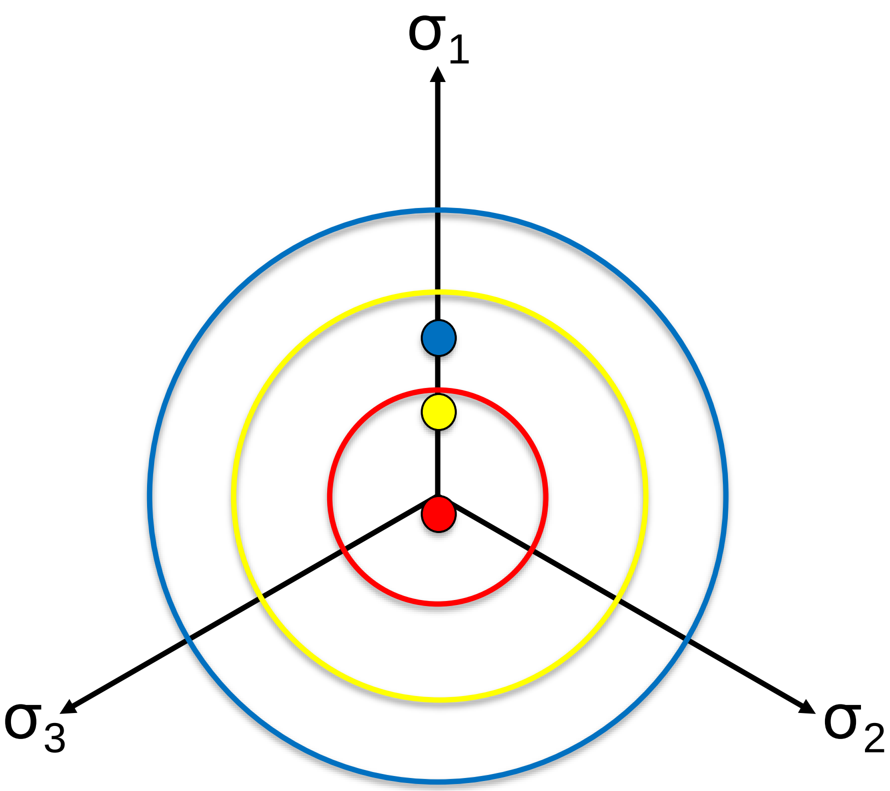
Nested layers





Shear response: hysteresis

Nested layers



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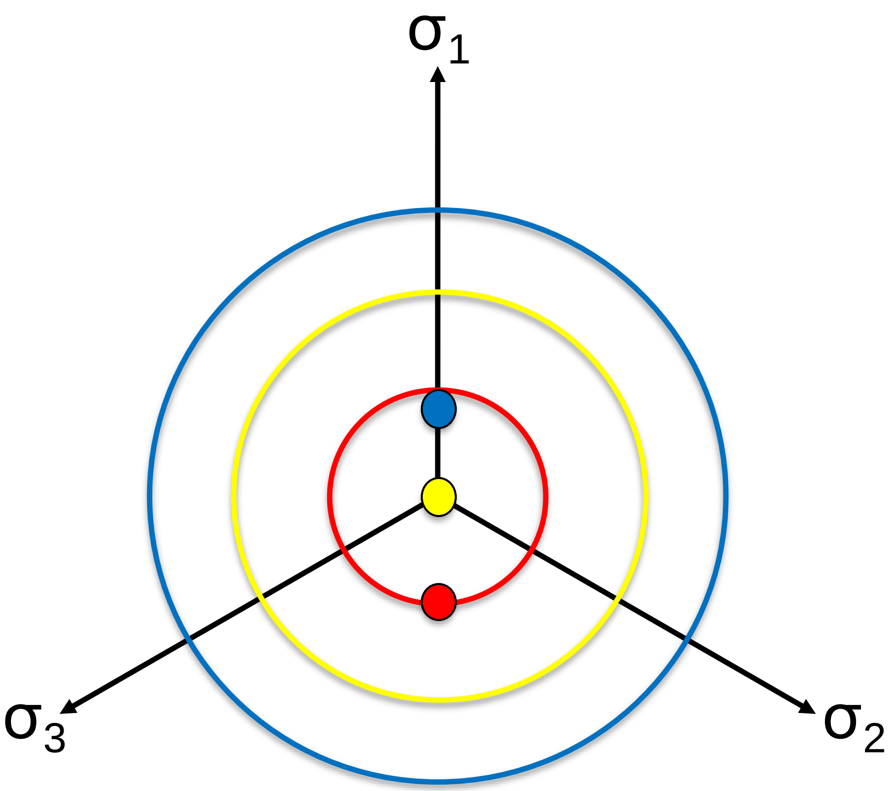
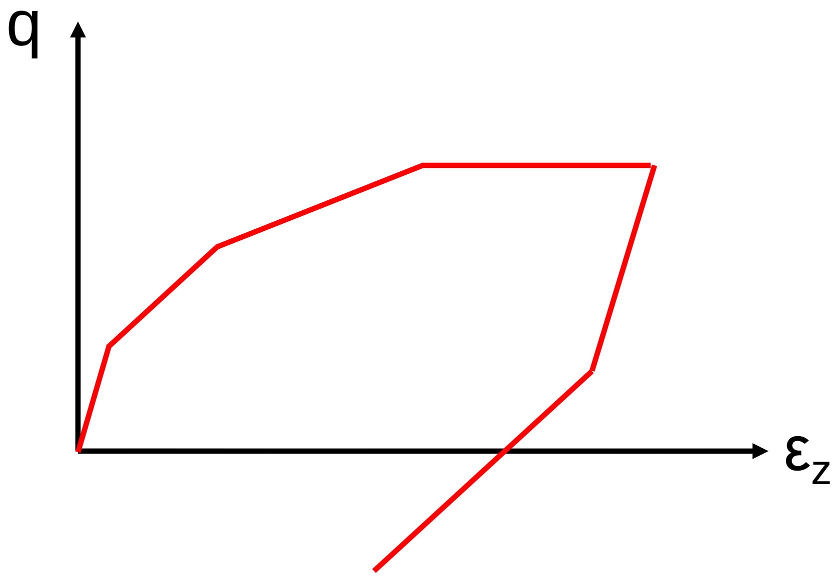
Shear response: hysteresis

Nested layers





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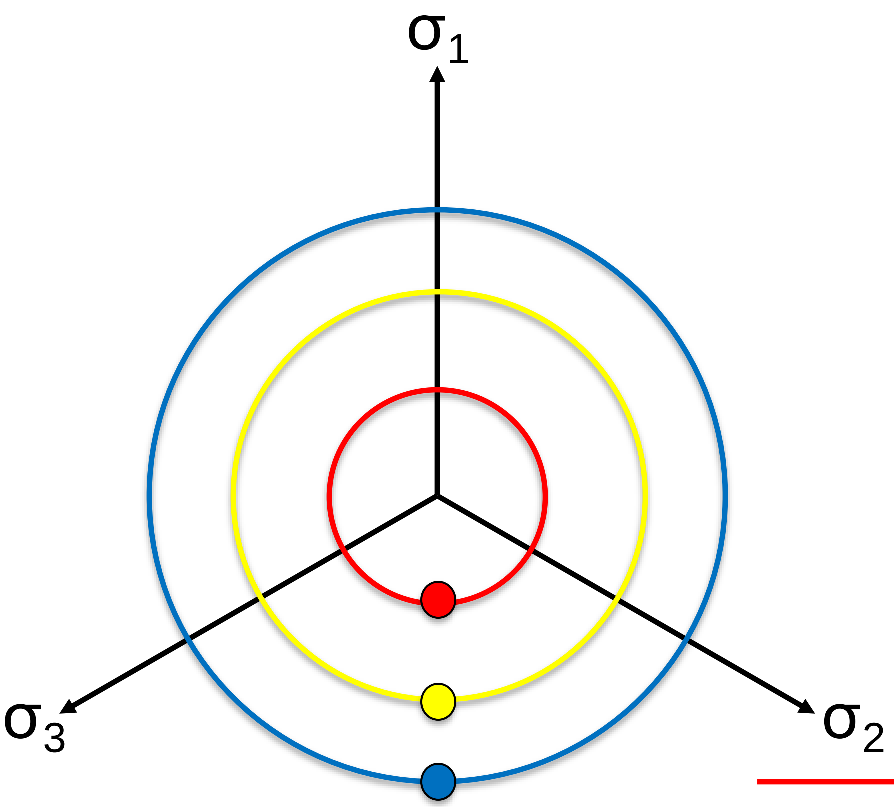
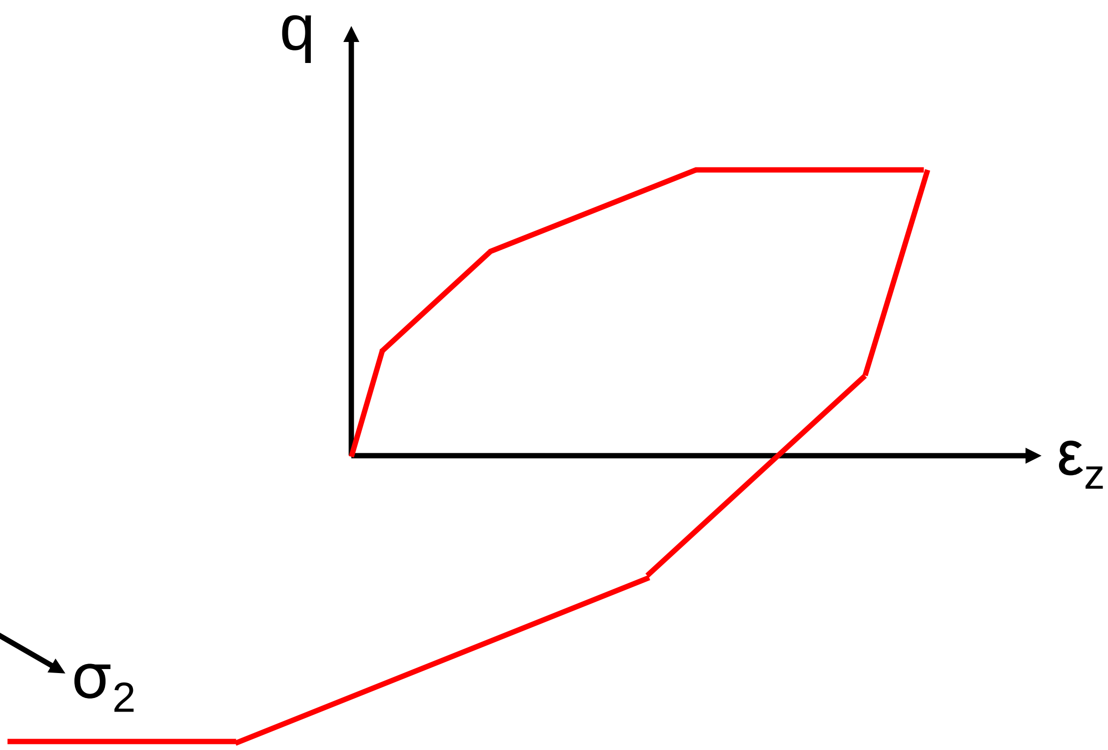
Shear response: hysteresis

Nested layers

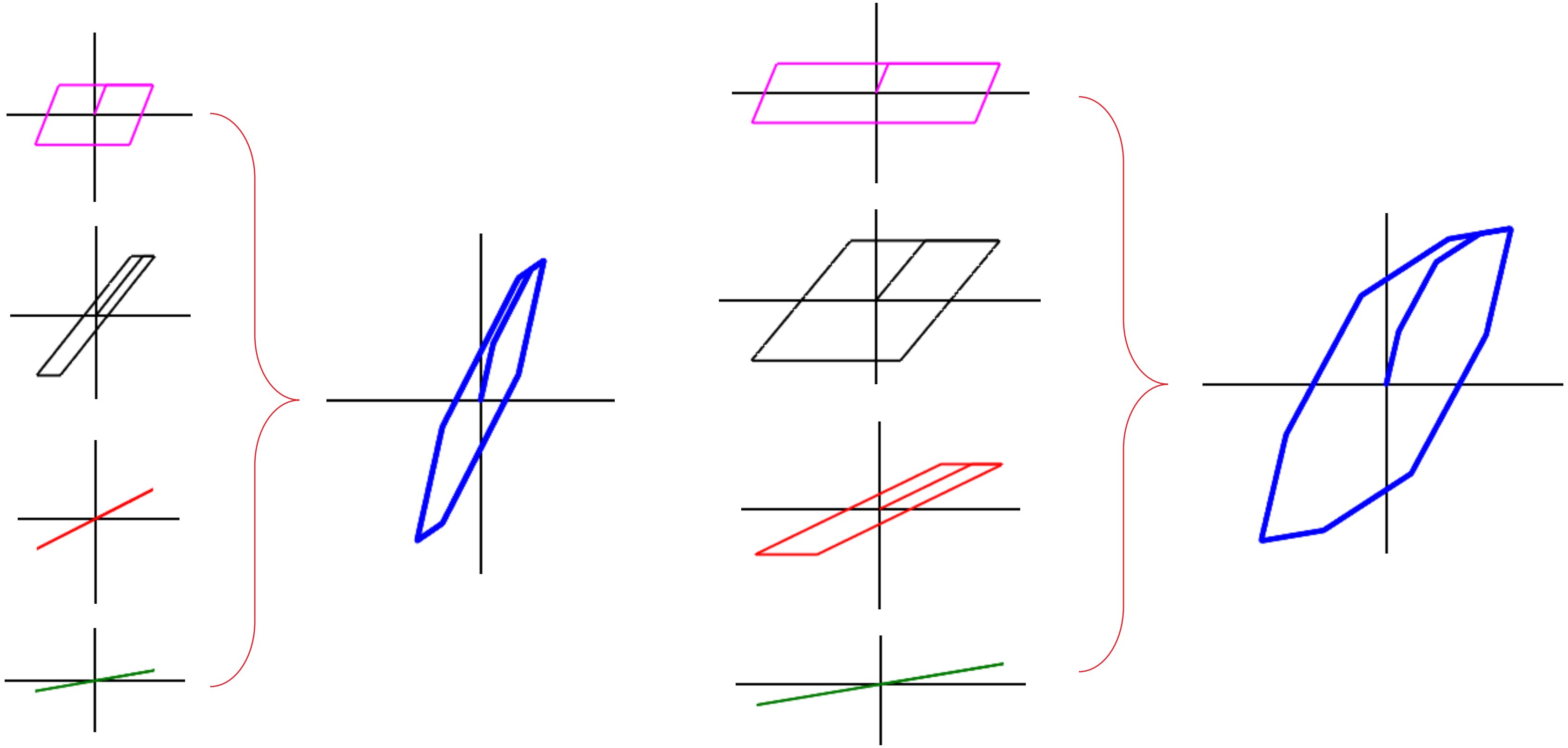




Shear response: hysteresis

Influence of strain amplitude

- At higher strain amplitudes, the hysteresis loops become fatter, i.e. damping increases.



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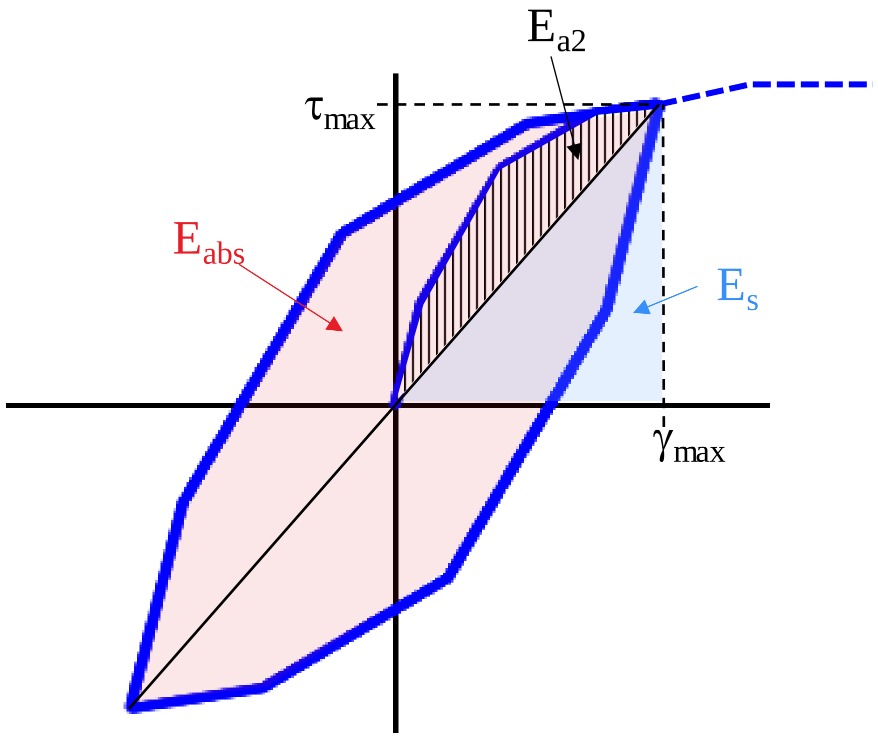
Shear response: hysteresis

Damping

- The amount of material damping at any given strain amplitude depends on the shape of the input shear stress-strain curve.
- When the hysteresis loops follow Masing's rule, the amount of damping is often described as *Masing Damping*
- The material damping ratio, D, can be calculated from the ratio of energy absorbed per cycle (E_{abs}) to the maximum stored energy during the cycle (E_s), shown by the red and blue-shaded areas respectively:



$$D = \frac{E_{abs}}{4\pi E_s}$$

$$E_{abs} = 8E_{a2} = 8\left(\int_0^{\gamma_{max}} f(y)\,d\gamma - E_s\right)$$

$$E_s = {\tau_{max}\gamma_{max}}/{2}$$

f(γ) is the monotonic stress-strain curve (backbone curve)

Shear response: hysteresis

Modulus reduction and damping curves

- In the literature, soils are often characterised by curves of G/G_0 **and** material damping, both versus shear strain amplitude.
- In regular usage of *MAT_079, only one curve is input (LCID). Both the G/G_0 and damping curves follow as a consequence of that (*Masing damping*).
- Masing damping may be unrealistic, especially at higher strains.
- LCID can usually be tweaked to obtain a reasonable compromise between the desired G/G_0 and the desired damping curve within the strain range of interest.
- There is an option, LCD, enabling the user to input a separate damping curve, described on a later slide.

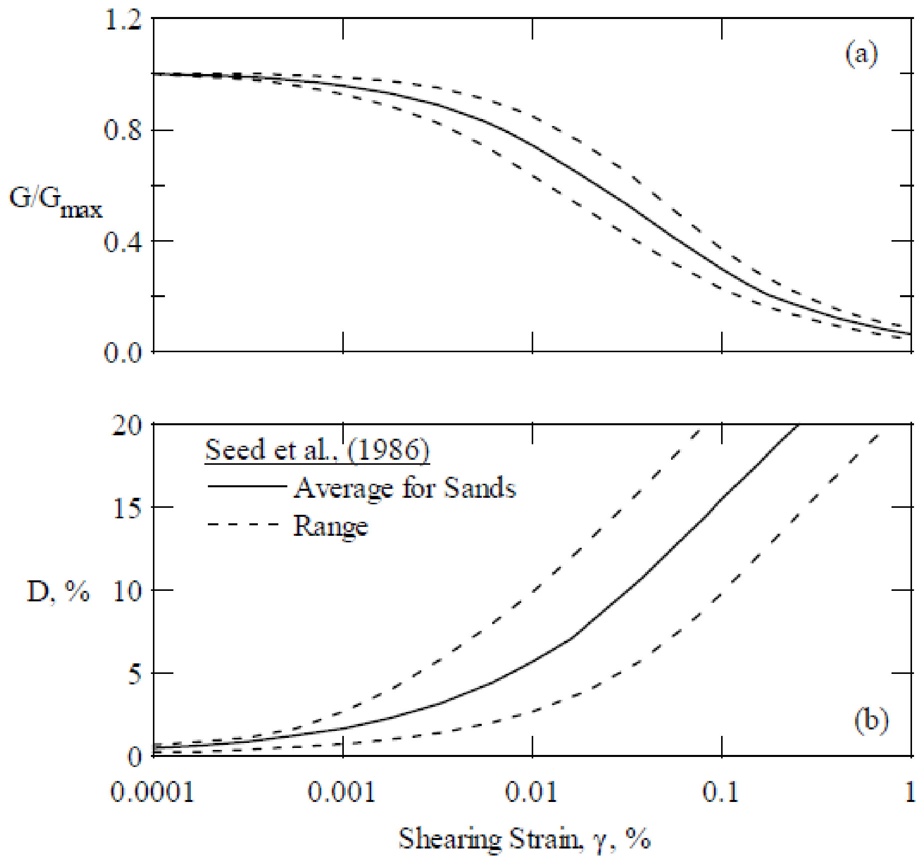


Figure 9.10 Empirical (a) normalized modulus reduction, and (b) material damping curves proposed for sands by Seed et al. (1986)

Volumetric response and pressure sensitivity

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Volumetric response

Elastic bulk volumetric response

- K0 is the elastic bulk modulus
- Linear volumetric response
- **No dilation or contraction caused by shearing**
  - **No pore pressure generation**
- No consideration of critical states
- Input parameters DIL_A through DIL_D are not recommended
  - A historical attempt to model the contraction & dilation effects leading to liquefaction.
  - Too simplistic to capture realistic behaviors.

Card 1	1	2	3	4	5	6	7	8
Variable	MID	R0	$\boxed{K0}$	P0	B	A0	A1	A2
Type	A	F	F	F	F	F	F	F

Card 2	1	2	3	4	5	6	7	8
Variable	DF	RP	LCID	SFLC	DIL_A	DIL_B	DIL_C	DIL_D
Type	F	F	F	F	F	F	F	F

Card 3	1	2	3	4	5	6	7	8
Variable	GAM1	GAM2	GAM3	GAM4	GAM5	LCD	LCSR	PINIT
Type	F	F	F	F	F	I	I	I

Card 4	1	2	3	4	5	6	7	8
Variable	TAU1	TAU2	TAU3	TAU4	TAU5	FLAG5		
Type	F	F	F	F	F			

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Pressure sensitivity of shear response

Influence on stiffness and strength: inputs

Pressure can influence the shear behavior of MAT_079 in two distinct aspects:

- **Stiffness** (input field B)
- **Strength** (input fields A0 A1 and A2)
- P0, RP and PINIT are relevant to both aspects

Card 1	1	2	3	4	5	6	7	8
Variable	MID	R0	K0	P0	B	A0	A1	A2
Type	A	F	F	F	F	F	F	F

Card 2	1	2	3	4	5	6	7	8
Variable	DF	RP	LCID	SFLC	DIL_A	DIL_B	DIL_C	DIL_D
Type	F	F	F	F	F	F	F	F

Card 3	1	2	3	4	5	6	7	8
Variable	GAM1	GAM2	GAM3	GAM4	GAM5	LCD	LCSR	PINIT
Type	F	F	F	F	F	I	I	I

Card 4	1	2	3	4	5	6	7	8
Variable	TAU1	TAU2	TAU3	TAU4	TAU5	FLAG5		
Type	F	F	F	F	F			

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Pressure sensitivity of shear response

Inputs for influence of pressure on strength

- Backbone curve (LCID with SFLC) is the shear stress-strain curve at the reference pressure (RP, called p_{ref} in the equations)
- A0, A1 and A2 govern how the strength from the curve gets scaled at different pressures:
  - Shear strength at pressure p = K_p*shear strength at pressure RP
  - Generic formula: K_p is proportional to $(A0 + A1p + A2p^2)^{1/2}$
- Special cases:
  - No pressure dependence: A0=1, A1=0, A2=0
  - Frictional (strength proportional to pressure): A0=0, A1=0, A2=1
- P0 is for numerical stability. Set it to a small negative value (tensile pressure, e.g. -0.001 kPa) such that there is a small non-zero strength at zero pressure.

$$\frac{\tau(p,\gamma)}{\tau(p_{\text{ref}},\gamma)} = \sqrt{\frac{[a_0 + a_1(p - p_0) + a_2(p - p_0)^2]}{[a_0 + a_1(p_{\text{ref}} - p_0) + a_2(p_{\text{ref}} - p_0)^2]}}$$

Cohesive

A0=1, A1=0, A2=0:

$$K_p = \frac{\tau(p,\gamma)}{\tau(p_{\text{ref}},\gamma)} = 1.0$$

Sand/granular

A0=0, A1=0, A2=1:

$$K_p = \frac{\tau(p,\gamma)}{\tau(p_{\text{ref}},\gamma)} = \frac{(p - p_0)}{(p_{ref} - p_0)} \approx \frac{p}{p_{ref}}$$

Pressure sensitivity of shear response

Inputs for influence of pressure on stiffness

- Backbone curve (LCID with SFLC) is the shear stress-strain curve at the reference pressure (RP, called p_{ref} in the equations)
- B (called *b* in the equations) governs how the stiffness from the curve gets scaled at different pressures:
  - Stiffness at pressure p = K_e*stiffness at pressure RP
  - Generic formula: K_e is proportional to p^b
- B=0 means no influence of pressure on stiffness.
- B must be < 1.
- Both the bulk modulus and the shear modulus are scaled by the same factor K_e

$$G(p) = \frac{G_0 (p - p_0)^b}{(p_{\text{ref}} - p_0)^b}$$

$$K(p) = \frac{K_0 (p - p_0)^b}{(p_{\text{ref}} - p_0)^b}$$

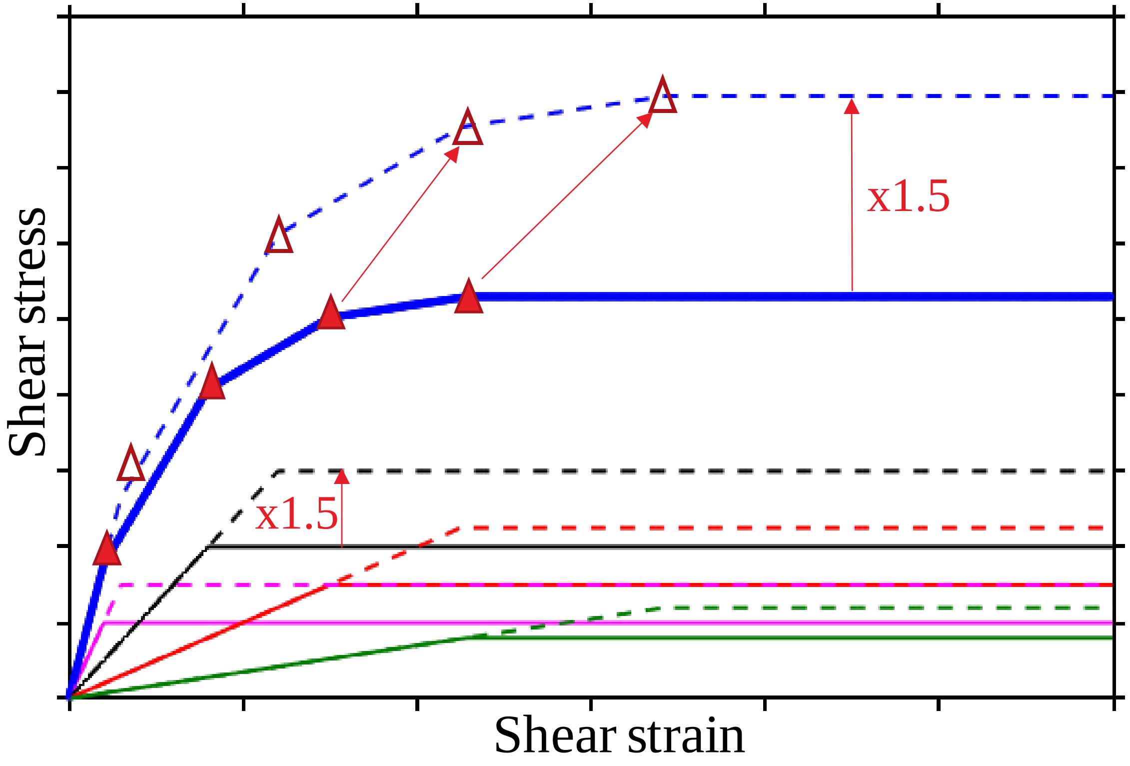
e.g. B=0.5:

$$K_e = \frac{(p - p_0)^{0.5}}{(p_{ref} - p_0)^{0.5}} \approx \left(\frac{p}{p_{ref}}\right)^{0.5}$$

Pressure sensitivity of shear response

Influence of pressure on strength

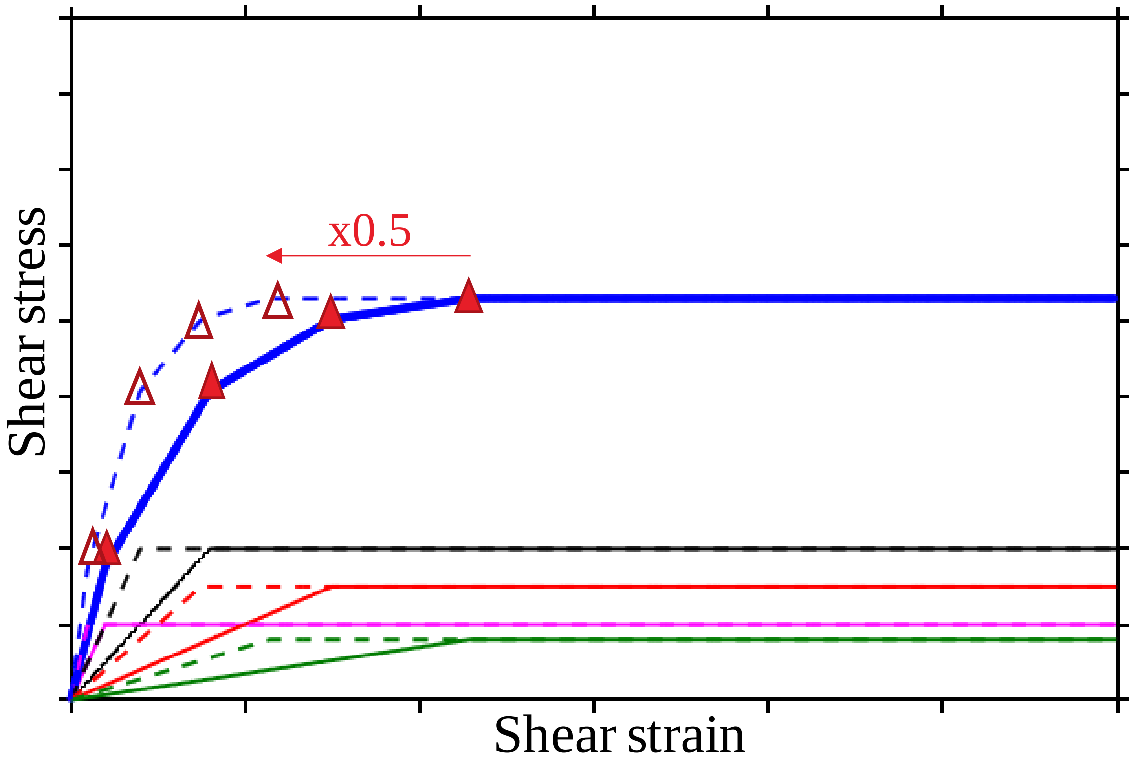
- K_p applies to yield stress of each layer
- Example shows K_p =1.5, K_e=1 (strength scaling only)
- Note that the points on the backbone curve shift along the strain axis (as well as along the stress axis) because the stiffness is unchanged in this example.



Pressure sensitivity of shear response

Influence of pressure on stiffness

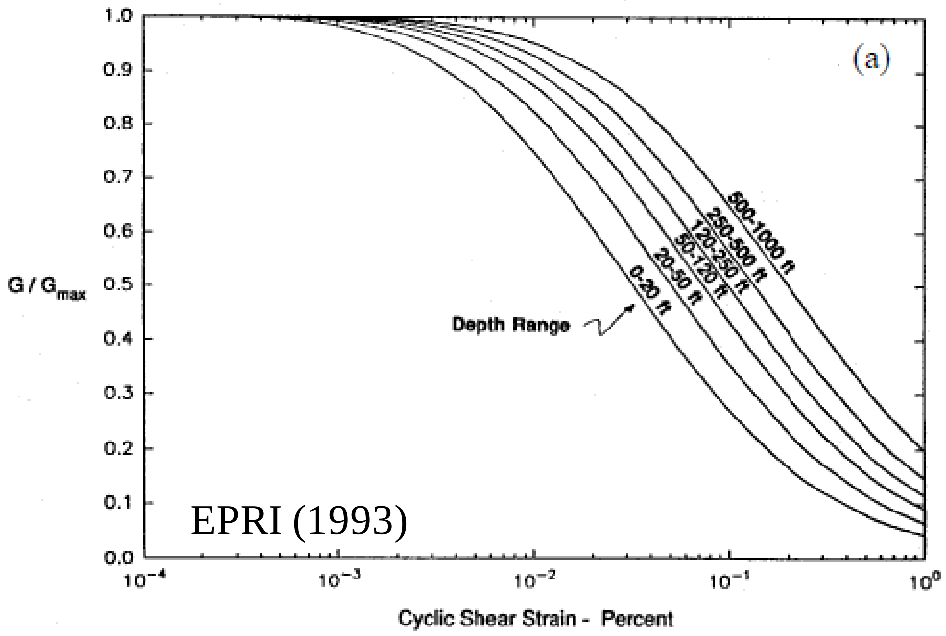
- K_e applies to elastic shear stiffness of each layer
- Example shows K_p =1, K_e=2.0 (stiffness scaling only)



Pressure sensitivity of shear response

Pressure sensitivity – influence on backbone curve

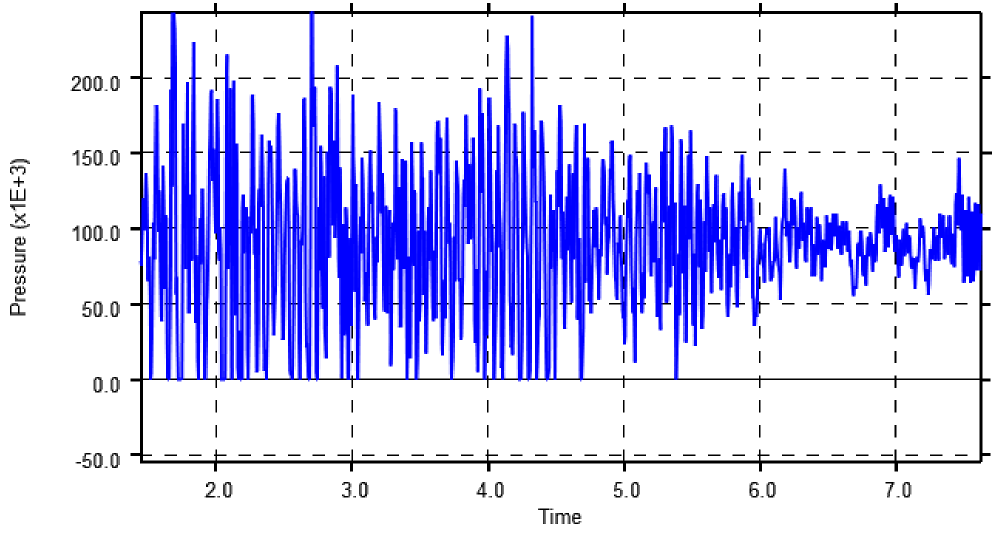
- The scaling of the backbone curve along the strain axis caused by the pressure-sensitivity algorithms is not unrealistic in principle – measured data also shows a shift of response along the strain axis with increasing pressure.



Pressure sensitivity of shear response

Watch-it: dynamically-varying properties

- During an earthquake analysis the pressure in any one element may oscillate rapidly, due to both realistic dynamic effects and also high-frequency element vibration.
- *MAT_079 responds to the instantaneous pressure, which includes the high-frequency effects. Thus, when pressure sensitivity is modelled, the stiffness and strength can also oscillate dynamically.
  - Letting the shear properties vary dynamically might not lead to more accurate solution if the pressure is noisy
  - Dynamically-varying properties can add to the noise problem (positive feedback effect). Dynamically-varying stiffness (K_e, input parameter B) is particularly prone to do this.



Element pressure time-histories can be very noisy.

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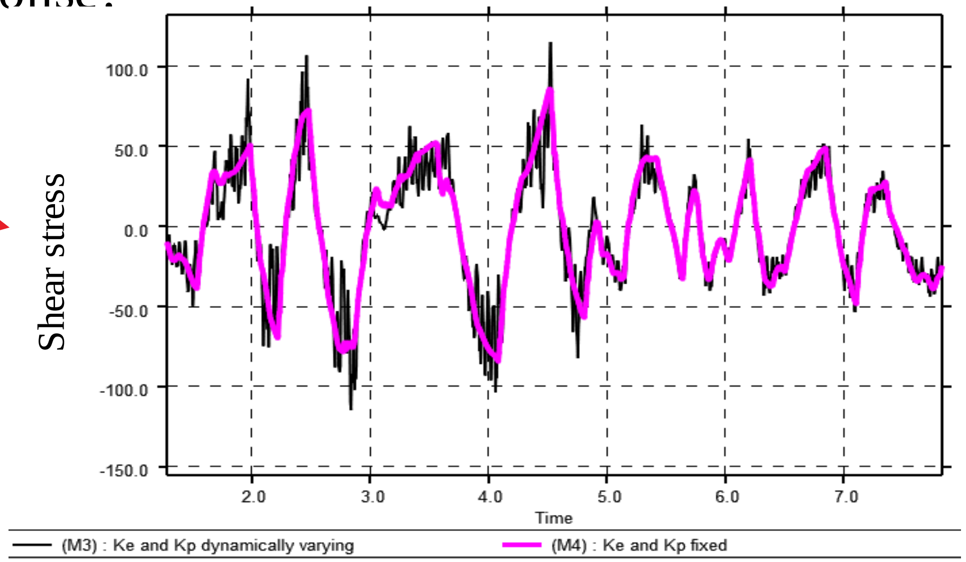
Pressure sensitivity of shear response

Watch-it: dynamically-varying properties

- General recommendations:
  - Dynamically-varying K_p (e.g. input field A2) is generally safe to use.
  - Take steps to damp the high-frequency pressure response (see later slide on *DAMPING)
  - Avoid dynamically-varying K_e (input field B), unless workarounds on next slides are used.
  - Check some element time-histories (pressure, shear stress). Is the high-frequency noise small compared to the underlying response?

Soil column model with horizontal and vertical earthquake input.
Pressure sensitivity:
A2=1, B=0.5



This example is relatively benign – only little additional noise caused by dynamically-varying pressure sensitive shear behavior. Full 3D models may be less forgiving with B>0.

Pressure sensitivity of shear response

Reasons for modeling pressure-sensitivity

- Before describing work-arounds, first consider the reasons for wishing to specify pressure-sensitive properties:
  - As a way of setting material properties as a function of depth, based on the free-field stress state arising from **self-weight of the soil;**
  - Soil may be stiffer/stronger locally due to **static loading from buildings, dams, etc;**
  - Stiffness and strength may change dynamically due to **rocking or bouncing of buildings, dams, etc.**

Work-around 1 or 2

Work-around 3

No work-around

Pressure sensitivity of shear response

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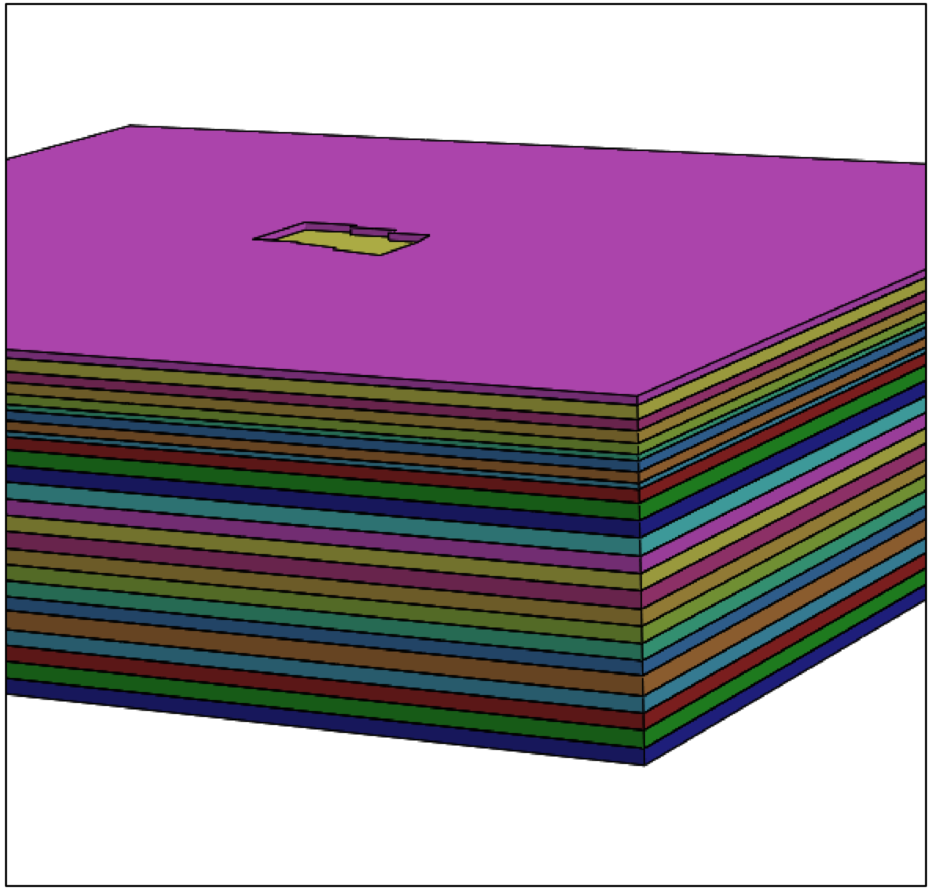
Work-around 1 - layered model

- This method allows only for **self-weight of the soil** on flat sites – not for additional loads from buildings etc.
- The soil is modelled in many thin layers.
- Each layer has shear stiffness and strength properties calculated by the user from the assumed stress state. Generally, these increase with depth.
- The properties do not vary with pressure: A0=1, A1=A2=B=0.
- Advantages are:
  - The depth-dependent material properties are explicitly stated in the input data for each layer and easier to check.
  - Initial stress does not need to be applied.
- Disadvantage: cannot easily model strengthening effect of weight of buildings, dams etc or non-horizontal sites



Pressure sensitivity of shear response

Work-around 2 - PINIT

- This method allows only for **self-weight of the soil** on flat sites – not for additional loads from buildings etc.
- Input field PINIT tells LS-DYNA to apply the pressure sensitivity (K_e and K_p) according the the pressure distribution that exists <u>at the first timestep</u>, but then to freeze K_e and K_p for the rest of the analysis.
- Negative value of PINIT means freeze only K_e at the first timestep, leaving K_p able to change dynamically: **recommended**.
- Free-field stress state can be generated at the first timestep with *INITIAL_STRESS_DEPTH, i.e. stress is a function of z-coordinate.
- Advantage: no need to create multiple materials and parts
- Disadvantage: ignores strengthening effect of weight of buildings, dams etc or non-horizontal sites

Card 1	1	2	3	4	5	6	7	8
Variable	MID	RO	K0	P0	B	A0	A1	A2
Type	A	F	F	F	F	F	F	F

Card 2	1	2	3	4	5	6	7	8
Variable	DF	RP	LCID	SFLC	DIL_A	DIL_B	DIL_C	DIL_D
Type	F	F	F	F	F	F	F	F

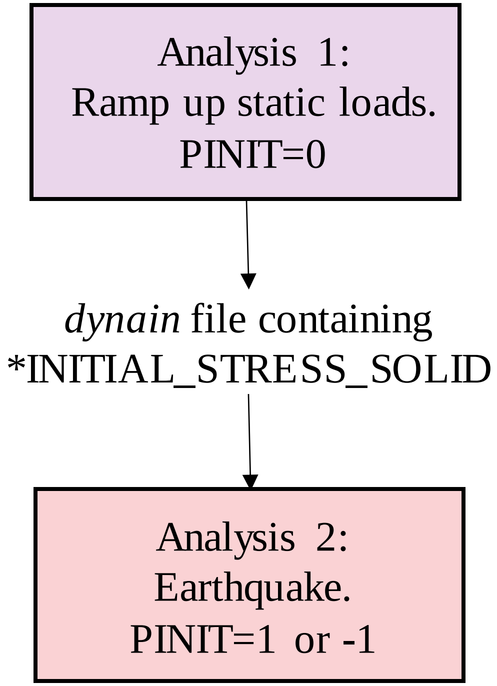
Card 3	1	2	3	4	5	6	7	8
Variable	GAM1	GAM2	GAM3	GAM4	GAM5	LCD	LCSR	$\boxed{\text{PINIT}}$
Type	F	F	F	F	F	I	I	I

Card 4	1	2	3	4	5	6	7	8
Variable	TAU1	TAU2	TAU3	TAU4	TAU5	FLAG5		
Type	F	F	F	F	F			

Pressure sensitivity of shear response

Work-around 3 – PINIT with 2-stage analysis

- This method allows for **self-weight of the soil** together with additional **static loads from buildings, dams**, etc.
- Analysis 1 sets up the gravity loading (pre-earthquake state) and writes a *dynain* file containing *INITIAL_STRESS_SOLID.
  - In Analysis 1, PINIT=0.
  - Heavy damping can be applied while the model settles down.
- Analysis 2 models the earthquake. The dynain file is part of the input.
  - In Analysis 2, PINIT is non-zero (e.g. -1).
  - K_e and/or K_p get frozen at the first timestep according to the stress state carried over from Analysis 1.
- To obtain a *dynain* file, use **Staged Construction** keywords (see later slide) or *INTERFACE_SPRINGBACK_LSDYNA
- In LS-DYNA R14, PINIT can freeze K_e and/or K_p at a user-input time



Advanced capabilites

Advanced capabilities

Strain-rate effects

- Input field LCSR references a curve.
- Rate effect acts on the strength of the material, i.e. The yield stress of each layer. No influence on elastic behavior or stiffness.
- Visco-plasticity algorithm to reduce noise: rate enhancement factor is calculated separately for each layer, based on plastic strain rate.
  - Note: strain rate effects are commonly used in SRA analyses. If instabilities arise in SSI analysis may consider turning off this feature and ”locking in” a higher shear strength via the backbone curve.

Card 1	1	2	3	4	5	6	7	8
Variable	MID	R0	K0	P0	B	A0	A1	A2
Type	A	F	F	F	F	F	F	F

Card 2	1	2	3	4	5	6	7	8
Variable	DF	RP	LCID	SFLC	DIL_A	DIL_B	DIL_C	DIL_D
Type	F	F	F	F	F	F	F	F

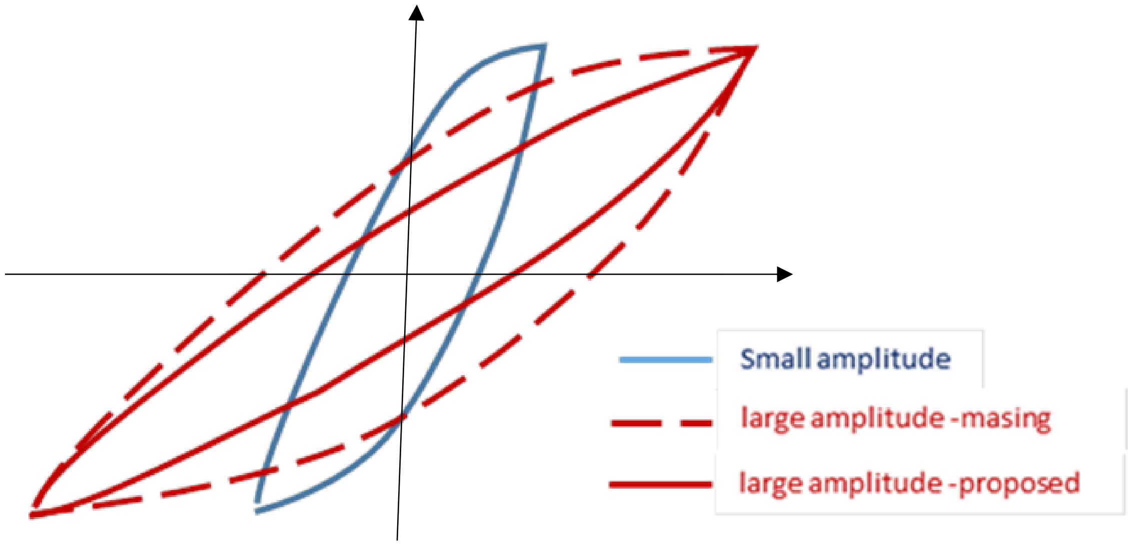
Card 3	1	2	3	4	5	6	7	8
Variable	GAM1	GAM2	GAM3	GAM4	GAM5	LCD	$\boxed{\text{LCSR}}$	PINIT
Type	F	F	F	F	F	I	I	I

Card 4	1	2	3	4	5	6	7	8
Variable	TAU1	TAU2	TAU3	TAU4	TAU5	FLAG5		
Type	F	F	F	F	F			

Advanced capabilities

Non-Masing damping

- Masing damping can overestimate the actual hysteretic damping, especially at high strain amplitudes. This feature is intended to reduce the material damping.



Card 1	1	2	3	4	5	6	7	8
Variable	MID	R0	K0	P0	B	A0	A1	A2
Type	A	F	F	F	F	F	F	F

Card 2	1	2	3	4	5	6	7	8
Variable	DF	RP	LCID	SFLC	DIL_A	DIL_B	DIL_C	DIL_D
Type	F	F	F	F	F	F	F	F

Card 3	1	2	3	4	5	6	7	8
Variable	GAM1	GAM2	GAM3	GAM4	GAM5	LCD	LCSR	PINIT
Type	F	F	F	F	F	I	I	I

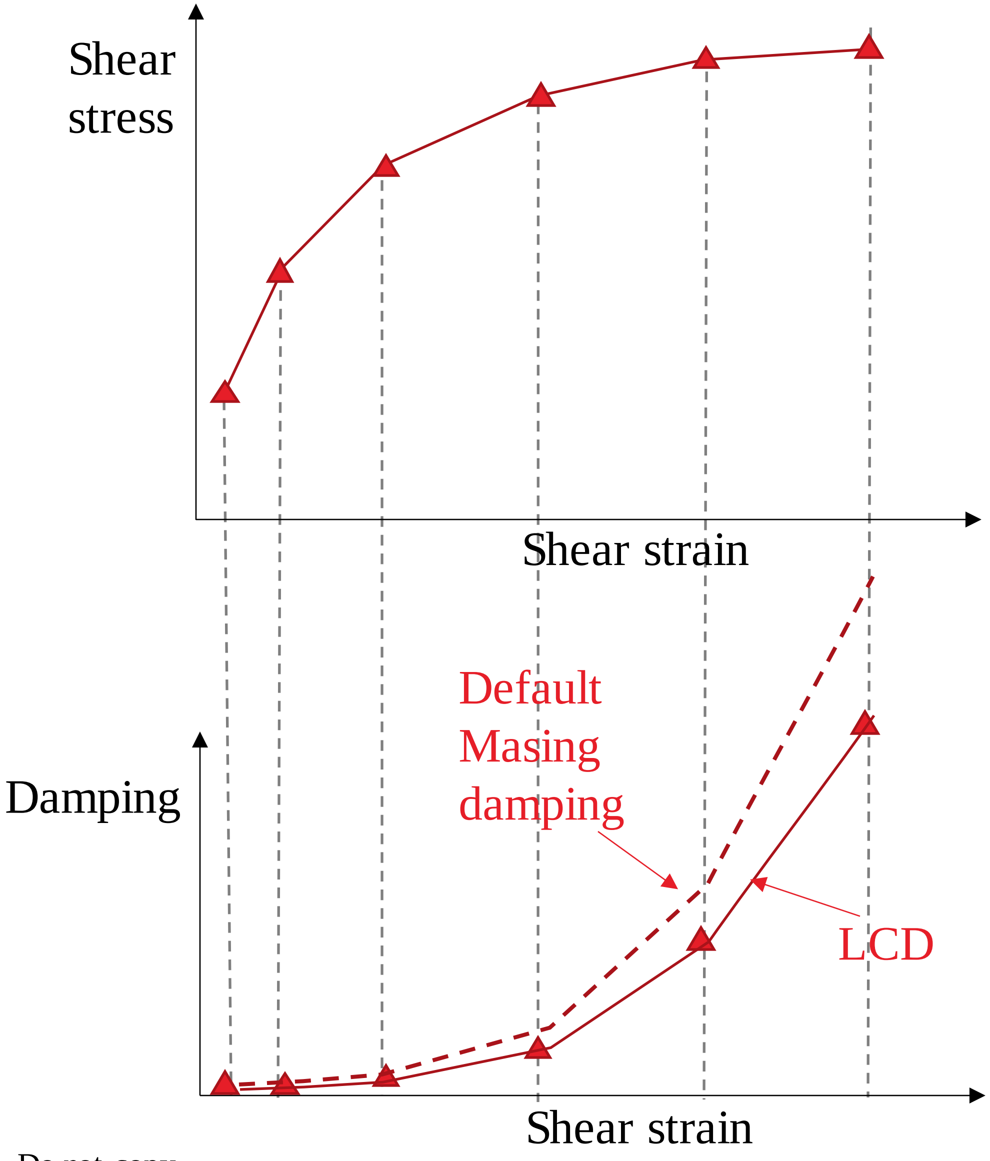
Card 4	1	2	3	4	5	6	7	8
Variable	TAU1	TAU2	TAU3	TAU4	TAU5	FLAG5		
Type	F	F	F	F	F			

Advanced capabilities

Non-Masing damping

- LCD references a curve. X-axis is equivalent shear strain. Must have same x-axis points as LCID. Y-axis is damping ratio.
- It can only reduce damping compared to Masing, not increase it.
- The damping difference (Masing – LCD) must increase monotonically with increasing strain.
- If these rules are not followed, data is output in Messag/mes0000 file

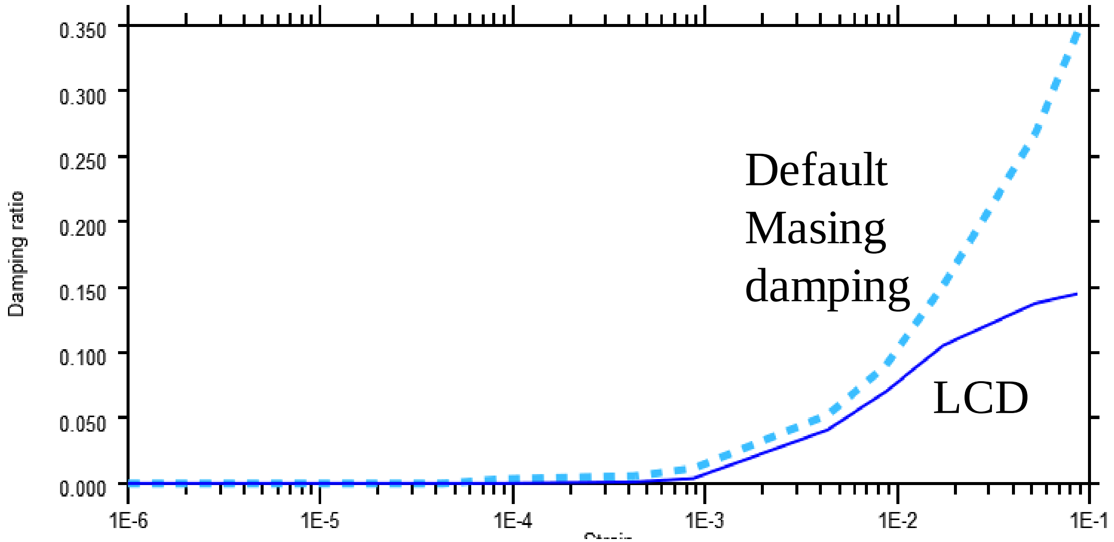
strain	Masing Damp	LCD damp	Difference
8.70000E-07	2.71795E-16	0.00000E+00	2.71795E-16
4.35000E-05	1.29144E-04	3.28780E-05	9.62663E-05
8.70000E-05	3.17420E-03	1.00760E-04	3.07344E-03
4.35000E-04	9.77809E-03	1.34730E-03	8.43079E-03
8.70000E-04	2.98717E-02	4.05580E-03	2.58159E-02
[illegible]	[illegible]	[illegible]	[illegible]

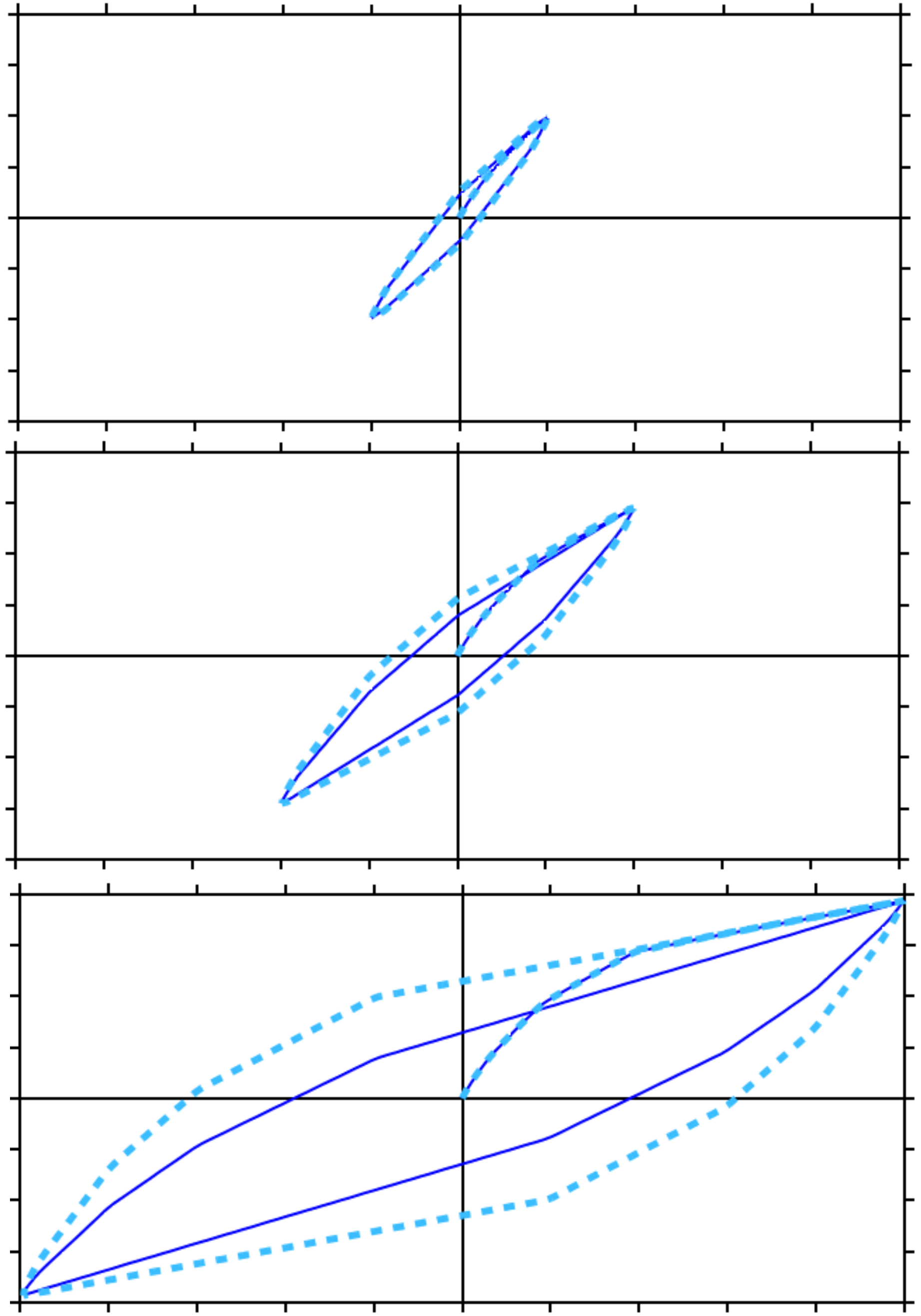


Advanced capabilities

Non-Masing damping

- Example:





- It works by scaling down the stiffness of the layers in response to high-tide shear strain.

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Advanced capabilities

Cyclic degradation

- This feature models the strain-induced cyclic degradation (or *destructuration*) shown by some clay soils.
- Uses the principal of ”damage strain” to reduce the size of yield surfaces, similar to references like Baudet & Stallebrass, 2004 and Rouania & Wood, 2000
- Damage is driven by shear strains occurring while the shear stress is above a threshold value.
- Damage affects strength, i.e. The yield stress of all layers.
- See User Manual for details.

Card 1	1	2	3	4	5	6	7	8
Variable	MID	R0	K0	P0	B	A0	A1	A2
Type	A	F	F	F	F	F	F	F

Card 2	1	2	3	4	5	6	7	8
Variable	DF	RP	LCID	SFLC	DIL_A	DIL_B	DIL_C	DIL_D
Type	F	F	F	F	F	F	F	F

Card 3	1	2	3	4	5	6	7	8
Variable	GAM1	GAM2	GAM3	GAM4	GAM5	LCD	LCSR	PINIT
Type	F	F	F	F	F	I	I	I

Card 4	1	2	3	4	5	6	7	8
Variable	TAU1	TAU2	TAU3	TAU4	TAU5	FLAG5		
Type	F	F	F	F	F			

This card is included if FLAG5 = 1.

Card 5	1	2	3	4	5	6	7	8
Variable	SIGTH	SIGR	CHI					
Type	F	F	F					

Modeling advice

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Modeling advice

Element formulation etc

- ELFORM=1 is recommended for MAT_079
- Pressure-sensitive materials do not work well with ELFORM=2
  - Pressure-smearing across integration points produces wrong results
- Hourglass control (*HOURGLASS): advice not specific to MAT_079
  - Stiffness-method (IHQ=4) often works better, especially for quasi-static applications
  - When using stiffness-method, reduce factor QM below its default value, e.g. to 0.01
  - Check that results are insensitive to further reduction of QM, i.e. not stiffening the response
  - Check d3plot results with magnified deformation, is hourglass deformation excessive?
  - Check hourglass energy in time-history results
- With pressure-sensitive soil materials, the top layer of elements is especially prone to misbehave during the earthquake (large unrealistic deformations when pressure drops to zero). We can increase P0 or replace the top layer with a crust of cohesive material.

Modeling advice

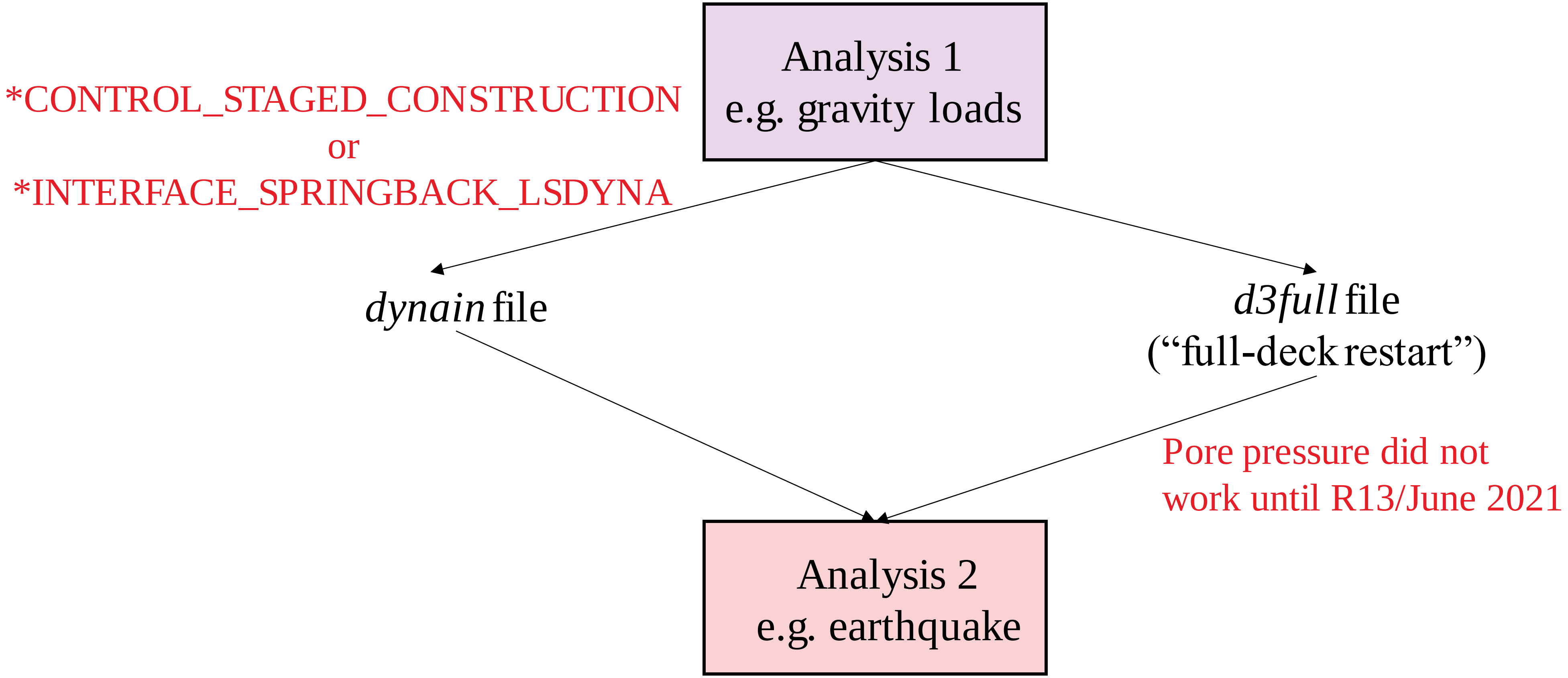
Element formulation etc (cont)

Damping:

- *DAMPING_GLOBAL
  - For low-frequency and rigid body motion, e.g. for settling under gravity
  - Do NOT apply during earthquake
- *DAMPING_FREQUENCY_RANGE_DEFORM
  - For real physical modes of vibration (does not damp rigid body modes).
  - Compensates for numerical model's lack of damping at small strains: typically 1-2%.
  - Also offers some benefit for reducing numerical noise.
- *DAMPING_PART_STIFFNESS
  - For high-frequency (element-level) vibration, i.e. numerically-driven noise.
  - Recommended when MAT_079 properties are pressure-sensitive (especially for B>0).

Multi-stage modeling

Methods



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Multi-stage modeling

Methods

All-in-one analysis e.g. gravity load then earthquake

*CONTROL_STAGED_CONSTRUCTION or
*INTERFACE_SPRINGBACK_LSDYNA

Analysis 1
e.g. gravity loads

dynain file

d3full file
(“full-deck restart”)
Pore pressure did not
work until R13/June 2021

drdisp file
Dynamic relaxation

Analysis 2
e.g. earthquake

Multi-stage modeling

Staged construction

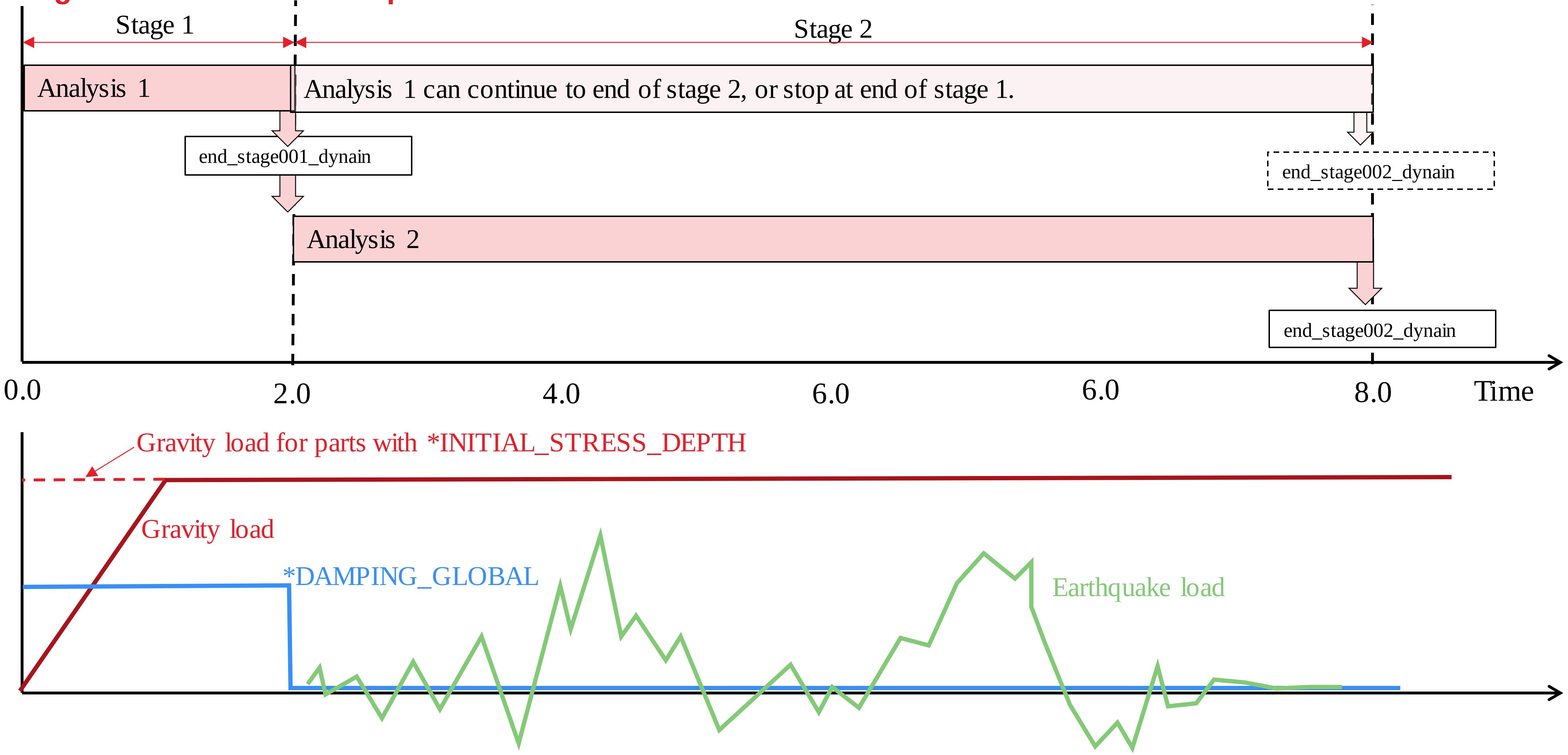
Functions of Staged Construction keywords:

• Define *stages* by start time and end time	*DEFINE_CONSTRUCTION_STAGES
• Define which stages will be run in this analysis (e.g. we could start from Stage 2)	*CONTROL_STAGED_CONSTRUCTION
• Define which parts are present during which stages (e.g. A part can be added at Stage 2; another part can be deleted at Stage 3)	*DEFINE_STAGED_CONSTRUCTION_PART
• Apply gravity loading to parts when present; ramp up loading smoothly when parts are added.	(included in the above)
• Write out *dynain* file at end of each stage	(included in the above)

See also: https://ftp.lstc.com/anonymous/outgoing/support/PRESENTATIONS/Staged_construction_notes_03apr2018.pdf

Multi-stage modeling

Staged construction example



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Multi-stage modeling

Staged construction example: keyword input file contents

Analysis 1:

*CONTROL_STAGED_CONSTRUCTION (run stage 1)
*CONTROL_...
*DATABASE_...
*MAT_...
*SECTION_...
*PART_...
*DEFINE_CURVE
*DEFINE_CONSTRUCTION_STAGES
*DEFINE_STAGED_CONSTRUCTION_PART
*LOAD_...

*NODE
*ELEMENT_...
*BOUNDARY_SPC
*INITIAL_STRESS_DEPTH

Copy

Analysis 2:

*CONTROL_STAGED_CONSTRUCTION (run stage 2)
*CONTROL_...
*DATABASE_...
*MAT_...
*SECTION_...
*PART_...
*DEFINE_CURVE
*DEFINE_CONSTRUCTION_STAGES
*DEFINE_STAGED_CONSTRUCTION_PART
*LOAD_...

*INCLUDE
end_stage001_dynain

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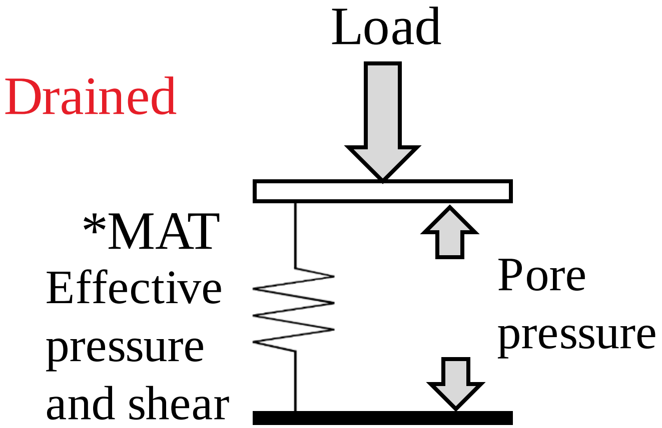
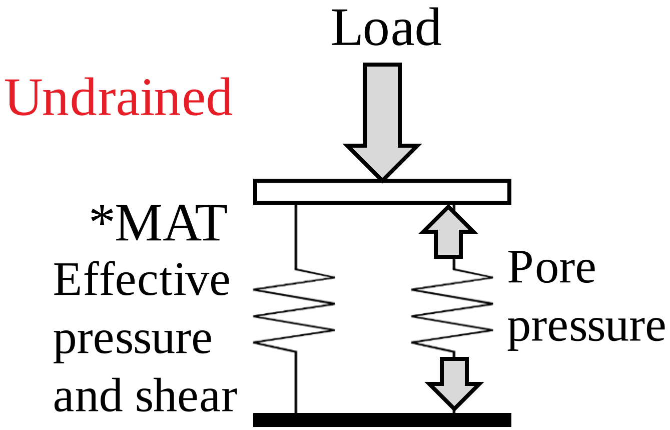
Pore pressure modeling

Overview

- Terzaghi's concept of effective stress:
  - Total stress = effective stress + pore pressure
  - Pore pressure "overlay" acts in parallel with soil skeleton: agnostic to (and separate from) the material model
  - Effective stress = soil skeleton behavior = *MAT cards
  - E.g. MAT_079 friction relates to effective pressure not total pressure
- Pore pressure "analysis types":
  - **Drained**: pore pressure is set by the user as a function of depth. Pore pressure not sensitive to volume change.
  - **Undrained**: pore water is locked within each element, adds volumetric stiffness
  - **Time-dependent consolidation**: similar to undrained, plus the pore fluid flows through mesh according to Darcy's law.

Set by user per Part





Pore pressure modeling

Overview

- Time-dependent consolidation:

$$\mathbf{v} = \kappa \nabla (p + z)$$

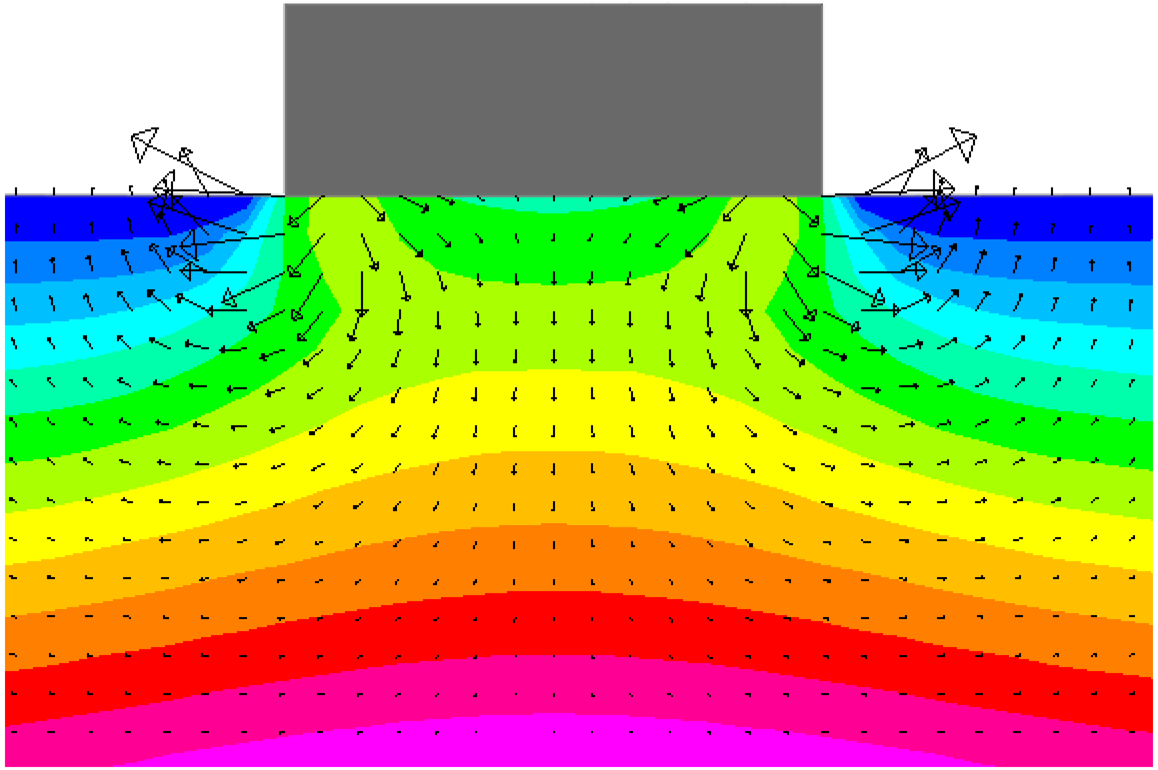
$\mathbf{v}$ = Seepage velocity = volume flow rate/area
κ = permeability
p = pressure head = pressure/ρg
z = z-coordinate.

- Output of pore pressure and effective stress
  - D3plot
  - D3thdt

  Stresses: see OUTPUT on *CONTROL_PORE_FLUID
  Pore pressure: Extra Variable NEIPH+1

  - In R14, also in elout/binout provided that OPTION1 is set on *DATABASE_ELOUT
  - See also *DATABASE_PWP_OUTPUT

Colours: pore pressure
Arrows: seepage velocity



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Pore pressure modeling

Input (keyword file)

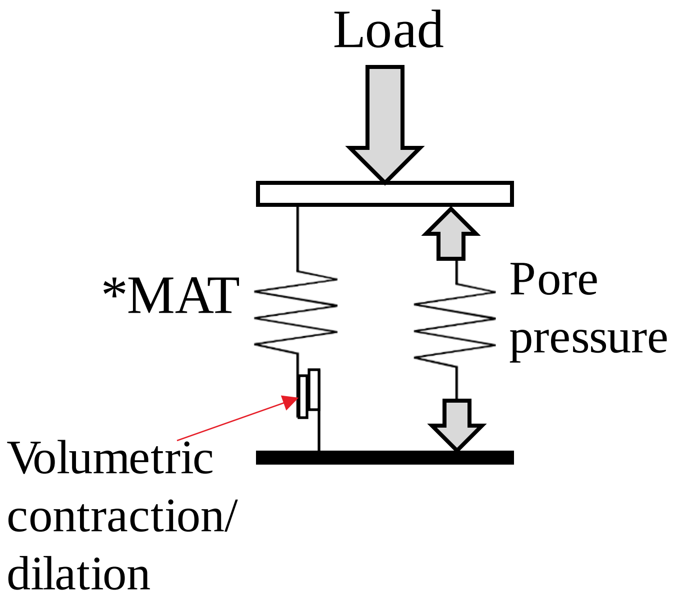
- *CONTROL_PORE_FLUID
  - General inputs, pore fluid bulk modulus and density…
- *BOUNDARY_PORE_FLUID
  - Per-part analysis type (drained/undrained/TDC), water table level, suction limit…
- *MAT_ADD_PERMEABILITY
- *BOUNDARY_PWP
  - Pore pressure versus time for a node set. Influences pore pressure in the attached elements but does not apply external loads.
- *INITIAL_PWP_DEPTH
  - Initial PWP as a function of depth
- *DATABASE_PWP_OUTPUT
  - Determines contents of d3plot/d3thdt files e.g. seepage velocity in place of particle velocity

Dynain file:
*INITIAL_PWP_NODAL_DATA
- node-by-node pore pressure data

Pore pressure modeling

Application to earthquake modeling

- Pore pressure generation in sands requires a material model that exhibits realistic volumetric contraction/dilation of the soil skeleton during shearing
  - *MAT_079 has DIL_A through DIL_D but too simplistic
  - *MAT_SOIL_SANISAND (developmental)
- LS-DYNA pore pressure keyword cards take care of the pore pressure itself
- Time-dependent consolidation enables pore pressure dissipation during and after the earthquake.
- Automatic time-scaling of pore water seepage enables long duration post-earthquake dissipation to be modelled.



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LS-DYNA *MAT_HYSTERETIC_SOIL (*MAT_079)

Session 2 - Validation and benchmarks

Richard Sturt, Kirk Ellison, Ben Shao

May 2022

BC hydro

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Validation and Benchmarks – Single Element Tests

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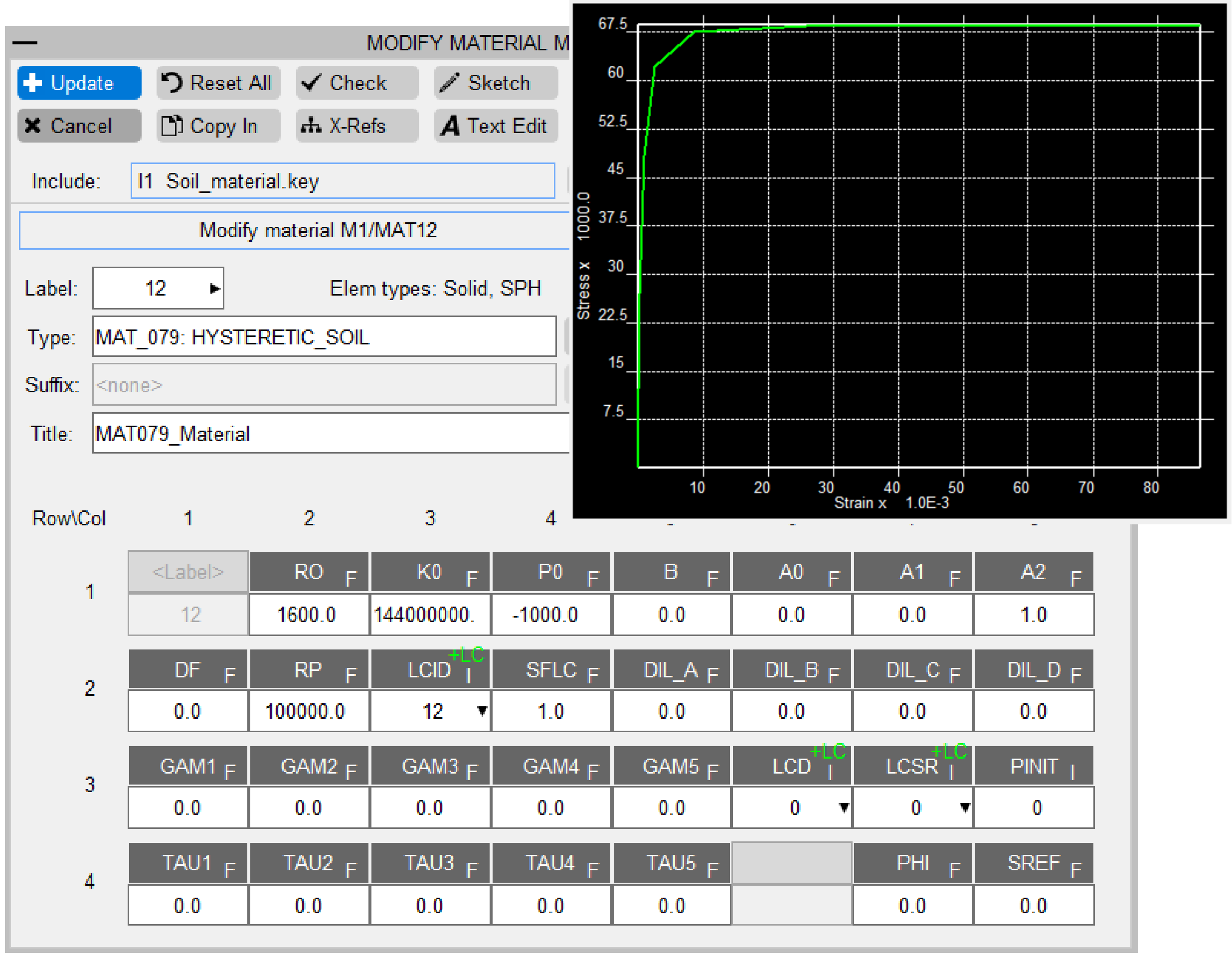
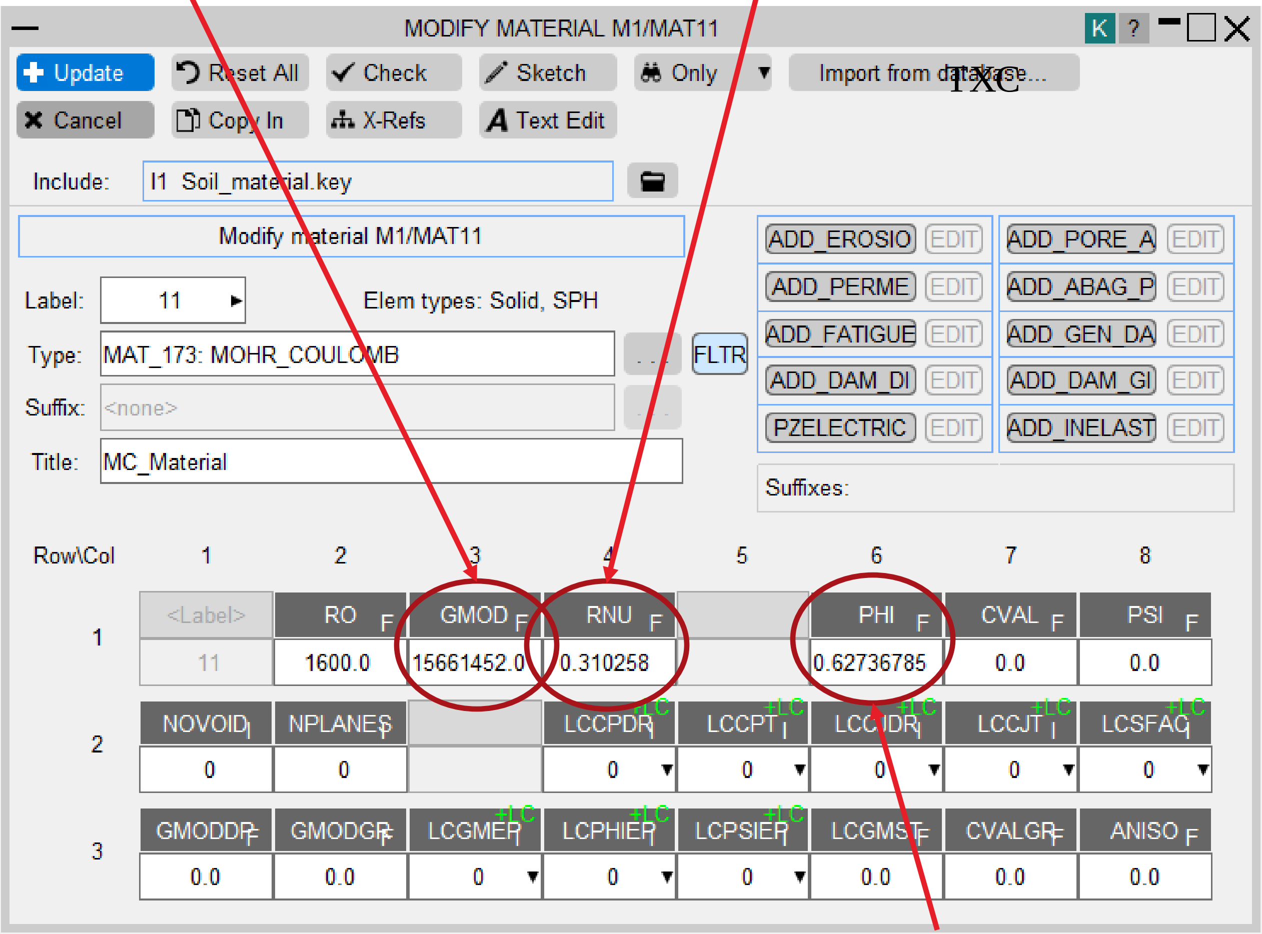
Validation and Benchmarks – Single Element Tests

Parameters for Example Stress Path Tests

Selected to be 4xG0 from MHS

Selected to match small strain v from MHS





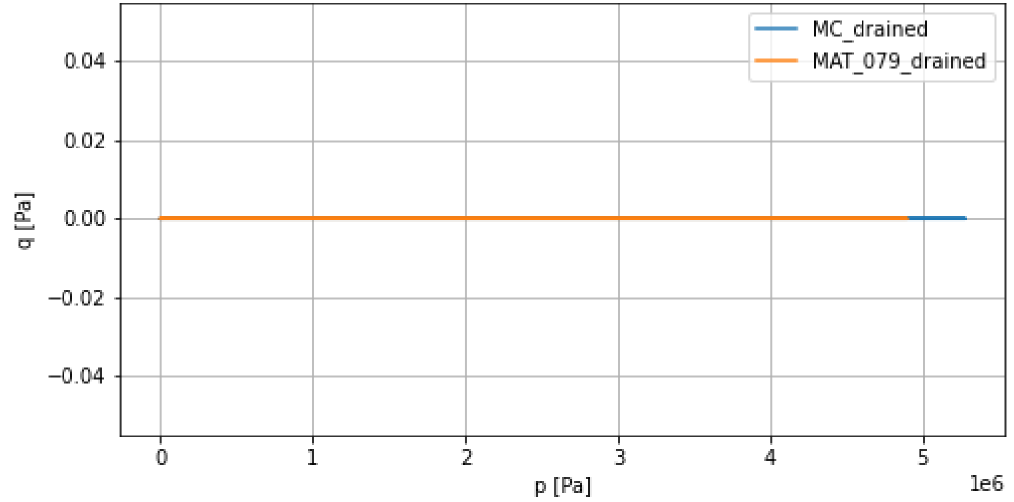
Selected to approximately match MHS strength for TXC

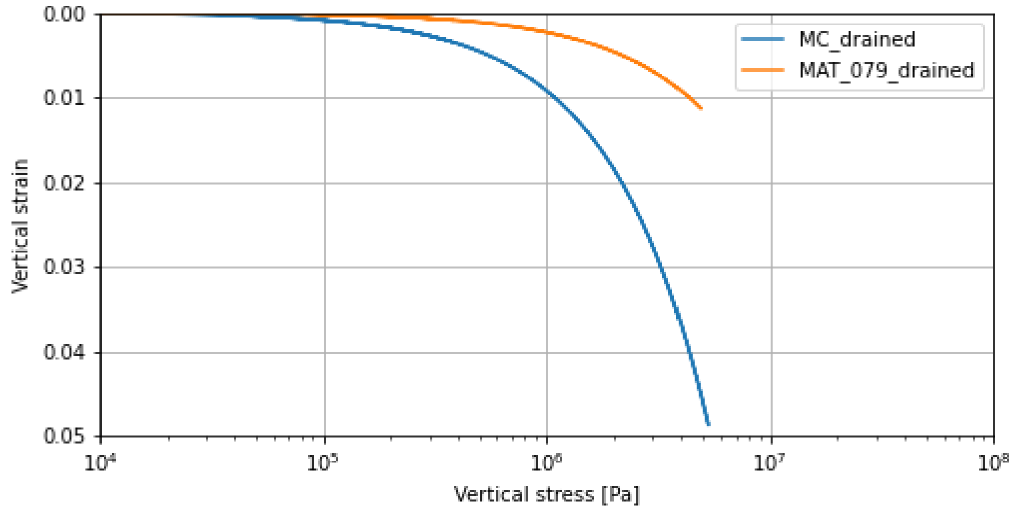
Validation and Benchmarks – Single Element Tests

Isotropic Consolidation

Loading Sequence

Point	X value	Y value
1	0.0	-1000.0
2	5.0	-1000.0
3	55.0	-383040.0
4	105.0	-47880.0
5	155.0	-4788000.0
6	205.0	-4788.0



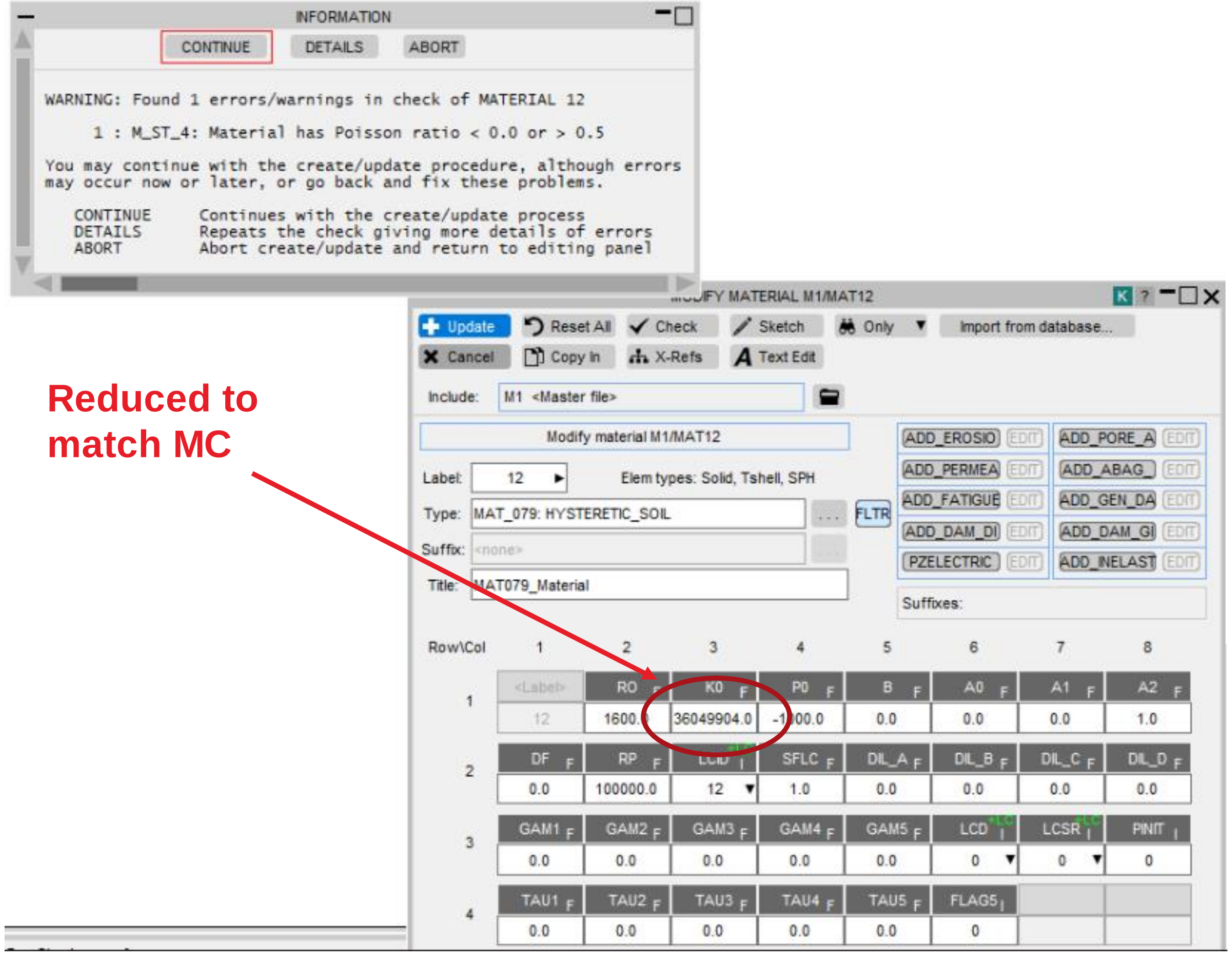


Notes:
- Relatively stiff behavior in MHS with realistic Poisson's Ratio
- No unload-reload loops

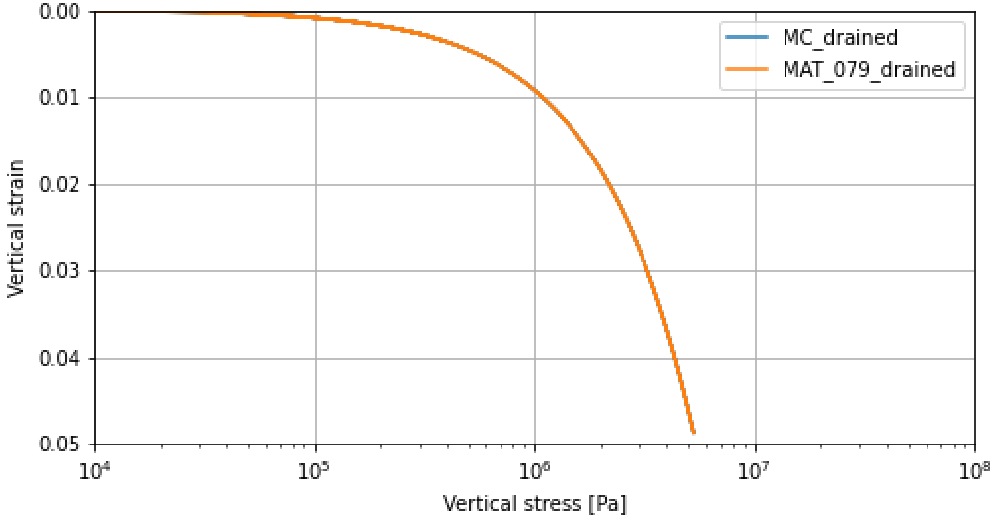
p: average stress
q: max principal stress – min principal stress

Validation and Benchmarks – Single Element Tests

Isotropic Consolidation – Unrealistic Poisson's Ratio



Reduced to match MC



Notes:

- Possible to reduce volumetric stiffness in Mat079, but will result in an unrealistic Poisson's ratio for elastic strains

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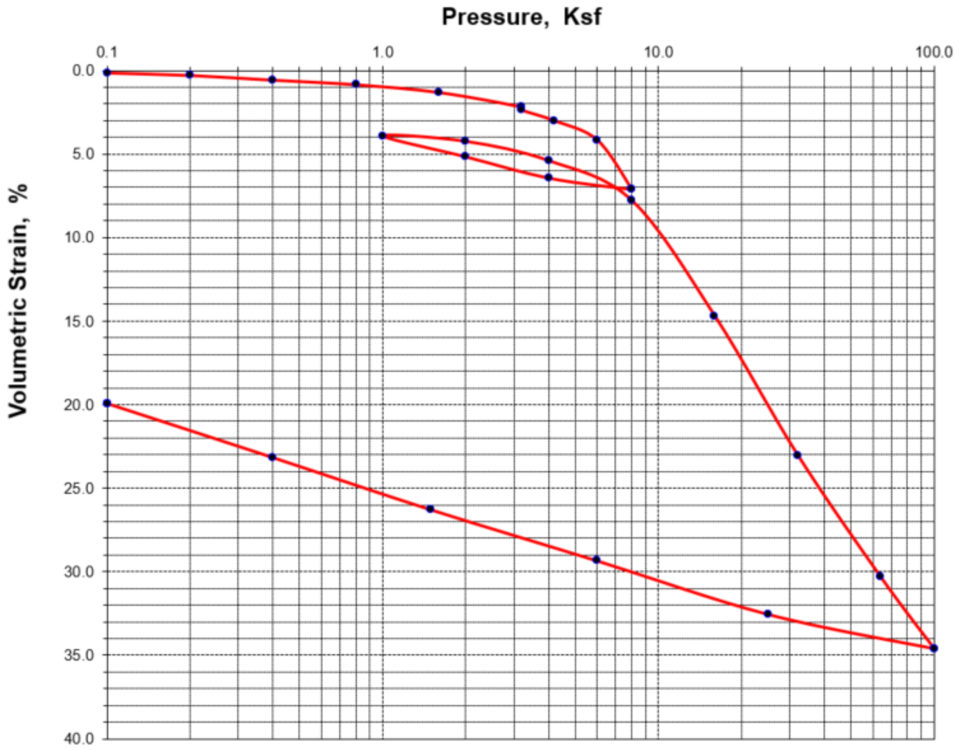
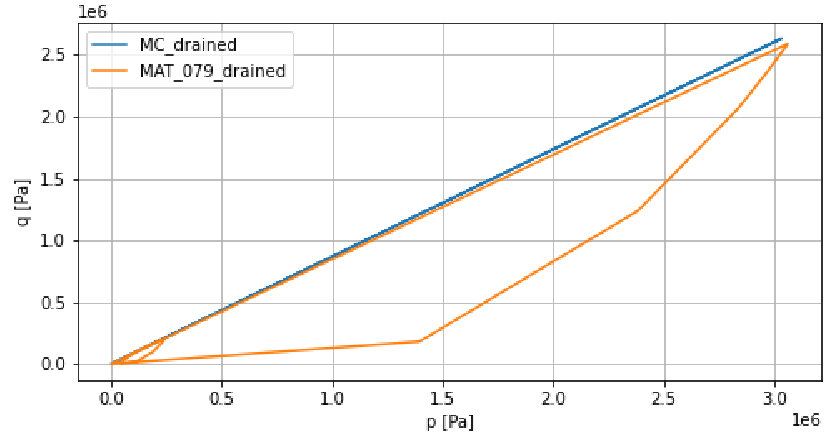
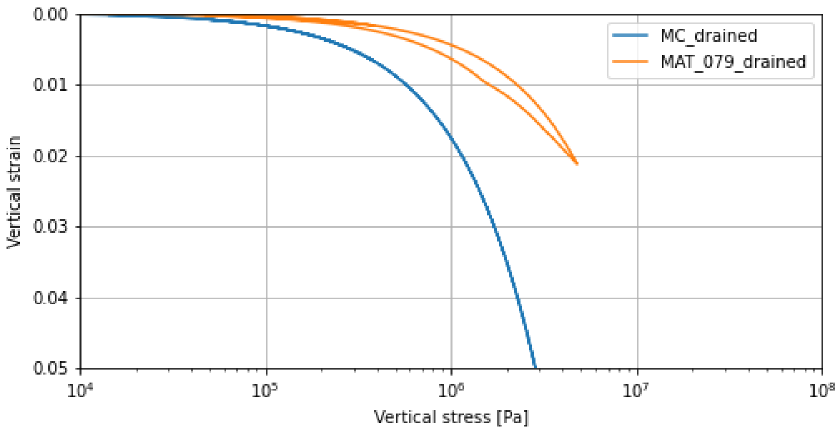
Validation and Benchmarks – Single Element Tests

1D Consolidation

Example Oedometer Data (for reference only)







p: average stress
q: max principal stress – min principal stress

Notes:

- Some difference in unloading due to changes in shear stress in Mat079 but not with Mohr Coulomb
- No concept of overconsolidation vs normal consolidation

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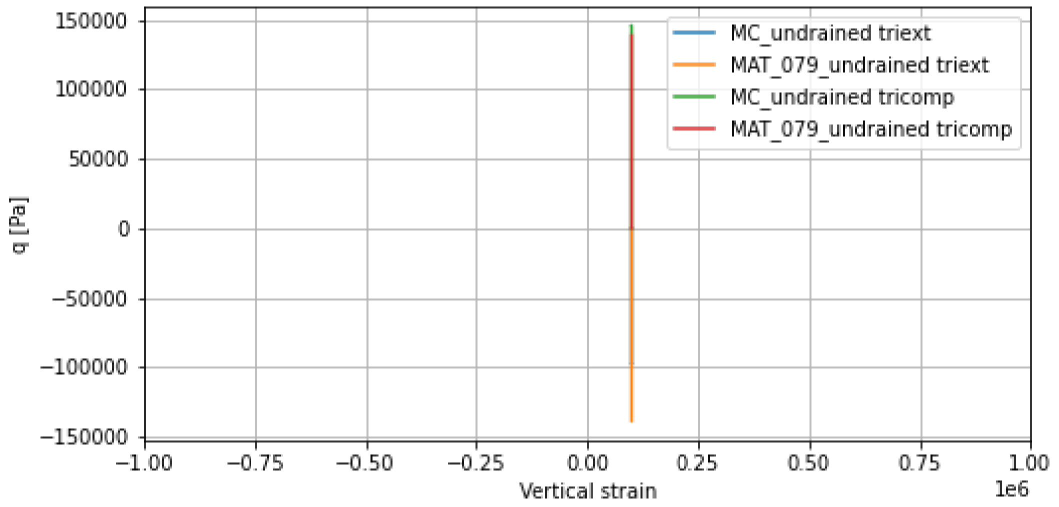
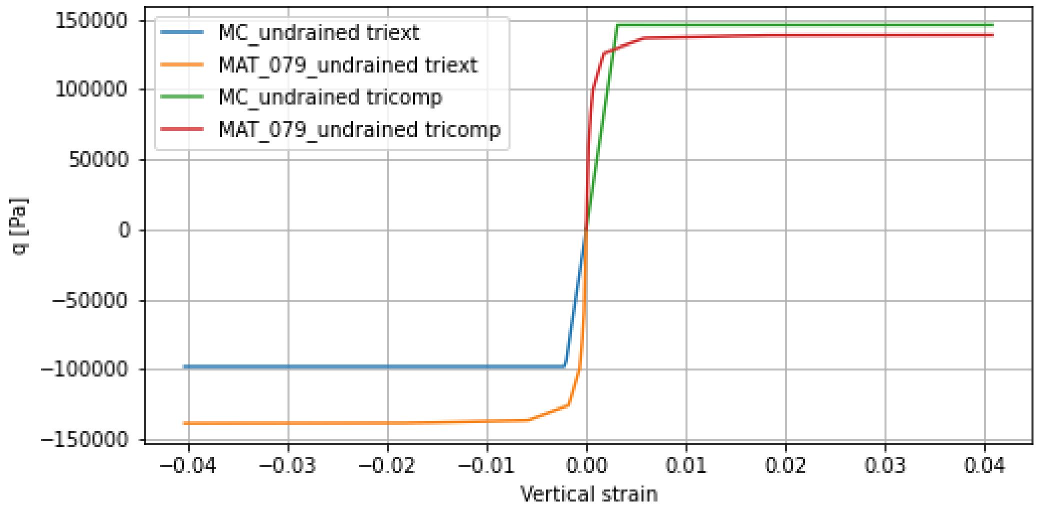
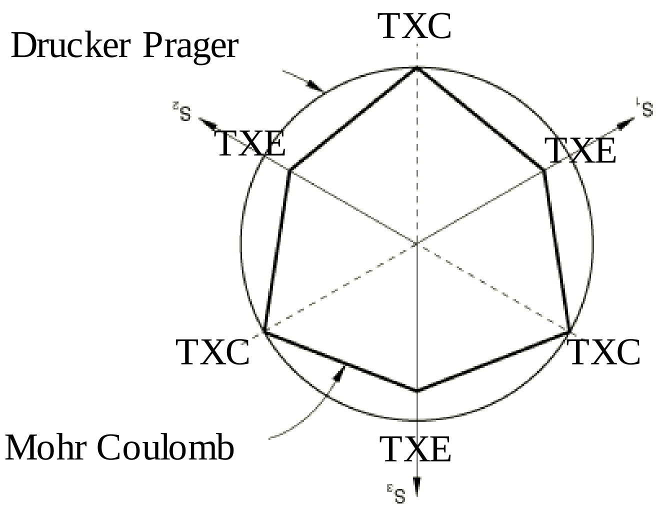
Validation and Benchmarks – Single Element Tests

Undrained Triaxial Compression and Tension







Notes:

- Mat079 has piecewise nonlinear behavior
- Mat079 material stronger in extension relative to MC
- No shear-induced volume change

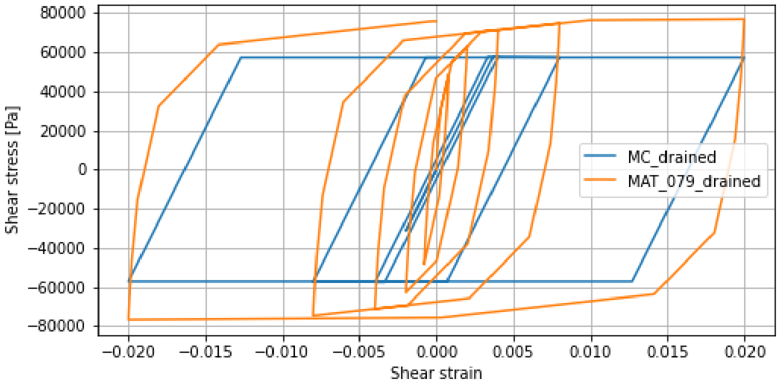
p: average stress
q: vertical stress – horizontal stress

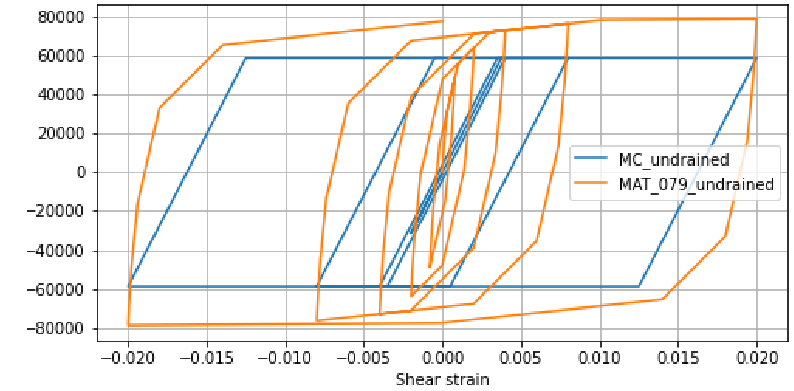
Validation and Benchmarks – Single Element Tests

Cyclic Simple Shear

Drained

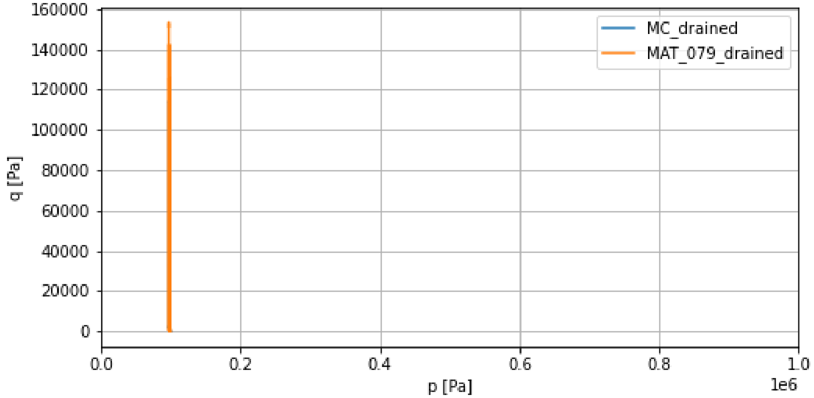
Undrained

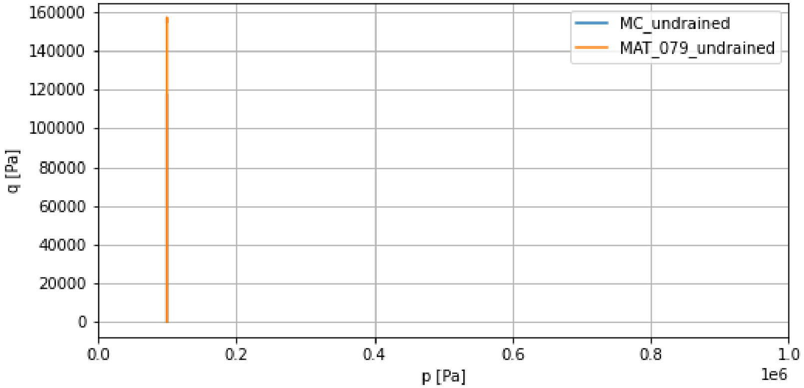




Notes:

- Mat079 has more realistic hysteretic behavior
- No shear-induced volume change
- Drained and undrained behavior is identical





Validation and Benchmarks – Single Element Tests

Key Points

- Beware that volumetric stiffness may be overestimated beyond small strains
- No shear-induced changes in volume or pore pressure (default parameters)
- Higher shear strength in extension relative to Mohr Coulomb
- Best suited for undrained monotonic or cyclic analyses where:
  - Undrained shear strength profile is well-defined
  - Large changes in pore pressure are not anticipated (e.g. no liquefaction)

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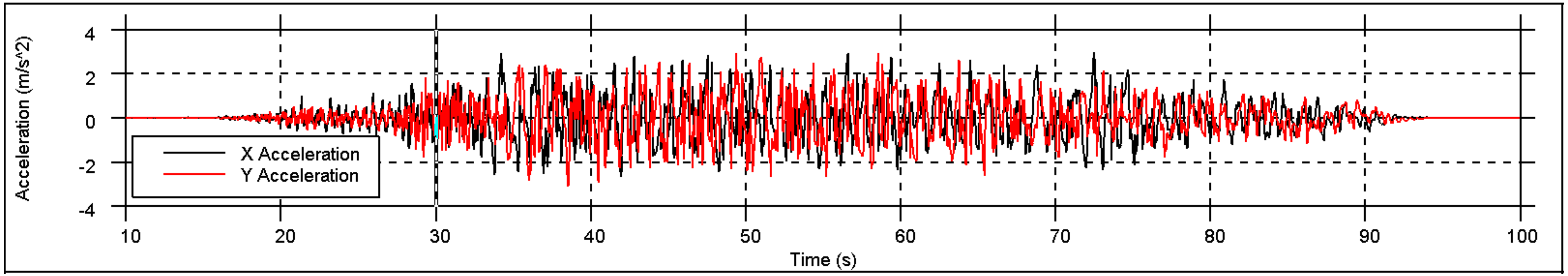
Validation and Benchmarks – Site Response

Validation and Benchmarks – Site Response

Sample SRA



Validation and Benchmarks – Site Response

References

- Bolisetti, C., Whittaker, A. S., Mason, H. B., Almufti, I., & Willford, M. (2014). Equivalent linear and nonlinear site response analysis for design and risk assessment of safety-related nuclear structures. Nuclear Engineering and Design, 275, 107-121.
- Motamed, R., Stanton, K. V., Almufti, I, Ellison, K. C., & Willford, M. (2016), Improved Approach for Modeling Nonlinear Site Response of Highly Strained Soils: Case Study of the Service Hall Array in Japan, Earthquake Spectra. 32(2): 1055-1074.
- Kumar, P., Ellison, K., Paul, N., Lee, J., Almufti, I. & Stanton, K. (2017) “Is there a Basin Effect in Mexico City? Validation of Three Urban Lakebed Sites Using Nonlinear Site Response Analysis Provides the Clue”.
- O’Riordan N.J., Almufti I, Lee J, Ellison K and Motamed R. (2018) Site response analysis for dynamic soil–structure interaction and performance-based design. Proceedings of the Institution of Civil Engineers – Geotechnical Engineering,

Validation and Benchmarks – Site Response

Bolisetti et al. 2014

- Study considers:
  - SHAKE vs DEEPSOIL vs LS-DYNA
  - Uni-directional ground shaking
  - Masing Rule Damping
  - 4 hypothetical ground profiles, i.e.
    - E1: hard rock
    - E2: 28 m of stiff sand over hard rock
    - W1: 100 m of soft rock
    - W2: 100 m of stiff sand
  - 9 ground motions (3 for E1 and E2, 3 ordinary motions for W1 and W2, 3 near-fault pulse motions for W1 and W2)

Validation and Benchmarks – Site Response

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Bolisetti et al. 2014

Generally demonstrates a good match between DEEPSOIL and LS-DYNA….

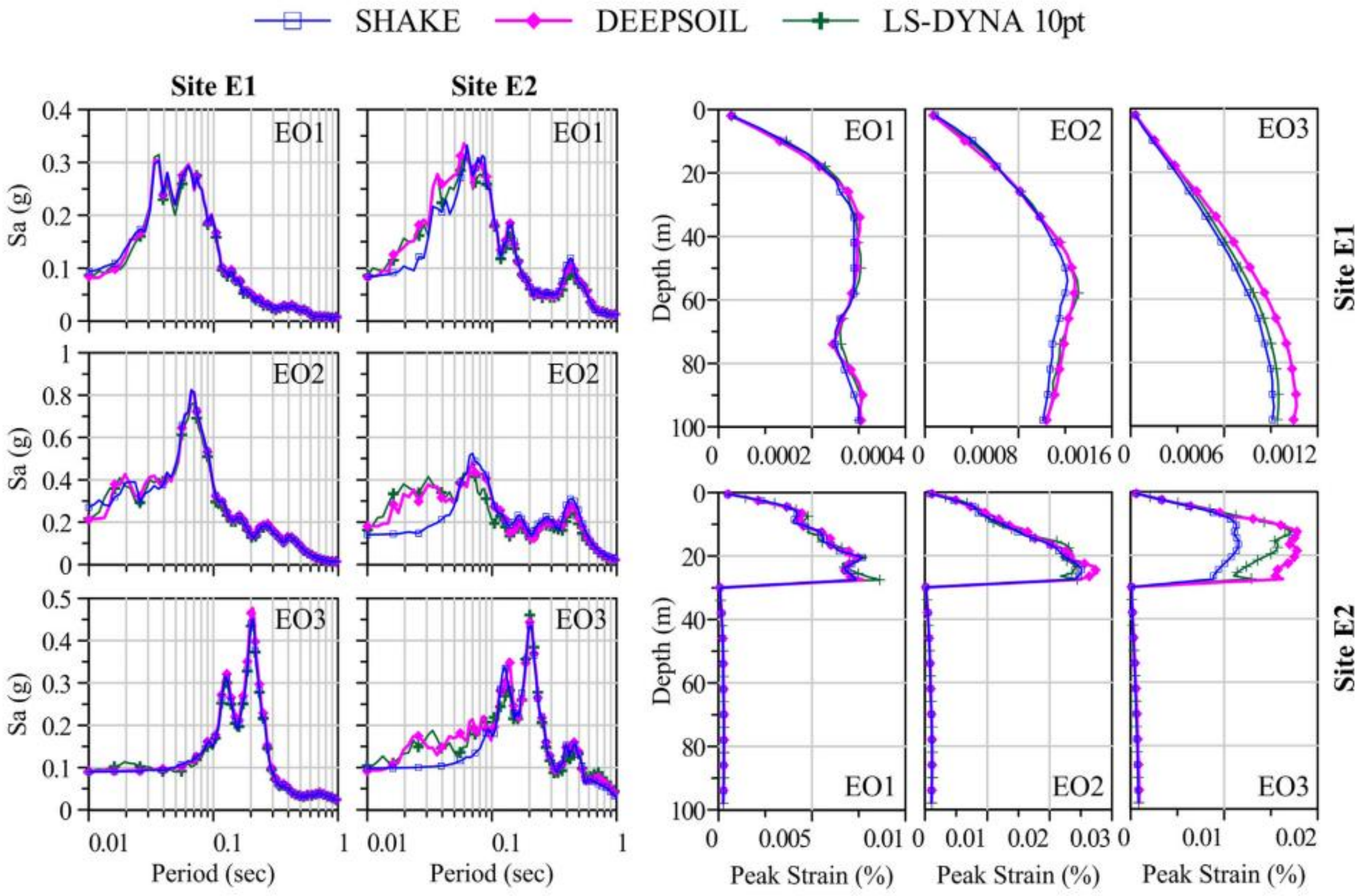


Fig. 4. Acceleration response spectra (left) and peak strain profiles (right) for sites E1 and E2.

Validation and Benchmarks – Site Response

Bolisetti et al. 2014

However, high frequency noise was observed for a few of the motions/profiles that was addressed by refining the backbone curve with coincident elements

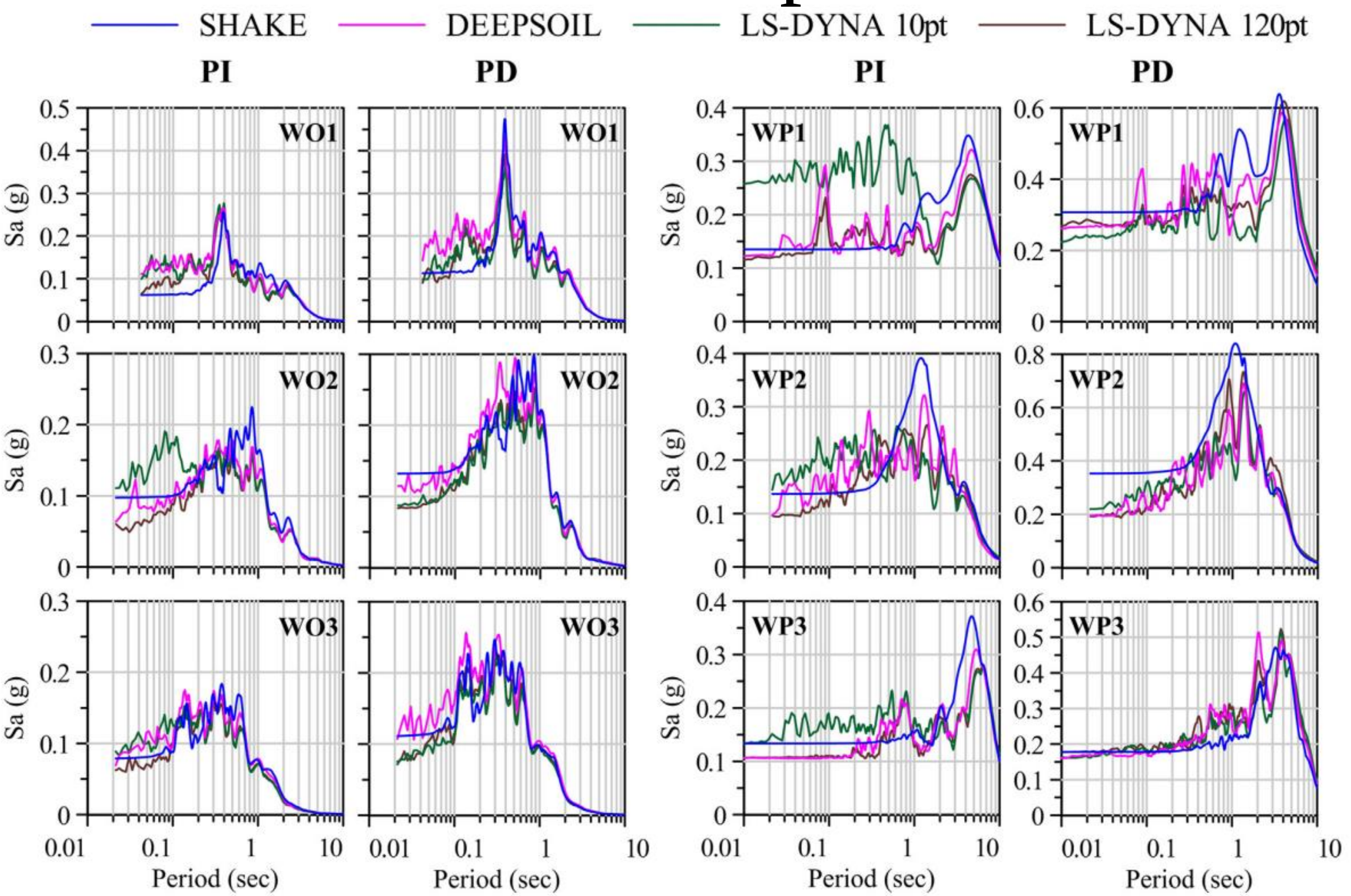


Fig. 8. Acceleration response spectra at the surface of site W2 (PI and PD indicate pressure-dependent and pressure-independent properties, respectively).

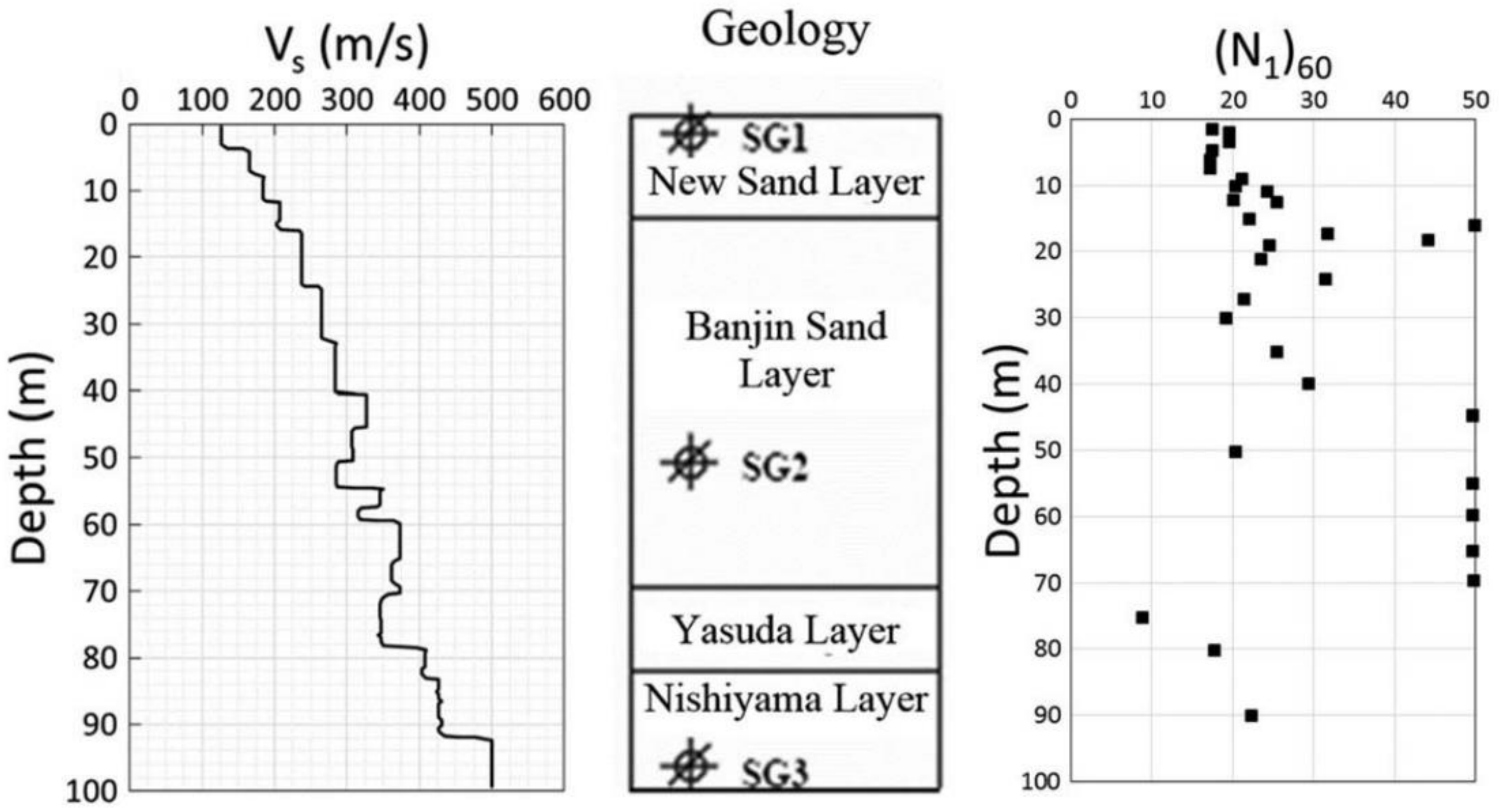
Validation and Benchmarks – Site Response

Motamed et al. 2016

- Study Considers:
  - Back analysis of a downhole array recording
  - Masing Damping
  - Importance of strength adjustment for backbone curves
  - Sensitivity studies for:
    - Strain rate effects
    - Uni-directional vs Bi-Directional Shaking

Validation and Benchmarks – Site Response

Motamed et al. 2016

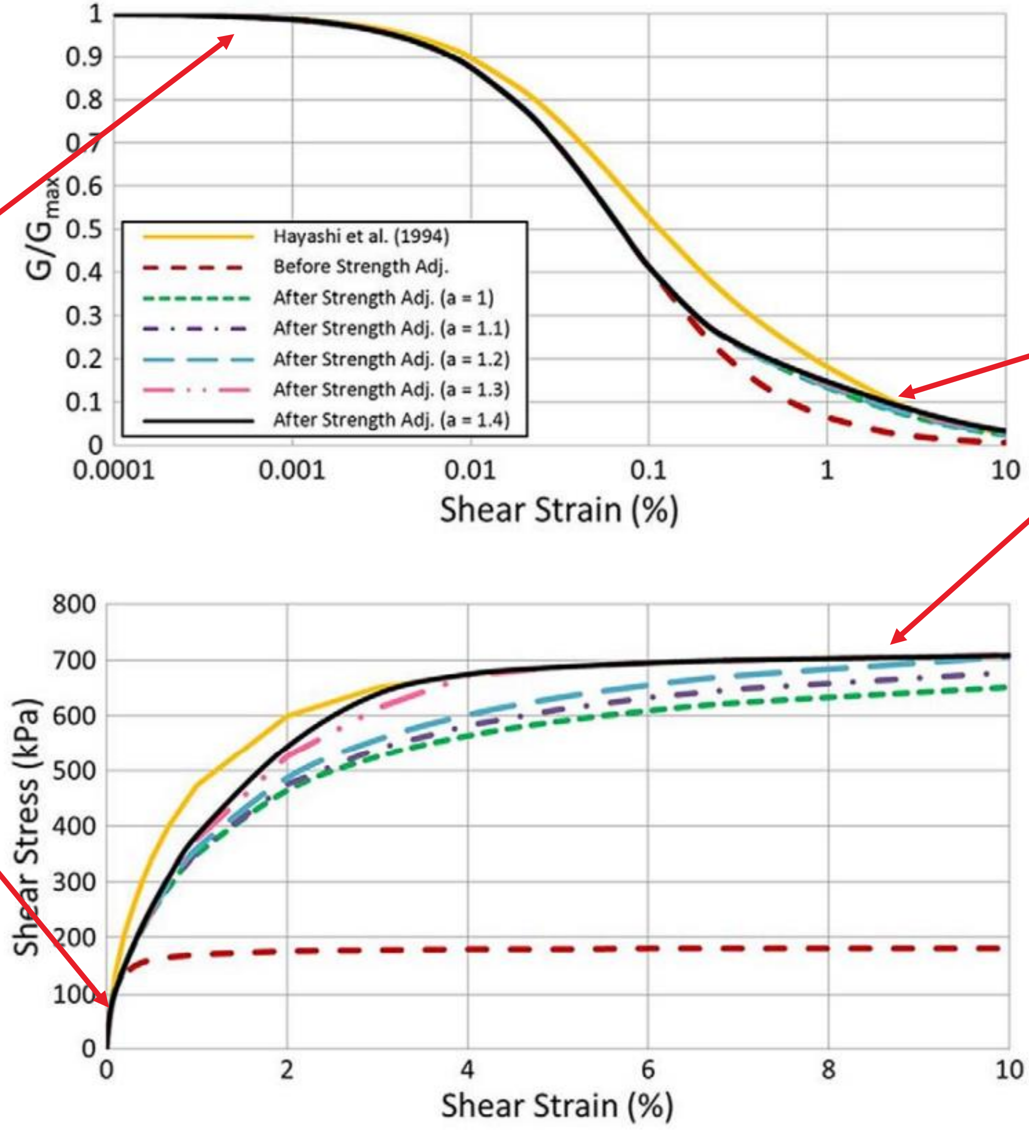


Validation and Benchmarks – Site Response

Motamed et al. 2016

Small strain modulus reduction based on Stewart and Yee (2012)

Large strain modulus reduction based on Hayashi et al (1994)



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Validation and Benchmarks – Site Response

Motamed et al. 2016

Bi-directional shaking tended to increase material damping and elongate natural period

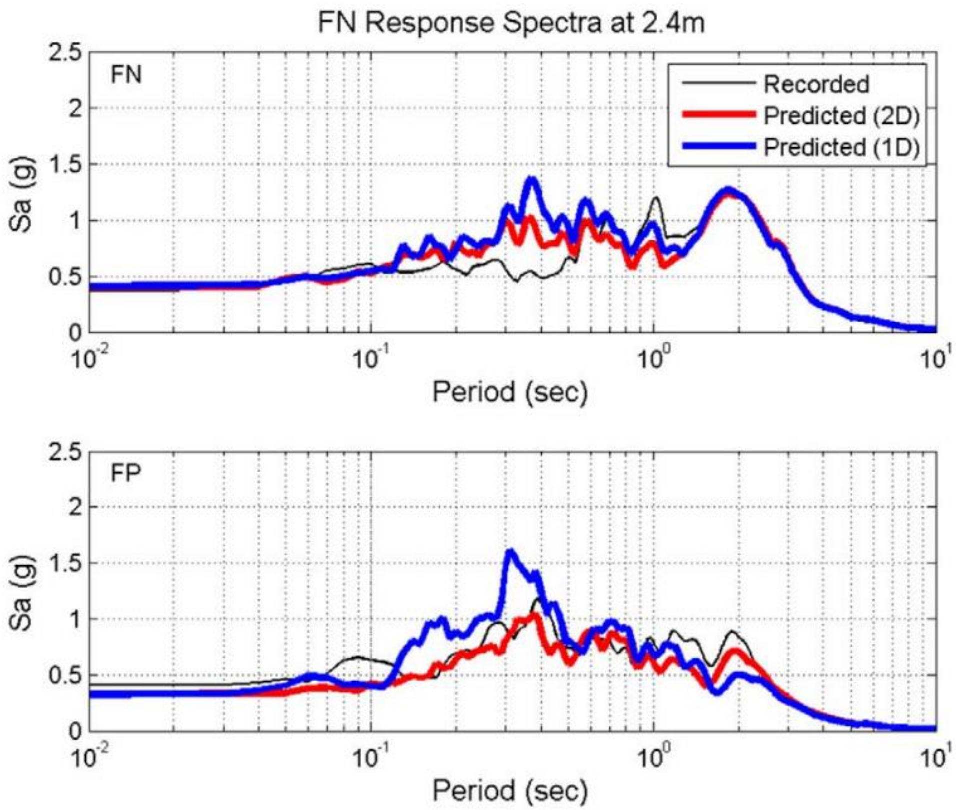

Figure 10. FN and FP measured and predicted response spectra for both 2-D and 1-D input motions at a depth of 2.4 m below the surface (the depth of accelerometer 1). All predicted data shown was developed with $D_{min} = 2\%$ and strain rate corrections on.

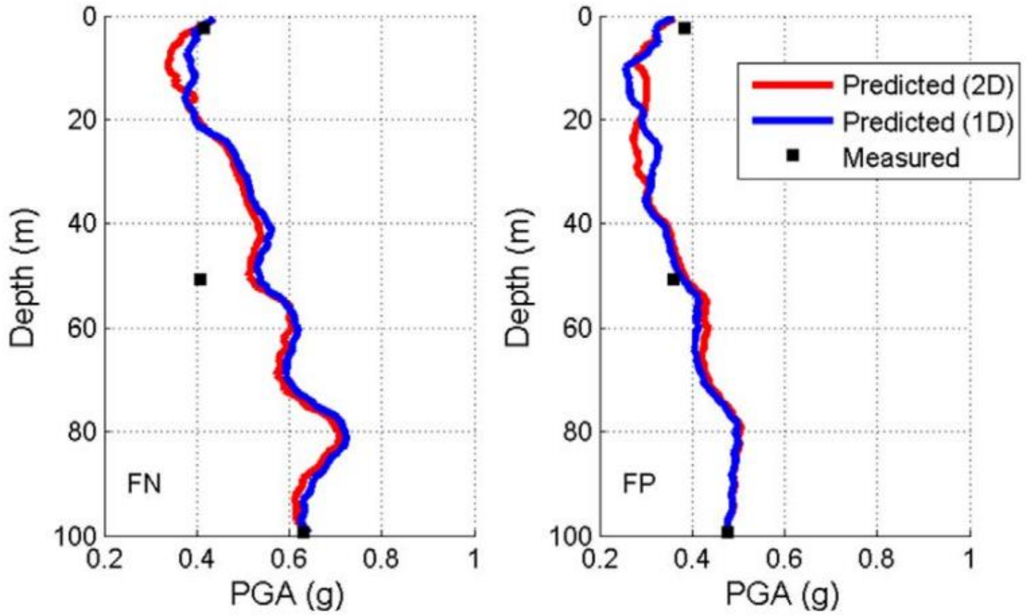

Figure 12. PGA profiles for the FN and FP directions. The data shown includes: measured (black squares), 1-D prediction (blue line) and 2-D prediction (red line). The predicted data was developed with $D_{min} = 2\%$ and strain rate corrections on.

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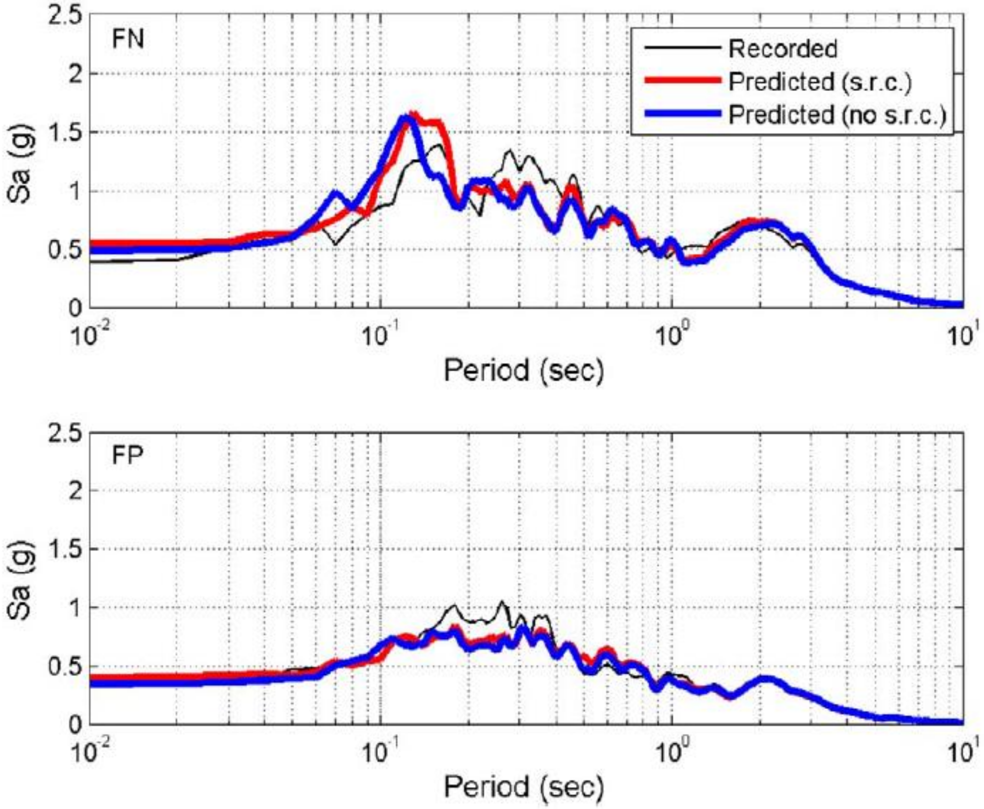
Validation and Benchmarks – Site Response

Motamed et al. 2016



Strain rate effects tended to slightly shorten the site period and increase accelerations

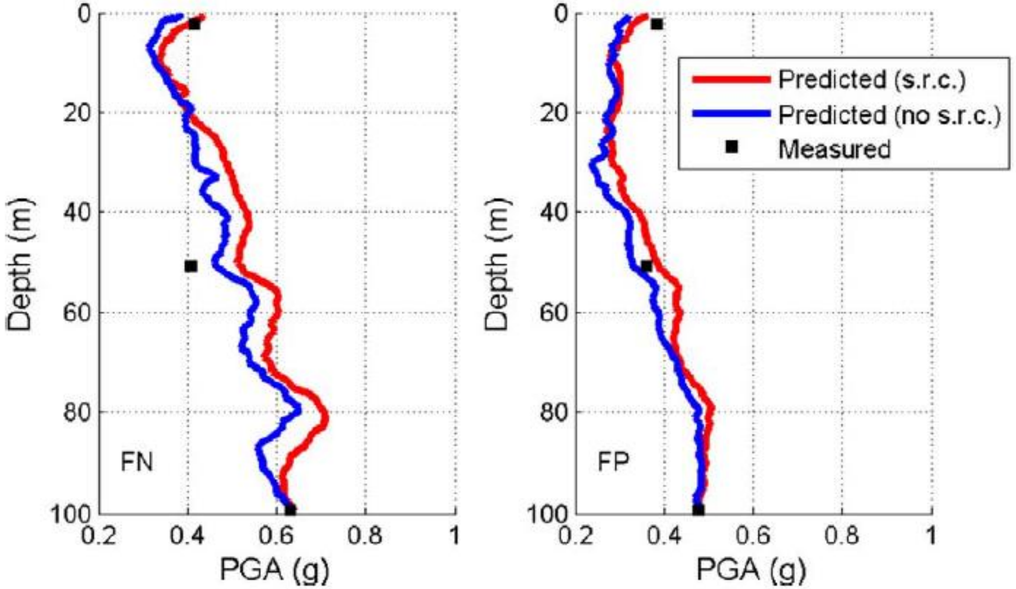


Figure 7. Two-dimensional FN and FP response spectra for a depth 50.8 m below the surface (the depth of accelerometer 2). The data shown includes: recorded at accelerometer 2 (thin black line), predicted with $D_{min} = 2\%$ and strain rate corrections on (thick red line), predicted with $D_{min} = 2\%$ and strain rate corrections off (thick blue line).

Figure 8. Two-dimensional PGA profiles for the FN and FP directions. The data shown includes: measured (black squares), predicted with $D_{min} = 2\%$ and strain rate corrections on (red line), predicted with $D_{min} = 2\%$ and strain rate corrections off (blue line).

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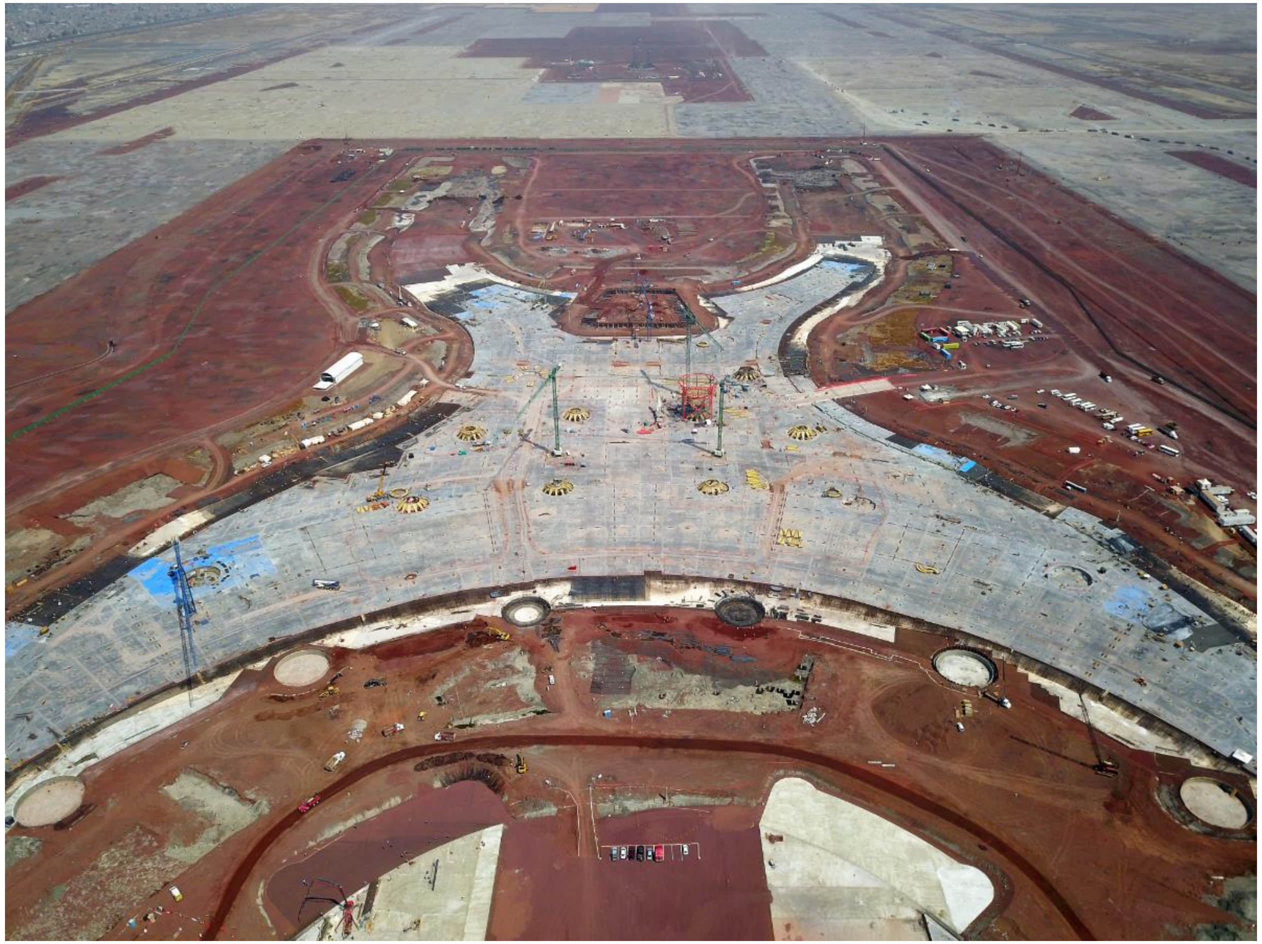
Validation and Benchmarks – Site Response

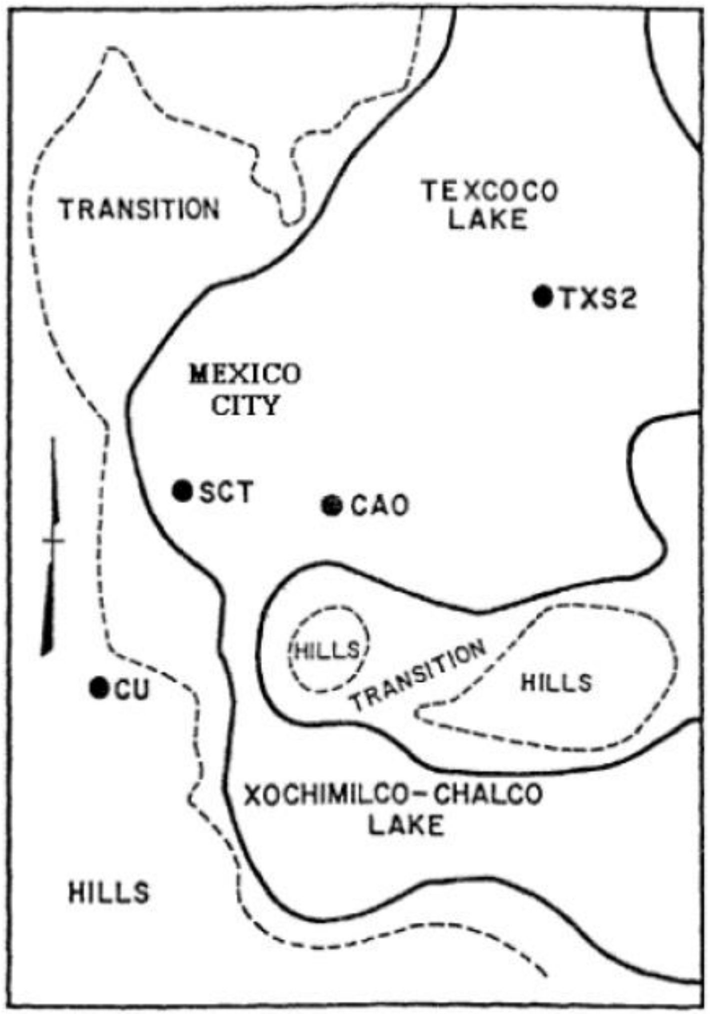
Kumar et al. 2017

- Study Considers:
  - Multiple recordings from 3 downhole arrays
  - Strain Rate Effects
  - Non-Masing Damping



Validation and Benchmarks – Site Response

Kumar et al. 2017



Event Date	Epicentral Distance (km)	Magnitude
Motions considered for CAO site		
10/24/1993	310	6.6
5/23/1994	215	5.6
10/9/1995	591	8
1/11/1997	446	7.1
6/21/1999	306	6.3
9/30/1999	438	7.4
9/19/1985*	420	8
Motions considered for SCT site		
9/14/1995	325	7.3
6/15/1999	220	6.9
9/19/1985*	425	8
Motions considered for TXS2 site		
6/15/1999*	212	6.9

*Downhole array data not available
Used CU outcrop motions with appropriate scale factors

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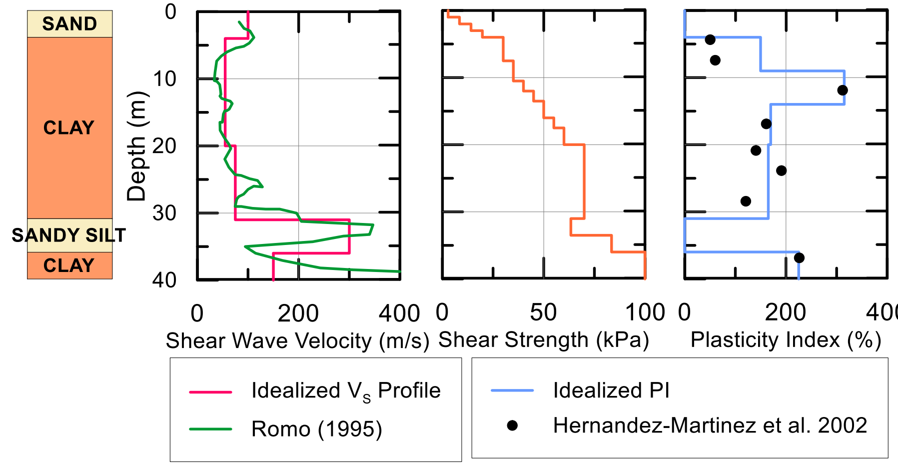
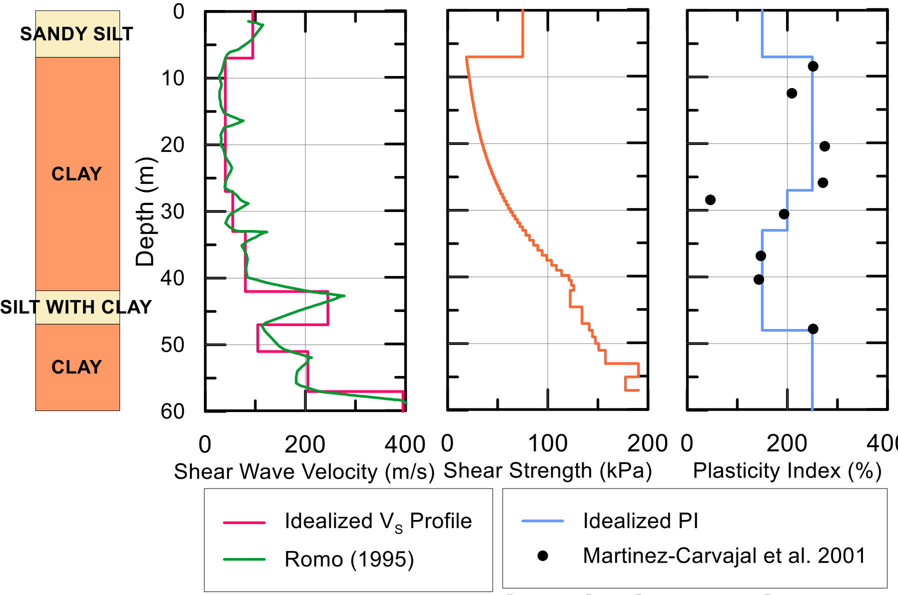
Validation and Benchmarks – Site Response

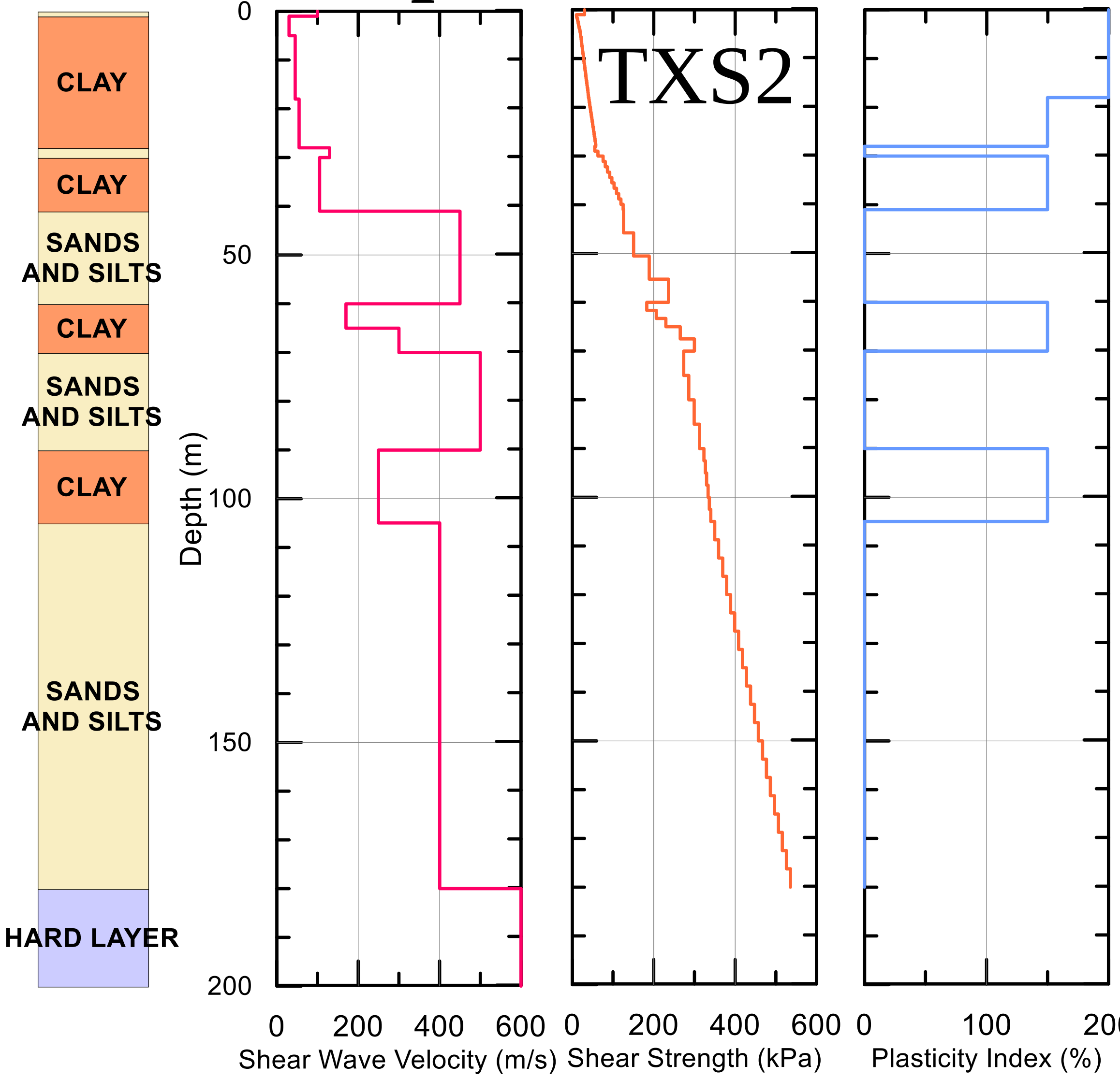
Kumar et al. 2017



SCT



CAO



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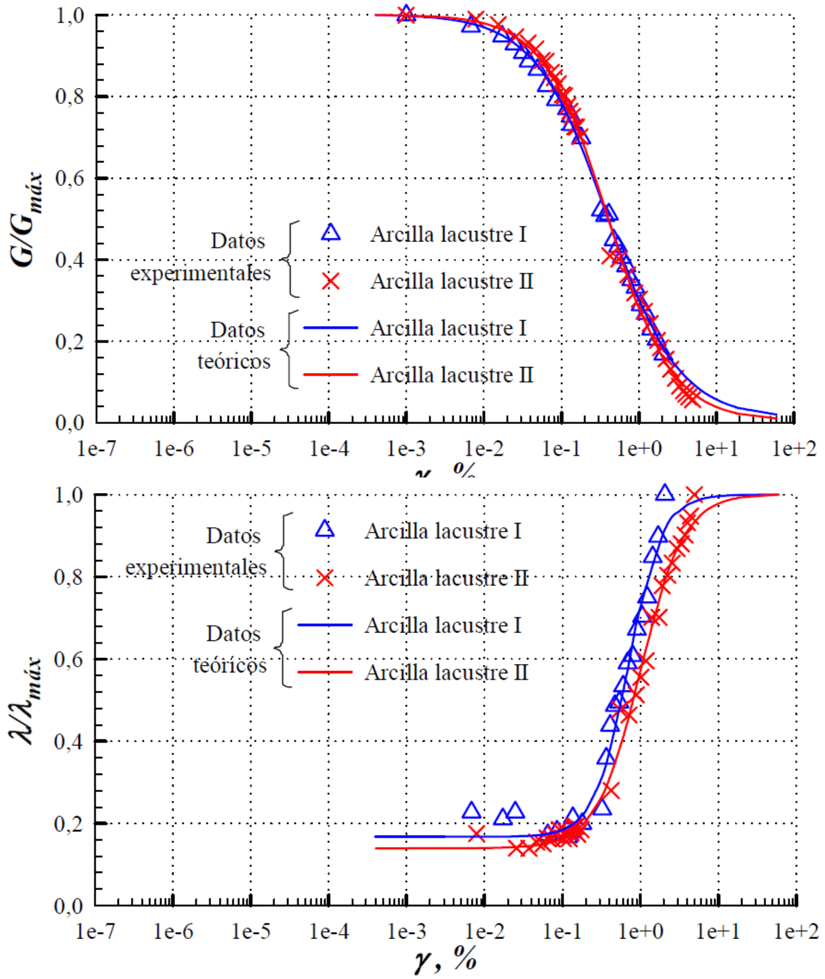
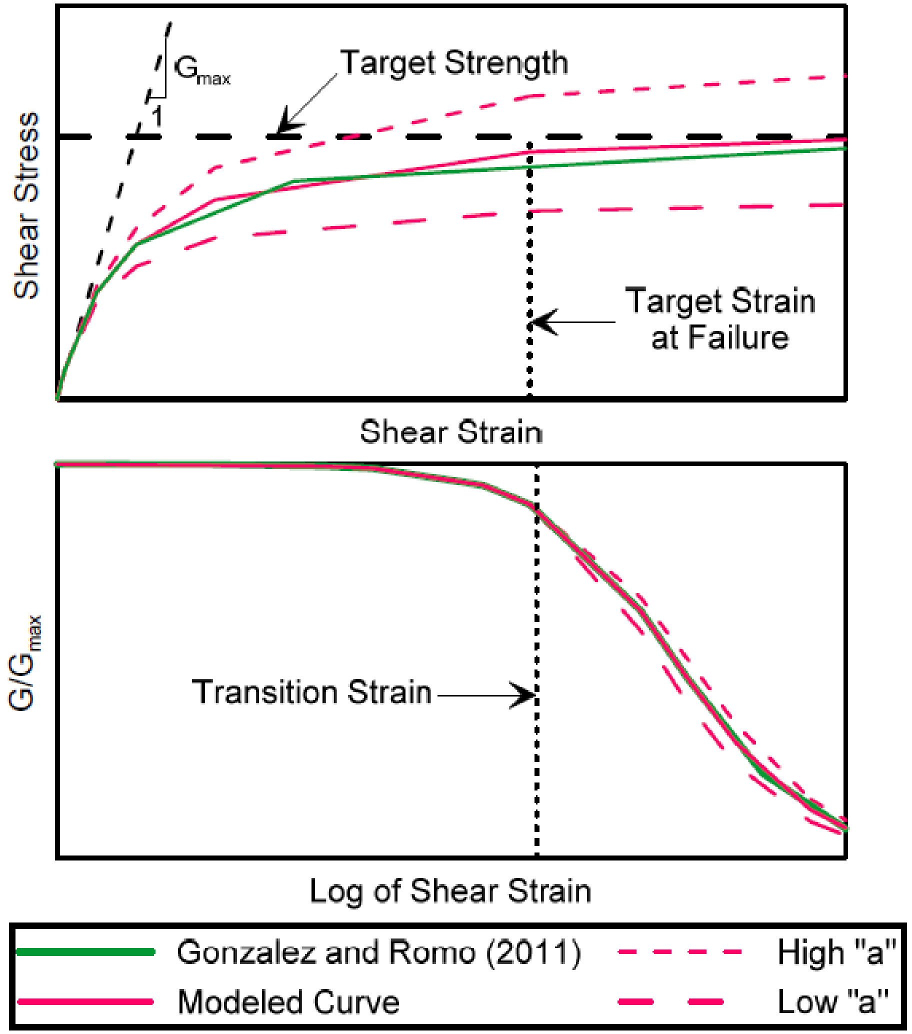
Kumar et al. 2017



after Gonzalez and Romo (2011)

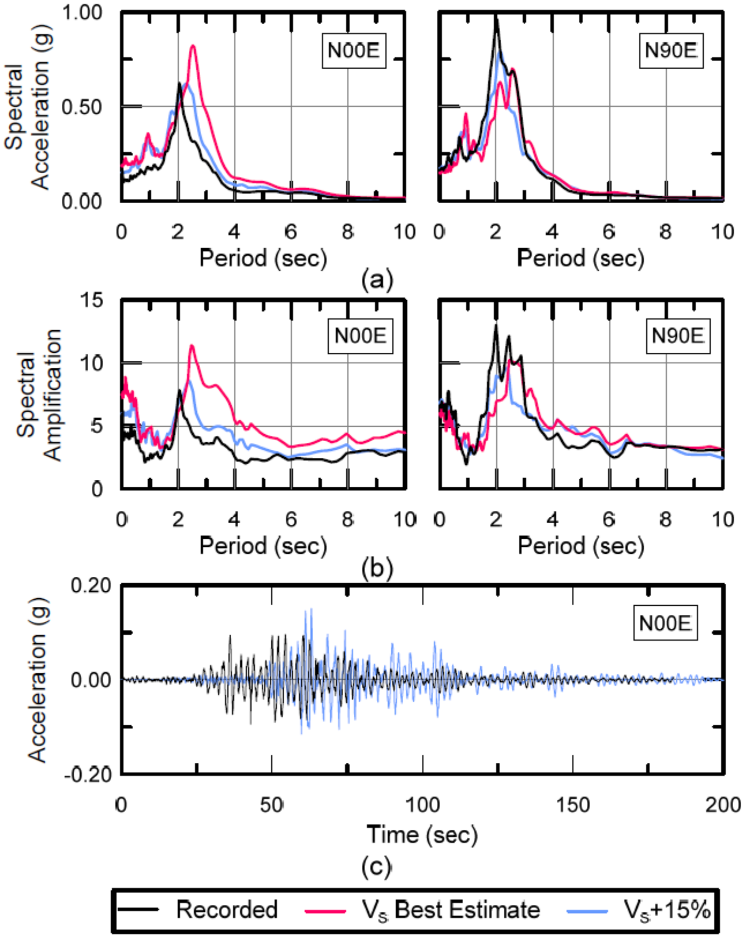


Gonzalez and Romo (2011) used to define Non-Masing Damping Curves and Initial Stiffness Degradation Curves

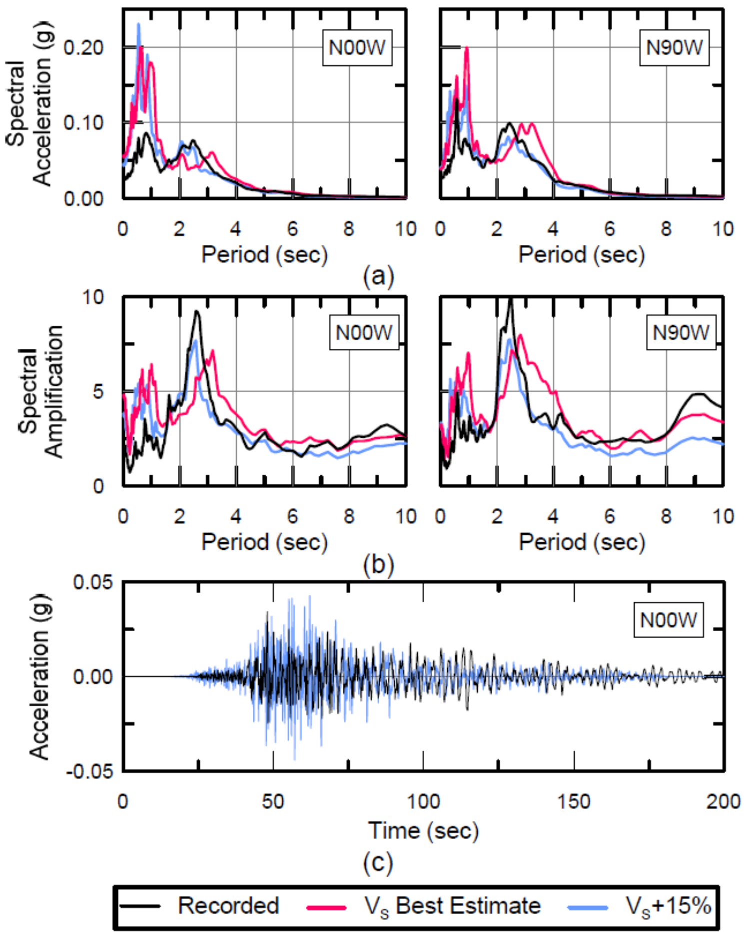
Validation and Benchmarks – Site Response

Kumar et al. 2017

SCT - September 19, 1985 M_W8.0



TXS2 – June 15, 1999 M_W6.9

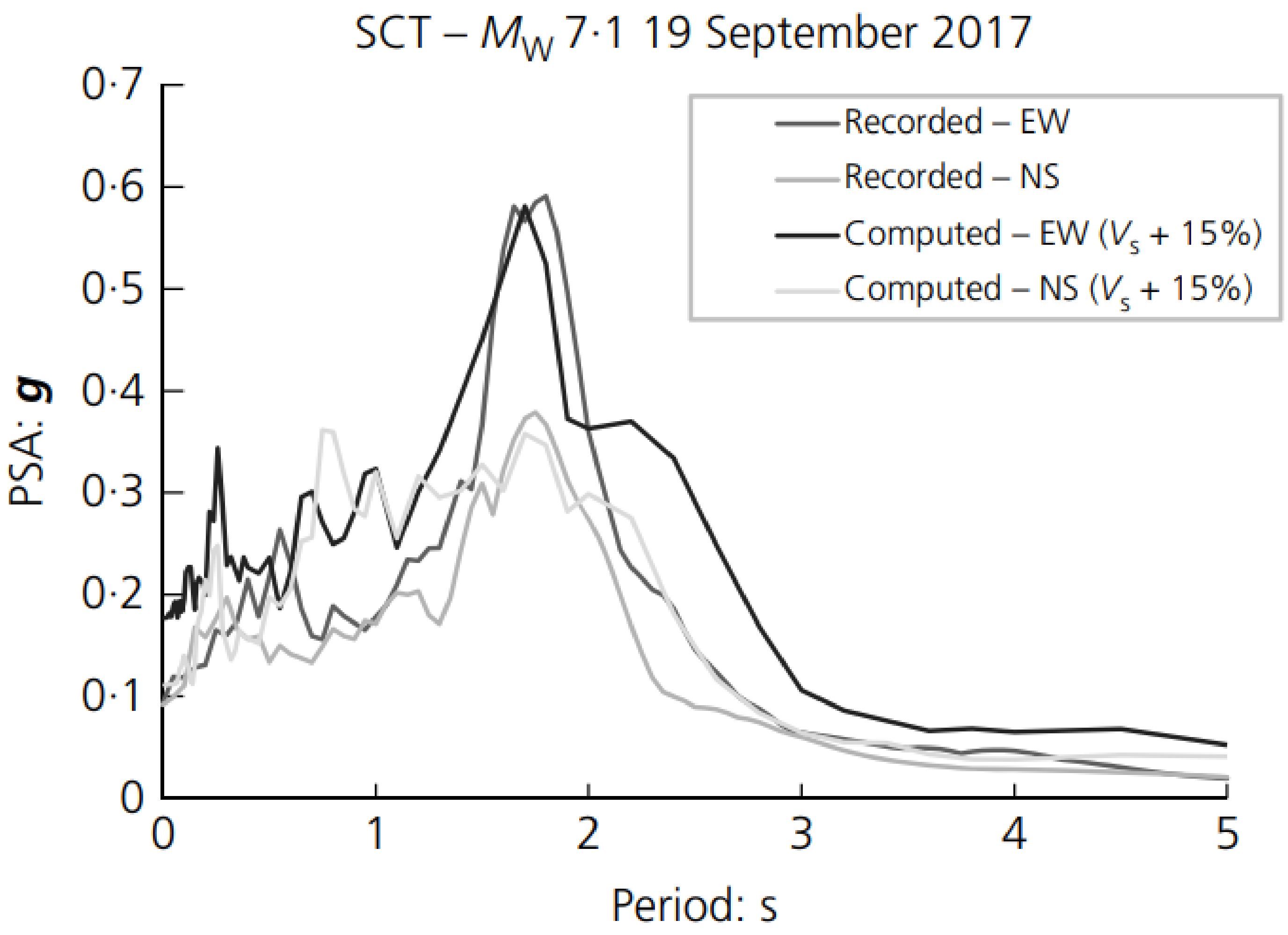


Notes:

- Non-Masing damping was needed to capture high spectral amplifications and second phase near-harmonic beating
- Results improved by Vs+15% assumption
- Assumes 15% strength increase per log cycle of shear strain rate

Validation and Benchmarks – Site Response

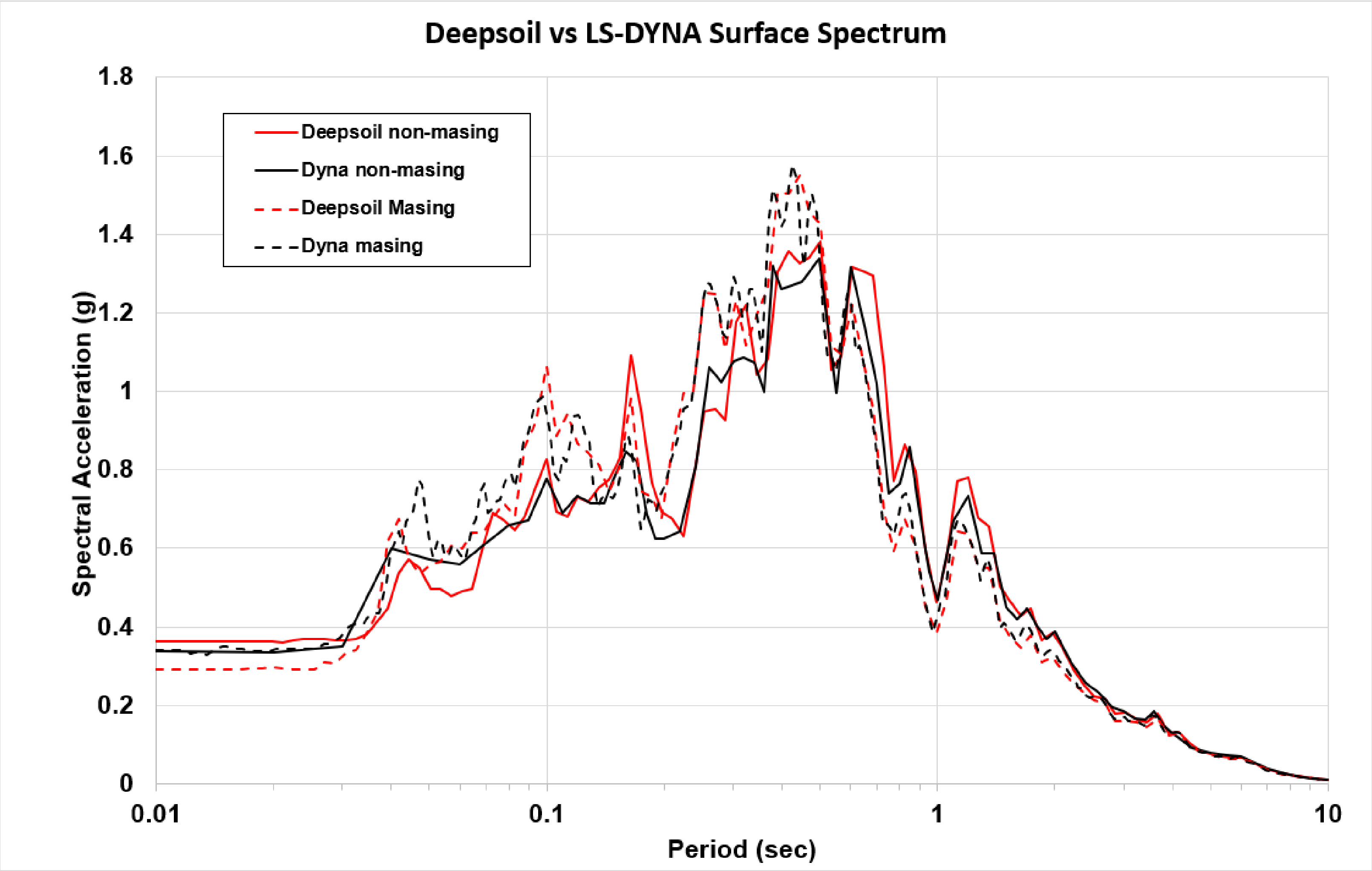
O'Riordan et al. 2017



Results from 2017 Puebla Earthquake, which occurred after publication of Kumar et al. (2017)

Validation and Benchmarks – Site Response

Non-Masing Validation



Validation and Benchmarks – SSI

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Validation and Benchmarks – SSI

References

- Ellison, K., Almufti, I., Masroor, A., Koutrouvelis, I. & Huang, Y. A. (2017) “Rupture to Rafters” Approach using Nonlinear Soil-Structure-Interaction Analysis for Performance-Based Earthquake Design. 3rd Int Conf on Performance-based Design in Earthquake Geotech Eng.
- Hashash, Y. M. A., Dashti, S., Musgrove, M., Gillis, K., Walker, M., Ellison, K., Yumar, I. B. (2018). Influence of Tall Buildings on the Seismic Response of Shallow Underground Structures. ASCE Journal of Geotechnical and Geoenvironmental Engineering. 144(12).
- A future publication in progress by Boushehri et al. provisionally entitled: Nonlinear Time-Domain Soil-Structure Interaction Analysis of a High-Rise Building Benchmarked Against Actual Recordings

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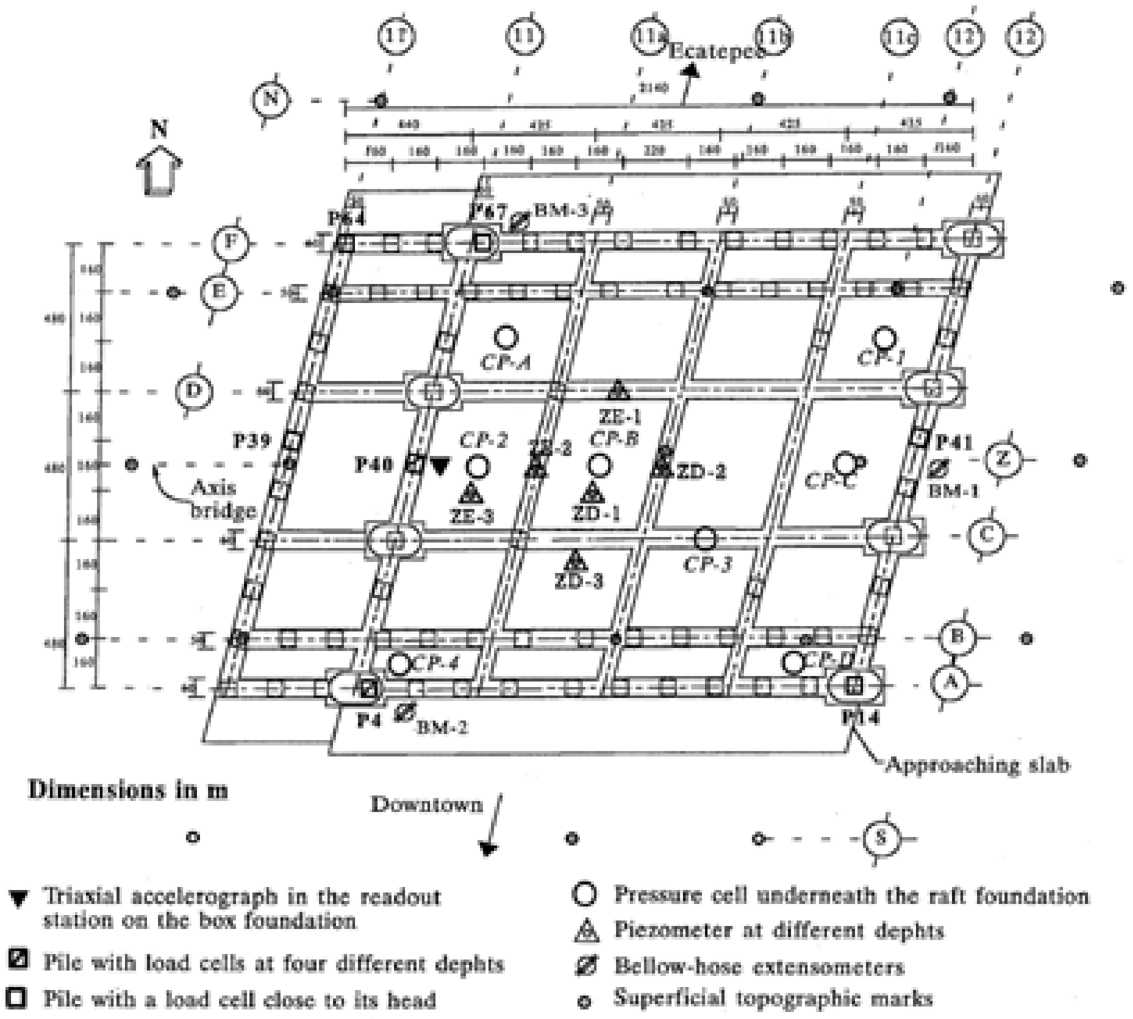
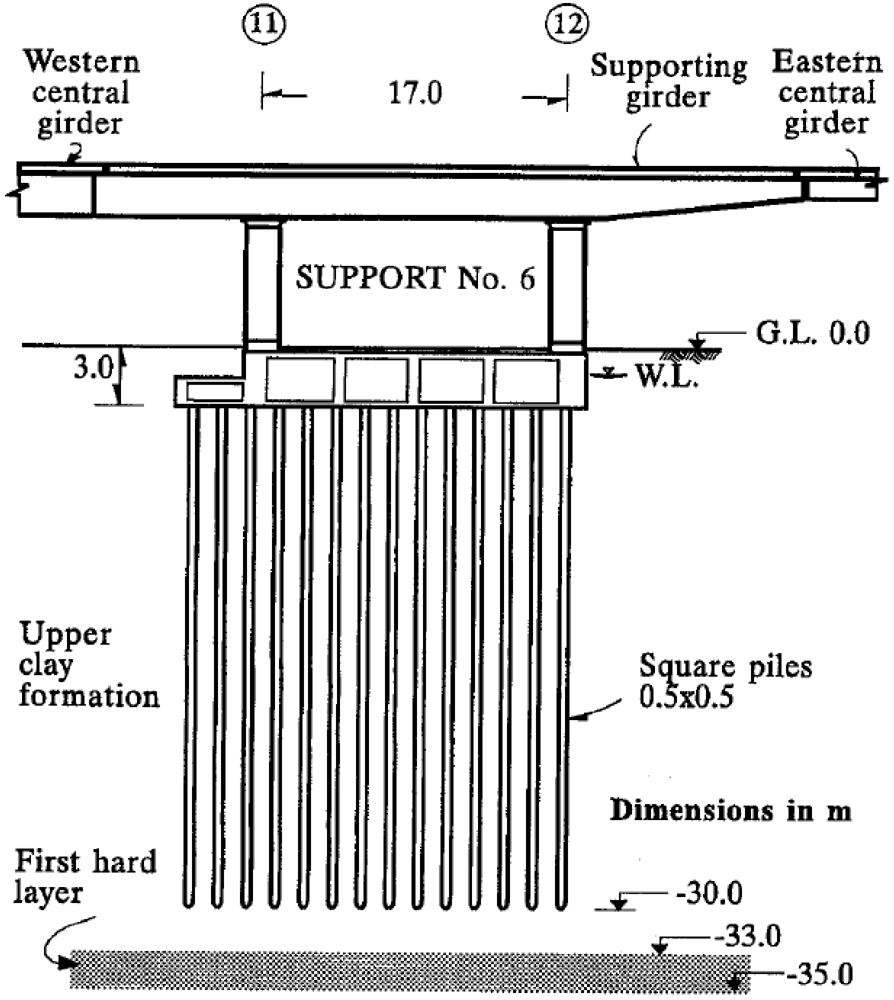
Validation and Benchmarks – SSI

Ellison et al. 2017 – Impulsora Bridge

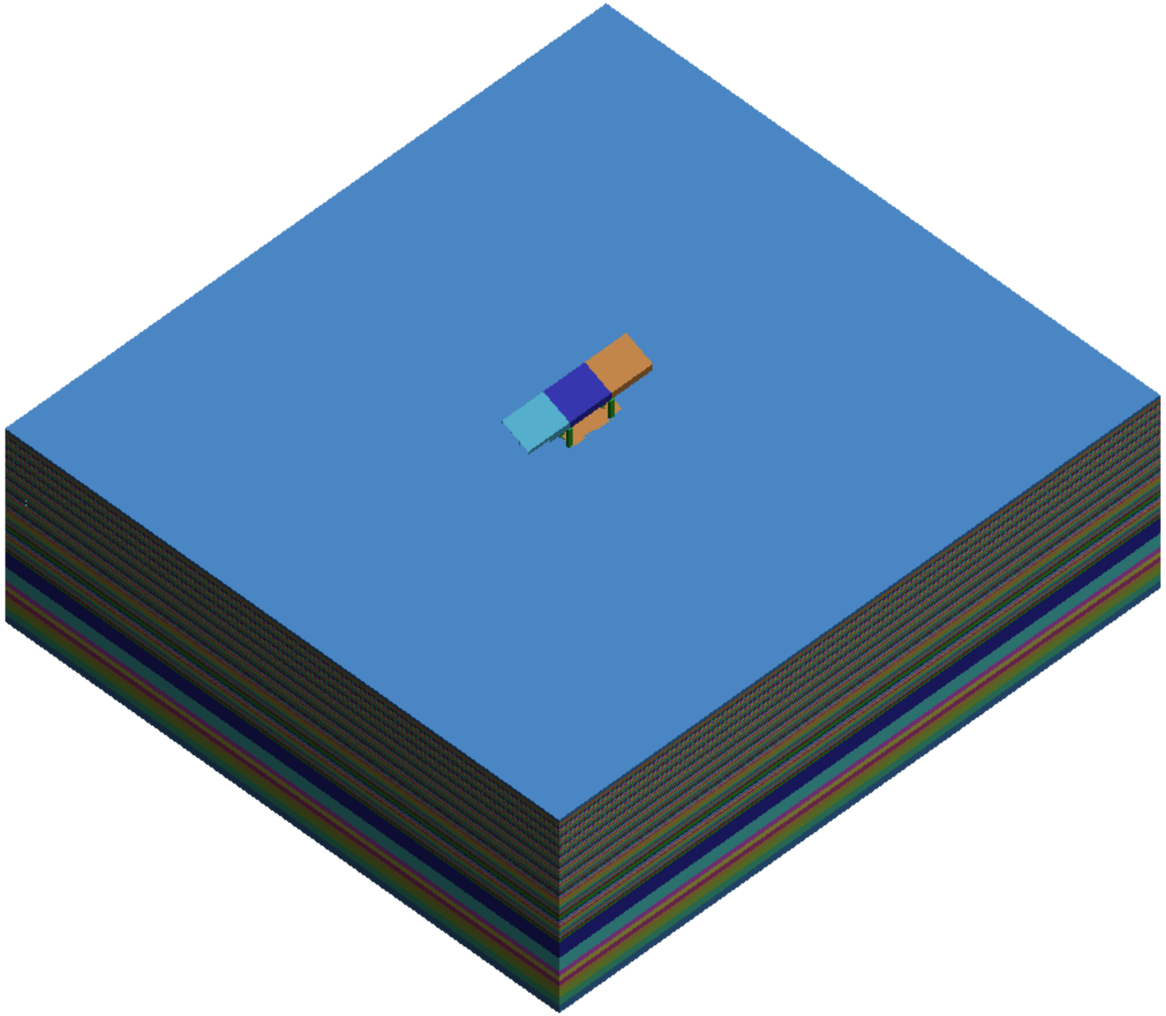


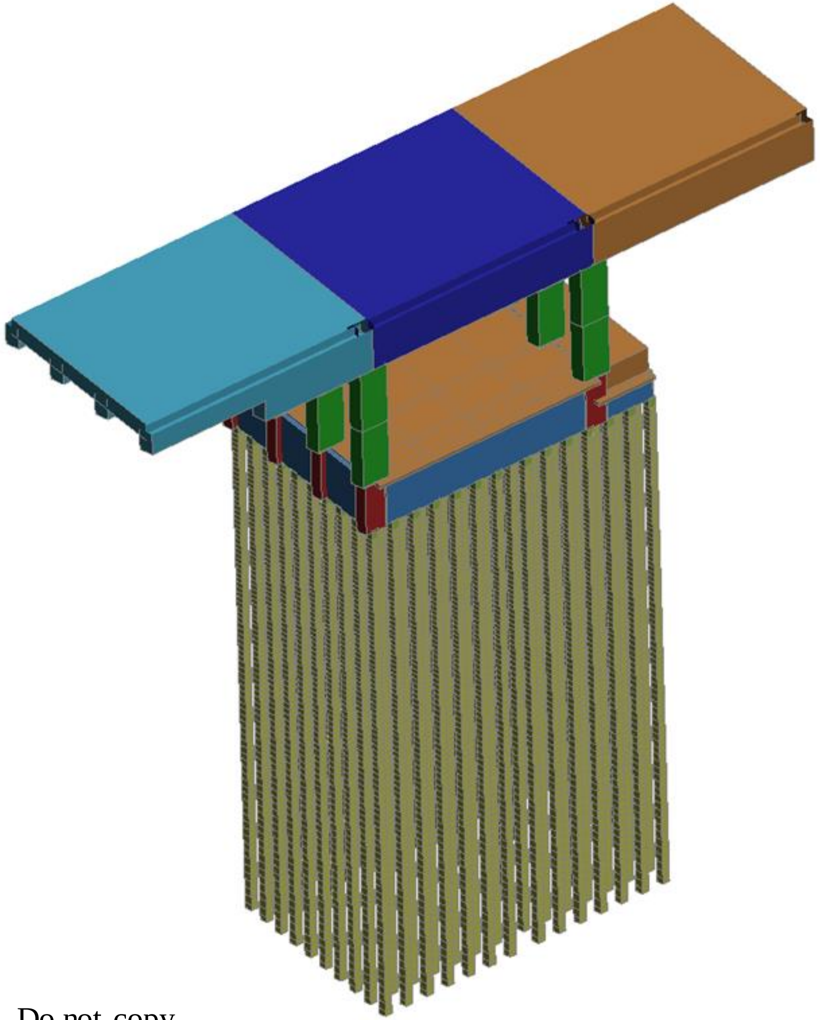


From Mendoza et al. (1998) and Mendoza et al. (2000)

Validation and Benchmarks – SSI

Ellison et al. 2017 – Impulsora Bridge





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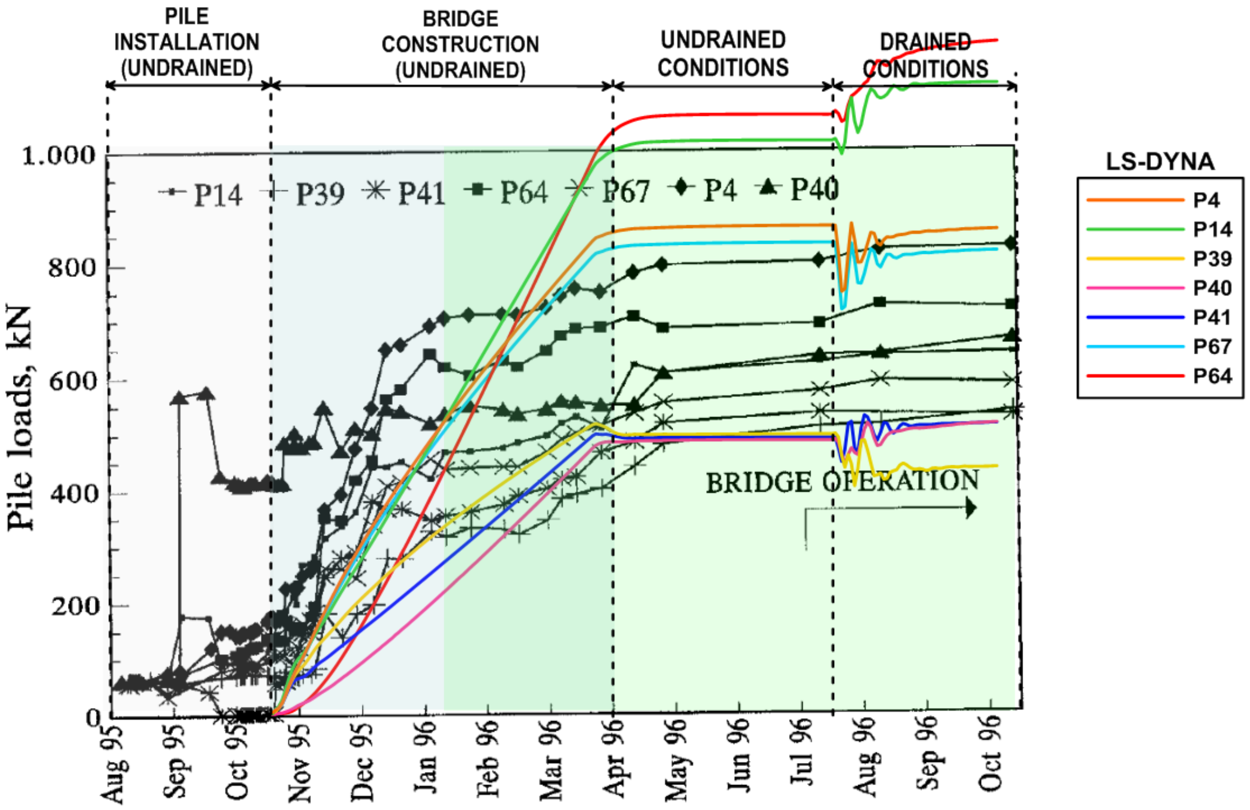
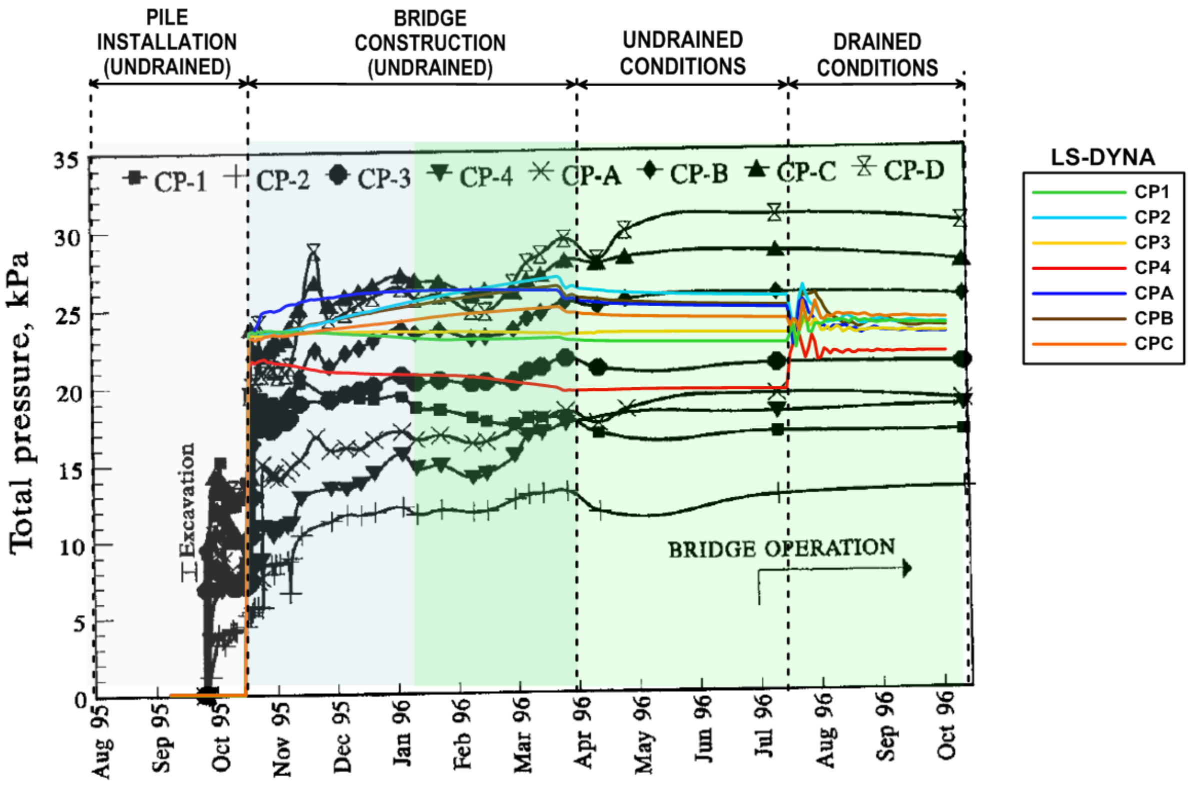
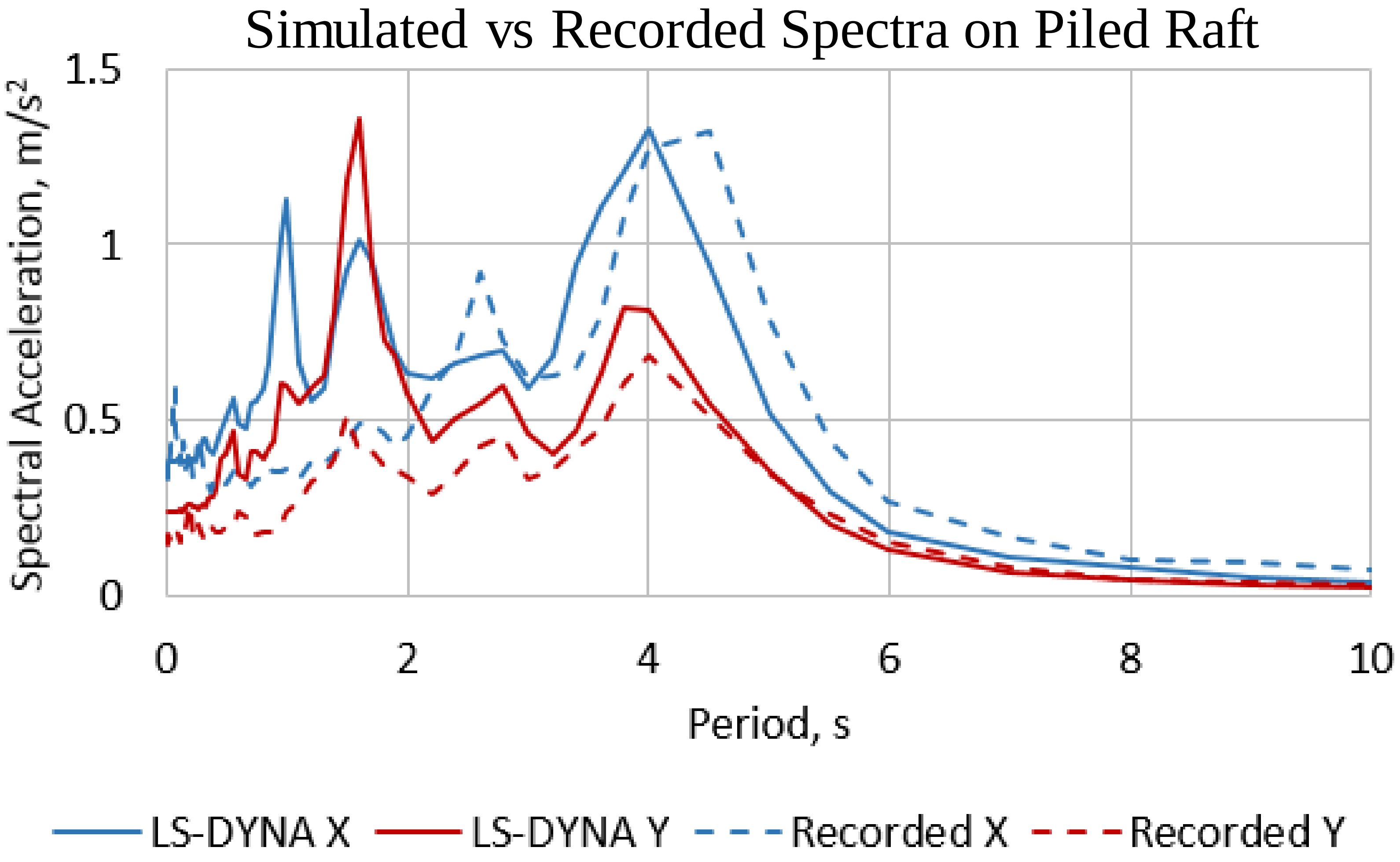
Ellison et al. 2017 – Impulsora Bridge





Validation and Benchmarks – SSI

Ellison et al. 2017 – Impulsora Bridge



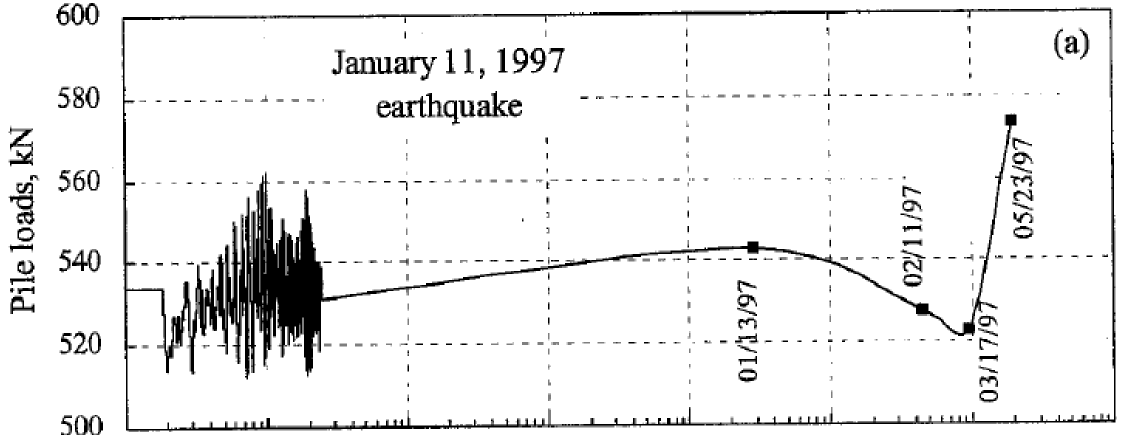
Reasonable agreement given:

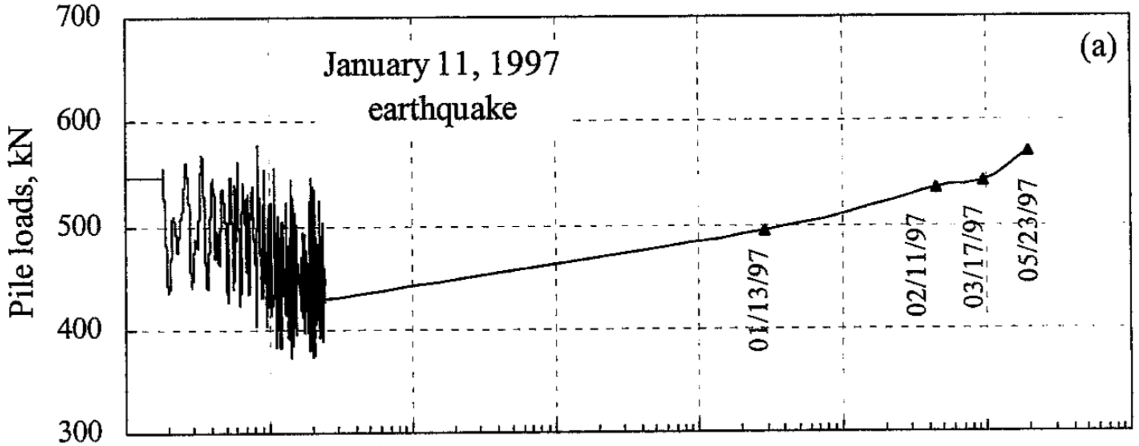
- Input motion based on "outcrop" recording 11 miles away
- No site-specific data below 25 m

Validation and Benchmarks – SSI

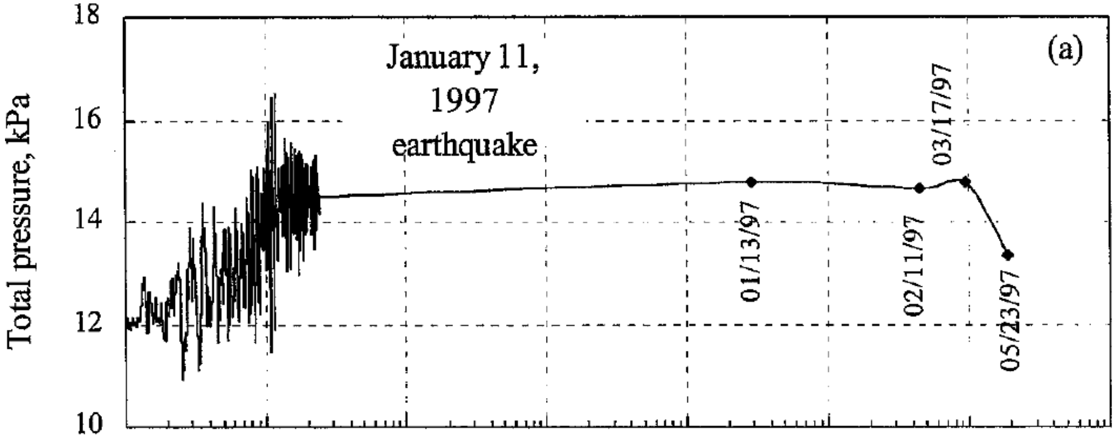
Ellison et al. 2017 – Impulsora Bridge

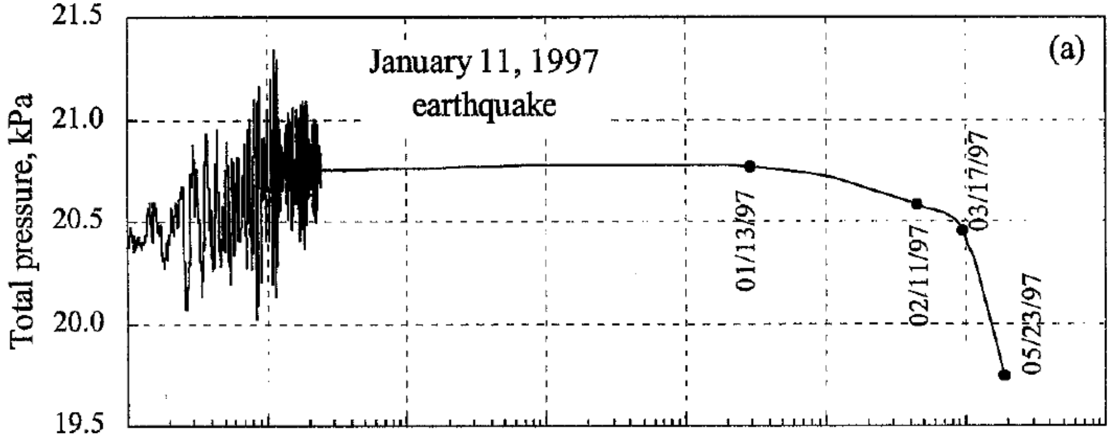
Sample pile loads





Sample pressure cell loads





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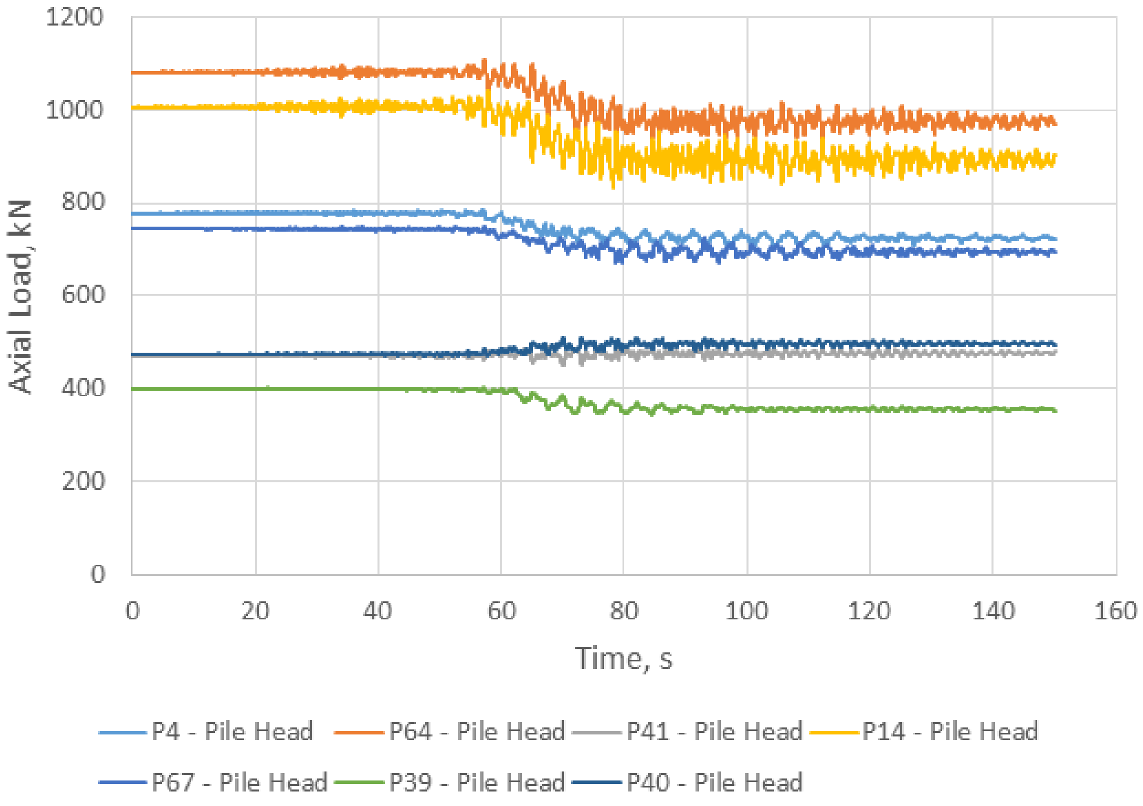
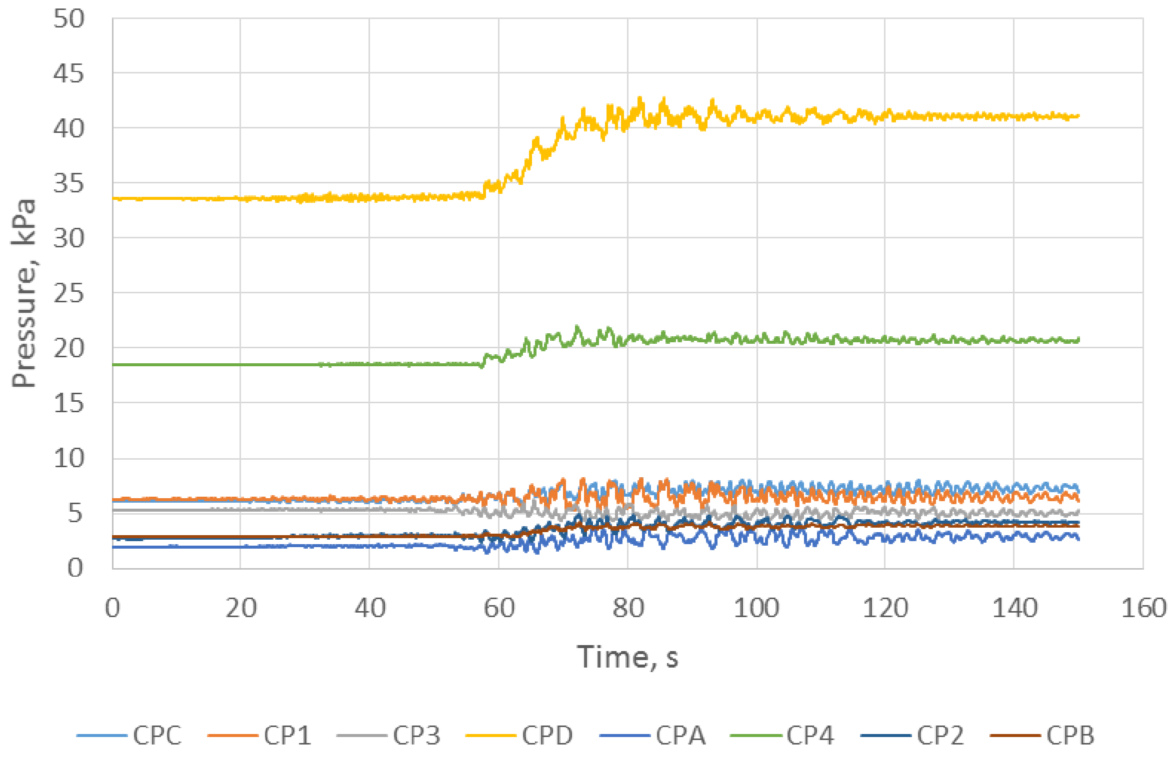
Validation and Benchmarks – SSI

Ellison et al. 2017 – Impulsora Bridge

Simulated pile loads

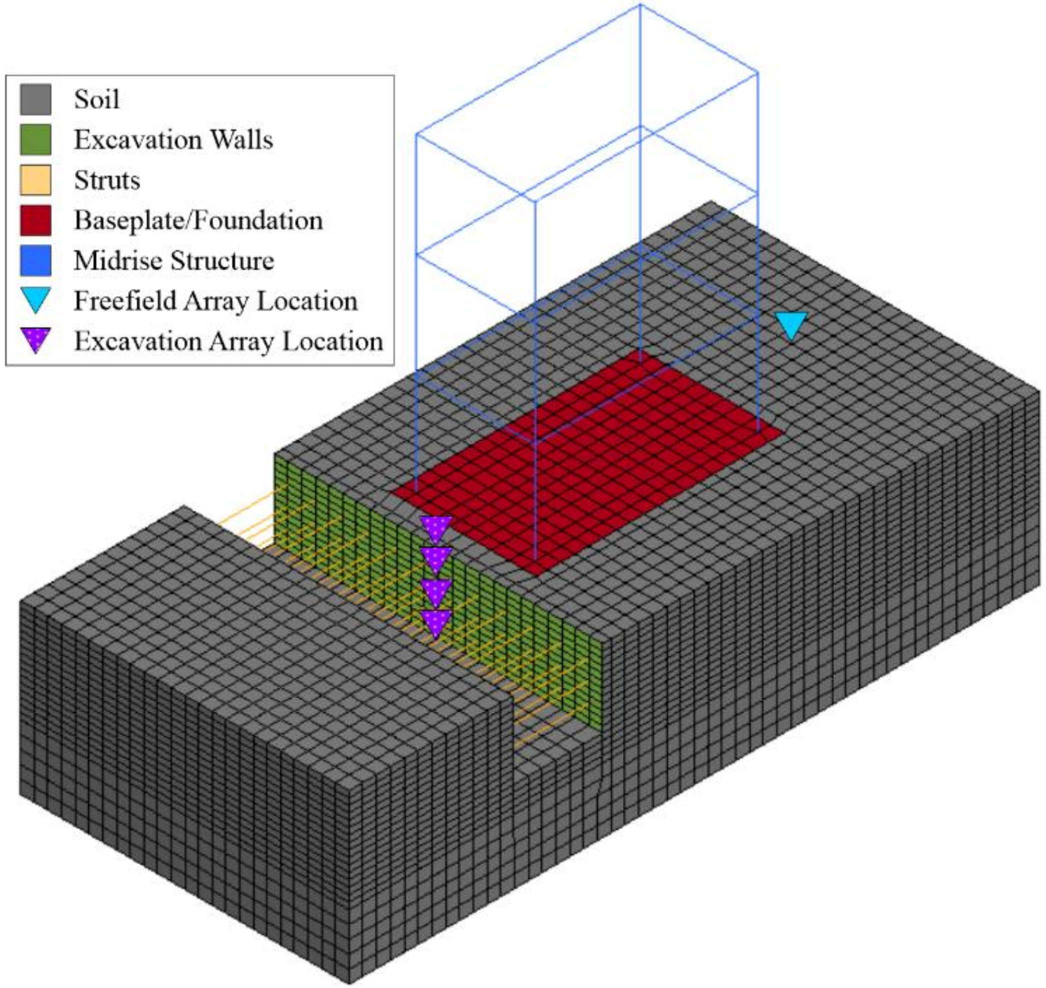


Simulated pressure cell loads



Validation and Benchmarks – SSI

Hashash et al. 2018



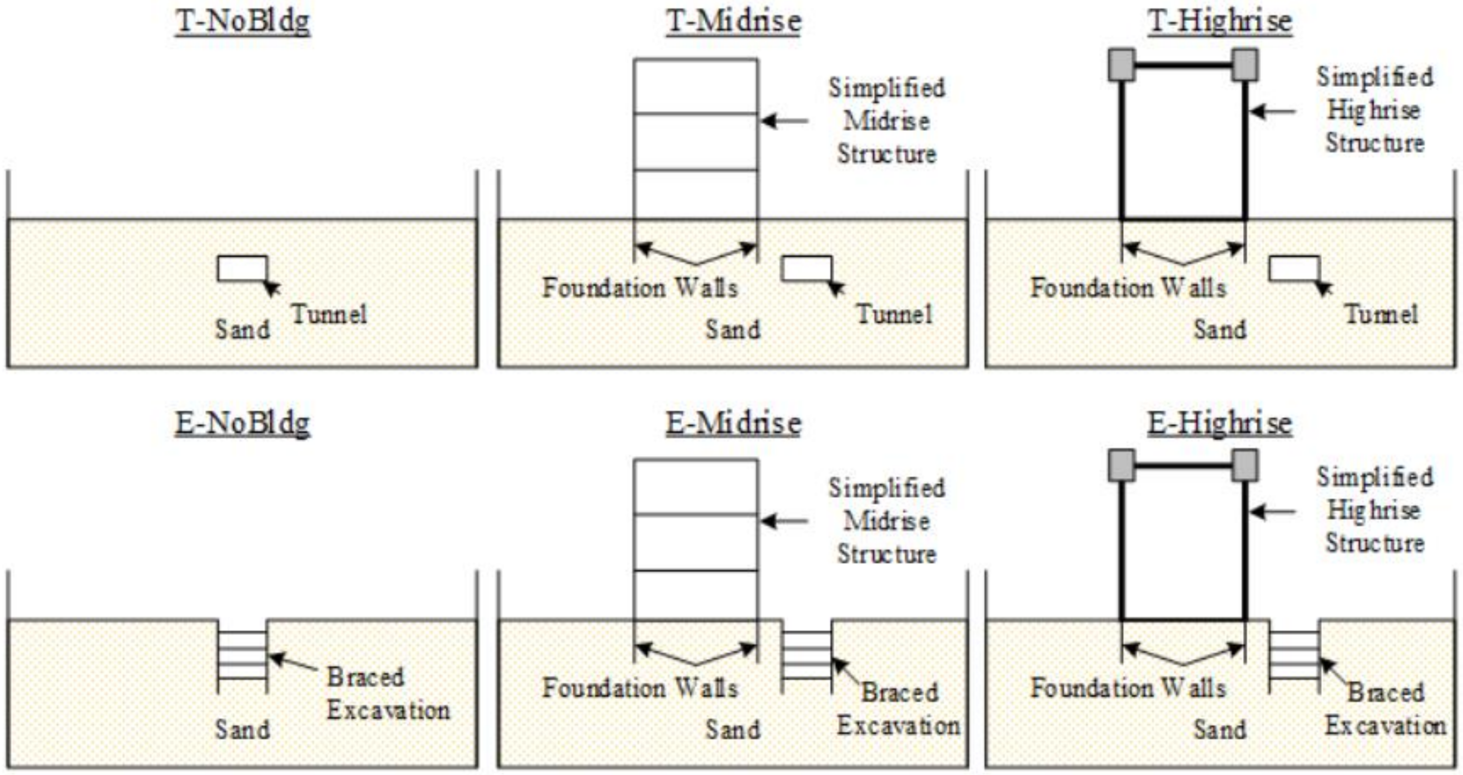


Fig. 1 – Centrifuge test configurations (T: Tunnel; E: Excavation).

Validation and Benchmarks – SSI

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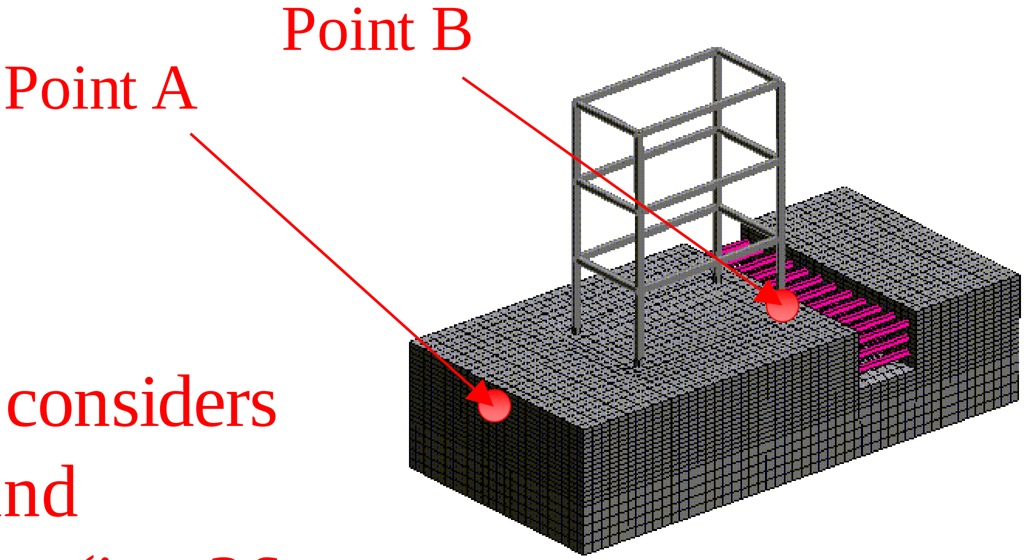
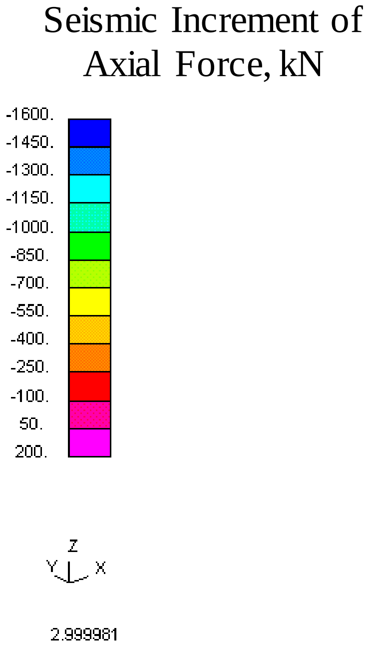
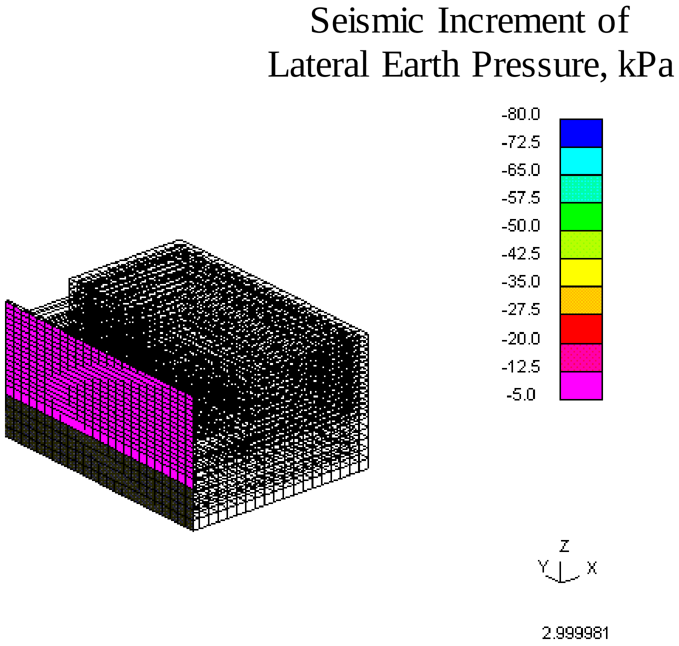
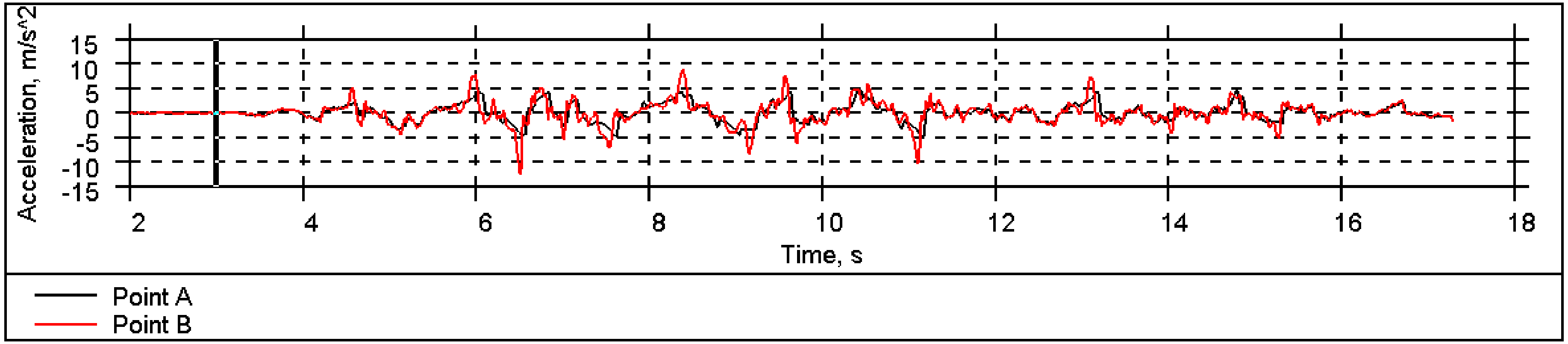
Hashash et al. 2018





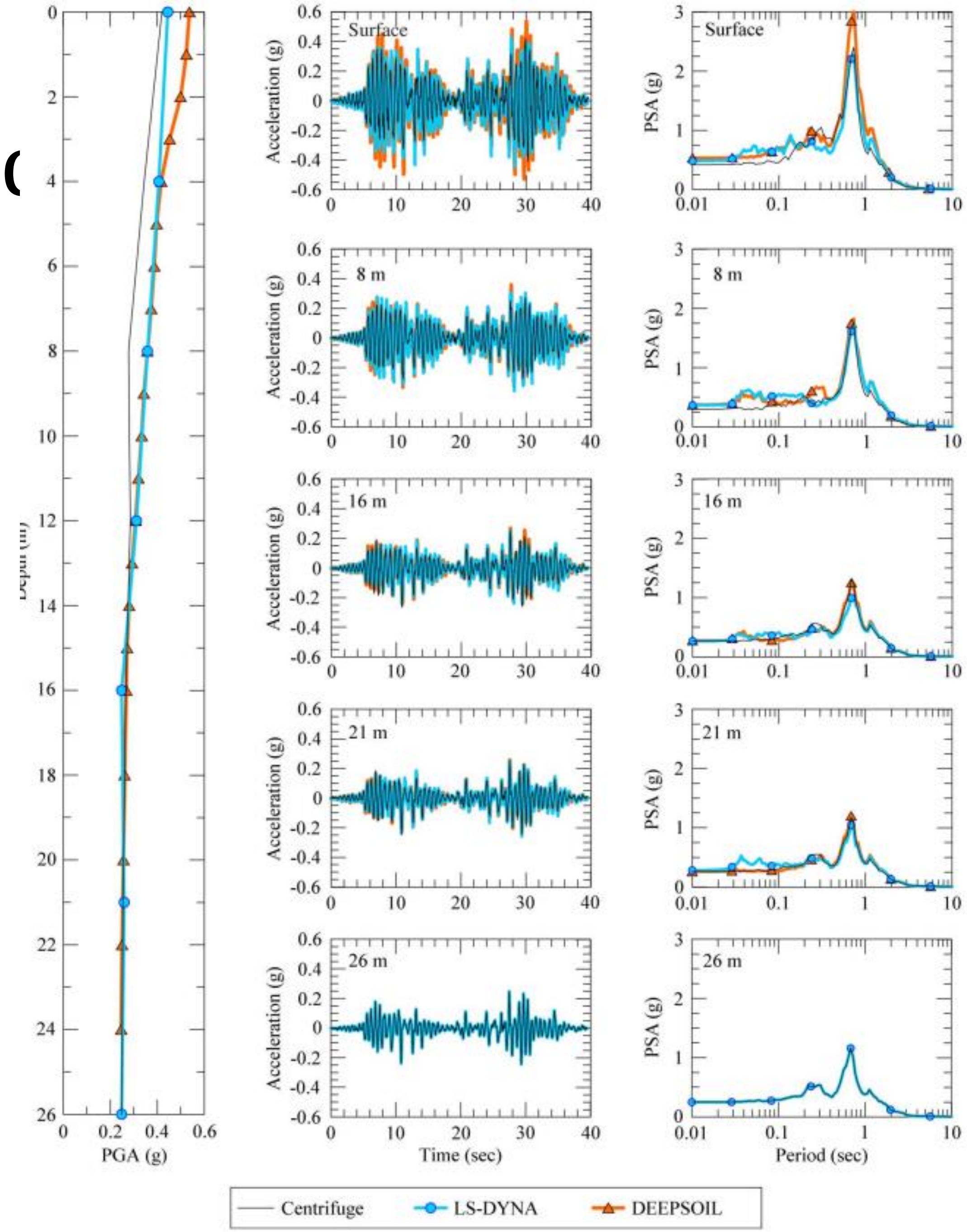


Study considers 6 ground motions (i.e. 36 simulations)



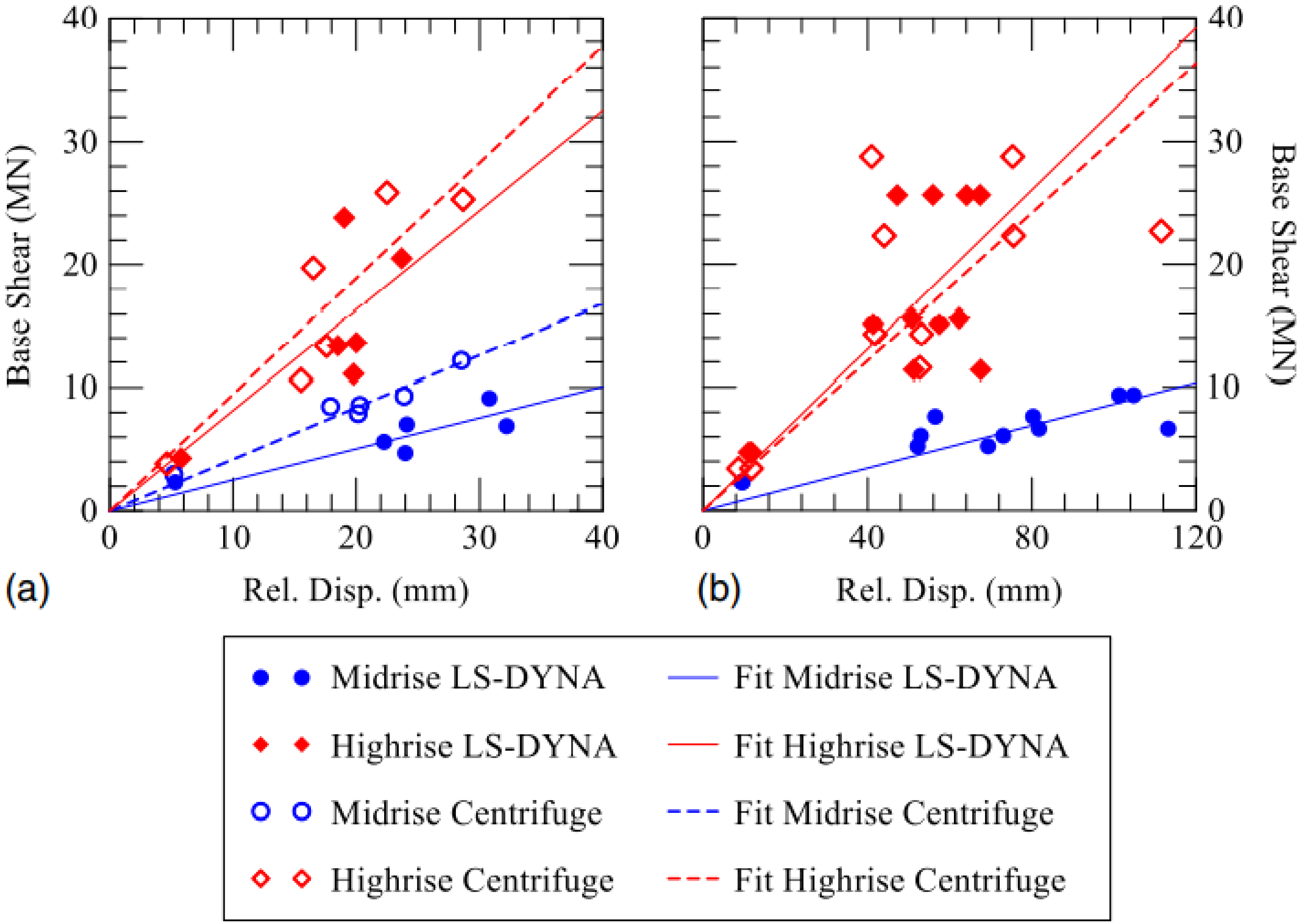
Validation anc

Hashash et al. 2018



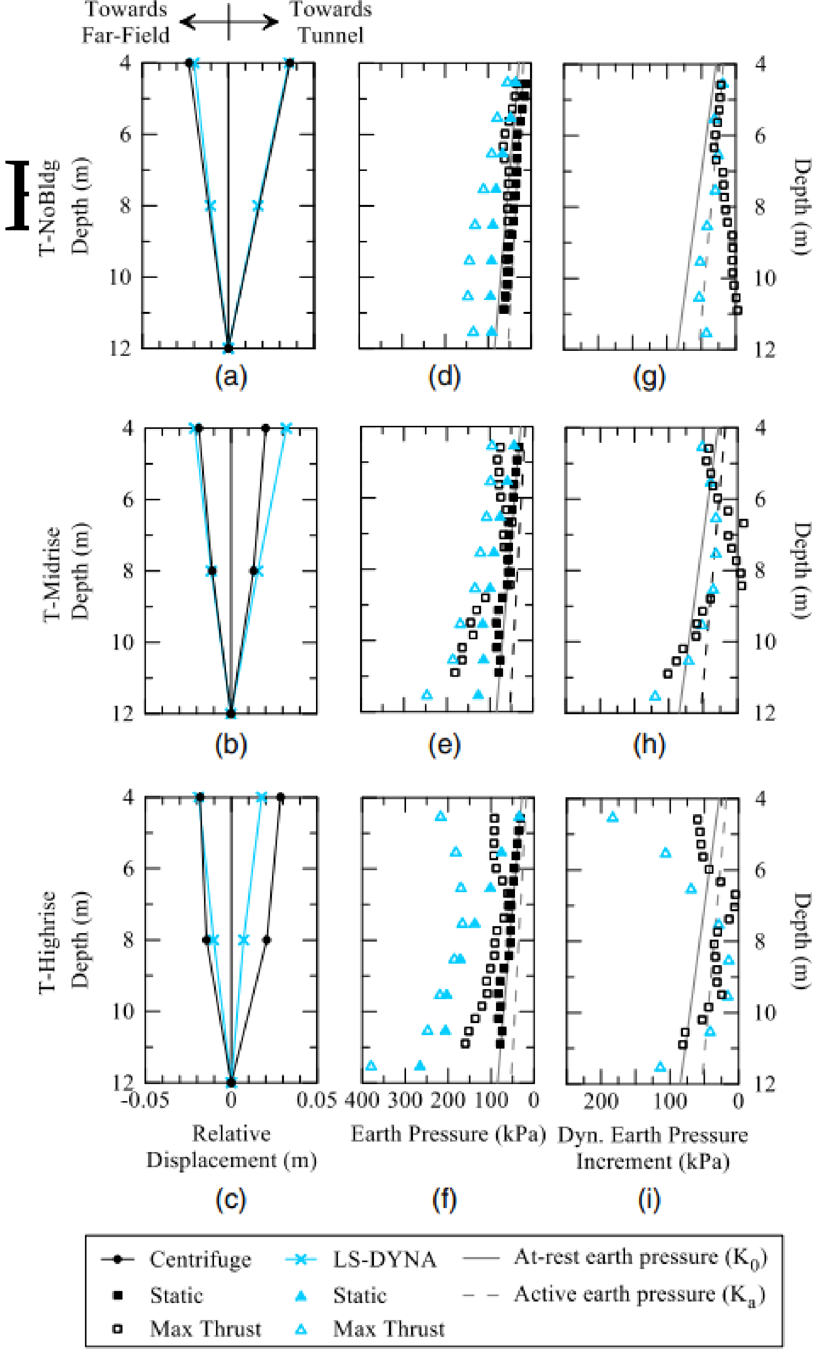
Validation and Benchmarks – SSI

Hashash et al. 2018



Validation and I

Hashash et al. 2018



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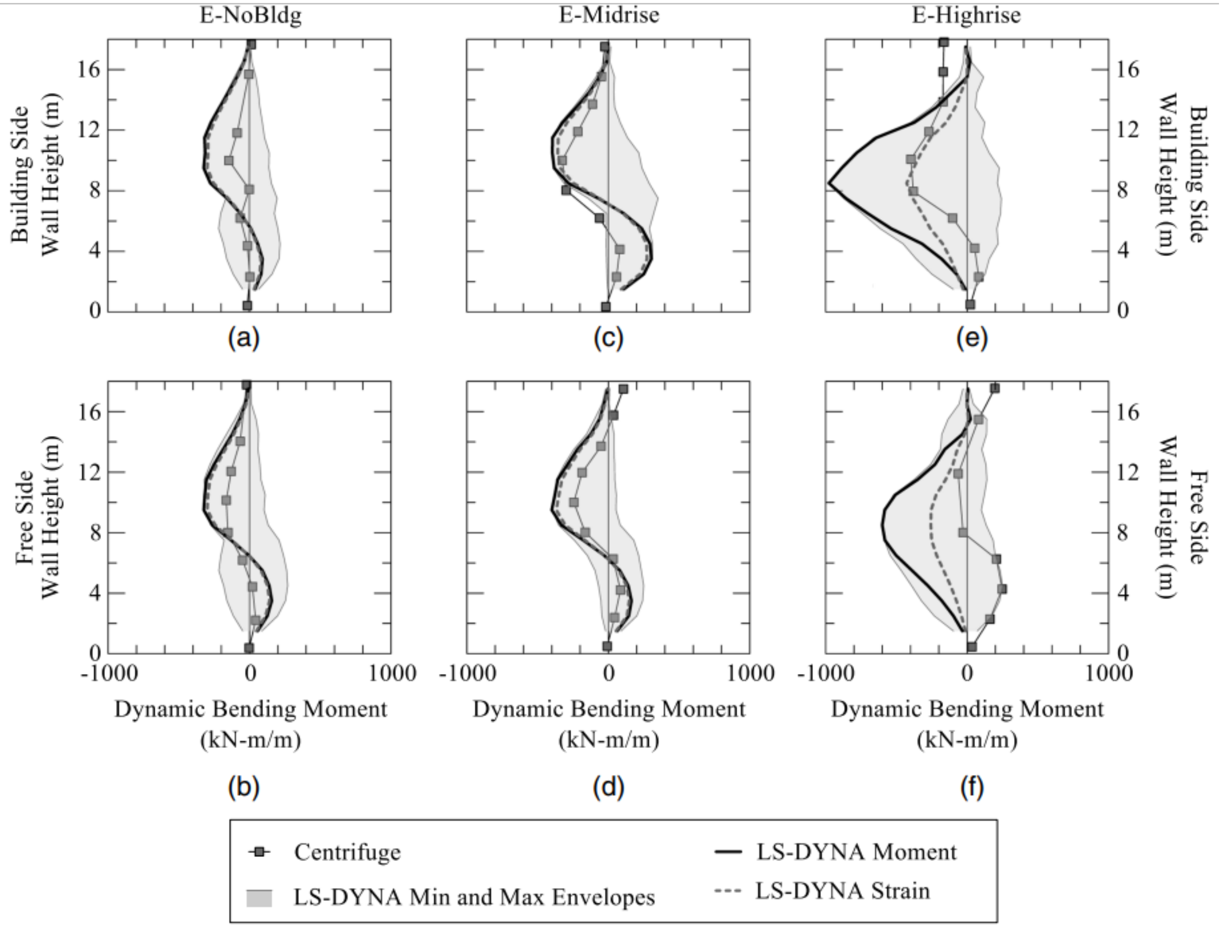
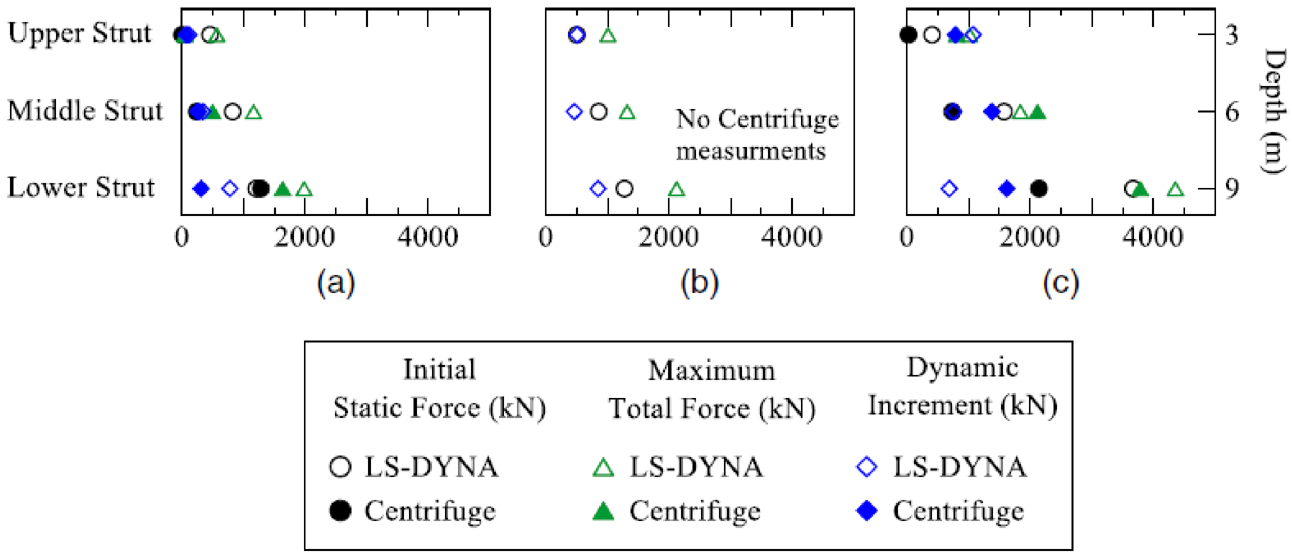
Validation and Benchmarks – SSI

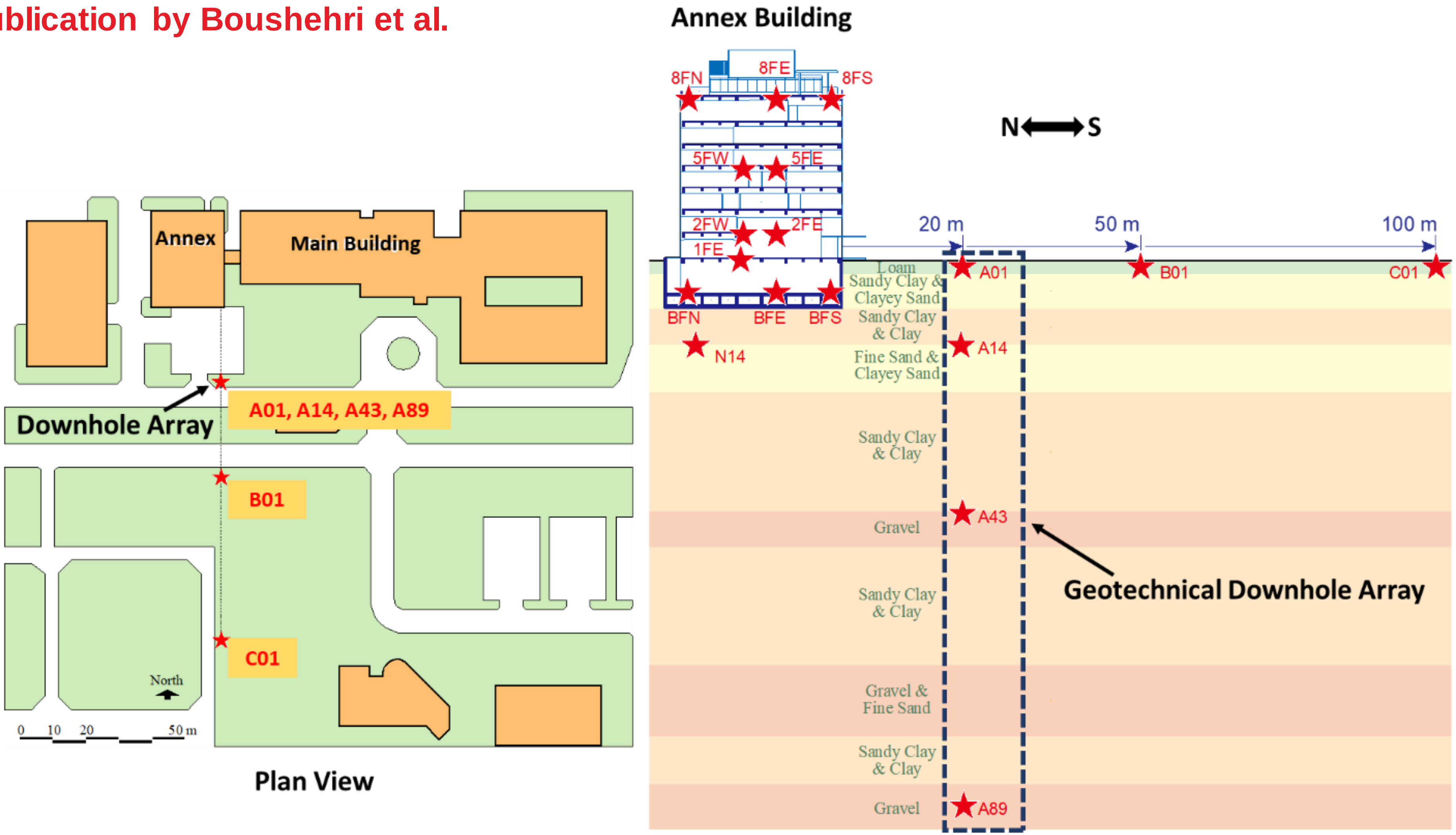
Hashash et al. 2018





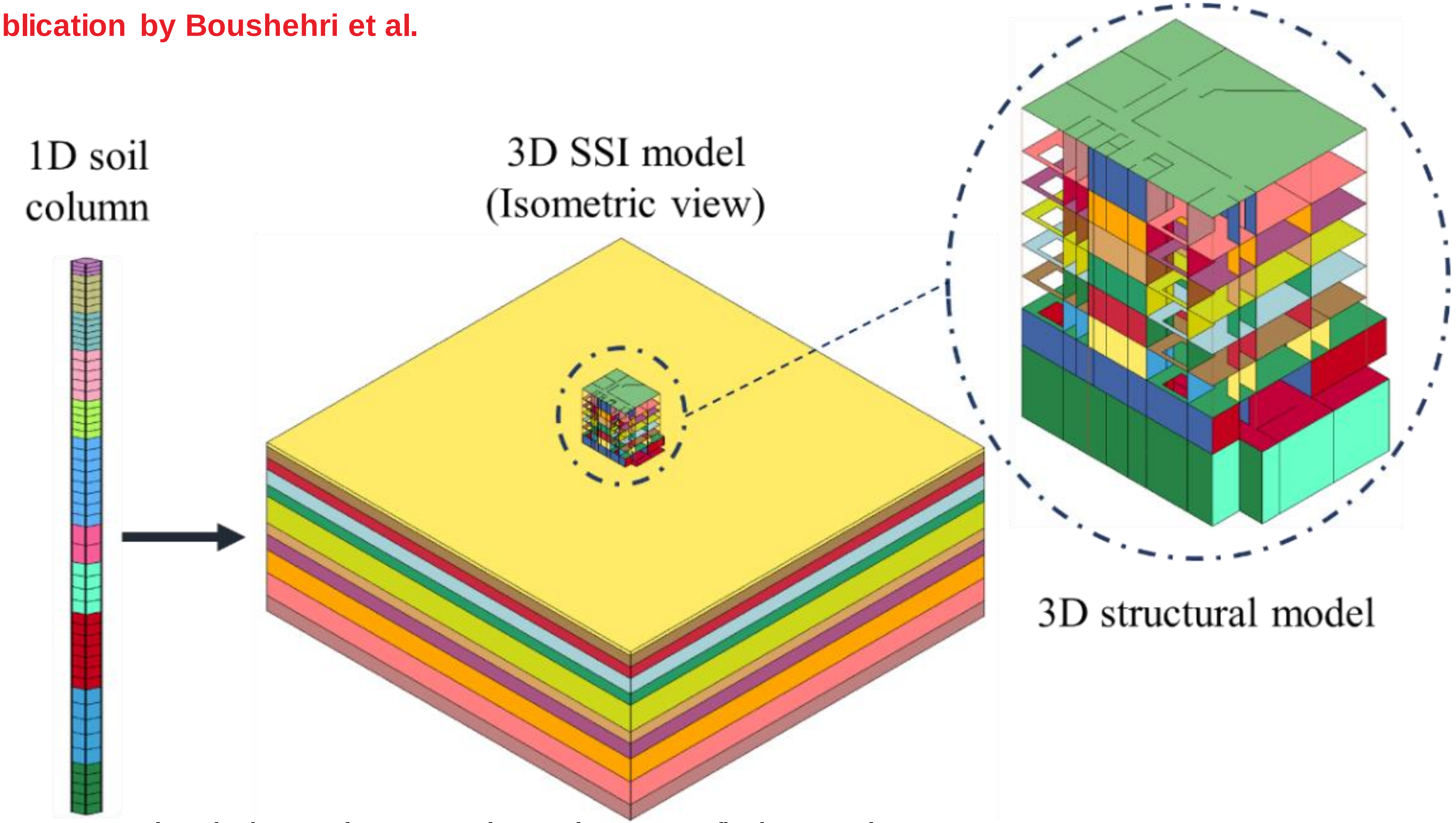
Validation and Benchmarks – SSI

Future Publication by Boushehri et al.



Validation and Benchmarks – SSI

Future Publication by Boushehri et al.



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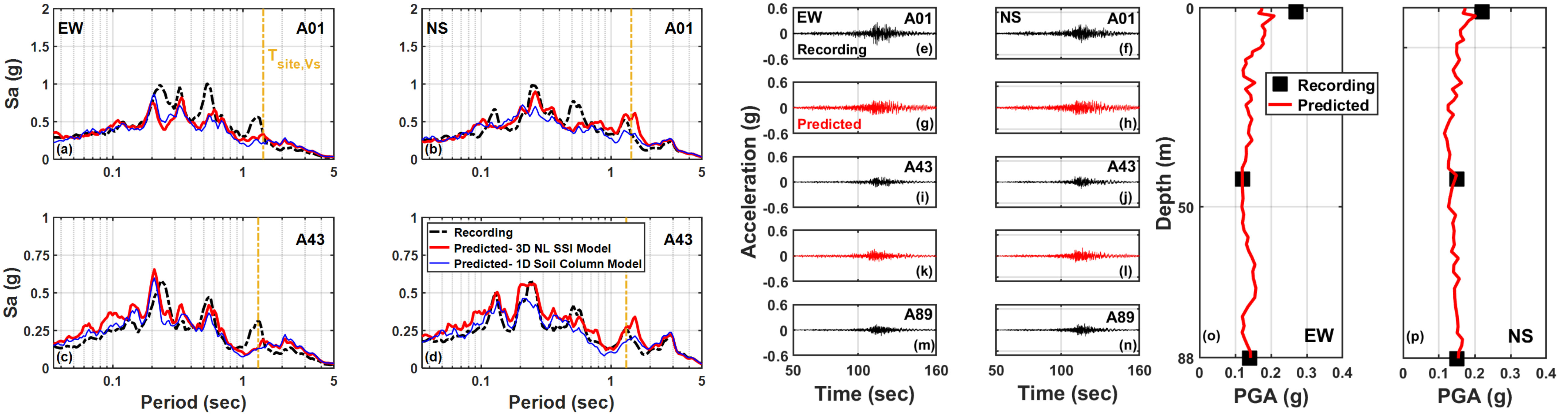
Validation and Benchmarks – SSI

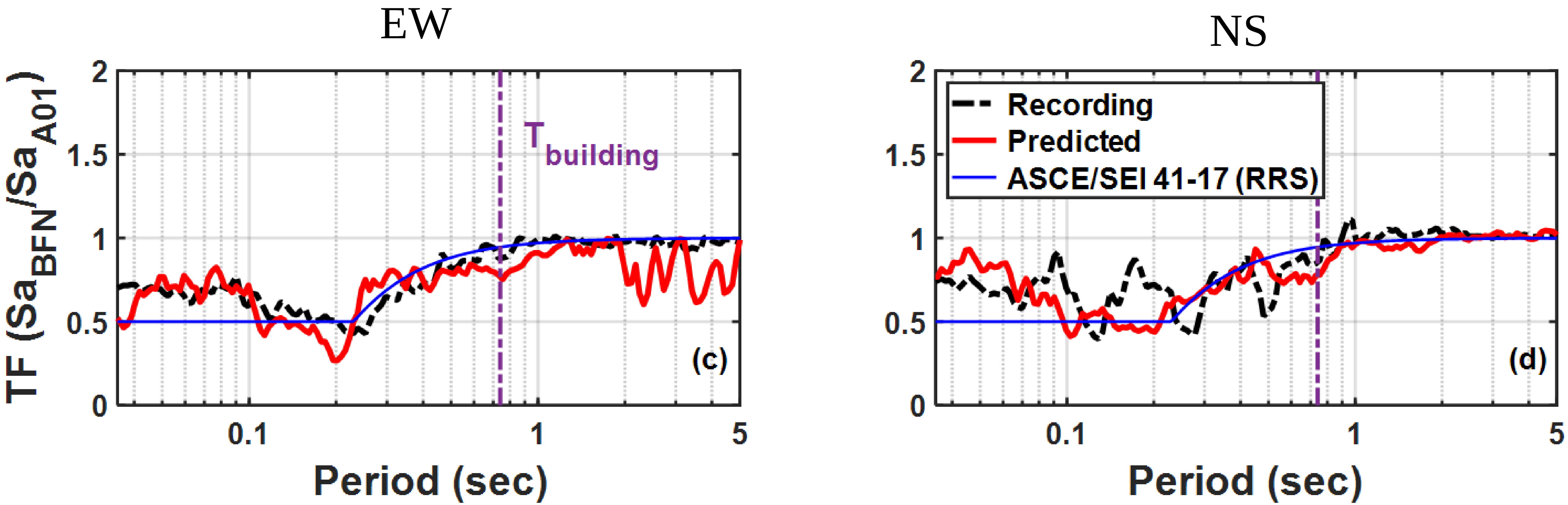
Future Publication by Boushehri et al.



Note: good match for mid-field soil behavior

Validation and Benchmarks – SSI

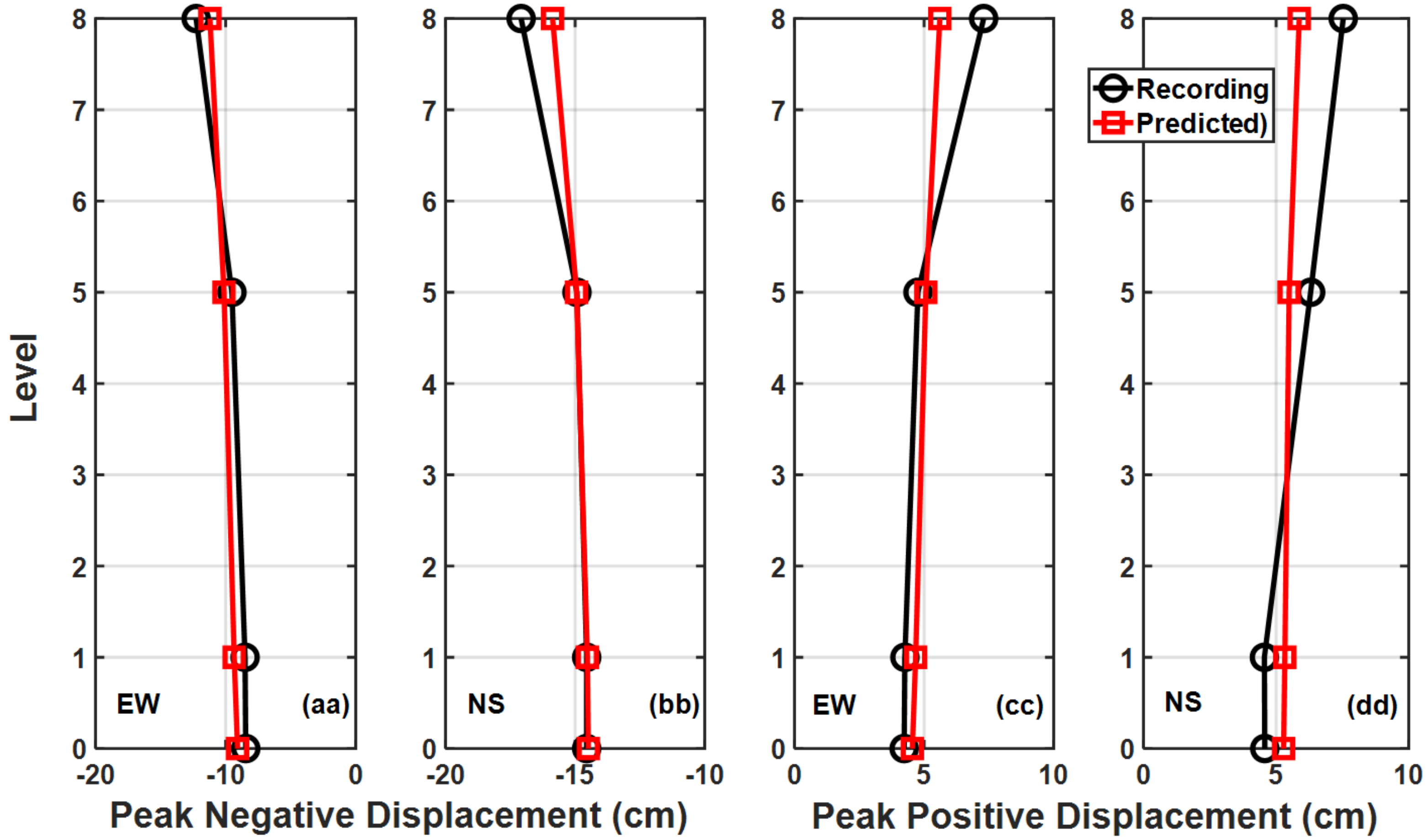
Future Publication by Boushehri et al.



Note: good match to expected kinematic effect on ground motion

Validation and Benchmarks – SSI

Future Publication by Boushehri et al.



Note: improved calibration of superstructure needed

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LS-DYNA *MAT_HYSTERETIC_SOIL (*MAT_079)

Session 3 - Project examples

Richard Sturt, Kirk Ellison, Ben Shao

May 2022

ARUP

Project Examples

A Typical Methodology for Performing Seismic SSI Analysis

1) Perform site characterization, with particular emphasis on:
   - Shear wave velocity vs depth
   - Shear strength vs depth
   - Index Parameters (OCR, K0, PI, unit weight, etc.)
   - Other characteristics known to be important for the regional soil strata (e.g. strength degradation, strain rate effects, nonlinear stiffness and damping characteristics, etc.)
2) Perform uni-directional SRA in DEEPSOIL for one or more soil columns with the GH/Q soil model
3) Export soil columns from DEEPSOIL to LS-DYNA
4) Update LS-DYNA model (e.g. for horizontal stress, bi-directional shaking) and MAT079 material cards (e.g. for non-Masing damping, pressure-dependent soil properties, strain rate effects, cyclic degradation, bi-directional shaking, etc.) to include pertinent parameters not available from DEEPSOIL.
5) Replicate uni-directional SRA results from DEEPSOIL in LS-DYNA
6) Extrapolate the SRA profiles to a 3D soil domain and carry out 3D SRA, 3D KSSI and/or 3D SSI analysis
7) Validate far-field soil behavior in the 3D model via comparison to SRA results

A Typical Methodology for Performing Seismic SSI Analysis

Project Examples

2 Rankin Shaft

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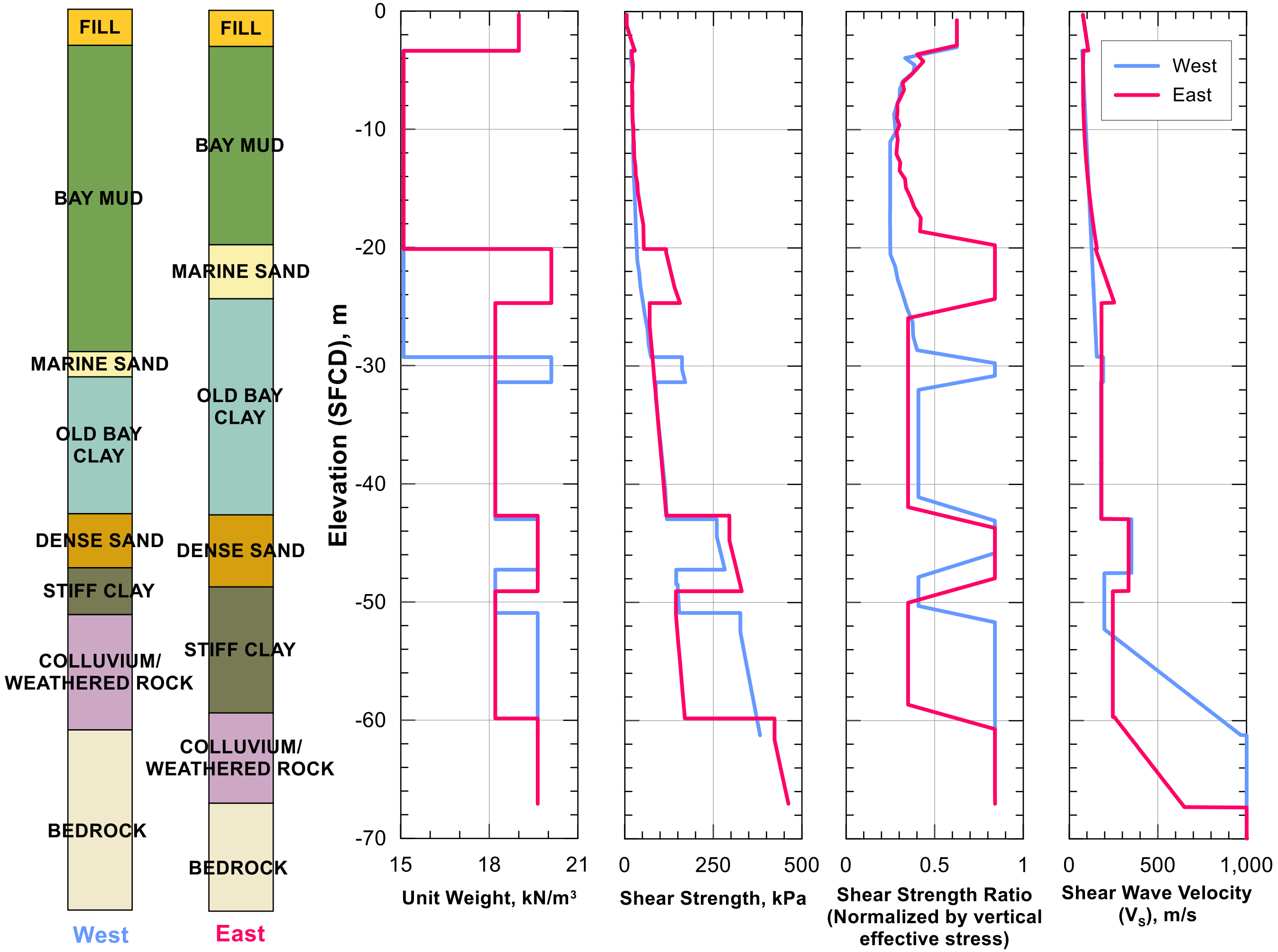
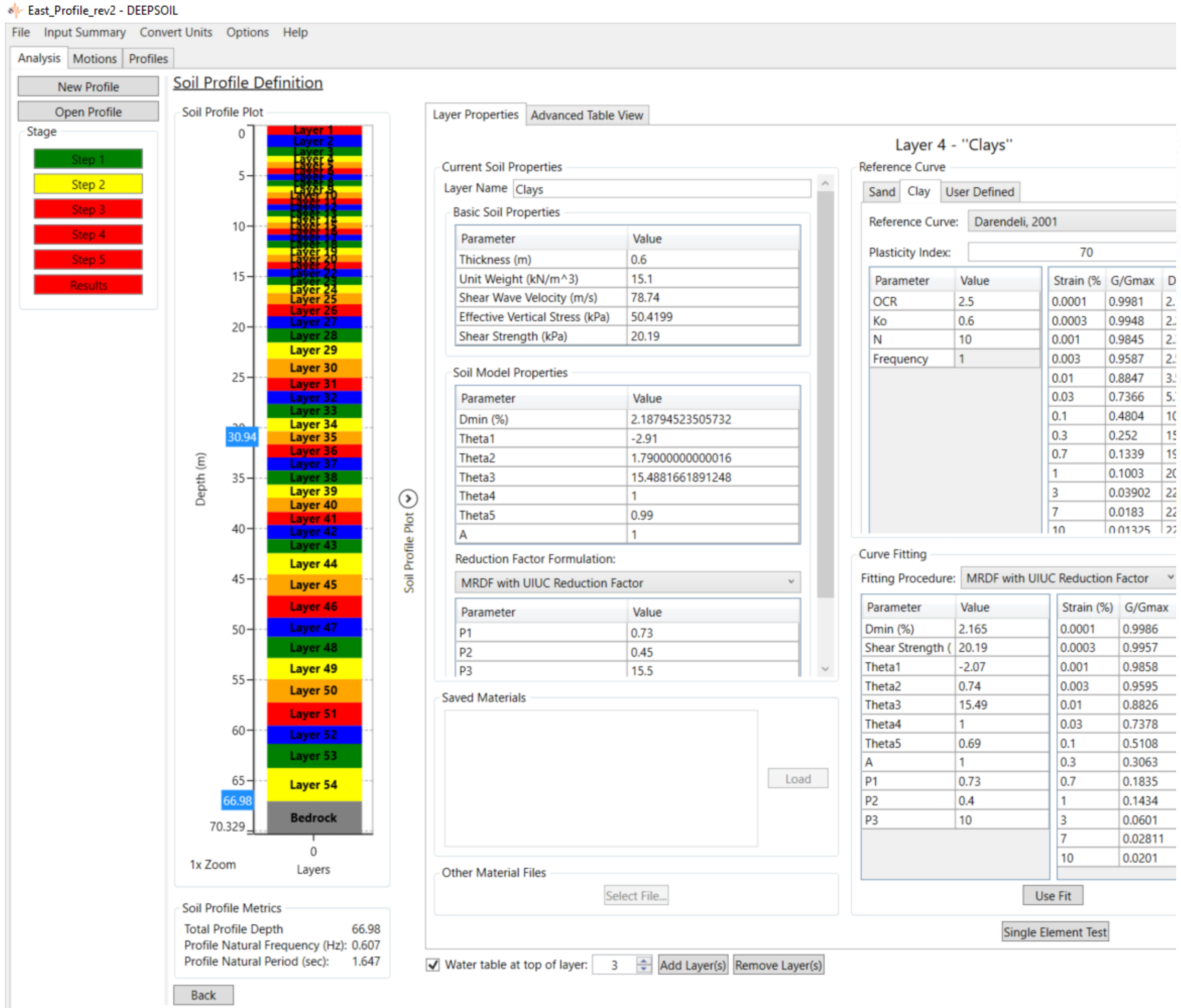
Project Examples

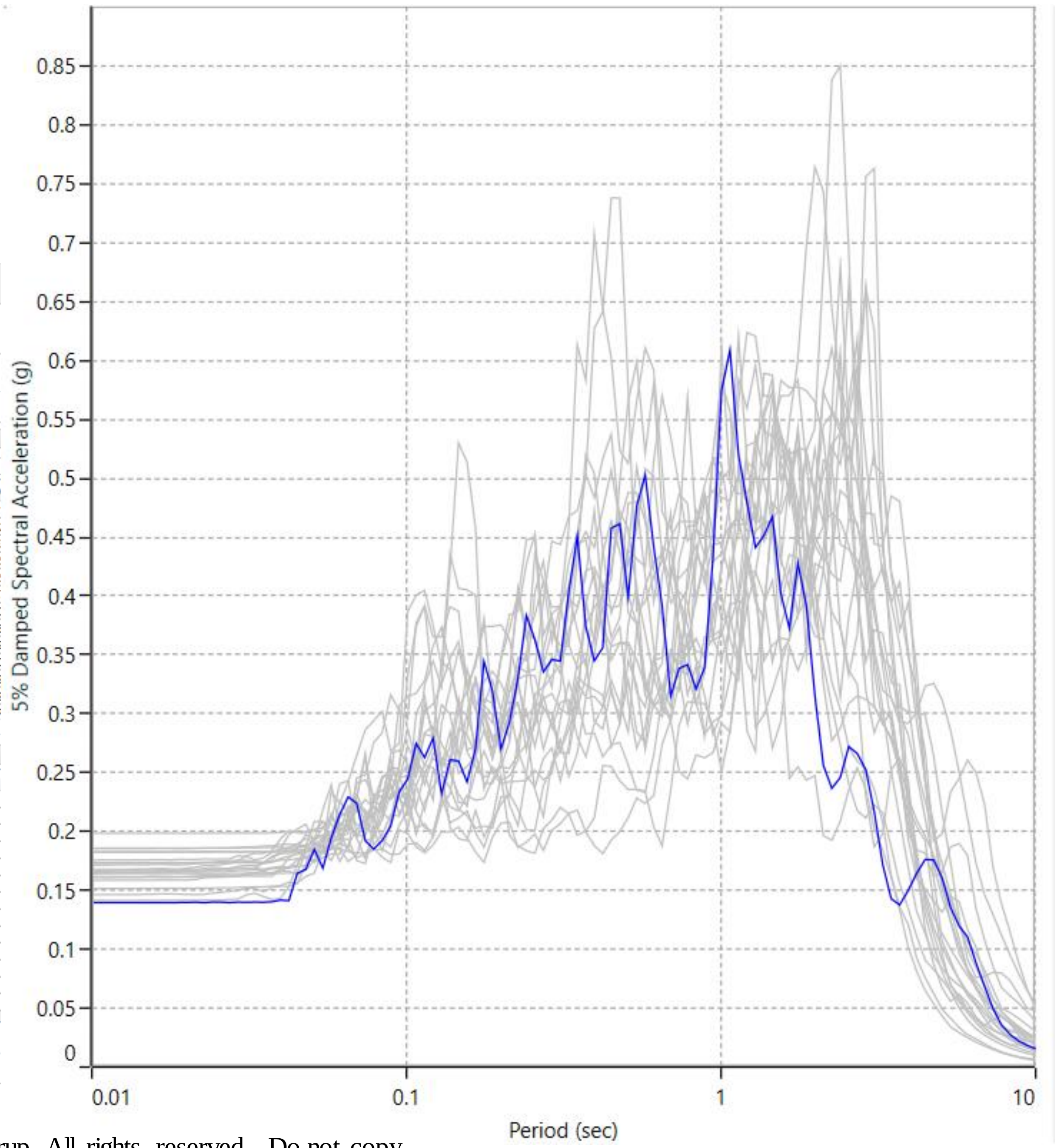
2 Rankin Shaft – Step 1: Site Characterization



Project Examples

2 Rankin Shaft – Step 2: SRA in DEEPSOIL





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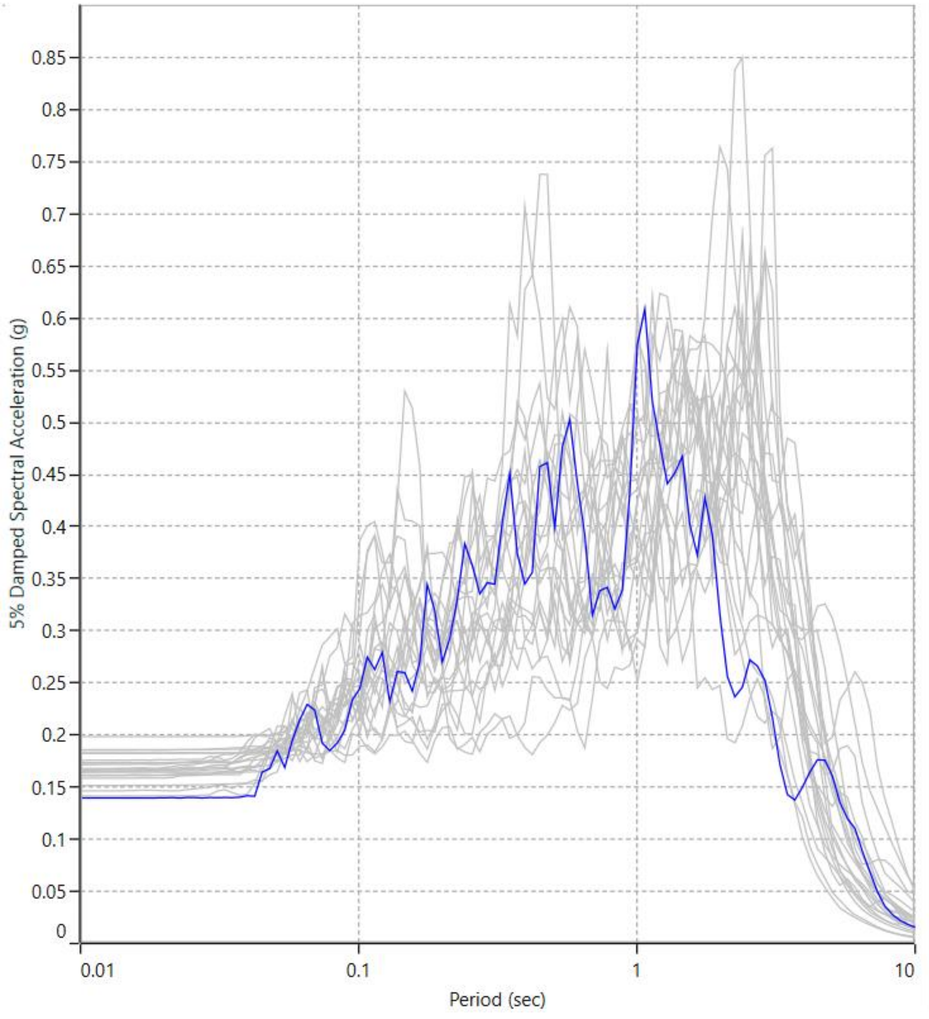
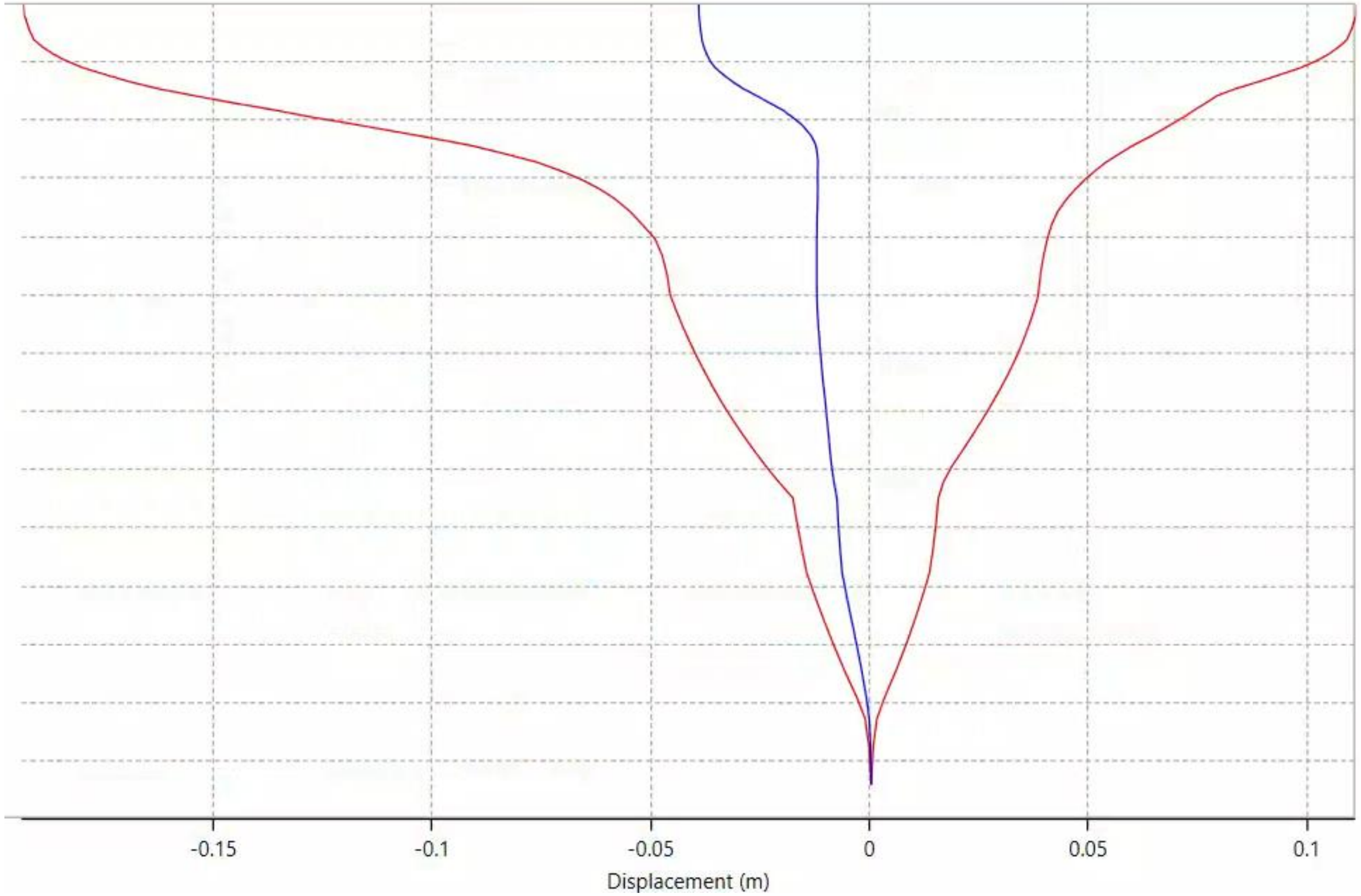
Project Examples

2 Rankin Shaft – Step 2: SRA in DEEPSOIL





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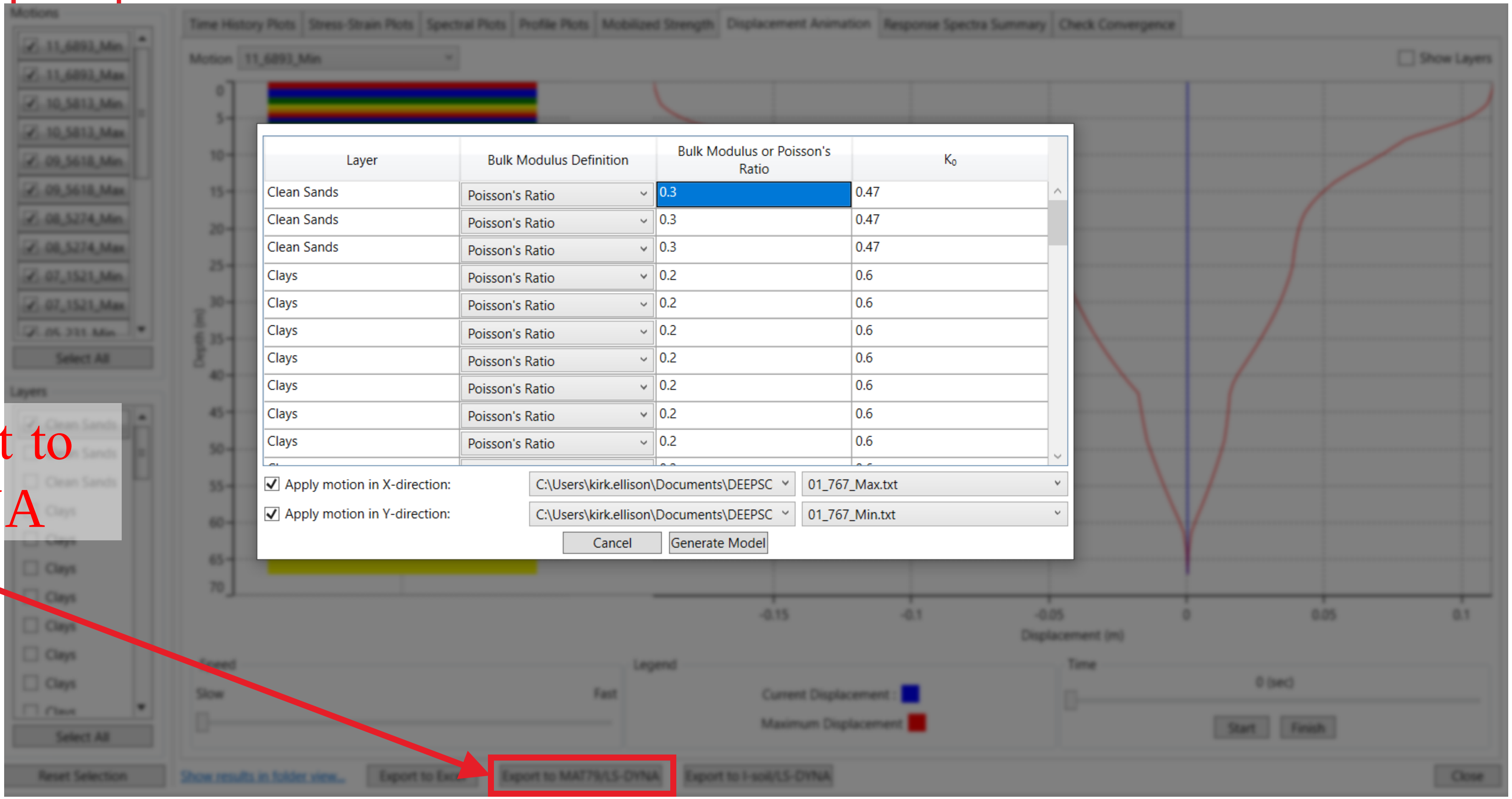
Project Examples

2 Rankin Shaft – Step 3: Export Soil Columns from DEEPSOIL to LS-DYNA



Button to “Export to MAT79/LS-DYNA

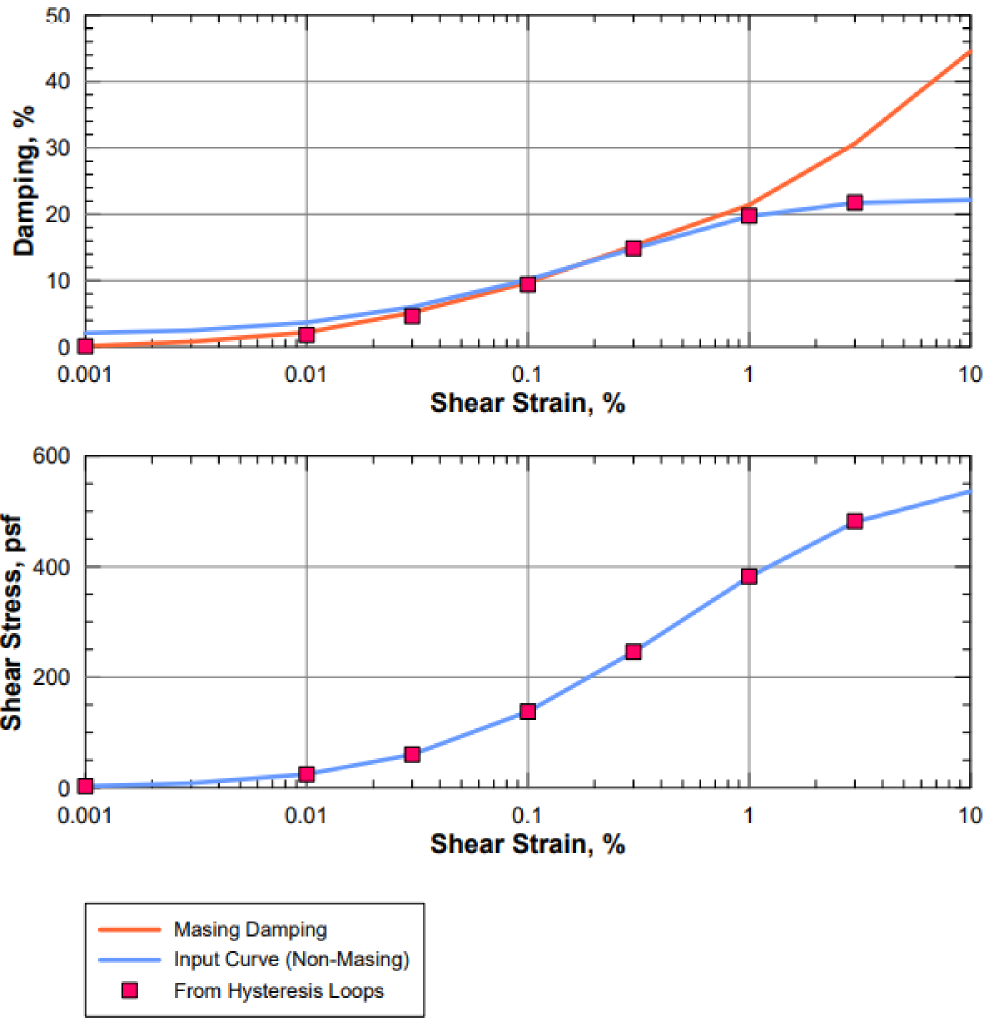
Project Examples

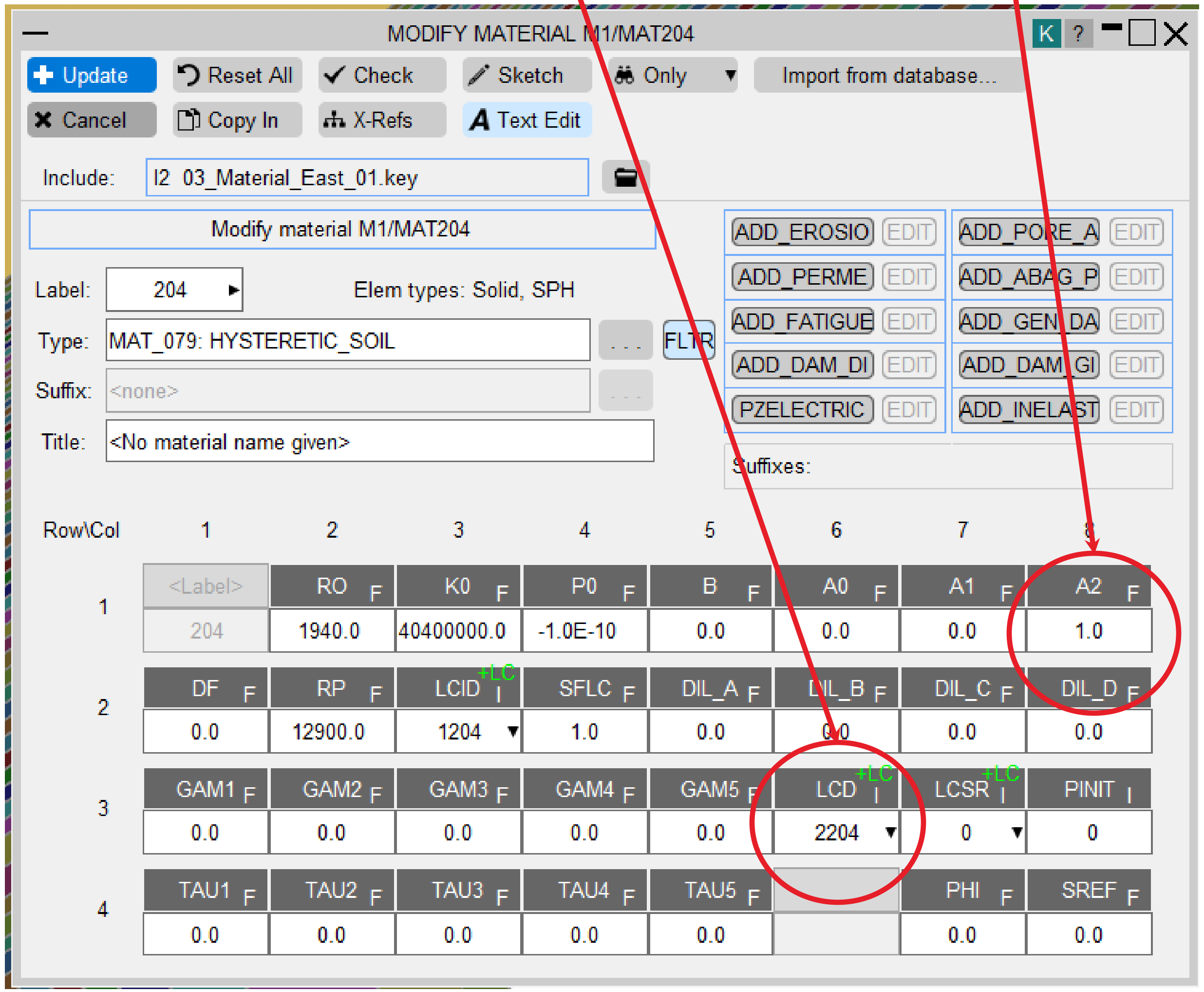
ARUP

2 Rankin Shaft – Step 4: Update LS-DYNA Model with Additional Features

Non-Masing Damping

Pressure-Dependent Strength/Stiffness





Project Examples

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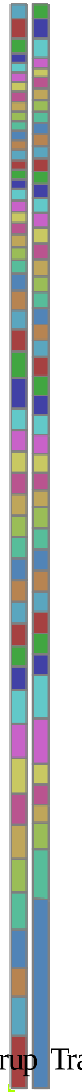
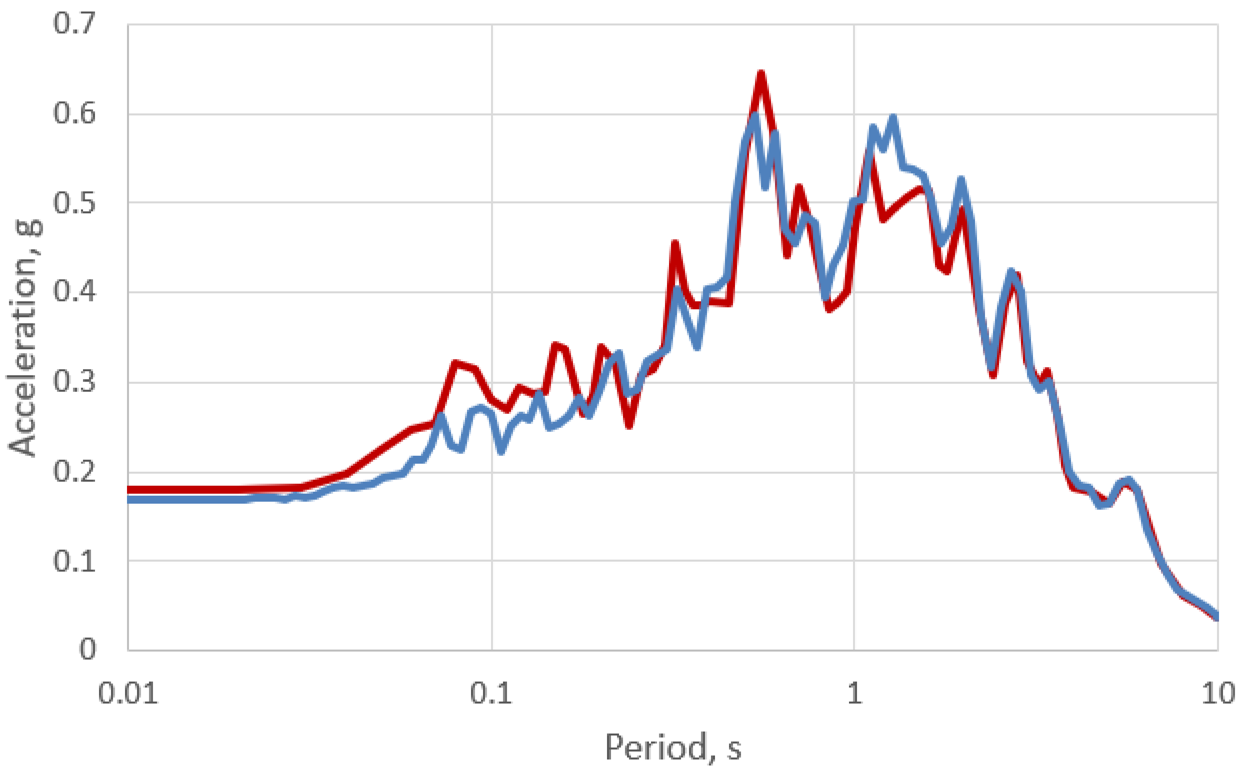
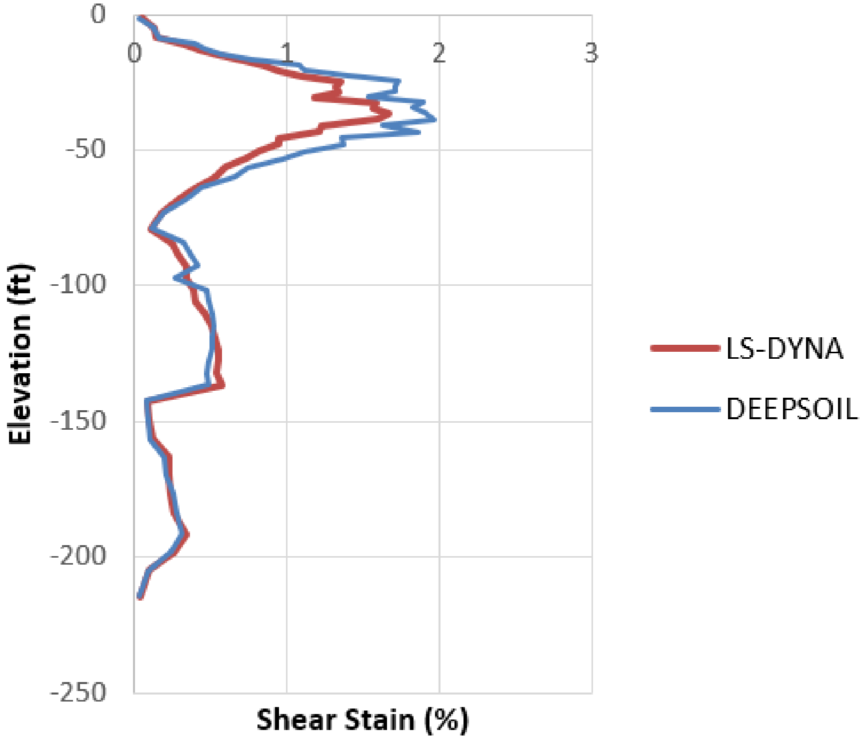
2 Rankin Shaft – Step 6: Compare Uni-Directional SRA in DEEPSOIL and LS-DYNA



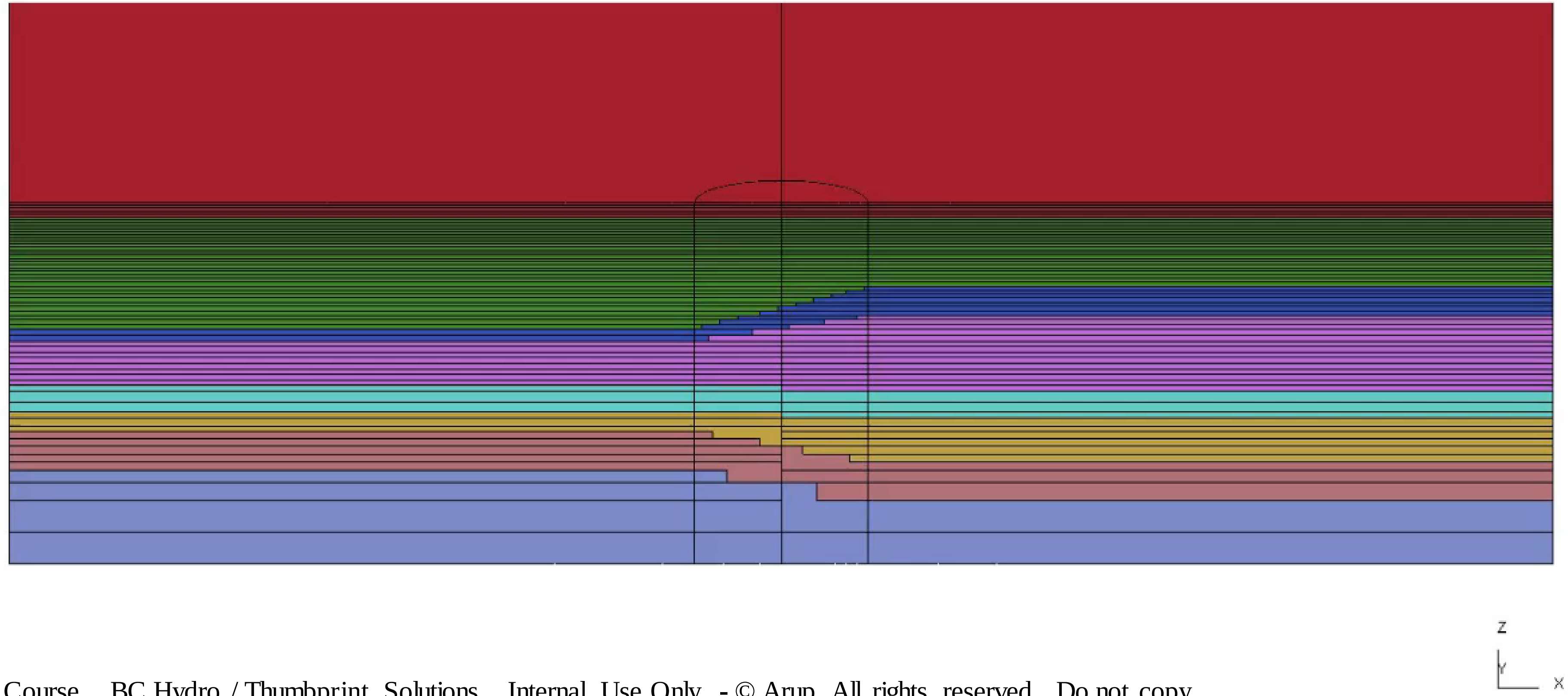
Average of 22 motions (11 orthogonal pairs) for east profile





Project Examples

2 Rankin Shaft – Step 6: Extrapolate SRA Profiles to 3D Soil Domain and Carry out SSI Analysis



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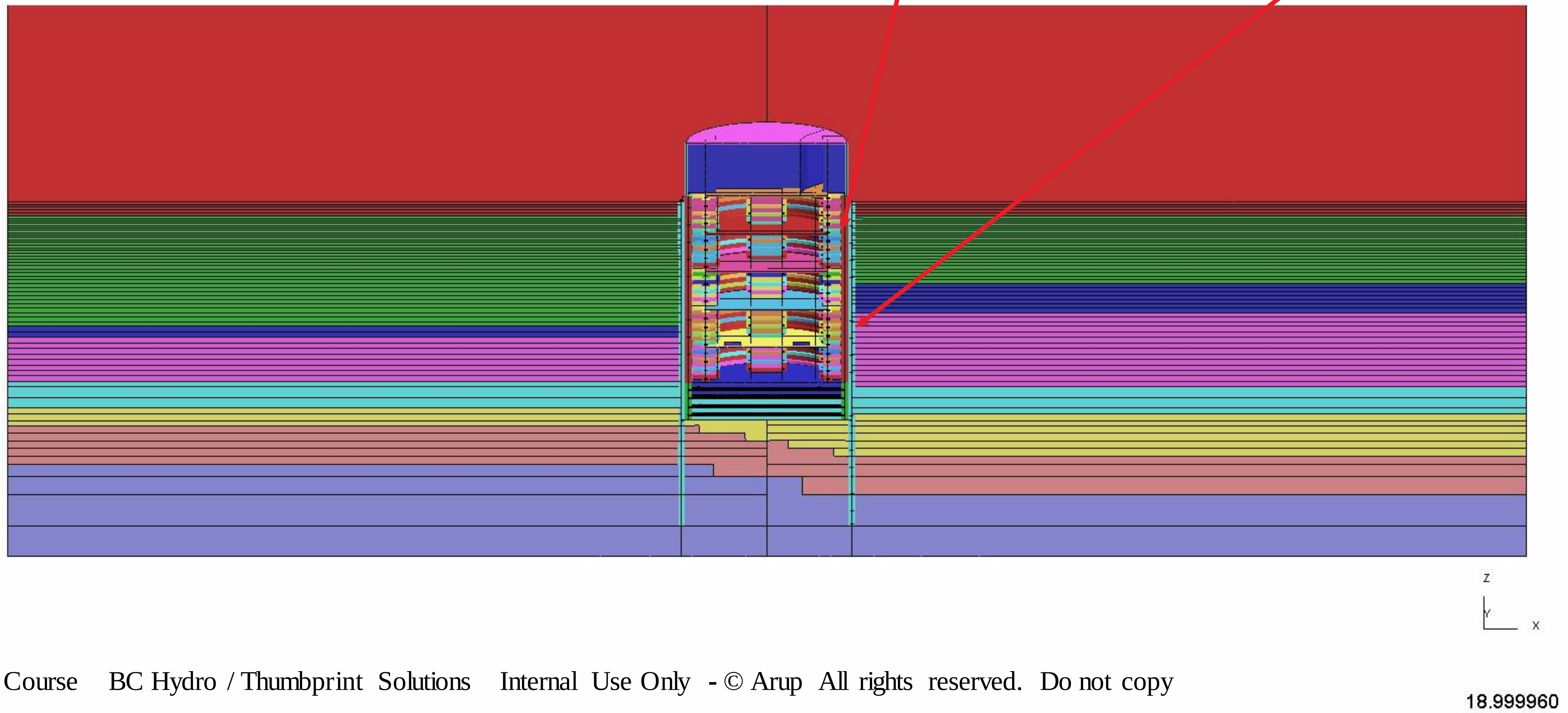
Project Examples

2 Rankin Shaft – Step 6: Extrapolate SRA Profiles to 3D Soil Domain and Carry out SSI Analysis

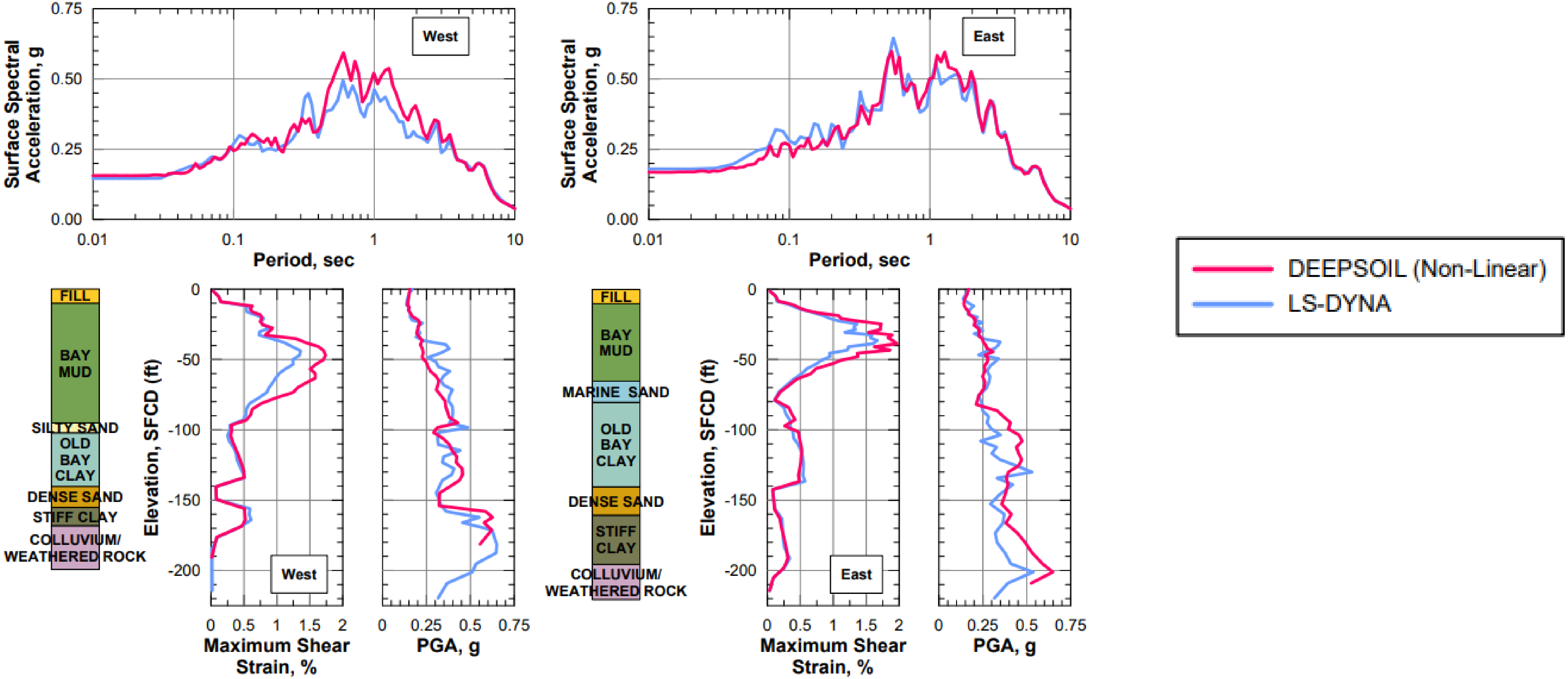
Basement-Diaphragm Wall Contact modeled via AUTOMATIC_SURFACE_TO_SURFACE

Diaphragm Wall Meshed Into Soil



Project Examples

2 Rankin Shaft – Step 7: Validate Far Field Soil Behavior



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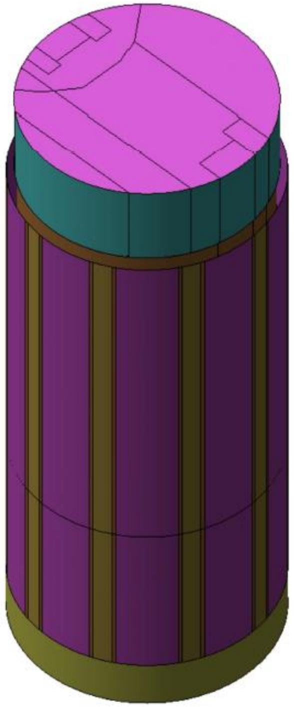
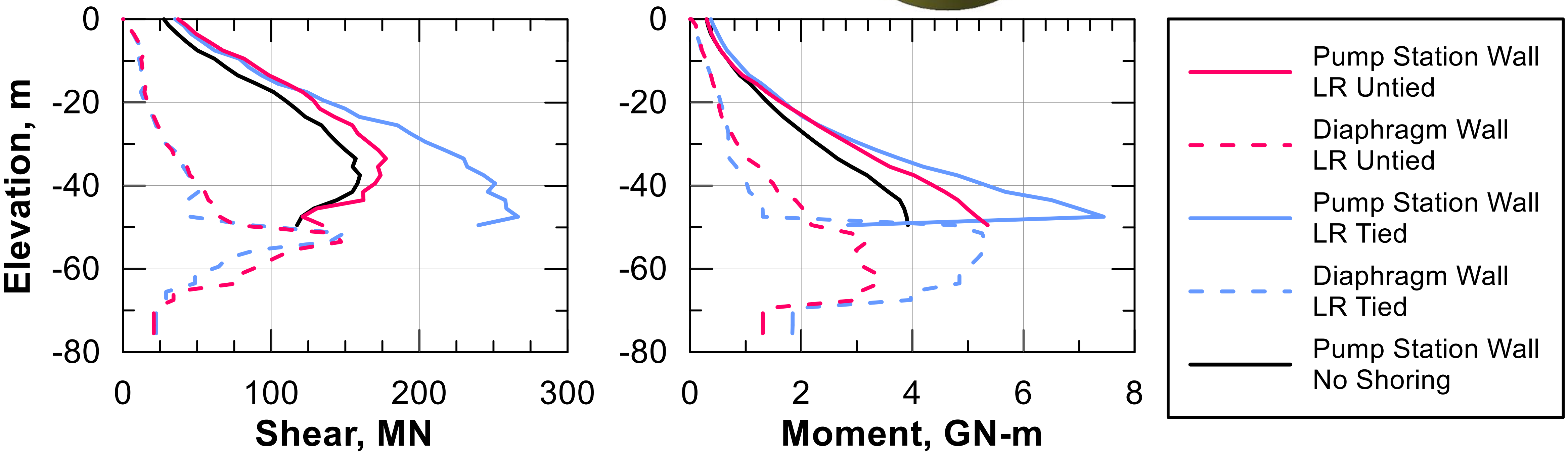
Project Examples

2 Rankin Shaft – Study Findings



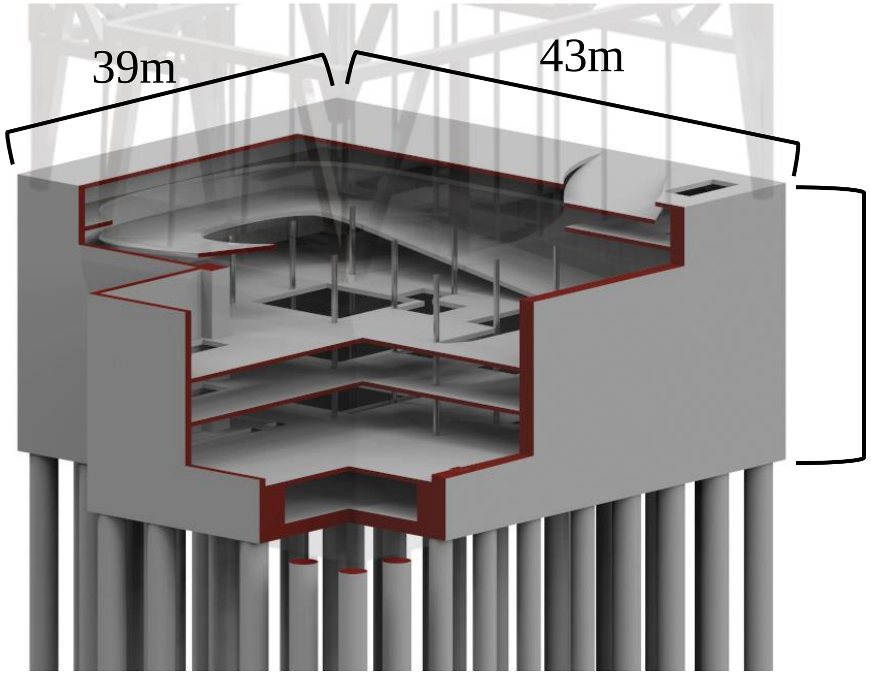


Project Examples

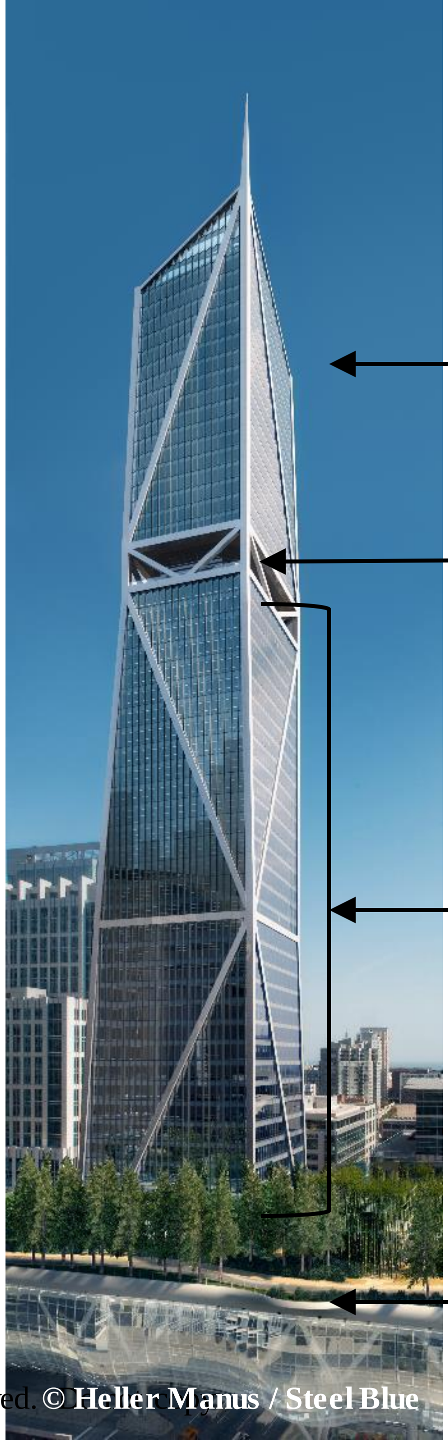
181 Fremont

Project Examples

181 Fremont – Building Specifications



5 levels, 18m deep



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245m tall, 56 stories

Between Level 39 and 55
Condominiums

Amenity Level
Open air terrace

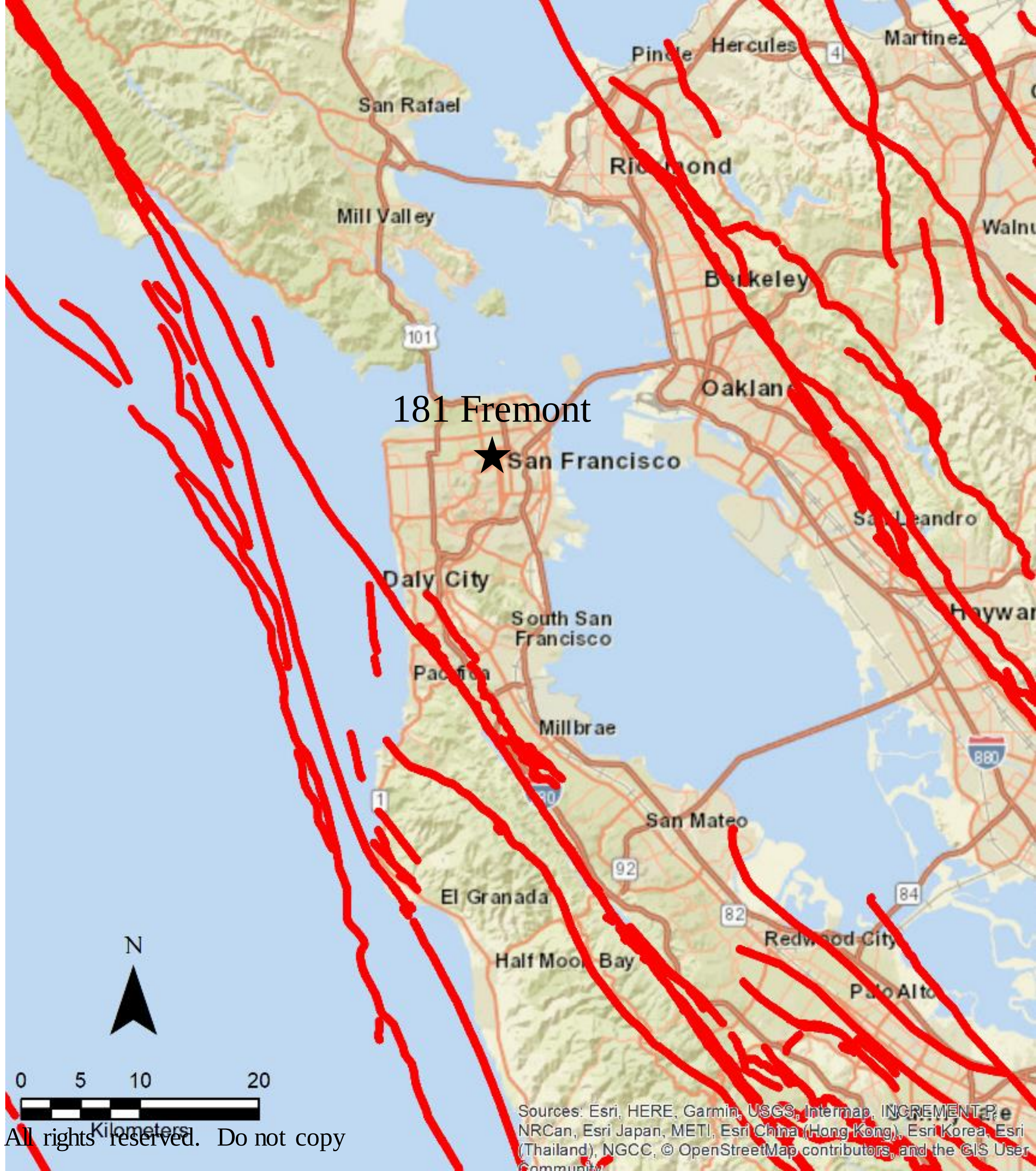
Between Level 2 and 36
Office space

Immediately Adjacent to
Salesforce Transit Center

Project Examples

181 Fremont – Key Issues

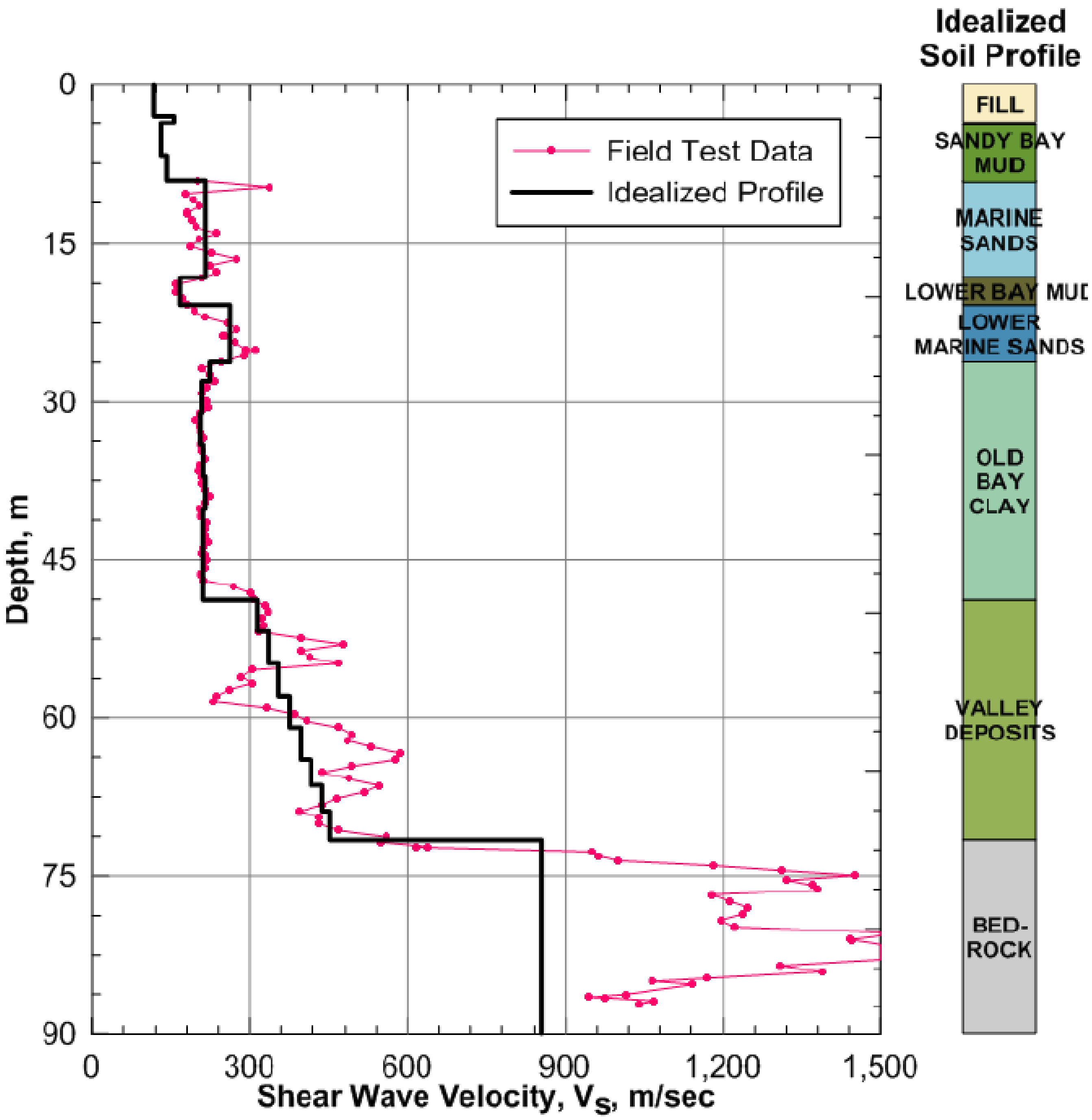
- High seismicity
- Deep clay soils
- Adjacent structures



Project Examples

181 Fremont – Key Issues

- High seismicity
- Deep clay soils
- Adjacent structures



Project Examples

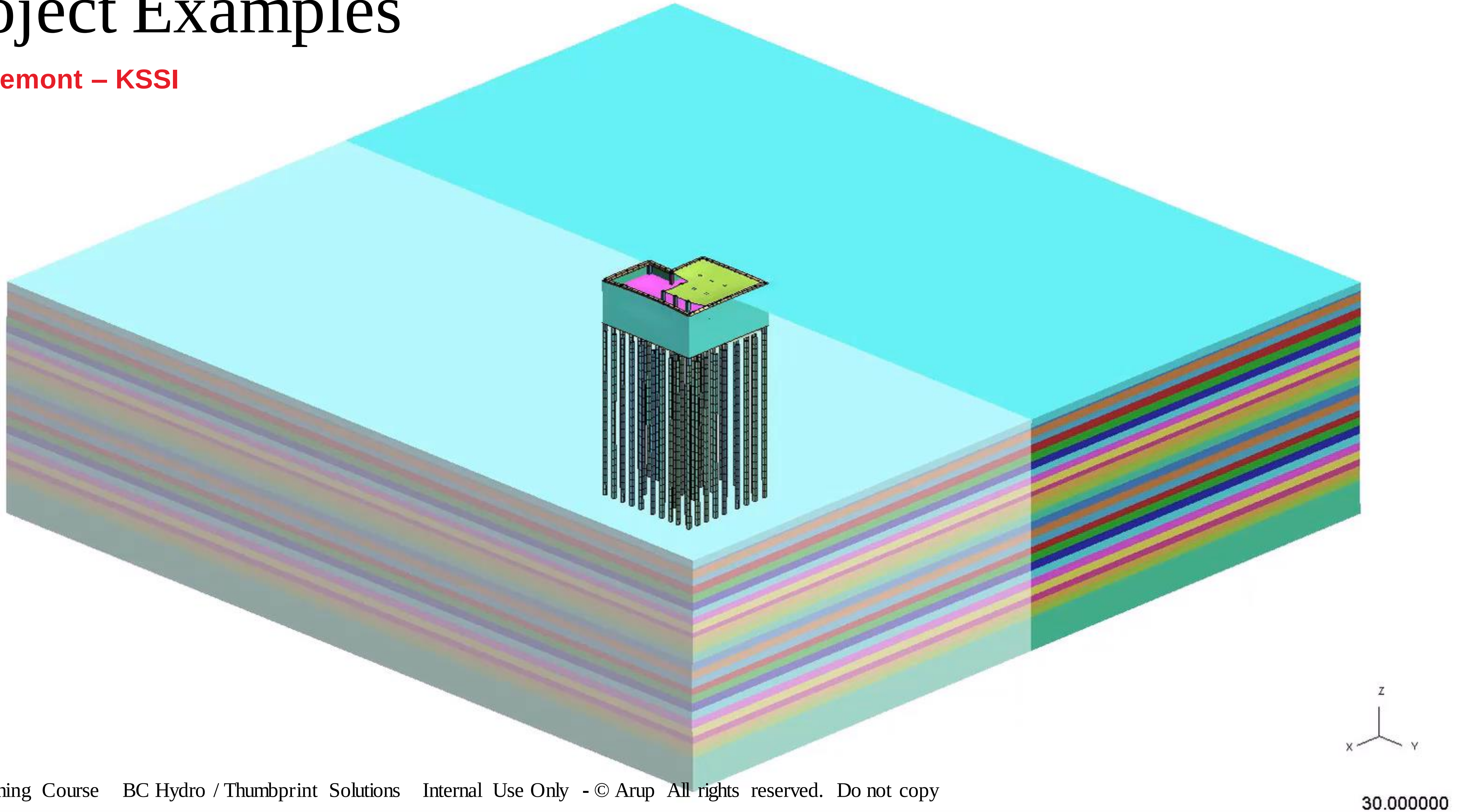
181 Fremont – Key Issues

- High seismicity
- Deep clay soils
- Adjacent structures

181 Fremont

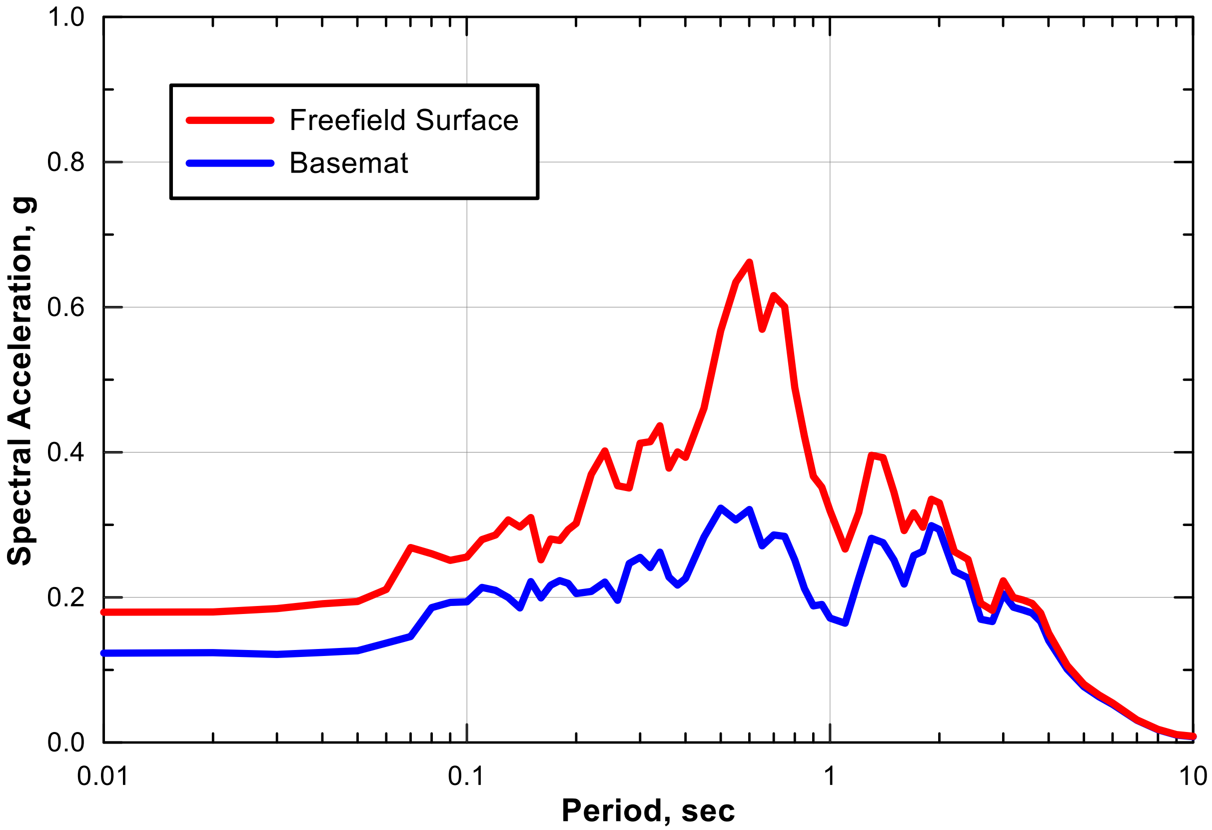
Project Examples

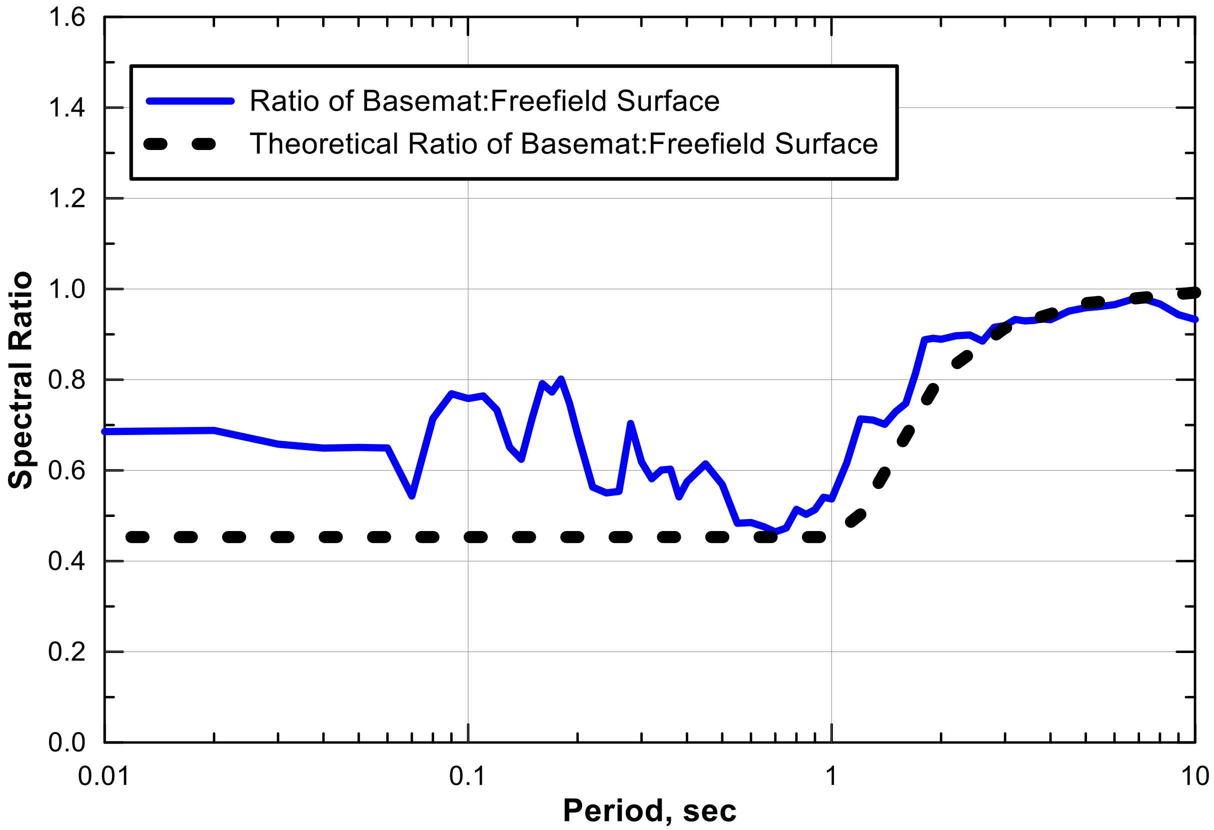
181 Fremont – KSSI



Project Examples

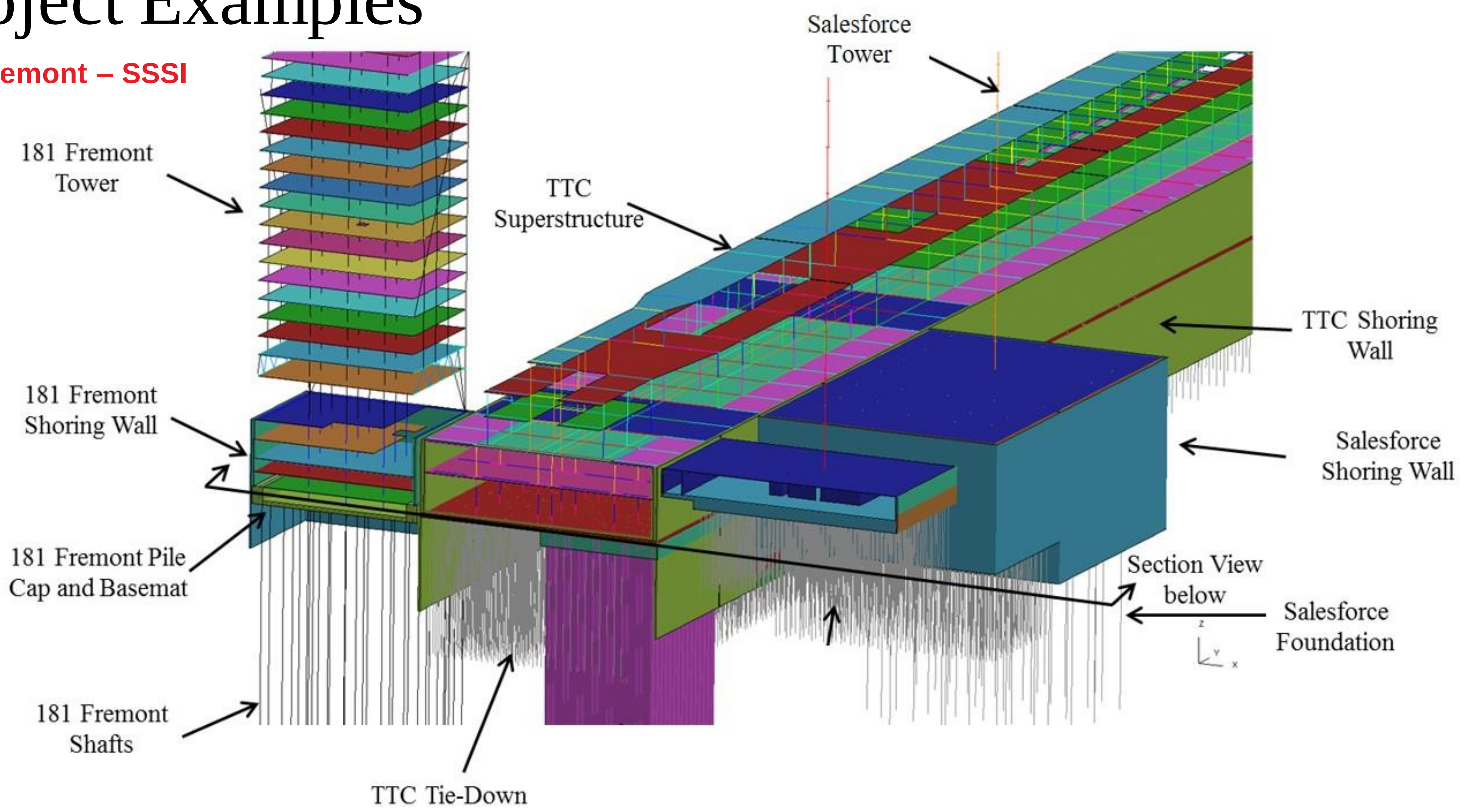
181 Fremont – KSSI





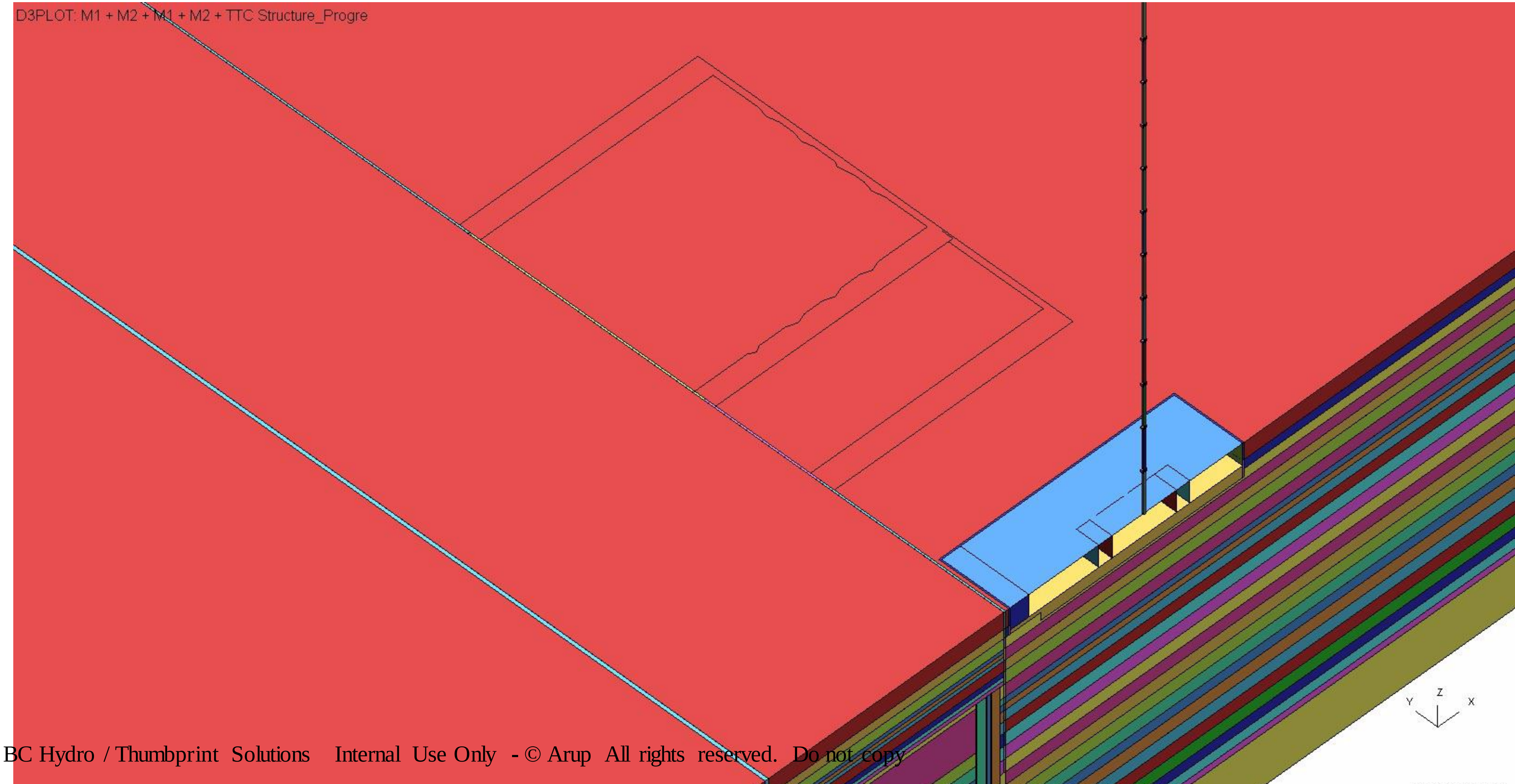
Project Examples

181 Fremont – SSSI



Project Examples

181 Fremont – SSSI



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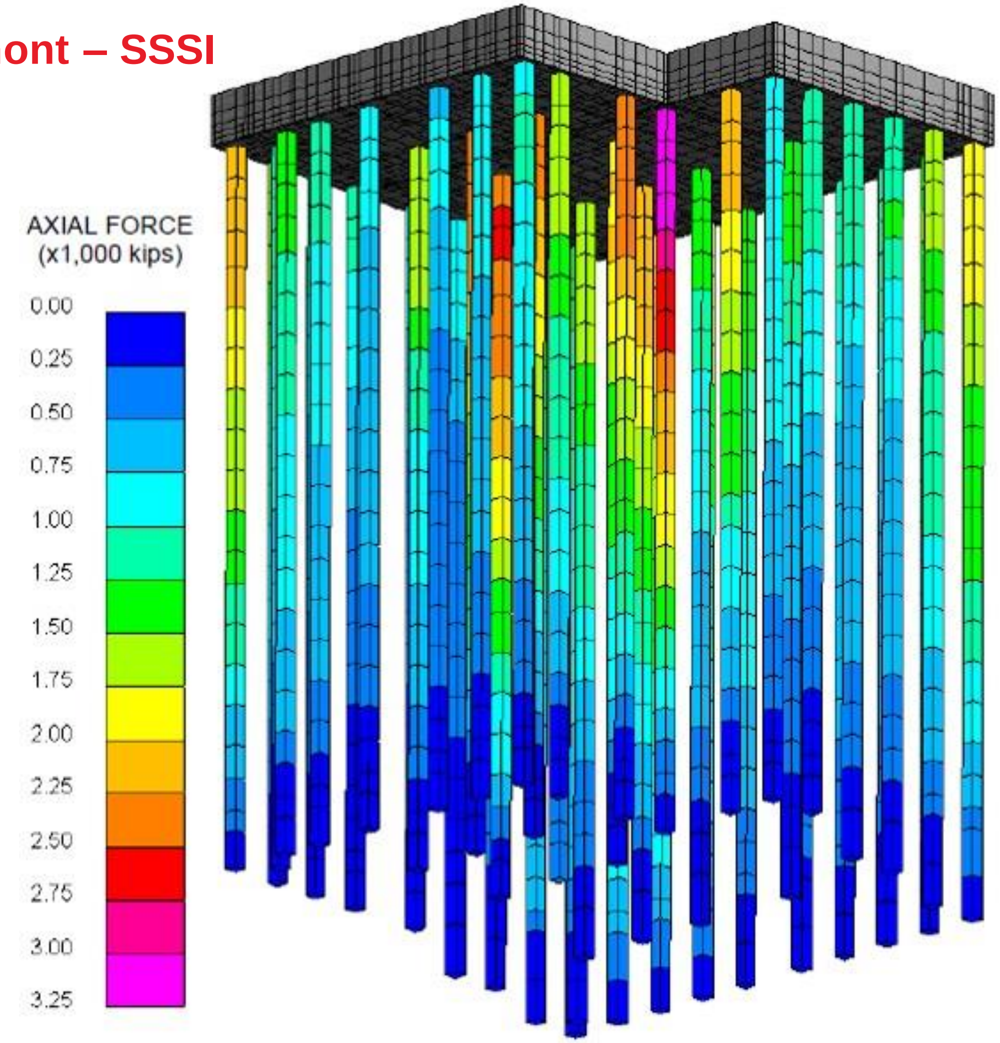
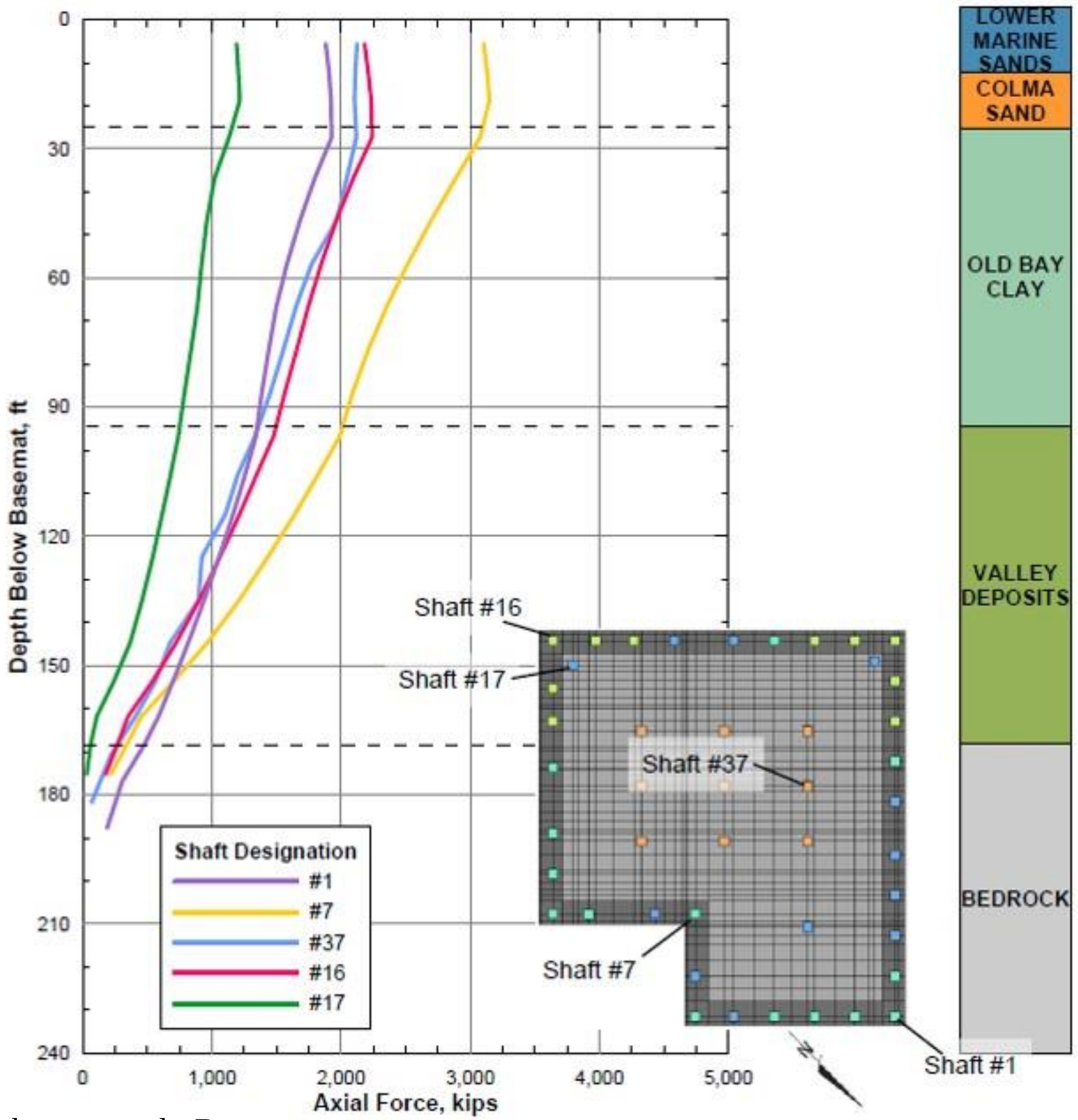
Project Examples

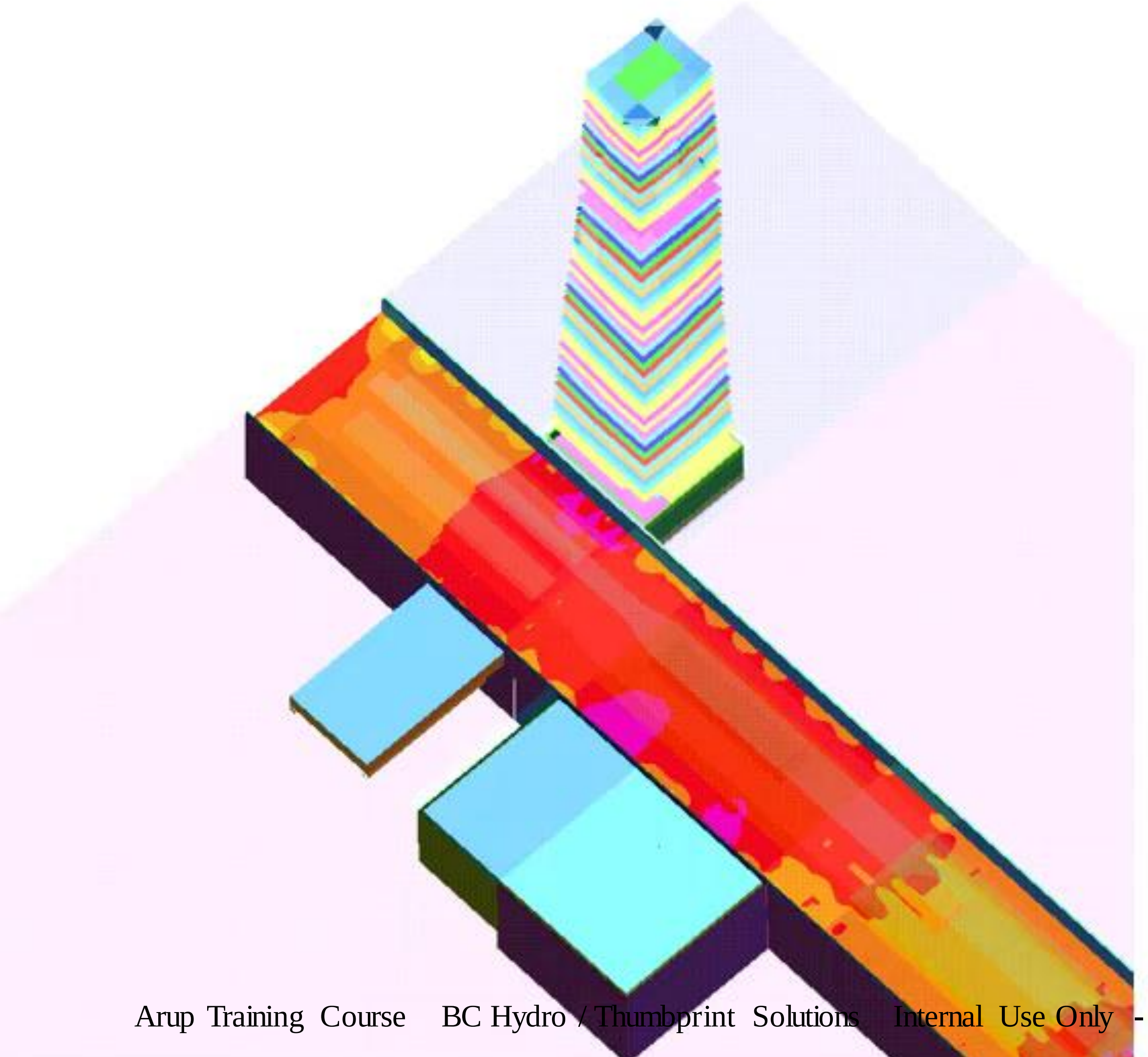
181 Fremont – SSSI

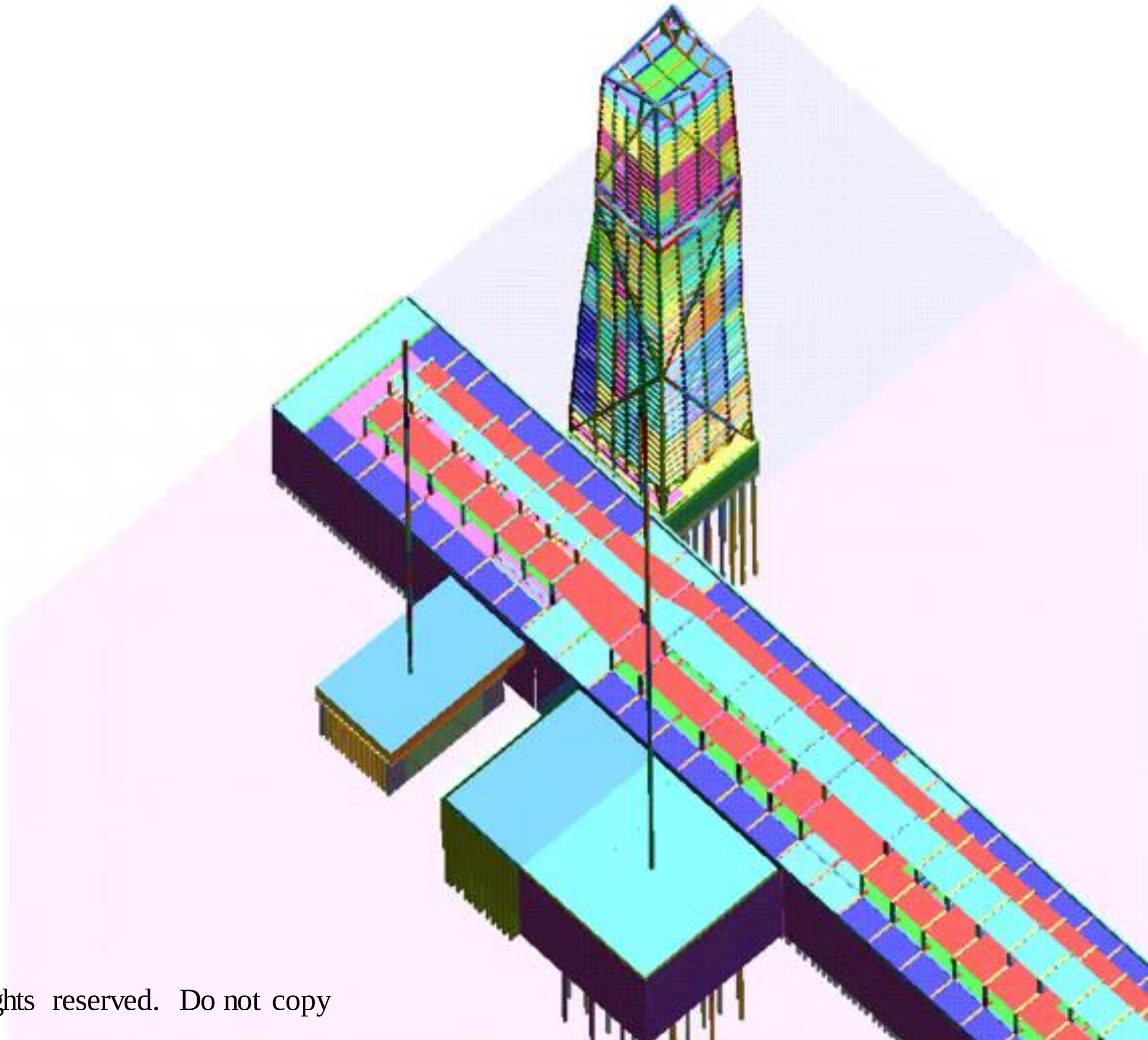




Project Examples

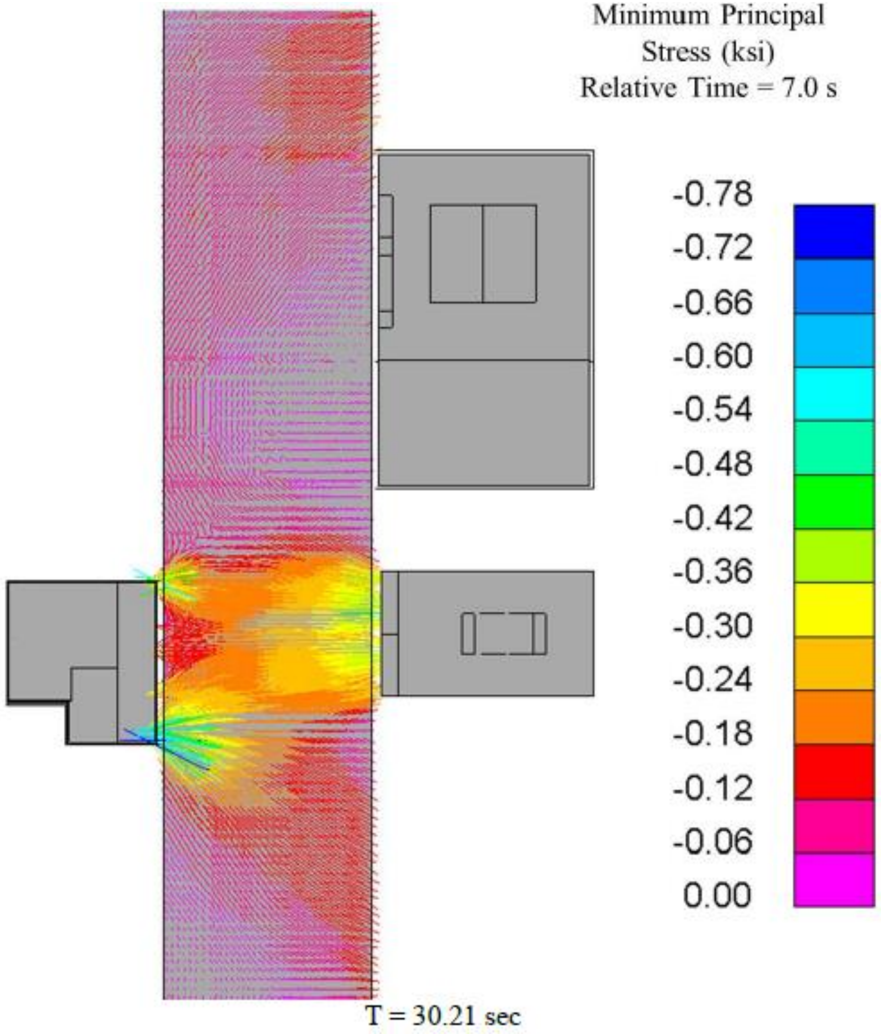
181 Fremont – SSSI

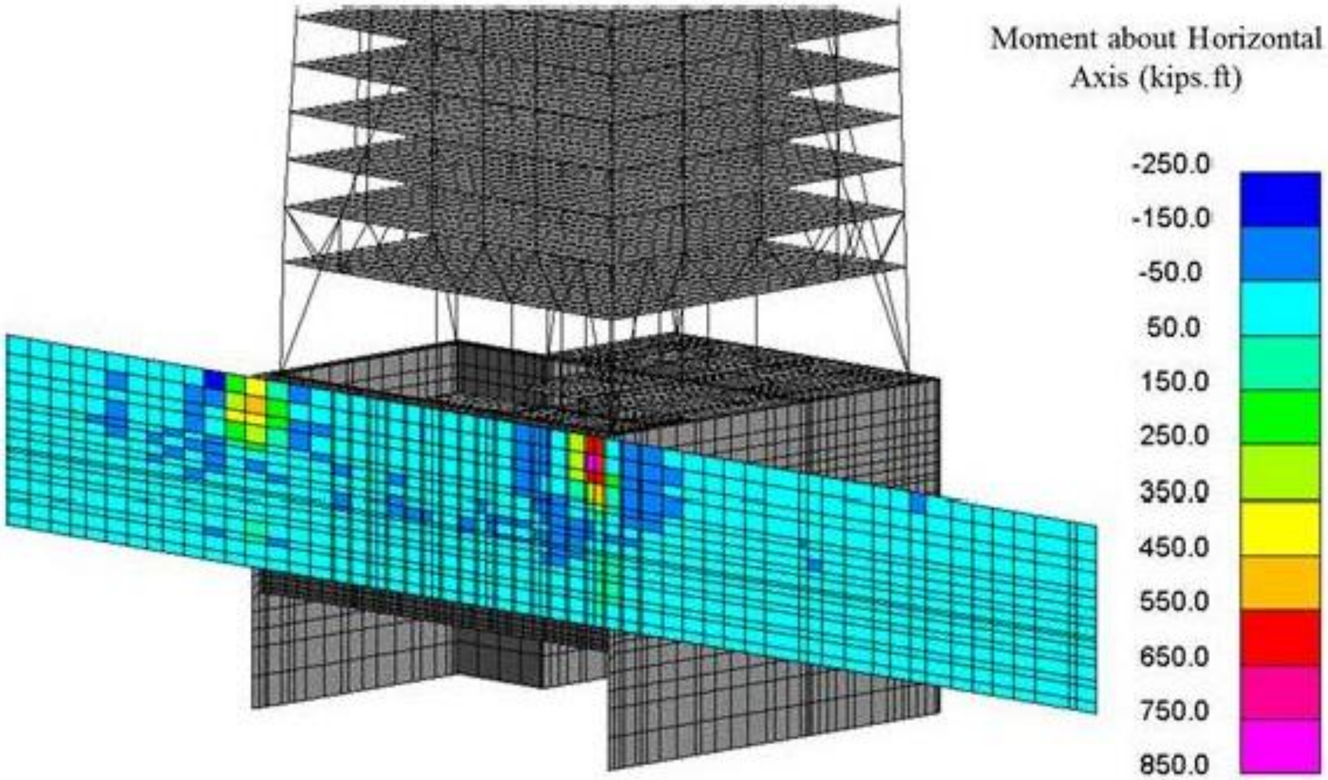


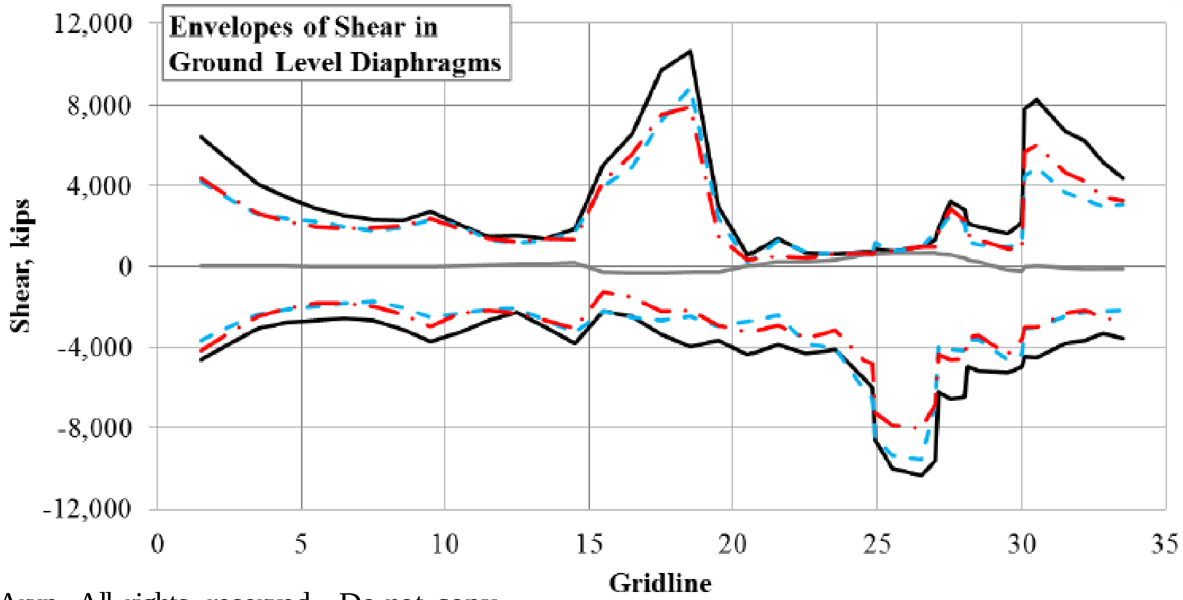


Project Examples

181 Fremont – SSSI







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LS-DYNA *MAT_HYSTERETIC_SOIL (*MAT_079)

Session 4 – Other Soil Models

Richard Sturt, Kirk Ellison, Ben Shao

May 2022

ARUP

LS-DYNA soil materials

Overview

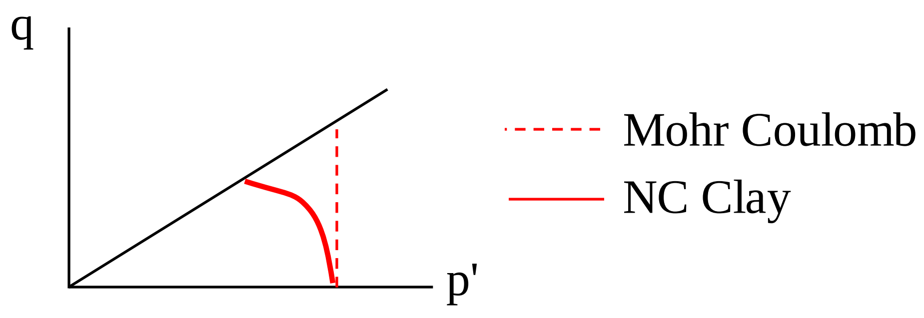
- *MAT_DRUCKER_PRAGER (*MAT_193)
  - Similar to MOHR_COULOMB in respect of limitations; little used in Arup
- *MAT_MOHR_COULOMB (*MAT_173)
  - Simple soil model, elastic-perfectly-plastic; not for seismic analysis.
- ***MAT_HYSTERETIC_SOIL (*MAT_079)**
  - Emphasis on shear hysteresis behavior, non-linear through full range of strain. Seismic applications
- *MAT_SOIL_BRICK (*MAT_192)
  - Over-consolidated clays. Quasi-static applications, foundation design, etc
- *MAT_SOIL_SANISAND
  - New, based on Dafalias/Manzari 2004. Includes dilation/contraction For sands.
  - PM4SAND implementation at developmental stage
- Others not used in Arup include MAT_005, MAT_014, MAT_025, MAT_078, MAT_147

ARUP

LS-DYNA soil materials

***MAT_MOHR_COULOMB**

- Similar to MAT_HYSTERETIC_SOIL with 1 yield surface, i.e.:
  - Elastic-perfectly plastic
  - No concept of critical state or overconsolidation
  - Dilatancy for dense sands can be captured in a crude manner
  - No contraction, so prone to overestimating undrained shear strength based on friction angle



- Option to include elastic anisotropy is available

LS-DYNA soil materials

*MAT_SOIL_SANISAND

- Sanisand is a traditional elasto-plastic critical state bounding surface model

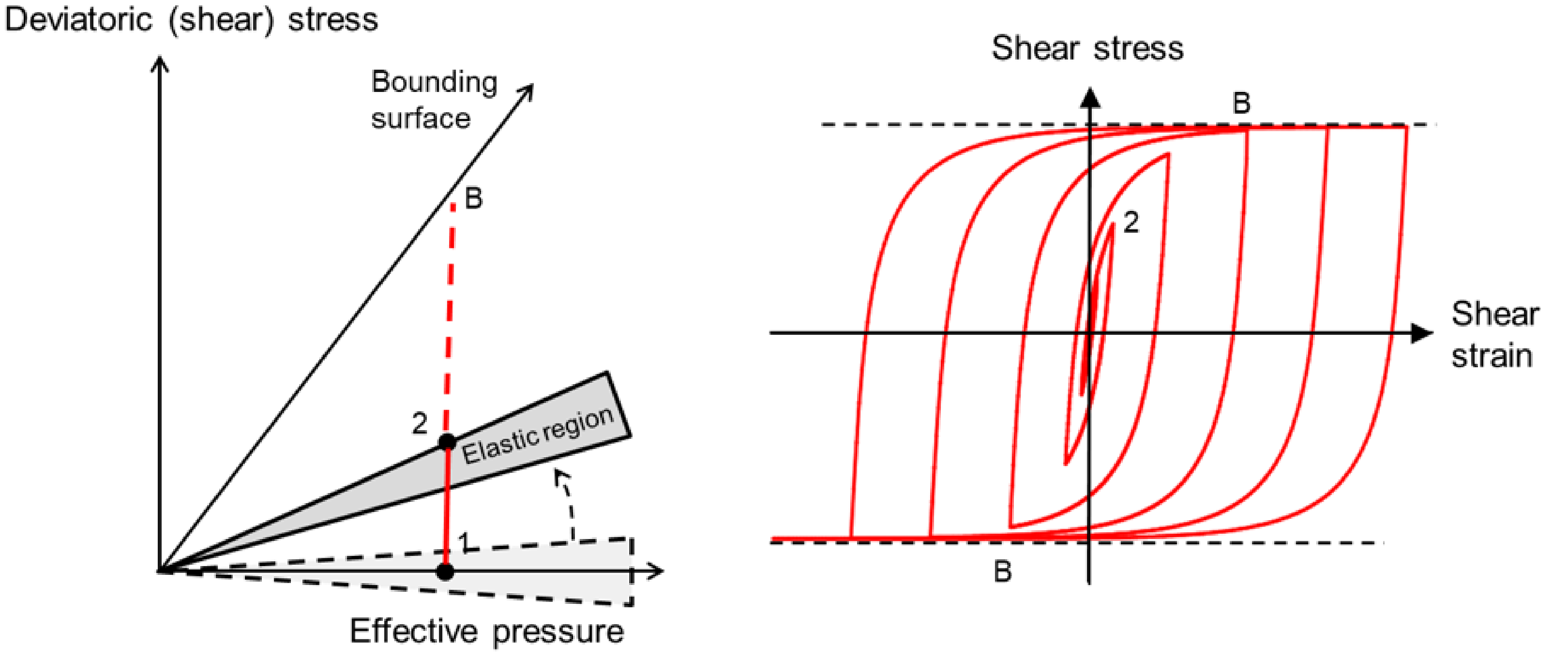


Fig.4: Schematic of SANISAND yield surface and bounding surface (left) and cyclic shear stress-strain response at constant effective pressure (right)

LS-DYNA soil materials

***MAT_SOIL_SANISAND**

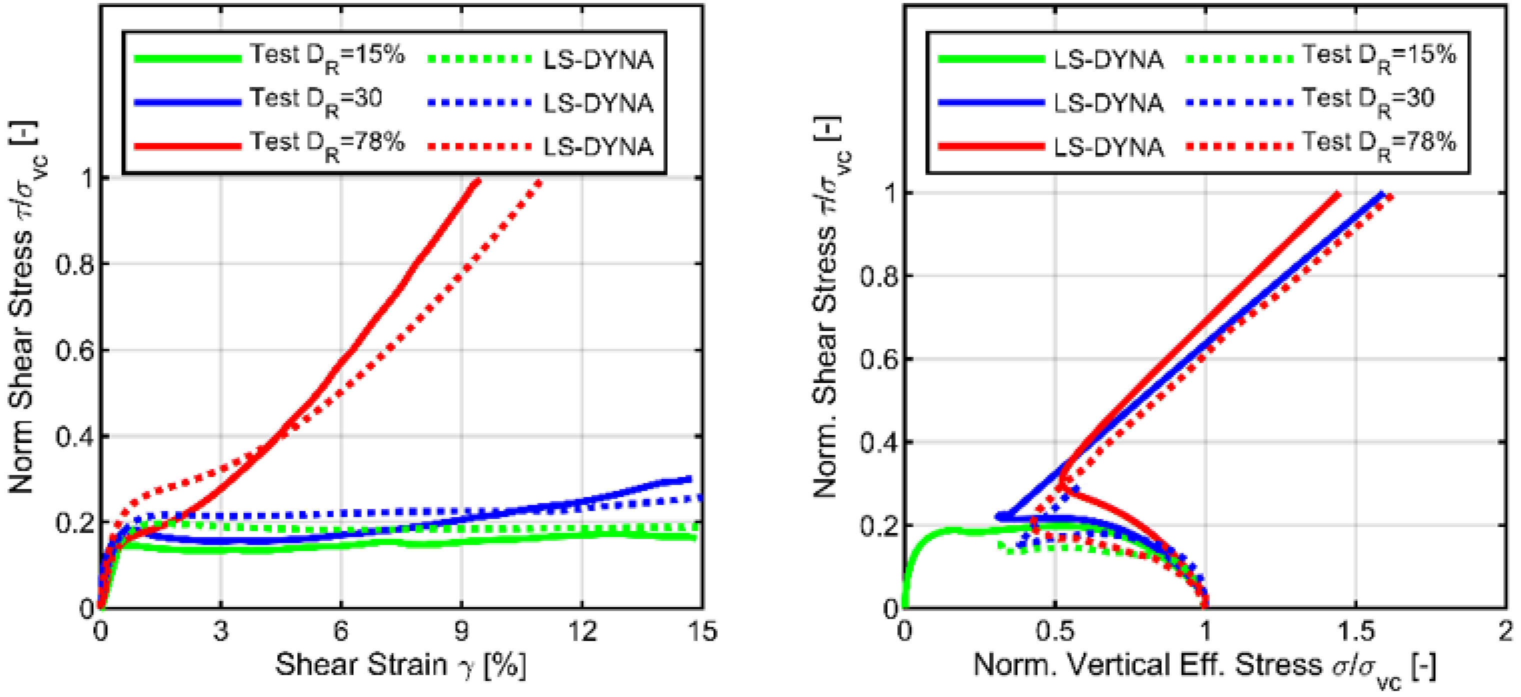


Fig.7: Comparison of the physical and numerical element test results under monotonic loading. Left: Normalized shear stress versus shear strain plot; right: normalized shear stress versus normalized effective stress plot.

Reference: Sturt et al 2021

LS-DYNA soil materials

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***MAT_SOIL_SANISAND**

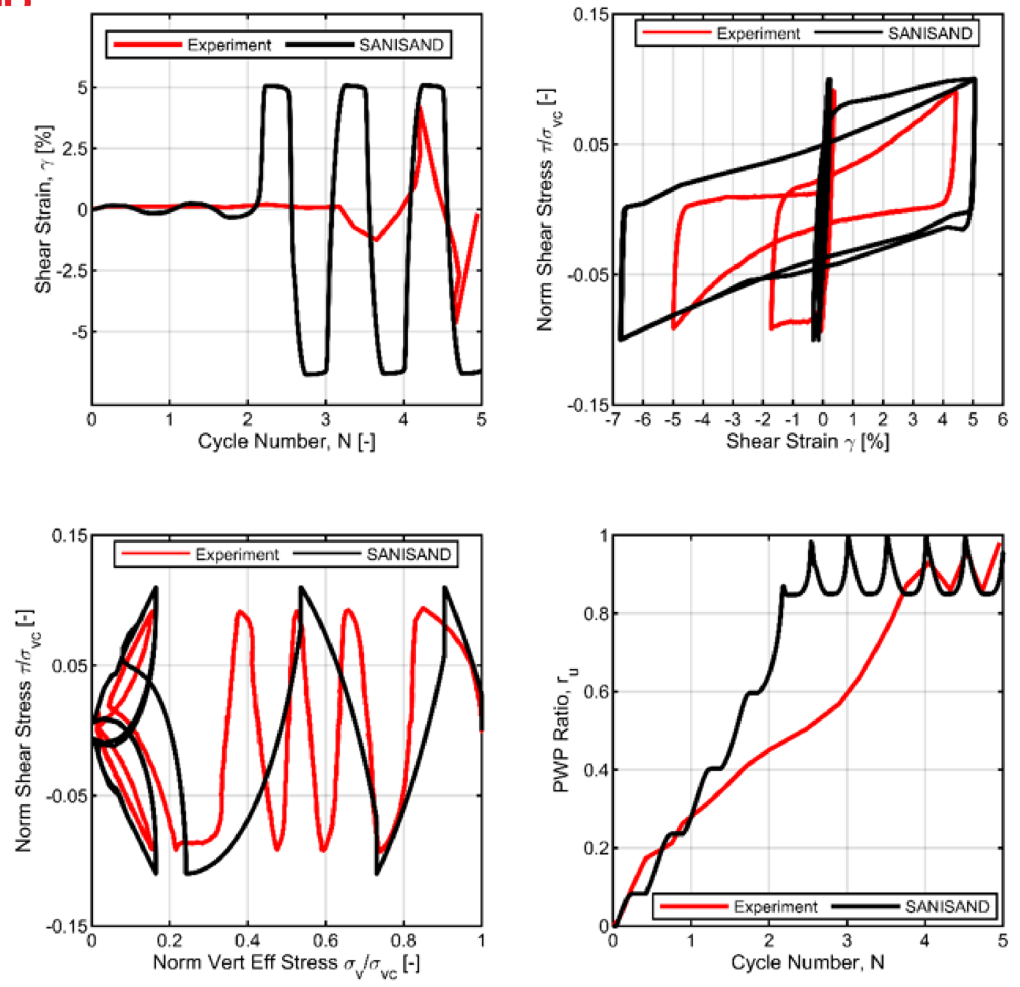
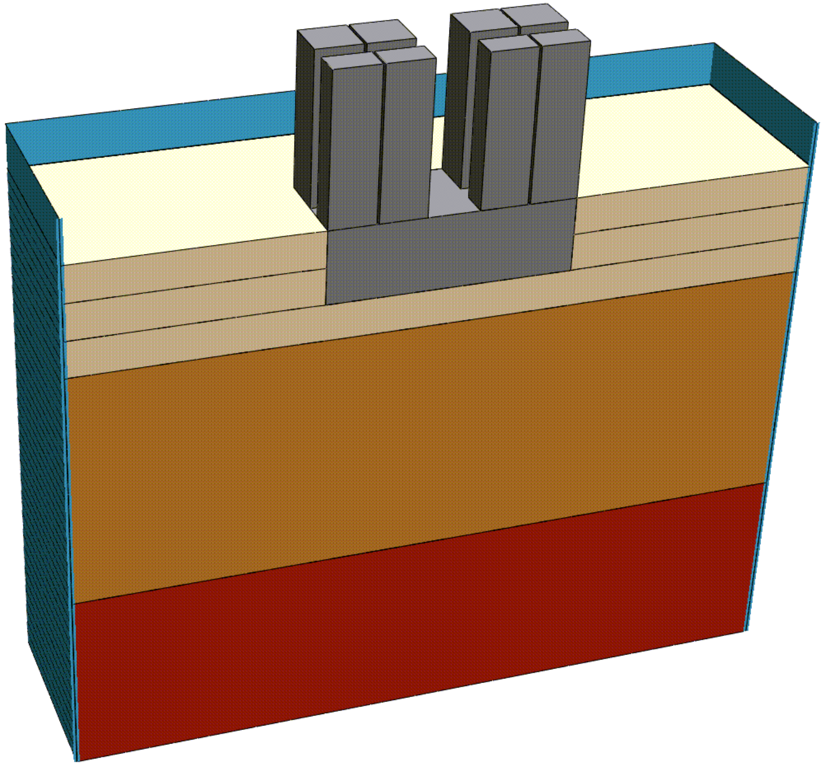


Fig.8: Comparison of physical and numerical element tests' results for Ottawa F-65 sand at a confining stress of 100 kPa

Reference: Sturt et al 2021

LS-DYNA soil materials

***MAT_SOIL_SANISAND**



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LS-DYNA soil materials

*MAT_SOIL_SANISAND References

- Dafalias, Y. F. (2004) Simple Plasticity Sand Model Accounting for Fabric Change Effects. ASCE Journal of Engineering Mechanics. 130(6).
- Sturt, R., Cengiz, C., Huang, H., Go, J., Bandara, S., Anton, P. (2021). Modelling Liquefaction of Soils with LS-DYNA using a SANISAND-based Material Model. 13th European LS-DYNA Conference, Ulm, Germany.

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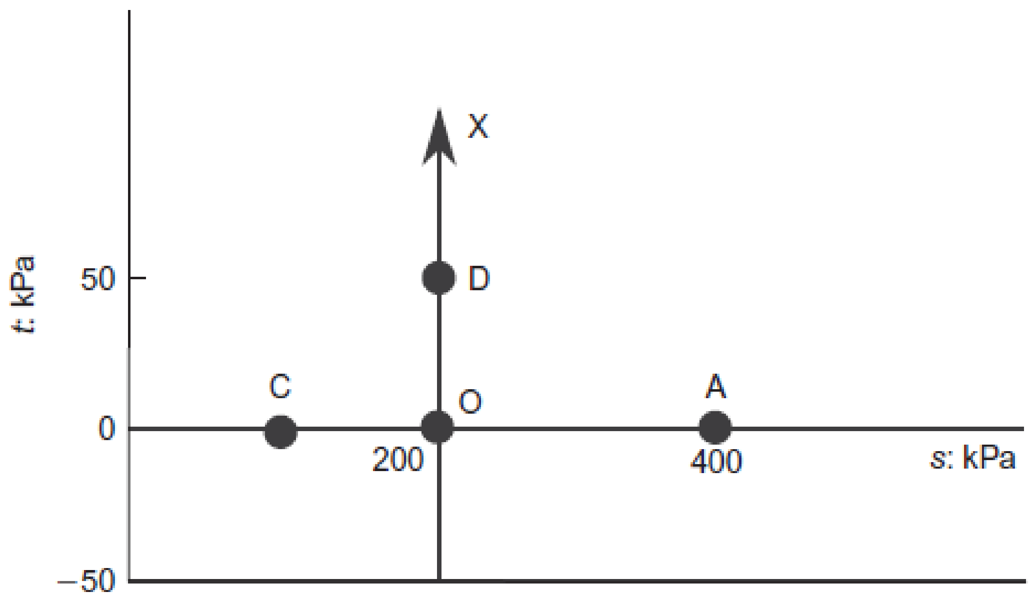
LS-DYNA soil materials

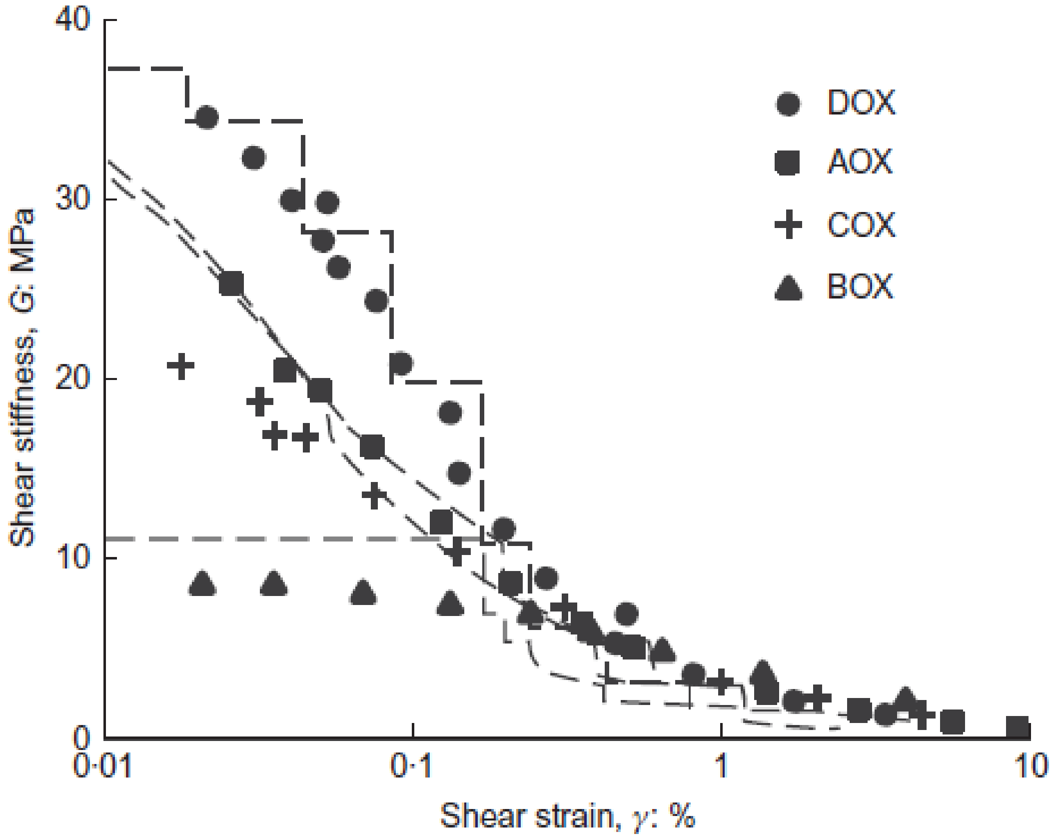
***MAT_BRICK**

- Like the MAT_HYSTERETIC_SOIL model, MAT_BRICK uses a backbone shear curve to define multiple elastic-plastic yield surfaces
- Differences:
  - Formulated in strain space
  - Shear behavior is influenced by stress history
  - Dilation/contraction behavior is influenced by stress history
  - Option to include anisotropic elasticity
  - Differentiates between normal consolidation and overconsolidation

LS-DYNA soil materials

***MAT_BRICK**

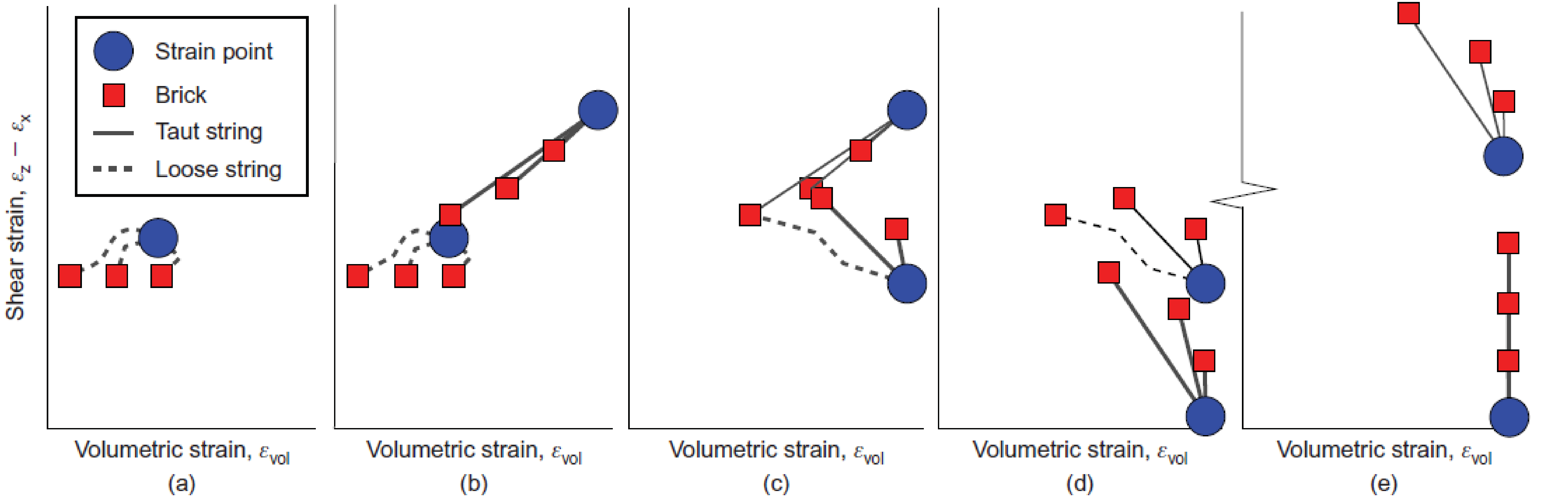




Reference: Simpson 1992

LS-DYNA soil materials

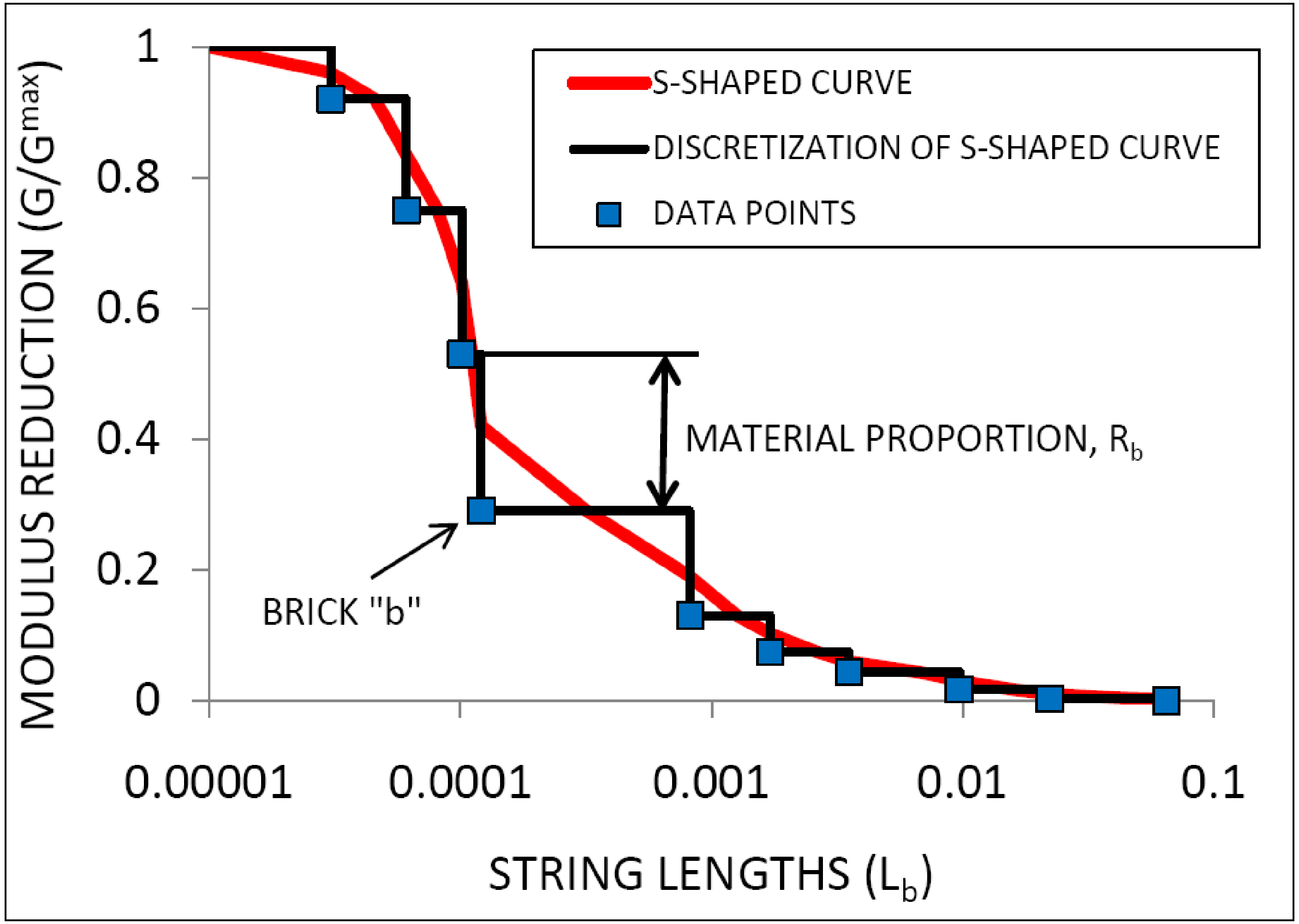
***MAT_BRICK**



Reference: Ellison et al 2012

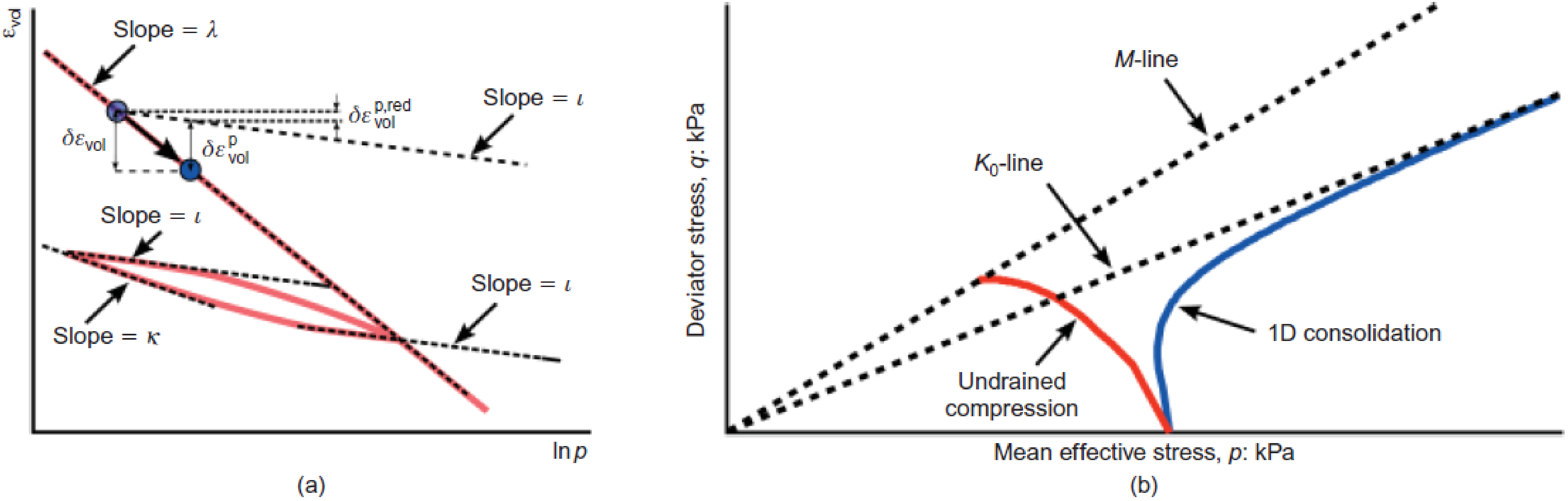
LS-DYNA soil materials

***MAT_BRICK**



LS-DYNA soil materials

***MAT_BRICK**



Reference: Ellison et al 2012

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LS-DYNA soil materials

*MAT_BRICK References

- Ellison, K. C., Soga, K., Simpson, B. (2012) A strain space soil model with evolving stiffness anisotropy. Geotechnique. 62(7).
- Simpson, B. (1992). Retaining structures: displacement and design. Geotechnique 42(4).

LS-DYNA soil materials

I-Soil

- Extension of MAT079 developed at UIUC
- Adds two parameters for calibration of shear-induced volume change, among other changes
- See link below for more information

https://www.youtube.com/watch?v=sQZHOxe_p-Q

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