Advanced Deep Excavation Workshop
23-Jan-2024

DEEP EXCAVATION

RELIABLE GEOEXPERTISE

Introduction

- Why this workshop
- Presenters

Dimitrios Konstantakos

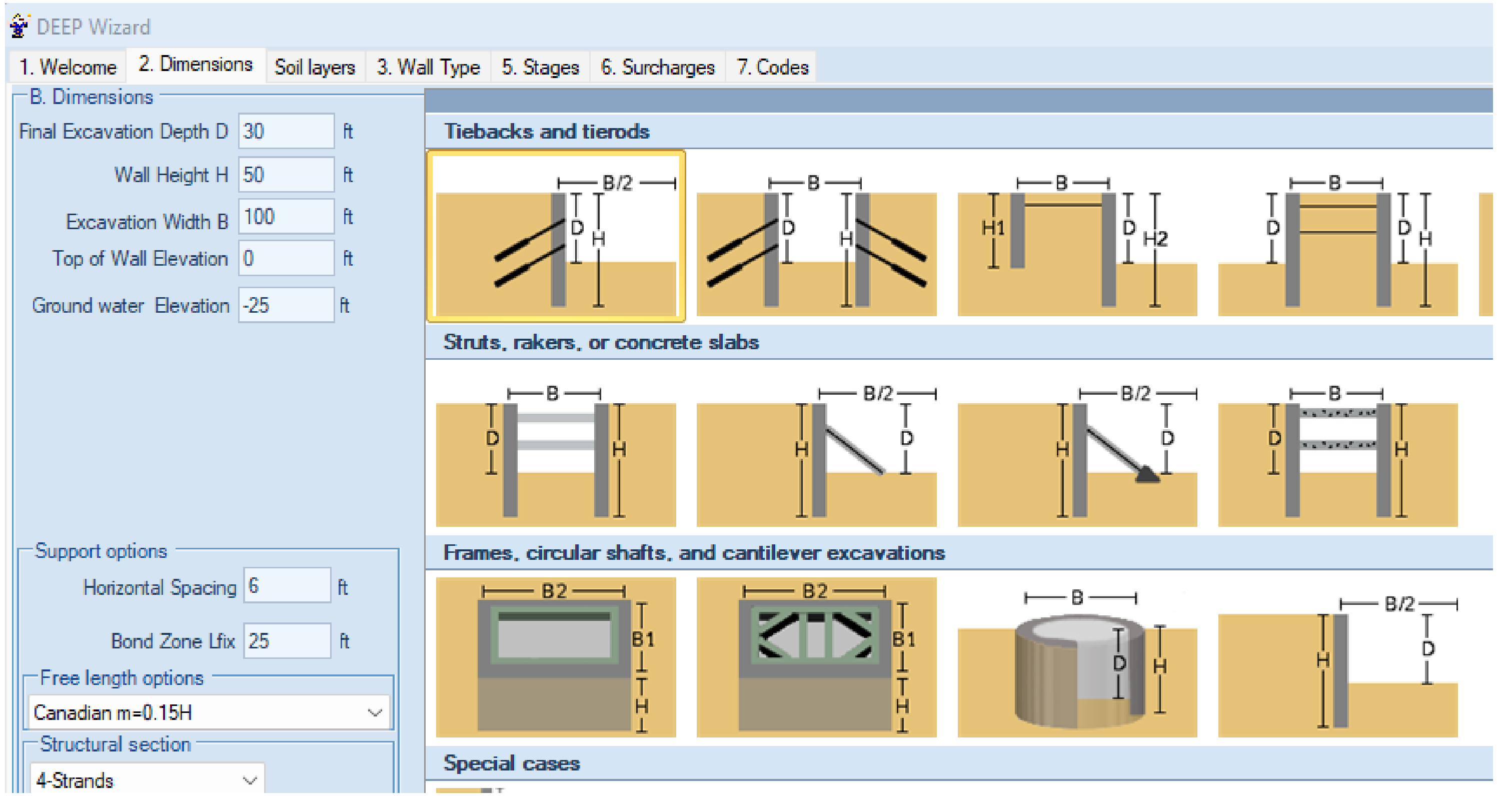
Dr. Nikolaos Lesgidis

Topics

- Optimization
- MSE Walls, pile abutments
- Finite element analysis – theoretical background and applications
- Design of sea walls
- Marine projects design methods
- Advanced 3D frame analysis
- Steel connections
- Practice – software applications

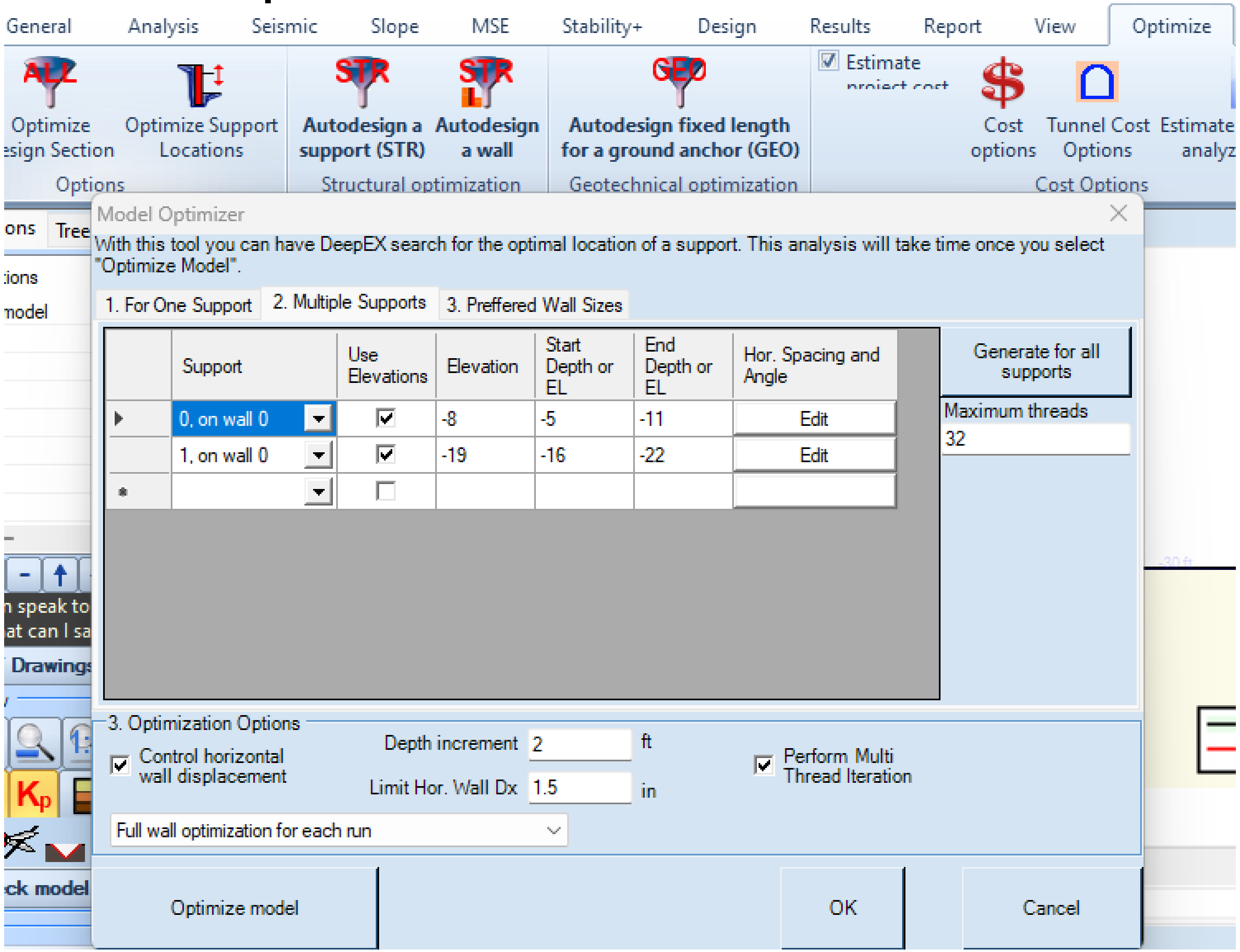
1.1 Support Location Optimization

- Obtain a range of optimized support locations
- 30ft deep excavation
- 2 levels of tiebacks

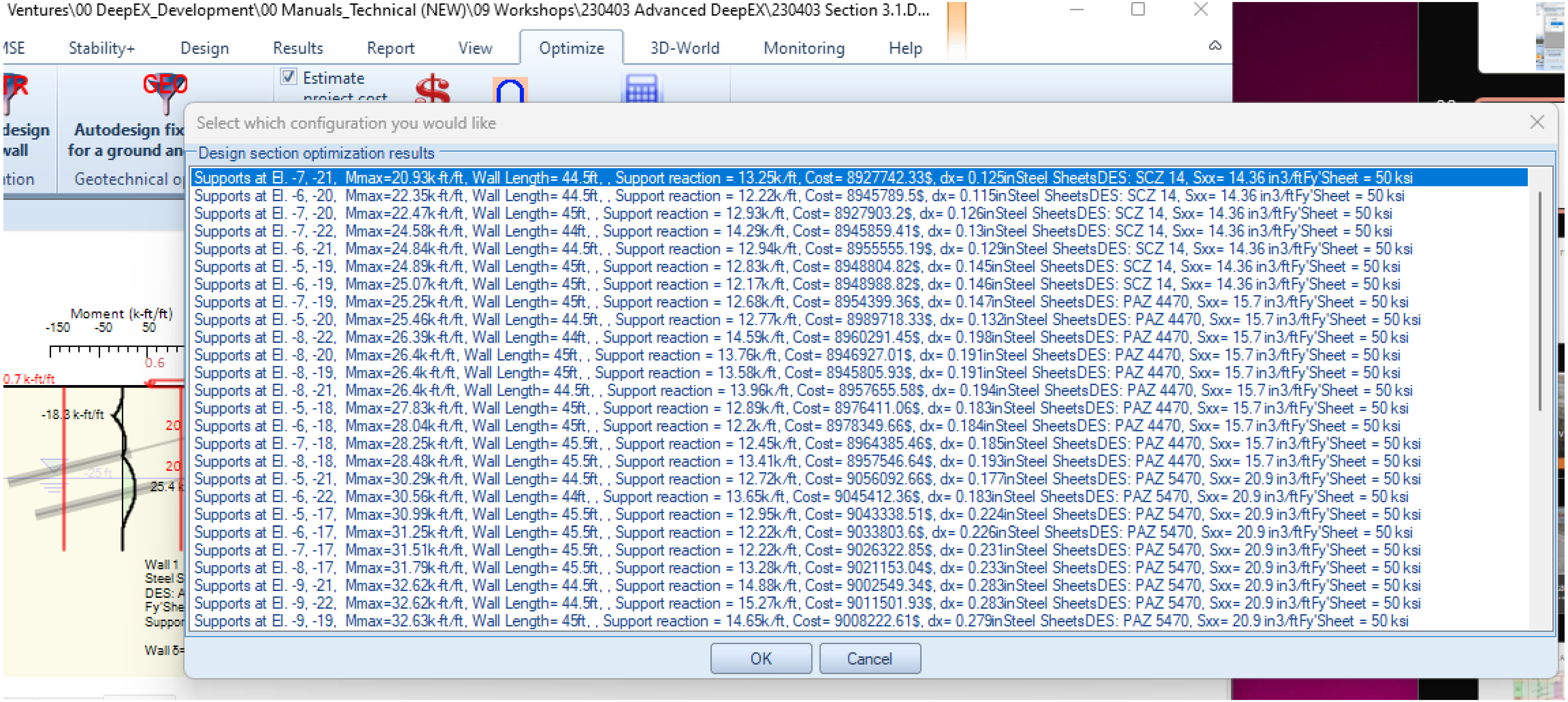


1.2 Optimization Options

- Optimize tab:



1.3 Results


Ventures\00 DeepEX_Development\00 Manuals_Technical (NEW)\09 Workshops\230403 Advanced DeepEX\230403 Section 3.1.D...
Stability+
Design
Results
Report
View
Optimize
3D-World
Monitoring
Help
Estimate project cost
Autodesign fix for a ground an
Geotechnical o
Moment (k-ft/ft)
-150
-50
50
0.6
0.7 k-ft/ft
-18.3 k-ft/ft
25.4
Wall 1
Select which configuration you would like
Design section optimization results
Supports at El. -7, -21, Mmax=20.93k-ft/ft, Wall Length= 44.5ft, , Support reaction = 13.25k/ft, Cost= 8927742.33$, dx= 0.125inSteel SheetsDES: SCZ 14, Sxx= 14.36 in3/ftFy'Sheet = 50 ksi
Supports at El. -6, -20, Mmax=22.35k-ft/ft, Wall Length= 44.5ft, , Support reaction = 12.22k/ft, Cost= 8945789.5$, dx= 0.115inSteel SheetsDES: SCZ 14, Sxx= 14.36 in3/ftFy'Sheet = 50 ksi
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Supports at El. -6, -19, Mmax=25.07k-ft/ft, Wall Length= 45ft, , Support reaction = 12.17k/ft, Cost= 8948988.82$, dx= 0.146inSteel SheetsDES: SCZ 14, Sxx= 14.36 in3/ftFy'Sheet = 50 ksi
Supports at El. -7, -19, Mmax=25.25k-ft/ft, Wall Length= 45ft, , Support reaction = 12.68k/ft, Cost= 8954399.36$, dx= 0.147inSteel SheetsDES: PAZ 4470, Sxx= 15.7 in3/ftFy'Sheet = 50 ksi
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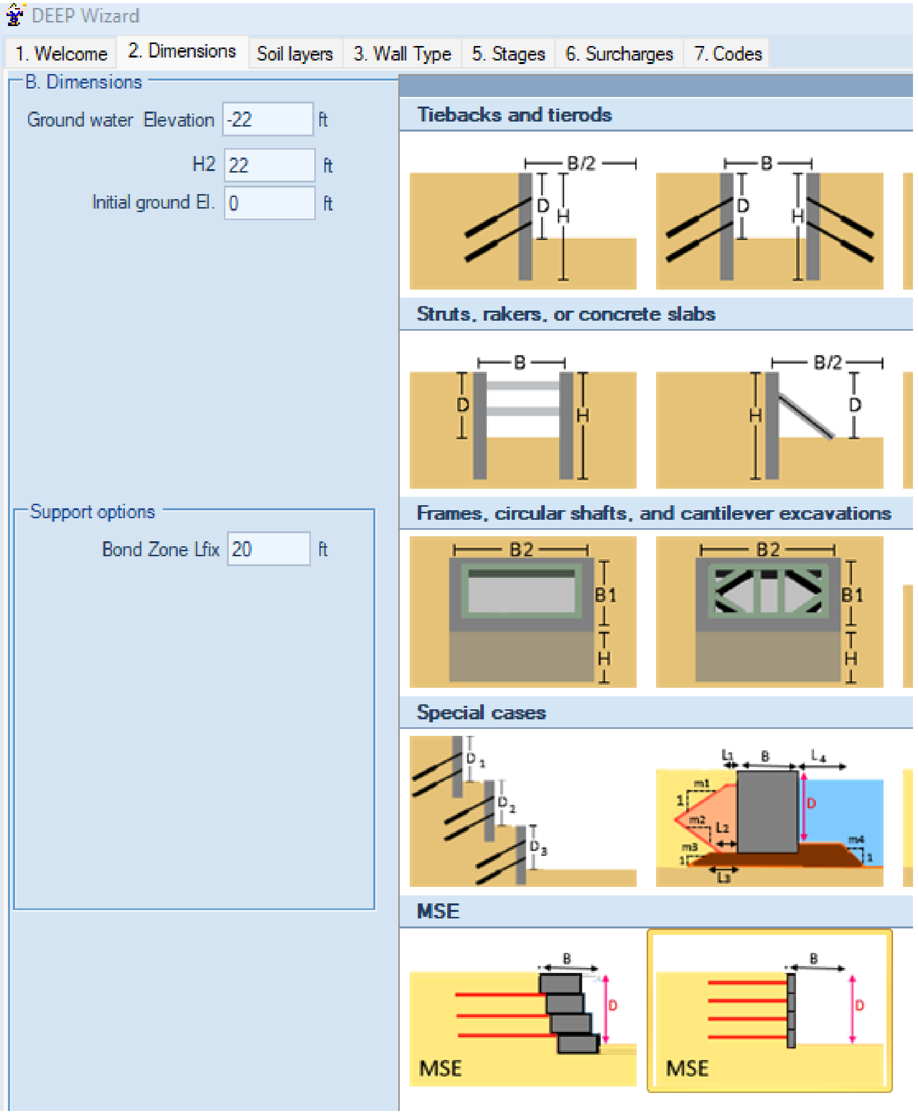
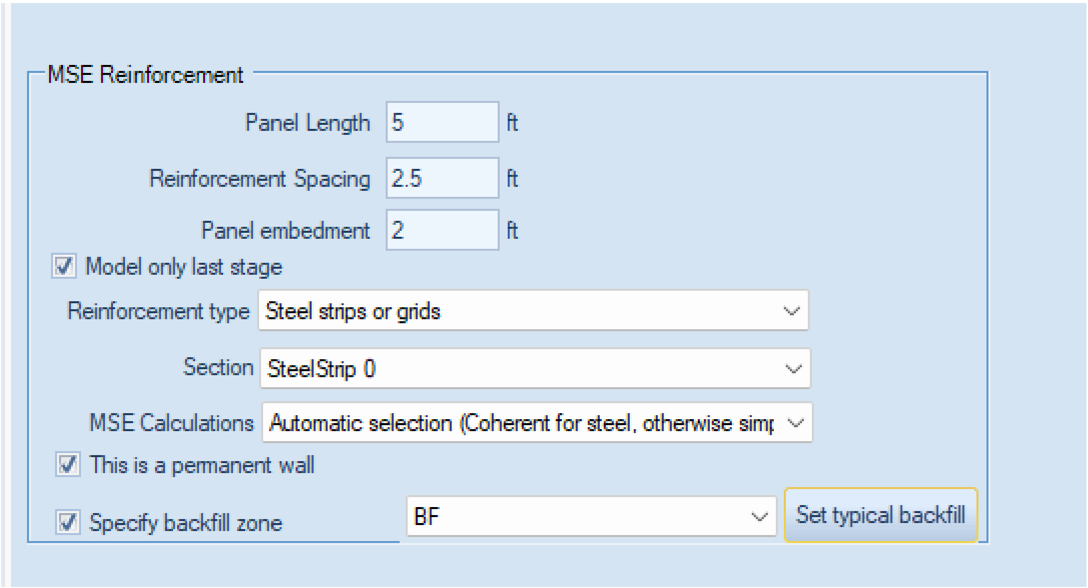
OK
Cancel

2. MSE Walls

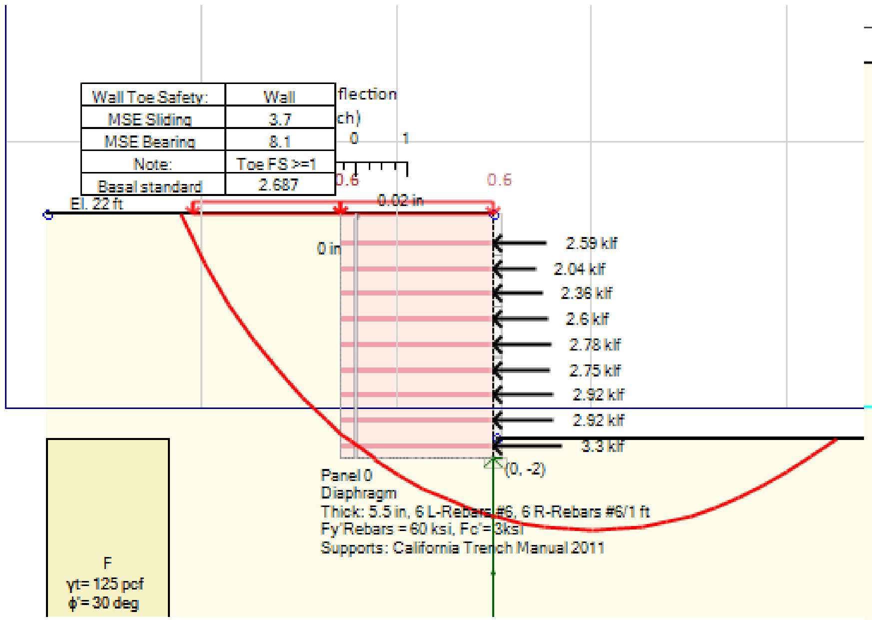
- 2023 Version MSE Wizard
- Automatic selection
- Coherent method
- AASHTO Stiffness
- Allen Stiffness

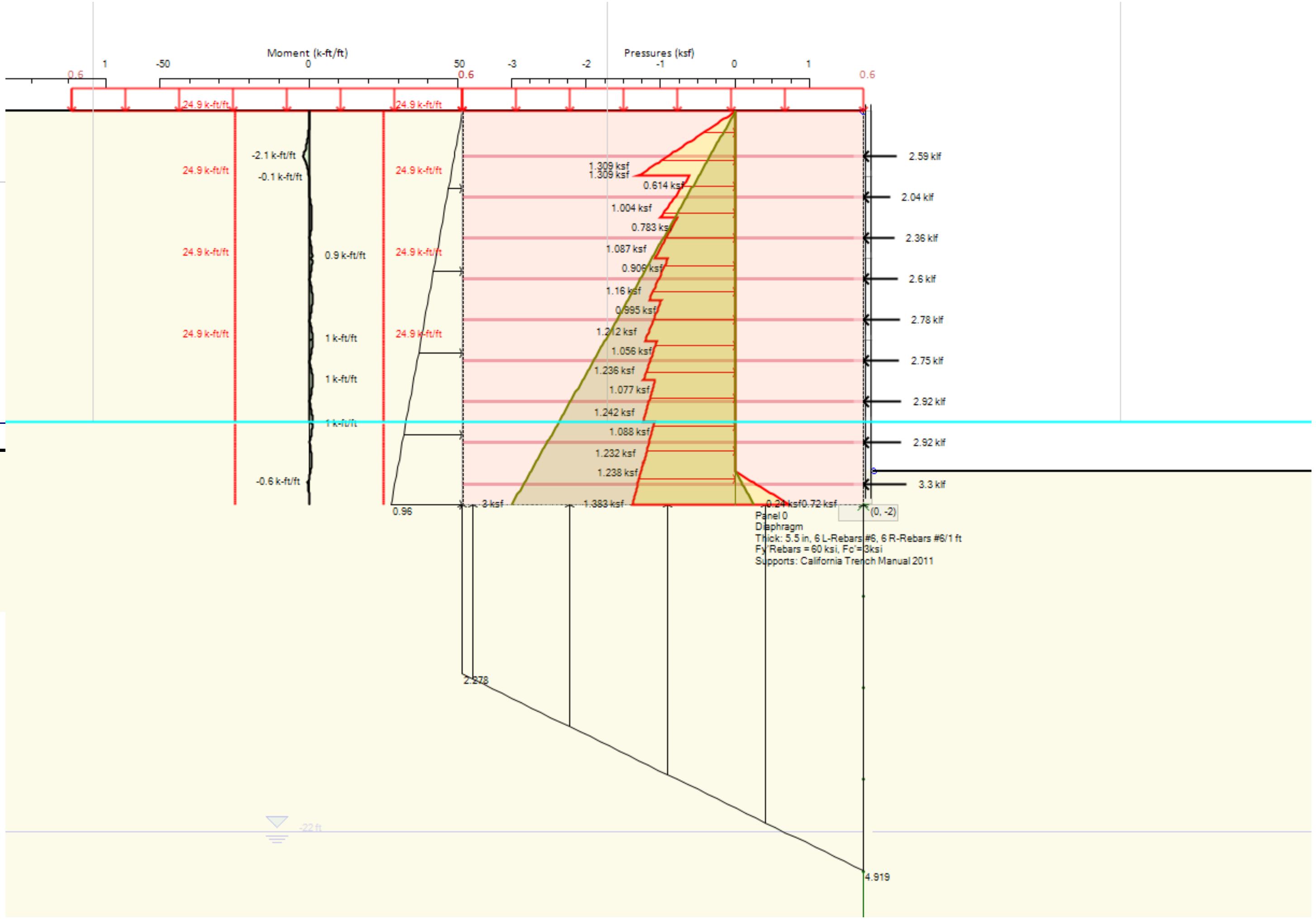
2.1 Use Wizard



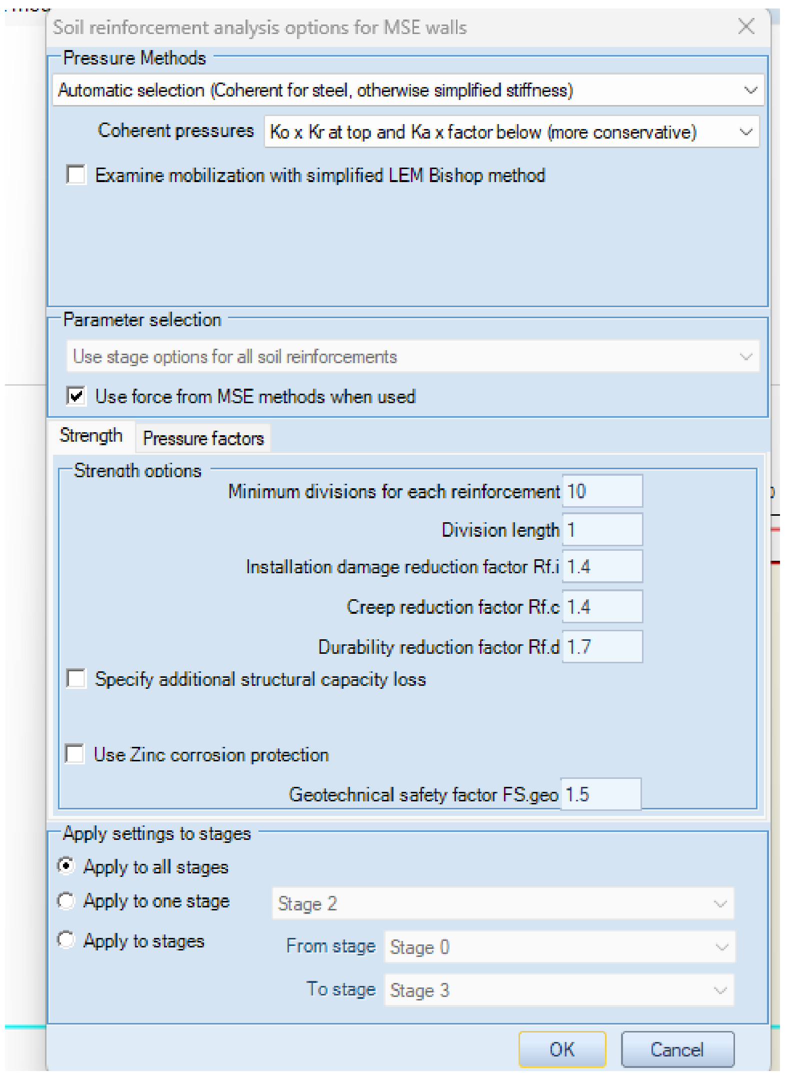


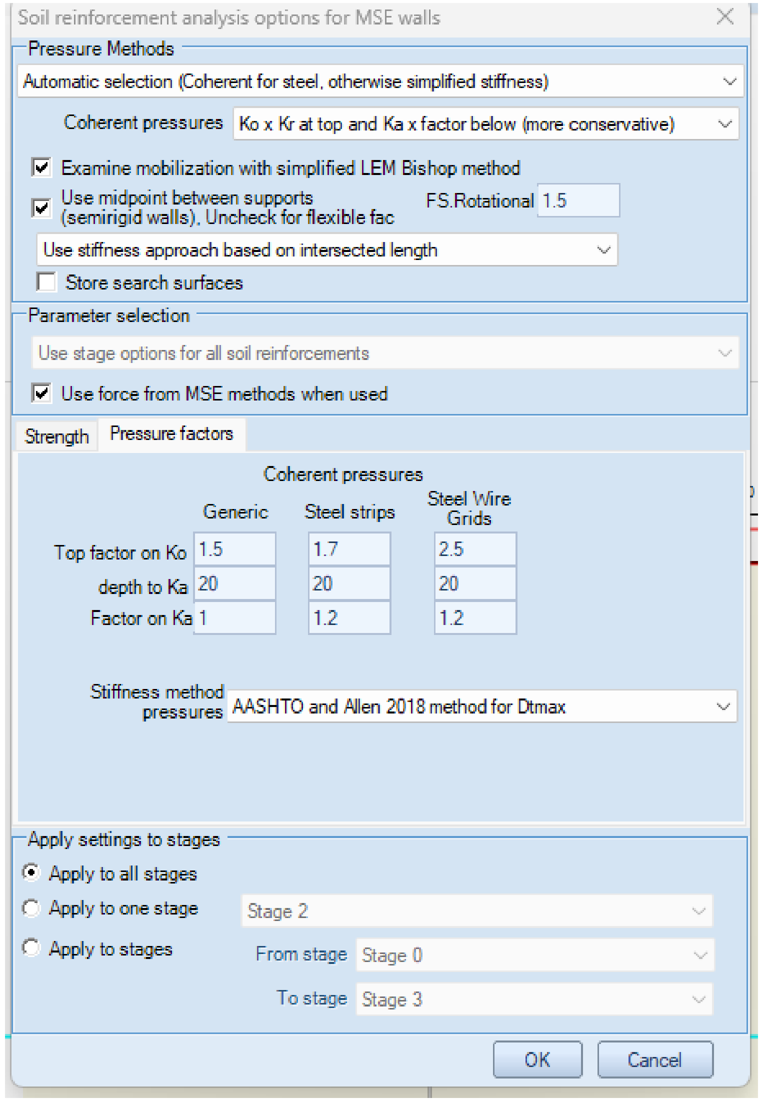
2.2 MSE Results



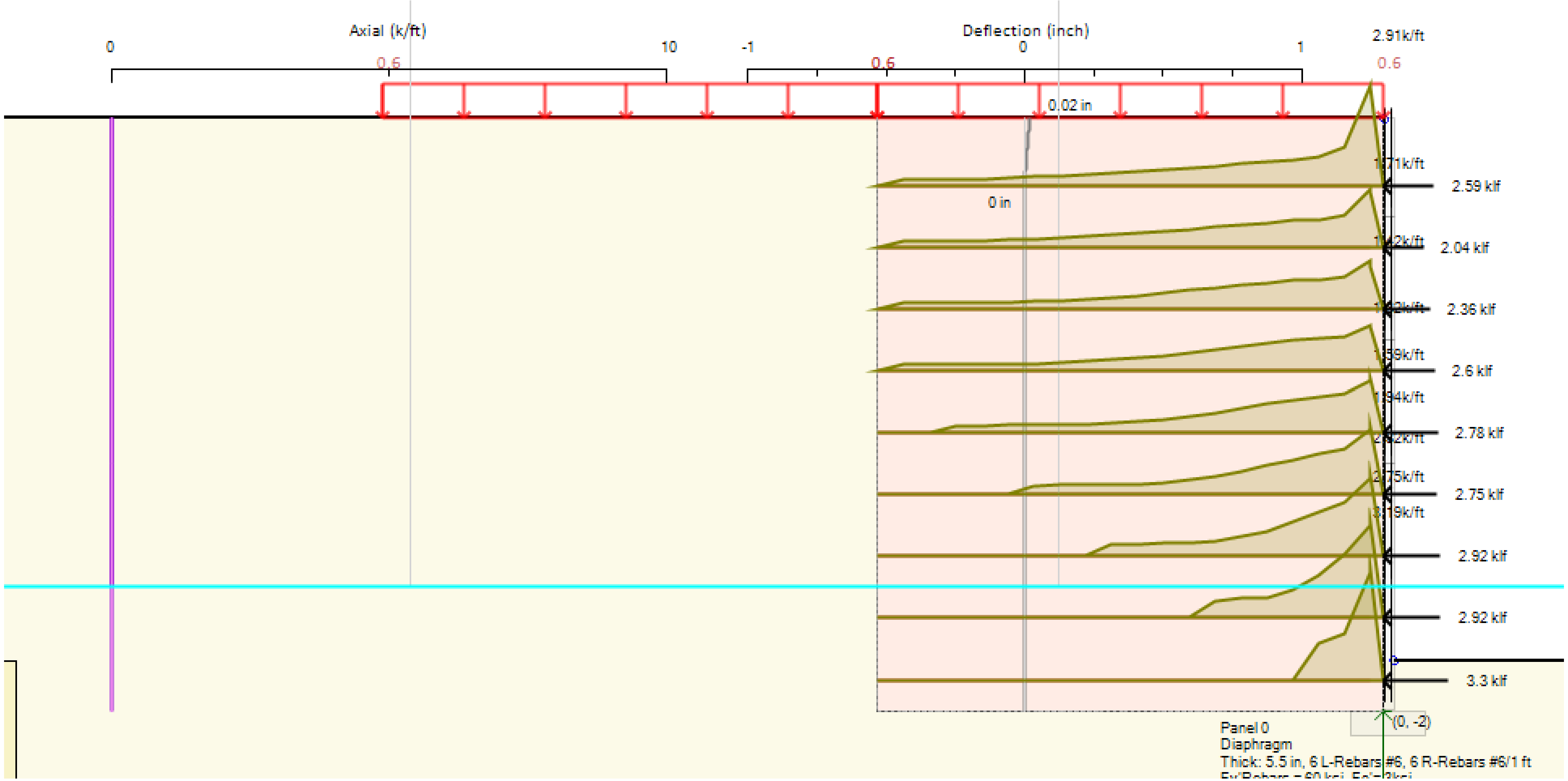


2.3 MSE Wall Options



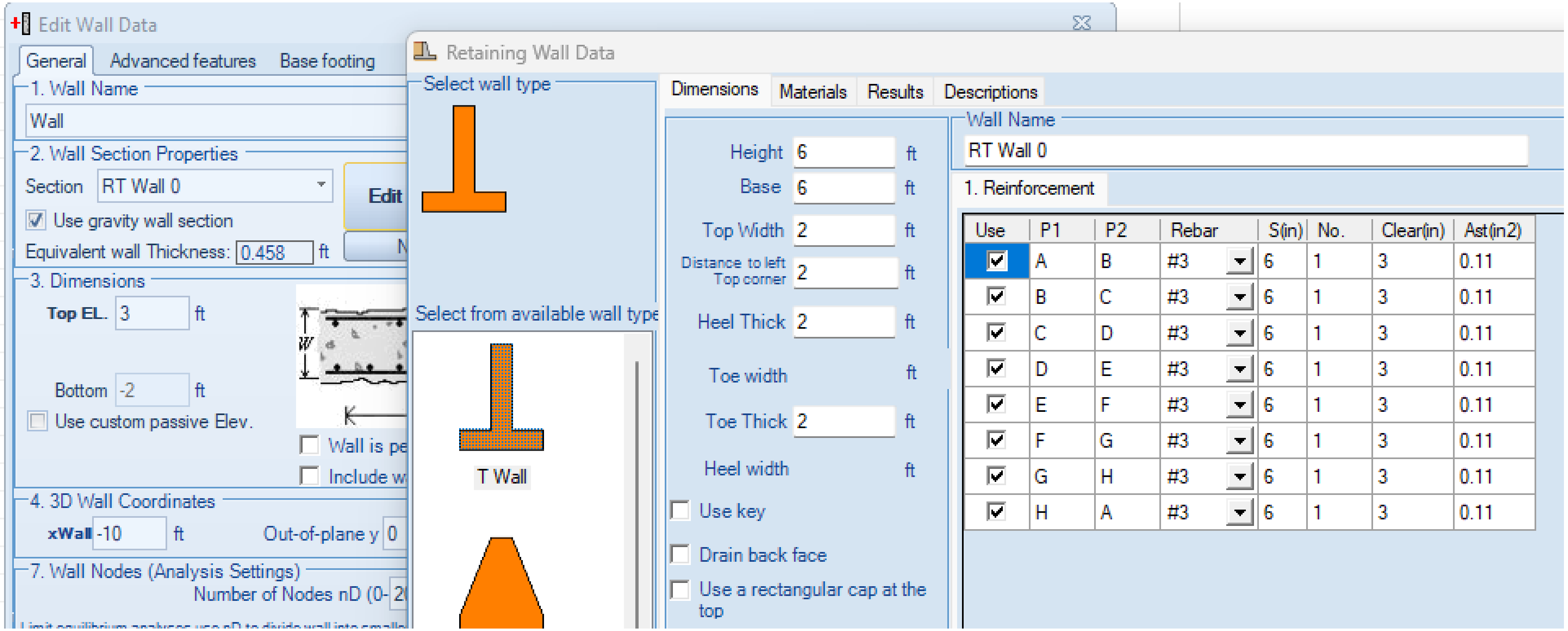


2.4 MSE Simplified Mobilization

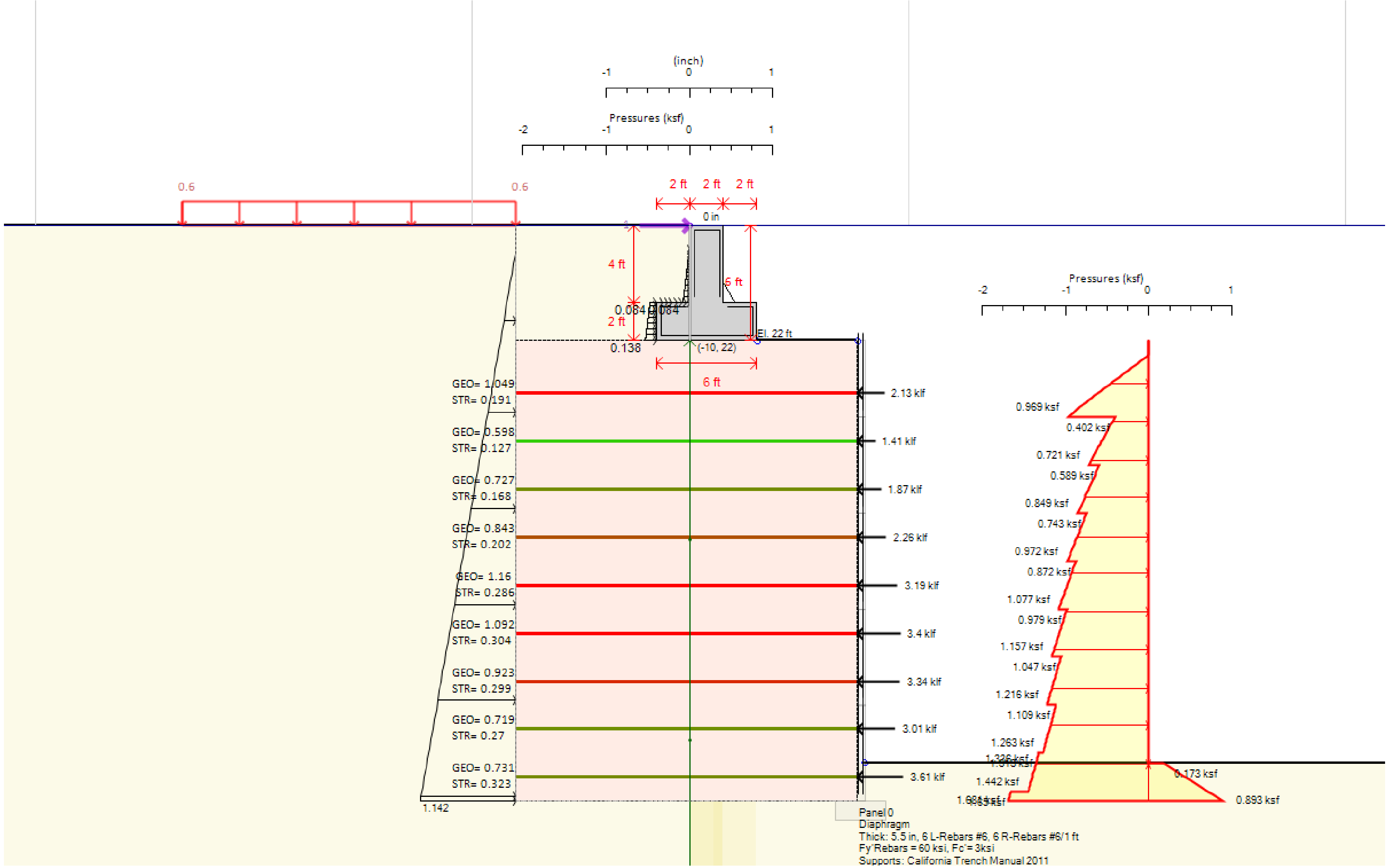


2.5 Adding an Impact Barrier

- Add wall on the left
- Draw impact load



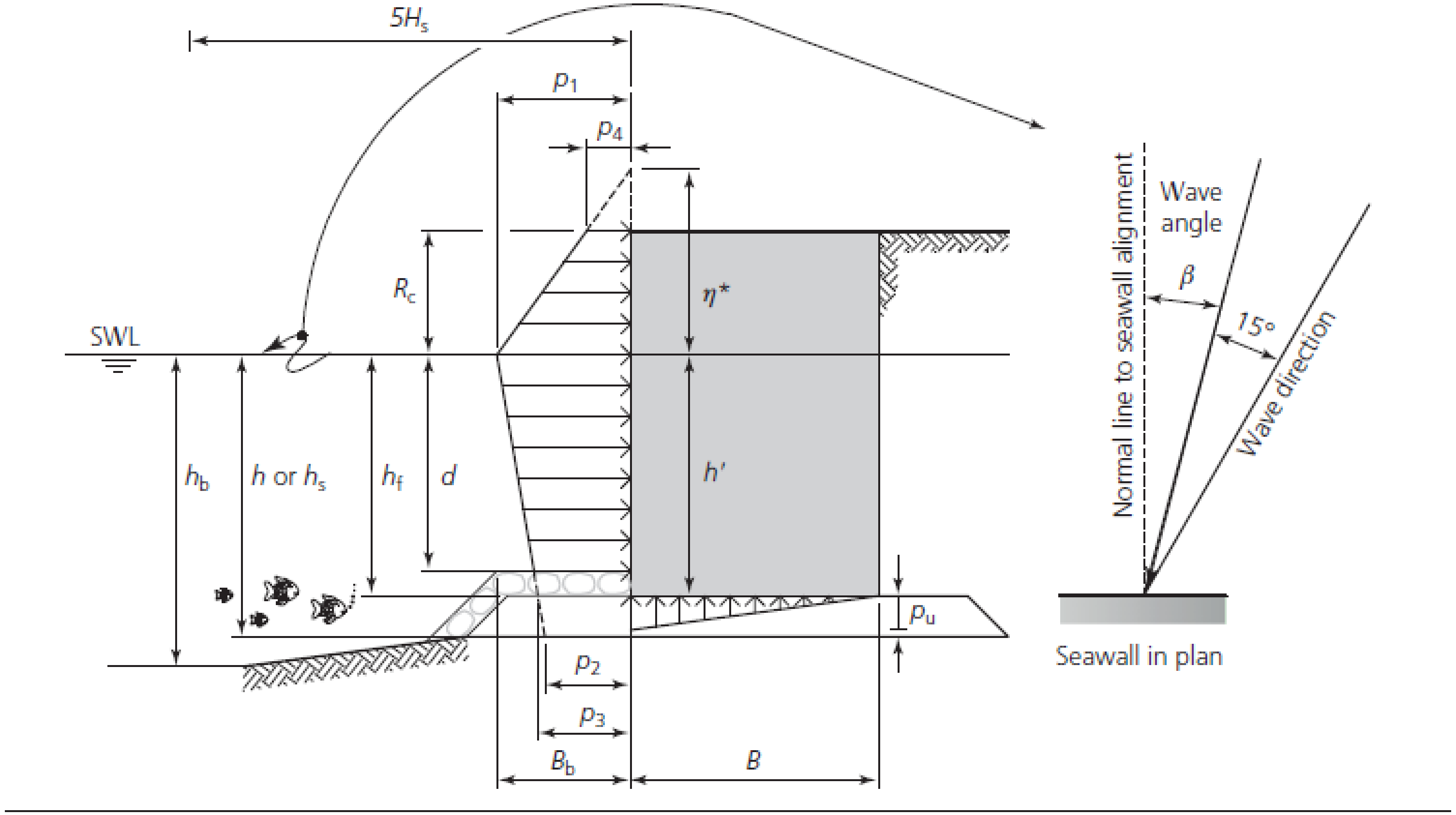
2.6 MSE Impact Load



3. Seawalls and Wave Pressures

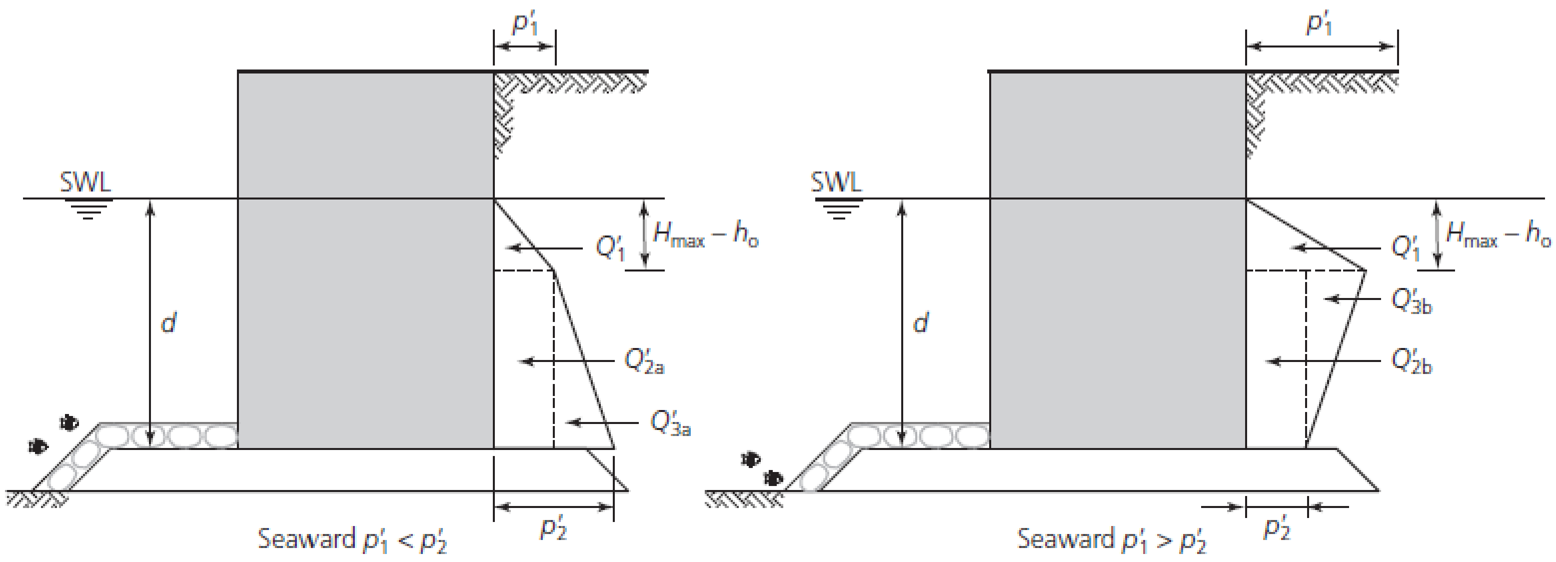
- Introduction to wave pressures
- Gravity walls
- Caisson walls

3.1.1 Wave Pressures - Shoreward



Goda's Wave Pressures

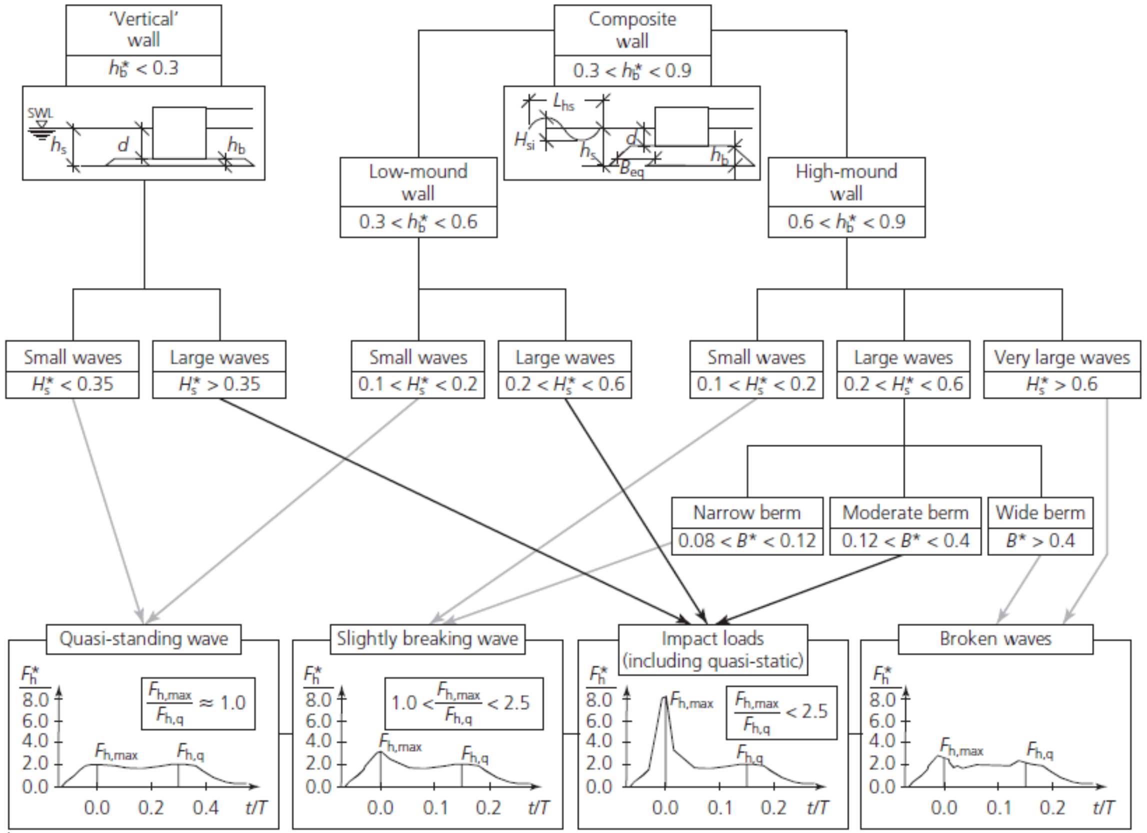
3.1.2 Wave Pressures - Seaward



Sainflou wave pressures

3.1.3 Impact vs. Pulsating Waves

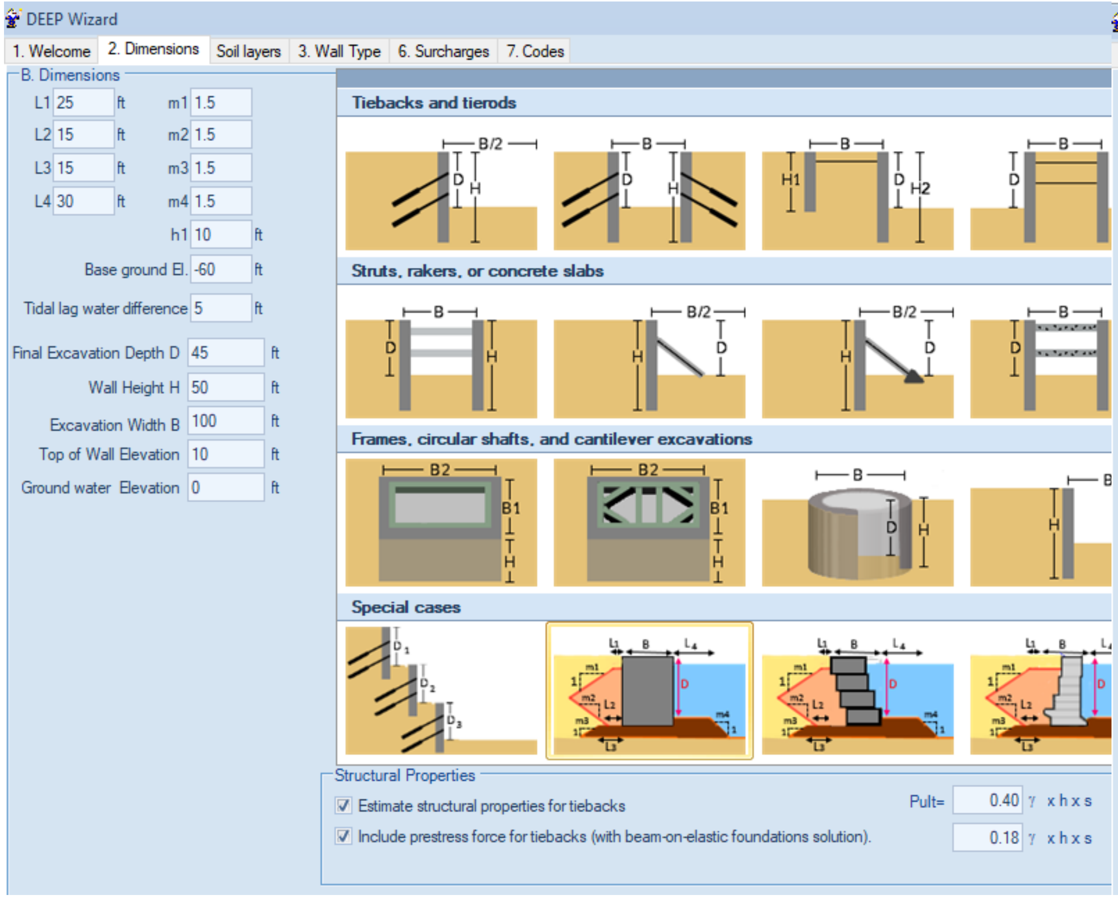
PROVERBS (European 1999)

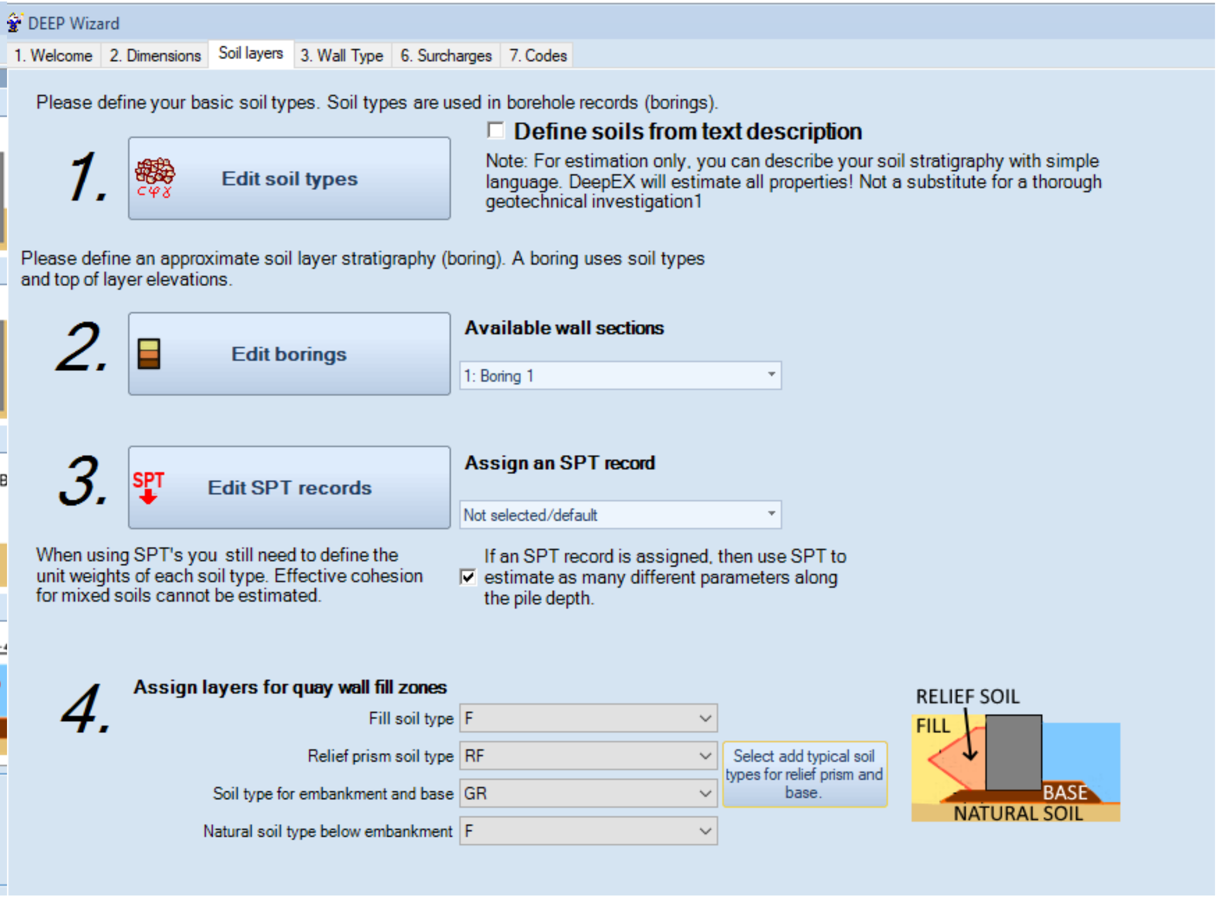


Pulsating/non-breaking/reflecting	Impulsive/plunging/breaking/impact	Broken
Empirical method to determine wave response/actions (refer to Chapter 5)		
Shoreward wave action: Goda (2000) Seaward wave action: Sainflou modified by McConnell *et al.* (1999) Uplift: Goda (2000)	Shoreward action: Allsop and Vicinanza modified by McConnell (2000) Cuomo *et al.* (2010) Seaward action: Sainflou modified by McConnell *et al.* (1999), model studies Uplift: Goda (2000)	Shoreward action: Blackmore and Hewson (1984) or Camfield (1991) as appropriate. Seaward action: Sainflou modified by McConnell *et al.* (1999) Uplift: Goda (2000)

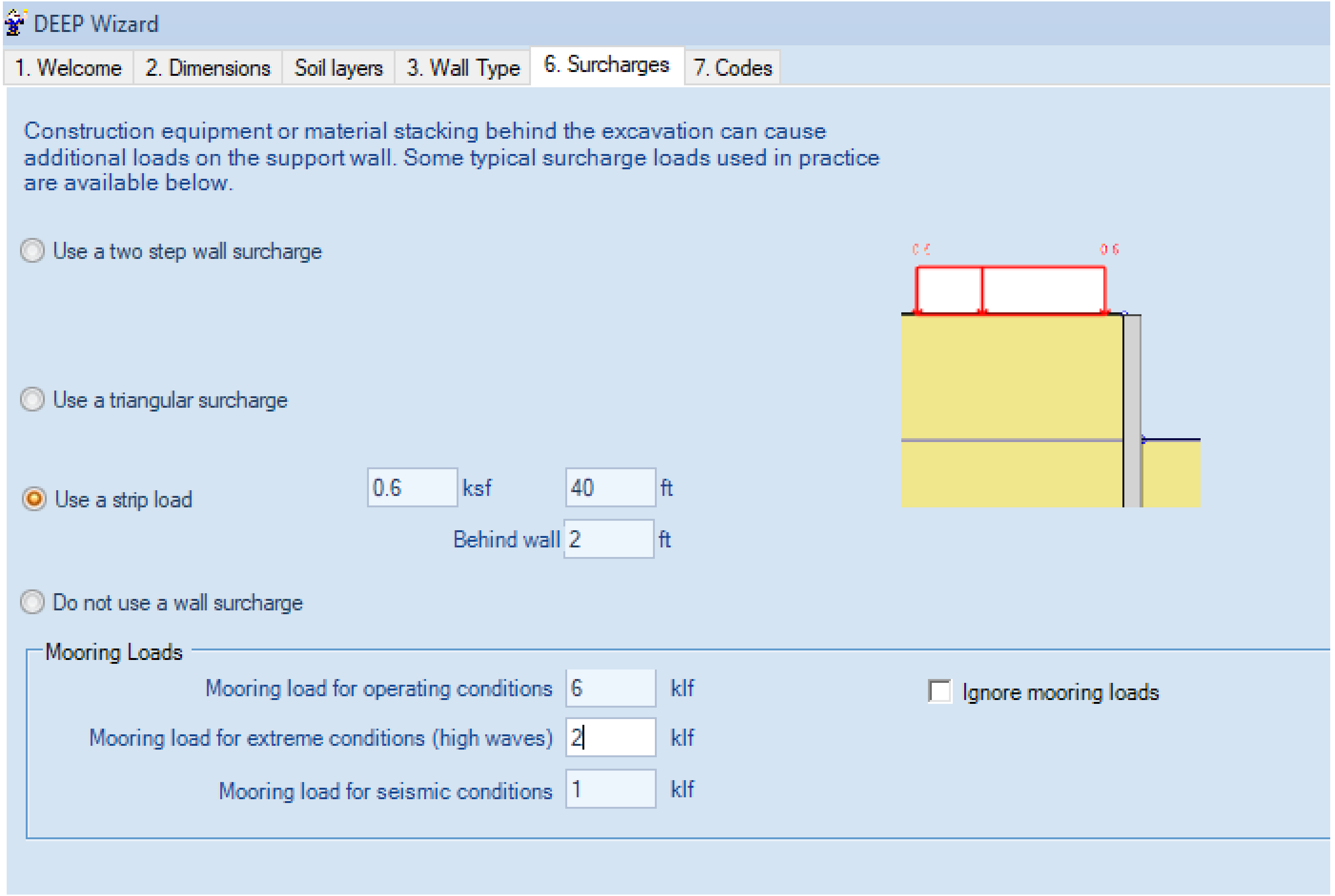
With $h_b^* = \frac{h_b}{h_s}$, $H_s^* = \frac{H_{si}}{h_s}$, $B^* = \frac{B_{eq}}{L_{hs}}$, $F_h^* = \frac{F_h}{p.g.H_b^2}$

3.2.1 Gravity Wall Example

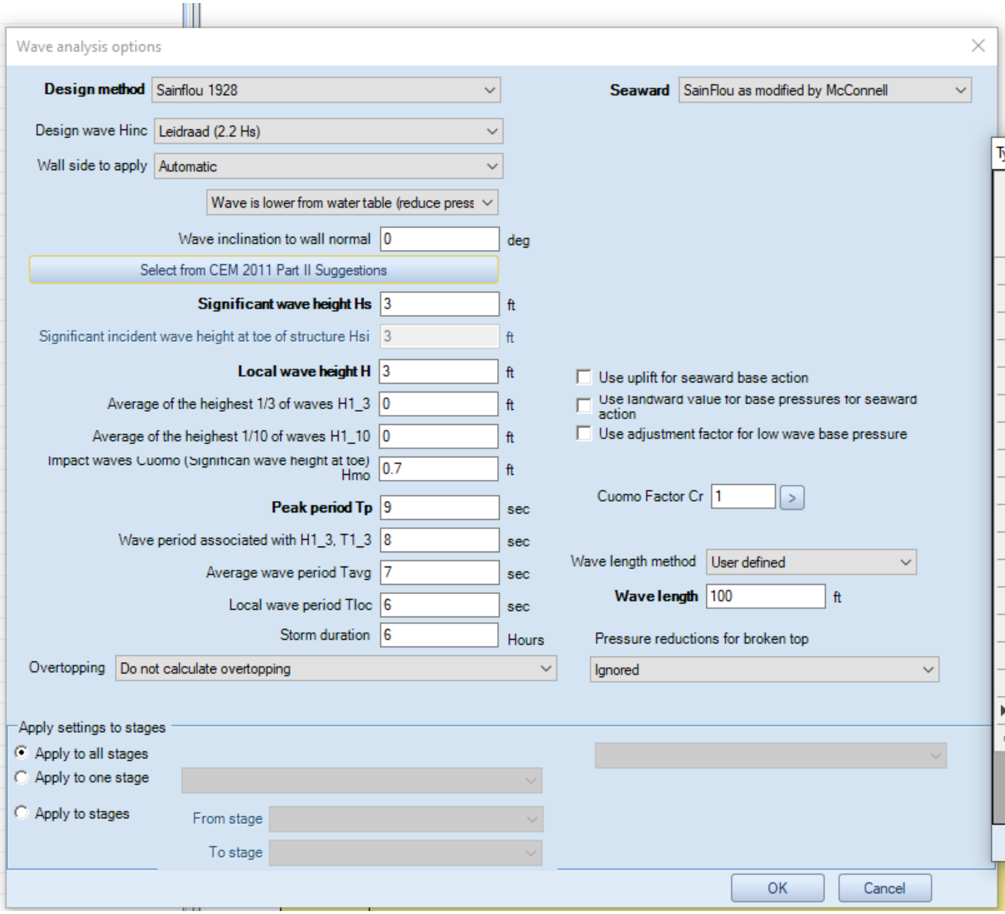


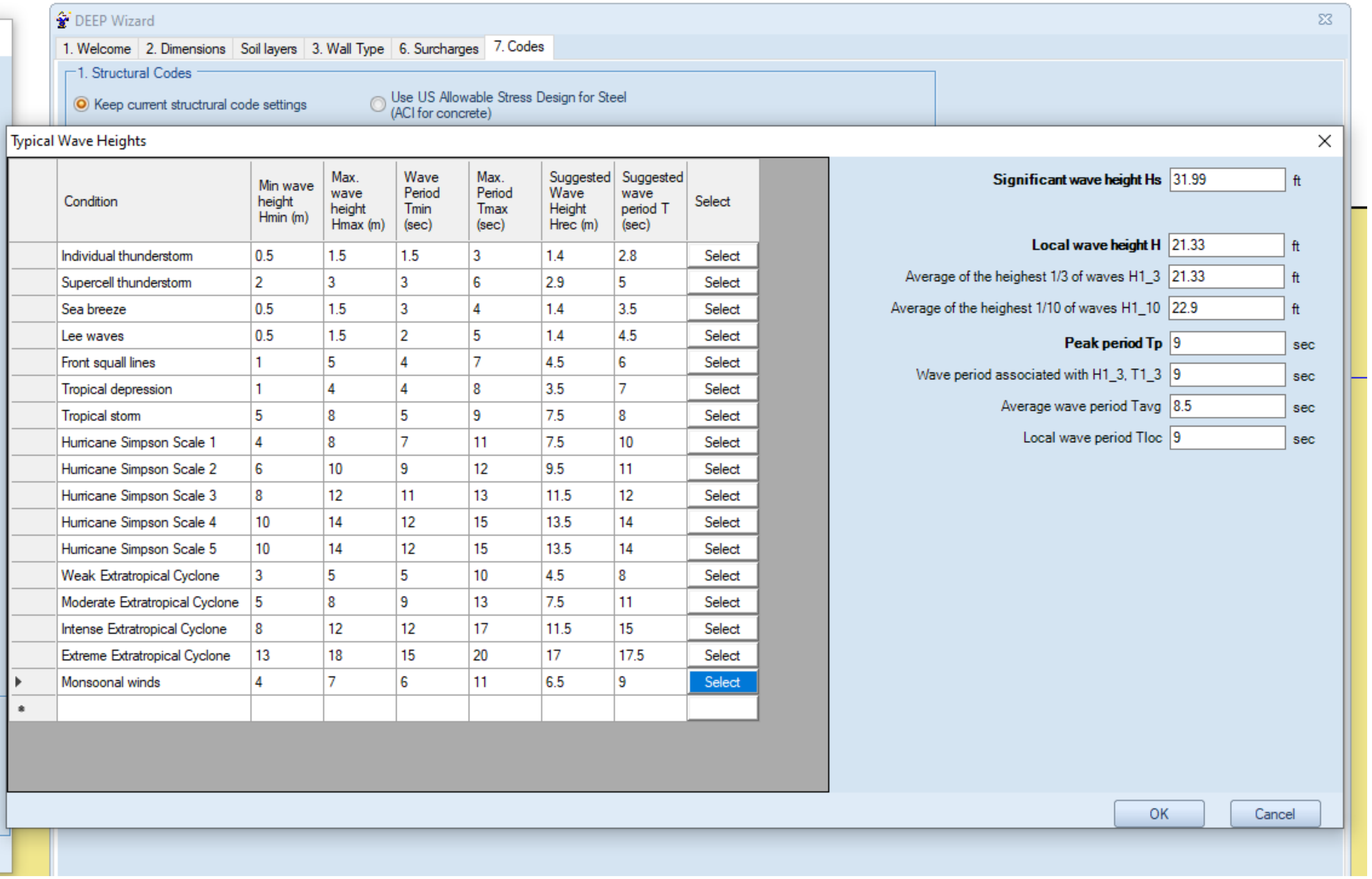


3.2.2 Gravity Wall Example - Loads



3.2.3 Wave Pressures – Sainflou, CEM

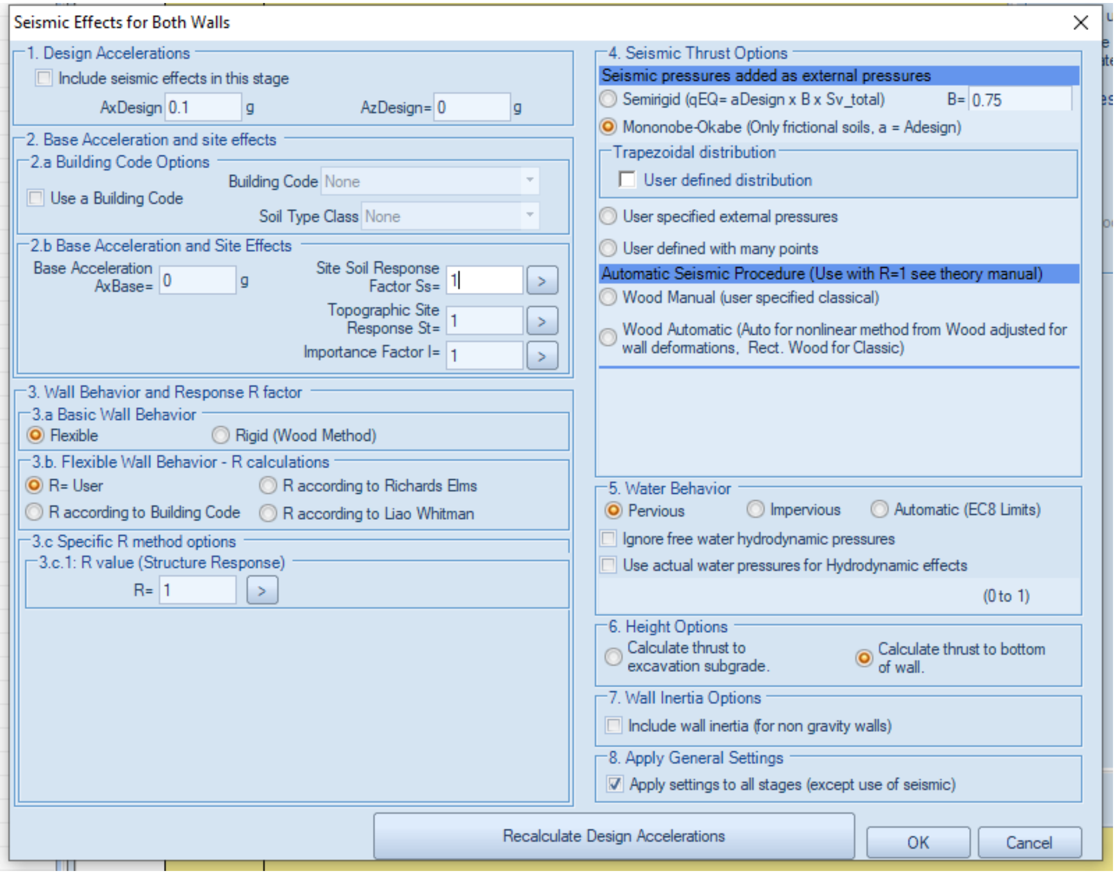




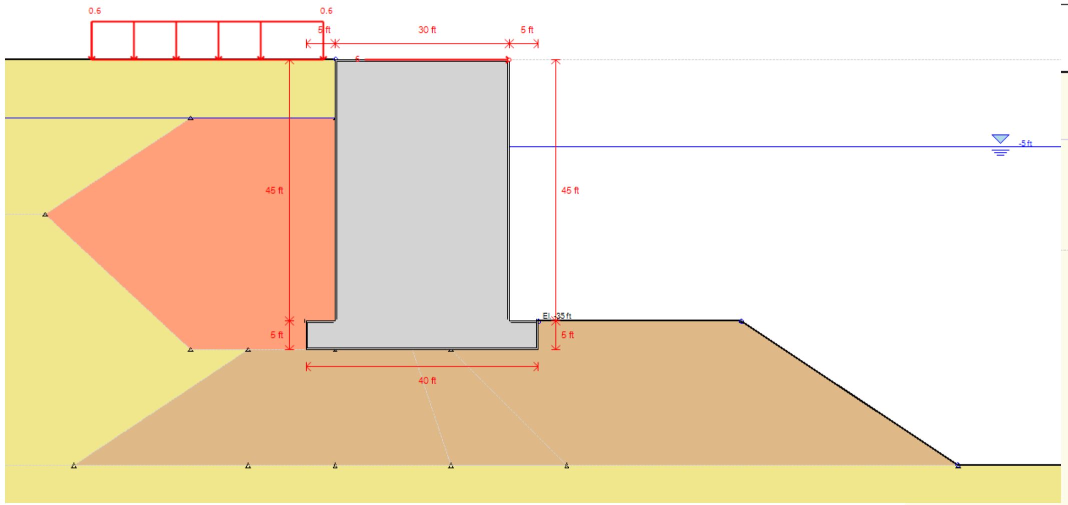
Condition	Min wave height Hmin (m)	Max. wave height Hmax (m)	Wave Period Tmin (sec)	Max. Period Tmax (sec)	Suggested Wave Height Hrec (m)	Suggested wave period T (sec)	Select
Individual thunderstorm	0.5	1.5	1.5	3	1.4	2.8	Select
Supercell thunderstorm	2	3	3	6	2.9	5	Select
Sea breeze	0.5	1.5	3	4	1.4	3.5	Select
Lee waves	0.5	1.5	2	5	1.4	4.5	Select
Front squall lines	1	5	4	7	4.5	6	Select
Tropical depression	1	4	4	8	3.5	7	Select
Tropical storm	5	8	5	9	7.5	8	Select
Hurricane Simpson Scale 1	4	8	7	11	7.5	10	Select
Hurricane Simpson Scale 2	6	10	9	12	9.5	11	Select
Hurricane Simpson Scale 3	8	12	11	13	11.5	12	Select
Hurricane Simpson Scale 4	10	14	12	15	13.5	14	Select
Hurricane Simpson Scale 5	10	14	12	15	13.5	14	Select
Weak Extratropical Cyclone	3	5	5	10	4.5	8	Select
Moderate Extratropical Cyclone	5	8	9	13	7.5	11	Select
Intense Extratropical Cyclone	8	12	12	17	11.5	15	Select
Extreme Extratropical Cyclone	13	18	15	20	17	17.5	Select
Monsoonal winds	4	7	6	11	6.5	9	Select

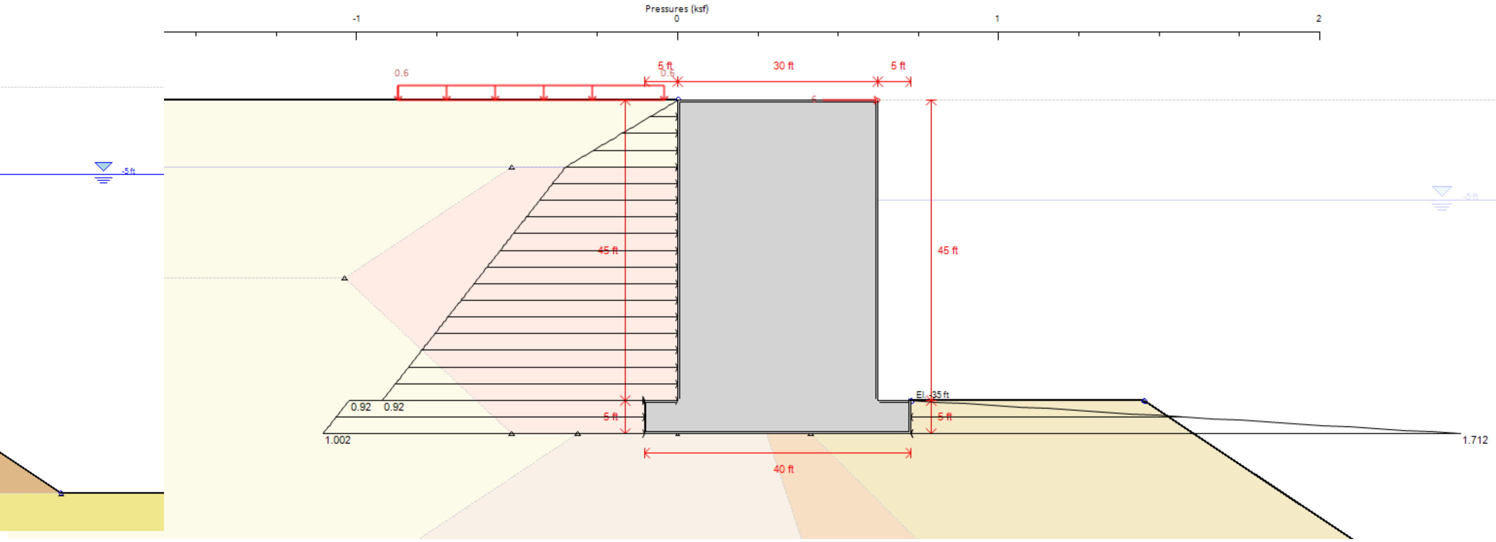
3.2.4 Seismic Pressures

- Seismic earth pressures
- Hydrodynamic pressures on seaside
- Mononobe-Okabe pressures
- Importance factor

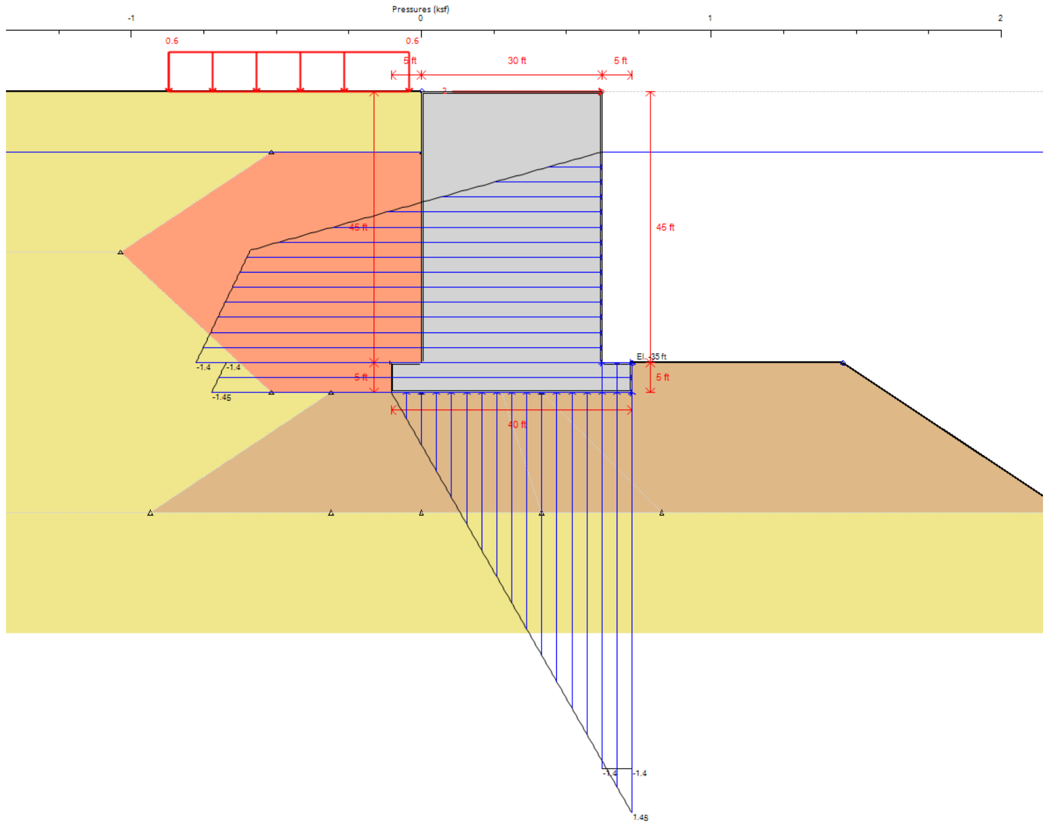


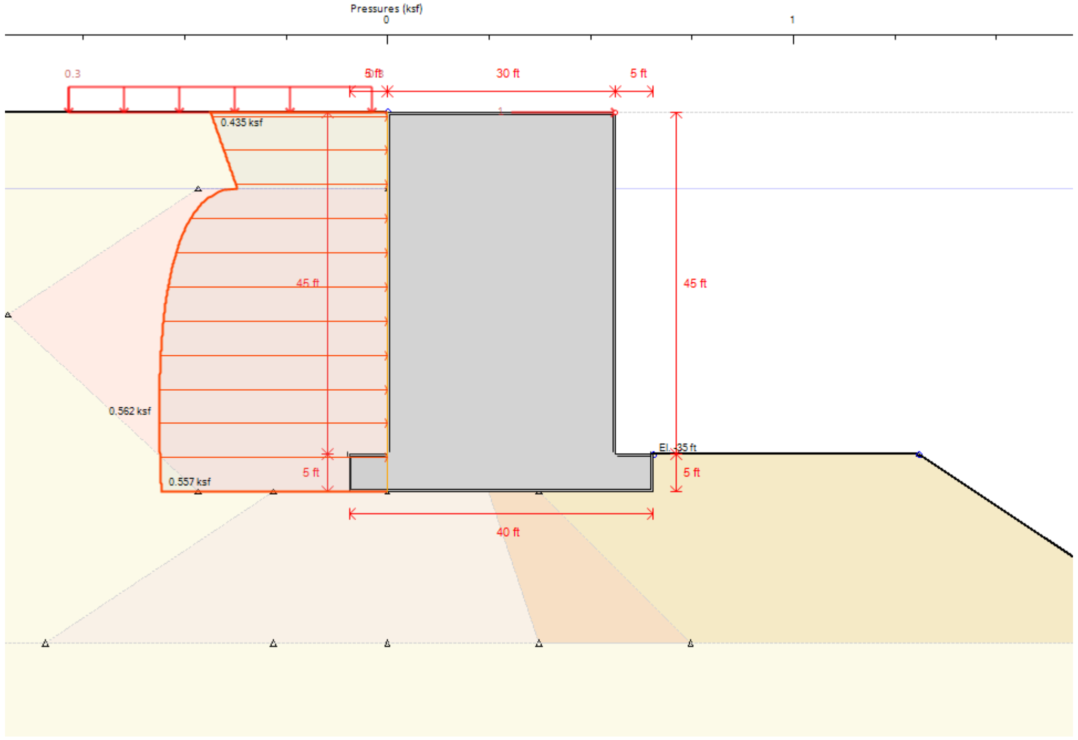
3.2.5 Gravity Wall Model



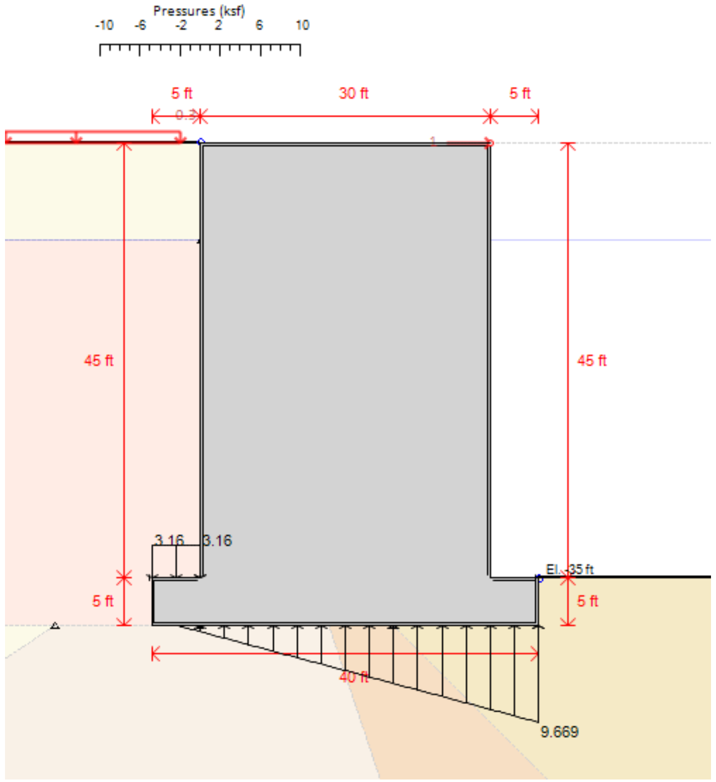


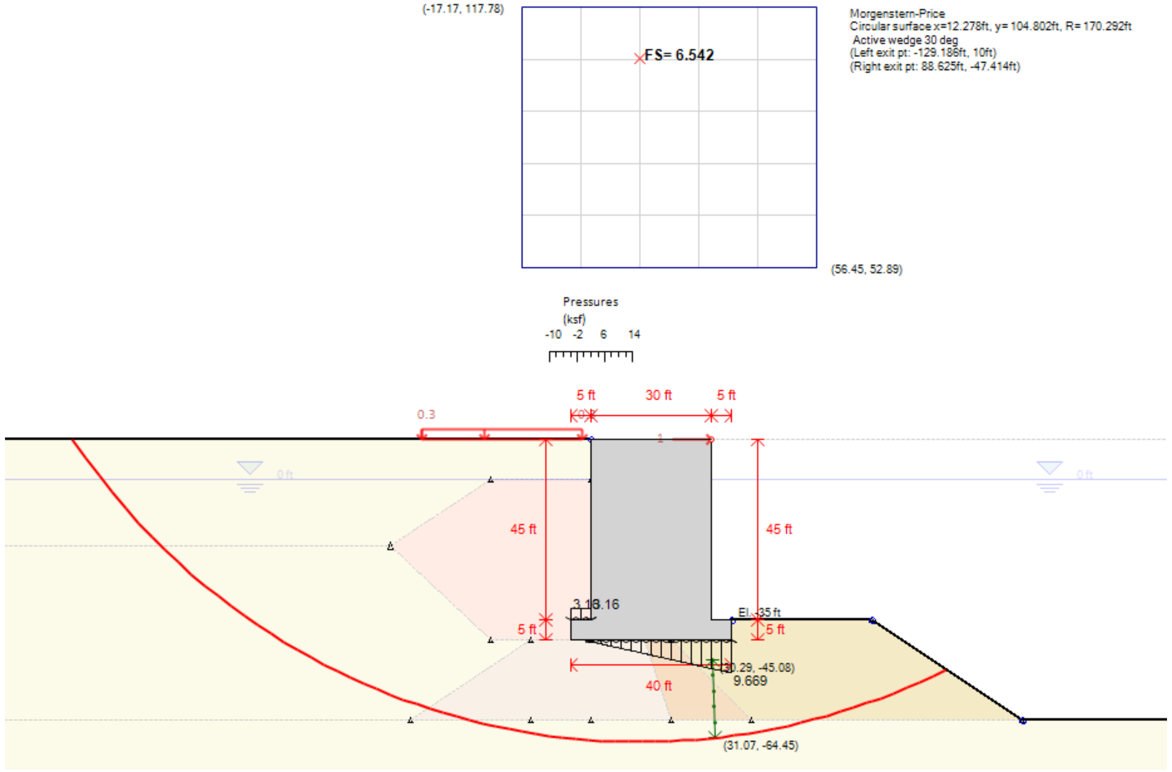
3.2.6 Wave & Seismic Pressures



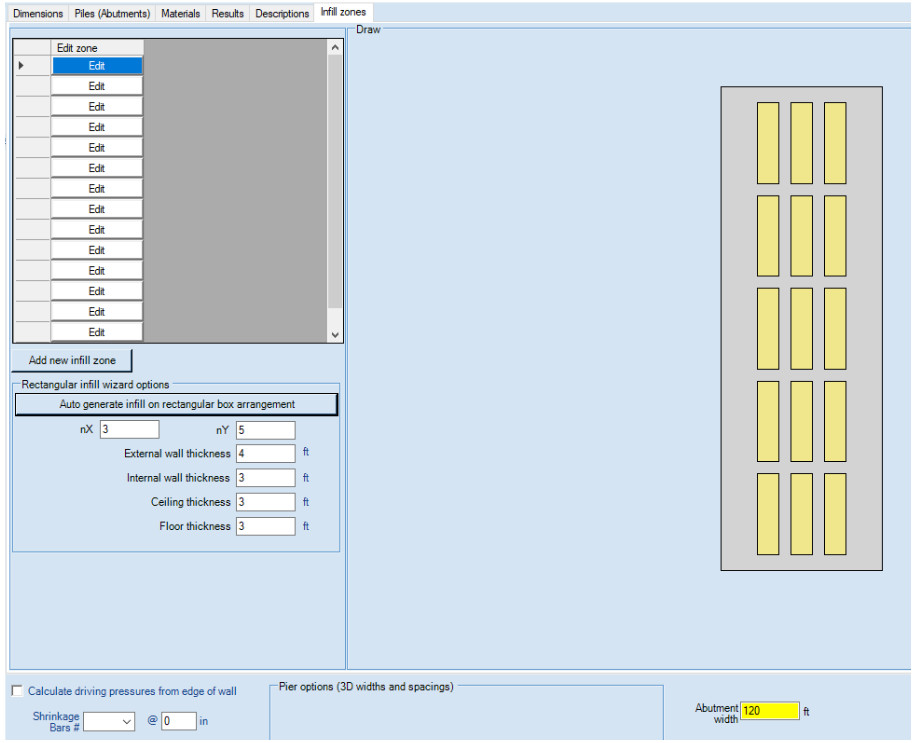


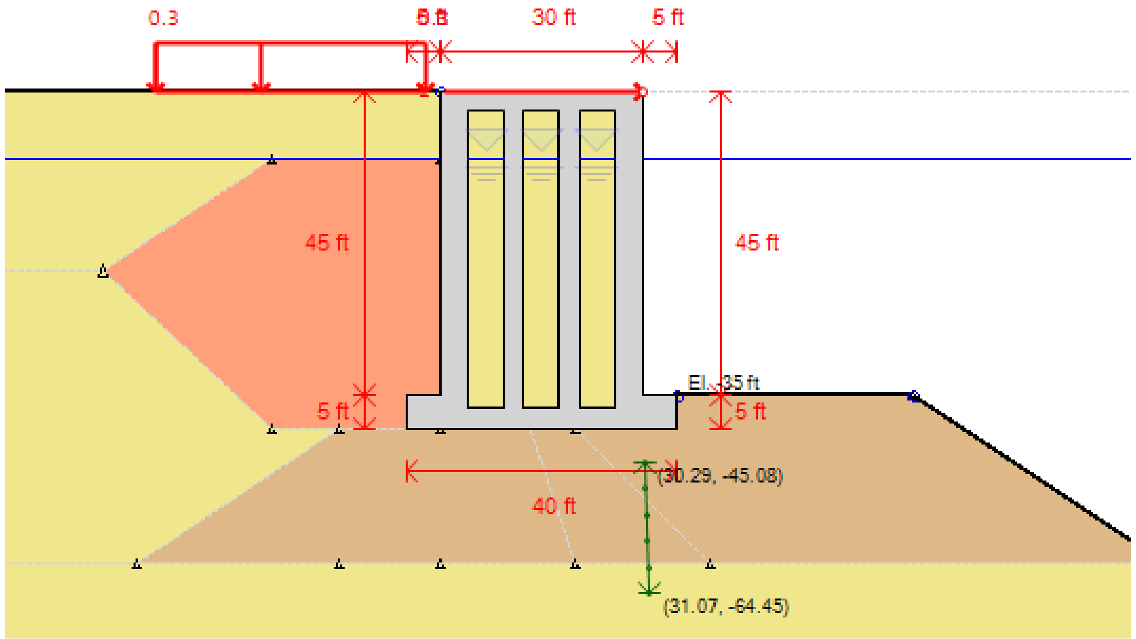
3.2.7 Bearing Capacity – Slope Stability





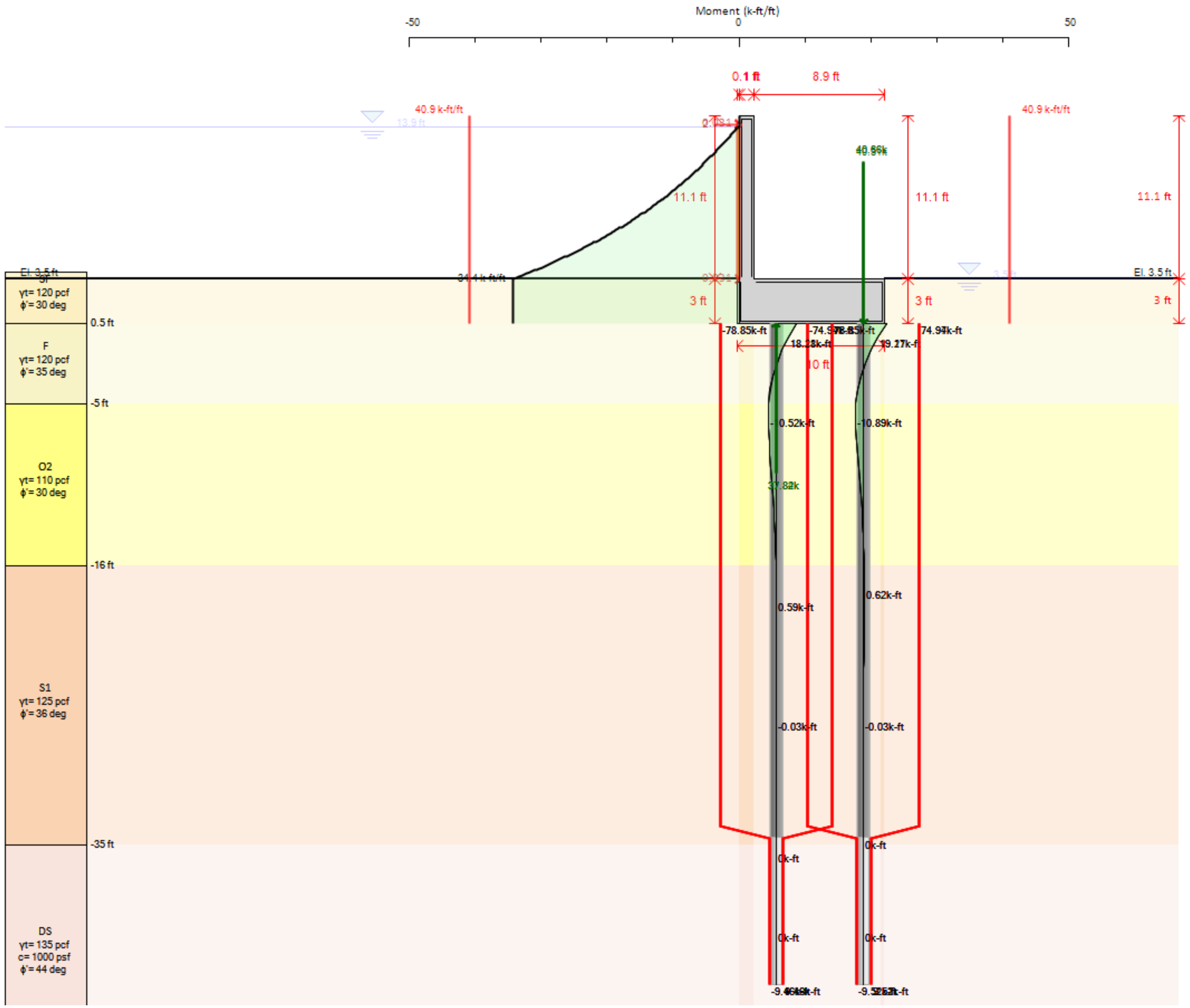
3.3 Caisson Walls



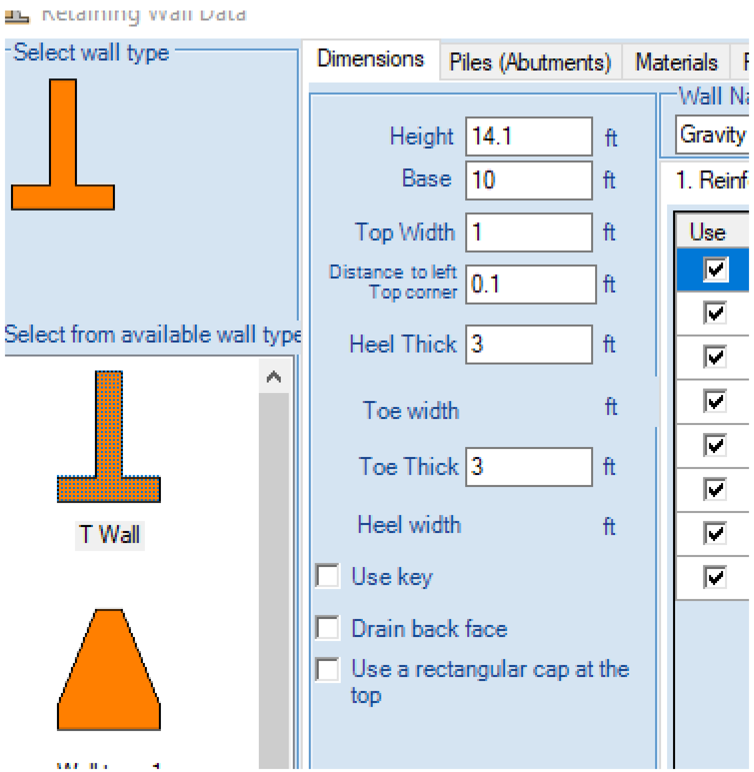


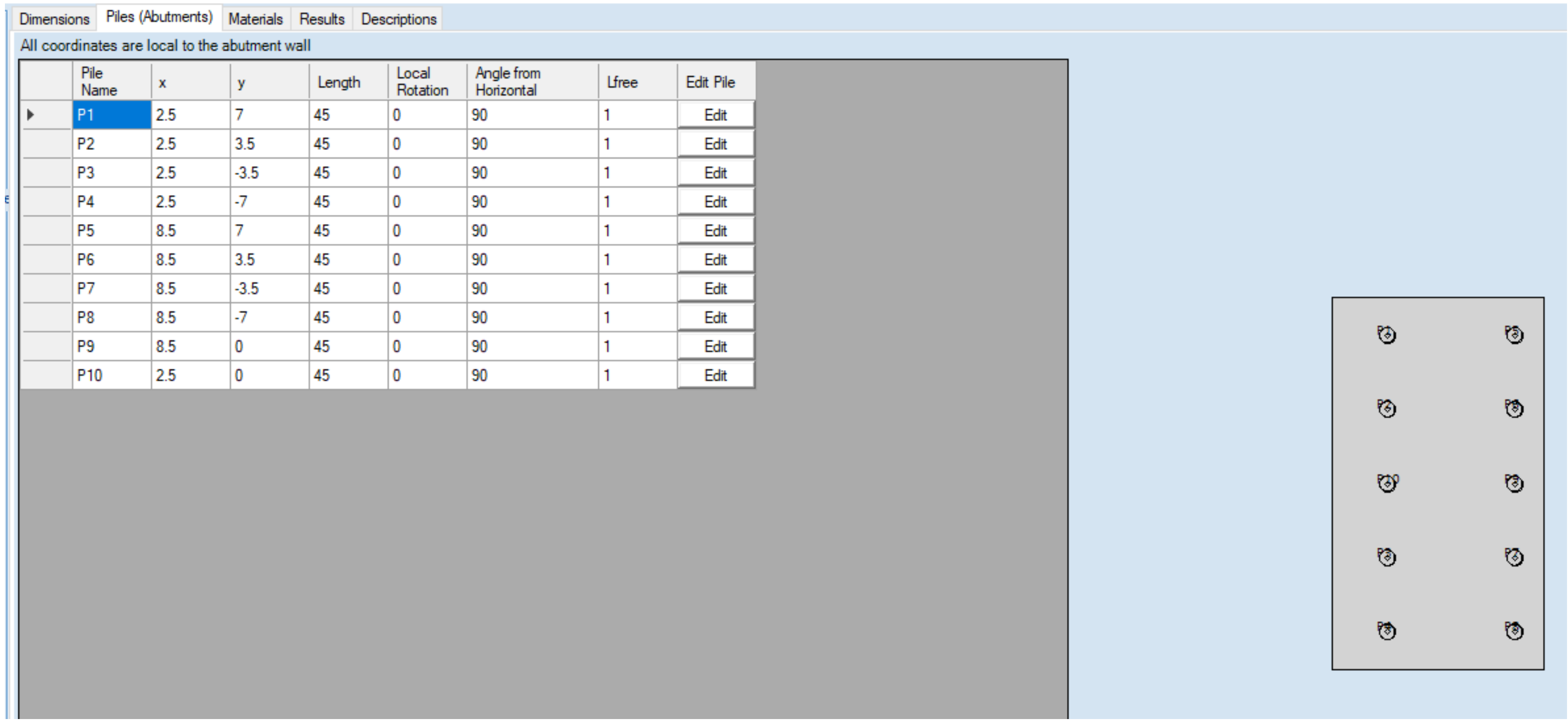
3.4.1 Floodwalls

- Use Gravity wall/Abutments
- Impact loads
- Wave pressures
- Local definitions



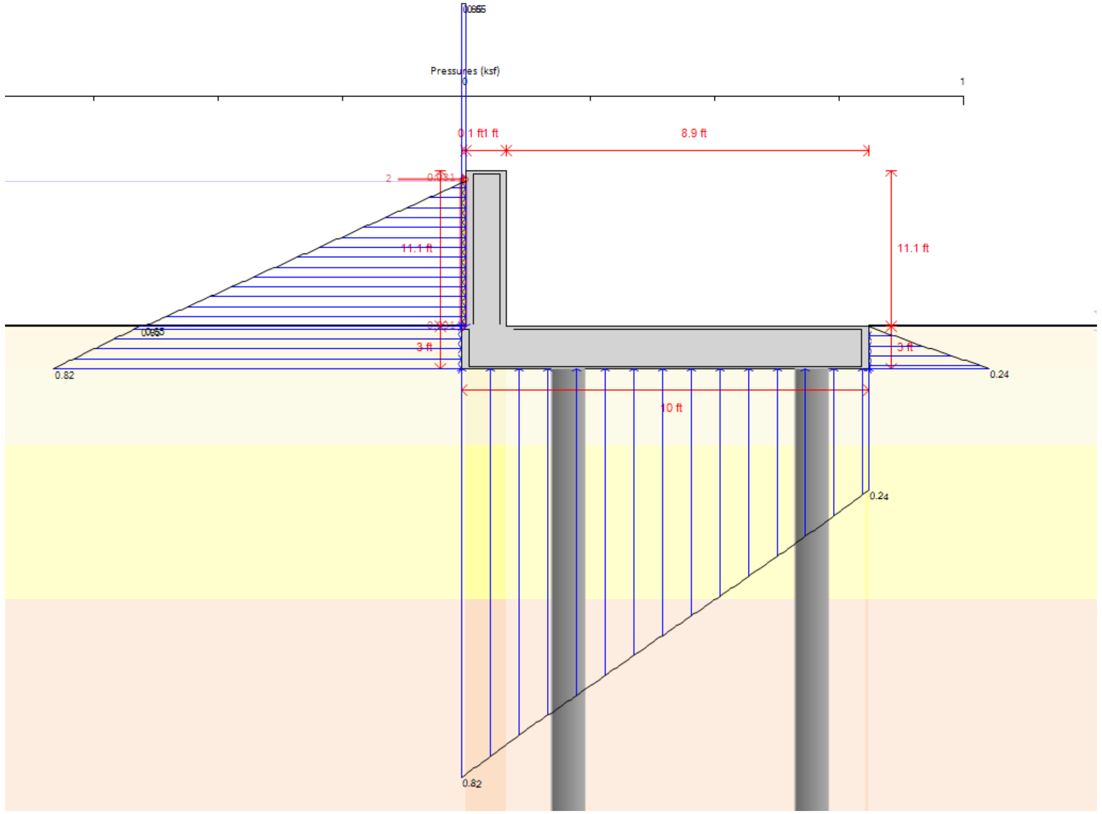
3.4.2 Abutment Wall (17.5 ft)

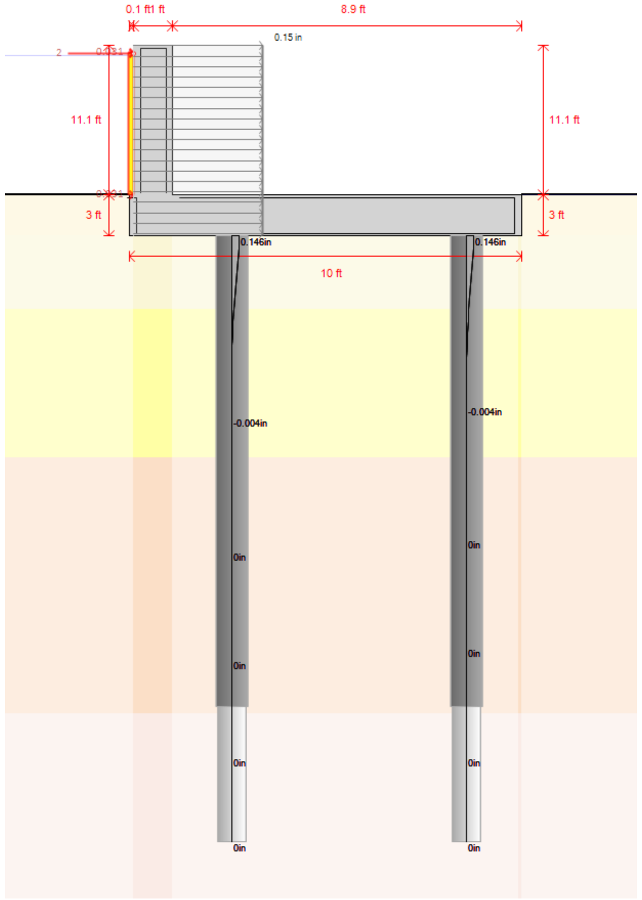




Pile Name	x	y	Length	Local Rotation	Angle from Horizontal	Lfree	Edit Pile
P1	2.5	7	45	0	90	1	Edit
P2	2.5	3.5	45	0	90	1	Edit
P3	2.5	-3.5	45	0	90	1	Edit
P4	2.5	-7	45	0	90	1	Edit
P5	8.5	7	45	0	90	1	Edit
P6	8.5	3.5	45	0	90	1	Edit
P7	8.5	-3.5	45	0	90	1	Edit
P8	8.5	-7	45	0	90	1	Edit
P9	8.5	0	45	0	90	1	Edit
P10	2.5	0	45	0	90	1	Edit

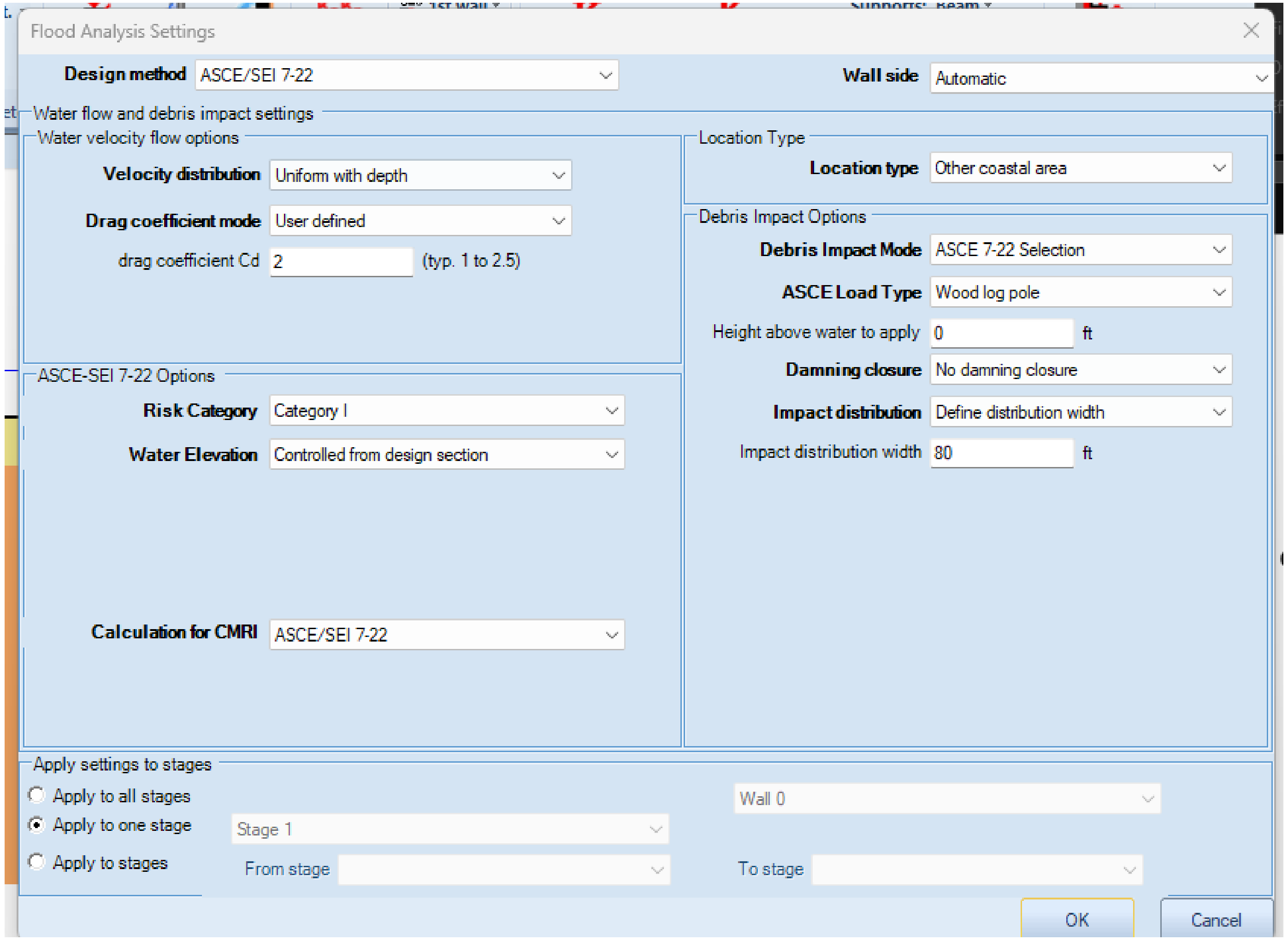
3.4.3 Water Pressures/Displacements





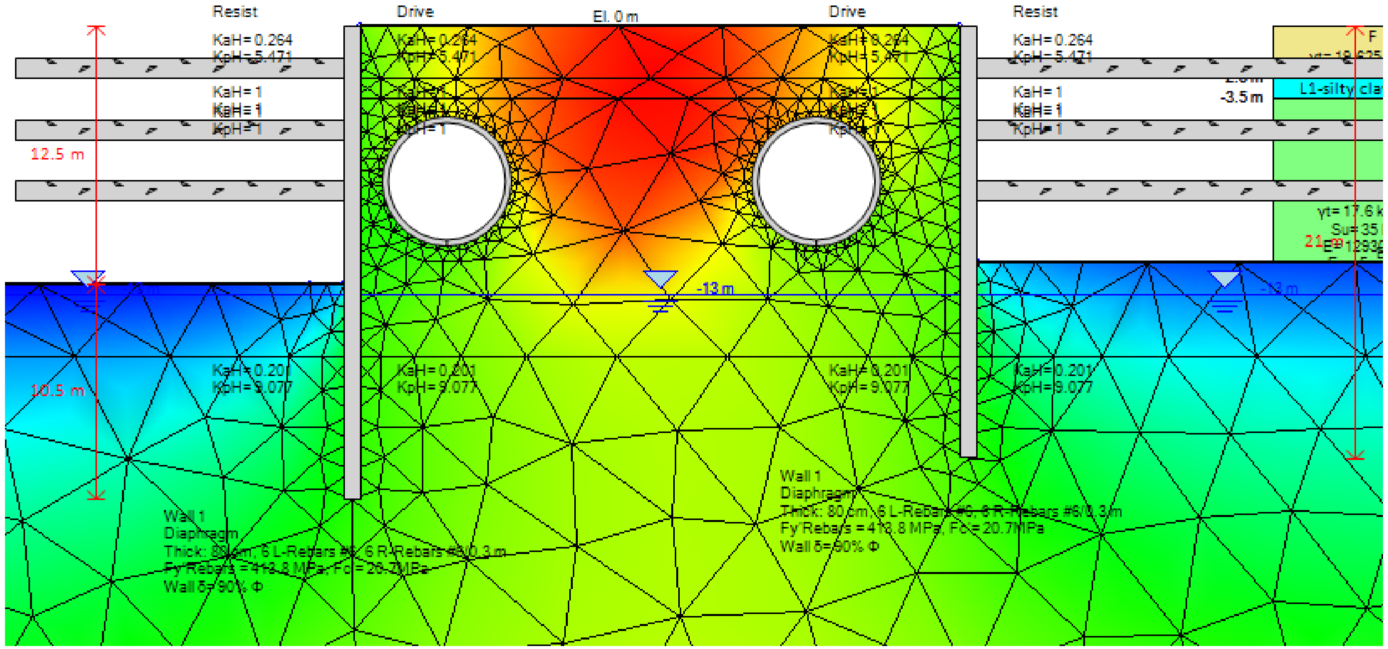
3.4.4 ASCE 7-22 Supplement 2

- Impact forces
- Load combinations
- Wave pressures
- Flow pressures

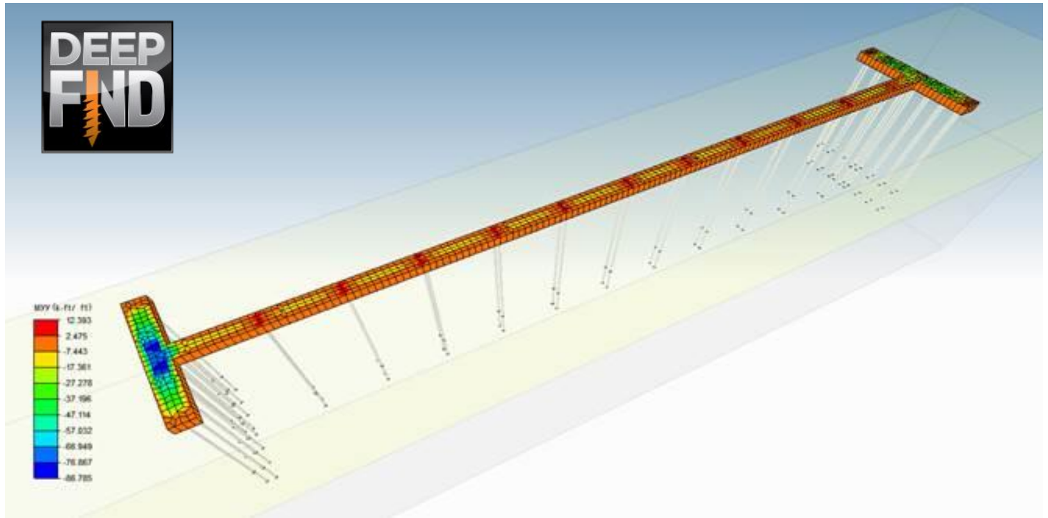


4. Finite element theory and application

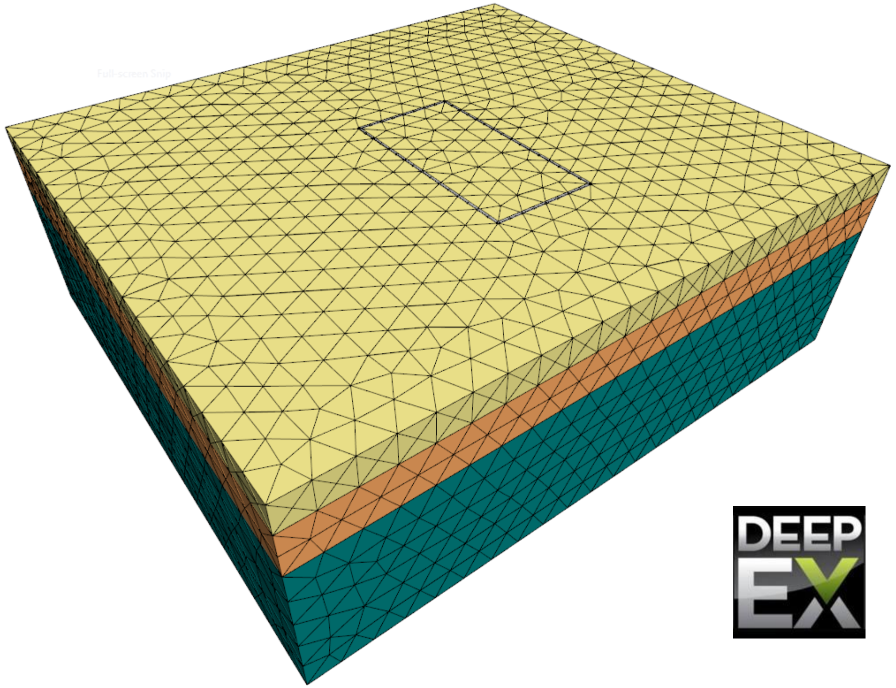
- Introduction to the finite element method
- Theoretical background
- Staging and initial conditions
- Soil constitutive laws
- structural components
- Meshing and analysis options
- Tunneling
- shear strength reduction analysis
- earthquake analysis



Introduction – Numerical Analysis



Numerical simulations are part of everyday design tasks.

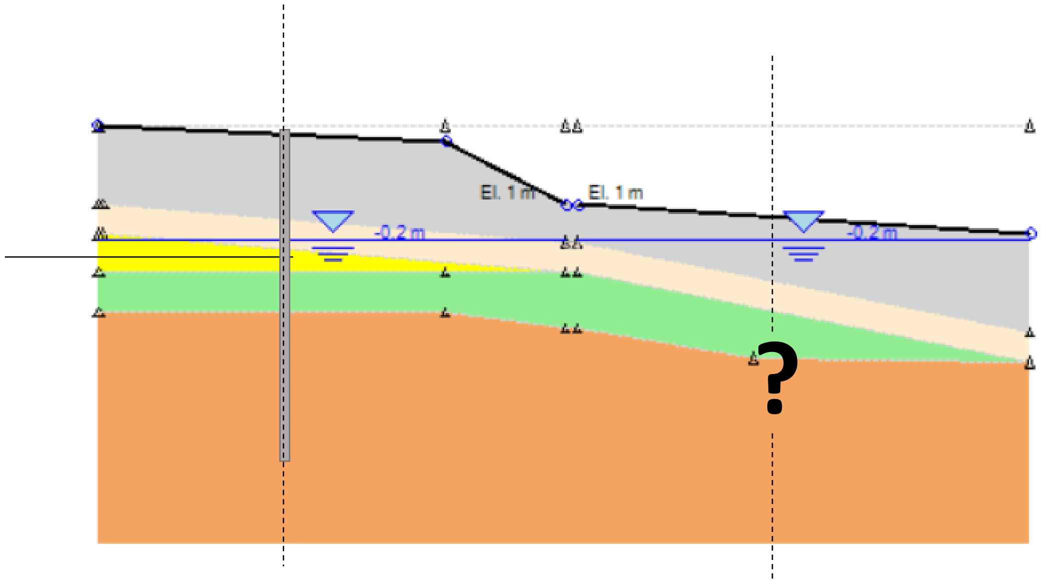


Why numerical simulations?

Cheap and fast way to forecast structure behavior.

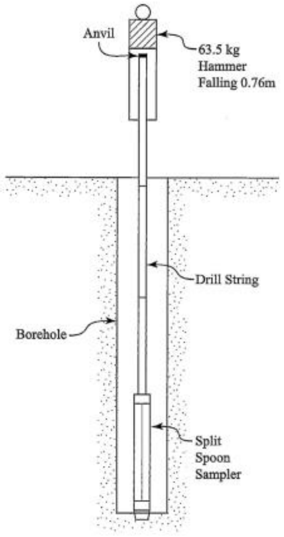
Introduction – Numerical Analysis

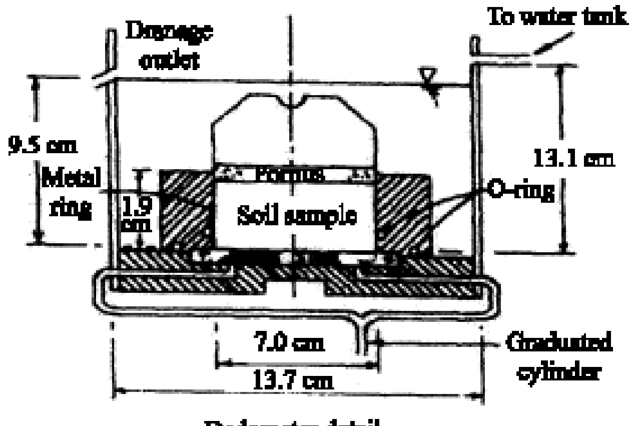
Simulating a geotechnical project → Not a straightforward task.



Limited information underneath the surface

On-site and lab tests not always reliable!

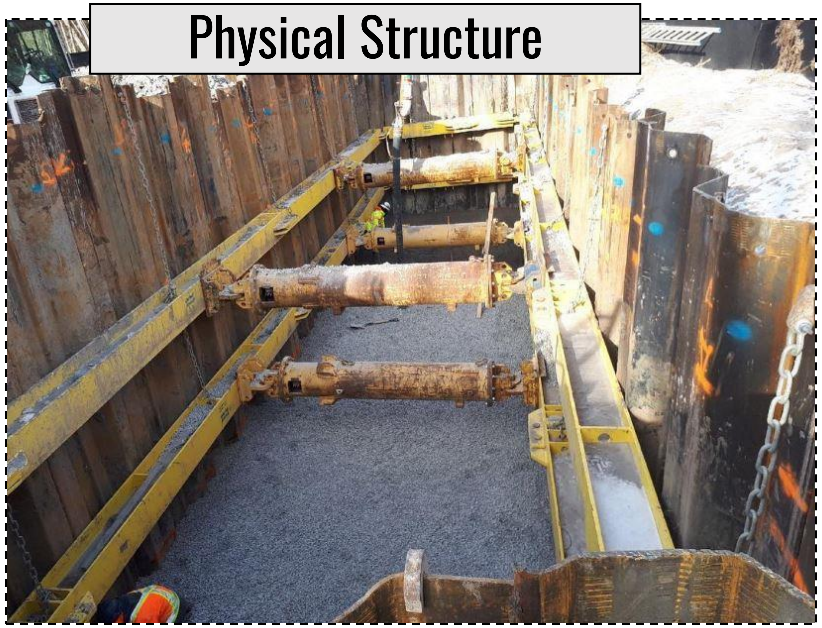




Introduction – Numerical Analysis

The designer must also cope with modeling decisions.

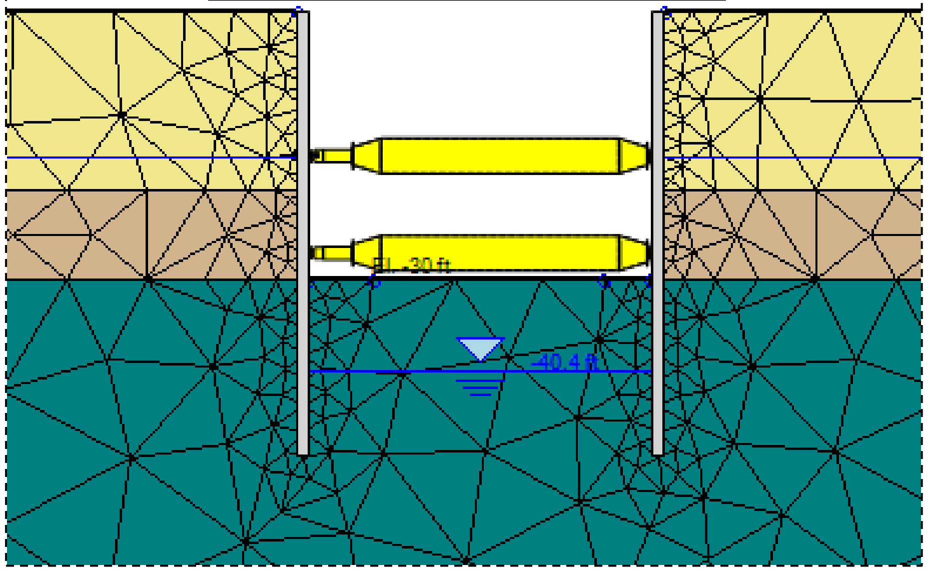
Physical Structure



(A) Soil stratigraphy, material properties, excavation geometry, support design

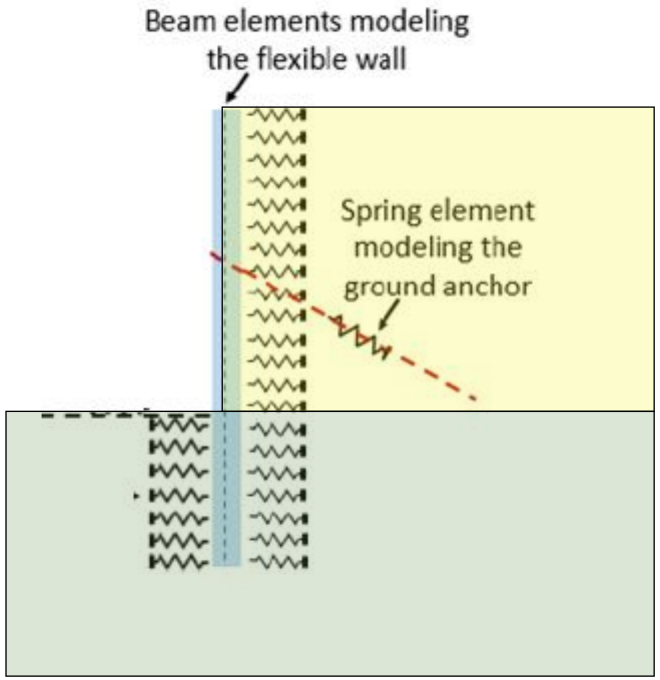
(B) Modelling decisions (Analysis method, constitutive models etc)

Numerical Model



Very often engineers are unaware of their model limitations.

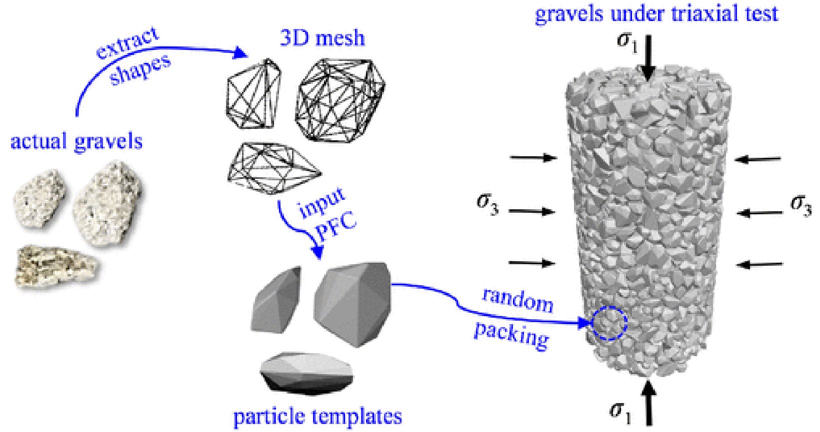
Introduction to the finite element method



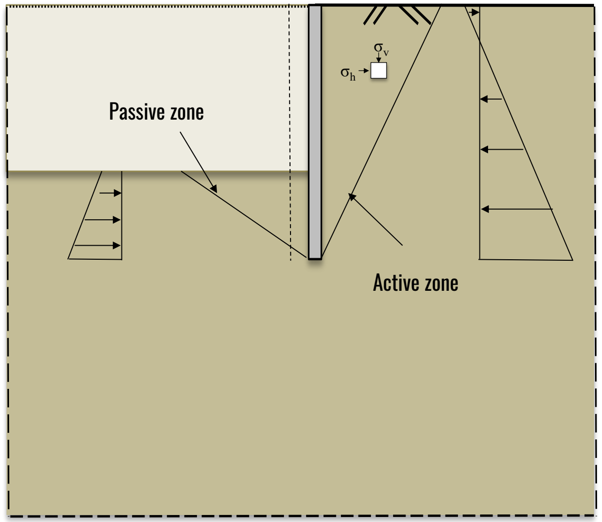
Nonlinear Spring
Winkler method

Numerical methods

Tan et al (2021)



Discrete Element Method

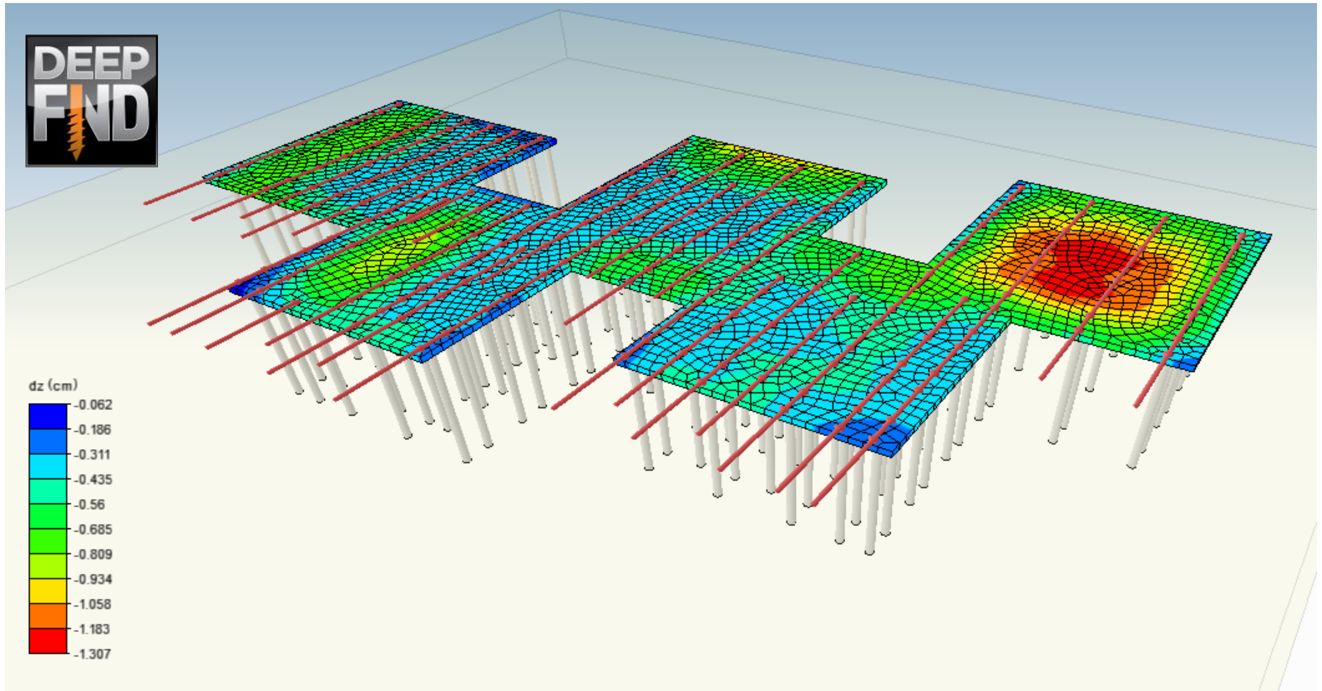


Limit equilibrium Method

Different strengths and limitations
from method to method!

Introduction to the finite element method

The Finite element method - a well-established numerical analysis method.



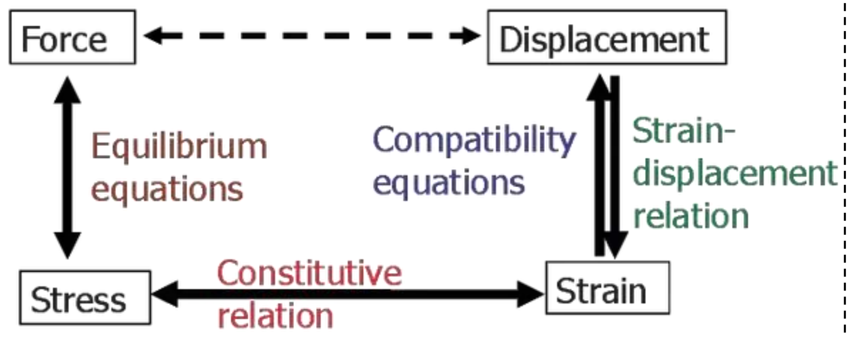
- Broad range of application
- complex geometry and boundary conditions
- simulation of the project as a whole

FEM Theoretical background

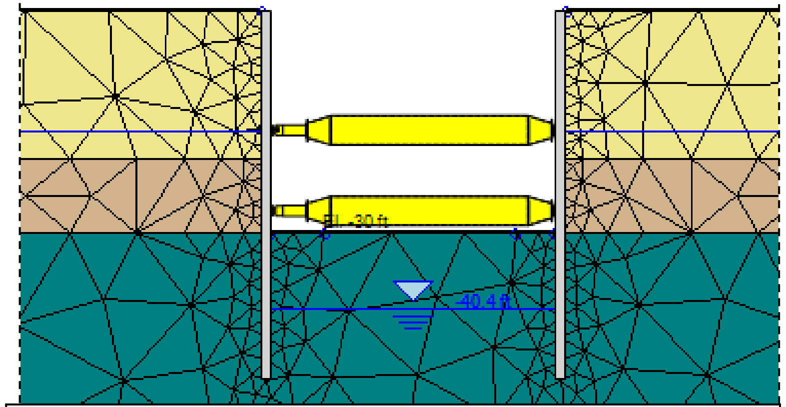
Physical structure

Modelling assumptions

Mathematical formulation



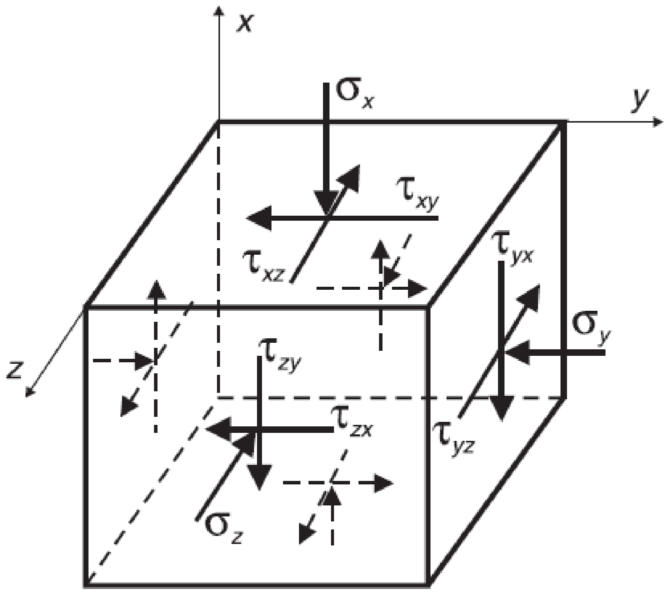
Discretization



FEM numerical model

FEM Theoretical background

(a) Equilibrium equations



Stresses within the soil medium must satisfy equilibrium

External body forces

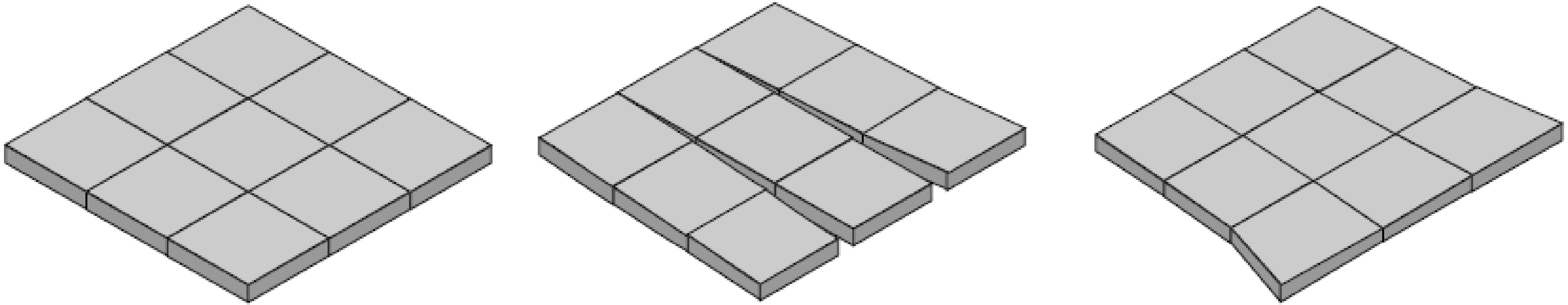
$$div_{\boldsymbol{x}}\boldsymbol{\sigma} + \boldsymbol{b} = \rho\ddot{\boldsymbol{u}}$$

Divergence of stress tensor

Acceleration (equal to zero for static analysis)

Mathematical form: Cauchy momentum equation

(b) Compatibility equations



(a) Original (b) Non-compatible (c) Compatible

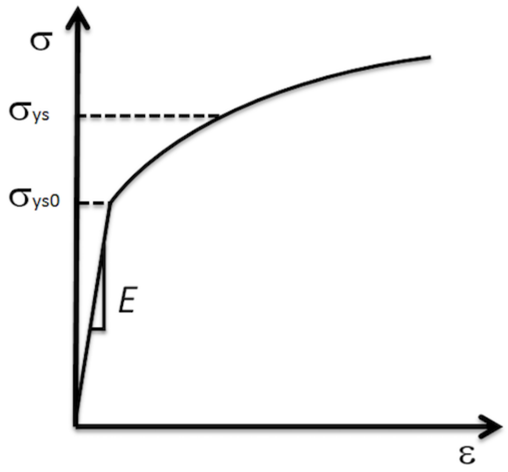
Strain to displacement relationship:

$$\varepsilon_x = -\frac{\partial u}{\partial x}; \quad \varepsilon_y = -\frac{\partial v}{\partial y}; \quad \varepsilon_z = -\frac{\partial w}{\partial z}$$

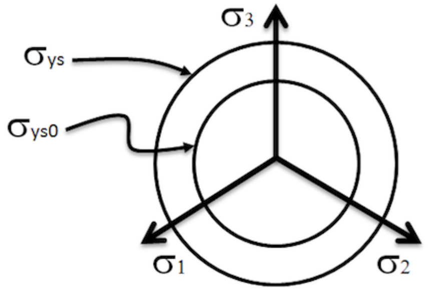
$$\gamma_{xy} = -\frac{\partial v}{\partial x} - \frac{\partial u}{\partial y}; \quad \gamma_{yz} = -\frac{\partial w}{\partial y} - \frac{\partial v}{\partial z}; \quad \gamma_{xz} = -\frac{\partial w}{\partial x} - \frac{\partial u}{\partial z}$$

FEM Theoretical background

(c) Constitutive law



Uniaxial stress-strain curve



Yield surface in principal stress-space

Numerical simulations are part of everyday design tasks.

Constitutive law

$$\boldsymbol{\Delta\sigma} = f_m(\boldsymbol{\Delta\varepsilon})$$

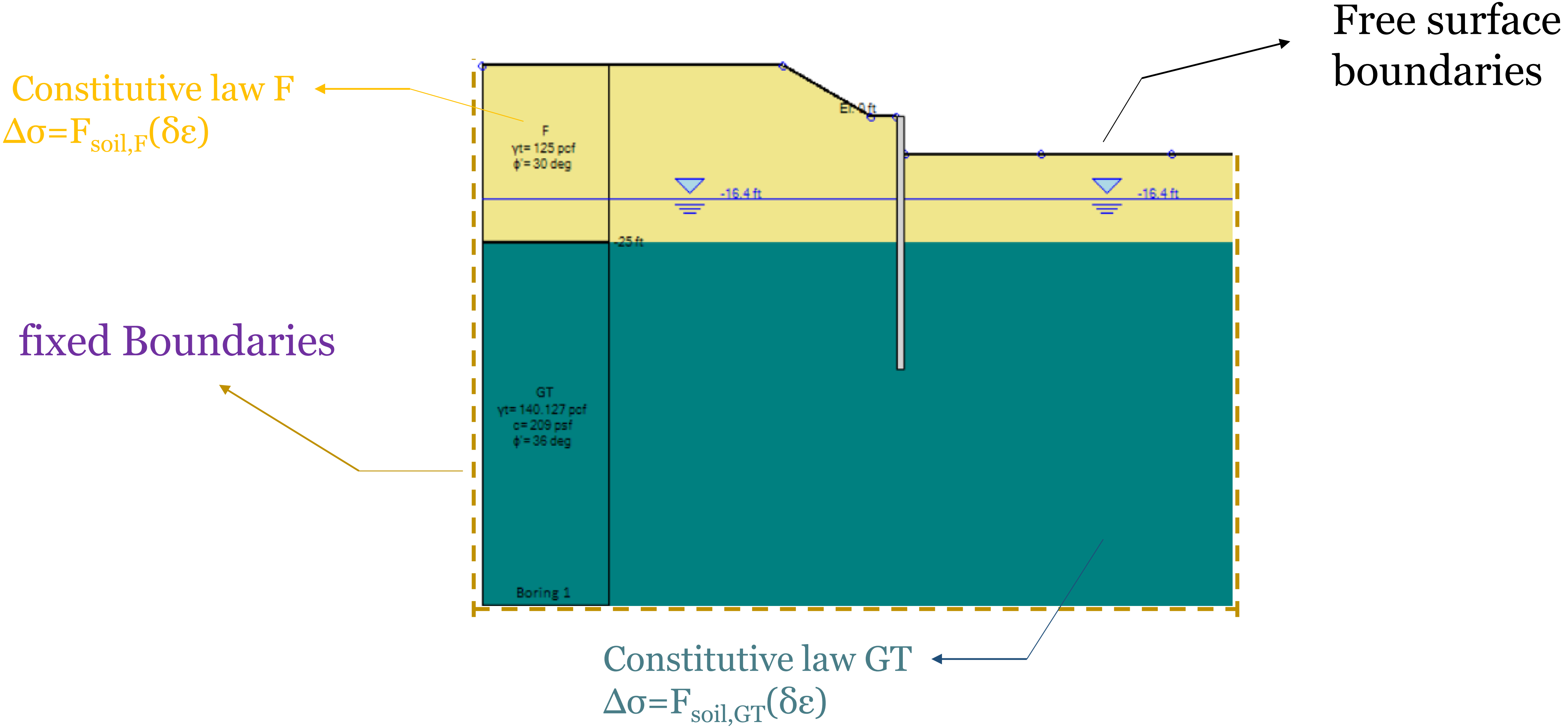
$$\begin{Bmatrix} \Delta\sigma_x \\ \Delta\sigma_y \\ \Delta\sigma_z \\ \Delta\tau_{xy} \\ \Delta\tau_{xz} \\ \Delta\tau_{zy} \end{Bmatrix} \qquad \begin{Bmatrix} \Delta\varepsilon_x \\ \Delta\varepsilon_y \\ \Delta\varepsilon_z \\ \Delta\gamma_{xy} \\ \Delta\gamma_{xz} \\ \Delta\gamma_{zy} \end{Bmatrix}$$

Stress increment

Strain Increment

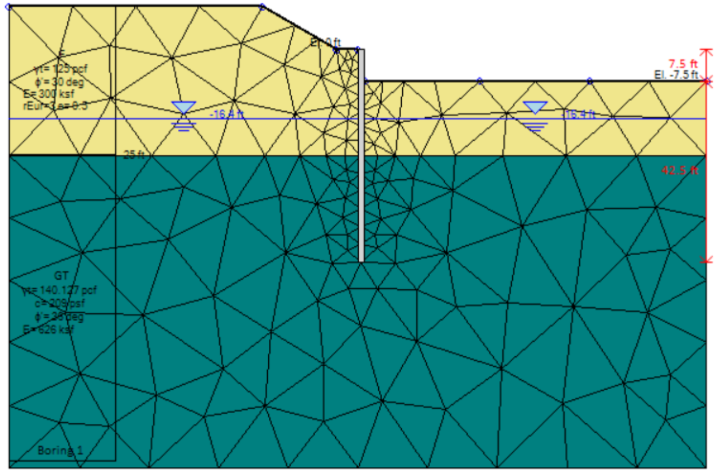
FEM Theoretical background

(e) Boundary Conditions and model geometry

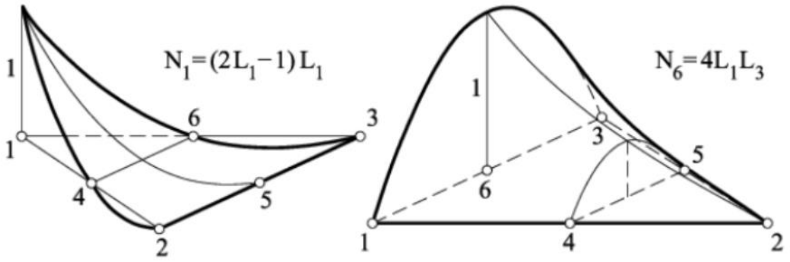


FEM Theoretical background

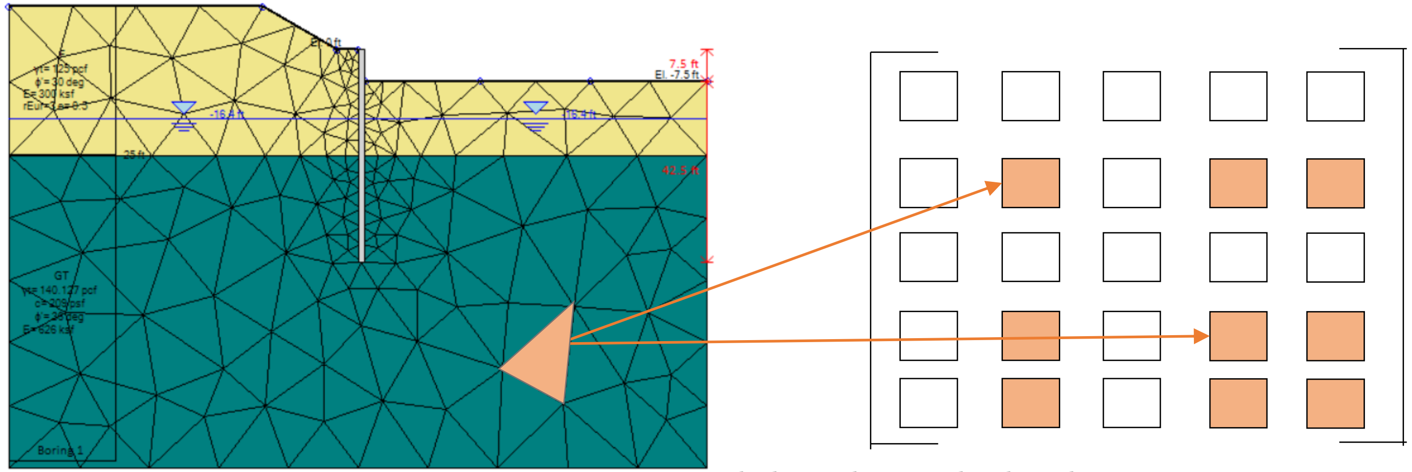
Reformulating the PDE system to an easily solvable system of algebraic equations!!



Step 1: Discretize the continuum



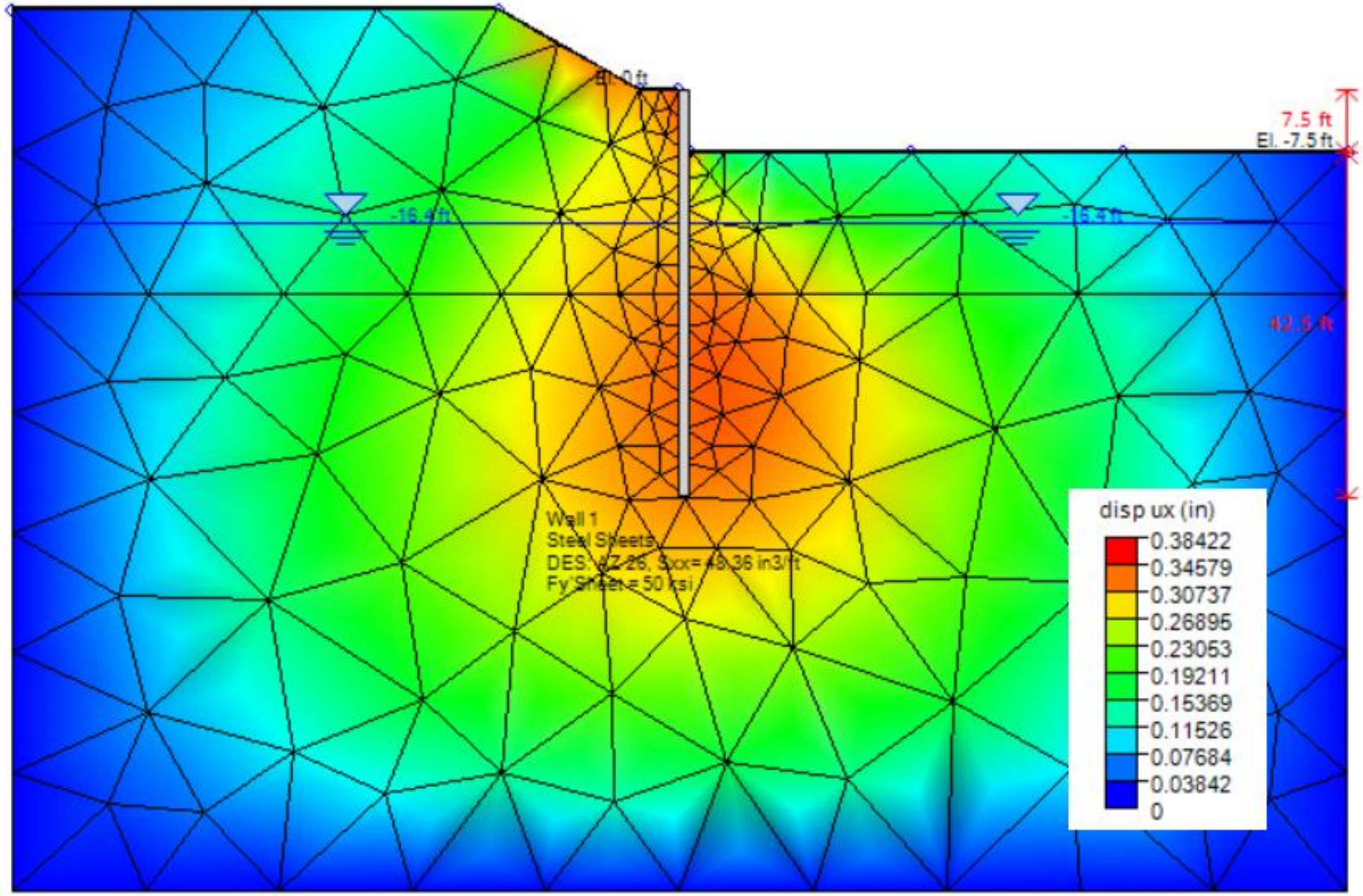
Step 2: Select interpolation functions



Step 3: Assemble the global equation system

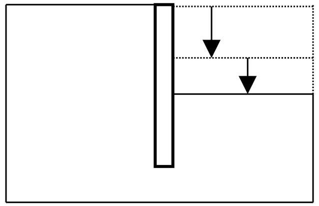
FEM Theoretical background

What results can you obtain from FEM?

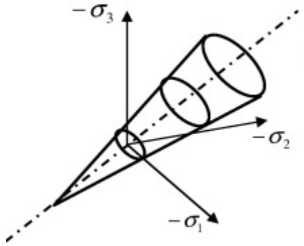


- Displacement and stress nodal properties
- displacement and stresses in any element location (interpolation)
- failure states!

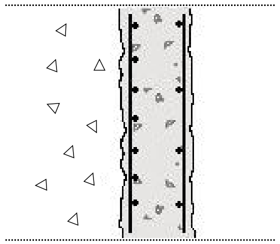
Good practice in FEM model construction



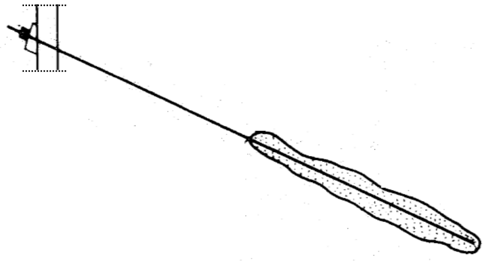
Staging



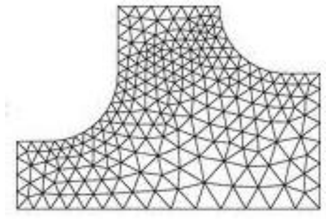
Soil constitutive laws



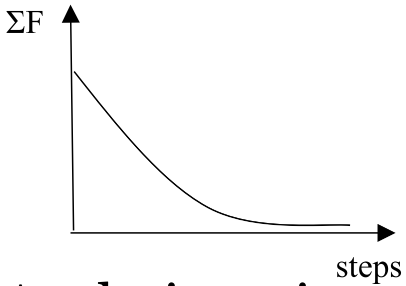
soil-structure interface



structural components

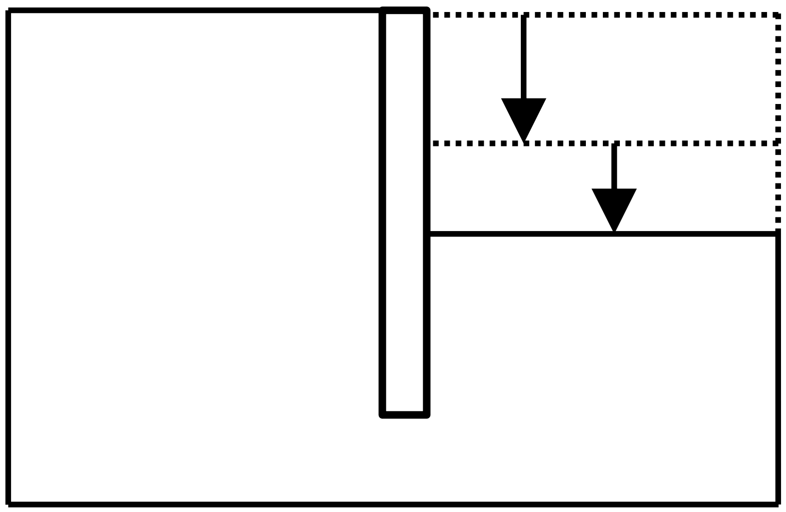


Meshing



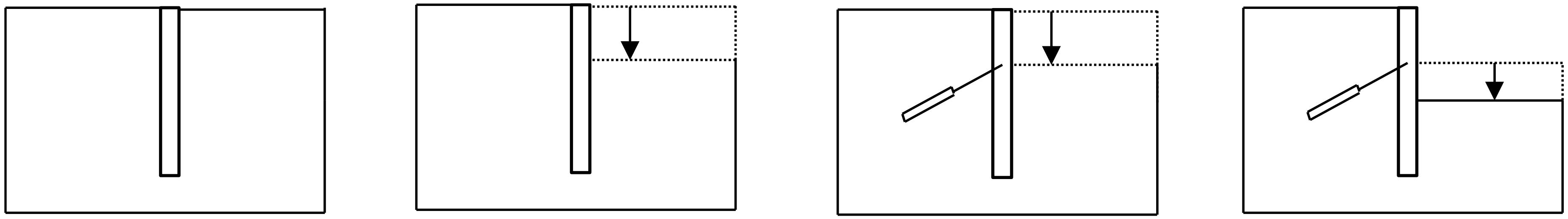
Analysis options

Good practice in FEM model construction



Staging

Staging sequence

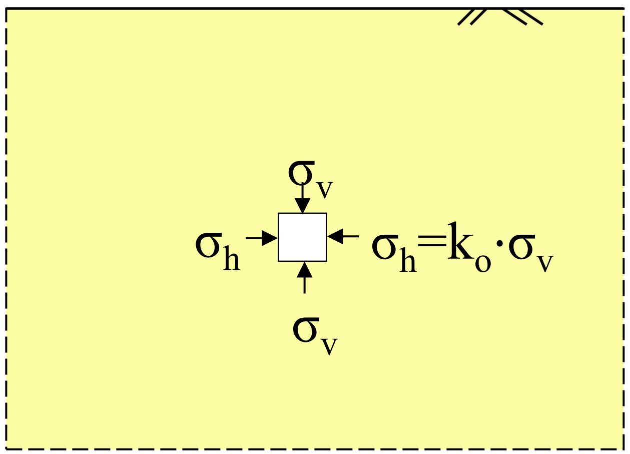


Why is staging necessary?

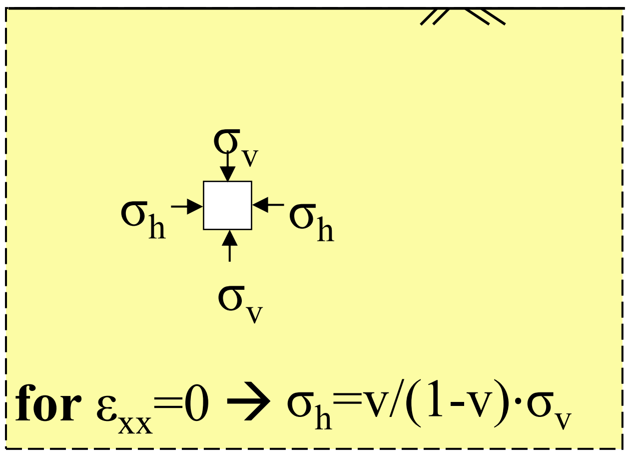
- Accurate representation of construction process
- smooth convergence for the FEM model

Staging sequence

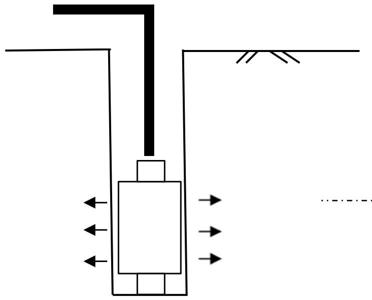
Initial stage in FEM analysis : Greenfield conditions.



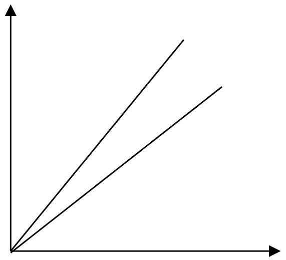
Imposed stress field
(ko)



Gravity Analysis



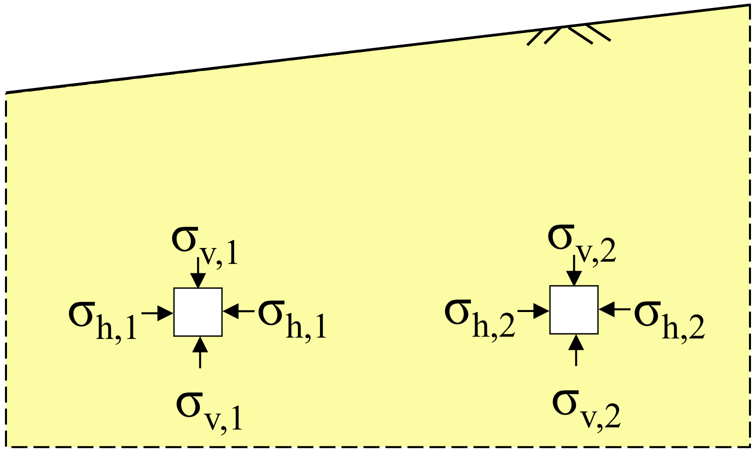
On site measurement
(pressuremeter test)



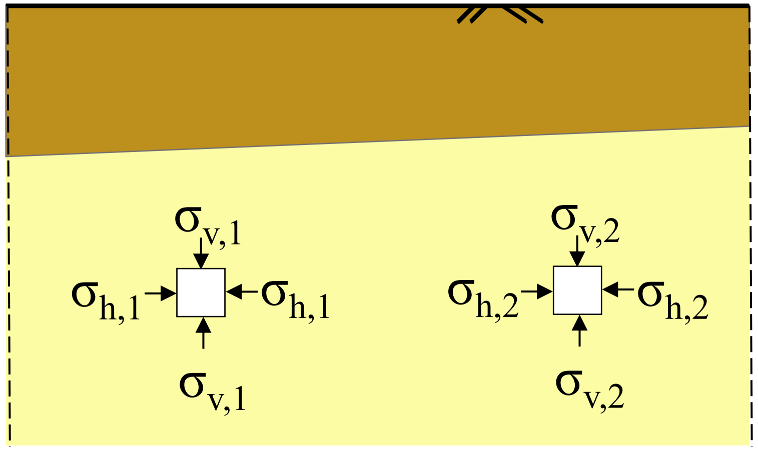
Corelation formulas
(k_o to OCR etc)

Model Staging Sequence

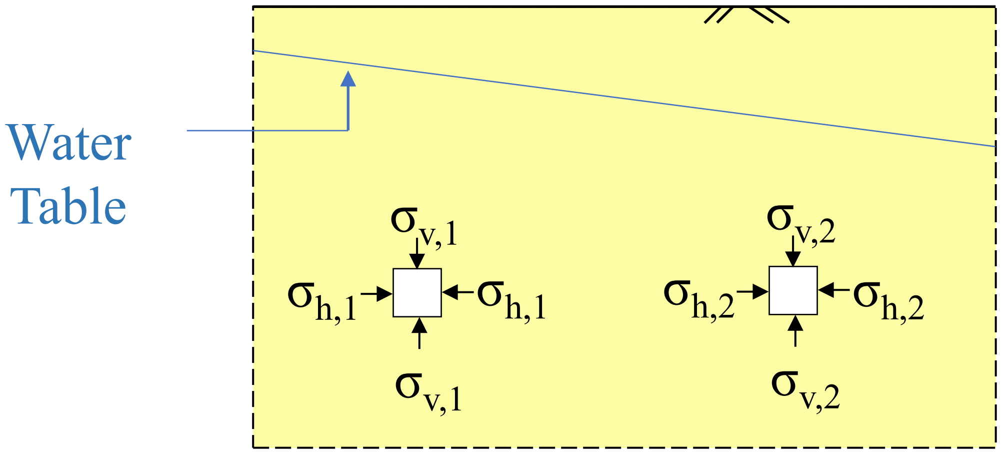
Imposed stress field: equilibrium **not** always satisfied!!



(a) Inclined surface



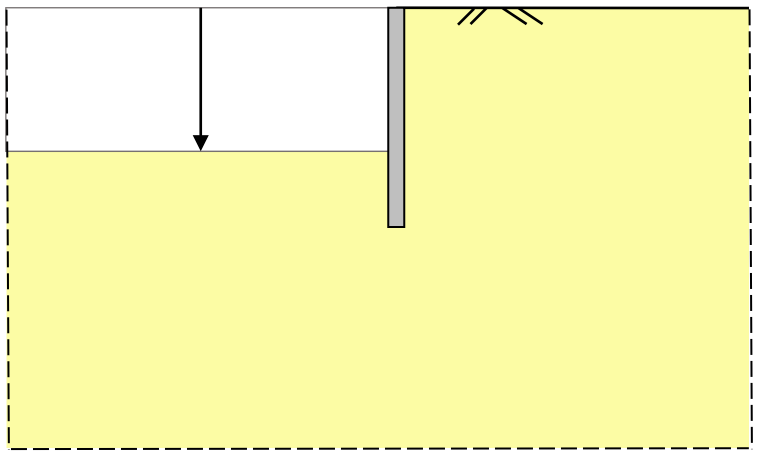
(b) Inclined layers



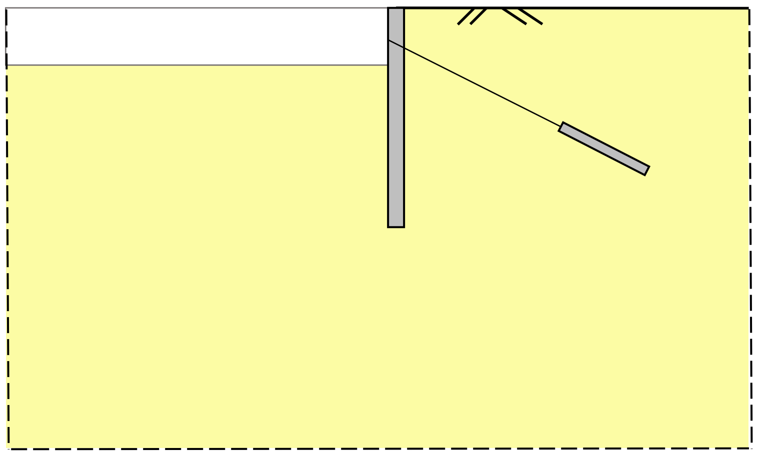
(c) Inclined water table

Model Staging Sequence

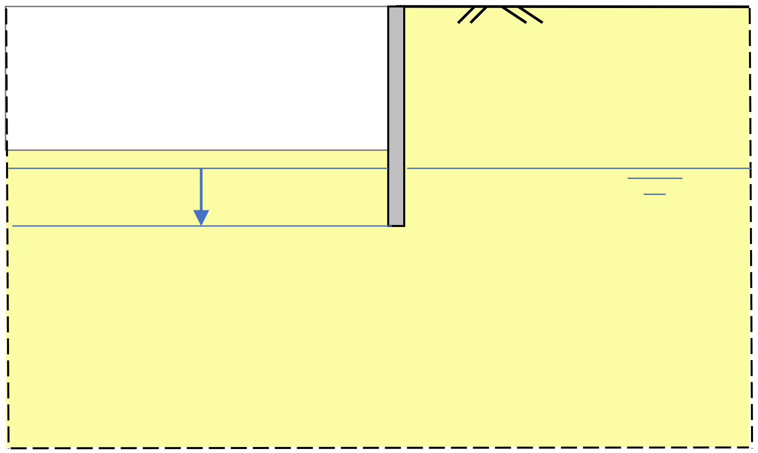
Rule of thumb: separate stage for each action.



(a)Excavation



(b)Support installation



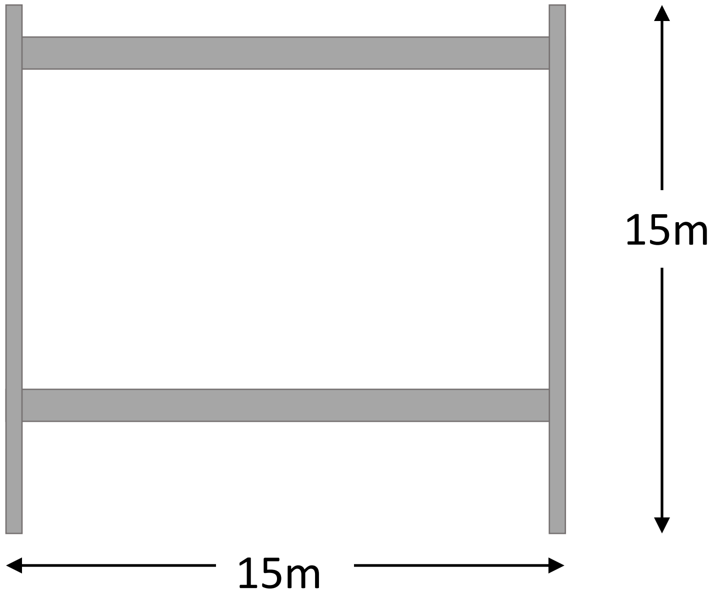
(c)Dewatering

Model Staging Sequence

Example 1: Staging in cut and cover tunnel construction.

Model Staging Sequence

Example 1: Staging in cut and cover tunnel.

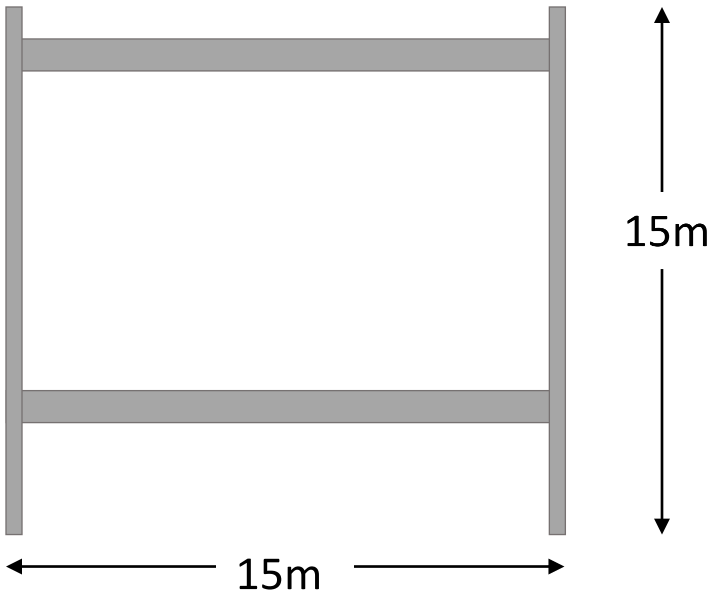


Bottom up construction:

Stage 0 : greenfield conditions
Stage 1: installation of walls (Diaphragm 0.6m)
Stage 2: excavation at depth **-3.5** m
Stage 3: add a temporary prop at depth -3m
stage 4: excavation at depth -8 m/ dewatering at -9m
Stage 5: add a temporary prop at depth -7m
stage 6: excavation at depth -12 m/ dewatering at -14.5m
stage 7: construct floor slab at -12 m (H=100cm)
stage 8: construct top slab at -1 m (H=70cm)
Stage 9 : remove temporary pros

Model Staging Sequence

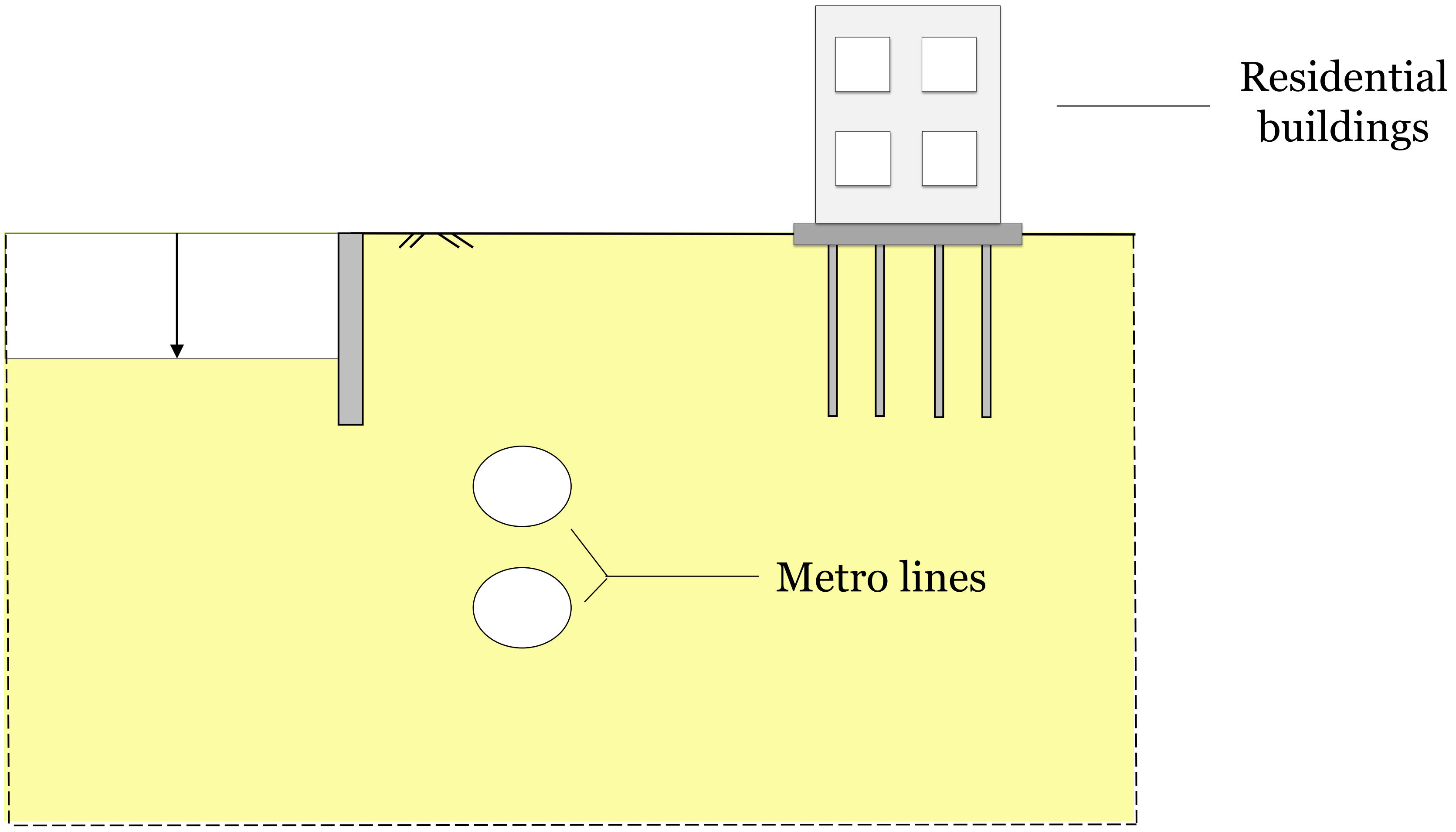
Example 1: Staging in cut and cover tunnel.



Top Down construction:
Stage 0 : greenfield conditions
Stage 1: installation of walls
Stage 2: excavation at depth **-1** m
Stage 3: construct top slab at -1 m
stage 4: excavation at depth -8 m / dewatering at -9m
Stage 5: add a temporary prop at depth -7m
stage 6: excavation at depth -12 m /dewatering at -14.5m
stage 7: construct floor slab at -12 m
Stage 8 : remove temporary pros

Model Staging Sequence

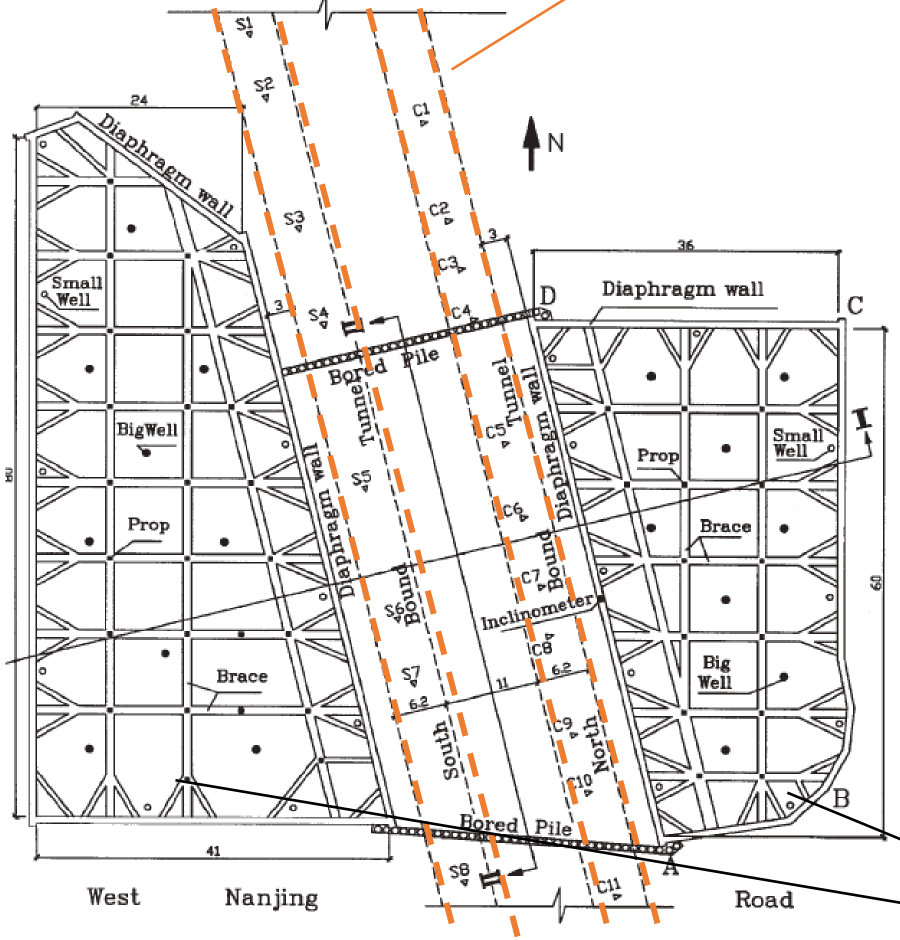
Presence of adjacent structures can **modify** state of stress!

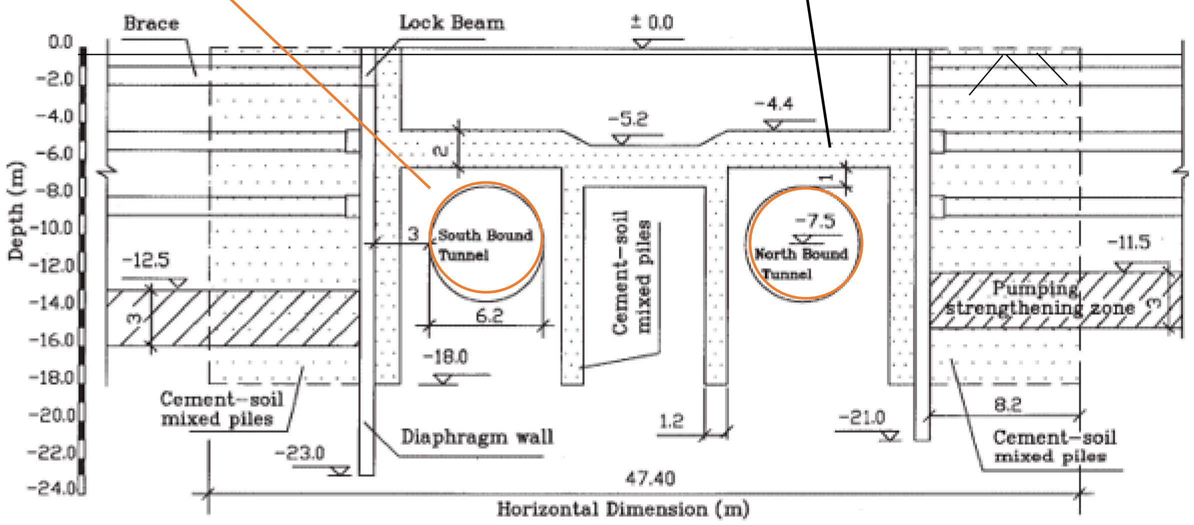


Model Staging Sequence

preexisting Shanghai Metro tunnels

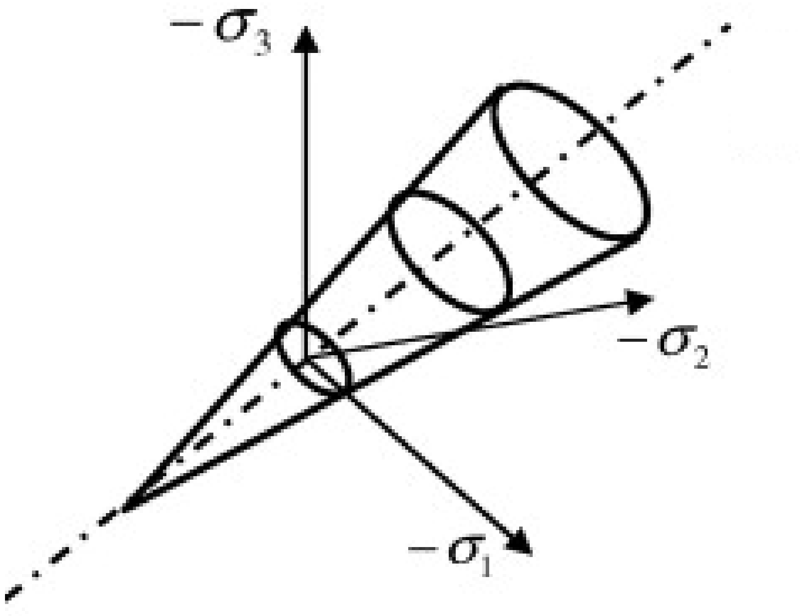
Cement – soil mixed piles





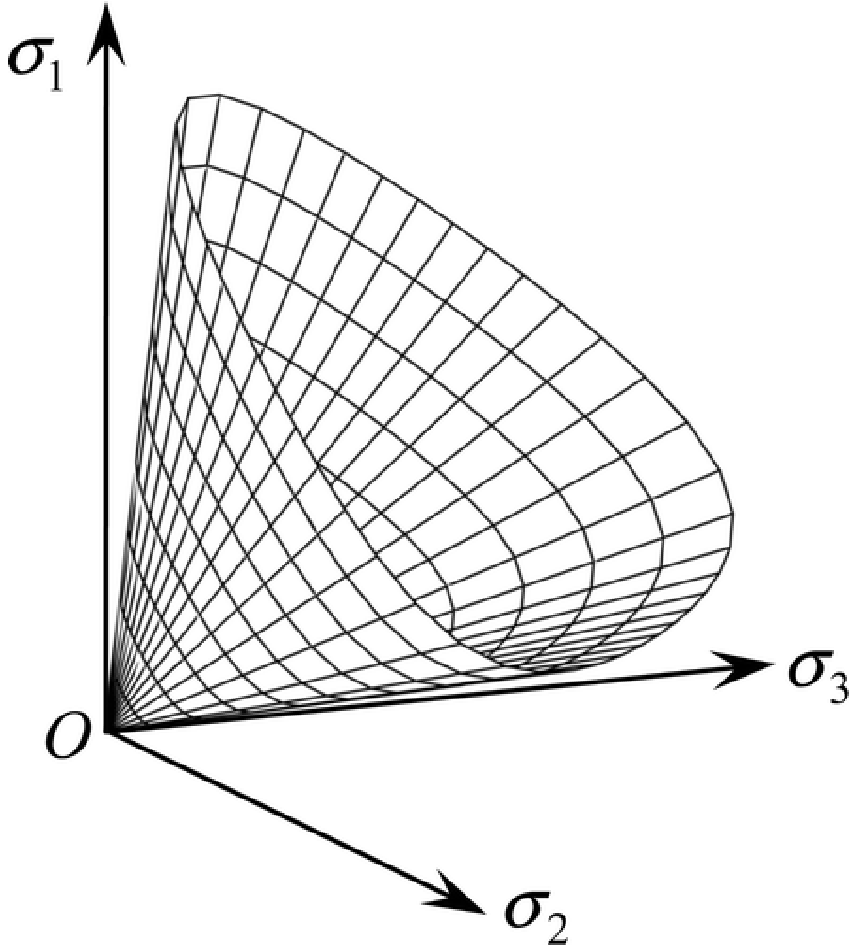
excavations for world Commercial City Complex

Good practice in FEM model construction



Soil constitutive laws

Soil Constitutive laws



Selection of soil constitutive law based on:

- Specific problem in question
- Available soil data

Constitutive law comprised of:

- Yield function
- plastic potential function
- hardening/softening rules

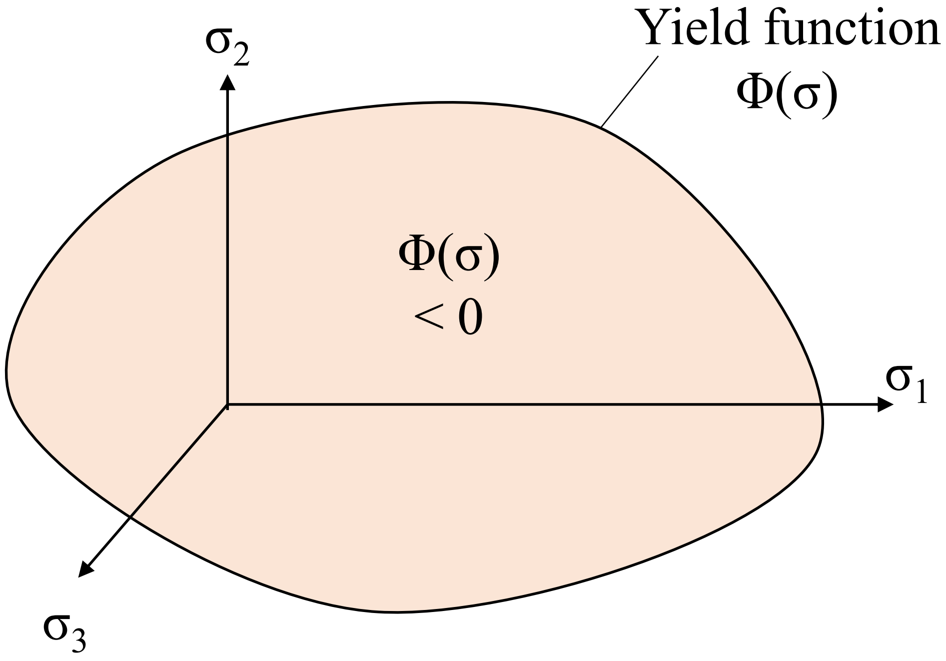
Model components: (a) Elastic behavior

$$\begin{bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{33} \\ \sigma_{23} \\ \sigma_{13} \\ \sigma_{12} \end{bmatrix} = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix} 1-\nu & \nu & \nu & 0 & 0 & 0 \\ \nu & 1-\nu & \nu & 0 & 0 & 0 \\ \nu & \nu & 1-\nu & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1-2\nu}{2} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1-2\nu}{2} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{1-2\nu}{2} \end{bmatrix} \begin{bmatrix} \varepsilon_{11} \\ \varepsilon_{22} \\ \varepsilon_{33} \\ 2\varepsilon_{23} \\ 2\varepsilon_{13} \\ 2\varepsilon_{12} \end{bmatrix}$$

Elastic parameters **linearly mapping** the strain tensor to the stress tensor.

Model Ingredients: (a) Yield Function

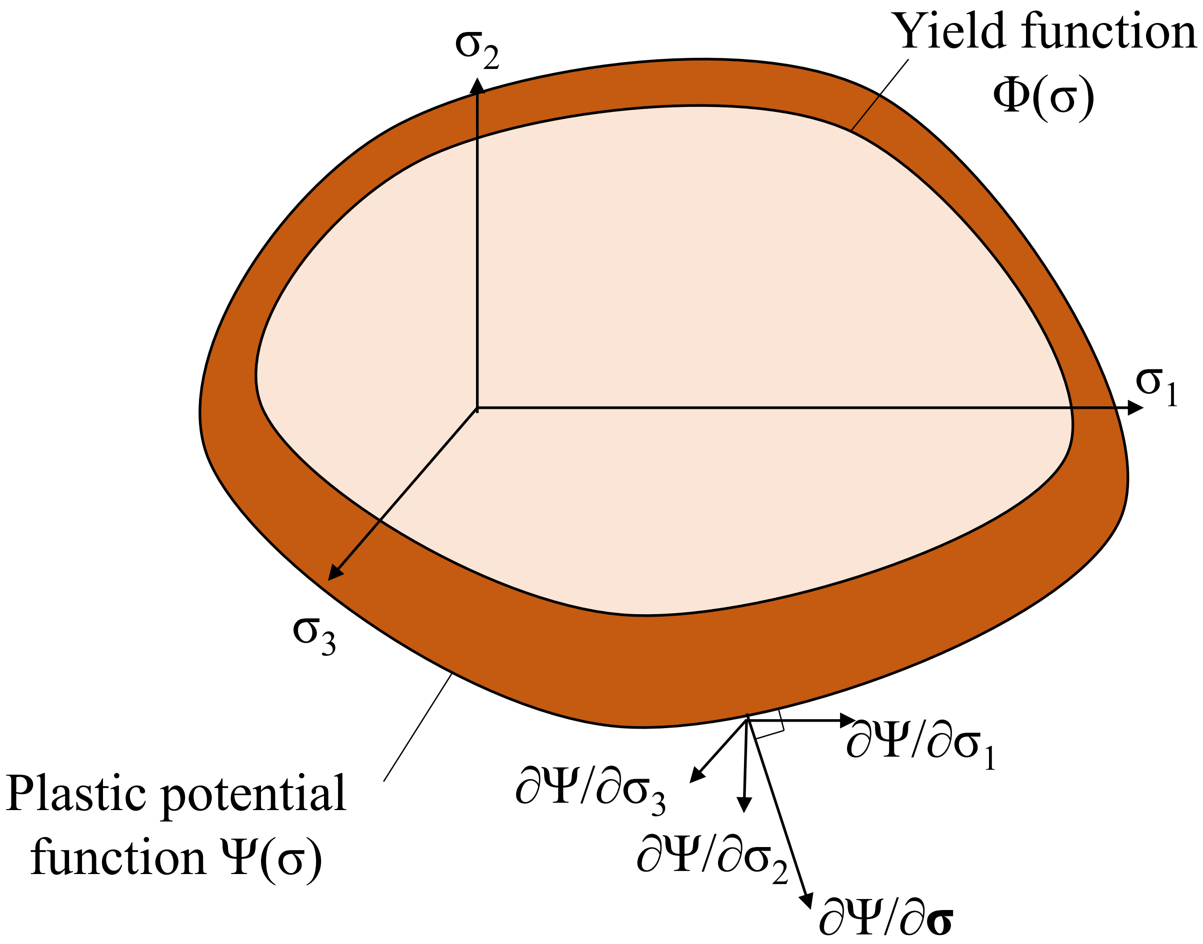
- separates purely elastic from elastoplastic behaviour.
- Function of stresses or stress invariants and hardening parameters



Soil Constitutive laws

Model Ingredients: (b) plastic potential function

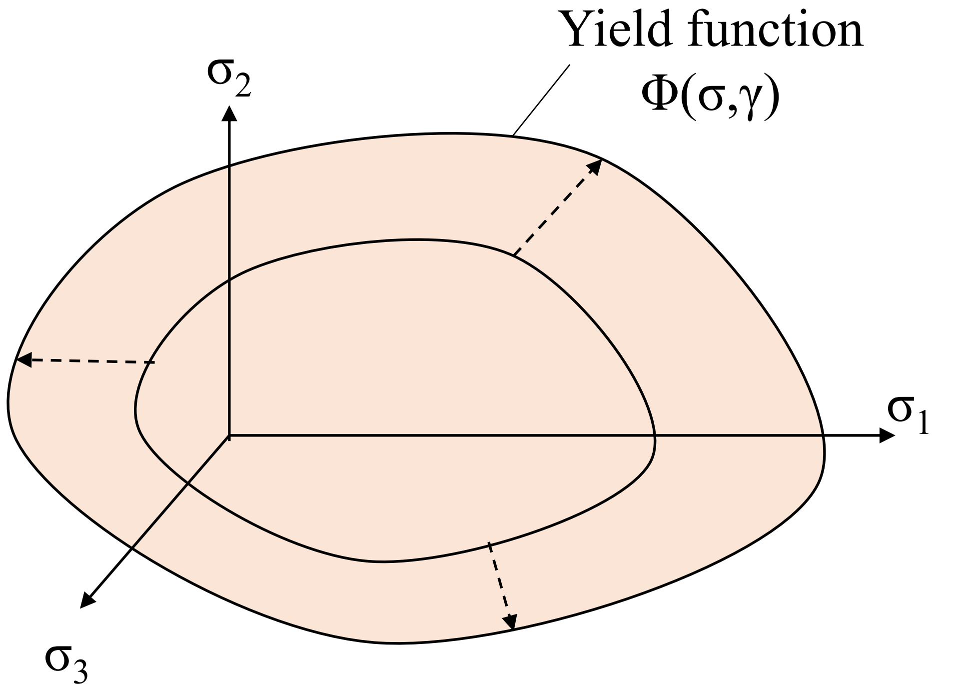
- specifying the direction of strains post yielding
- govern dilatancy effects



Soil Constitutive laws

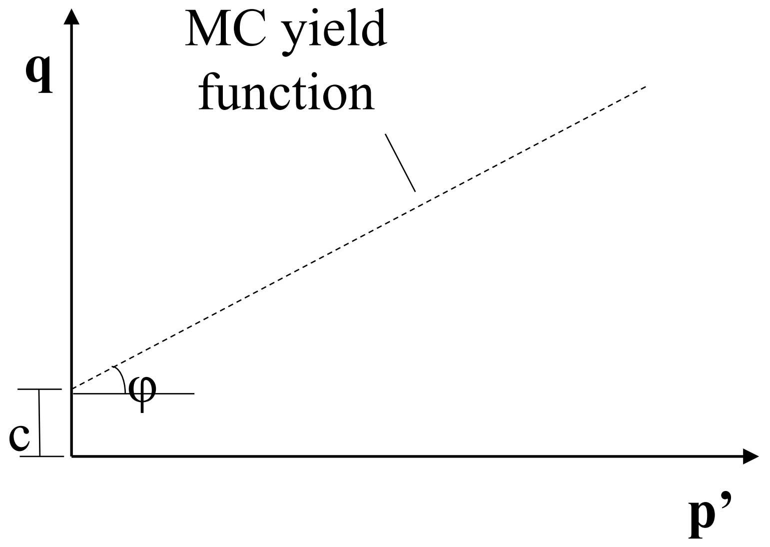
Model Ingredients: (c) Hardening/Softening rules

- Post failure hardening or softening behavior
- capture pre-failure inelastic behaviour



Soil Constitutive laws

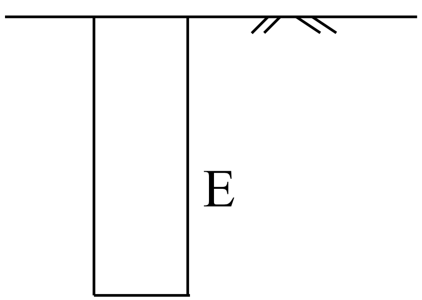
Mohr Coulomb:



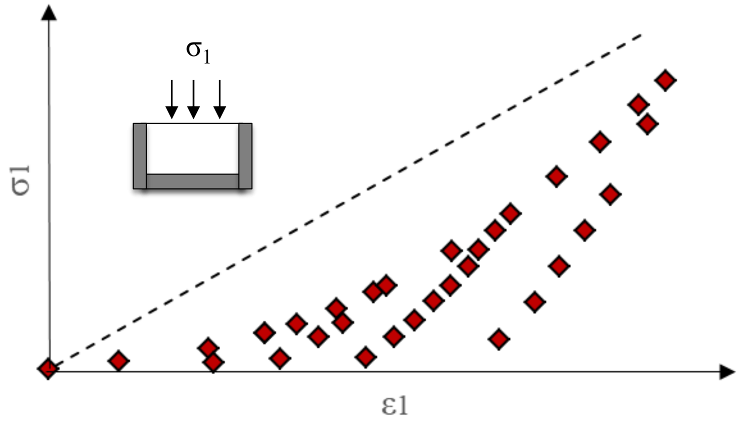
Elastic behavior:
E: young modulus
v: Poisson ratio

Post failure behavior:
φ: friction angle
c: cohesion
ψ: dilatancy angle

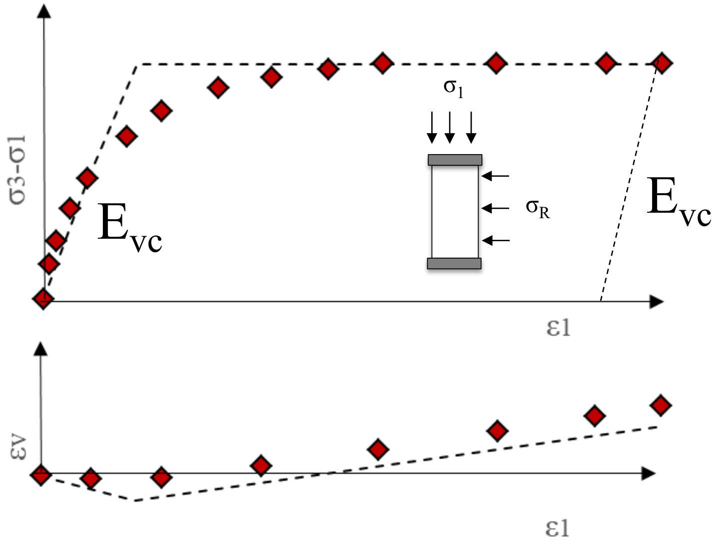
Mohr Coulomb:



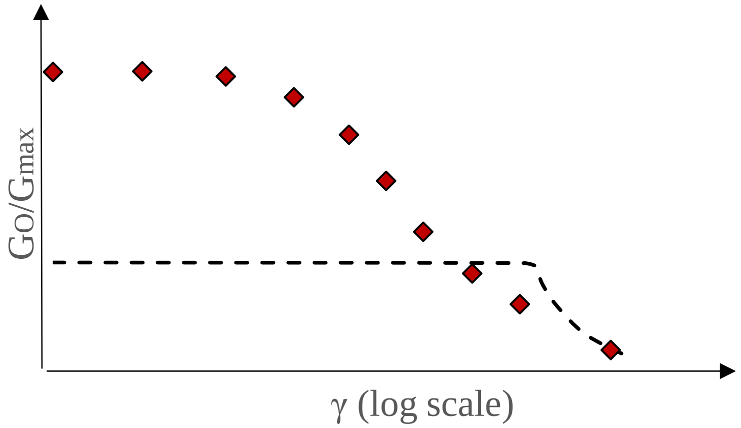
(a) Stress dependent elastic properties



(c) Oedometer test results



(b) Triaxial test results



(d) Small strain Stiffness

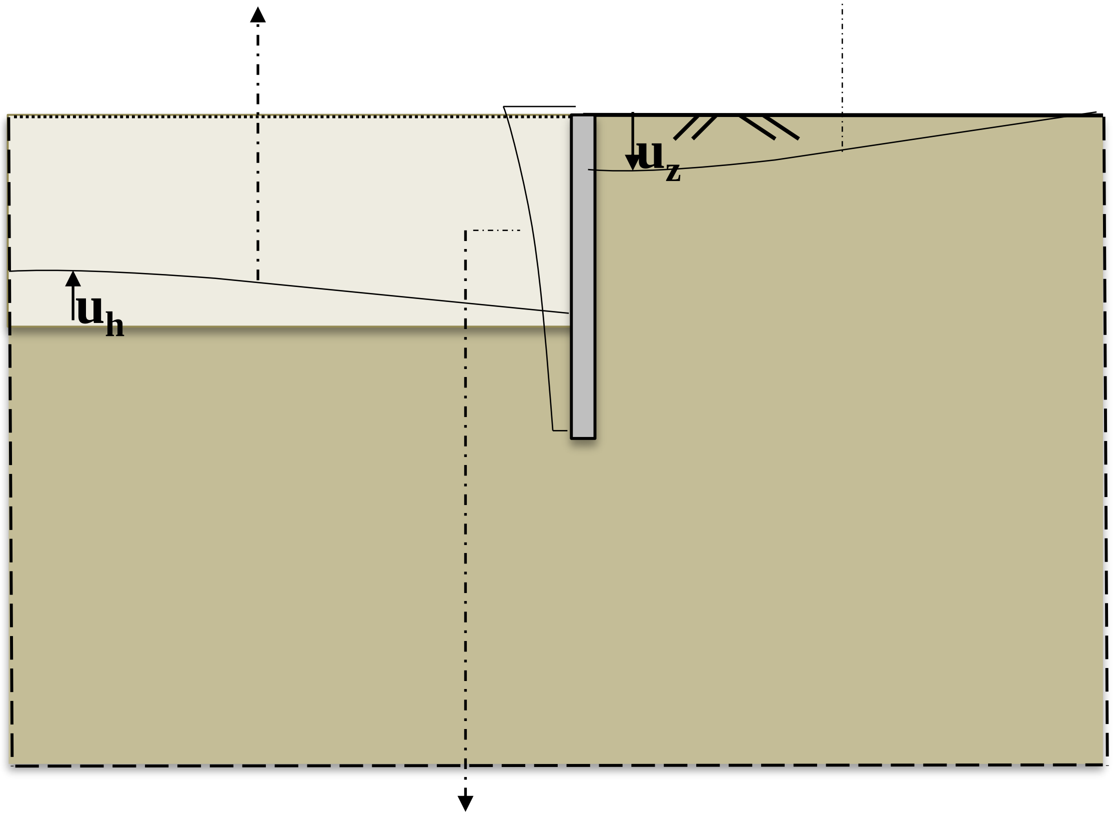
Used for preliminary estimate with 50% error in displacement.

Soil Constitutive laws

Dominating properties in excavations:

unrealistic bottom heave **[unloading/reloading stiffness]**

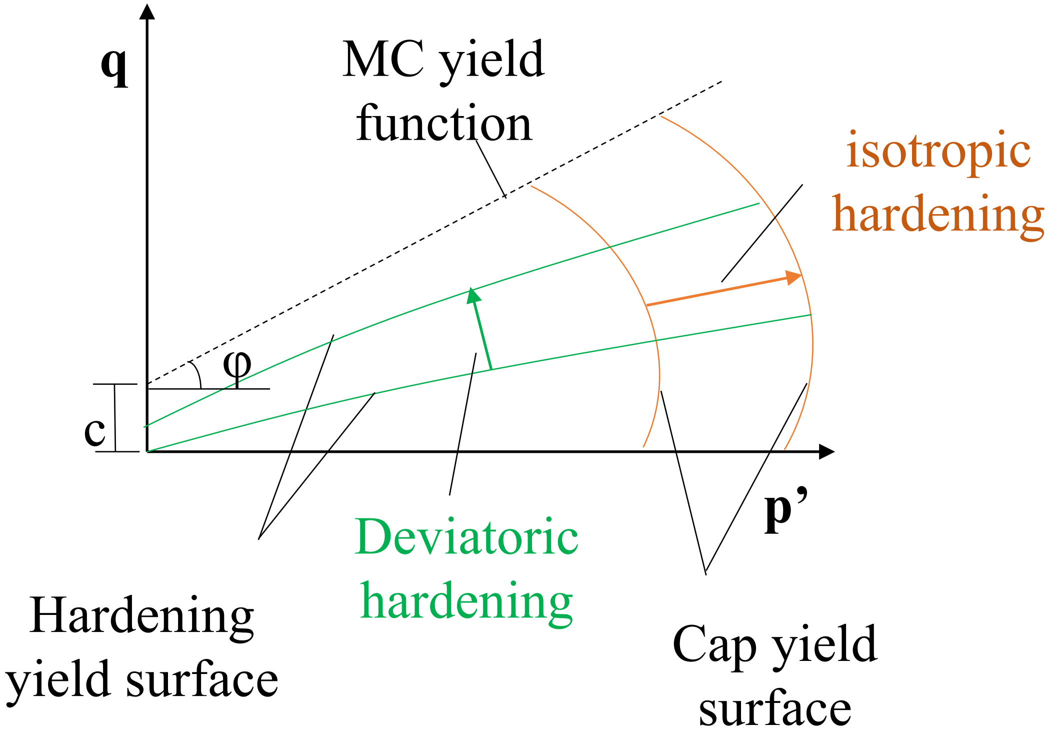
Unrealistic surface settlement [**pre yielding stiffness]**



Unrealistic wall displacement without [**small strain stiffness and stress dependency]**

Soil Constitutive laws

Soil hardening:



Stiffness parameters:

$\mathbf{E_{50}}$: secant stiffness in standard triaxial test

$\mathbf{E_{oed}}$: tangent stiffness for primary oedometer loading

$\mathbf{E_{ur}}$: unloading/reloading stiffness

$\mathbf{v_{ur}}$: Poisson ratio for unloading/reloading

$\mathbf{K_{0.nc}}$: K_0 value for normal consolidation

m: power of stress dependent stiffness

$\mathbf{P_{ref}}$: reference σ_1 pressure

ψ: dilatancy angle

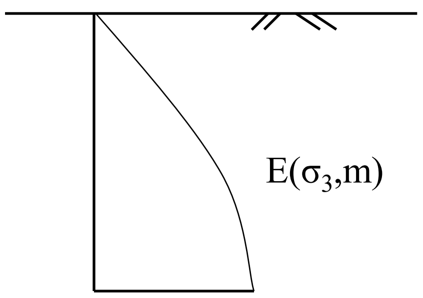
Strength parameters:

φ: friction angle

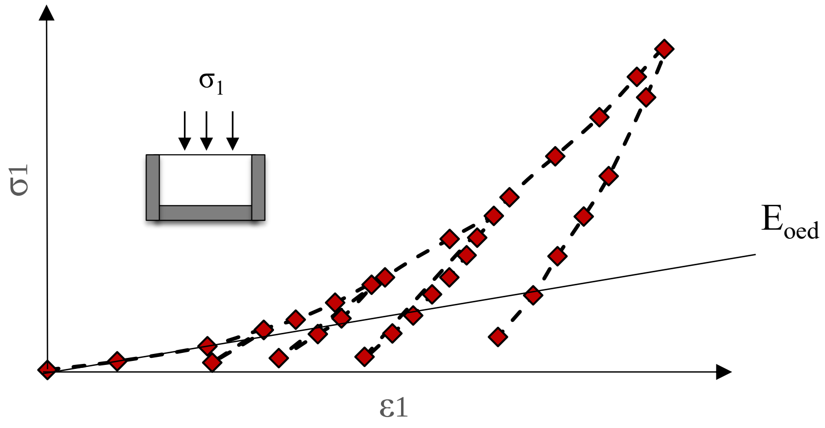
c: cohesion

Soil Constitutive laws

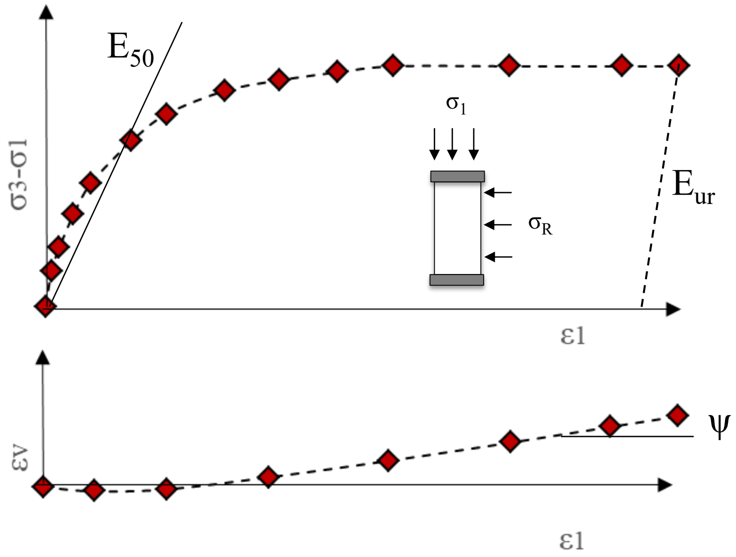
Soil hardening:



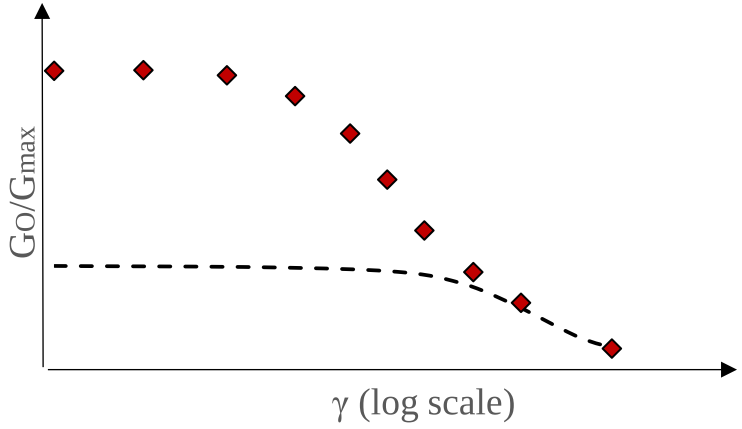
(a) Stress dependent elastic properties



(c) Oedometer test results

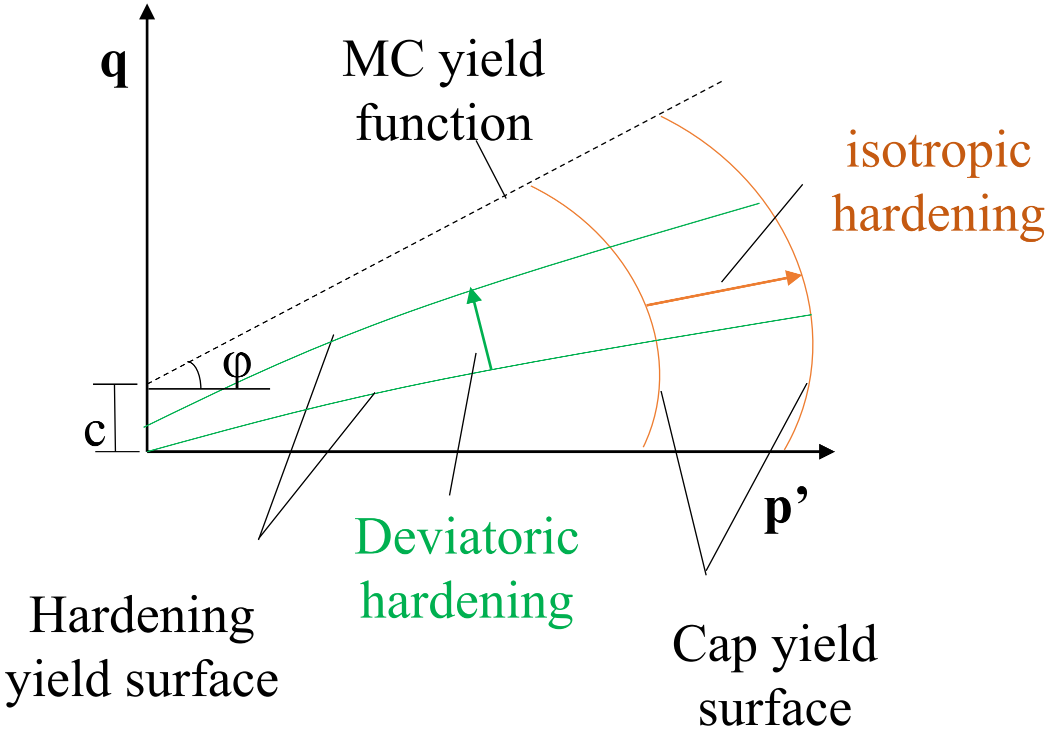


(b) Triaxial test results



(d) Triaxial test results

Small strain Soil hardening:



Stiffness parameters:

E_{50}: secant stiffness in standard triaxial test

E_{oed}: tangent stiffness for primary oedometer loading

E_{ur}: unloading/reloading stiffness

v_{ur}: Poisson ratio for unloading/reloading

$K_{0.nc}$: K_0 value for normal consolidation

m: power of stress dependent stiffness

P_{ref}: reference σ_1 pressure

ψ: dilatancy angle

Strength parameters:

φ: friction angle

c: cohesion

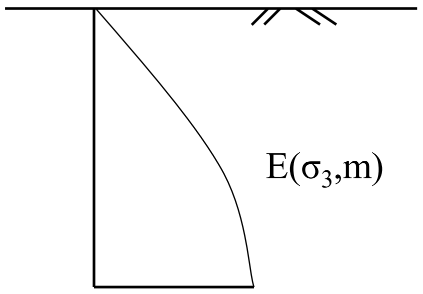
Very small strain stiffness parameters:

$E_{0,ref}$: initial young modulus

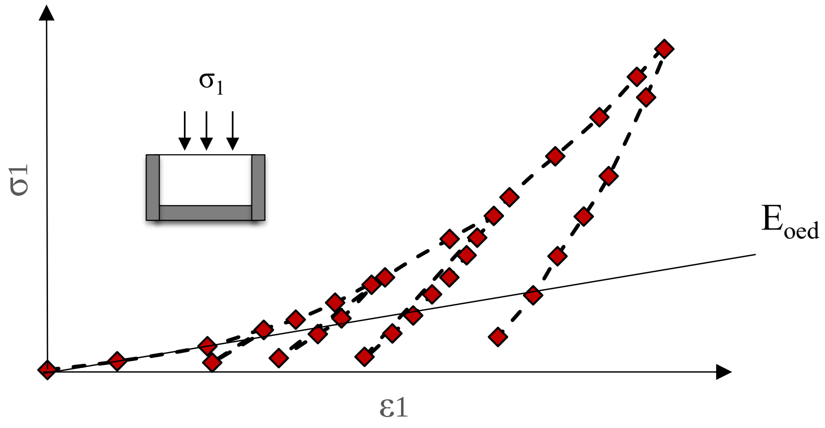
$\gamma_{0.7}$:shear strain for 70% shear modulus reduct.

Soil Constitutive laws

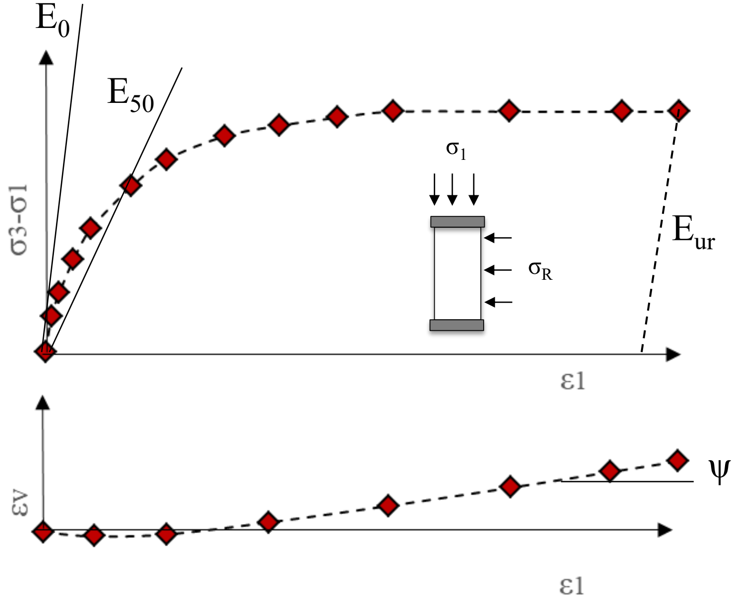
Small strain Soil hardening:



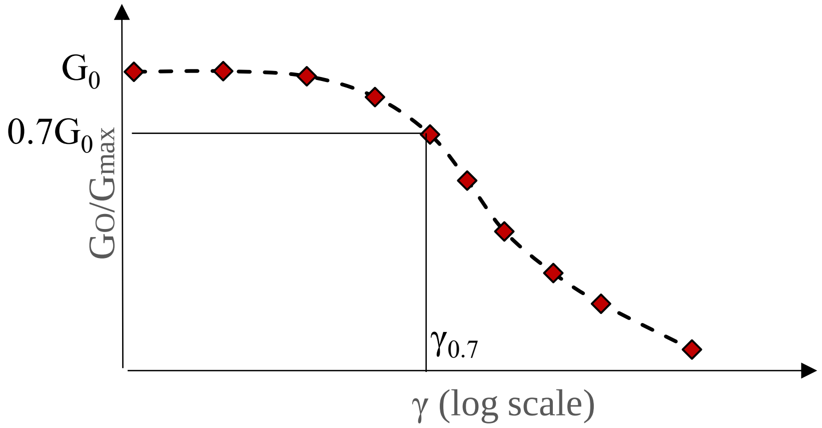
(a) Stress dependent elastic properties



(c) Oedometer test results



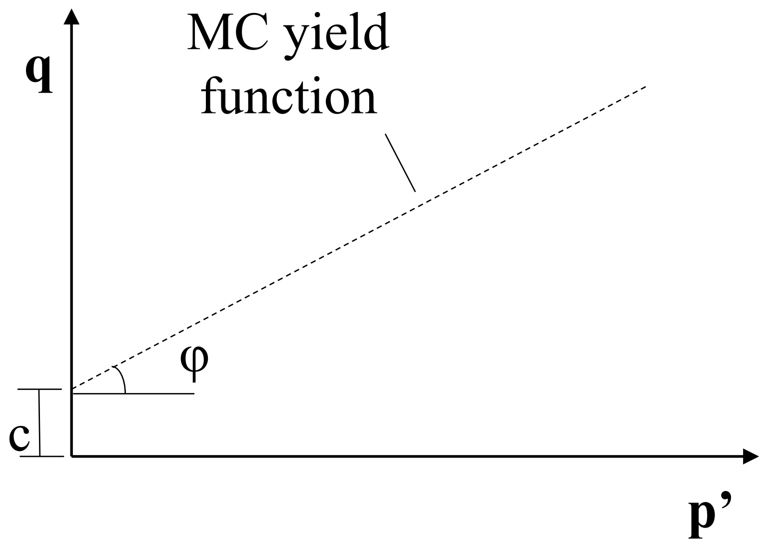
(b) Triaxial test results



(d) Triaxial test results

Soil Constitutive laws

Modified Mohr Coulomb:



$$(\sigma_1 - \sigma_3)/2 = (\sigma_1 + \sigma_3)/2 * \tan(\varphi) + c$$

Stiffness parameters:

E: young modulus

v: Poisson ration

ψ: dilatancy angle

m: power of stress dependent stiffness

P_{ref}: reference σ1 pressure

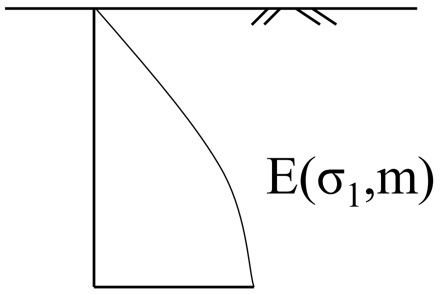
Strength parameters:

φ: friction angle

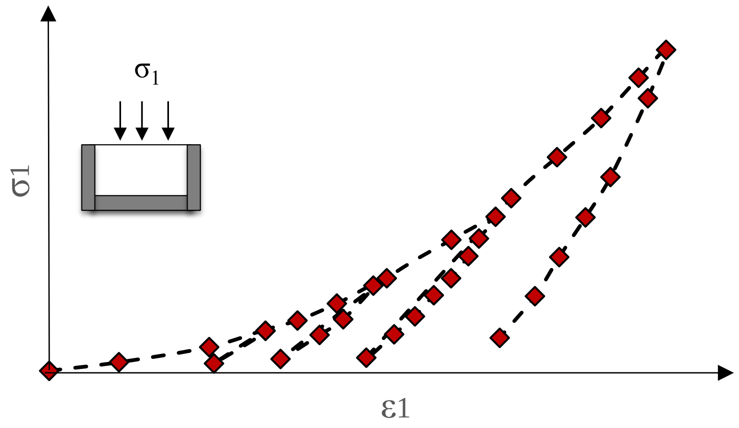
c: cohesion

Soil Constitutive laws

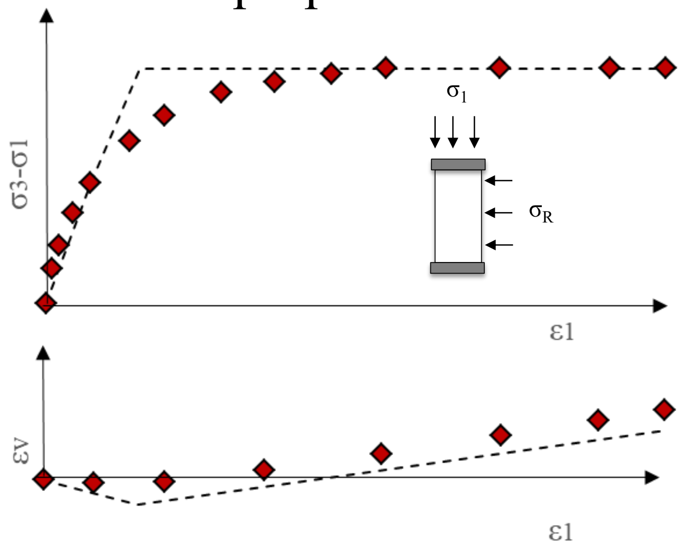
Modified Mohr Coulomb:



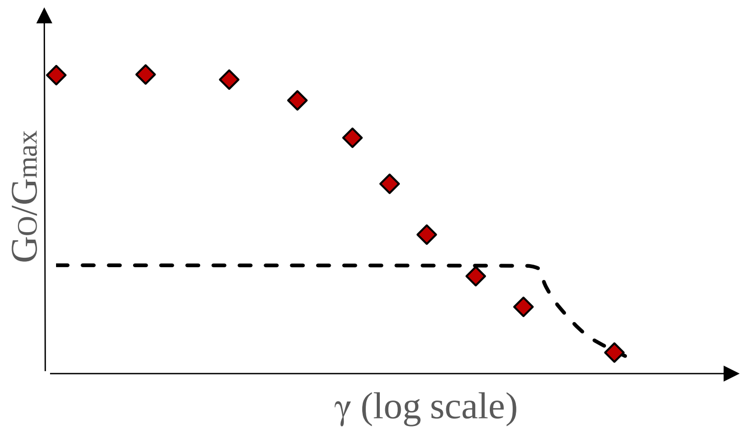
(a) Stress dependent elastic properties



(c) Oedometer test results

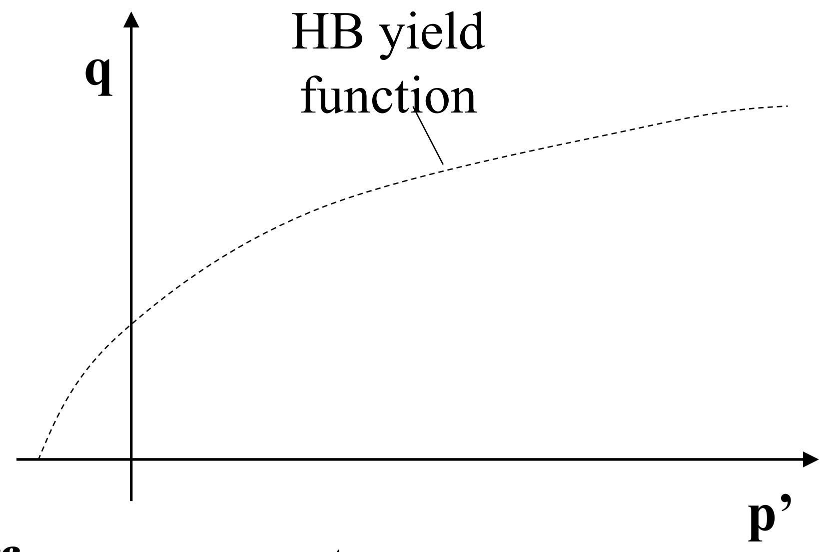


(b) Triaxial test results



(d) Small strain Stiffness

Hoek Brown:



Stiffness parameters:
E: young modulus
v: Poisson ration
ψ: dilatancy angle

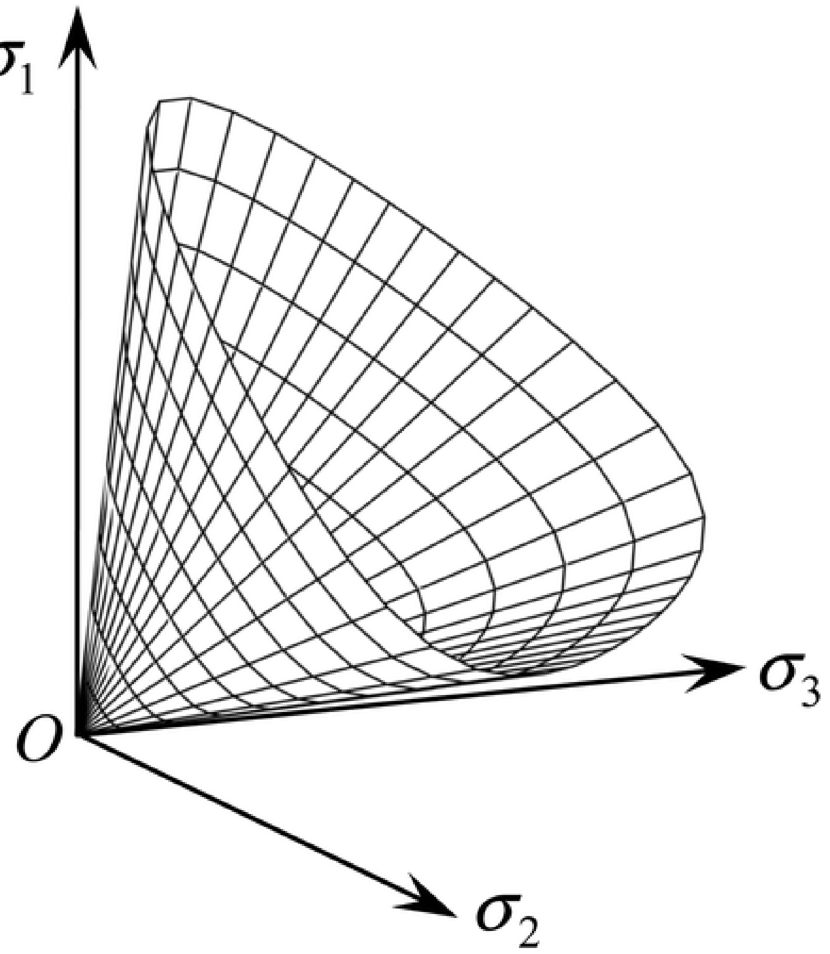
Strength parameters:
GSI:geological strength index
D:disturbance factor
σ_{ci}: compressive strength of intact rock

Appearance of rock mass	Description of rock mass	Suggested value of *D*
	Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	*D* = 0
	Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.	*D* = 0 *D* = 0.5 No invert
	Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass.	*D* = 0.8
	Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.	*D* = 0.7 Good blasting *D* = 1.0 Poor blasting
	Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal. In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less.	*D* = 1.0 Production blasting *D* = 0.7 Mechanical excavation

(Hoek et al 2002)

Soil Constitutive laws

The selection and parametrization of a constitutive law is **highly** influential but also **highly** complex!

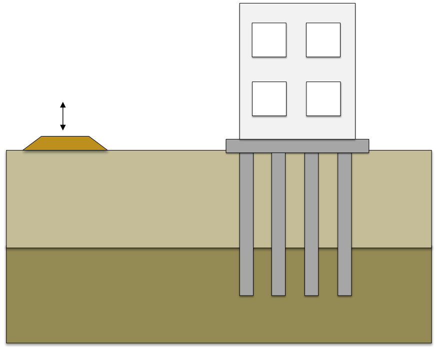


- Limited mathematical models
- High uncertainty in input data

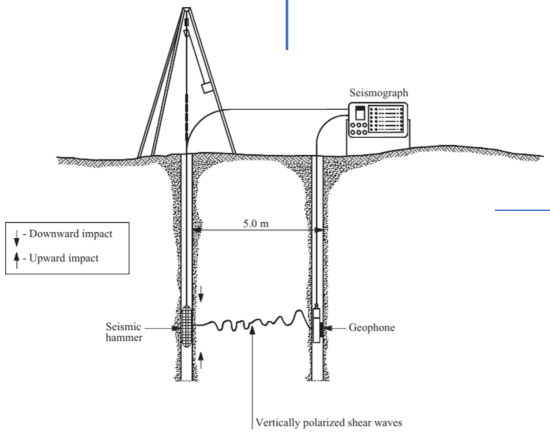
Soil Constitutive laws

Selection of constitutive law?

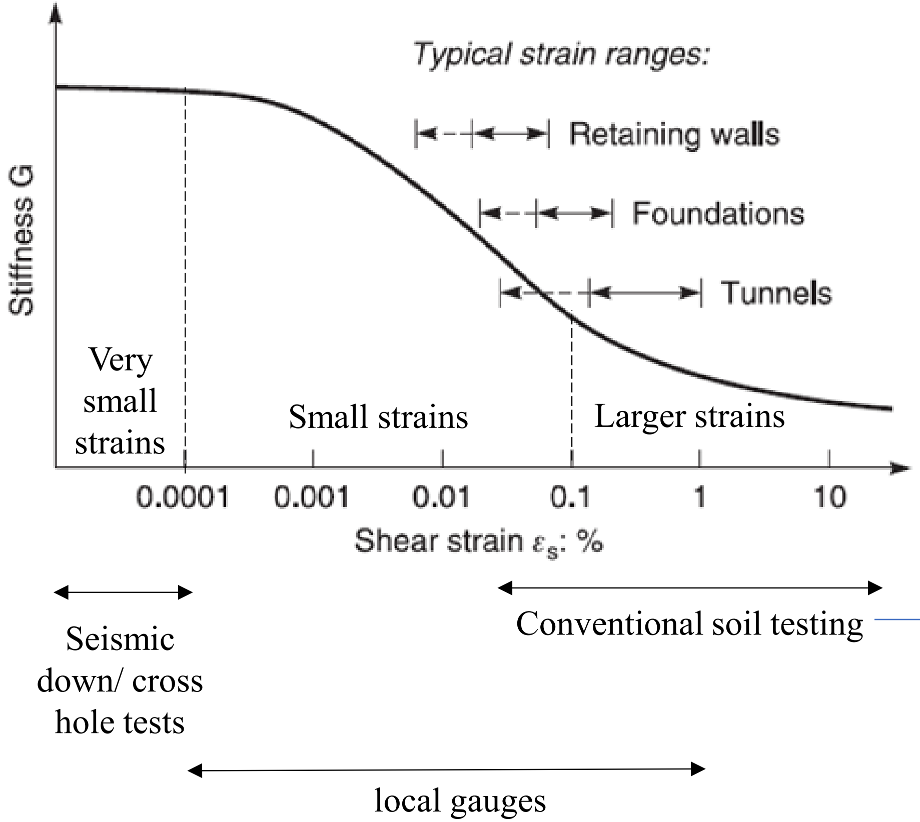
1) Level of strain

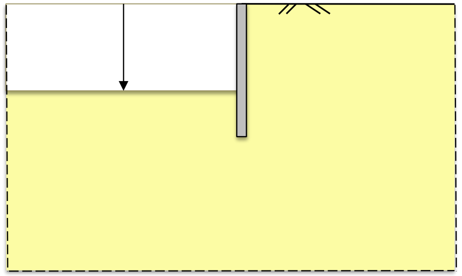


Railway vibration



Cross hole test

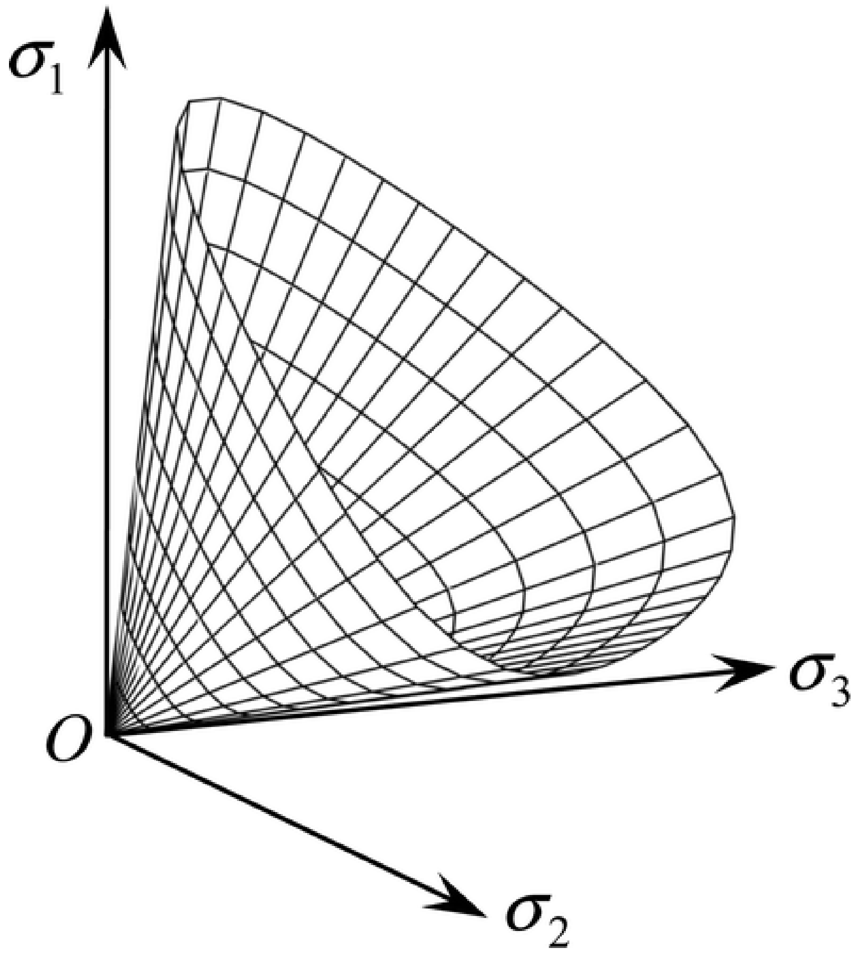




Excavation

Soil Constitutive laws

The selection and parametrization of a constitutive law is **highly** influential but also **highly** complex!

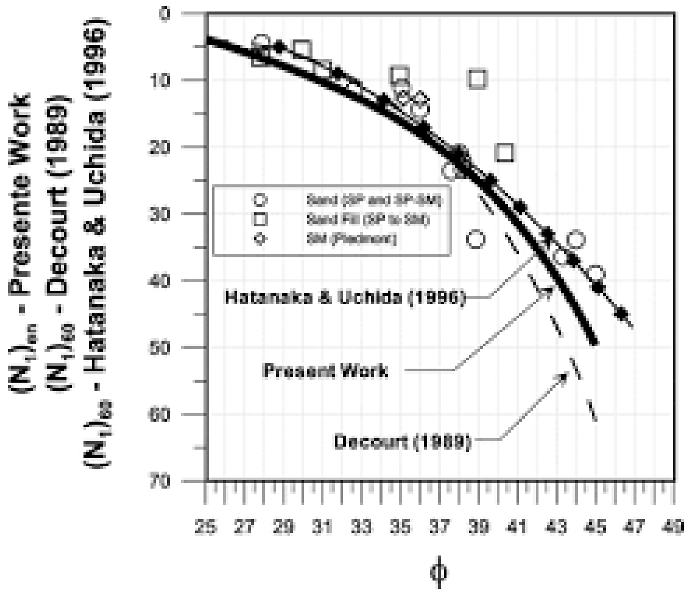


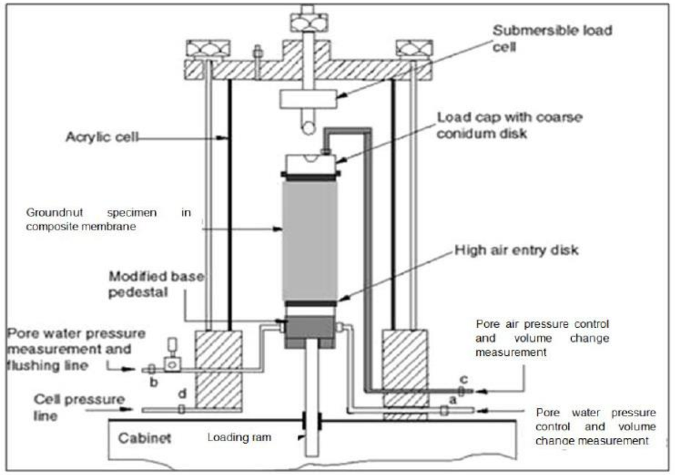
- Limited mathematical models
- High uncertainty in input data

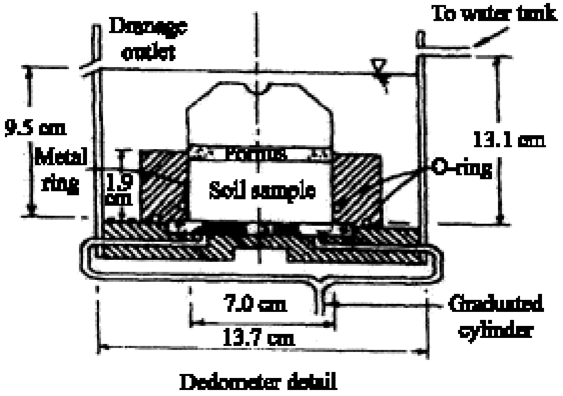
Soil Constitutive laws

Selection of soil parameters

corelation with SPT/CPT/ and
soil classification







Oedometer test

Directly from lab tests
(triaxial test, oedometer test,
direct shear test)

Local experience in the site from previous projects!

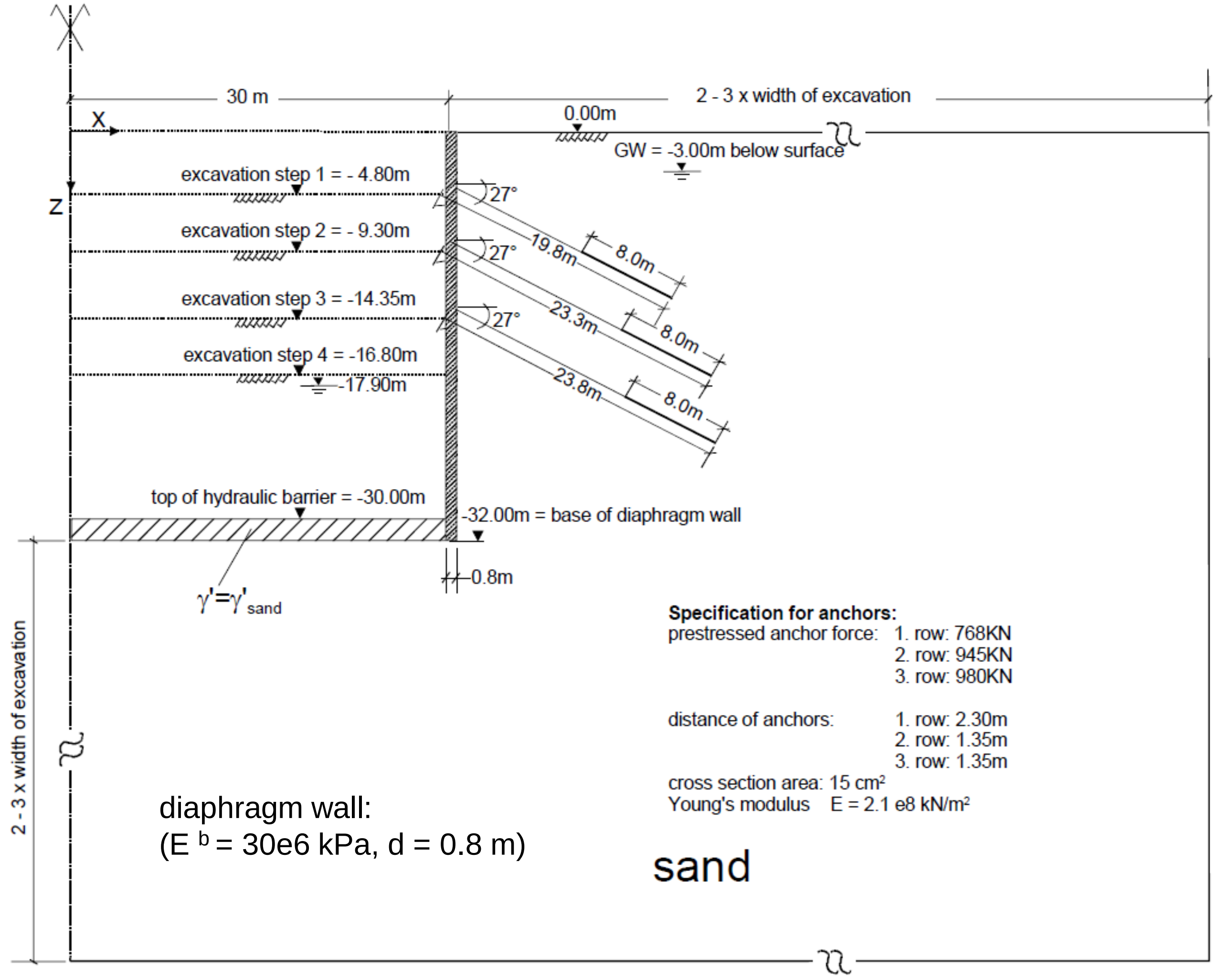
Soil Constitutive laws

Example 2: Anchored excavation in Berlin sand.

Soil Constitutive laws

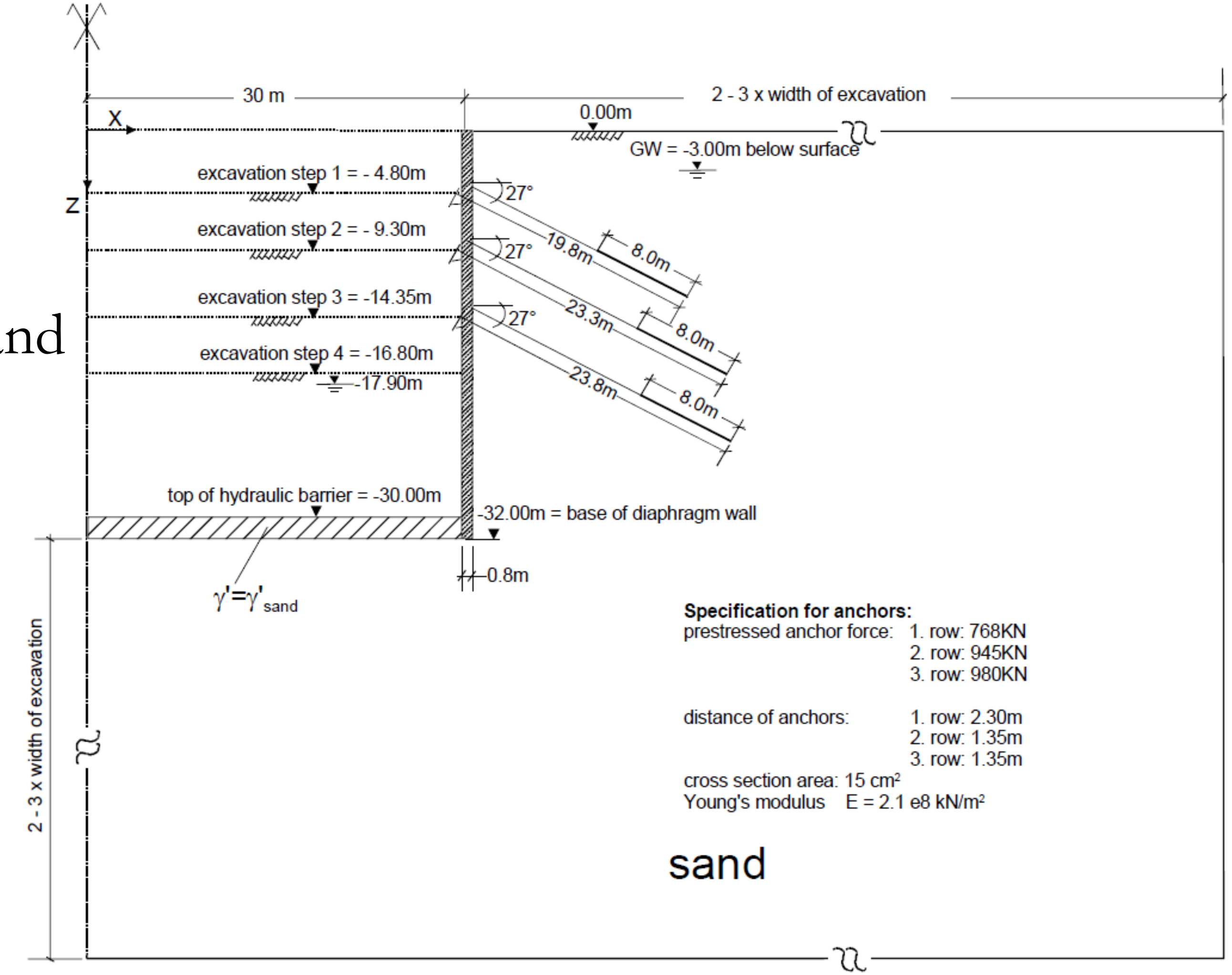
- Selected an existing excavation in berlin (with monitoring results and detailed soil investigation)
- Sent to various university institutes and consulting companies known to deal with Excavation numerical analysis.

Example with prescribed properties as published By Schweiger.



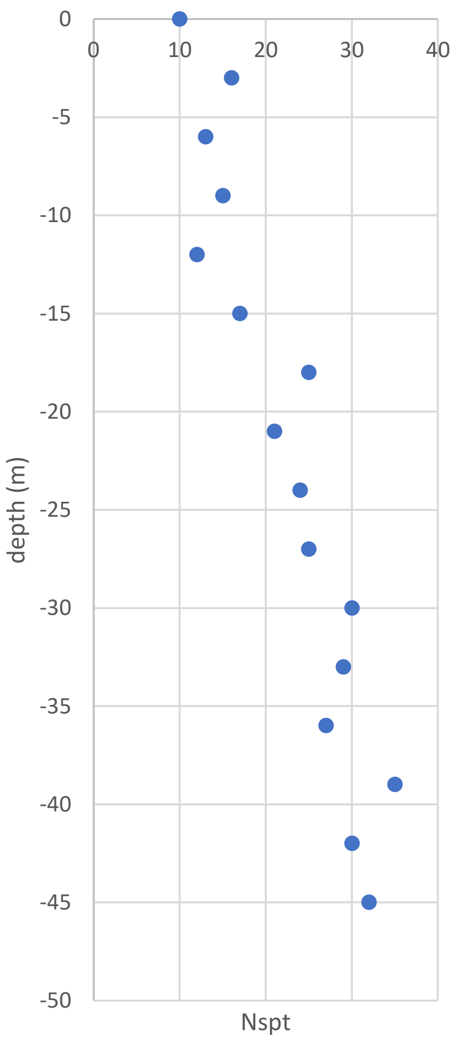
Soil Constitutive laws

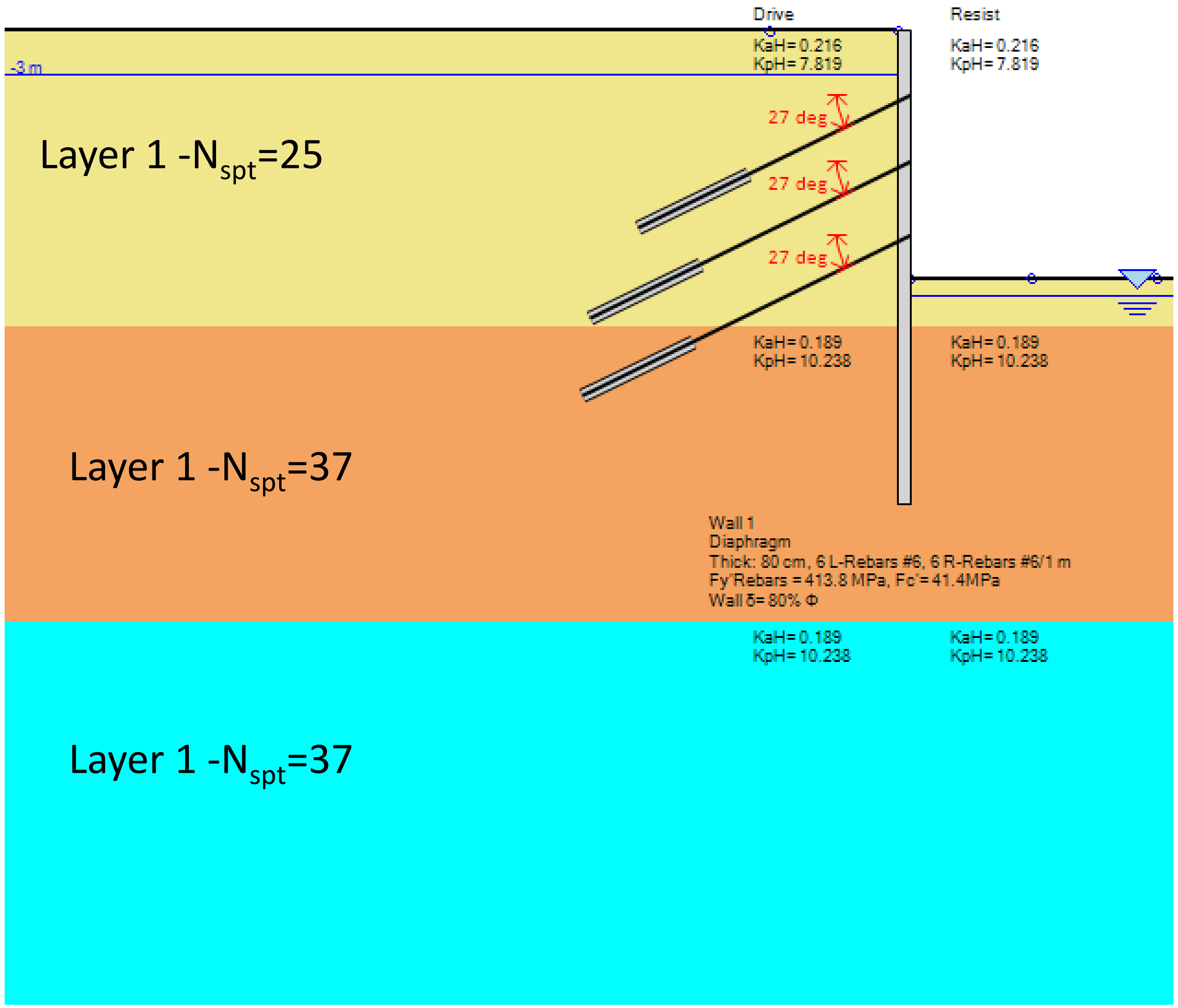
Stage GF: Greenfield conditions (Ko based imposed stress field)
Stage 0: activation of diaphragm wall
Stage 1: excavation at level -4.8m
Stage 2: activation of anchor 1 and prestressing and groundwater lowering to -9.4m
Stage 3: excavation at level -9.3m
Stage 4: activation of anchor 2 and prestressing
Stage 5: groundwater lowering to -14.5m
Stage 6: excavation at level -14.35m
Stage 7: activation of anchor 3 and prestressing
Stage 8: groundwater lowering to -17.9m
Stage 9: final excavation at level -16.8m



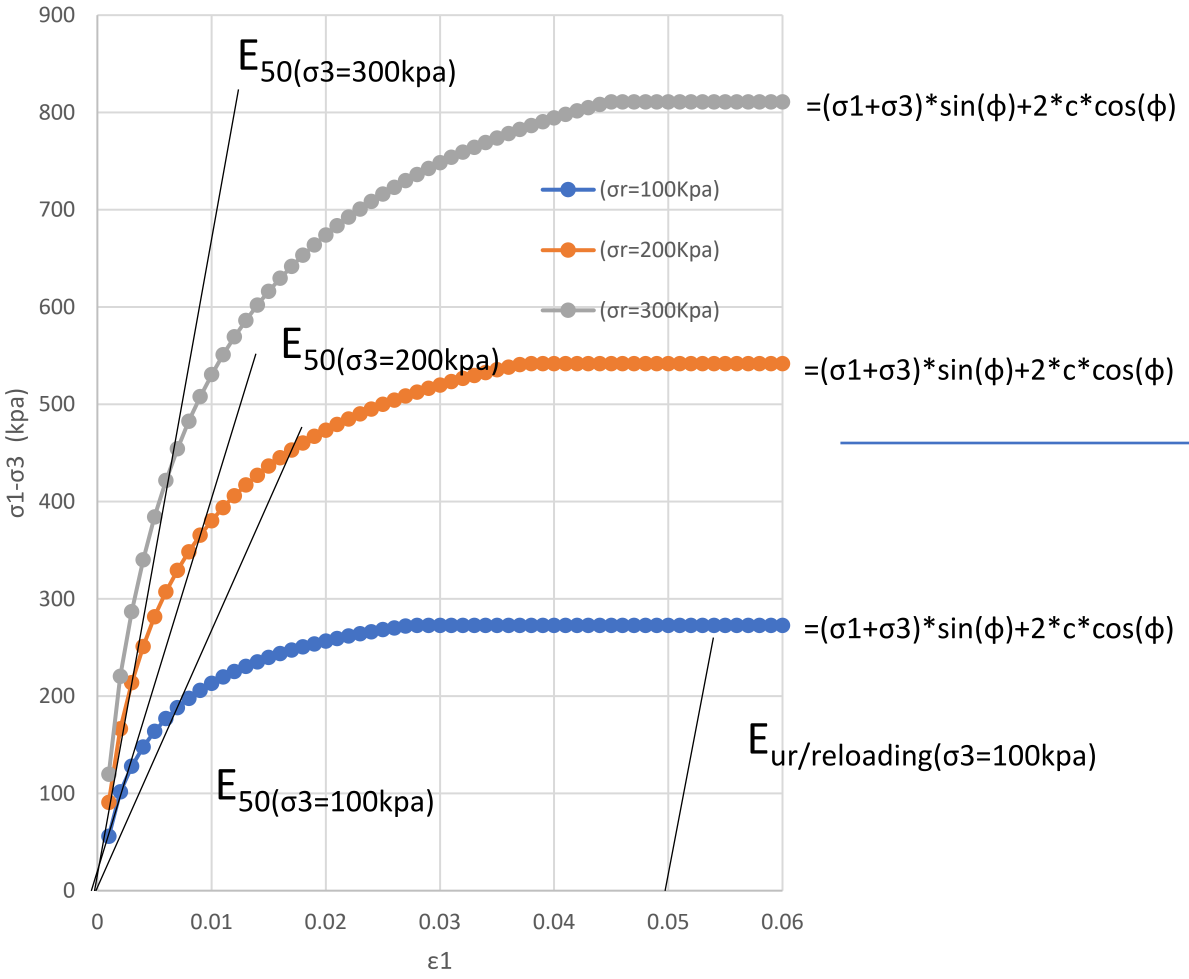
Soil Constitutive laws

Standard penetration test results:



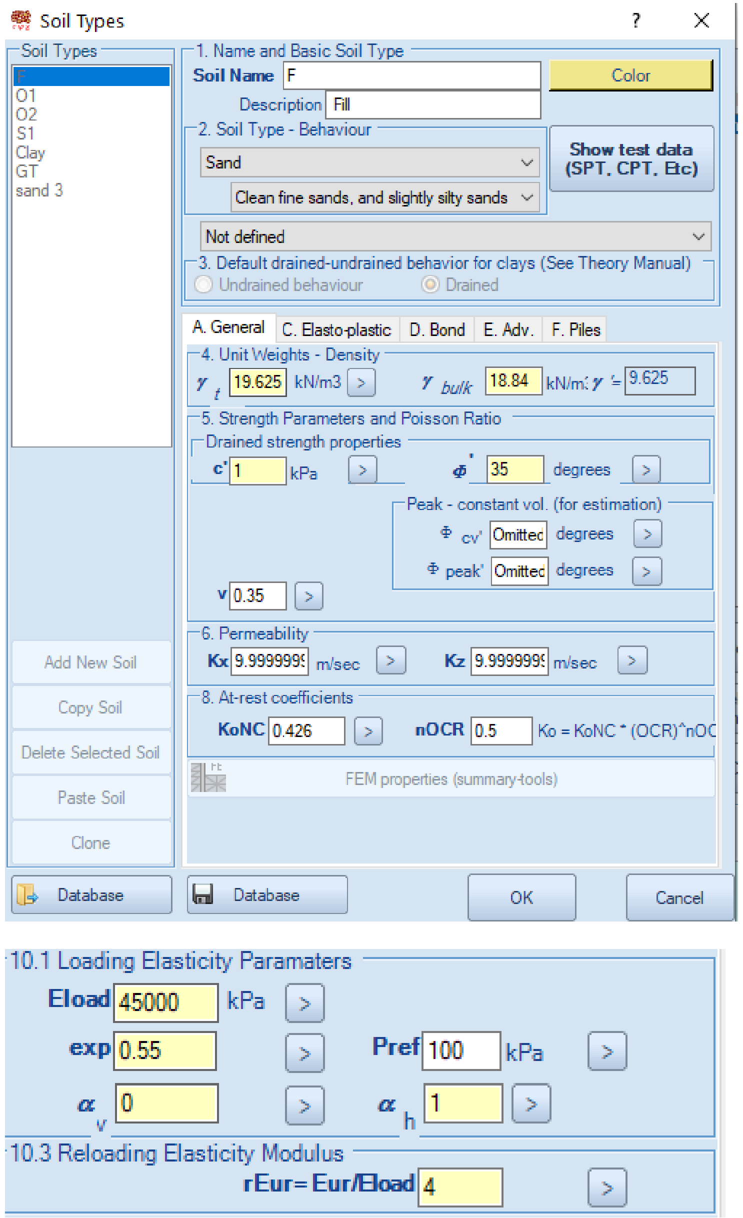


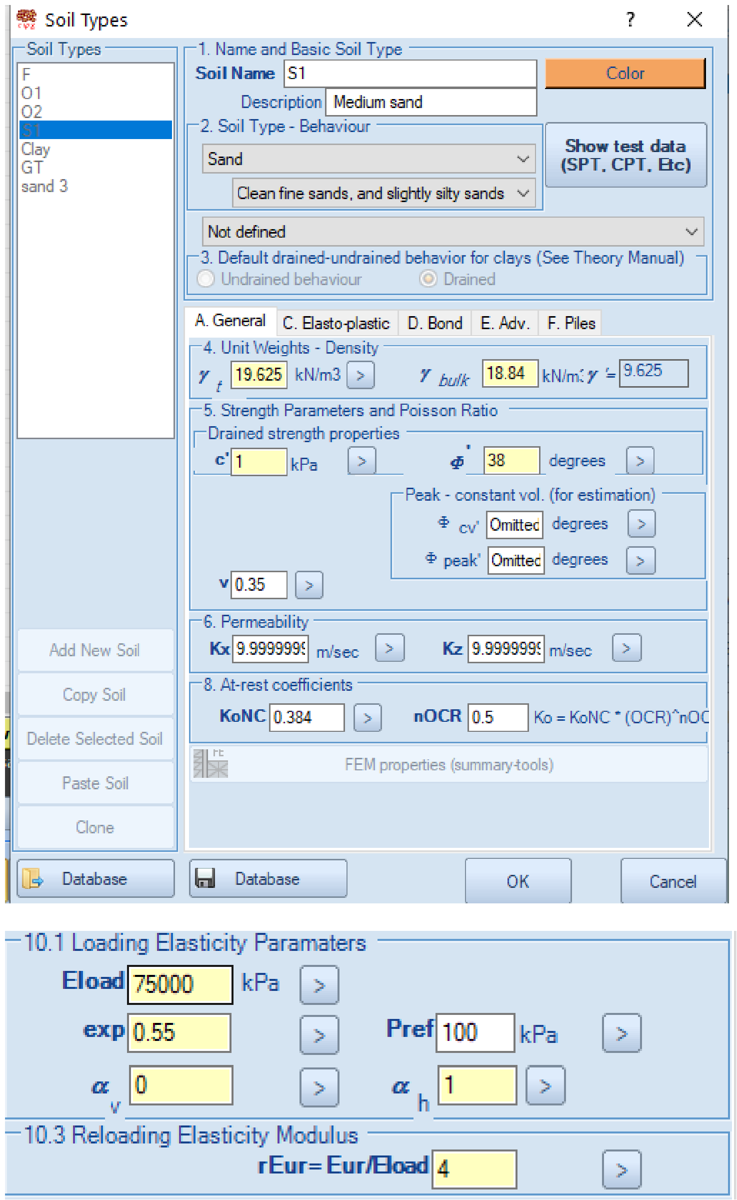
Case Studies : anchored wall - Berlin

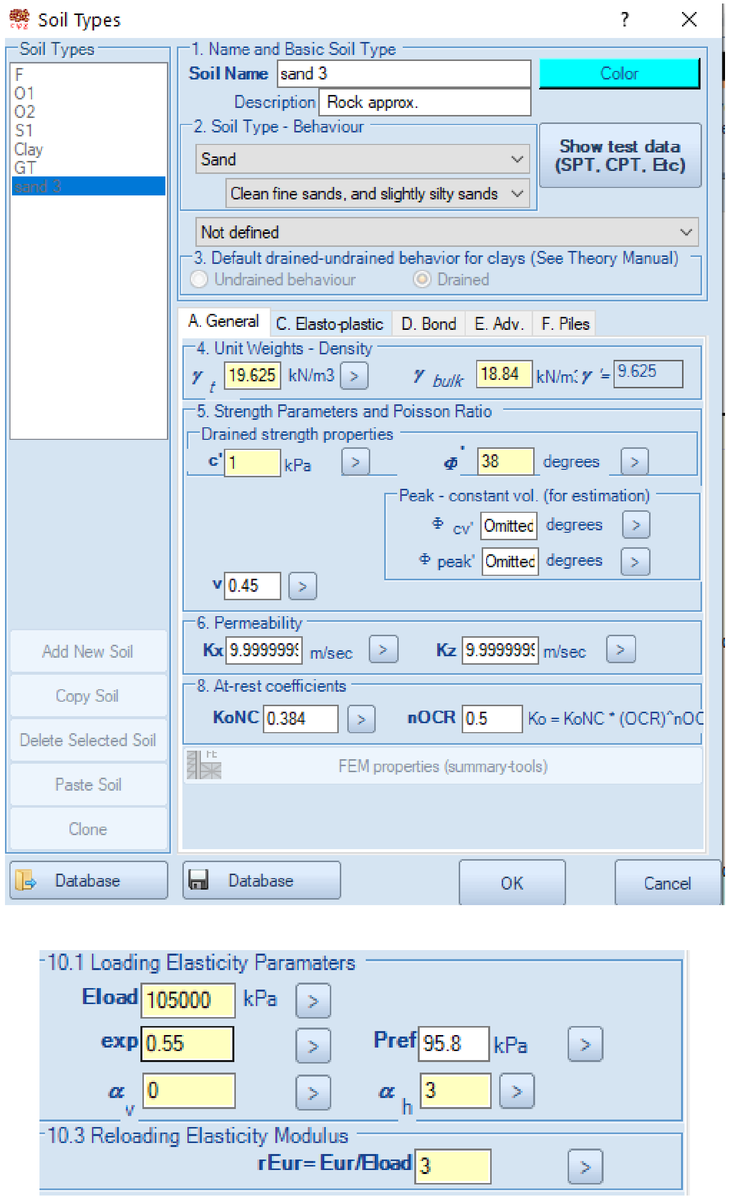


depth of layer	E_{50}^{ref}	E_{ur}^{ref}	E_{oed}^{ref}	ν	P	c	$<_{ur}$	p_{ref}	m	R_f	R_{inter}
m	kPa	kPa	kPa	°	°	kPa	-	kPa	-	-	-
0 - 20	45 000	180 000	45 000	35	5	1.0	0.2	100	0.55	0.9	0.8
20 - 40	75 000	300 000	75 000	38	6	1.0	0.2	100	0.55	0.9	0.8
> 40	105 000	315 000	105 000	38	6	1.0	0.2	100	0.55	0.9	-

Soil Constitutive laws



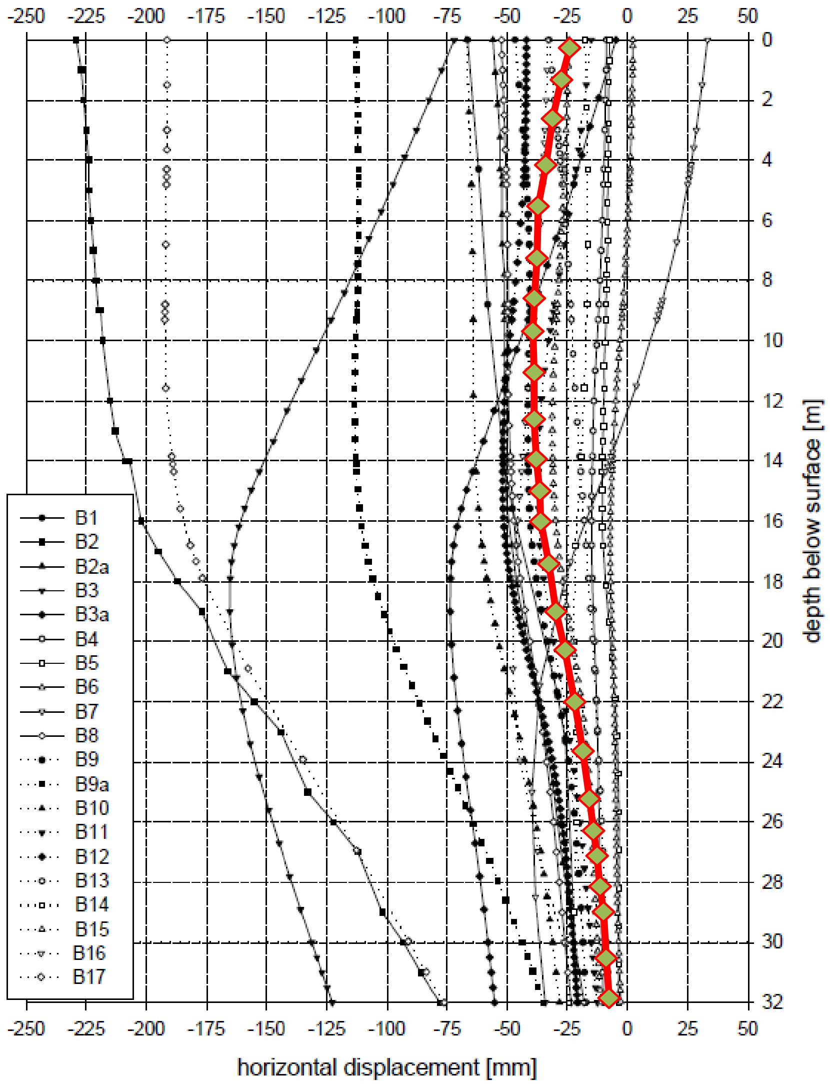




Soil Constitutive laws

- Most participants trusted the strength indicated from the experiments
- stiffness generated from the tests was considered too low by many participants
- significant variation was observed however in the assumption of the dilatancy angle ψ, ranging from 0° to 15°.

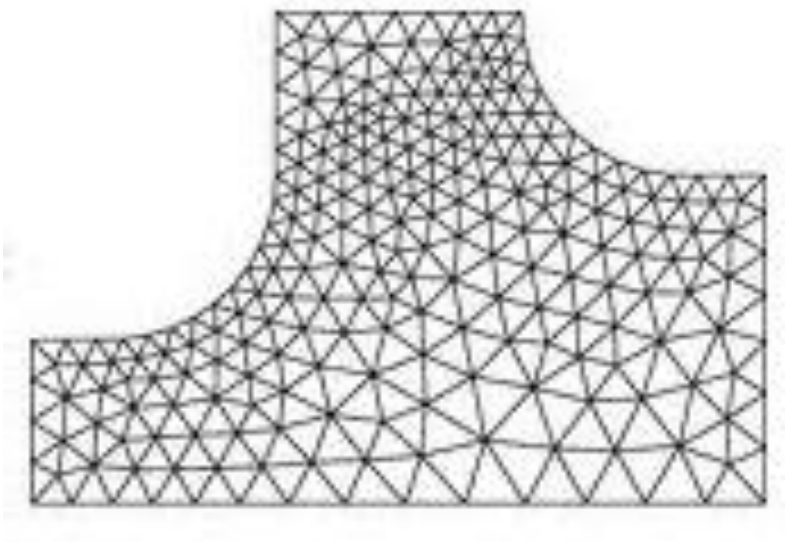
Soil Constitutive laws



- Some of the participants used an incorrect stiffness due to misguiding lab tests
- some participants incorrectly calculated the prestress force
- Mohr coulomb based models
- Fixed anchors
- the difference in modelling the groundwater lowering (0.5-1.5cm)

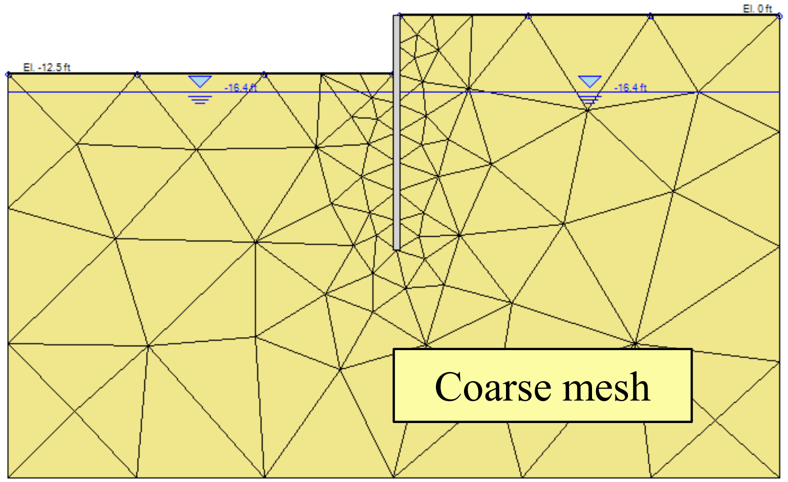
Major issues where caused by the selection of the soil properties based on lab tests (participants based on literature and local experience had better results).

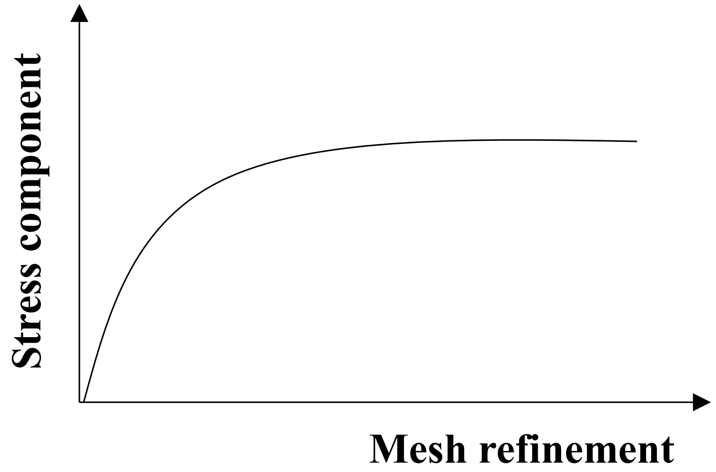
Good practice in FEM model construction

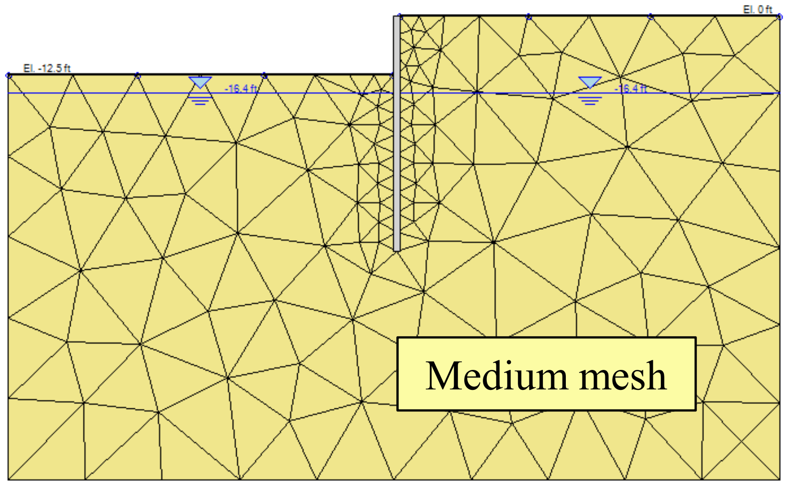


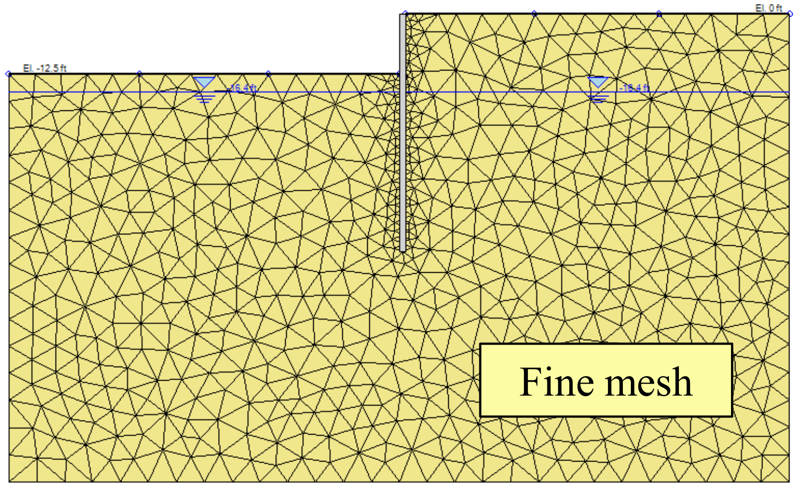
Meshing

Meshing and boundaries



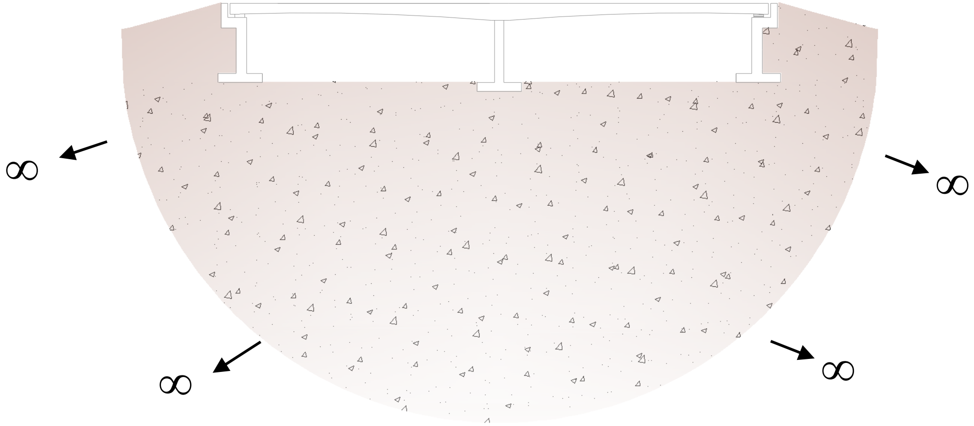






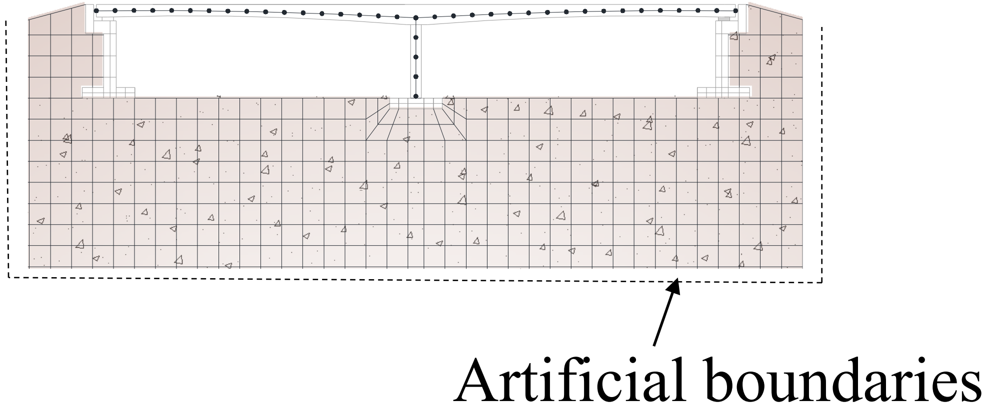
Meshing and boundaries

Boundary conditions



Reality: semi-infinite soil medium

FEM model: truncated boundaries

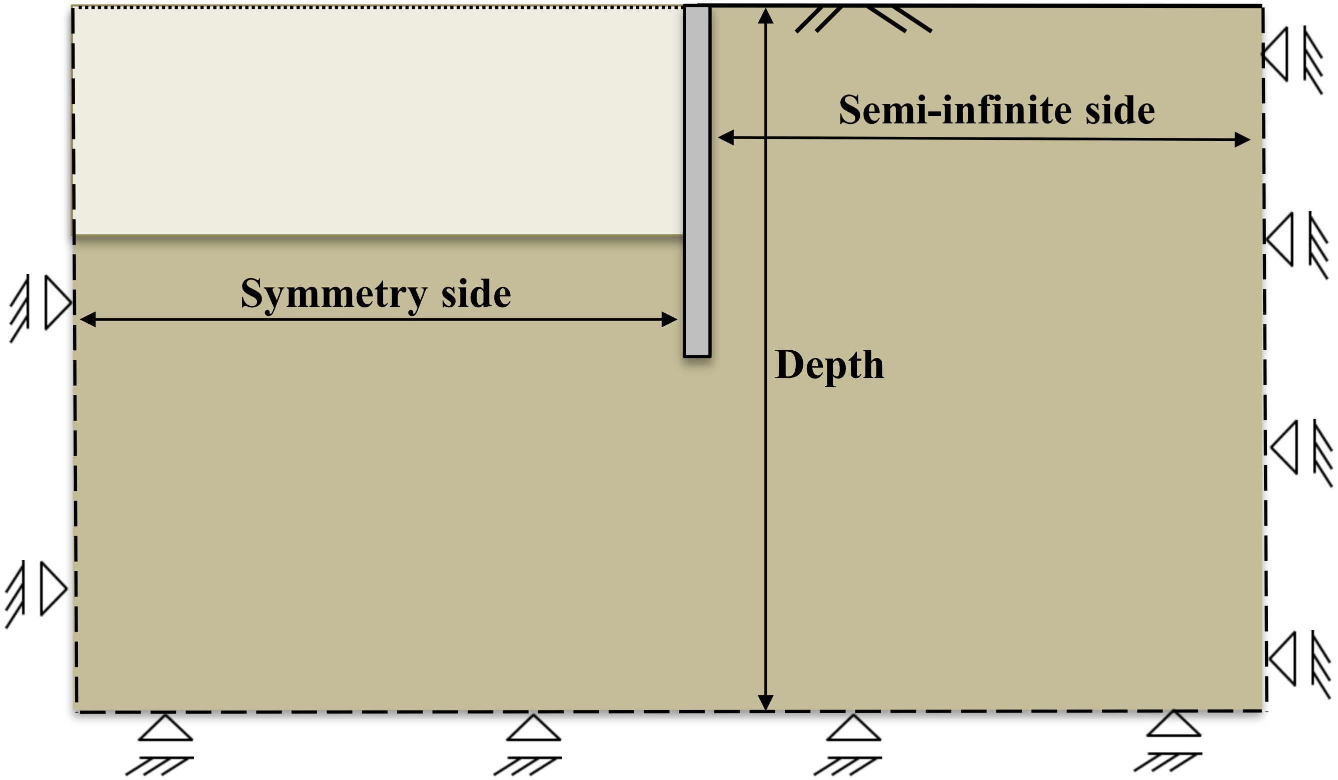


Meshing and boundaries

Soil medium boundaries

Semi-infinite side: based on surface settlement accuracy (1.5 to 3 times the excavation width)

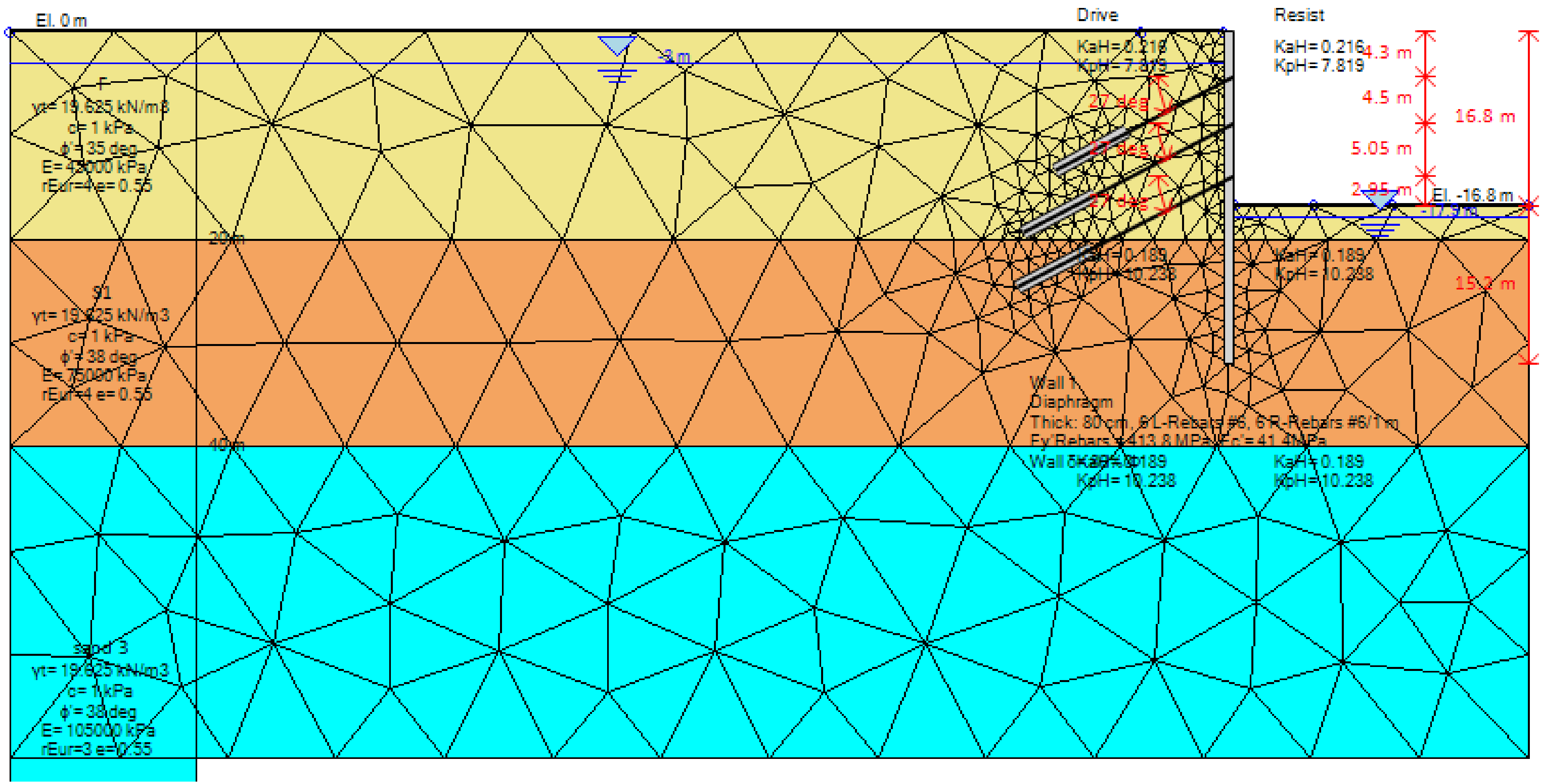
symmetry side : half distance



Depth : a stiff and strong layer is a good location to cut your model

Meshing and boundaries

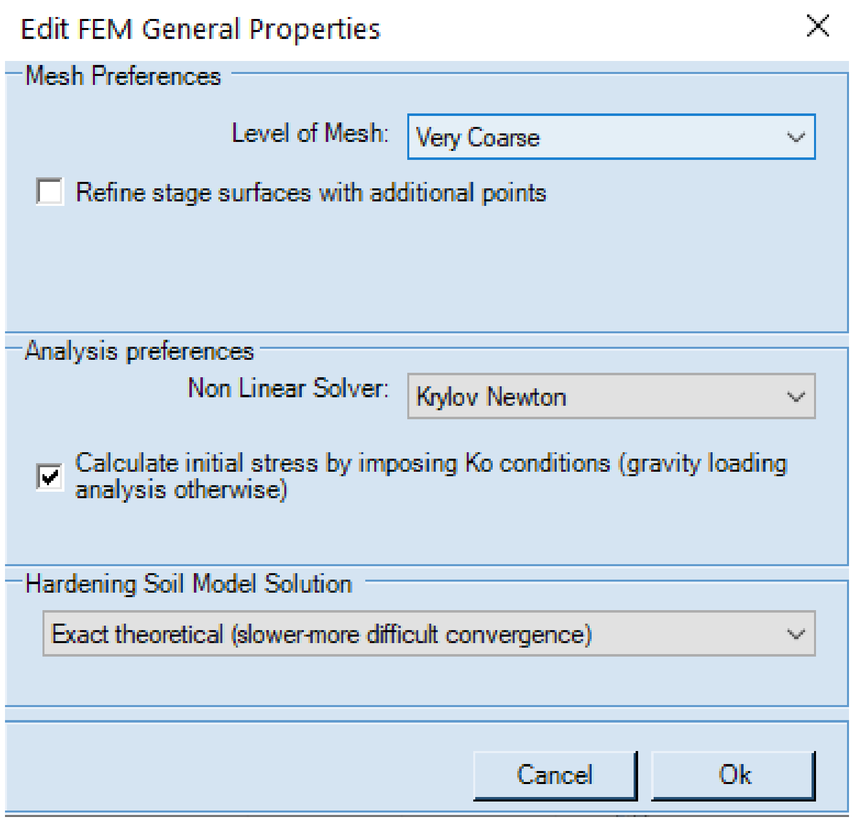
Example 3: Boundary dimensions



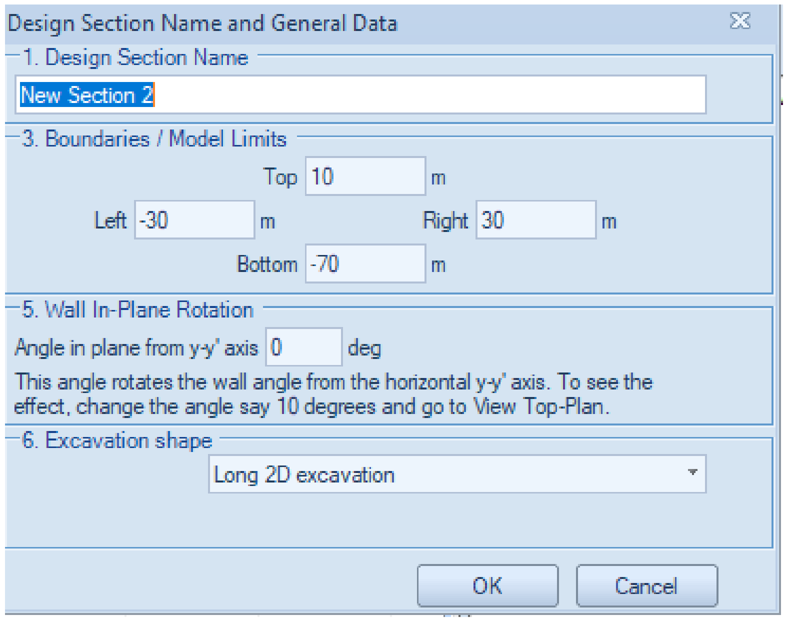
Meshing and boundaries

Example 3: Boundary dimensions

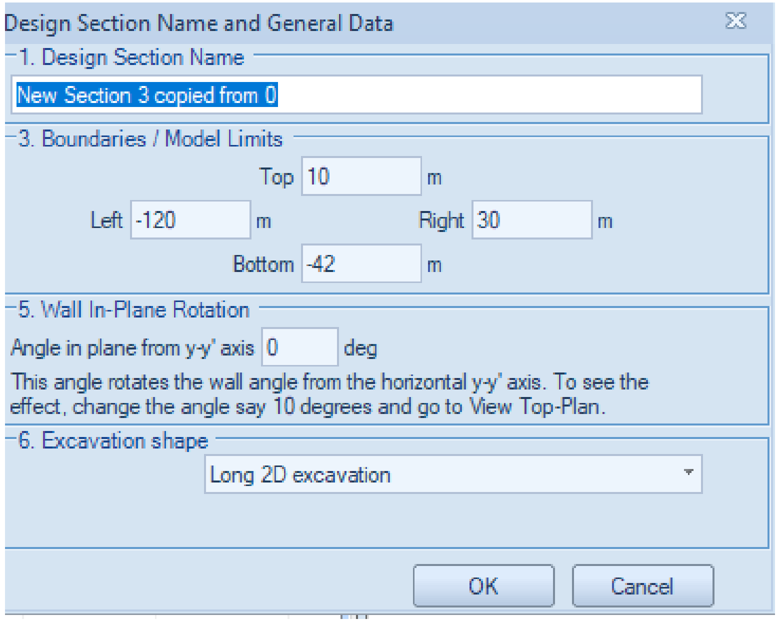
Change in mesh



Change in semi-infinite side

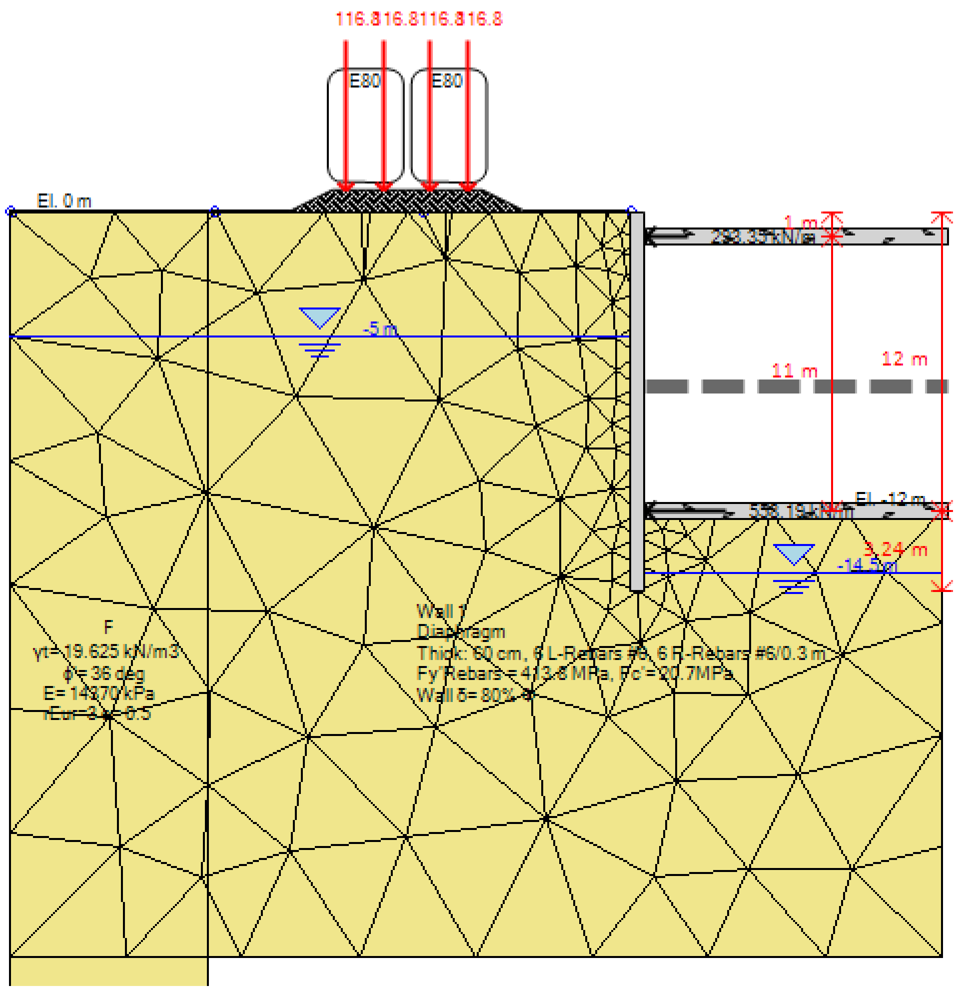


Change in bottom depth



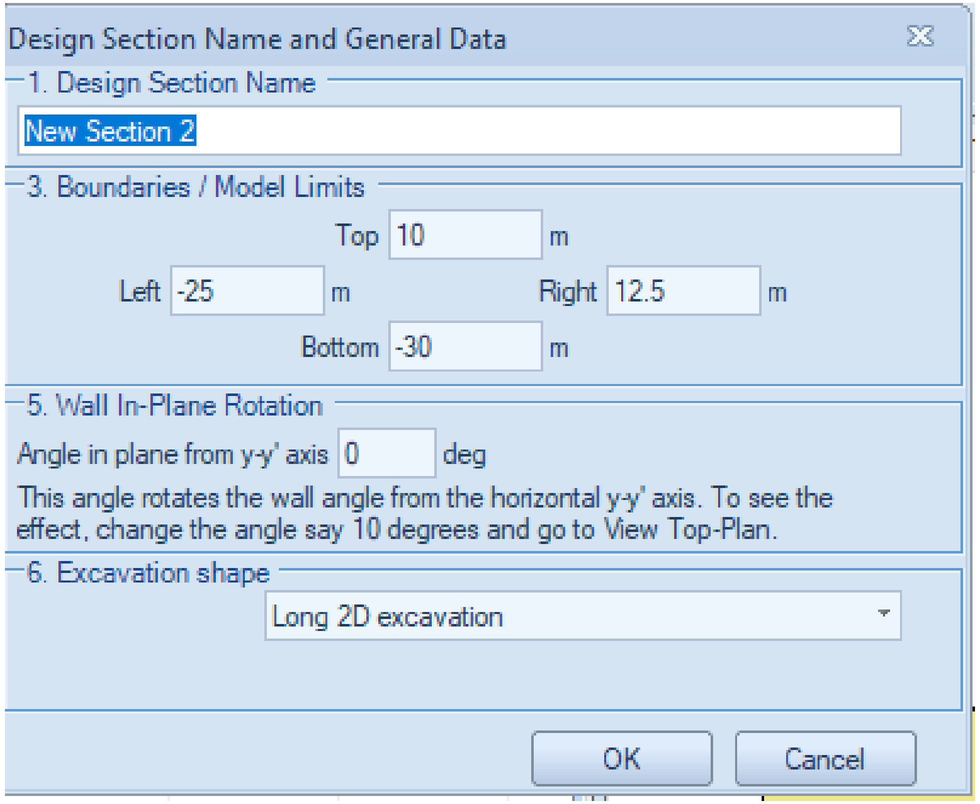
Meshing and boundaries

Example 4: symmetry conditions



Meshing and boundaries

<u>Example 4:</u> symmetry conditions

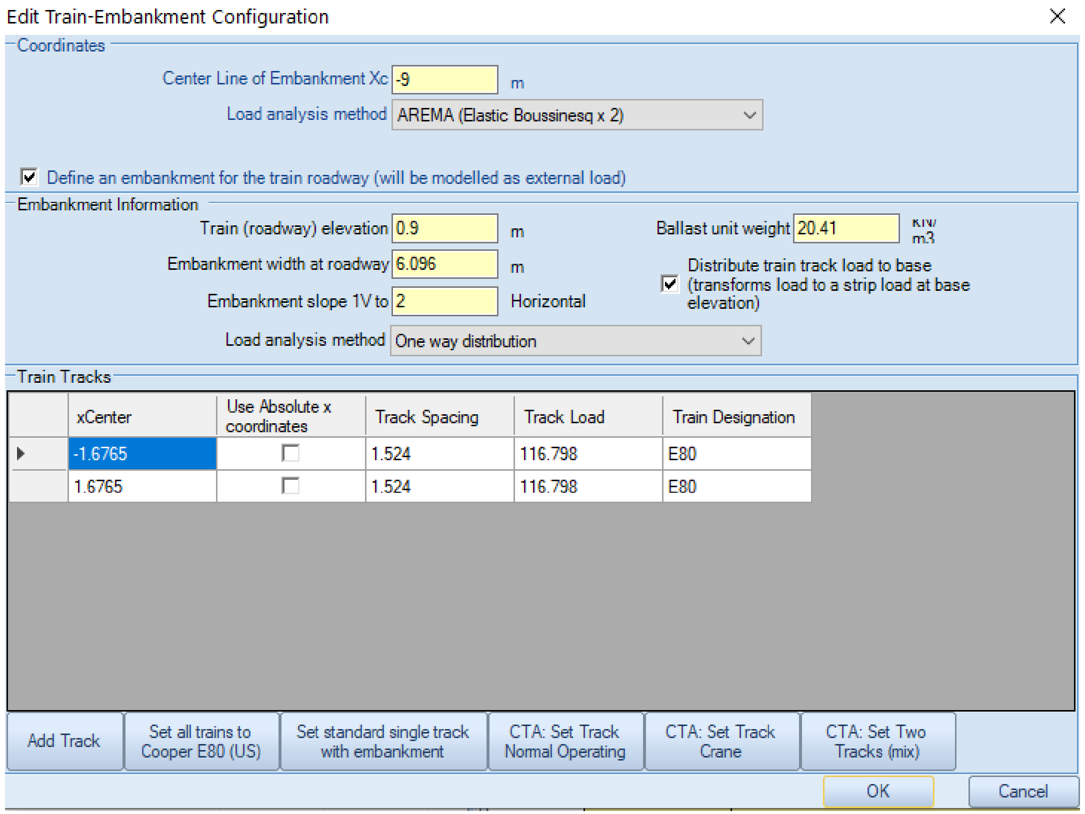


Top Down construction:

Stage 0 : greenfield conditions
Stage 1: installation of walls
Stage 2: excavation at depth **-1** m
Stage 3: construct top slab at -1 m
stage 4: excavation at depth -8 m
Stage 5: add a temporary prop at depth -7m
stage 6: excavation at depth -12 m
stage 7: construct floor slab at -12 m
Stage 8 : remove temporary pros

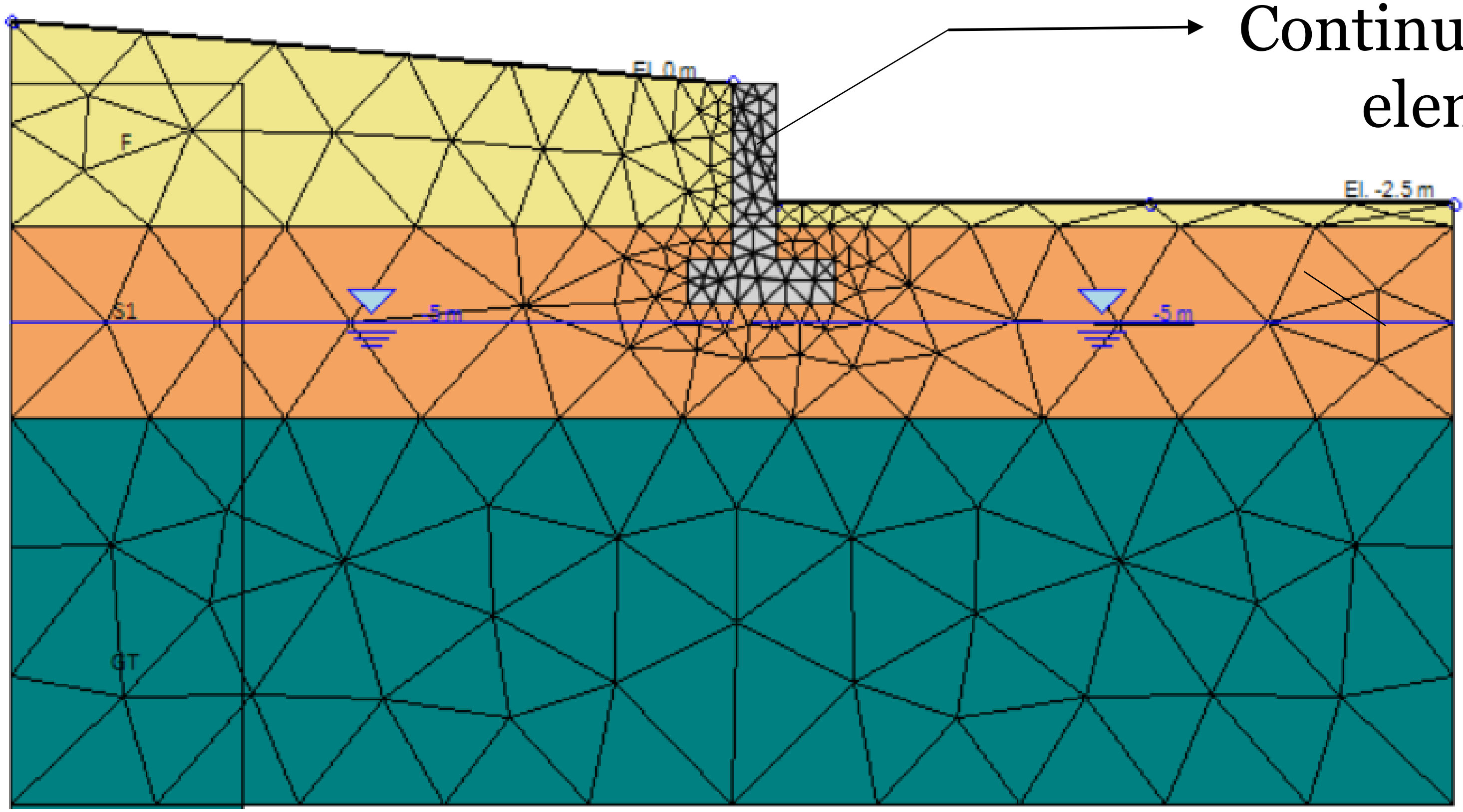
Meshing and boundaries

Example 4: symmetry conditions



Structural components and interface

Structural components **can** be simulated with continuum elements:



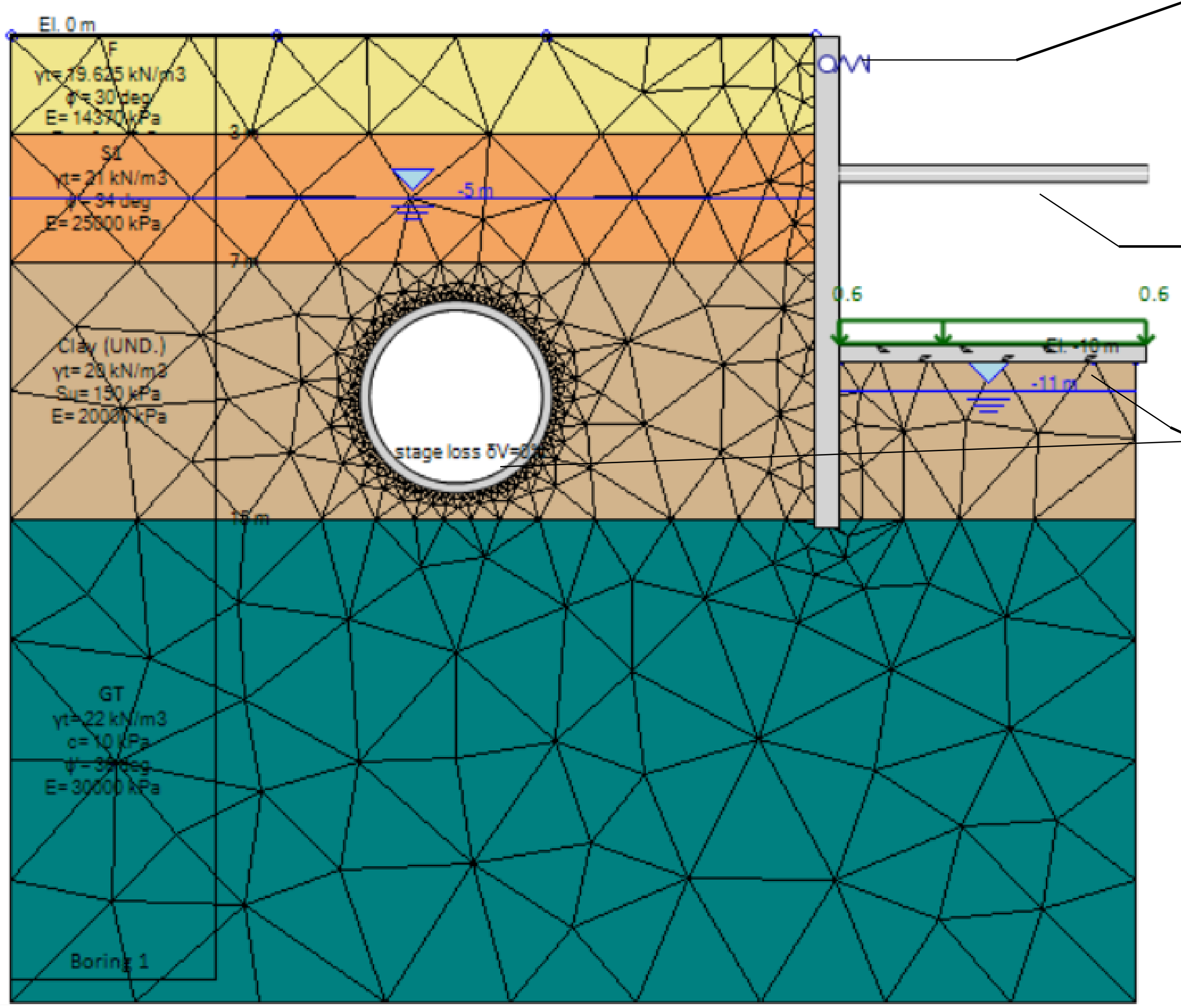
Continuum finite elements

Structural components and interface

Common use of additional **special** finite elements.

Advantages:

- inelastic behaviour
- computationally faster
- stress directly in member level



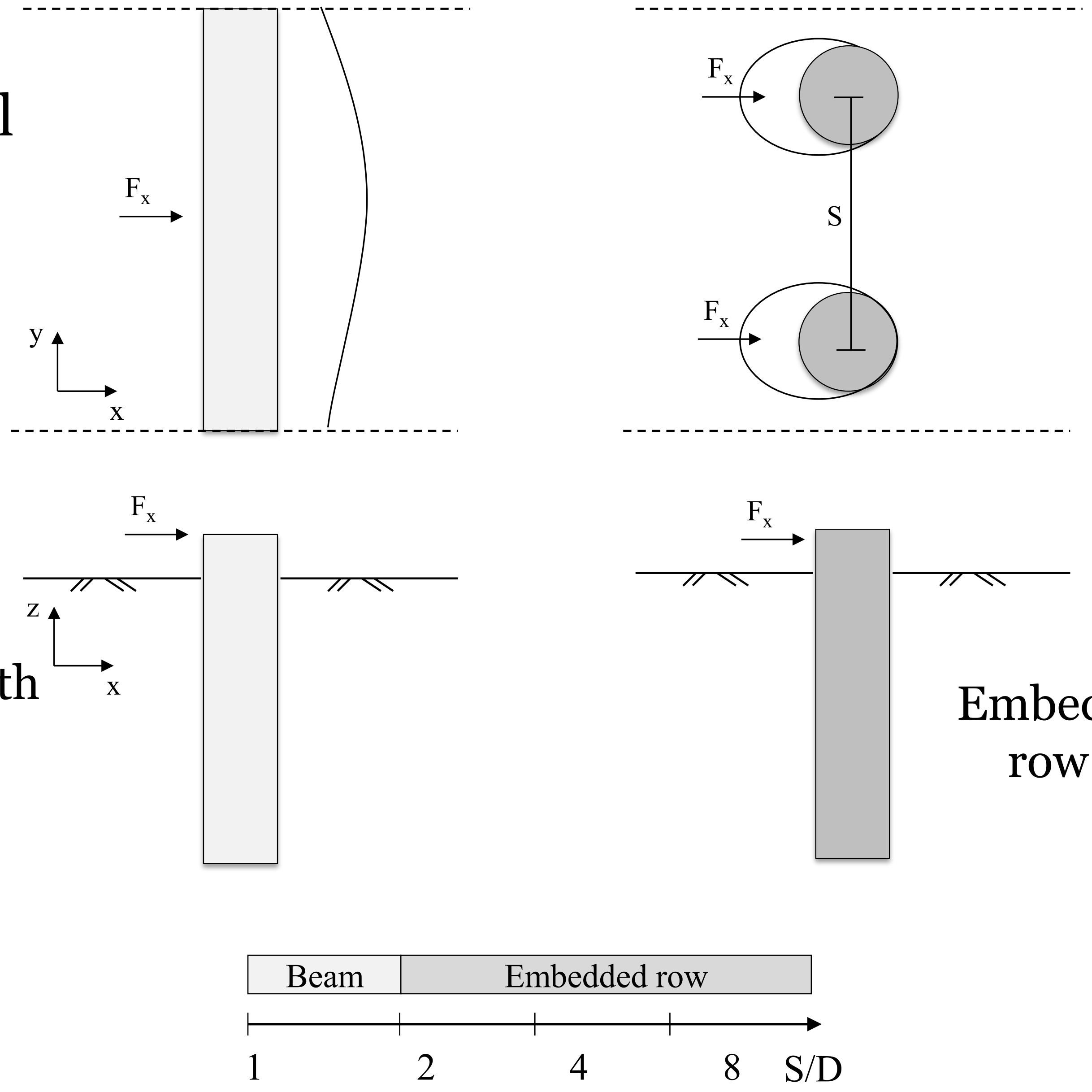
Spring element

Strut element

Plane strain shell element

Structural components and interface

Two modelling options for structural components in soil:

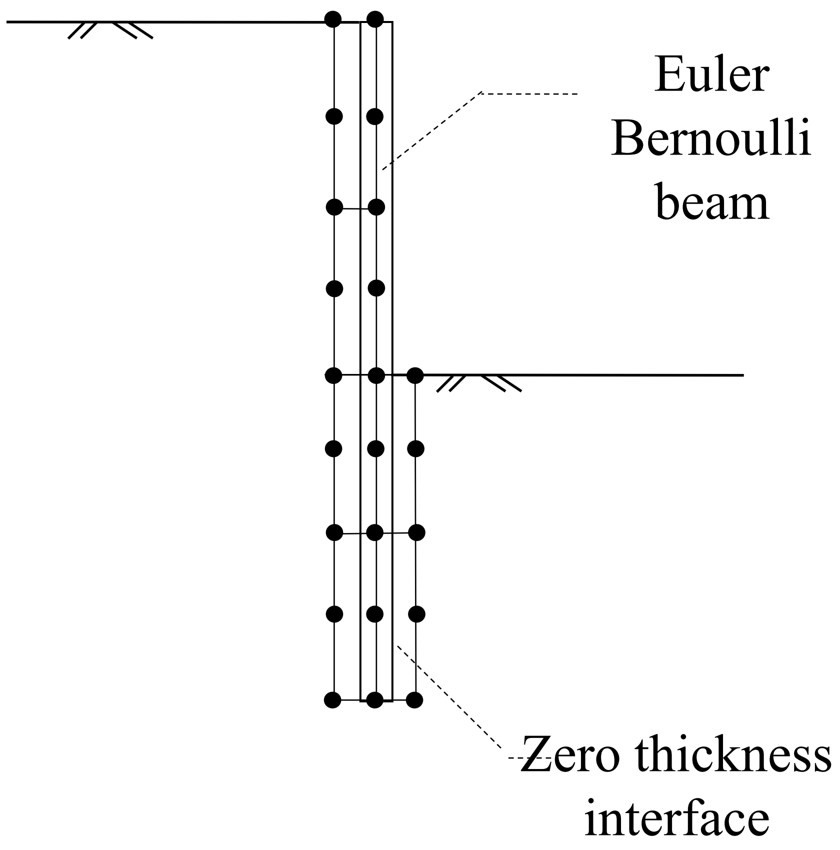


Beam method with zero thickness interface

Embedded beam row method

Structural components and interface

Beam method with zero thickness interface

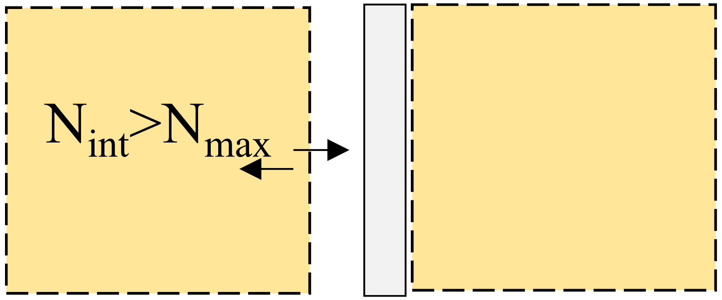


(soldier pile with lagging, sheet pile, secant pile, tangent pile, diaphragm,gravity walls)

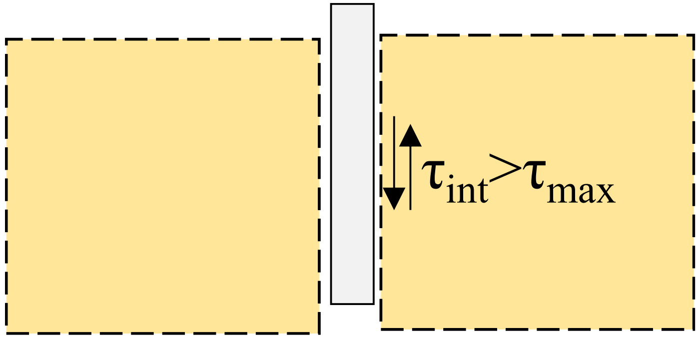
Structural components and interface

Zero thickness element

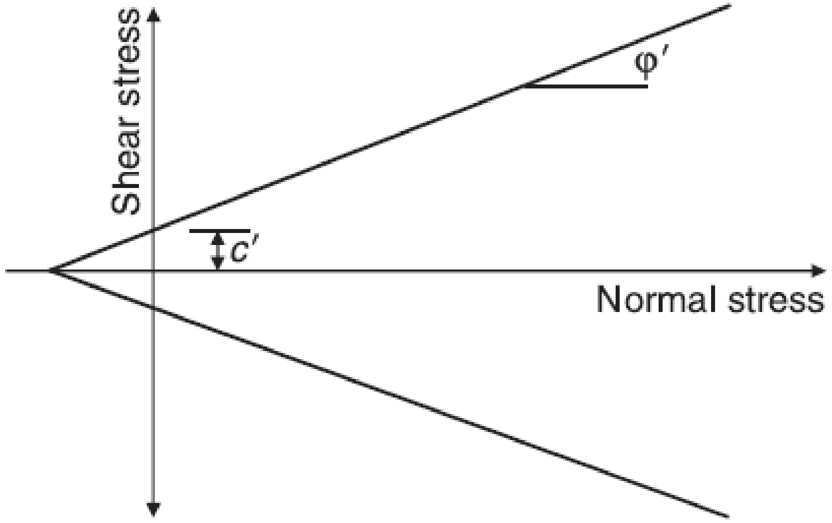
- δ angle
- R reduction factor (Strength/Stiffness)



Separation
(interface tension failure)



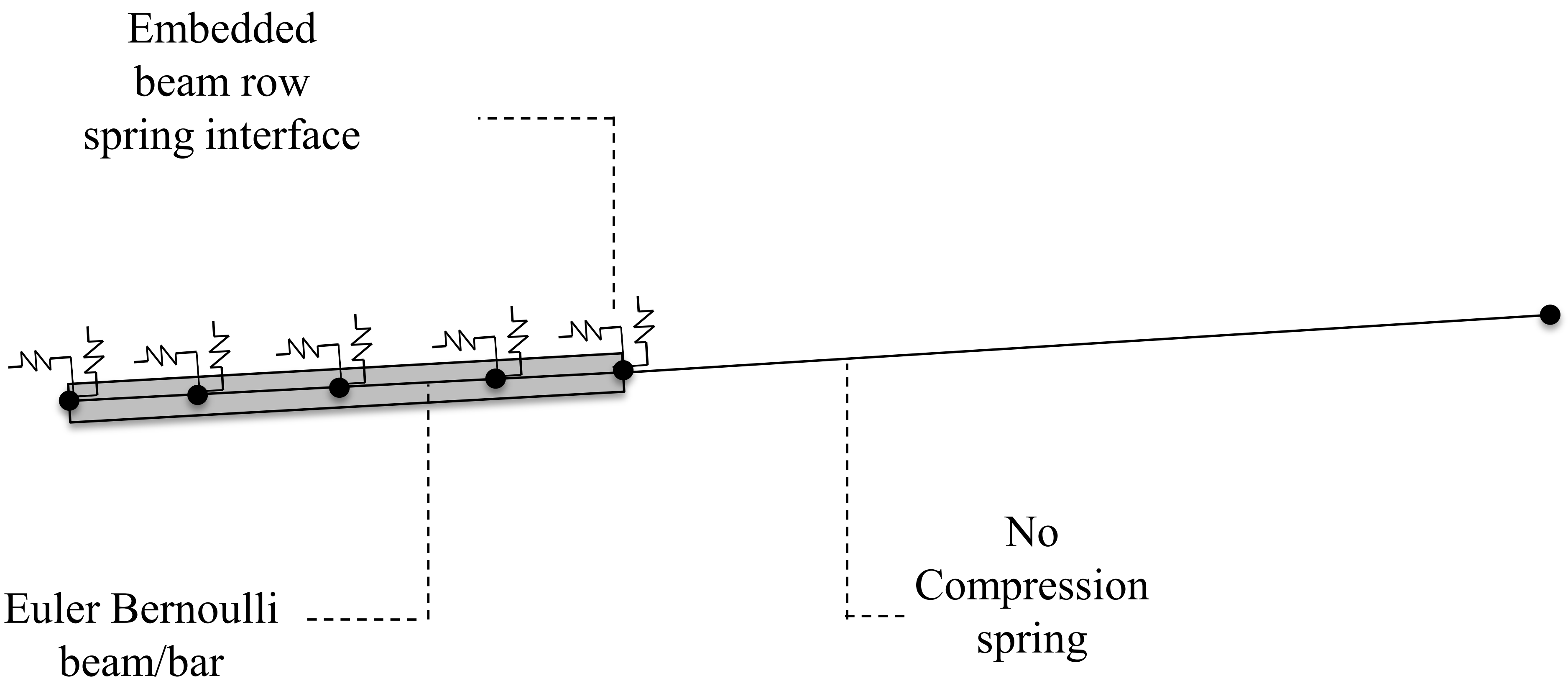
Slippage
(interface shear failure)



Mohr coulomb law

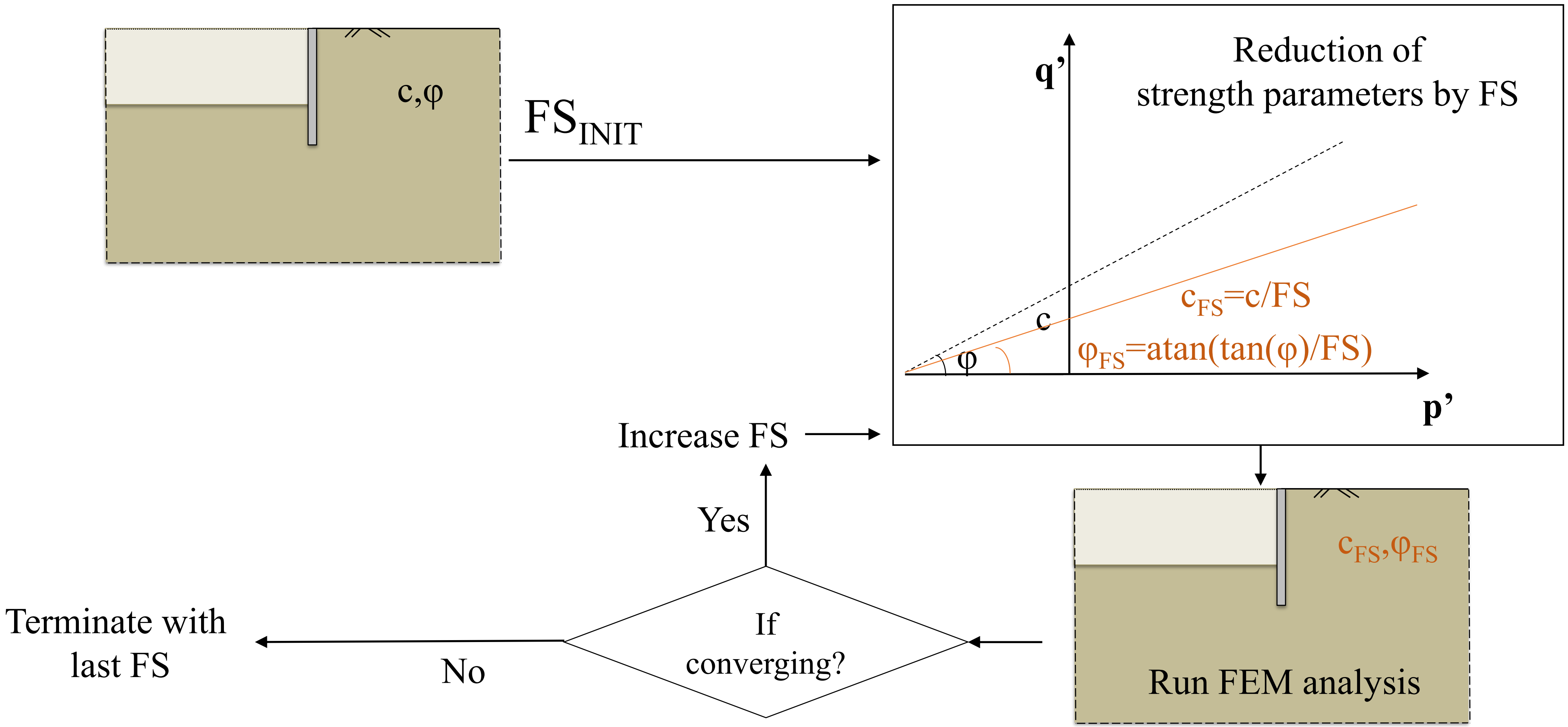
Structural components and interface

Embedded beam row method



(Pile rows, tiebacks, tiedowns, soil nails, rock bolts)

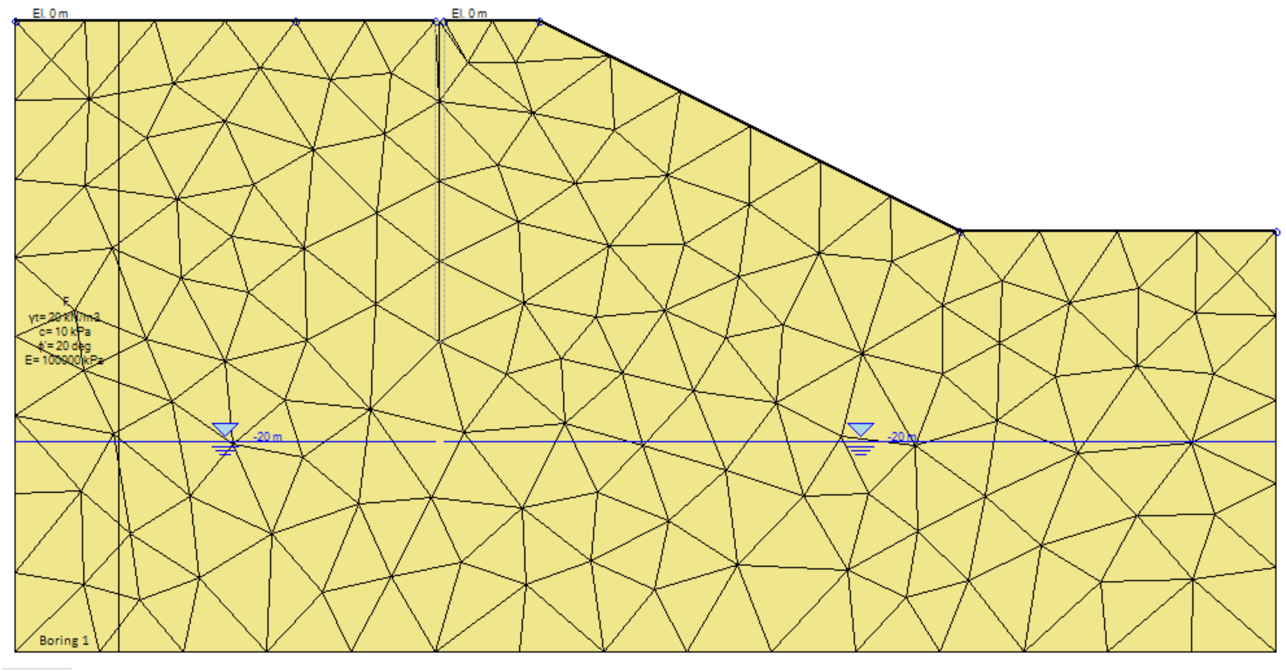
Strength reduction analysis



Strength reduction analysis

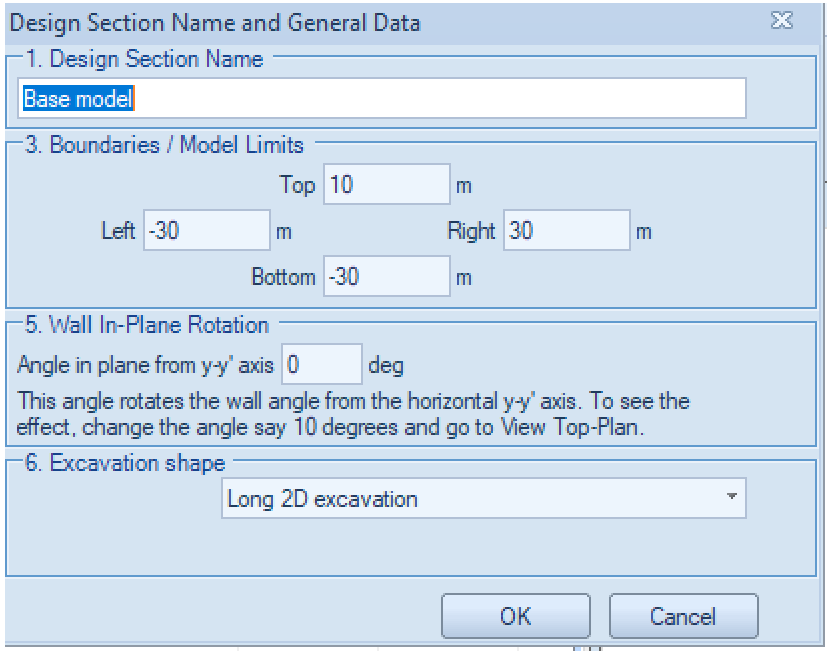
Example 5: Shear strength reduction analysis example

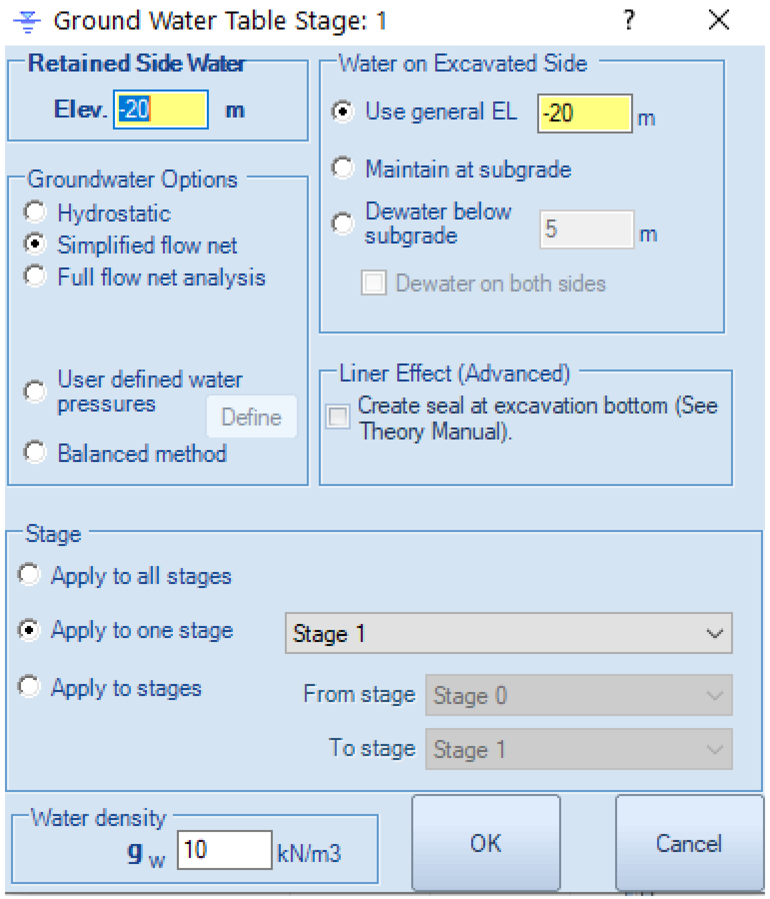
Parameter	Value	Unit
Elastic modulus	10^5	kN/m^2
Poisson's ratio	0.3	--
Unit weight	20	kN/m^3
Cohesion	10	kN/m^2
Friction angle	20	^{0}
Dilatancy angle	0	^{0}



Strength reduction analysis

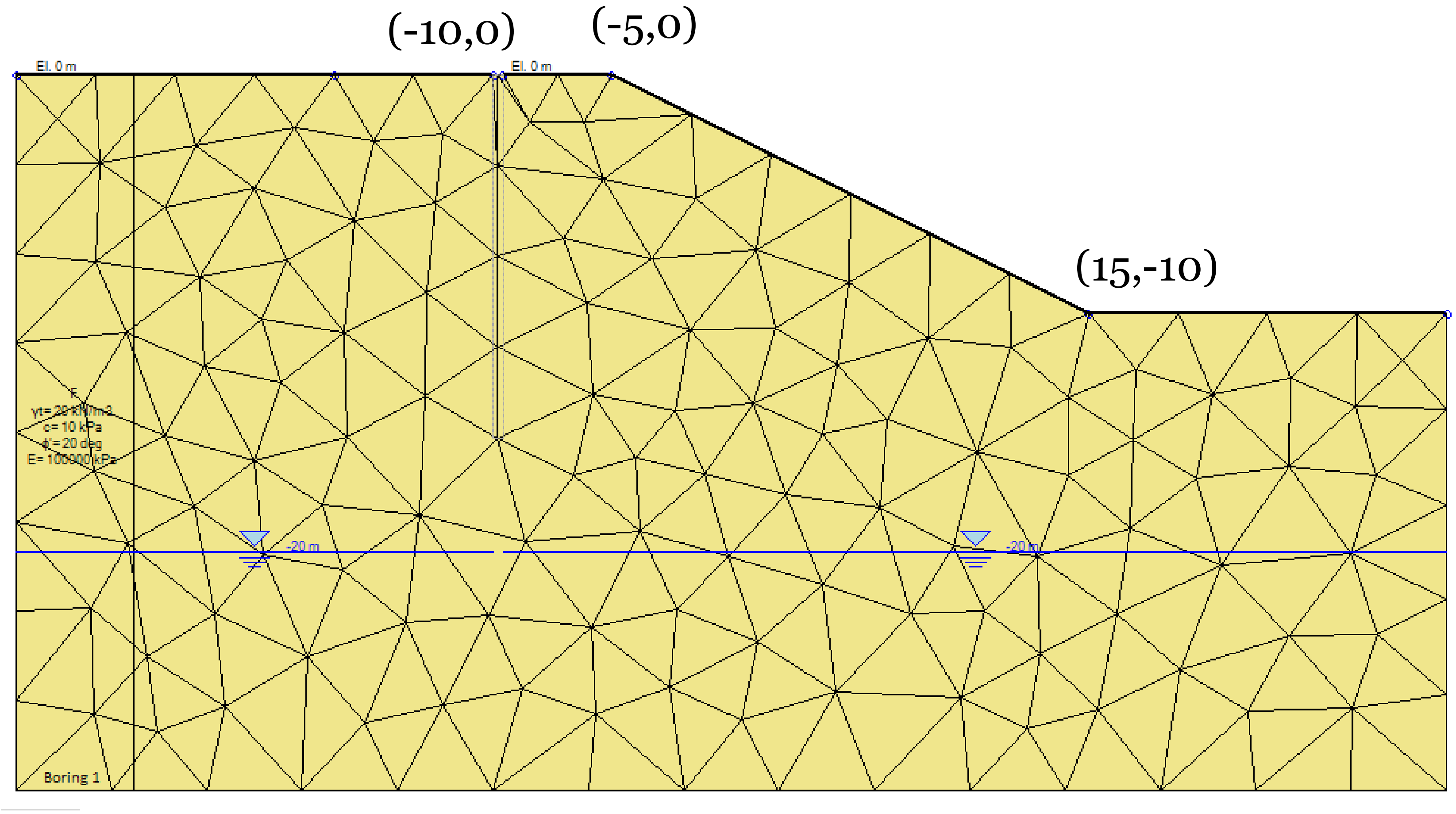
Example 5: Shear strength reduction analysis example



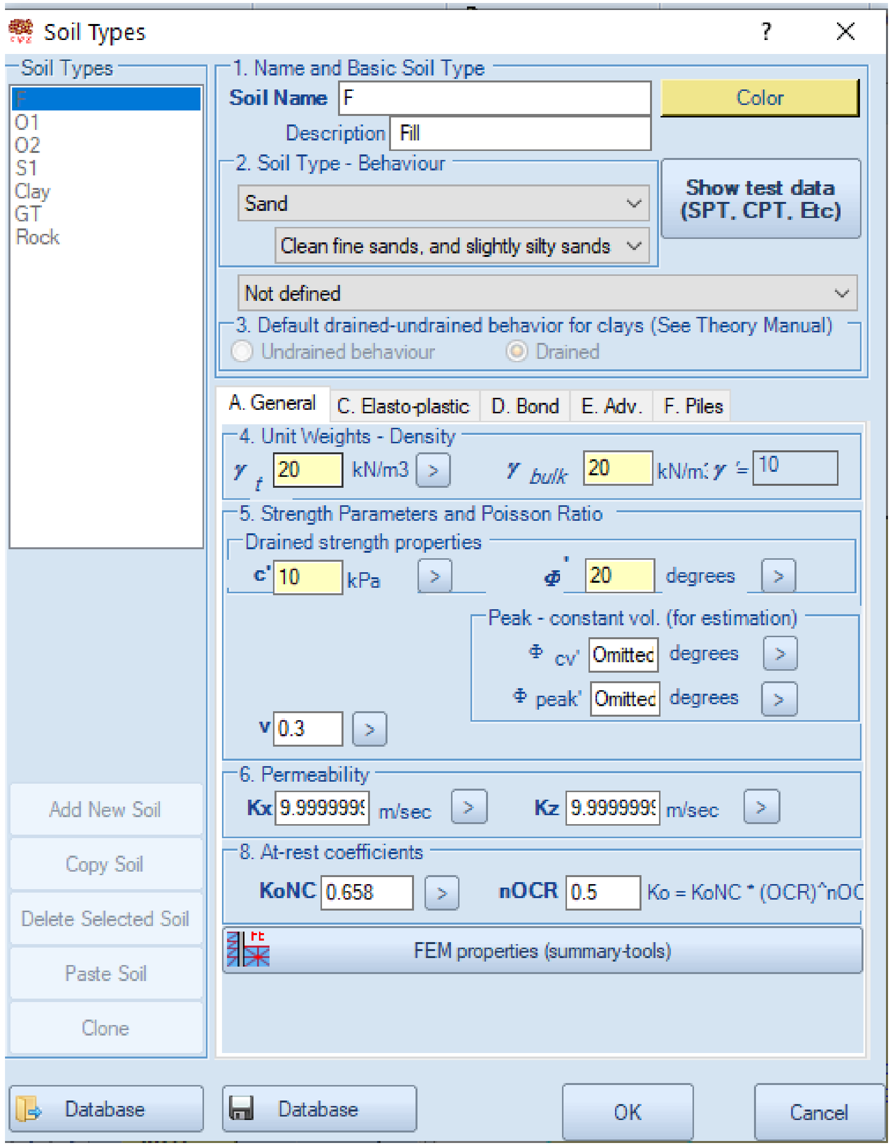


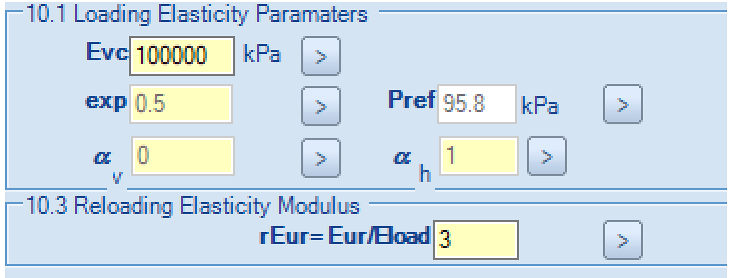
Strength reduction analysis

Example 5: Shear strength reduction analysis example



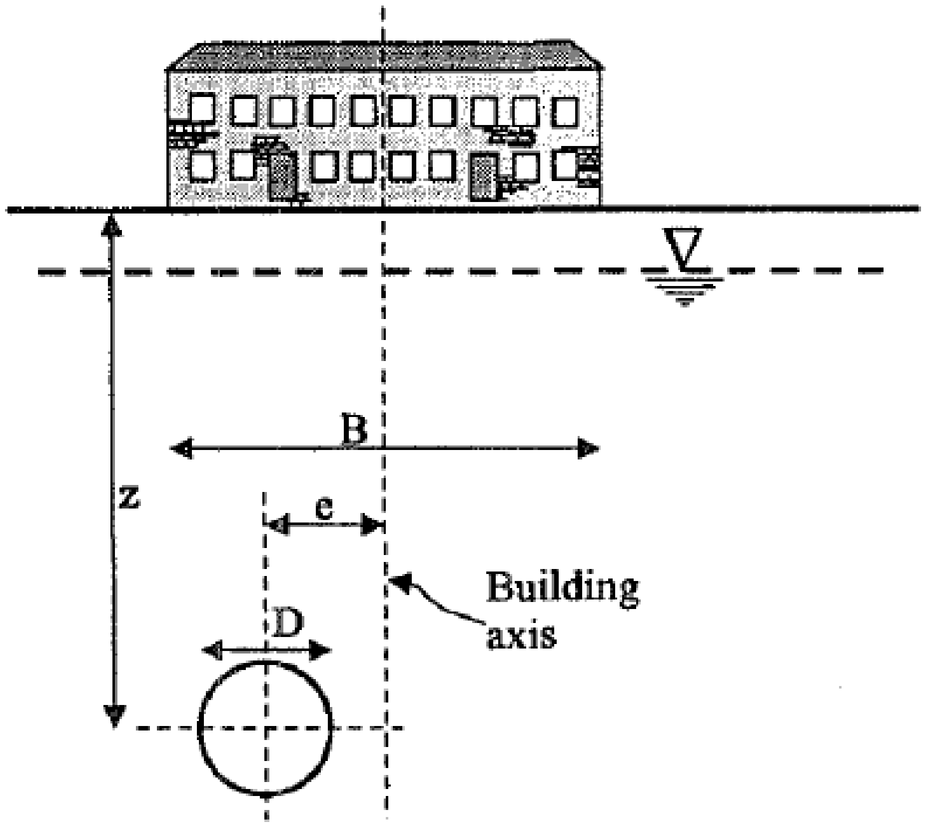
Example 5: Shear strength reduction analysis example





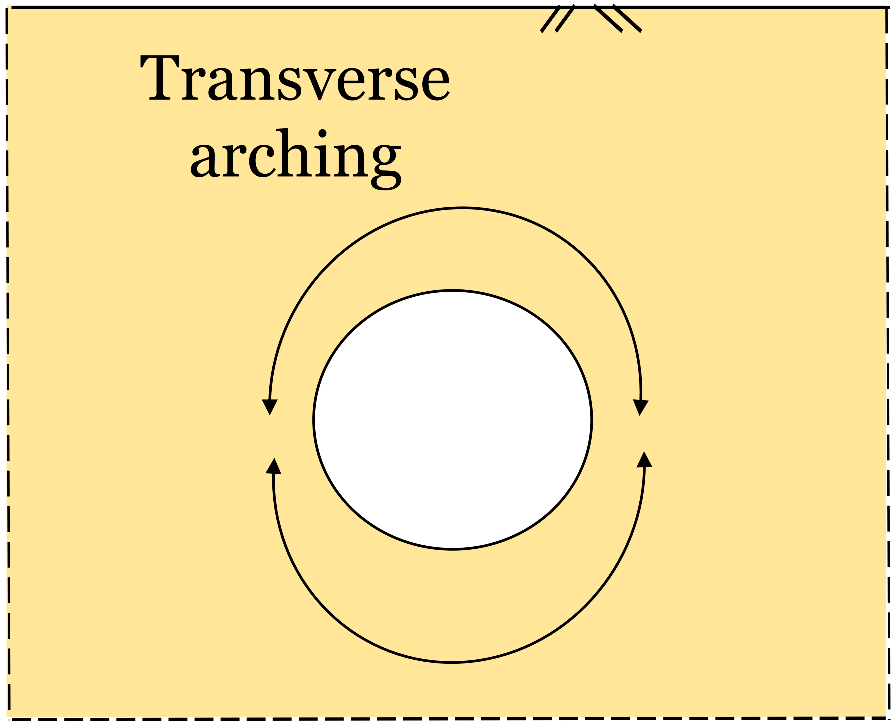
Tunneling

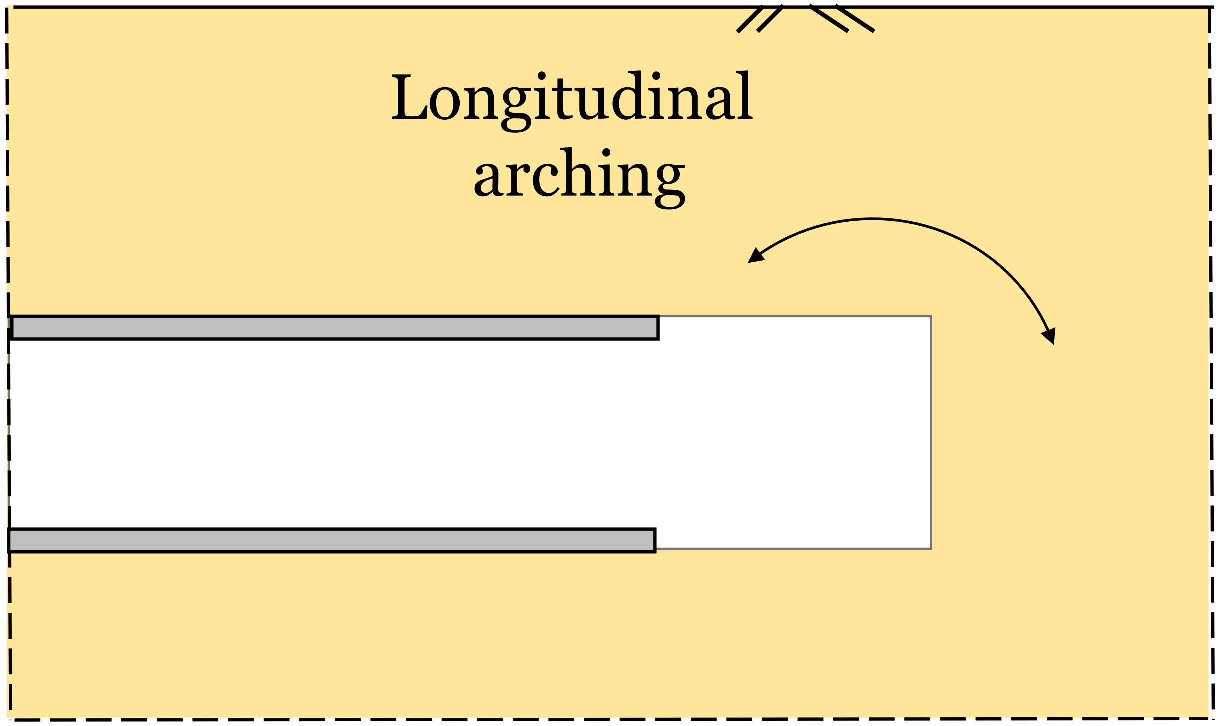
- Tunnel construction affects existing structures
- Complex soil structure interaction problem
- full numerical analysis essential



Tunneling

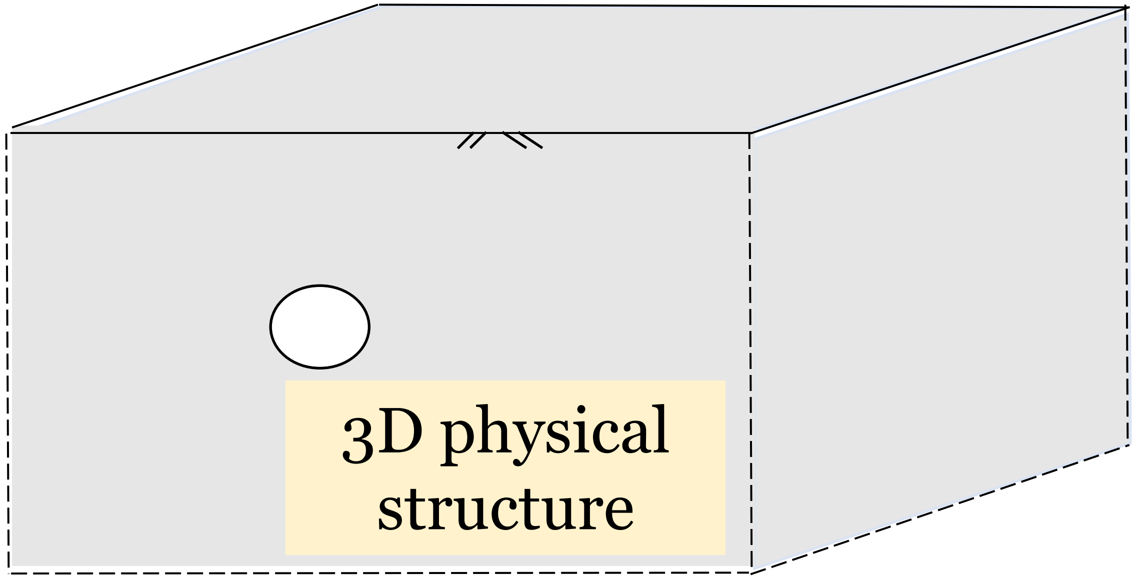
3D load transfer

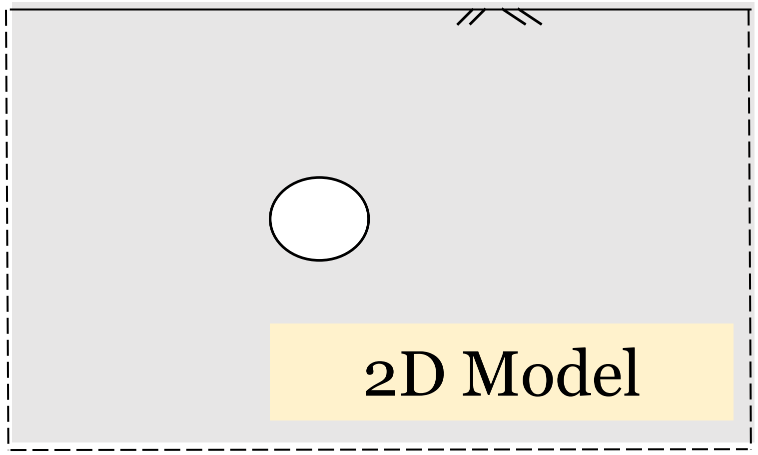




transverse and longitudinal arching around the unsupported section

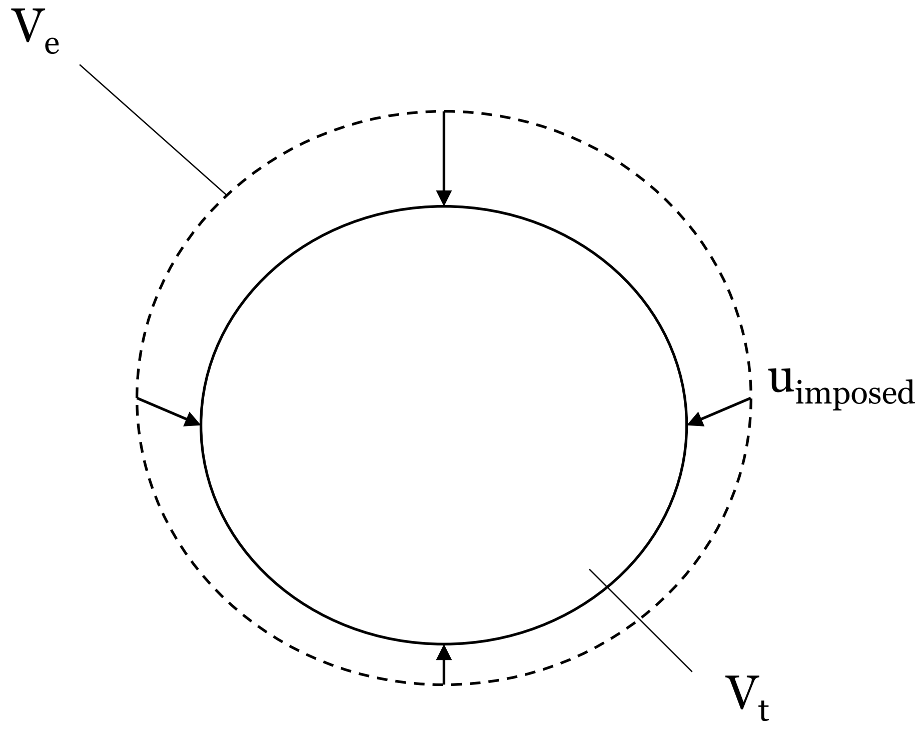
TBM tunnel construction stages

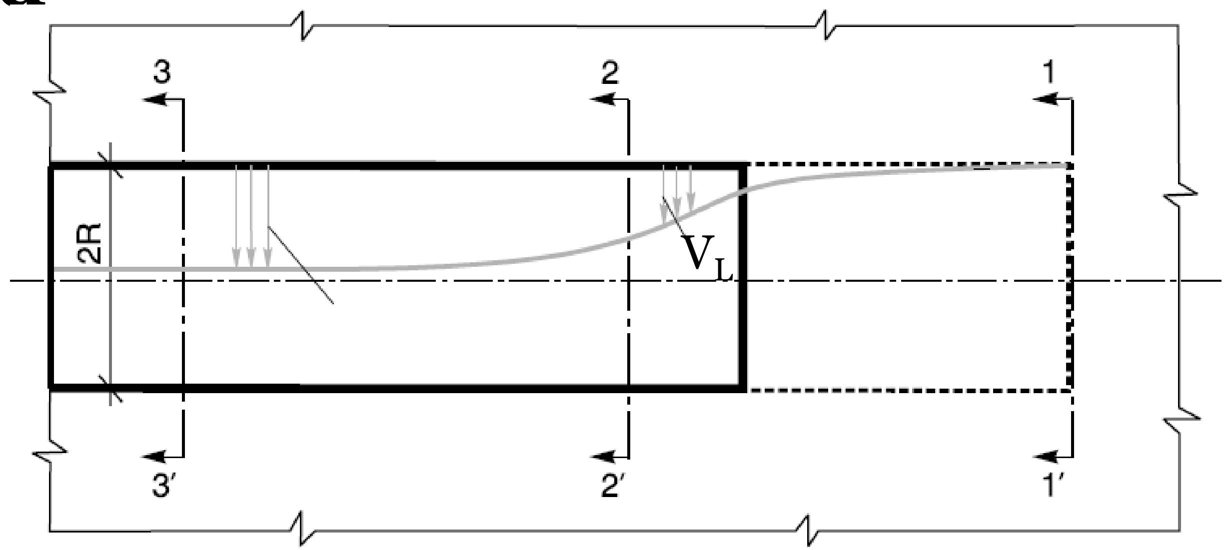




When modelling in plane strain something must be prescribed, rather than predicted.

The volume loss control method (contraction)





Displacement based contraction:

$V_L = V_e - V_t$

V_e: excavated tunnel volume
V_t: final tunnel volume

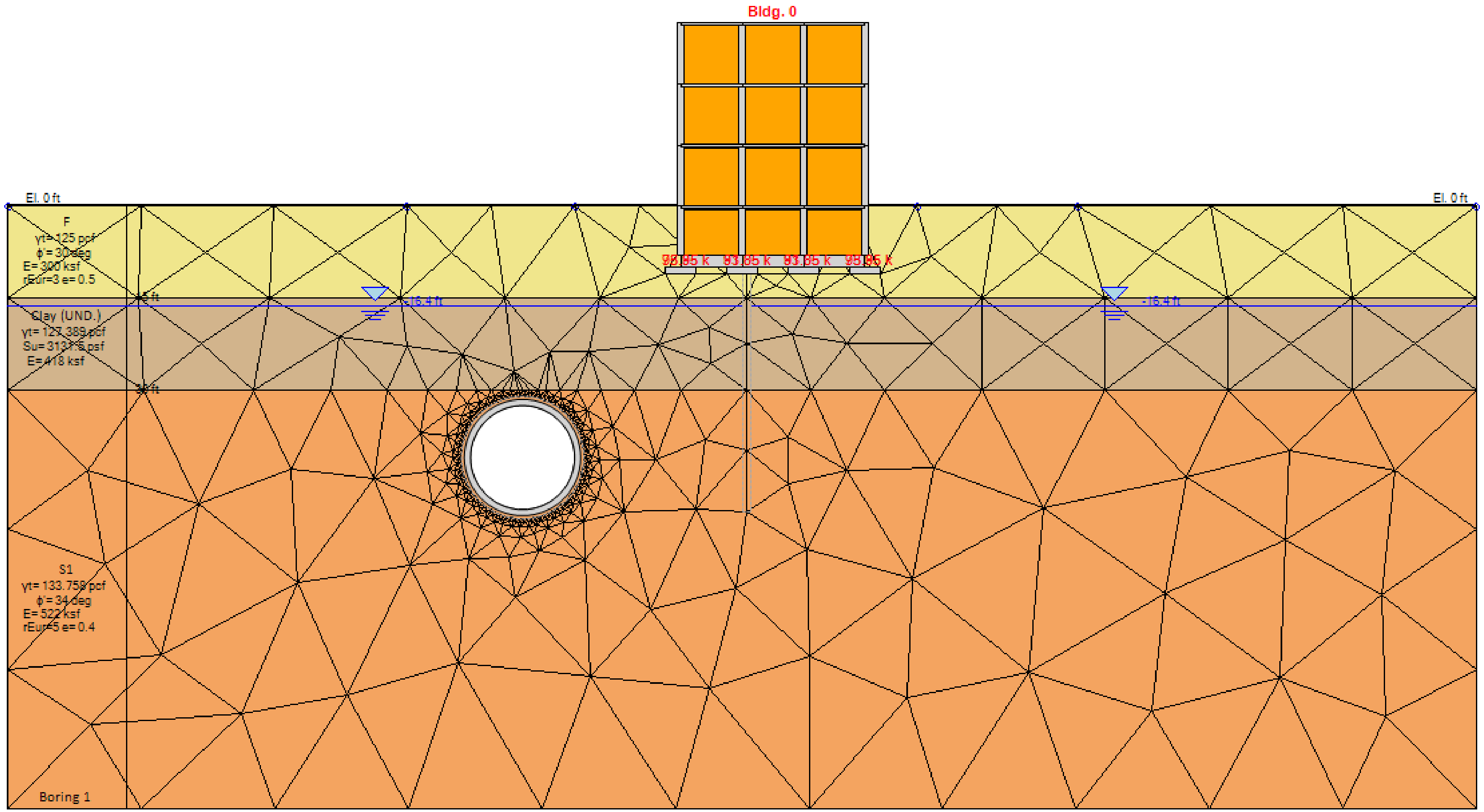
Indication of volume loss values

Table 7-7 Relationship between Volumes Loss and Construction Practice and Ground Conditions

Case	V_L **(%)**
Good practice in firm ground; tight control of face pressure within closed face machine in slowly raveling or squeezing ground	0.5
Usual practice with closed face machine in slowly raveling or squeezing ground	1.0
Poor practice with closed face in raveling ground	2
Poor practice with closed face machine in poor (fast raveling) ground	3
Poor practice with little face control in running ground	4.0 or more

Tunneling

Example 6: TBM constructed tunnel



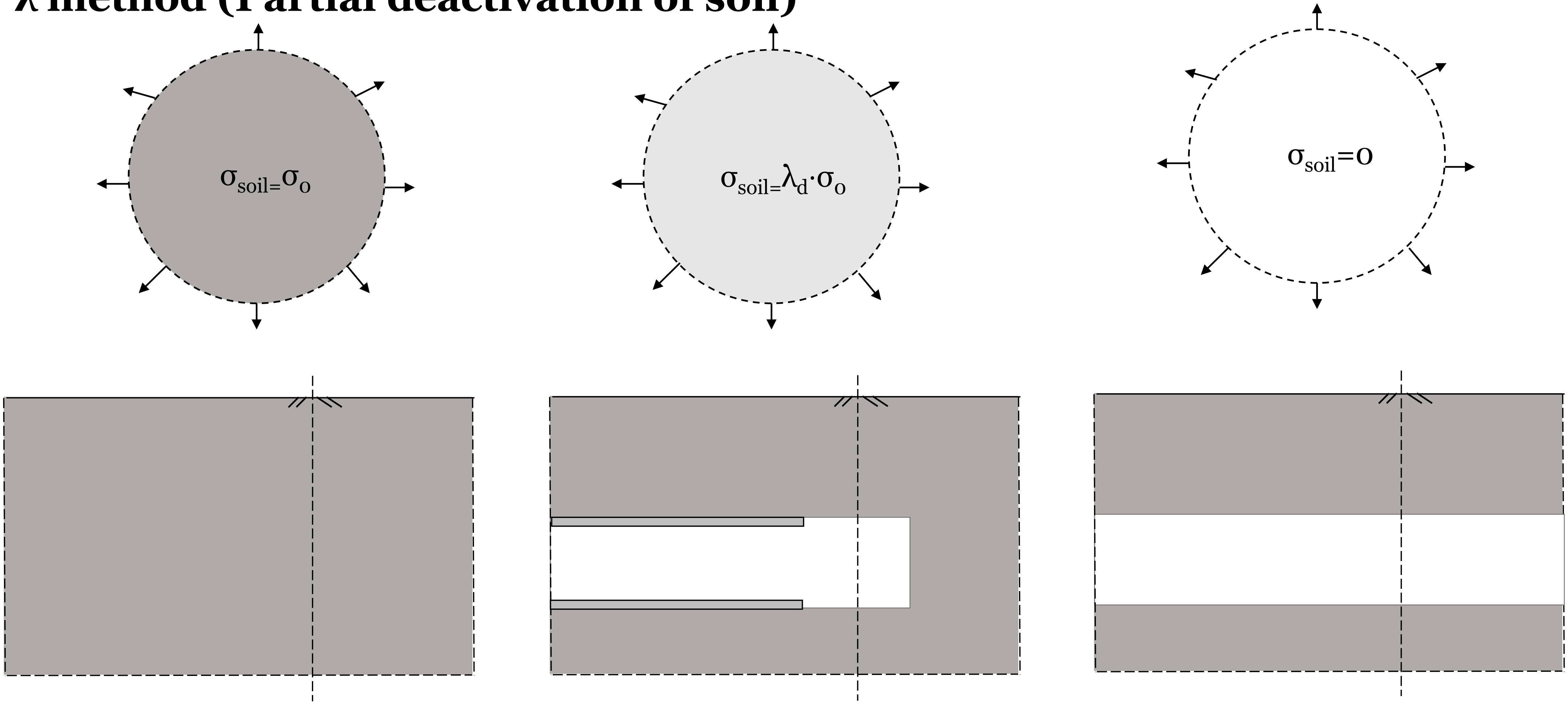
Tunneling

Example 6: TBM constructed tunnel

Stage 0 : greenfield conditions
Stage 1: building structure (conditions prior to tunnel construction)
Stage 2: excavation of tunnel and TBM activation
Stage 3: contraction (1 %)
Stage 4: grouting stage (3.5 ksf)
Stage 5: concrete lining installation

Tunneling

λ method (Partial deactivation of soil)



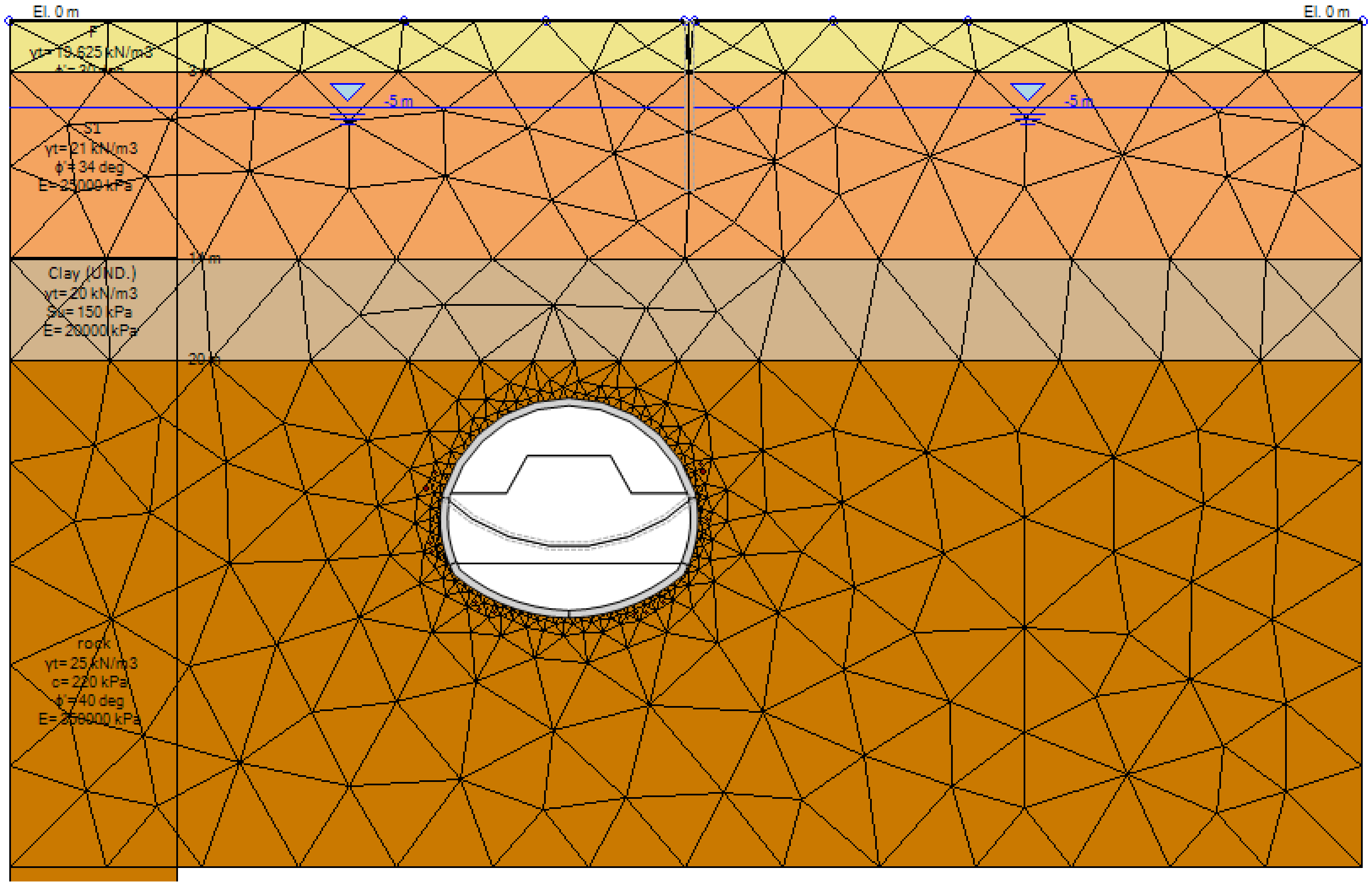
Prior to excavation

Tunnel excavation stage

Plain strain excavation

Tunneling

Example 7: SEM constructed tunnel



Tunneling

Example 7: SEM constructed tunnel

Stage 0: green field conditions.
Stage 1: excavation of the upper part of the top heading (partial 20%)
Stage 2: the tunnel crown is supported
Stage 3: Excavation of the support core (partial 20%)
Stage 4: A temporary lining is constructed at the bottom of the so far excavated section.
Stage 5: full excavation of top heading 100%
Stage 6: excavation of the bench part of the tunnel section (partial 20%)
Stage 7: construction of the remaining lining sections (left, right wall and invert)
Stage 8: full excavation of remaining tunnel 100%

Seismic design

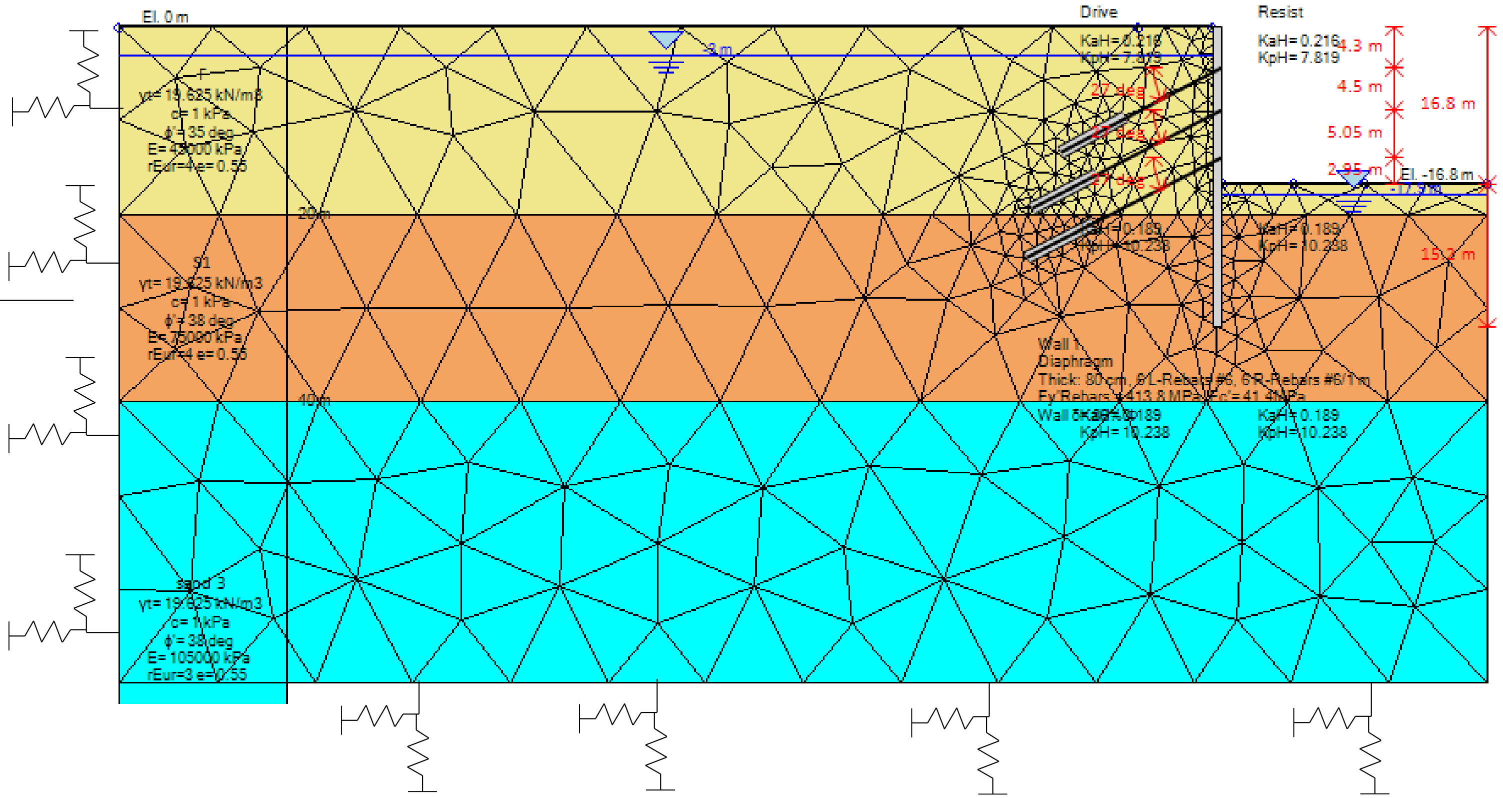
Novak Boundaries

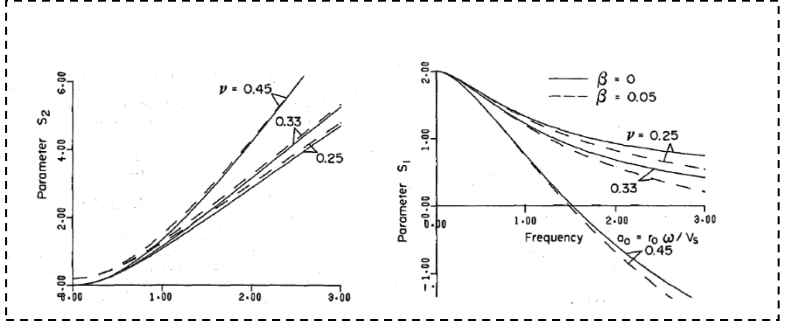
Fundamental Solution of wave equation in infinite disc:

$$\rho \frac{\partial^2 u}{\partial t^2} = (\lambda + \mu)\frac{\partial \bar{\varepsilon}}{\partial x} + \mu \nabla^2 u$$

$$\rho \frac{\partial^2 v}{\partial t^2} = (\lambda + \mu)\frac{\partial \bar{\varepsilon}}{\partial y} + \mu \nabla^2 v$$

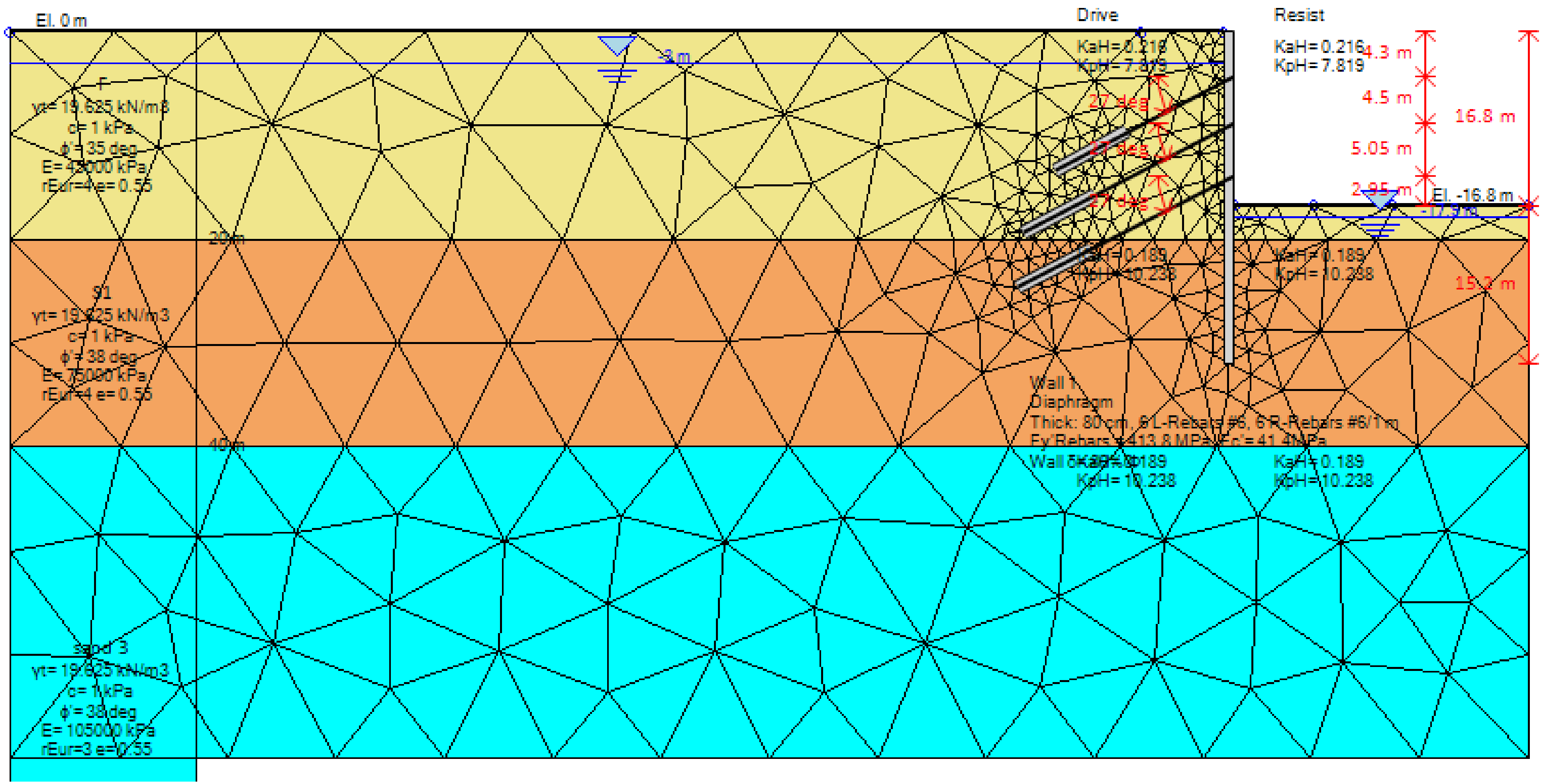
$$\rho \frac{\partial^2 w}{\partial t^2} = (\lambda + \mu)\frac{\partial \bar{\varepsilon}}{\partial z} + \mu \nabla^2 w$$





Seismic design

Example 8: Earthquake design

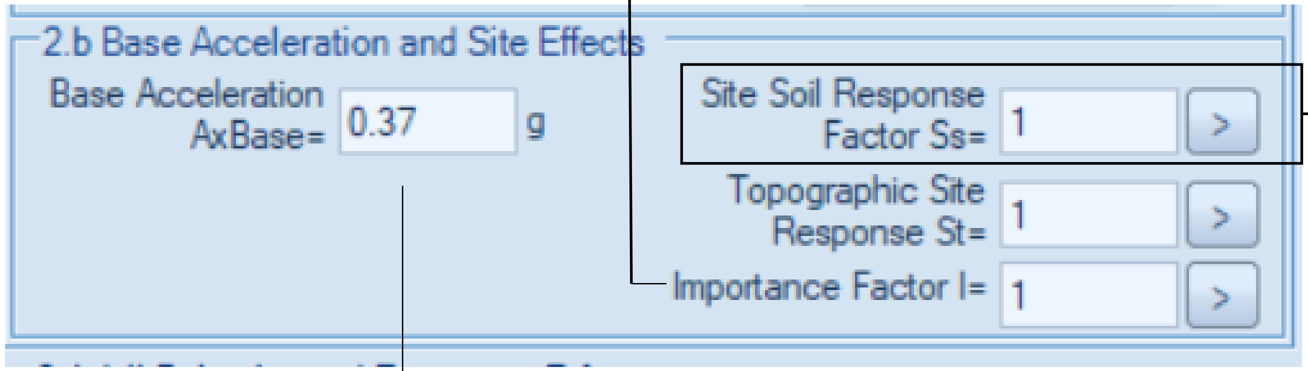


Seismic design

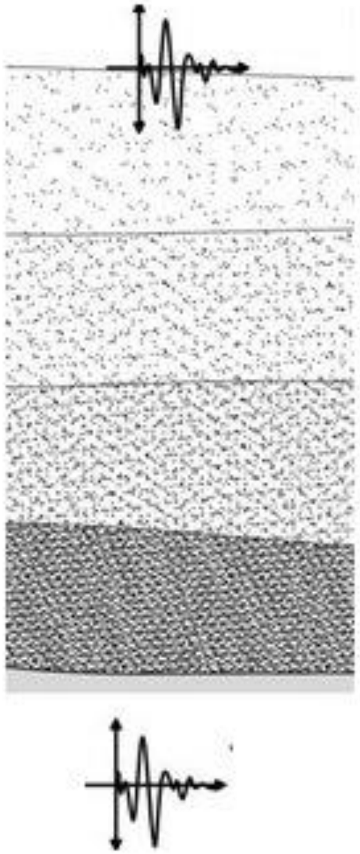
Calculation of design acceleration

- The user can directly prescribe the design acceleration
- Use bedrock acceleration and factors to generate a_{design}

Seismic code	Occupancy category		
	Higher	Moderate	Lower
IS 1893-Part1	1.5	1.2	1.0
Canadian Building Code Act	1.5	—	1.0
BS EN 1998-1	1.5	—	0.8
ASCE/SEI 7	1.5	1.25	1.0
NZS 1170 Part 5	1.3	—	0.6
NBC 105	2.0	1.5	1.0
EAK 2000	1.3	—	0.85
Iranian Standard 2800	1.2/1.4	1.0	0.8



Free-field motion



Earthquake at Bedrock

Seismic design

Calculation of Response R factor

- Rigid walls experience greater forces compared to yielding walls

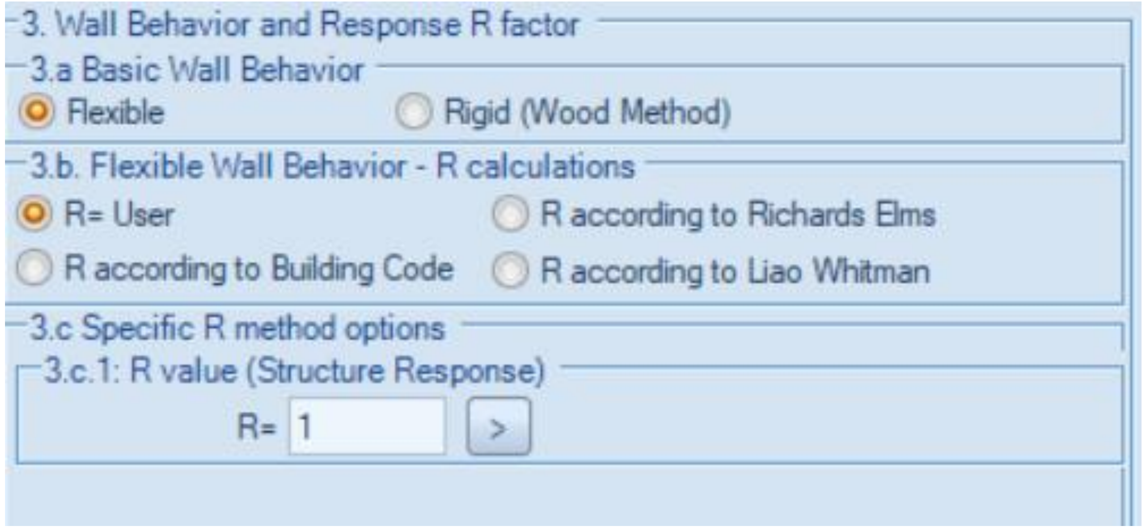


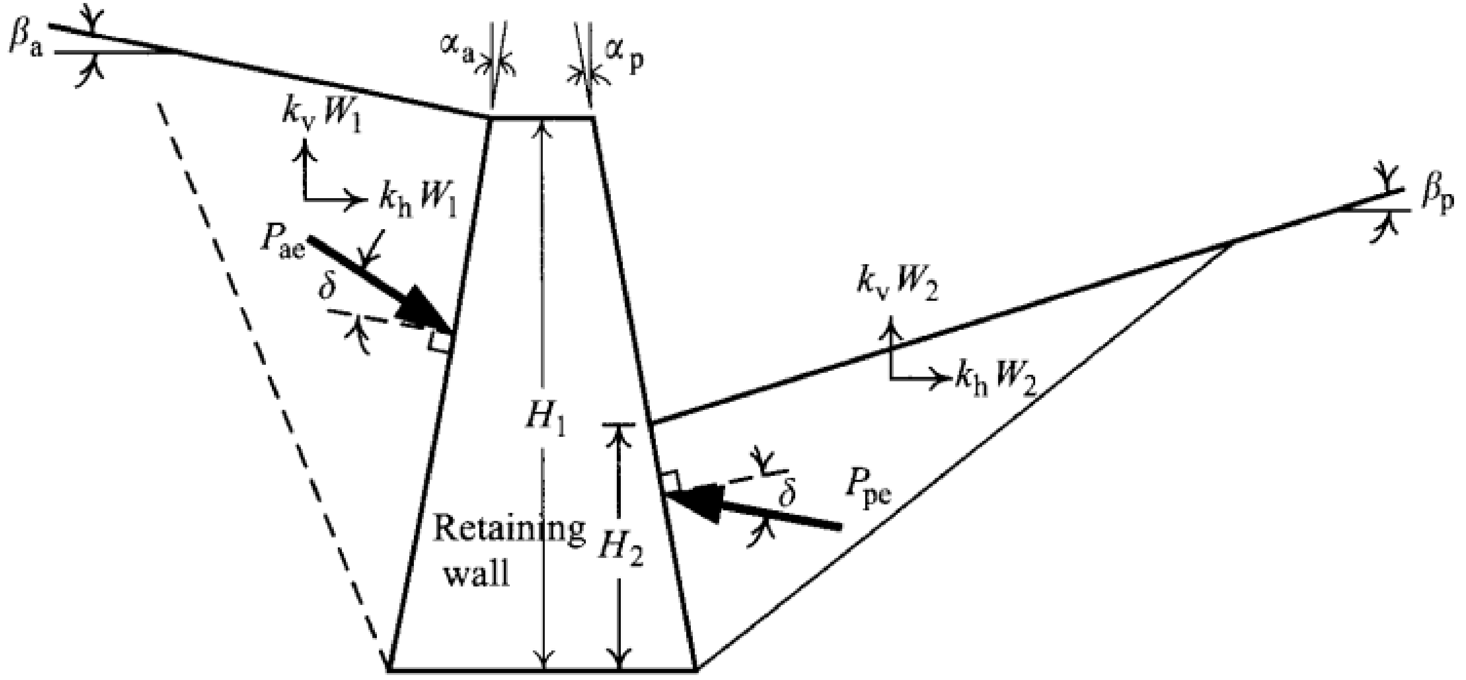
Table 7.1 — Values of factor *r* for the calculation of the horizontal seismic coefficient

Type of retaining structure	*r*
Free gravity walls that can accept a displacement up to $d_r = 300\ \alpha \cdot S$ (mm)	2
Free gravity walls that can accept a displacement up to $d_r = 200\ \alpha \cdot S$ (mm)	1,5
Flexural reinforced concrete walls, anchored or braced walls, reinforced concrete walls founded on vertical piles, restrained basement walls and bridge abutments	1

Eurocode EC8

Seismic design

Mononobe-Okabe :

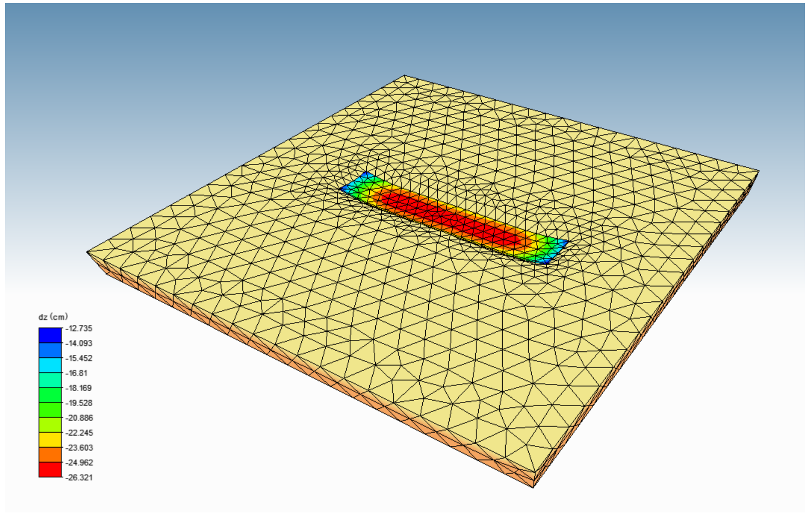


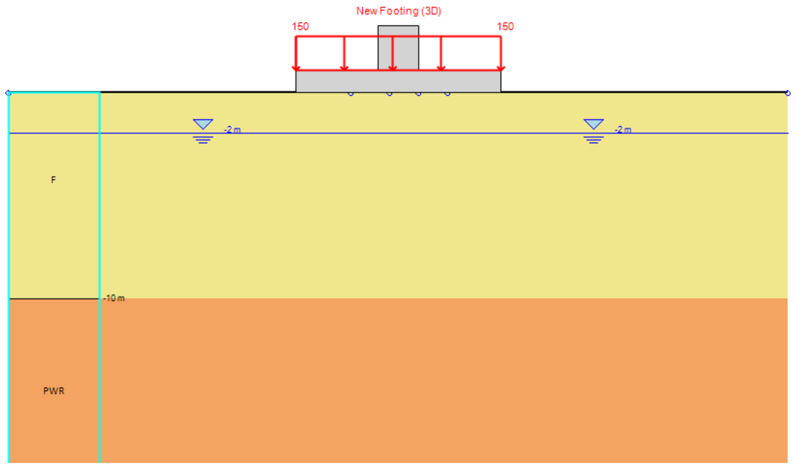
W_1 : Weight of wedge in the active zone

W_2 : Weight of wedge in the passive zone

2D plane strain and 3D models

Plane strain conditions:

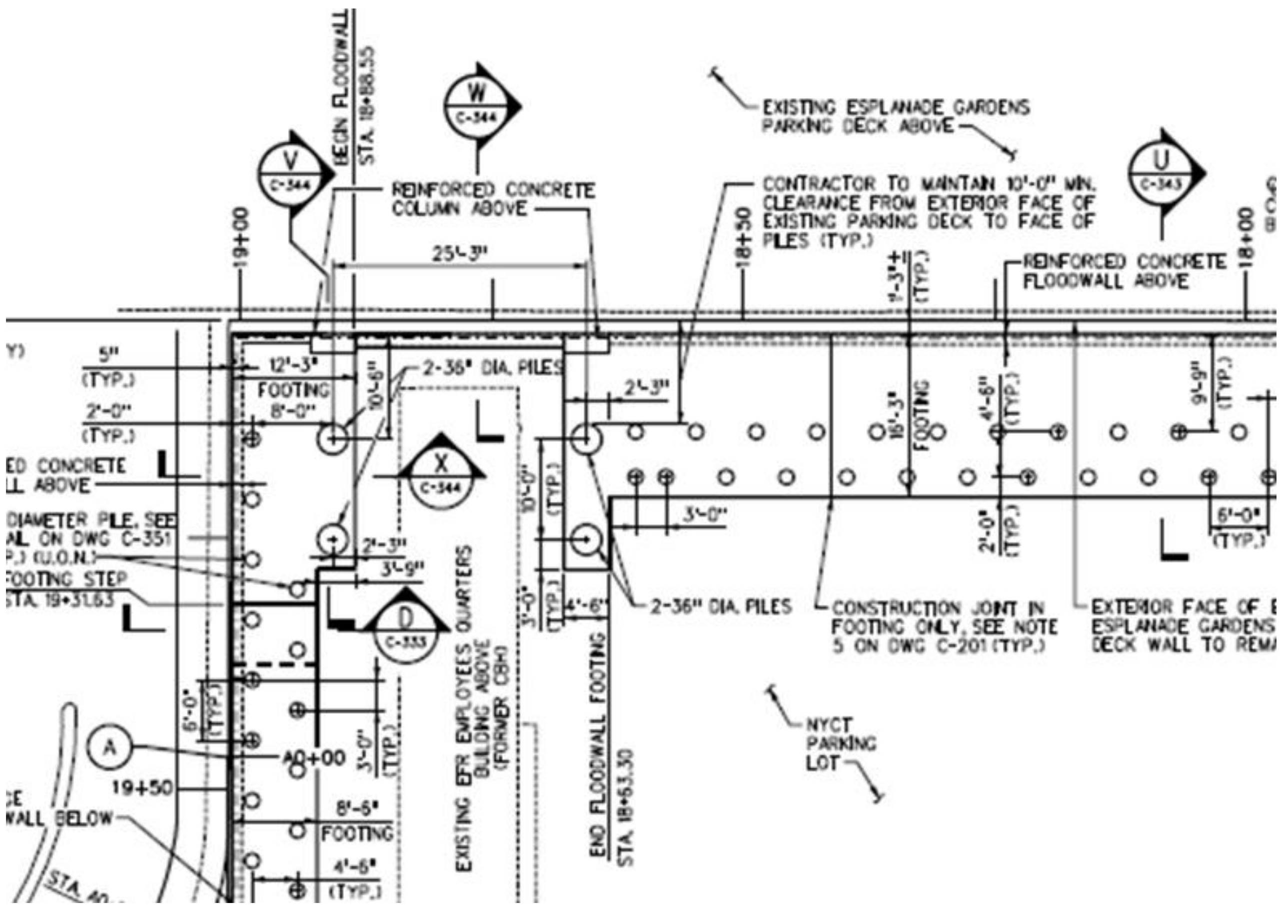


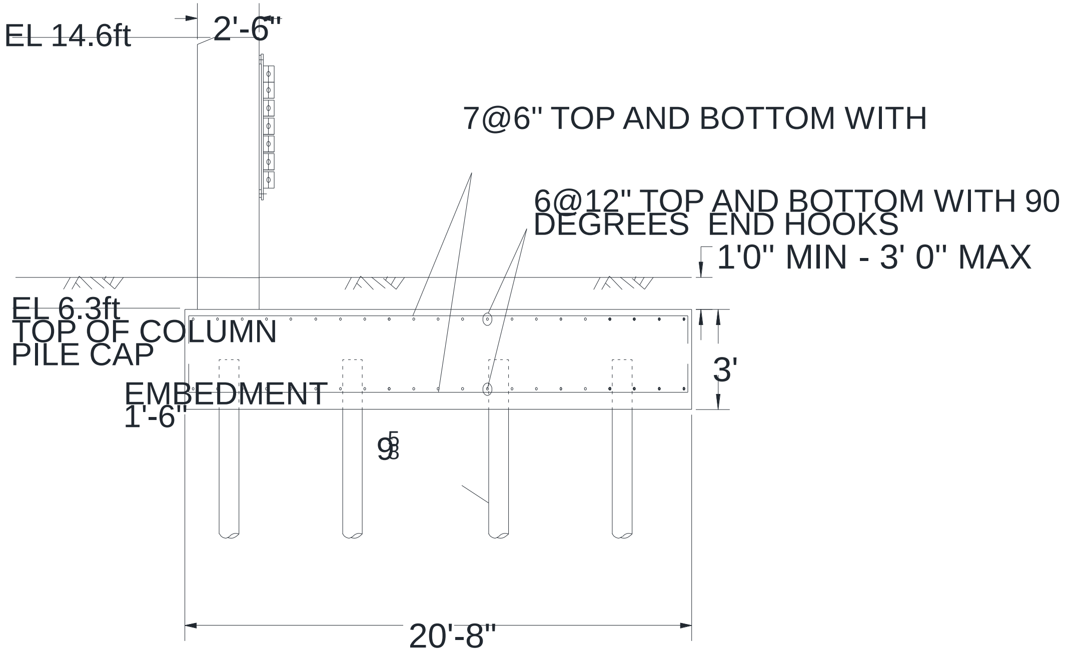


- Valid for systems with one dimension much larger than the others.
- Assumes zero strain on larger dimension
- stress on larger dimension is non-zero.

2D plane strain and 3D models

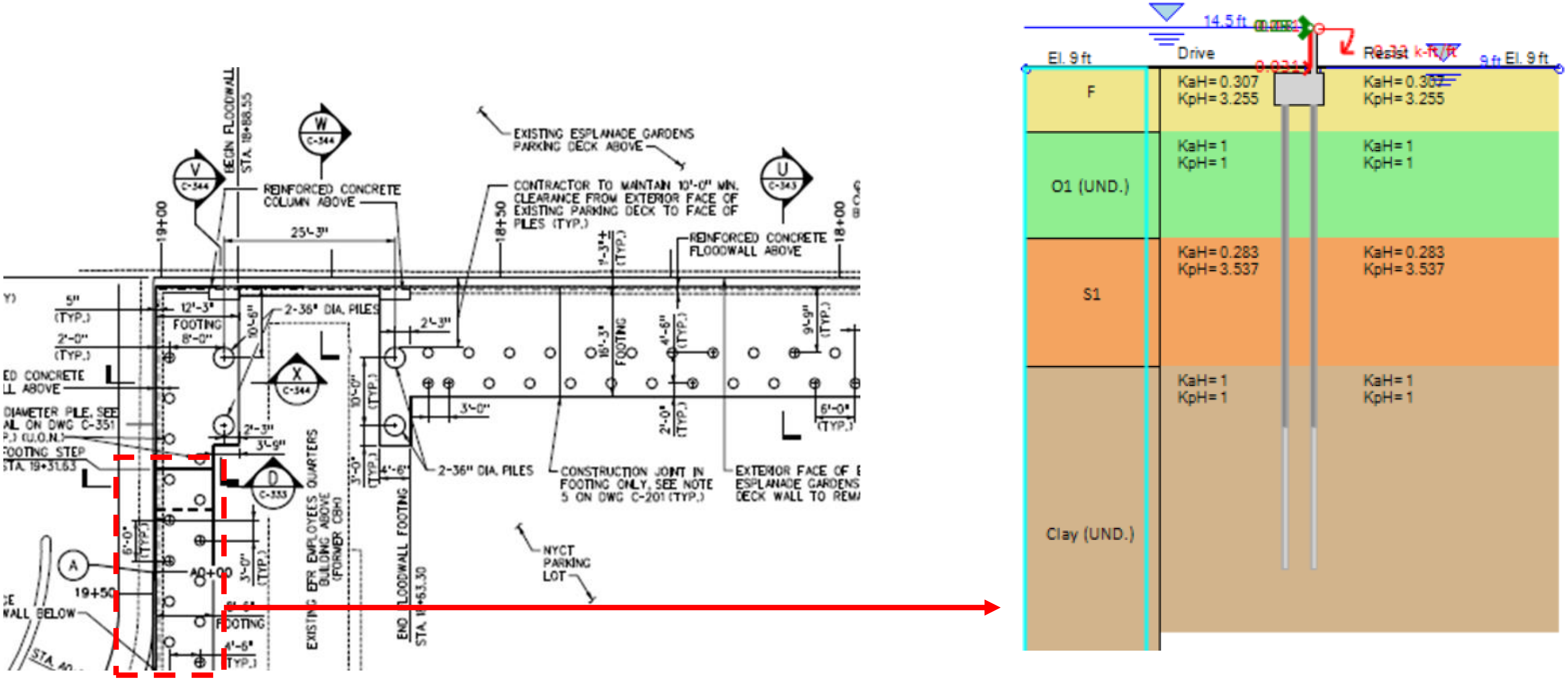
System conditions on floodwall project:





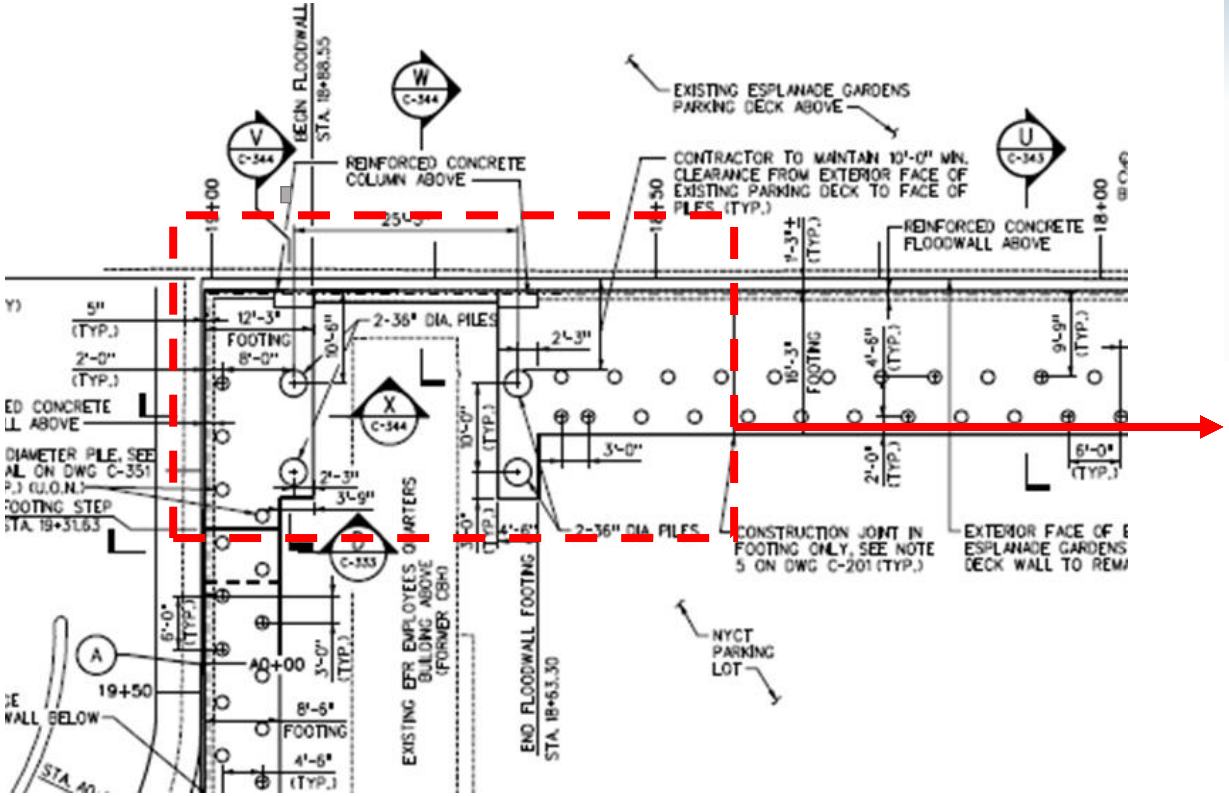
2D plane strain and 3D models

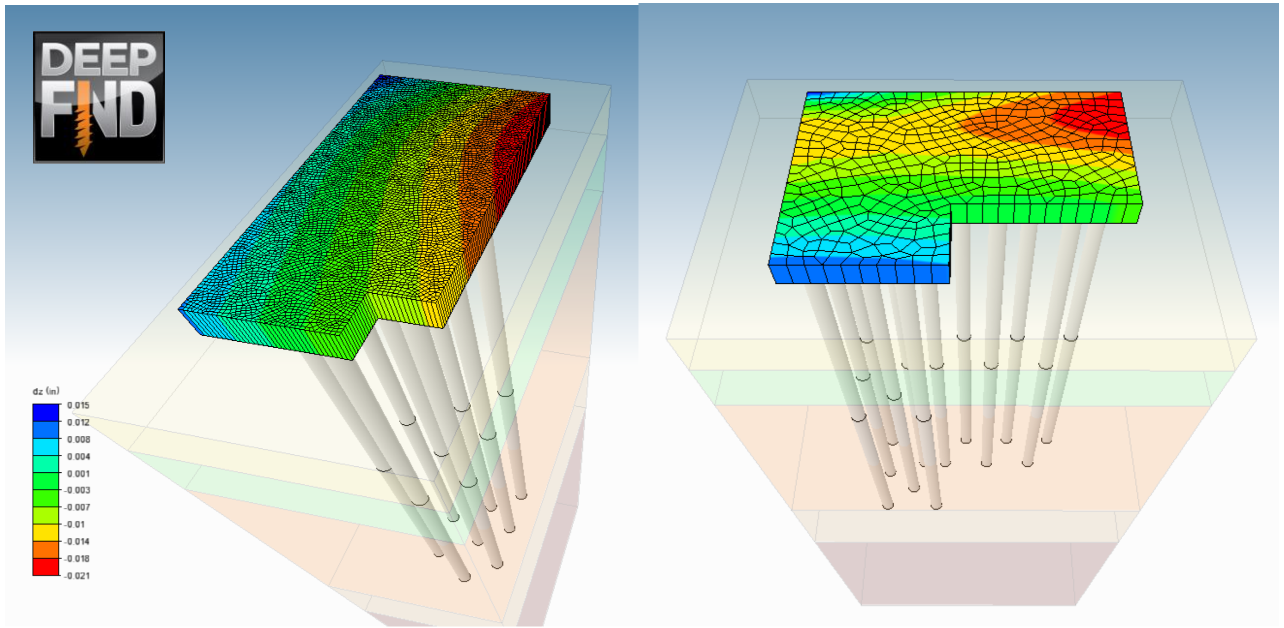
Plane strain conditions

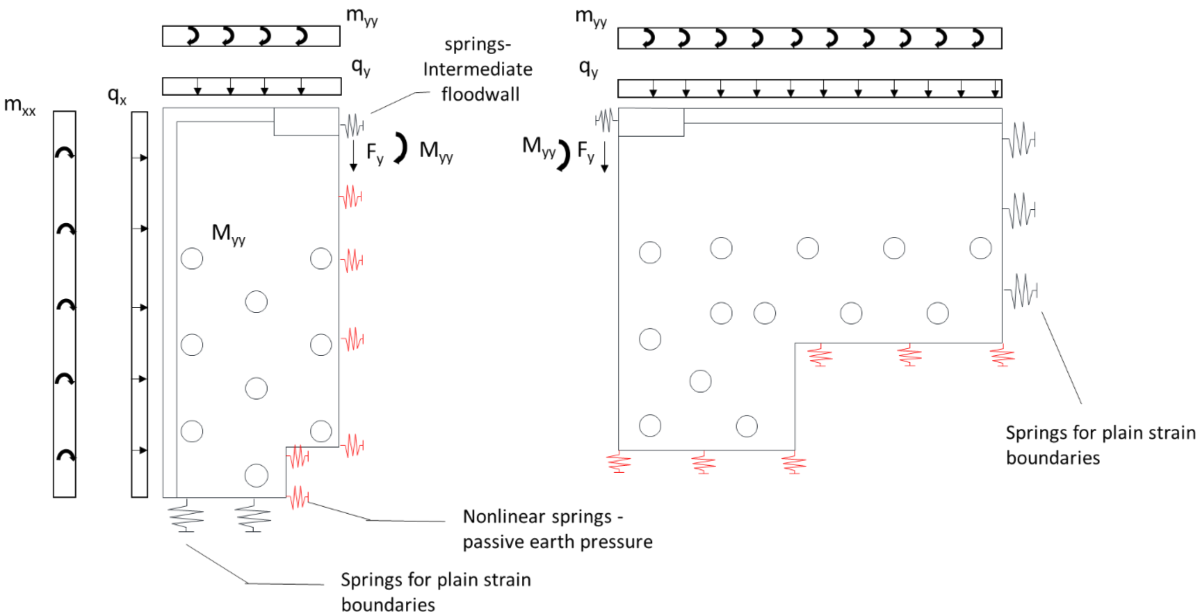


2D plane strain and 3D models

3D conditions







5. Deep Excavation FEM Cases

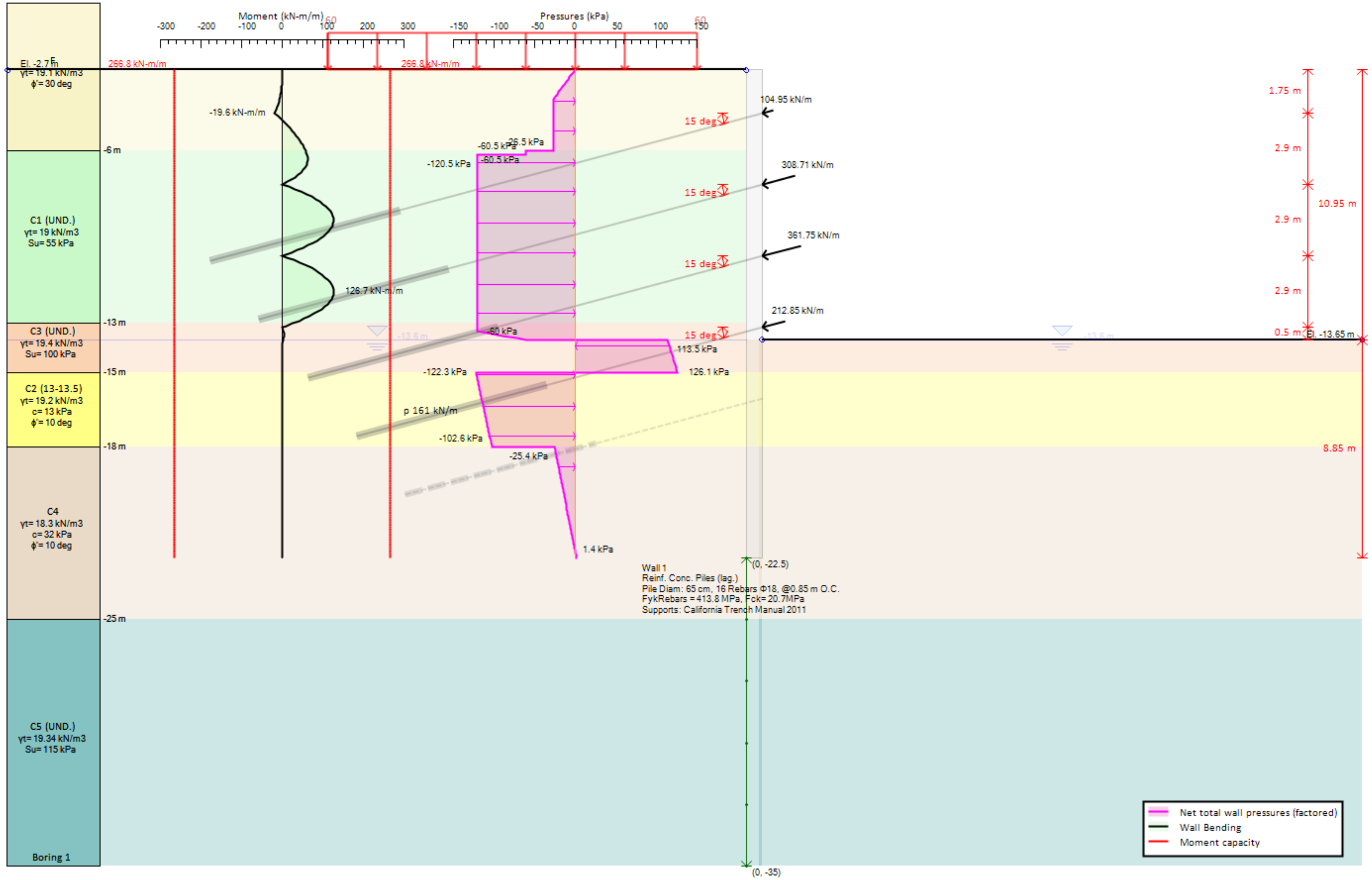
- Collapse case
- Measured deflections

5.1 Deep excavation collapse

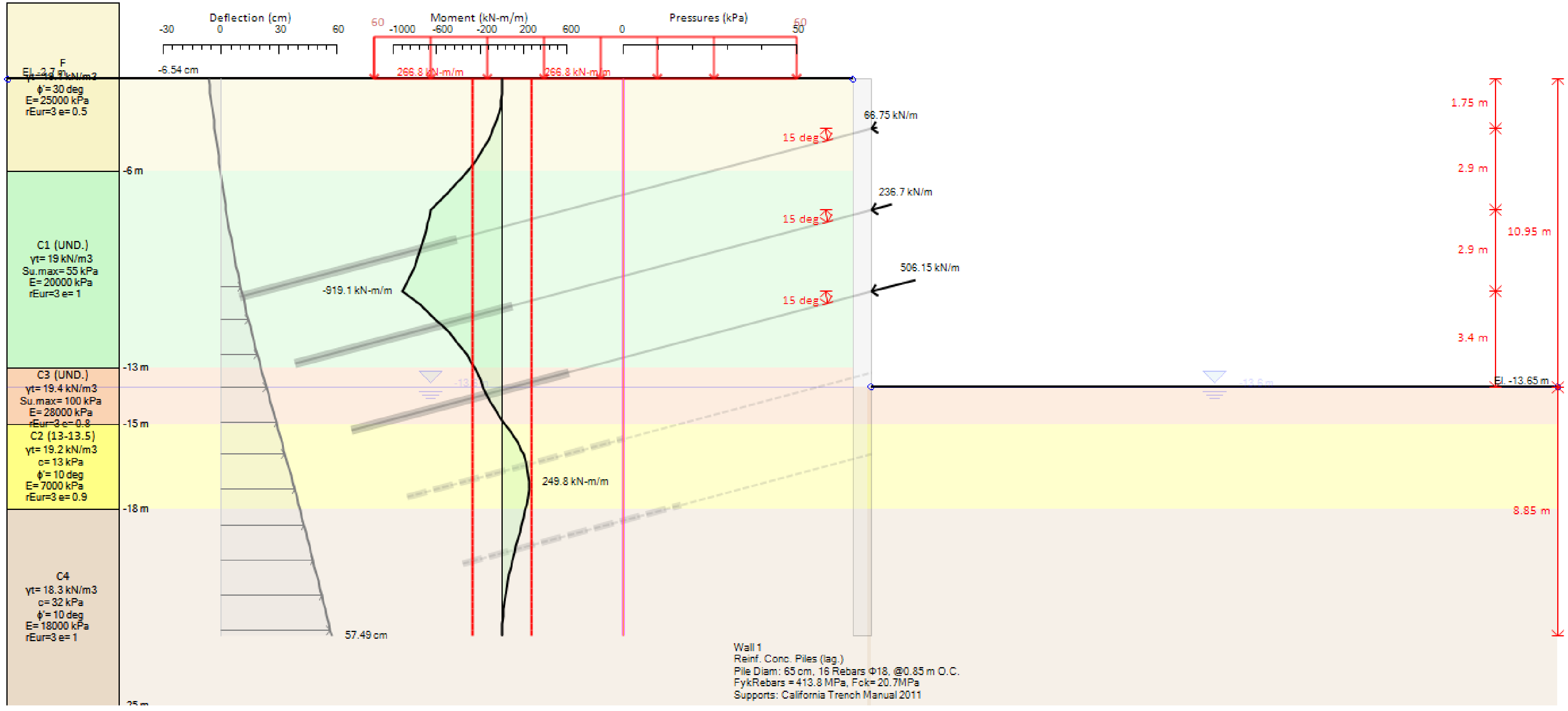
5.1.2 Deep Excavation Collapse

Mischaracterization of soil properties during investigation

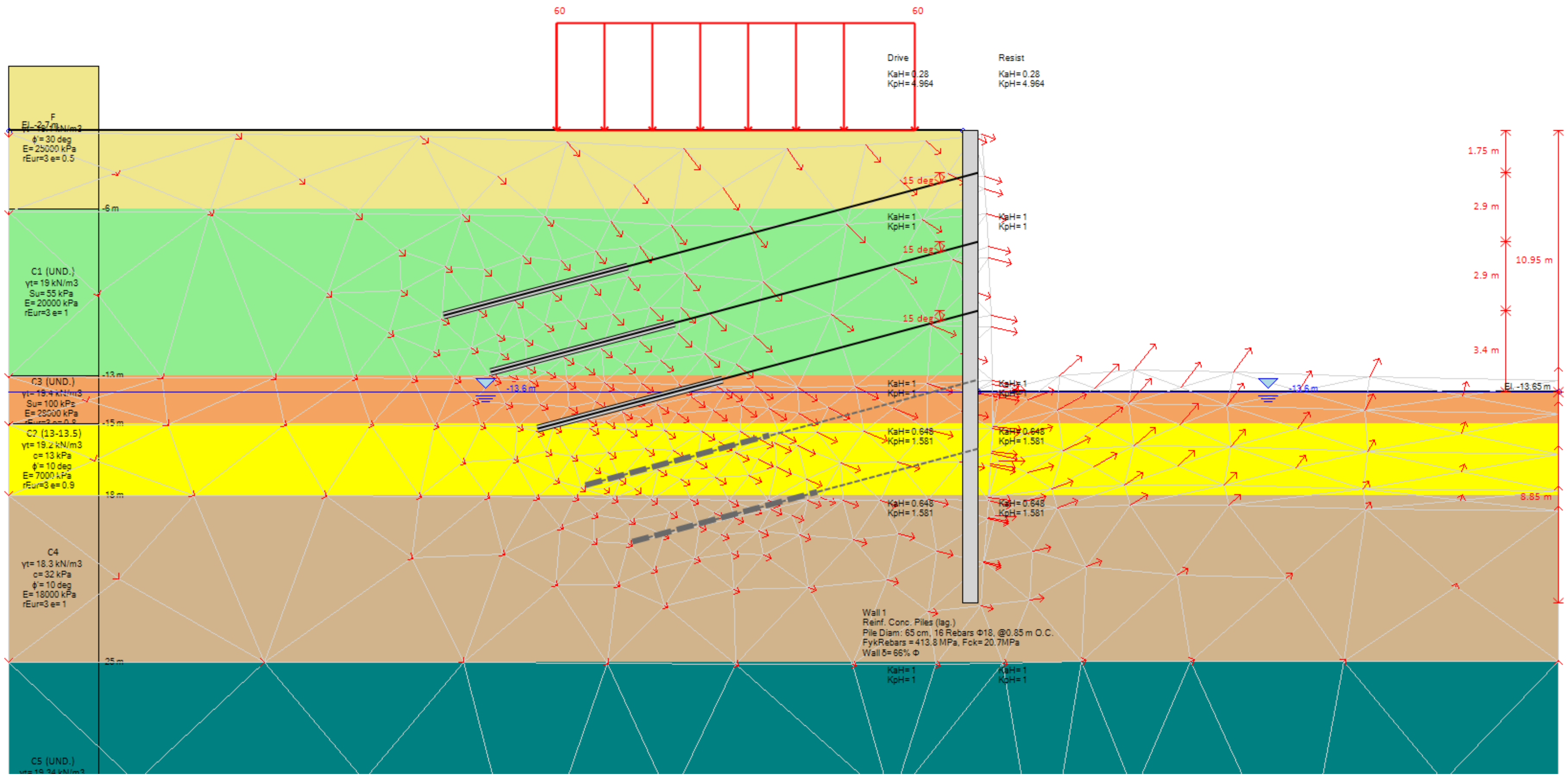
5.1.3 Conventional Analysis



5.1.4 Non-linear analysis



5.1.4 FEM Analysis

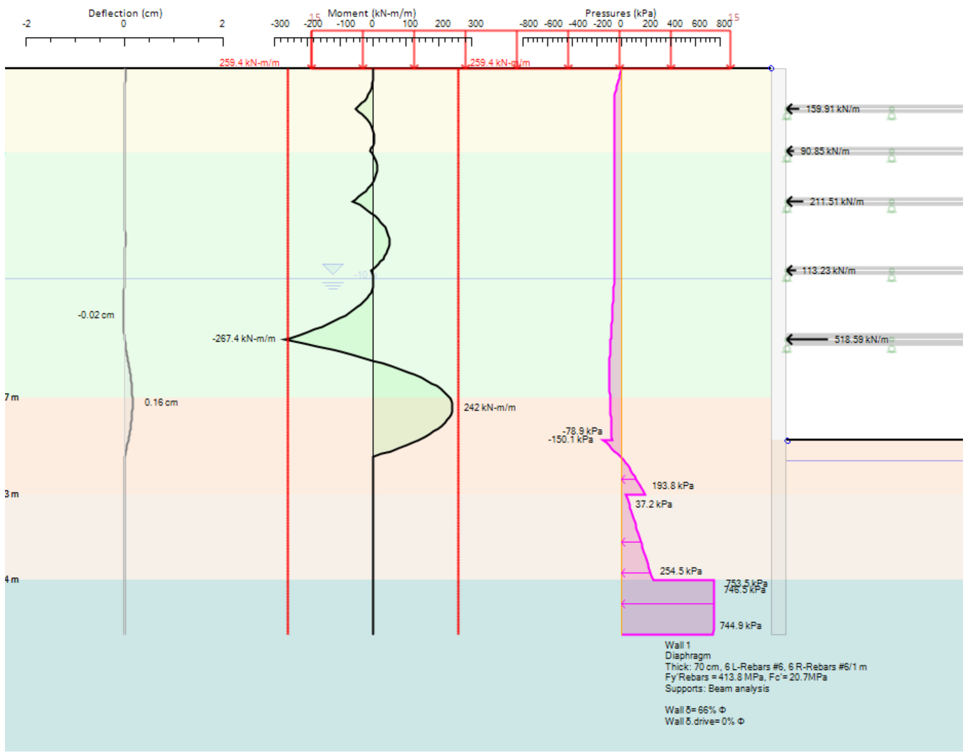


5.2.1 Excavation with LEM/NL/FEM

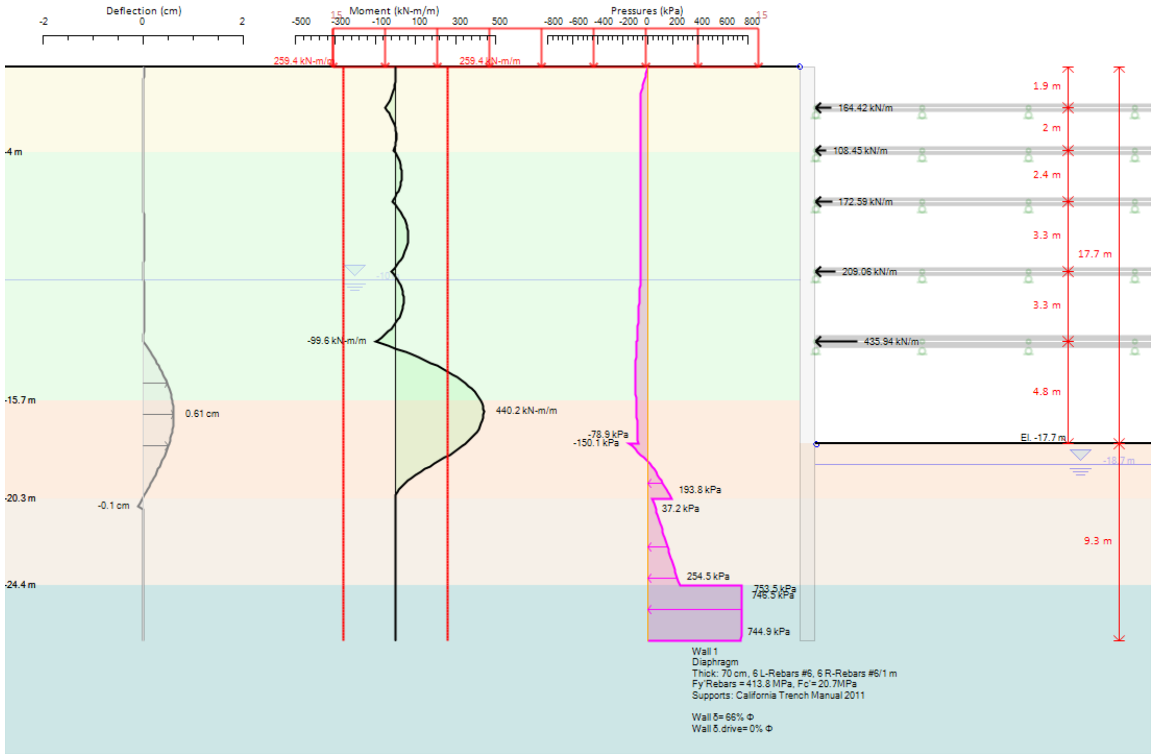
- 17.7m deep excavation in Taiwan
- Published case history
- 1cm of lateral displacement
- FEM Calibration with MC Model
- Water table at 10.0m

Ground layer description / Bottom depth of each layer	N value	γ_t kN/m^3	Total stress c kPa	Total stress ϕ °	Effective stress c' kPa	Effective stress ϕ °
1 The backfill layer gradually changed to yellowish brown clayey silt and silty sand mottled with gray including some gravel. Average thickness 4.0 m	1.5-19 (5)	19.3	* 9.8	* 21	0	30
2 The coarse gravel layer comprised yellowish brown coarse, medium, and fine silty sand. Average thickness 11.7 m	15 to > 50 (40)	21.6	—	—	* 4.9	* 38
3 The gravel layer comprised yellowish brown and gray coarse, medium, and fine silty sand. Average thickness 4.6 m	20-58 (35)	21.1	—	—	* 0	* 38
4 The composition of this layer was gray sand and clayey silt containing thin layers of sand and clay. Average thickness 4.1 m	12-26 (16)	19.4	9.8	24	0	32
5 The coarse gravel layer comprised yellowish brown coarse, medium, and fine silty sand. Average thickness 10.0 m, hole bottom	> 50 (50)	* 22.1	—	—	* 9.8	* 40

5.2.2 LEM Methods

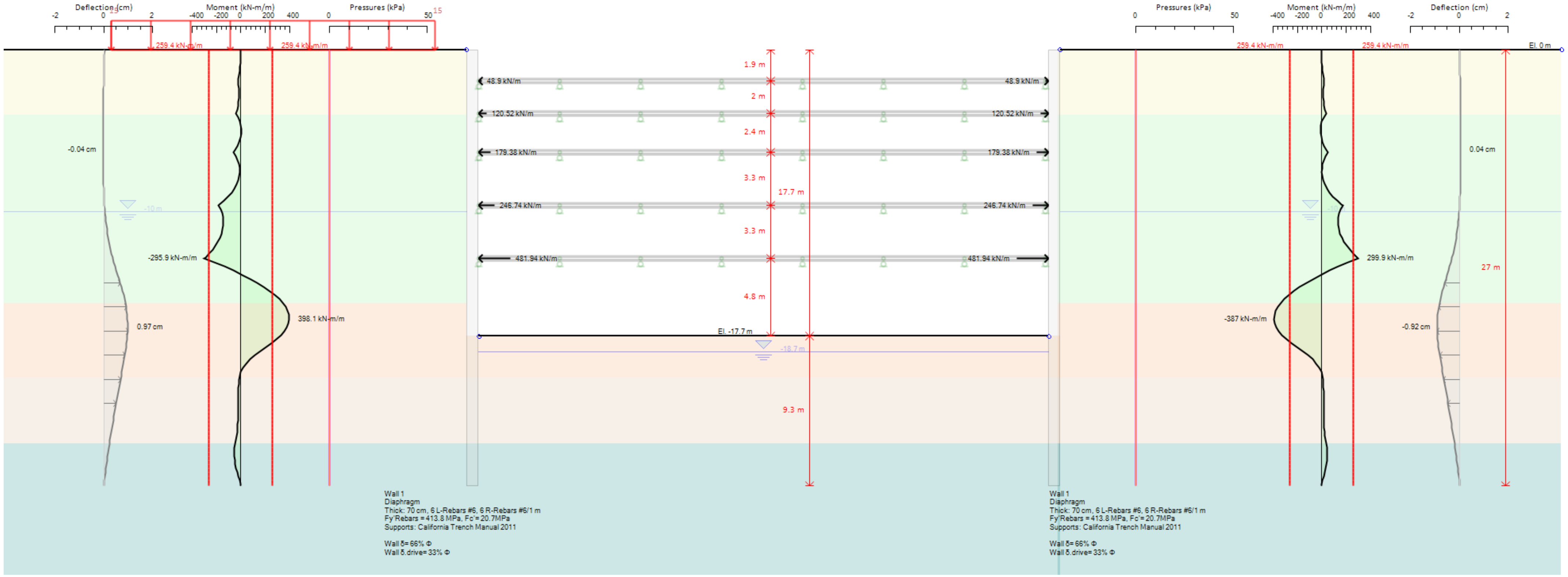


FHWA-Blum's

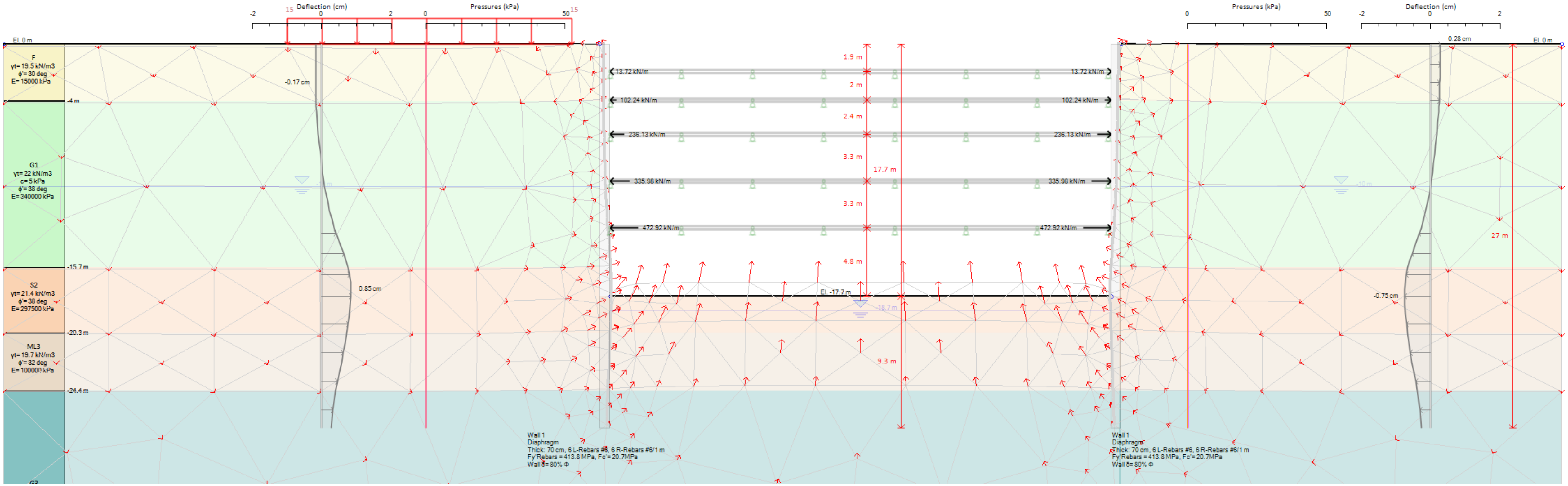


FHWA-CALTRANS

5.2.3 NL Analysis

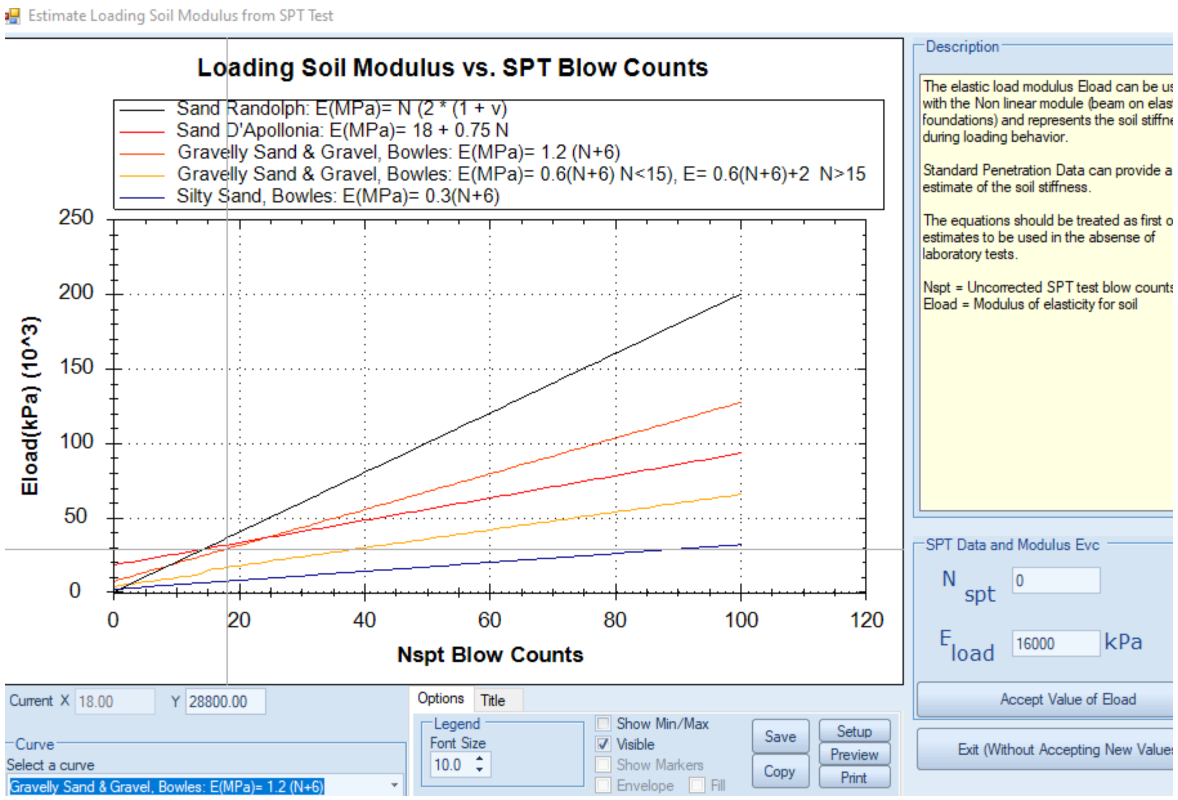


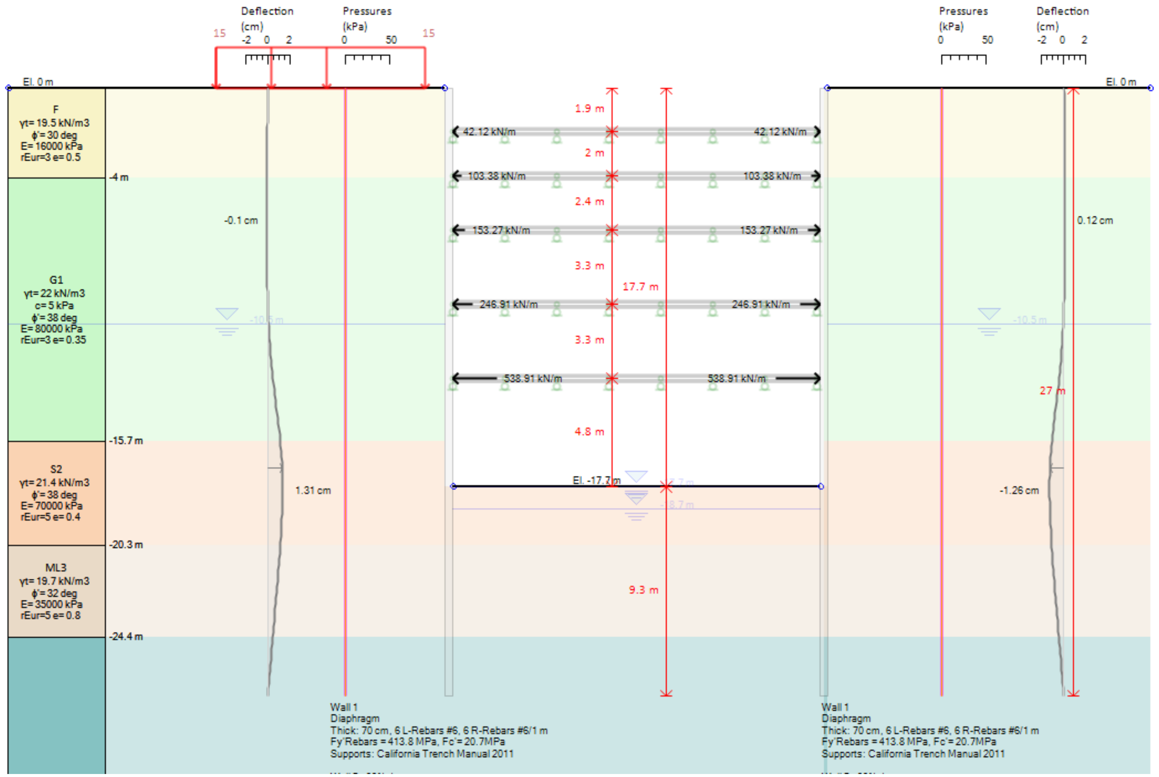
5.2.4 FEM with MC Model



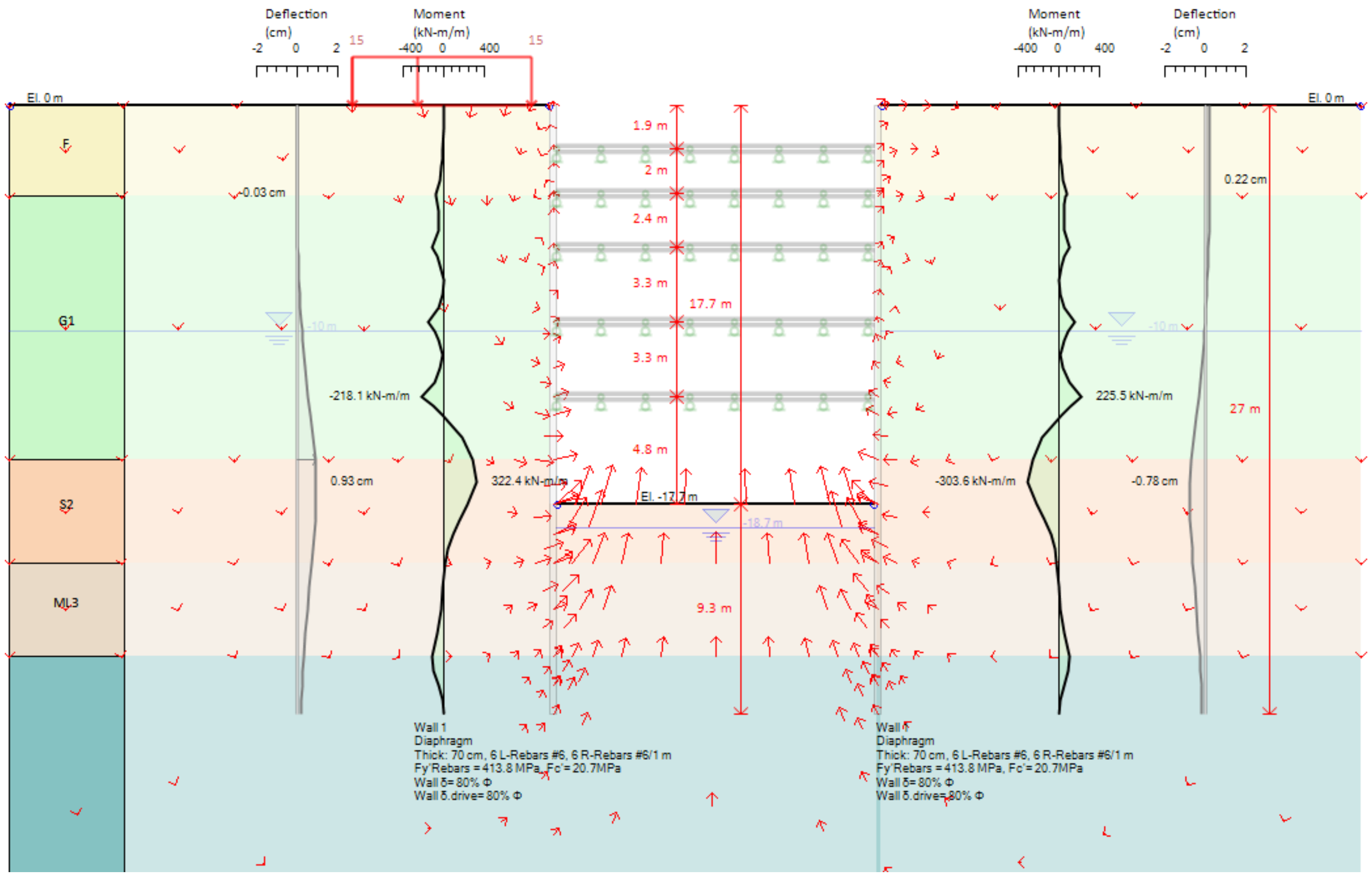
5.2.5 NL Model with EXP Soil Models

- Use SPT Estimate for wall displacement
- Randolph sand (least conservative Eq. most adequate)

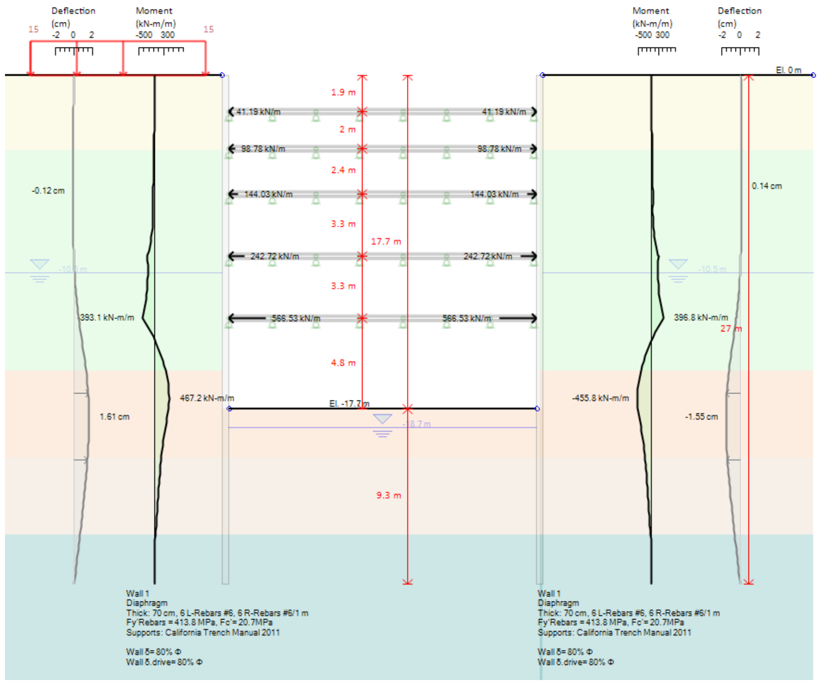


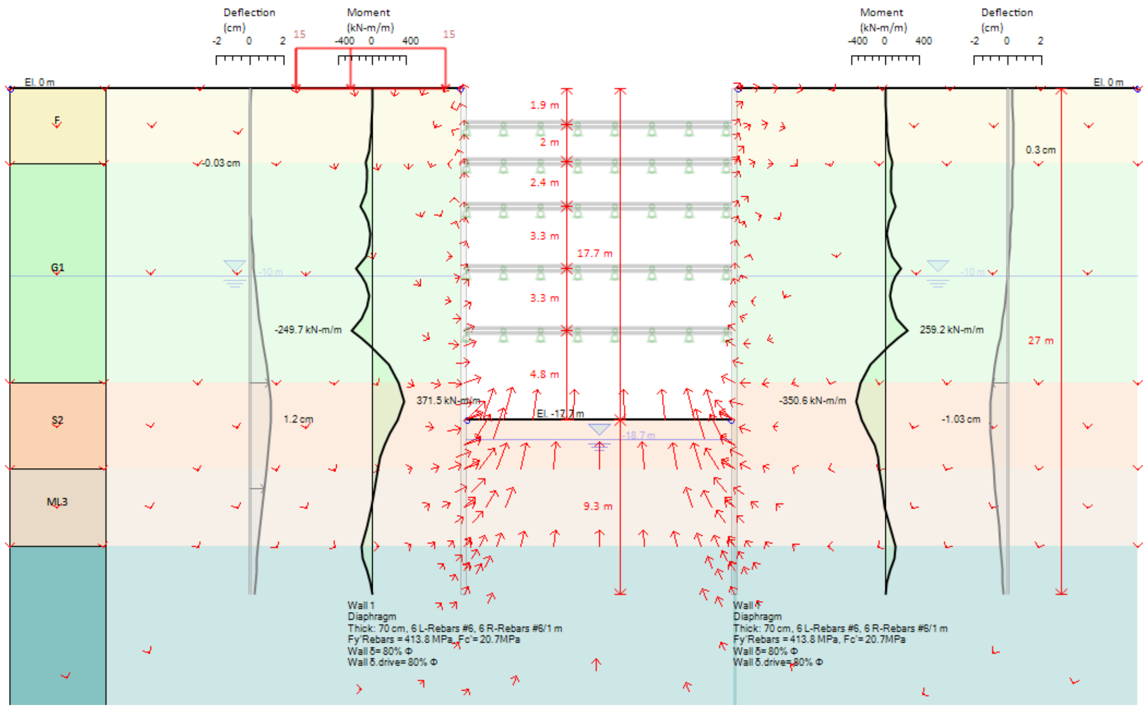


5.2.6 FEM Model with HS Randolph E Estimate

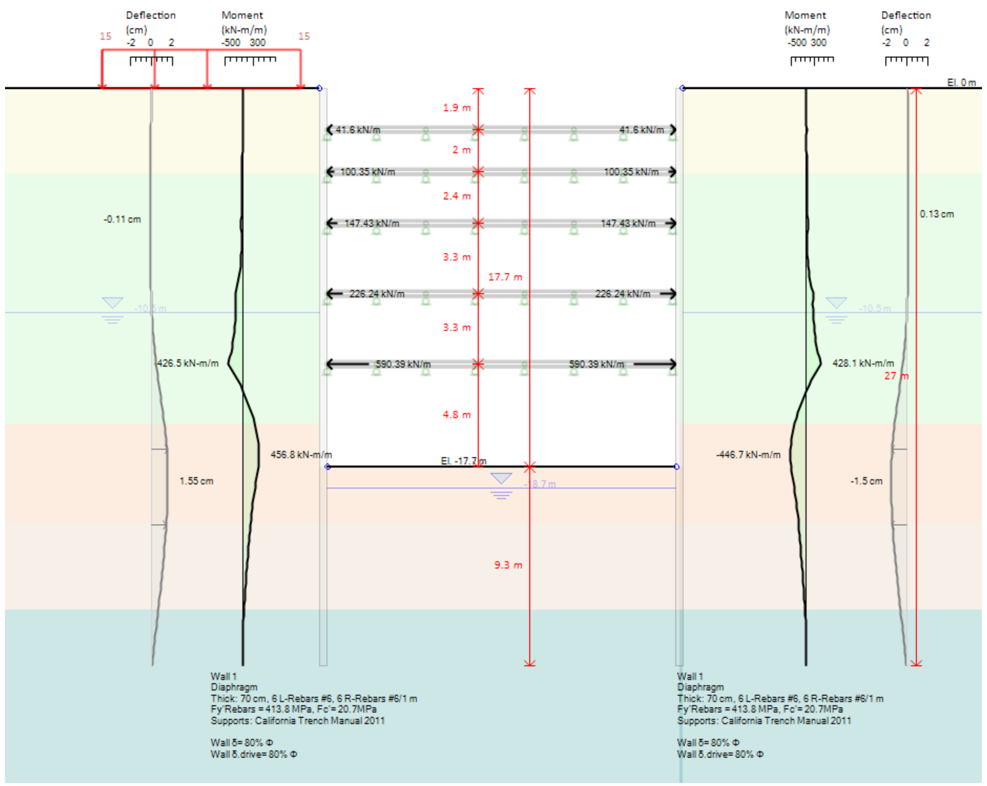


5.2.6 FEM and NL Model with Bowles Gravel E

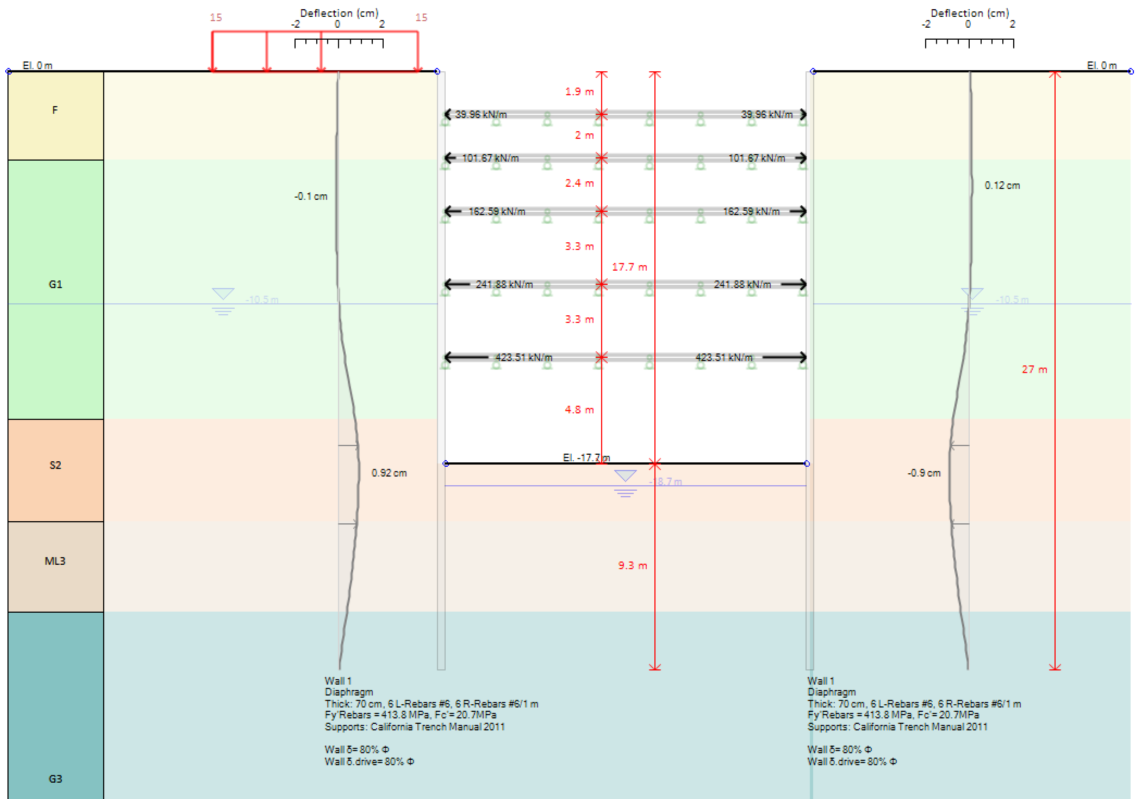




5.27 Influence of Water Modeling in NL Analysis Flownet vs. 1D



Flownet (2D)



Simplified flow (1D)

6. Marine Issues - Corrosion

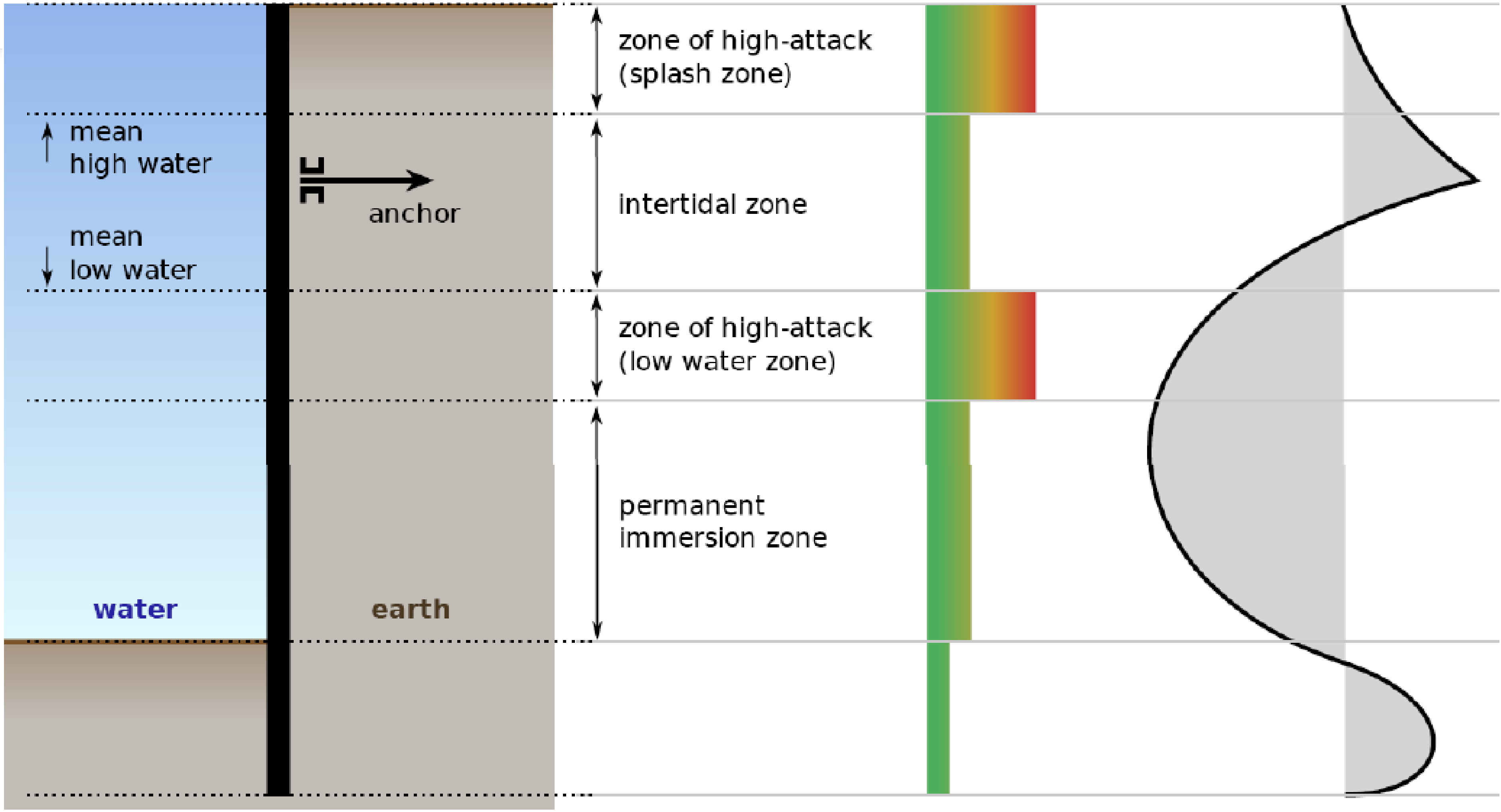
Courtesy of JKMC UK
info@jkmc.co.uk

On the bright side it is providing water pressure relief on the backside

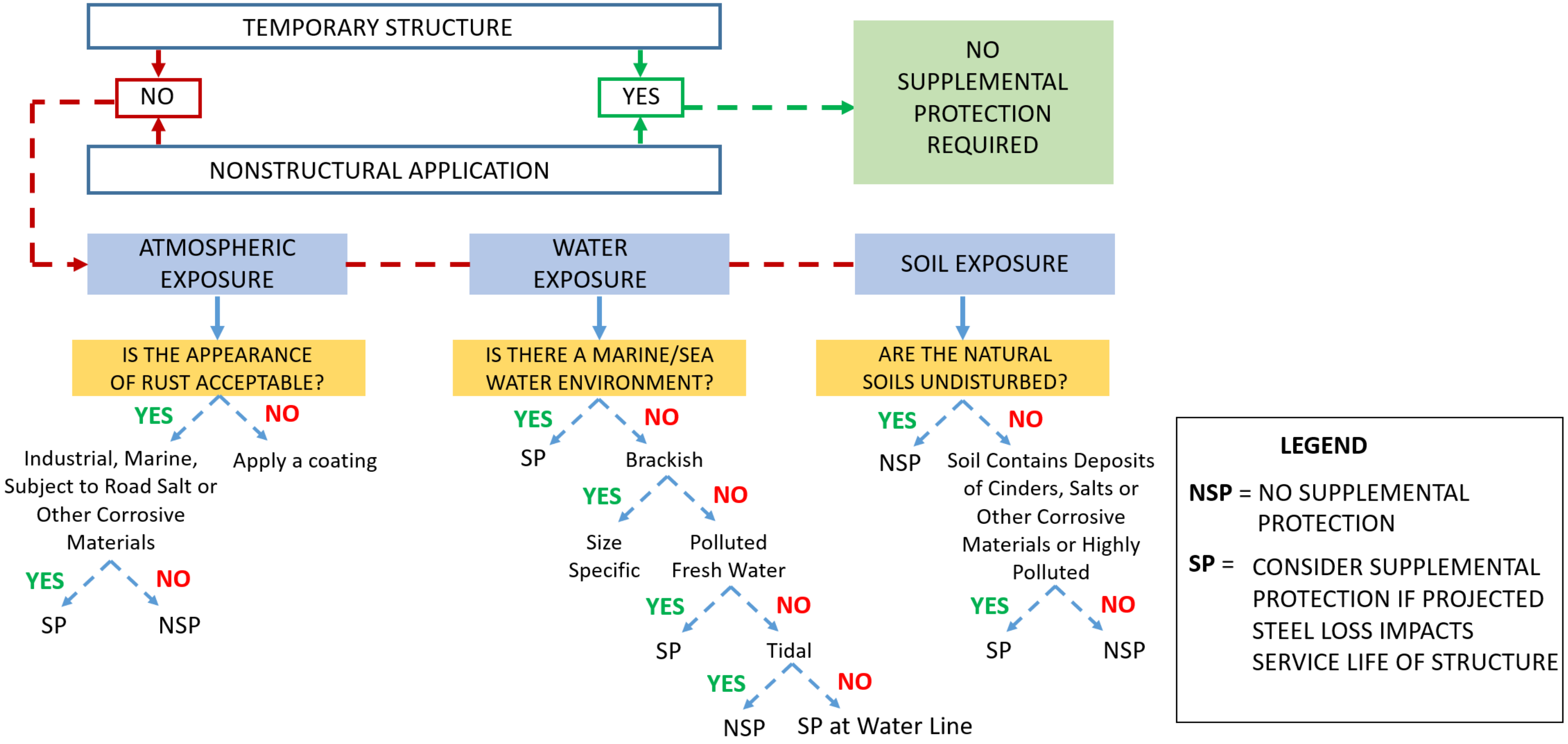
6.1.1 Steel sheet piling corrosion

- Sacrificial thickness
- Marine steel grade ASTM A690
- Higher yield steel to extend life
- Protective coatings
- Cathodic protection

6.1.2 Corrosion Zones



6.1.3 Decision Tree for Corrosion Protection



Eurocode 3
Design of Steel Structures

Part 5: Piling (ENV 1993-5)

Loss of thickness (mm)

TABLE 1. Loss of Thickness Due to Corrosion for Steel Sheet Pilings (Ref 4) [A]

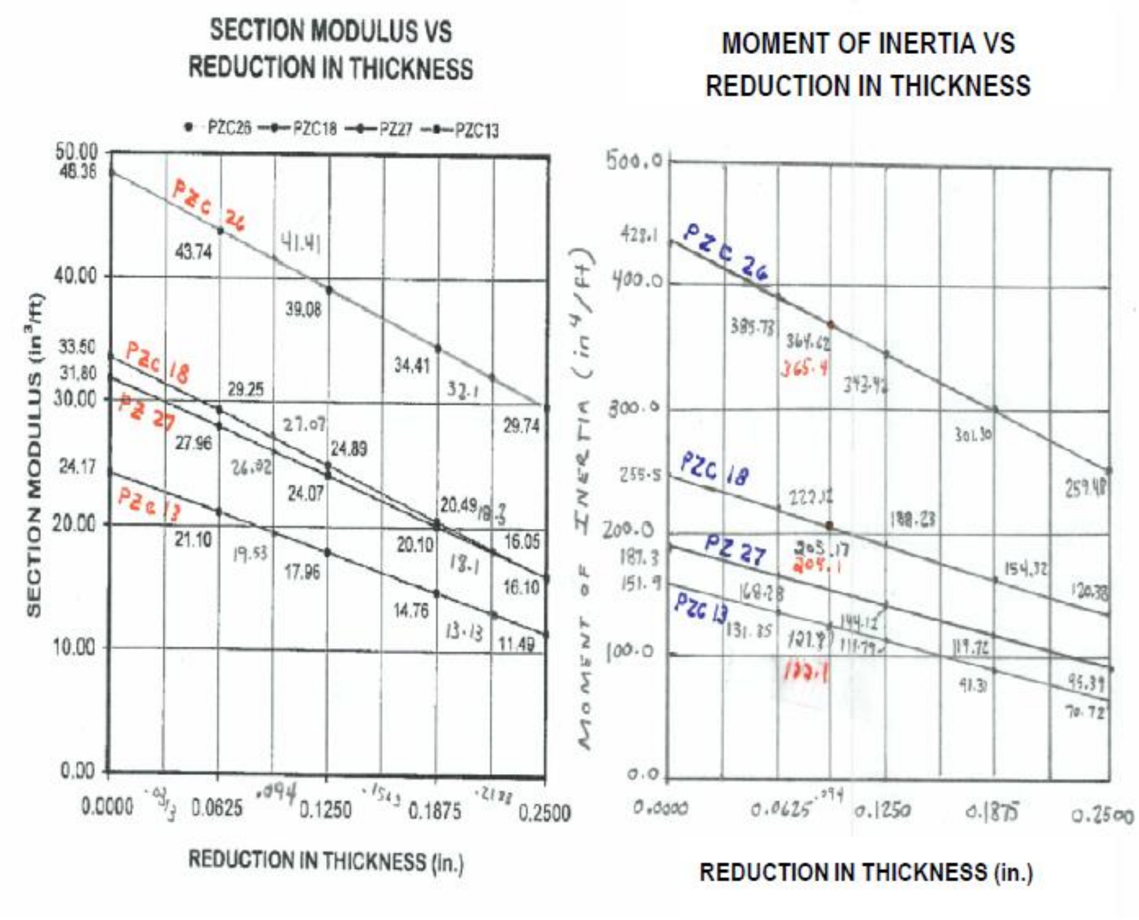
Soil, with or without groundwater:	DESIGN LIFE:				
	5 years	25 years	50 years	75 years	100 years
Undisturbed natural soils	0.00 mm	0.30 mm	0.60 mm	0.90 mm	1.20 mm
Polluted natural soils and industrial grounds	0.15 mm	0.75 mm	1.50 mm	2.25 mm	3.00 mm
Aggressive natural soils (swamp, marsh, peat...)	0.20 mm	1.00 mm	1.75 mm	2.50 mm	3.25 mm
Non-compacted and non-aggressive fills [B] (clay, schist, sand, silt...)	0.18 mm	0.70 mm	1.20 mm	1.70 mm	2.20 mm
Non-compacted and aggressive fills [B] (ashes, slag...)	0.50 mm	2.00 mm	3.25 mm	4.50 mm	5.75 mm
Water[C]:					
Common fresh water (river, ship canal-,...) in the zone of high attack (water line)	0.15 mm	0.55 mm	0.90 mm	1.15 mm	1.40 mm
Very polluted fresh water (sewage, industrial effluent-,...) in the zone of high attack (water line)	0.30 mm	1.30 mm	2.30 mm	3.30 mm	4.30 mm
Sea water in temperate climate in the zone of high attack (low water and splash zones)	0.55 mm	1.90 mm	3.75 mm	5.60 mm	7.50 mm
Sea water in temperate climate in the submerged zone or tidal zone	0.25 mm	0.90 mm	1.75 mm	2.60 mm	3.50 mm

A. Values are provided for general guidance only. Local knowledge may lead to the use of other values for design. The values given for 5 and 25 years are based on measurements, whereas other values are extrapolated.

B. In compacted fills, these corrosion losses should be divided by two.

C. The highest corrosion rate is usually found at the splash zone of marine environments or at the low water level in tidal waters. However in most cases, the highest bending stresses occur in the submerged zone.

6.1.4 Section modulus vs. reduction in thickness



6.1.5. Sheet pile design life

- EC, USACE consider end of design life when any part of the pile reaches a maximum permissible stress through corrosion loss
- Many state DOT’s (Florida, etc), require design life at 85% of full section values (note that could be area or section modulus)
- Latest USACE (2008) piling sections thinner than 0.250 inches restricted to uses with low bending, low corrosion, and low interlocked joint strength in tension.

6.1.6 Mariner Grade Steel ASTM A690

- ASTM A 690/A 690M - 07

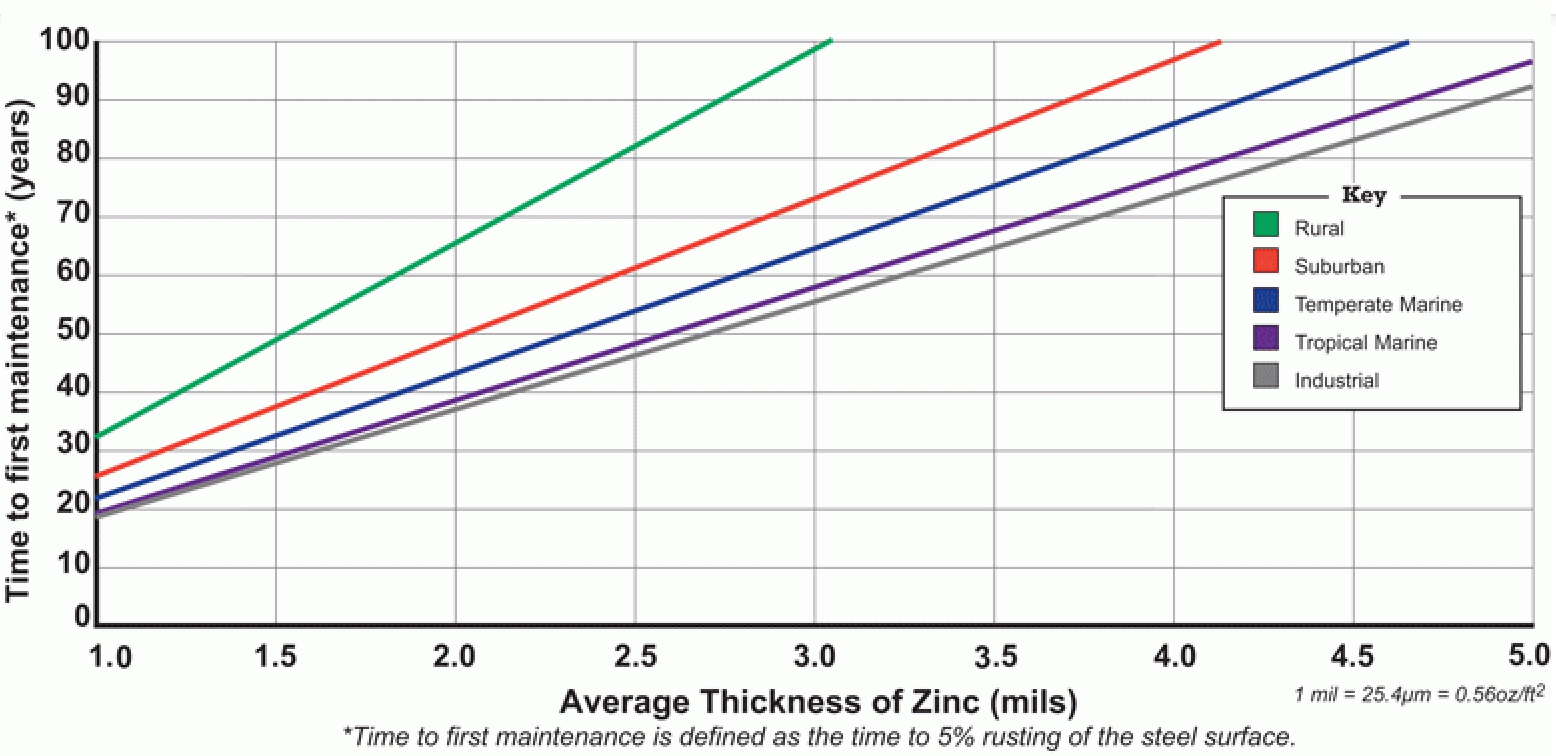
1.2 The atmospheric corrosion resistance of this steel is substantially better than that of ordinary carbon steels with or without copper addition (see Note 1). The steel has also shown to have substantially greater resistance to seawater "Splash Zone" corrosion than ordinary carbon steel (Specifications A 36/A 36M and A 328/A 328M) where exposed to the washing action of rain and the drying action of the wind or sun, or both. Where the steel is not boldly exposed, the usual provisions for the protection of ordinary carbon steel should be considered.

- ASTM stated, A690 exhibited 2 to 3 times more resistance in the splash zone...

6.1.7 Galvanized Steel

- Based on AGA (American Galvanizers Association) Tropical Marine exposure, a galvanized system will have a projected life (to 5% of surface rust) in excess of 75 years.
- Galvanized coating life determined by thickness and severity of exposure conditions.
- Typical protection 3 to 7 mils.
- Expected service life defined when 5% rusting of the steel substrate occurs. At 5% surface rust, no steel integrity lost; time to consider applying new corrosion protection.

6.1.8 Hot-dip galvanized steel service-life chart



6.2 Cathodic protection

- Required if design life cannot be achieved by other methods
- If used with a coating system, Cathodic protection becomes effective when the coating life ends.
- Types:
  - Impressed current
  - Sacrificial anode
- Continuous as long as system is maintained – 5 year typical system review (base metal anode replacement, electrical continuity upgrade at 20 years)

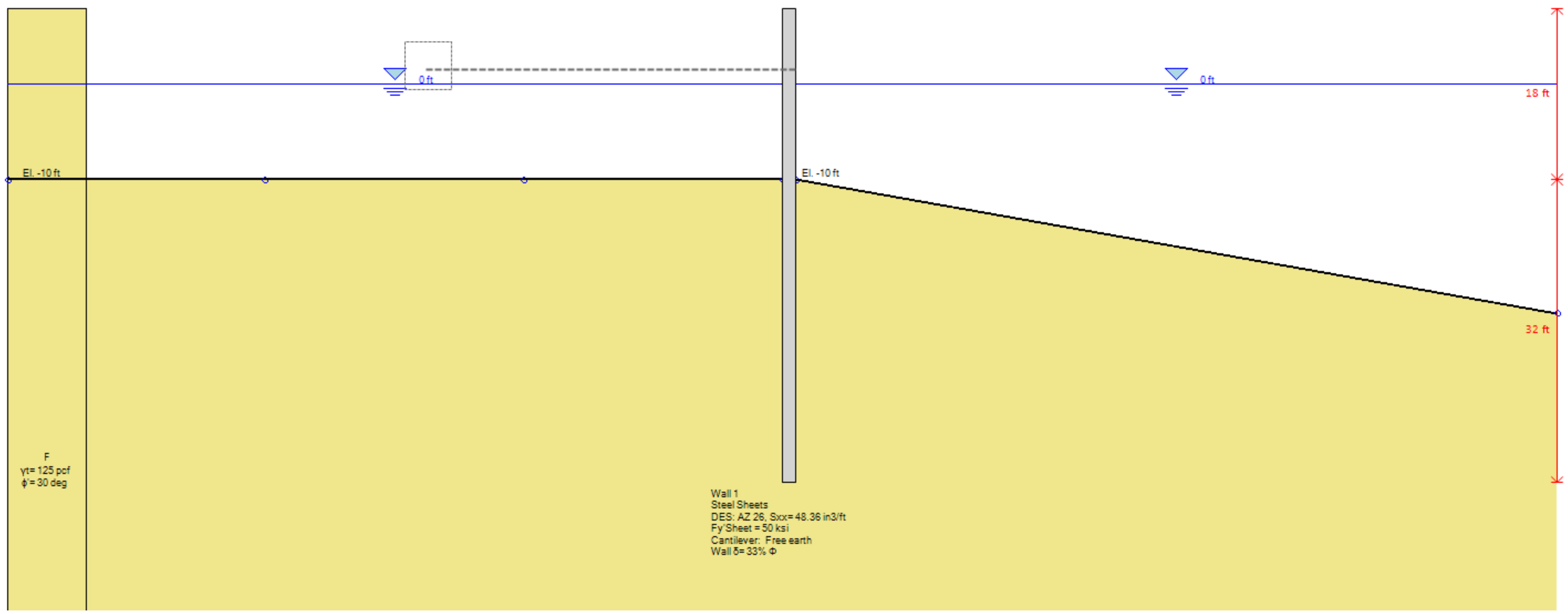
6.3 Indicative design life cost comparison

Option	Describe	AZ13	Life	AZ18	life
1	Base section	$8.0/ft^2	32 years	$9.0/ft^2	41 years
2	Marine grade A690	+$0.75/ft^2	+32 years	+$0.85/ft^2	+41 years
3	Higher grade steel	+$0.22/ft^2	+6 years	+$0.24/ft^2	+ 8 years
4	Coatings (both sides)	+$2.60/ft^2	+15 years	+$2.85/ft^2	+15 years
5	Cathodic protection	+$6.25/ft^2	+20 years	+$6.25/ft^2	+20 years

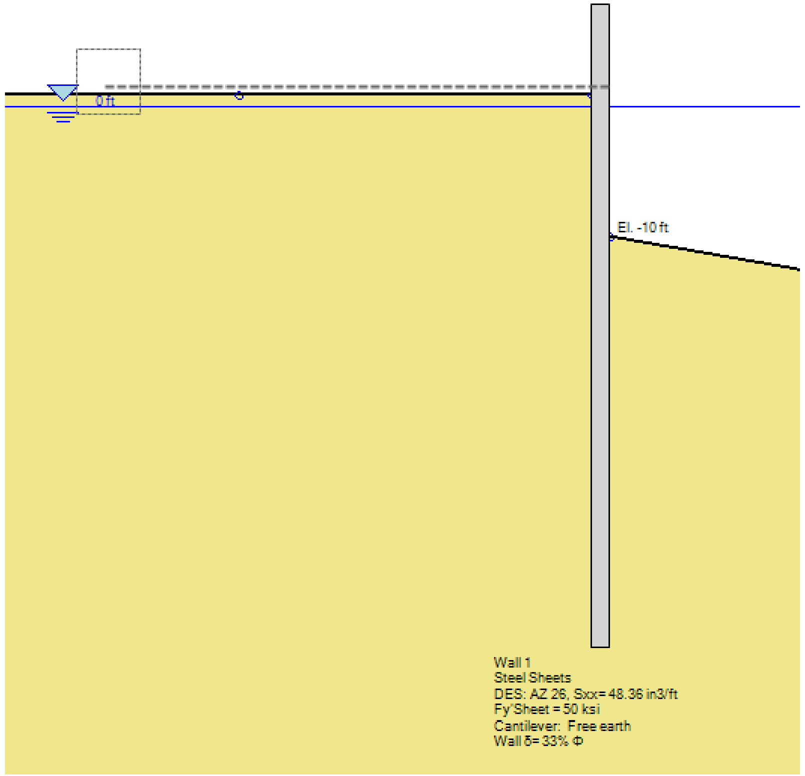
Adapted from Skyline Steel, LLC example for an 800ft x 40 ft wall without coating

6.4.1 Example

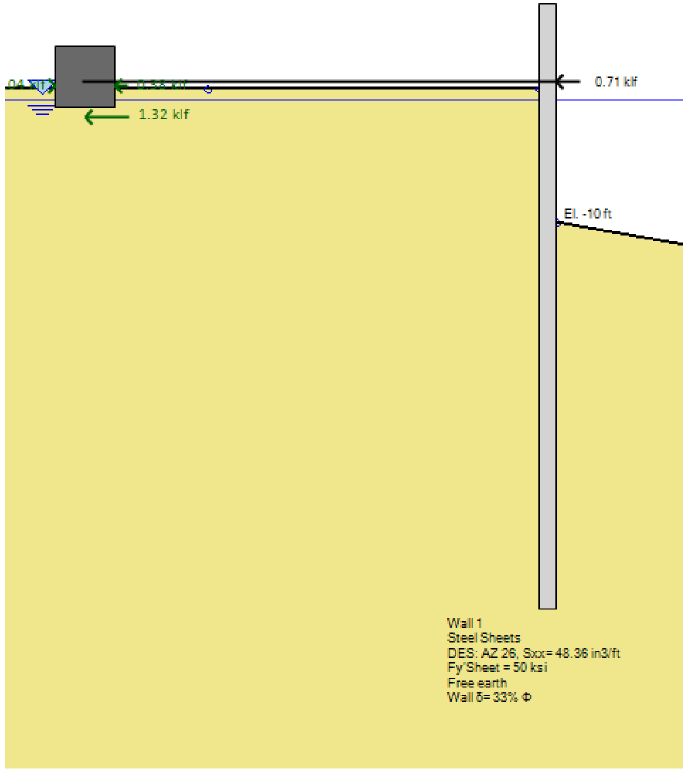
- Wall at El. +8
- Seafloor at El. -10, sloping down 10 deg
- Anchor rod at El. +1.5
- Deadman block at El. +4.5



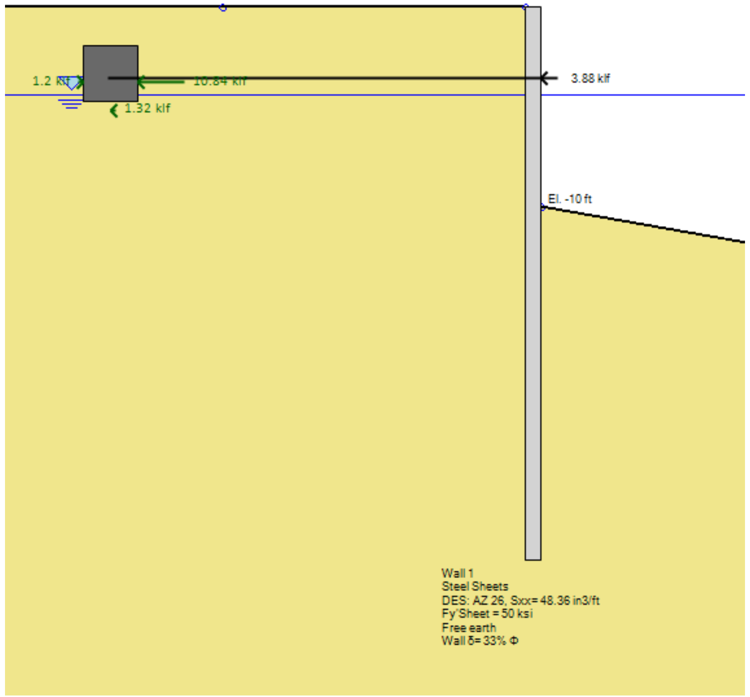
6.4.2 Staging



0 ft
El. -10 ft
Wall 1
Steel Sheets
DES: AZ 26, Sxx= 48.36 in3/ft
Fy'Sheet = 50 ksi
Cantilever: Free earth
Wall δ= 33% Φ

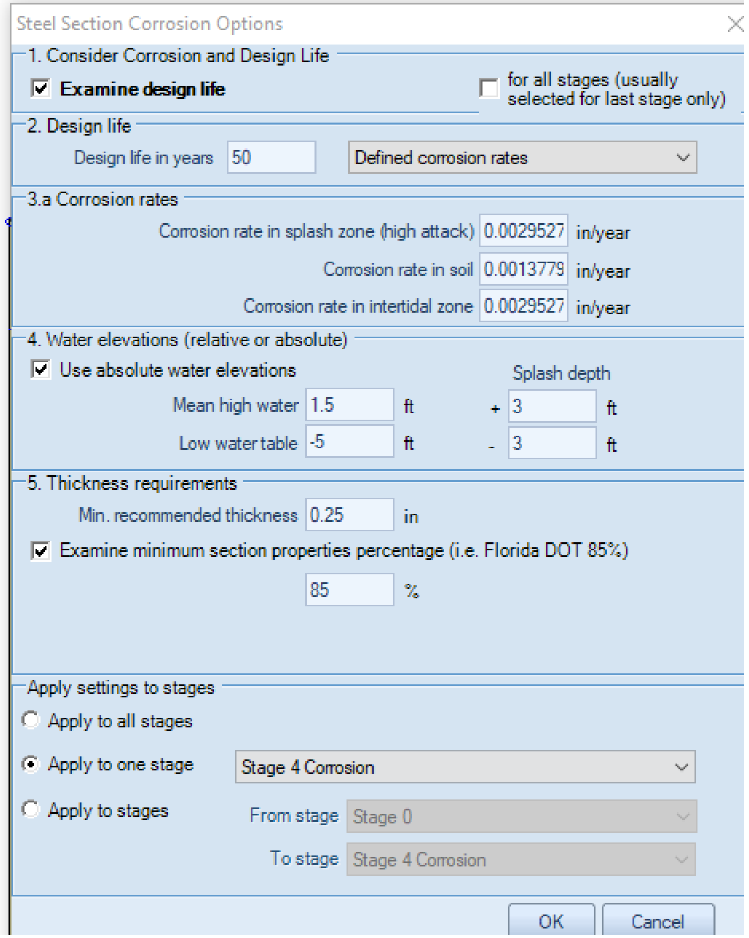


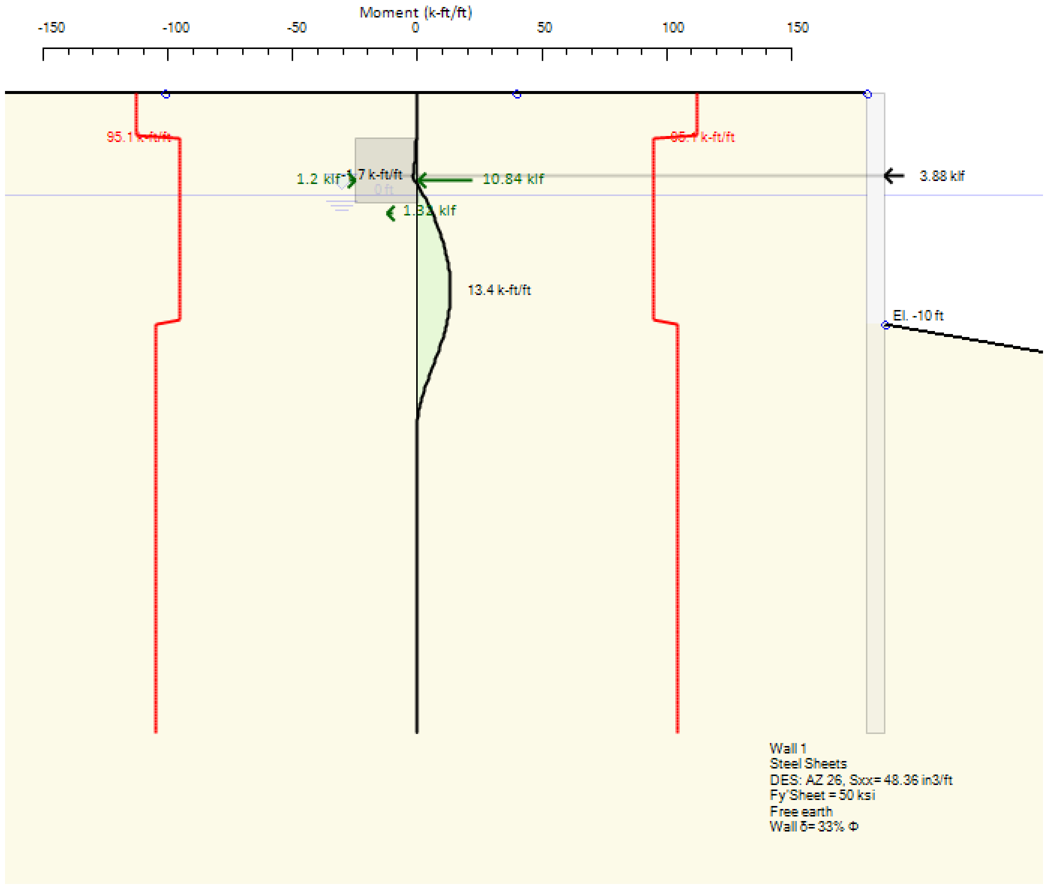
0.71 klf
1.32 klf
El. -10 ft
Wall 1
Steel Sheets
DES: AZ 26, Sxx= 48.36 in3/ft
Fy'Sheet = 50 ksi
Free earth
Wall δ= 33% Φ



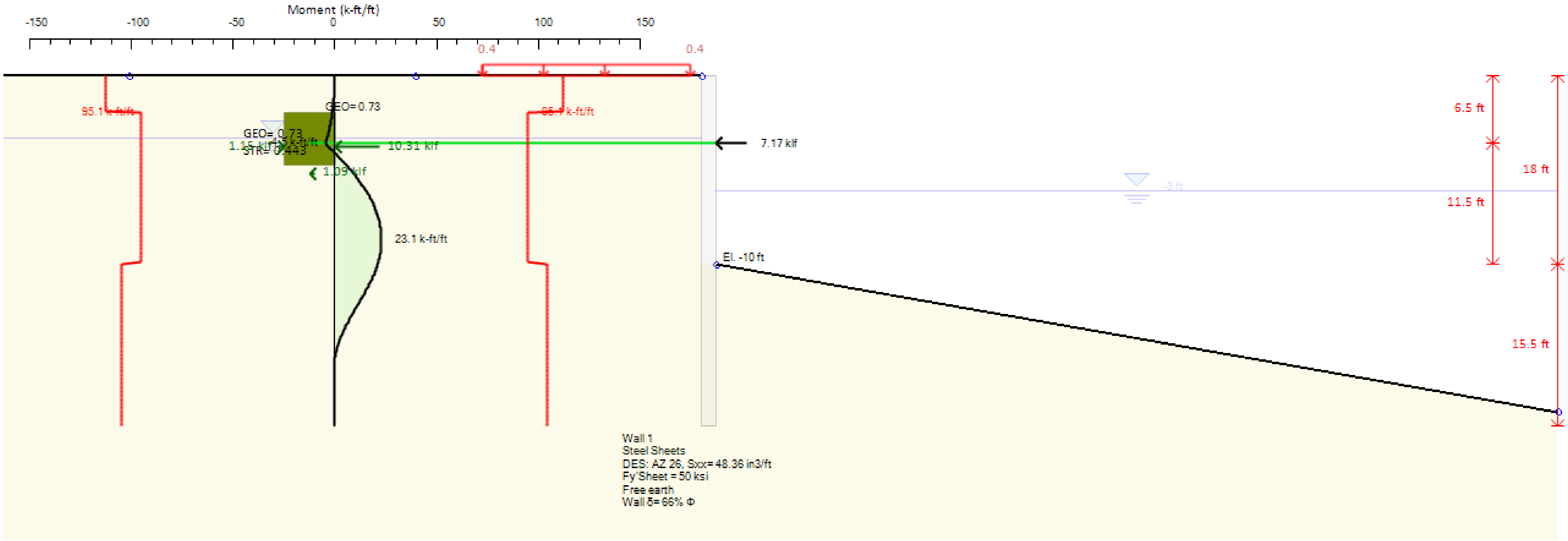
3.88 klf
1.32 klf
El. -10 ft
Wall 1
Steel Sheets
DES: AZ 26, Sxx= 48.36 in3/ft
Fy'Sheet = 50 ksi
Free earth
Wall δ= 33% Φ

6.4.3 Design Life Settings





6.4.4 Water Level Difference & Optimization

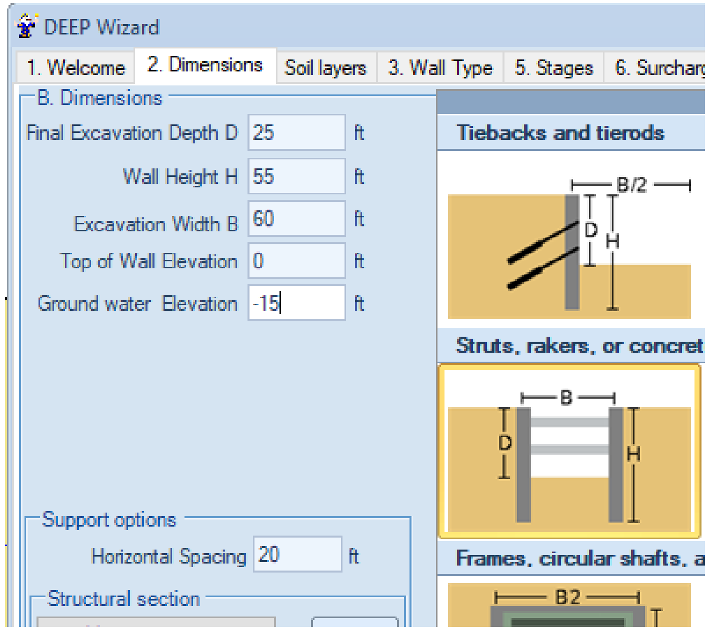


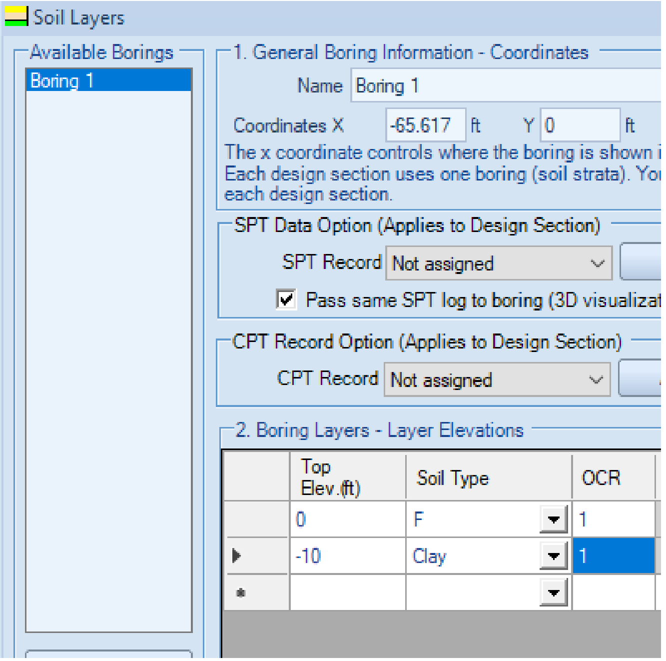
7. Advanced Frame Analysis

- Create typical soil conditions
- Import Buildings from Google Maps
- Draw wall perimeter on map
- Draw cross-sections
- Analyze
- Building damage assesment

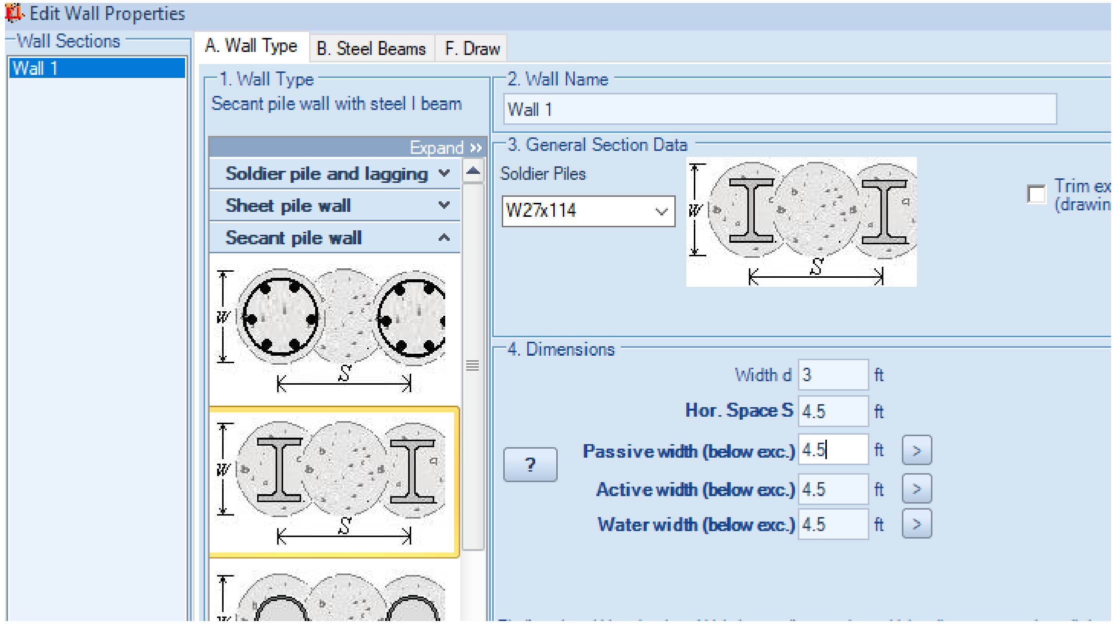
7.1.1 Set General Problem

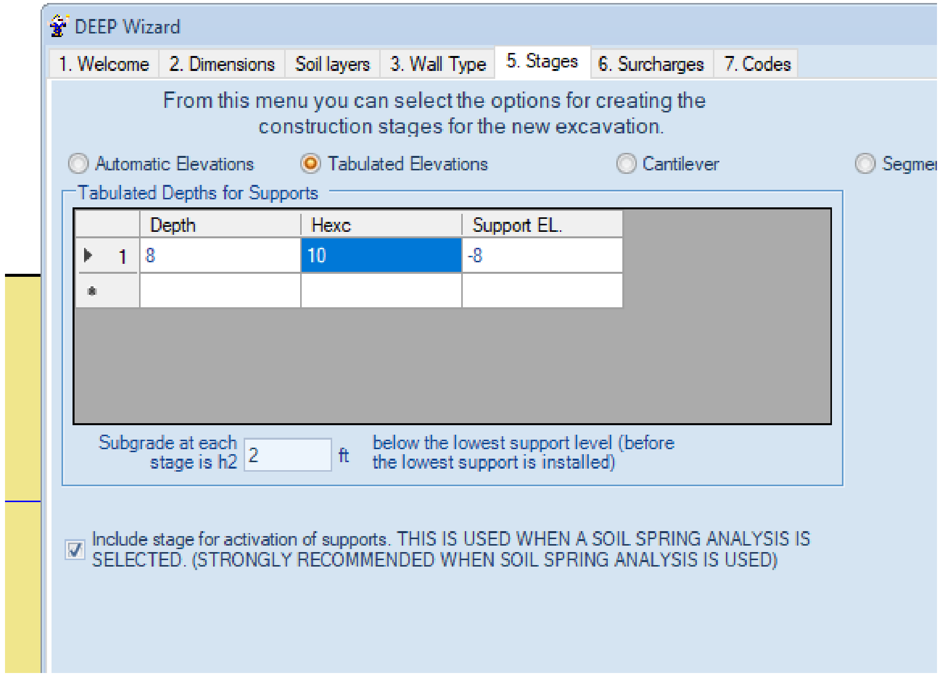
- 10ft fill, 30 deg, 125 pcf
- Clay Su= 2000 psf, 120 pfc
- Water table at -15ft
- Excavation 25ft
- One level of bracing at -8ft
- Secant pile wall W27





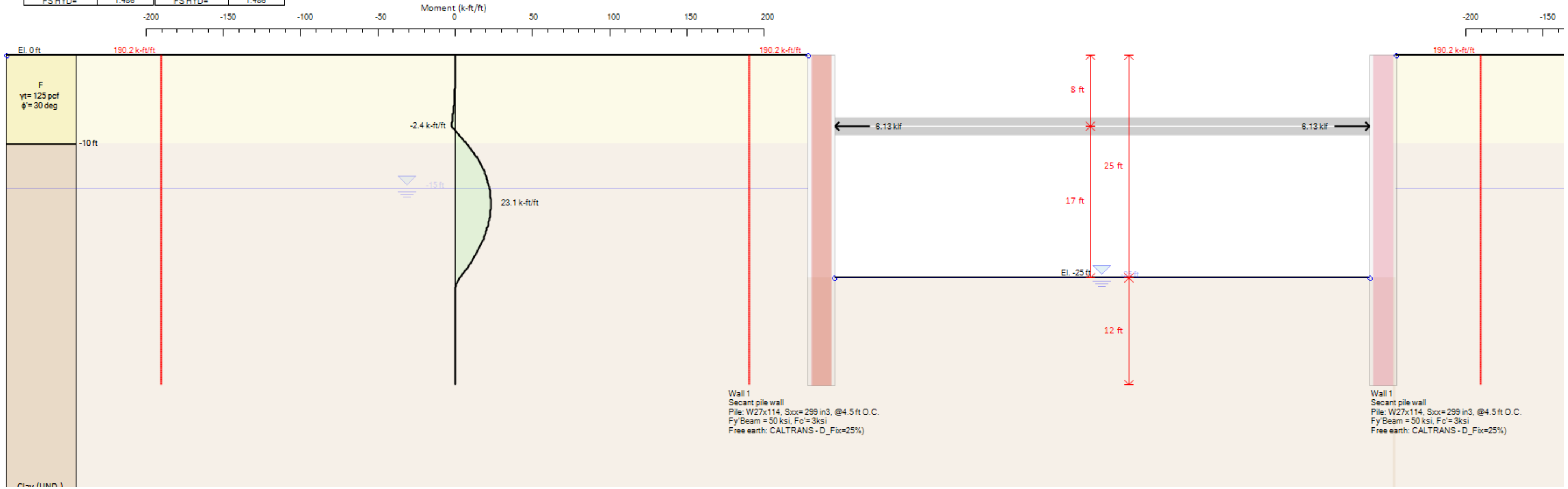
7.1.2 Setup Secant Wall & Braces





7.1.3 Preliminary Wall Optimization

Wall Toe Safety:	Wall	Wall Toe Safety:	Right wall
Min FS=	6.582	Min FS=	6.41
FS Embed=	8.901	FS Embed=	8.901
FS Rot.=	6.582	FS Rot.=	6.41
Req. toe FS=1:	1.348 ft	Req. toe FS=1:	1.348 ft
Toe El. FS=1:	-26.348 ft	Toe El. FS=1:	-26.348 ft
Note:	Toe FS >=1	Note:	Toe FS >=1
Basal standard	3.8	Basal standard	3.8
FS HYD=	1.486	FS HYD=	1.486



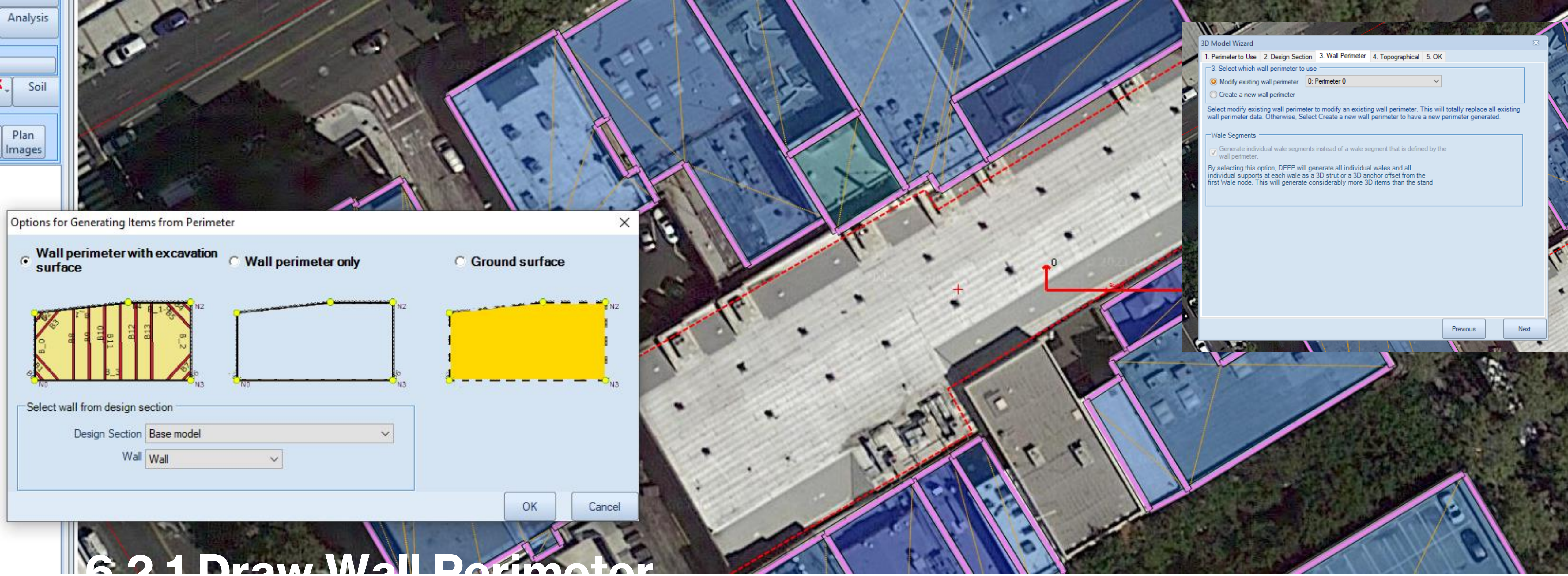
7.1.4 Open Map Function

- Draw project region
- Import buildings from Google Maps

6.1.6 Import Buildings

- Information from Google Earth for terrain
- Cadmapper for Buildings
- Certain buildings might be missing
- Then import in DeepEX

7.1.7 Remove & Move Structures

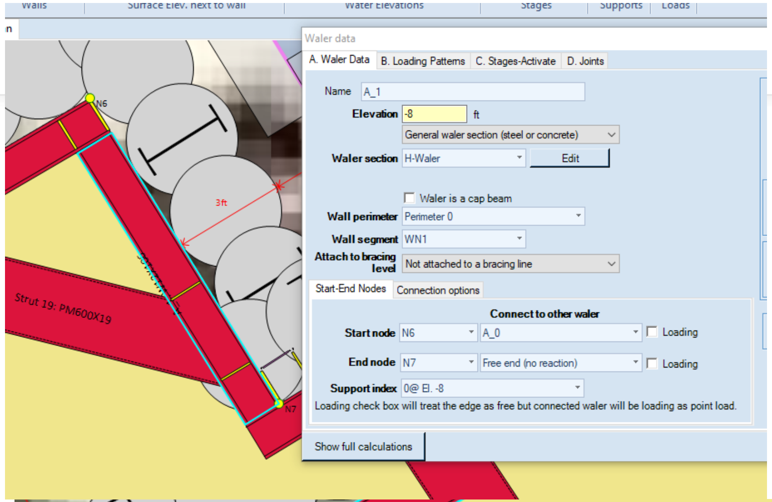


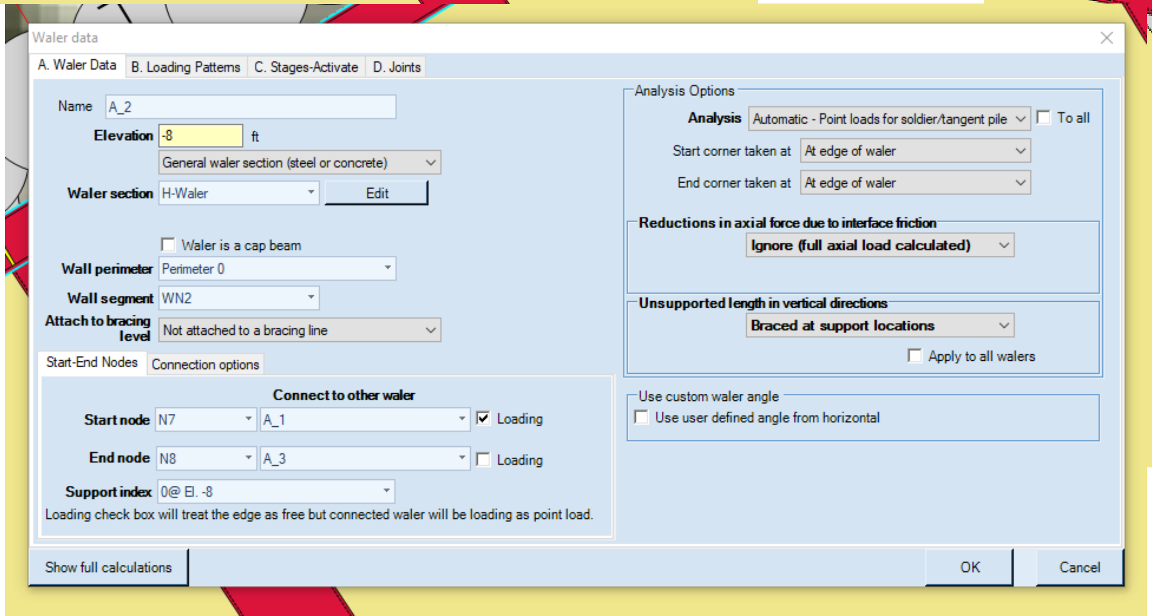
6.2.1 Draw Wall Perimeter

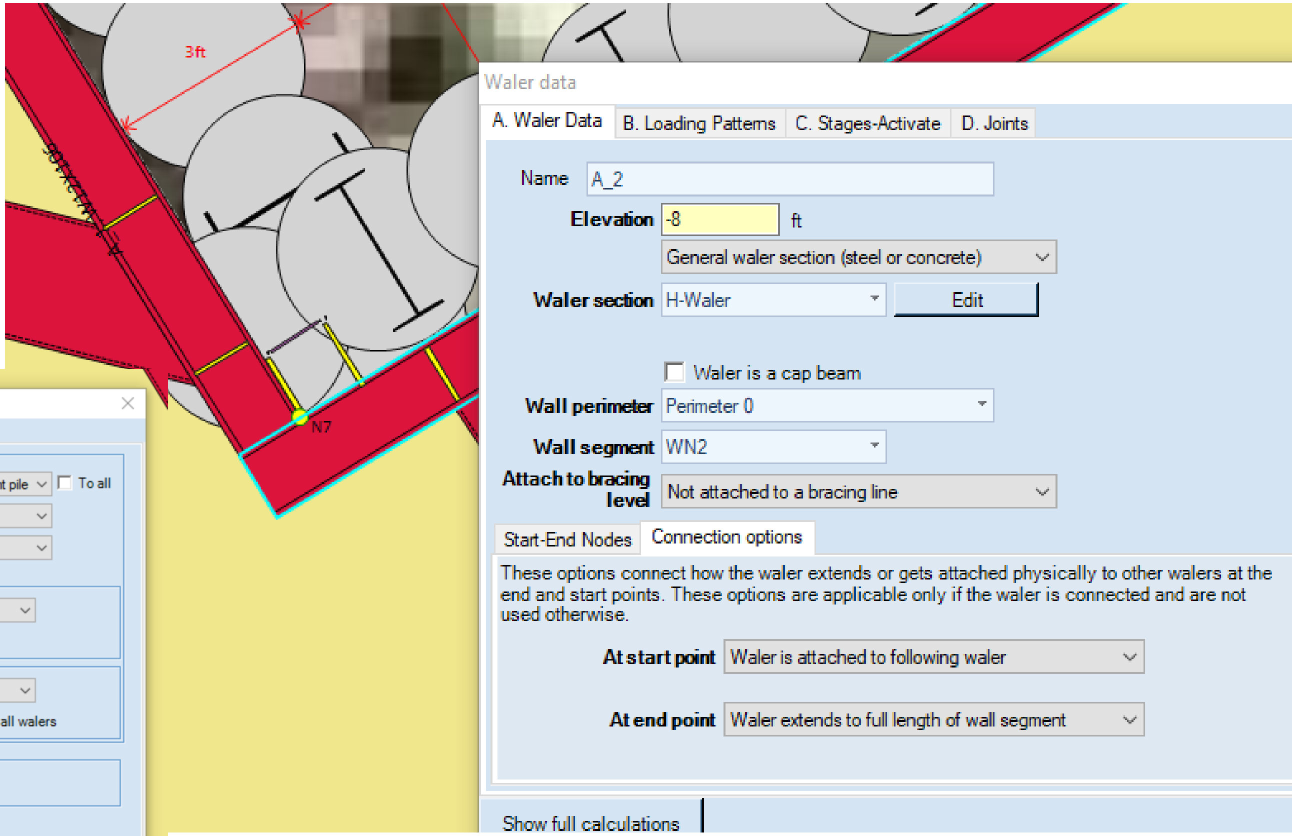
- Draw wall perimeter
- Generate preliminary layout
- Modify existing wall perimeter

7.2.2 Initial and Revised Strut Layout

7.2.3 Re-entrant Corner Walers

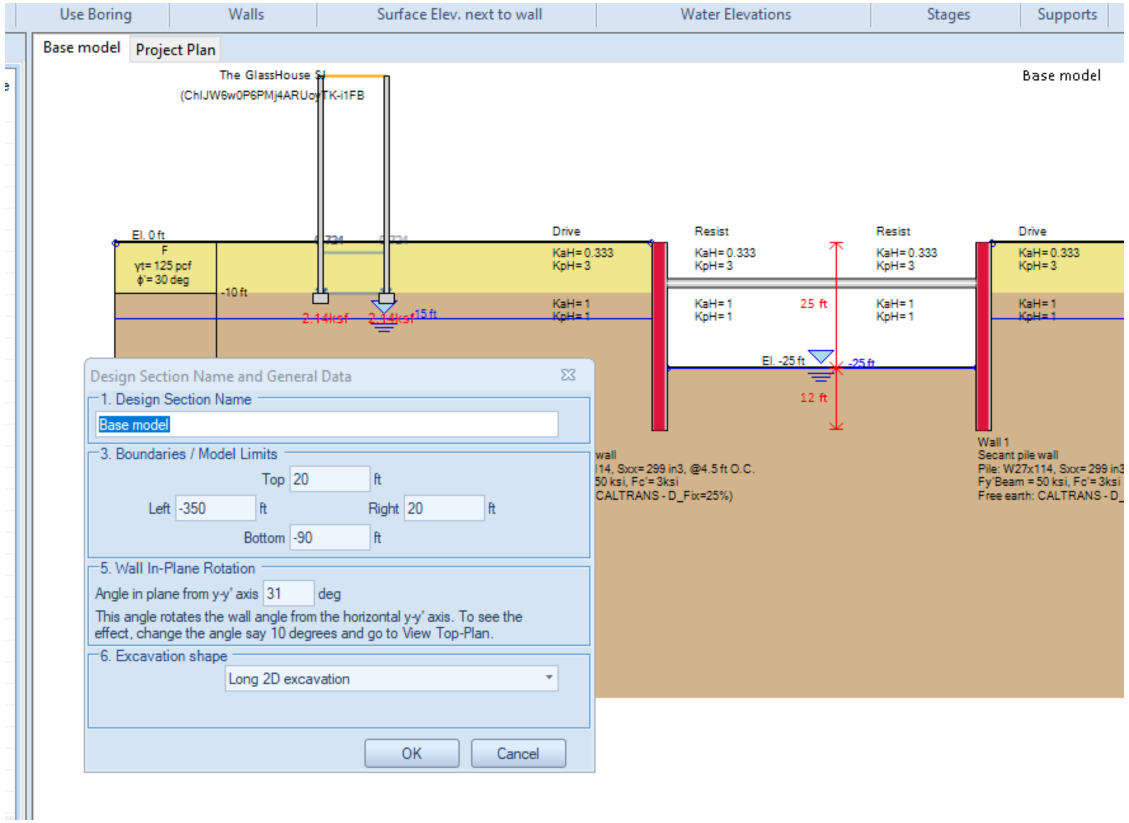




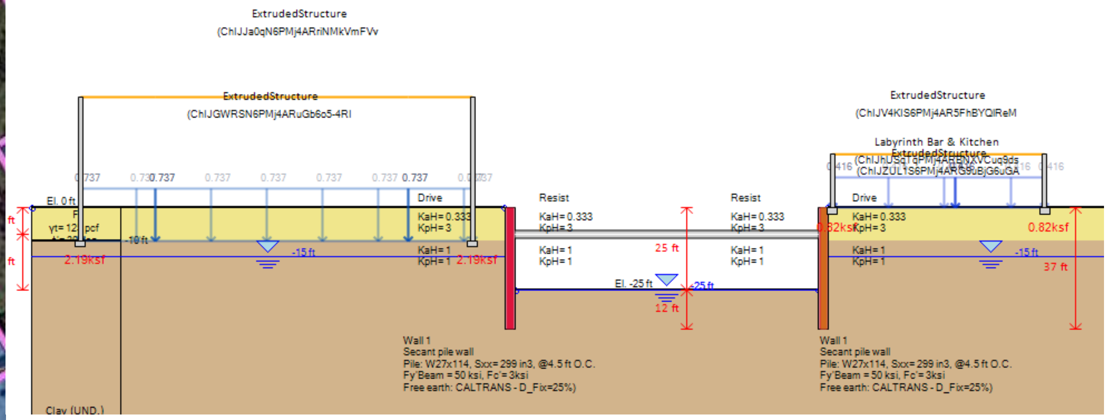


7.3.1 Design Sections

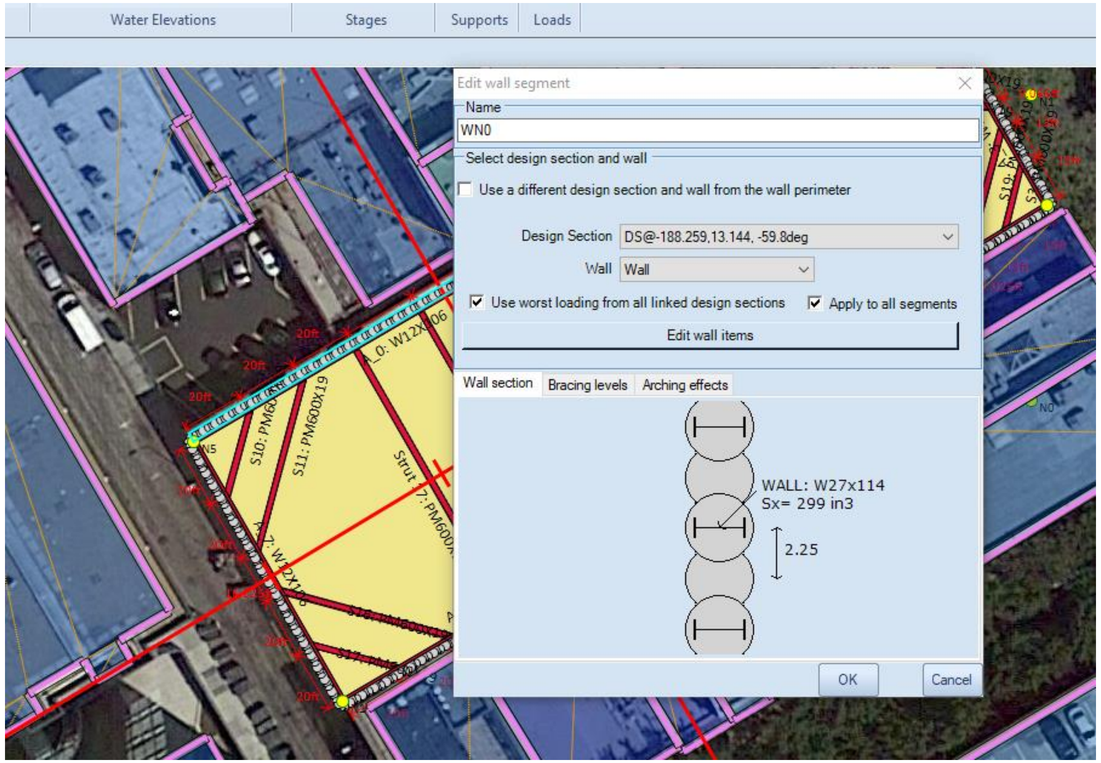
- Reposition 1st design section
- Change project limits

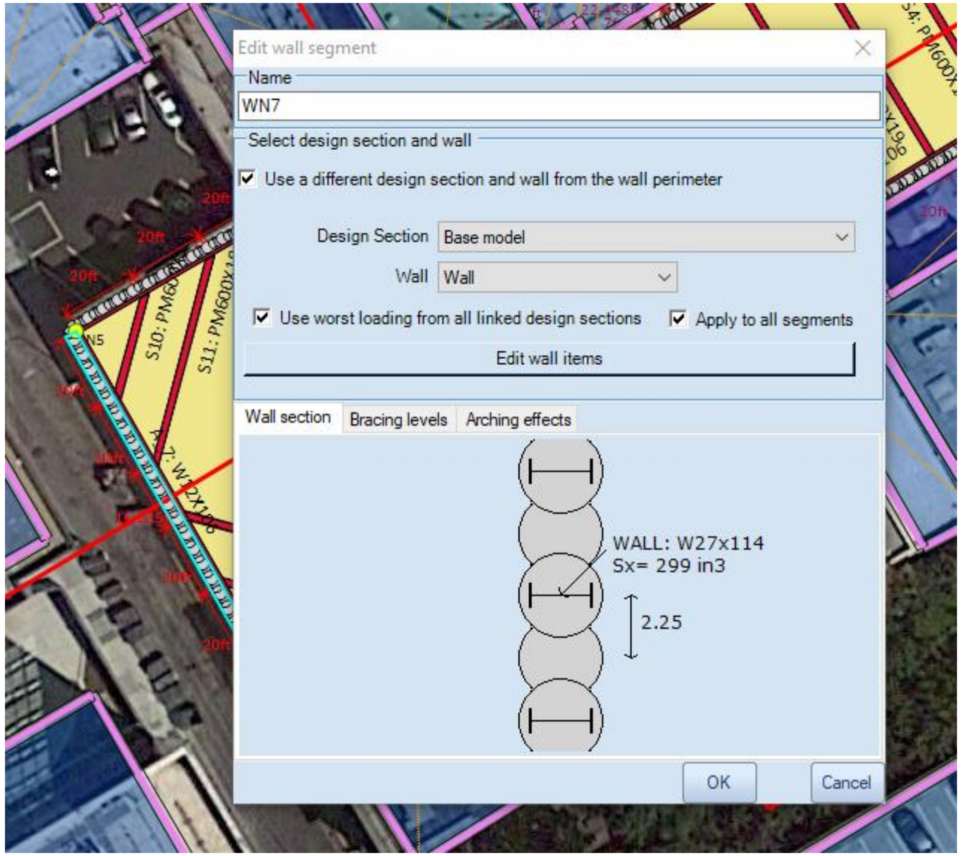


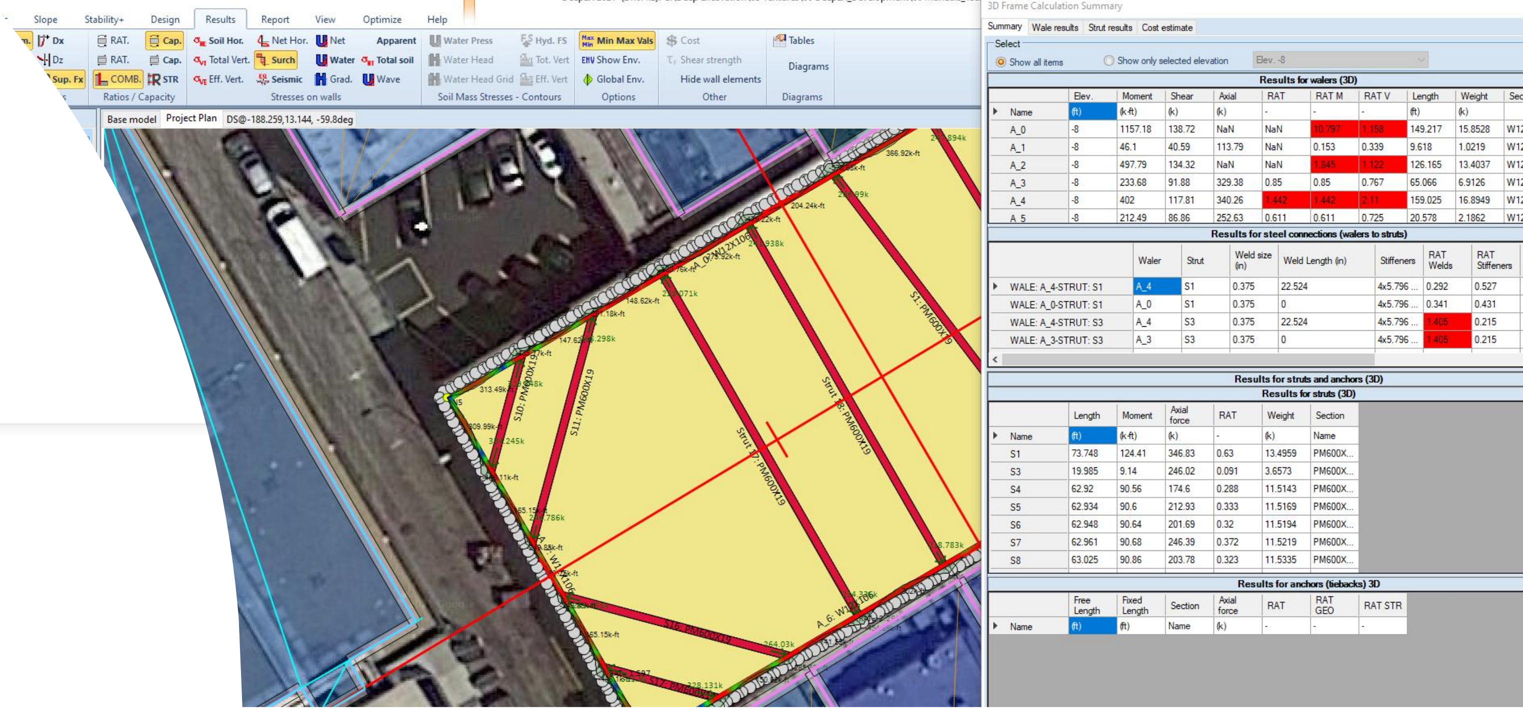
7.3.2 Cut Cross-Sections



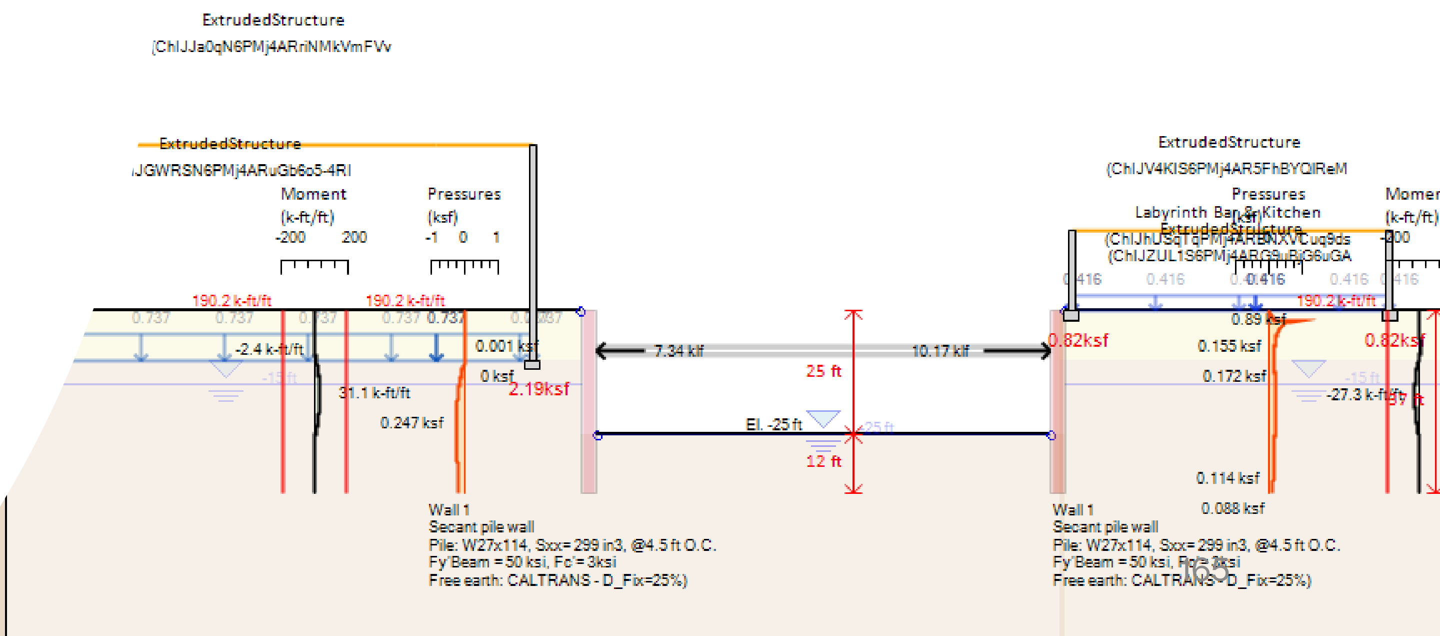
7.3.3 Pass Appropriate Design Sections

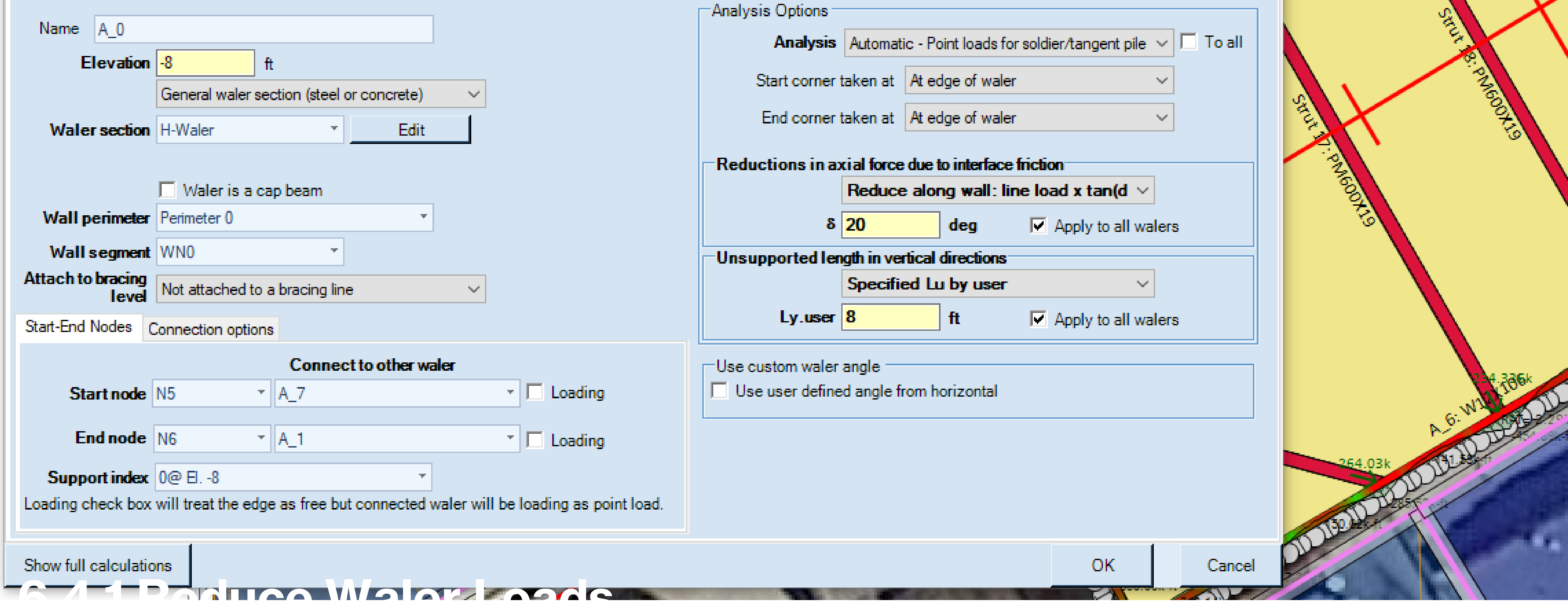





3D Frame Calculation Summary
Results for walers (3D)
Results for steel connections (walers to struts)
Results for struts and anchors (3D)
Results for struts (3D)
Results for anchors (tiebacks) 3D

7.4.0 Analyze

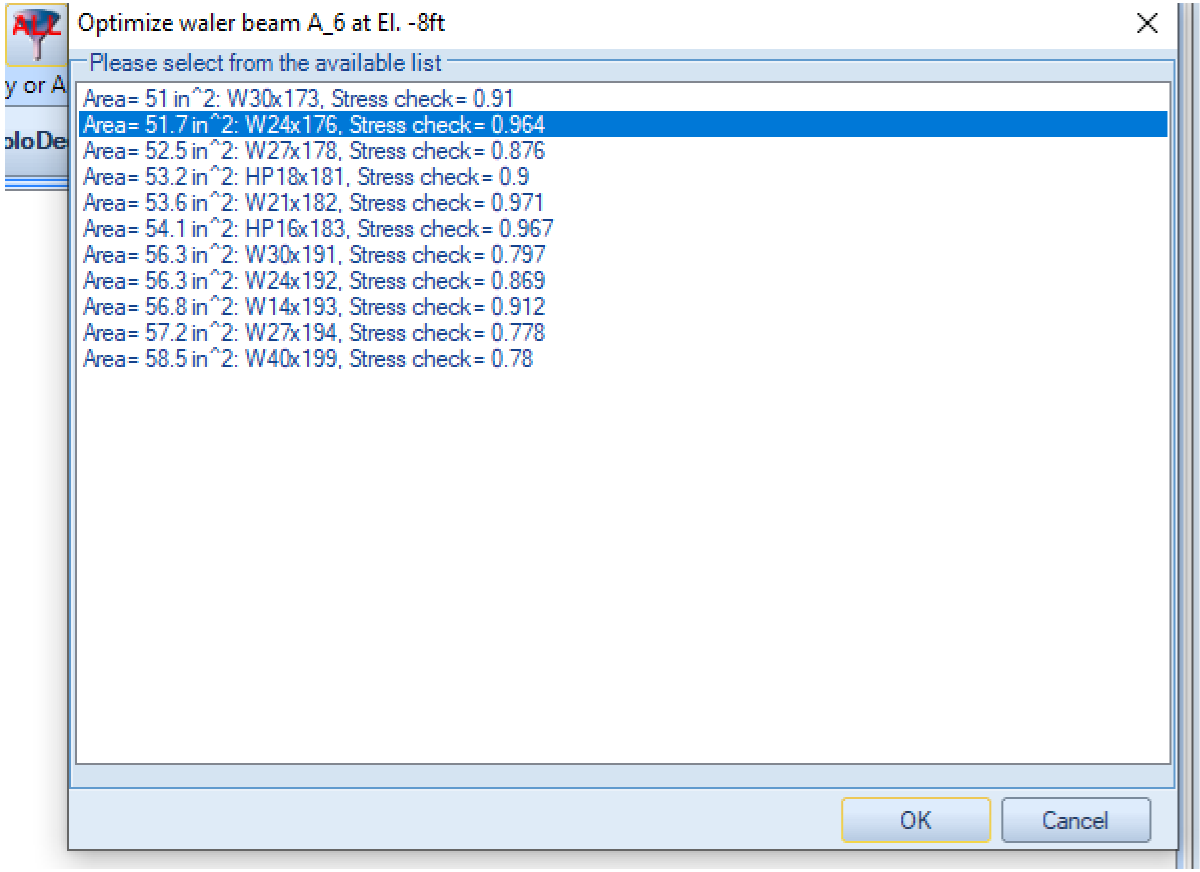

ExtrudedStructure
(ChIJJa0qN6PMj4ARriNMkVmFVv
Moment
(k-ft/ft)
Pressures
(ksf)
190.2 k-ft/ft
-2.4 k-ft/ft
31.1 k-ft/ft
0.247 ksf
2.19ksf
7.34 klf
10.17 klf
25 ft
El. -25 ft
12 ft
0.82ksf
0.155 ksf
0.172 ksf
-27.3 k-ft/ft
0.114 ksf
0.088 ksf
Wall 1
Secant pile wall
Pile: W27x114, Sxx= 299 in3, @4.5 ft O.C.
Fy Beam = 50 ksi, Fc'= 3ksi
Free earth: CALTRANS - D_Fix=25%)

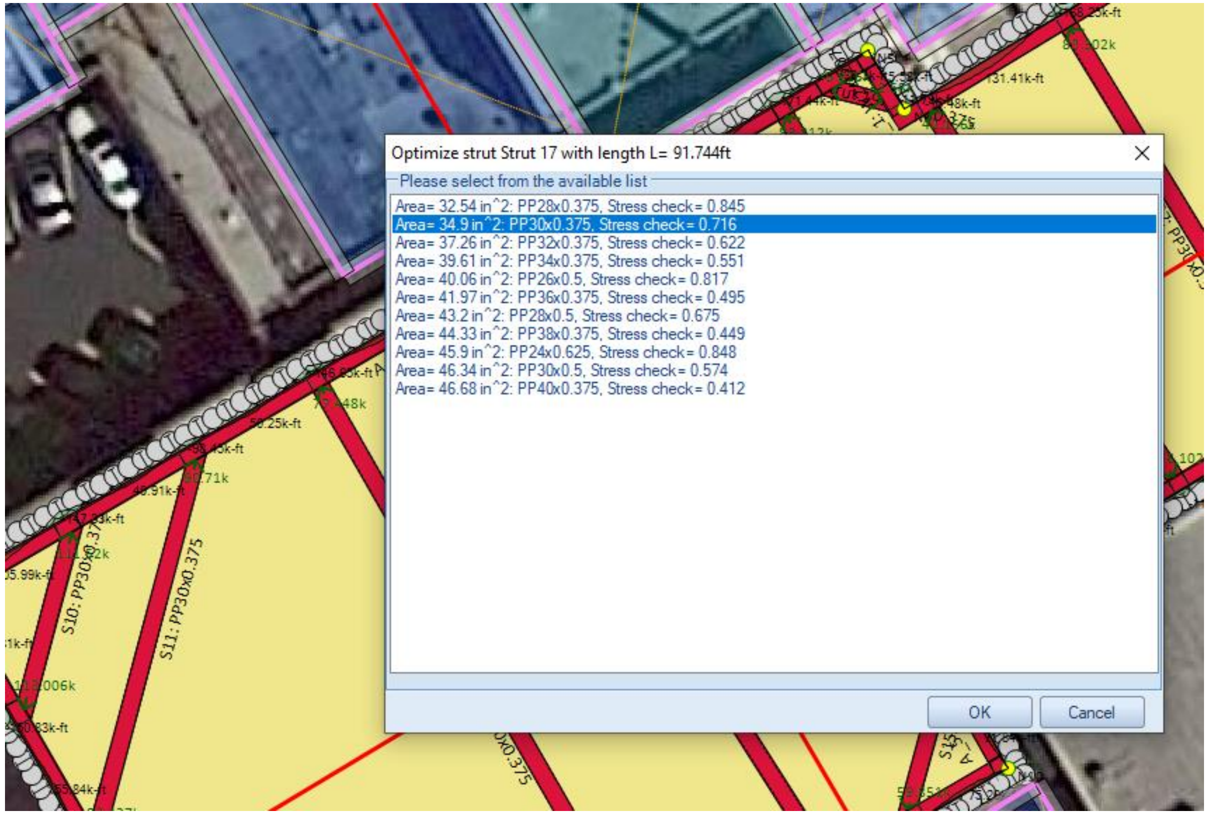


6.4.1 Reduce Waler Loads

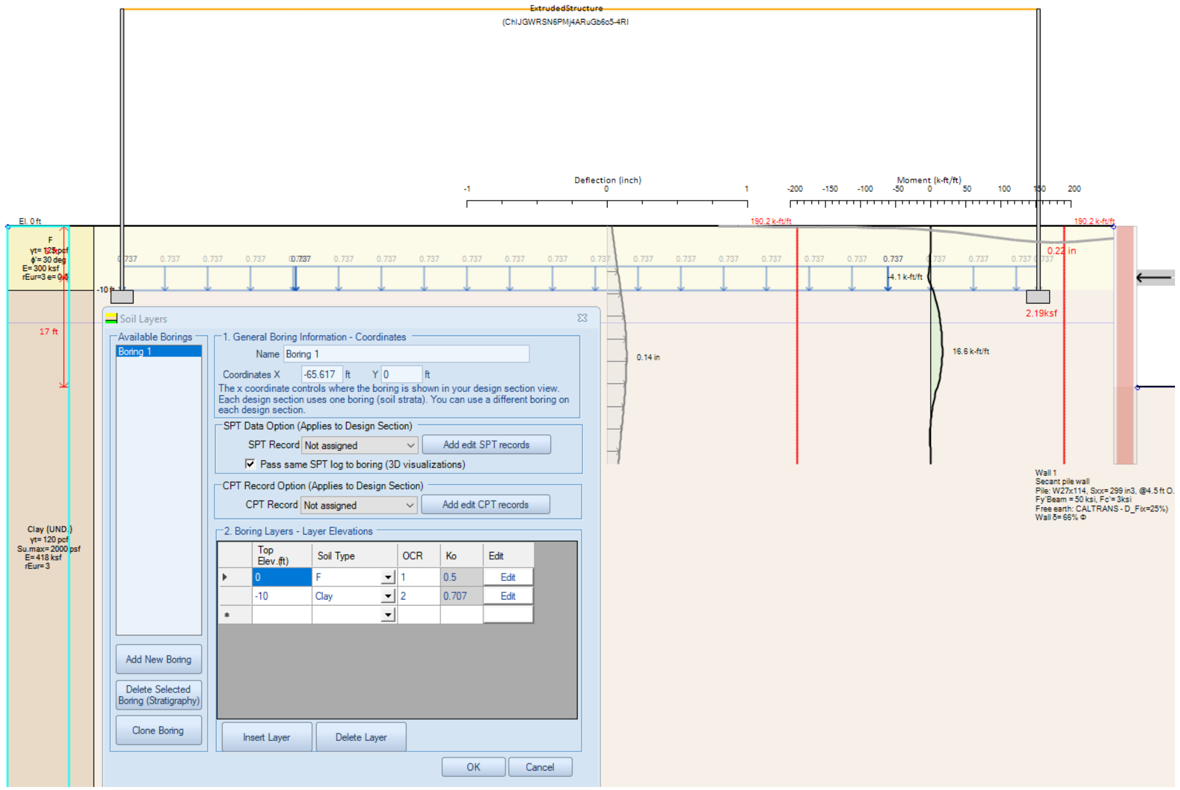
- Assume wall friction
- Cut vertical unbraced length

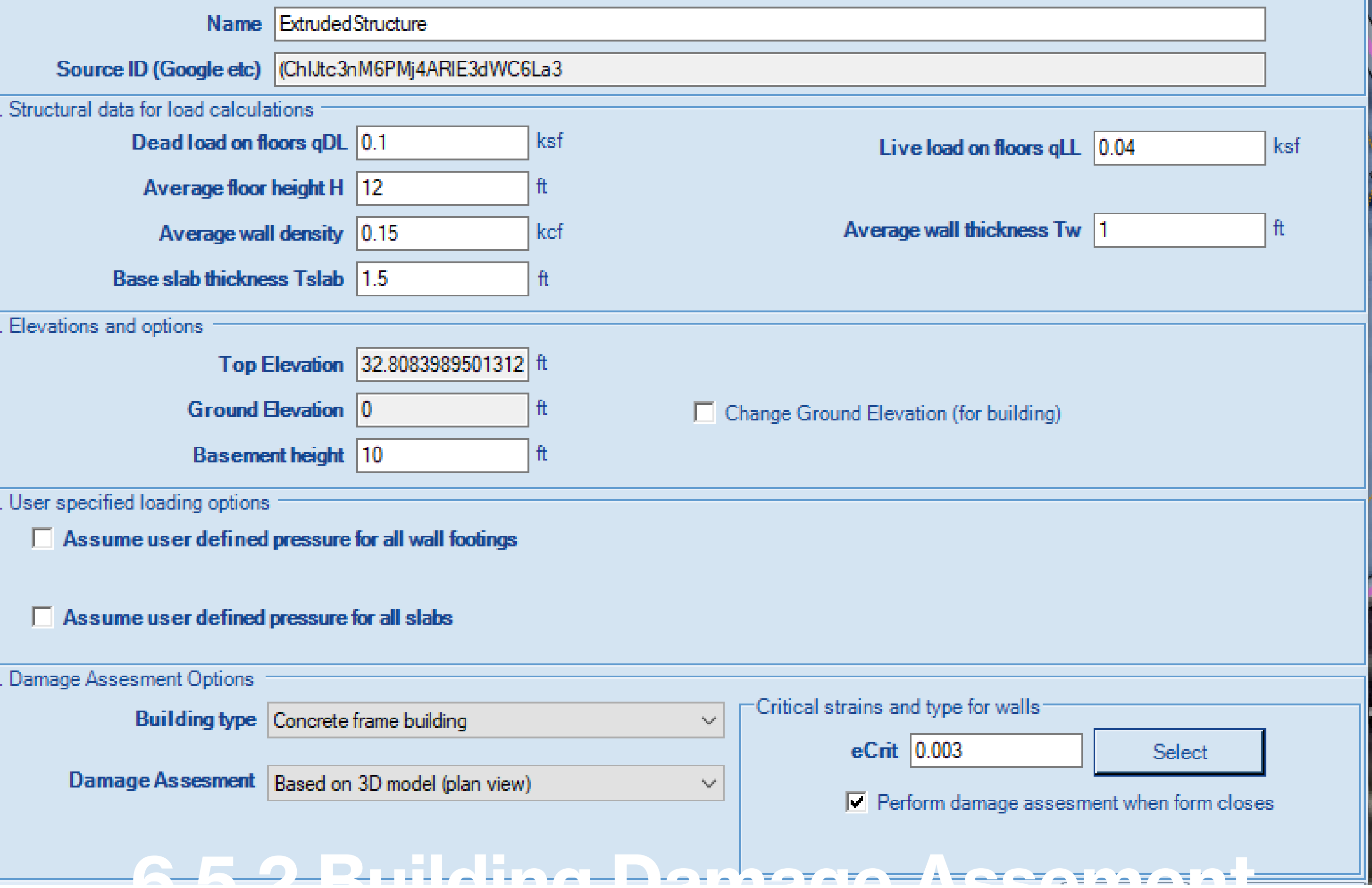
7.4.2 Optimize Waler-Strut Sizes

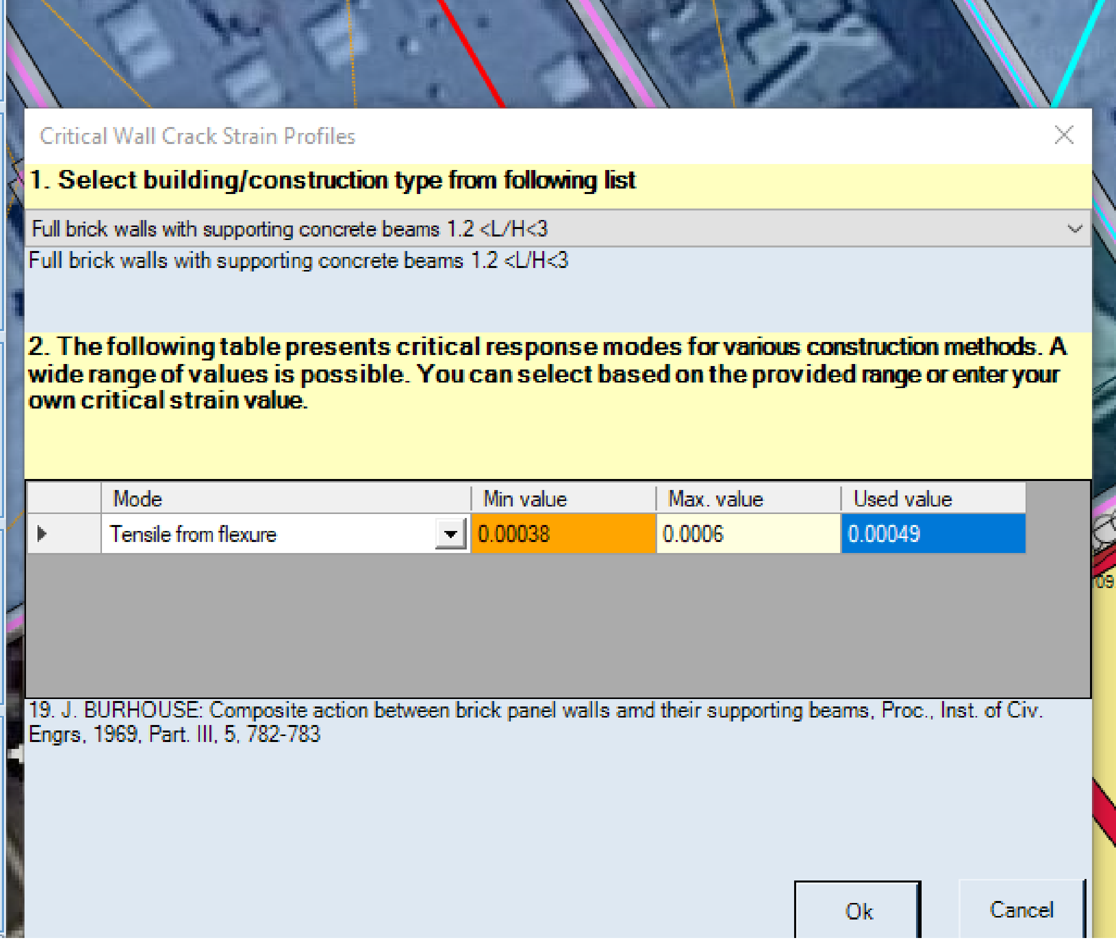




7.5.1 Non-linear analysis & Settlement



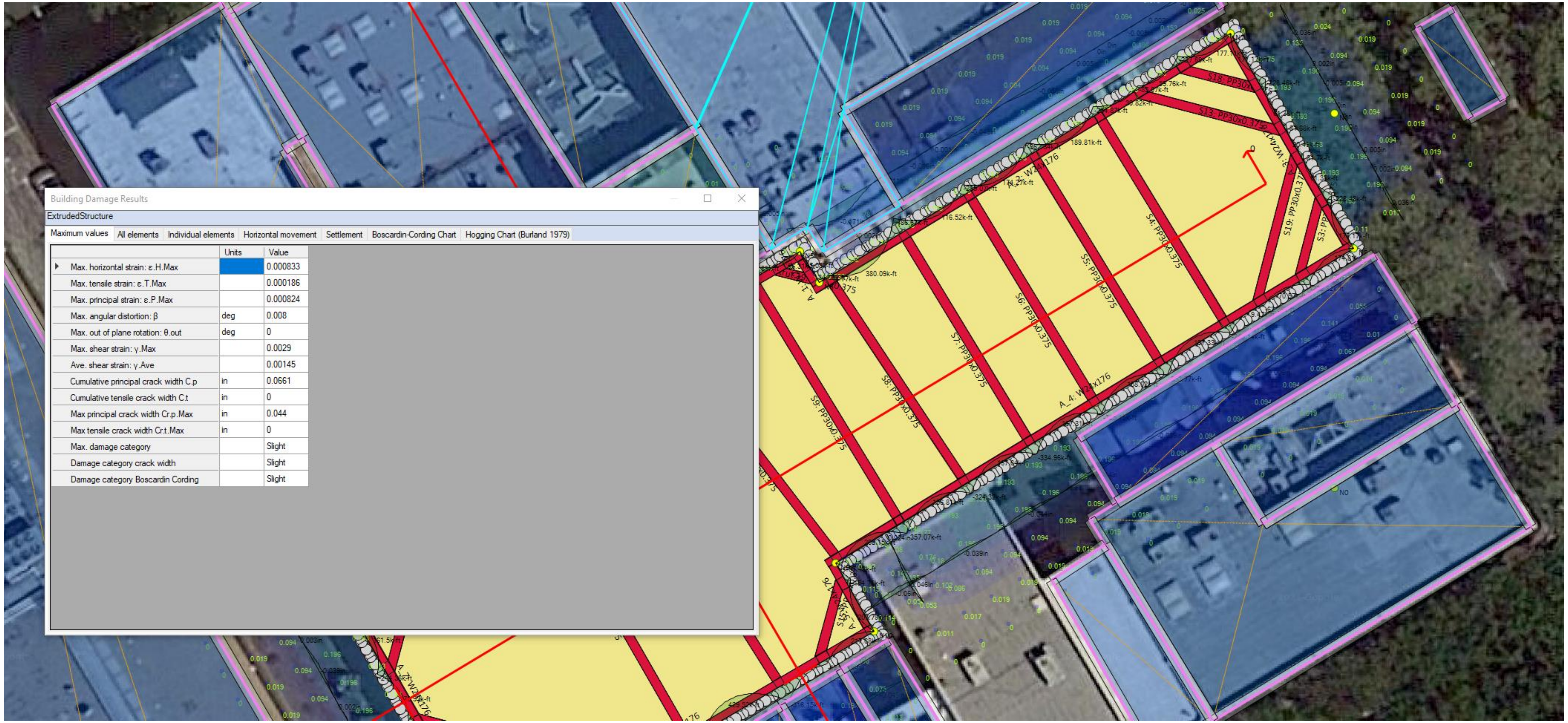




6.5.2 Building Damage Assesment

- Click on a building
- Define critical strains
- 2D-3D assessment

7.5.3 Building Damage Assesment

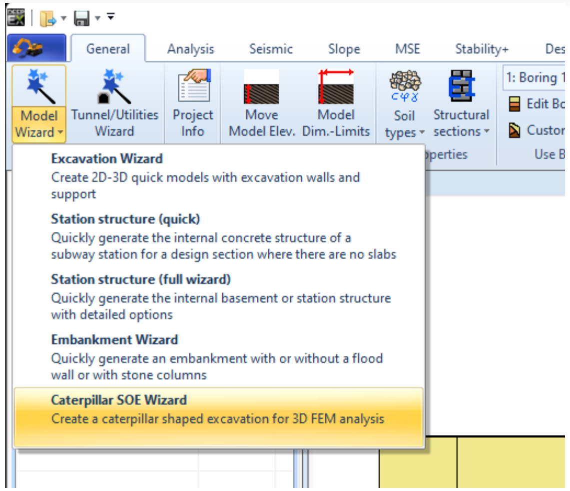


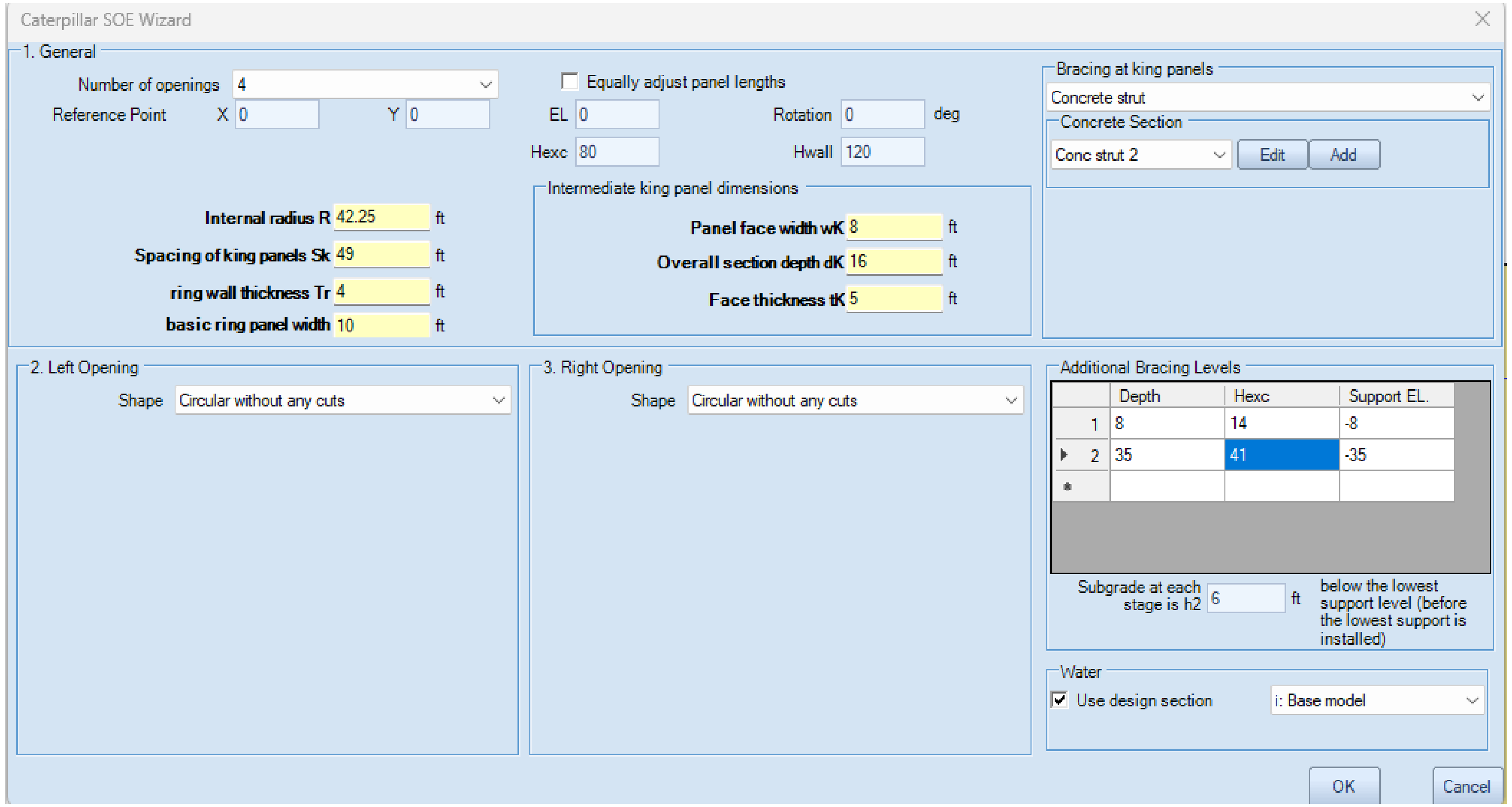
	Units	Value
Max. horizontal strain: ε.H.Max		0.000833
Max. tensile strain: ε.T.Max		0.000186
Max. principal strain: ε.P.Max		0.000824
Max. angular distortion: β	deg	0.008
Max. out of plane rotation: θ.out	deg	0
Max. shear strain: γ.Max		0.0029
Ave. shear strain: γ.Ave		0.00145
Cumulative principal crack width C.p	in	0.0661
Cumulative tensile crack width C.t	in	0
Max principal crack width Cr.p.Max	in	0.044
Max tensile crack width Cr.t.Max	in	0
Max. damage category		Slight
Damage category crack width		Slight
Damage category Boscardin Cording		Slight

7.6.0 Special 3D Wizard Applications

- Caterpillar excavations
- Arbitrary excavation

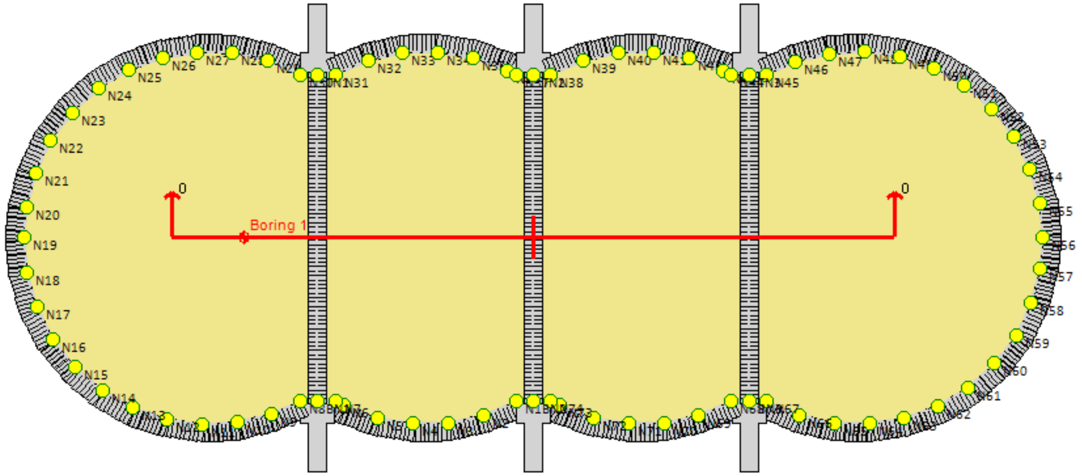
7.6.1 Caterpillar SOE

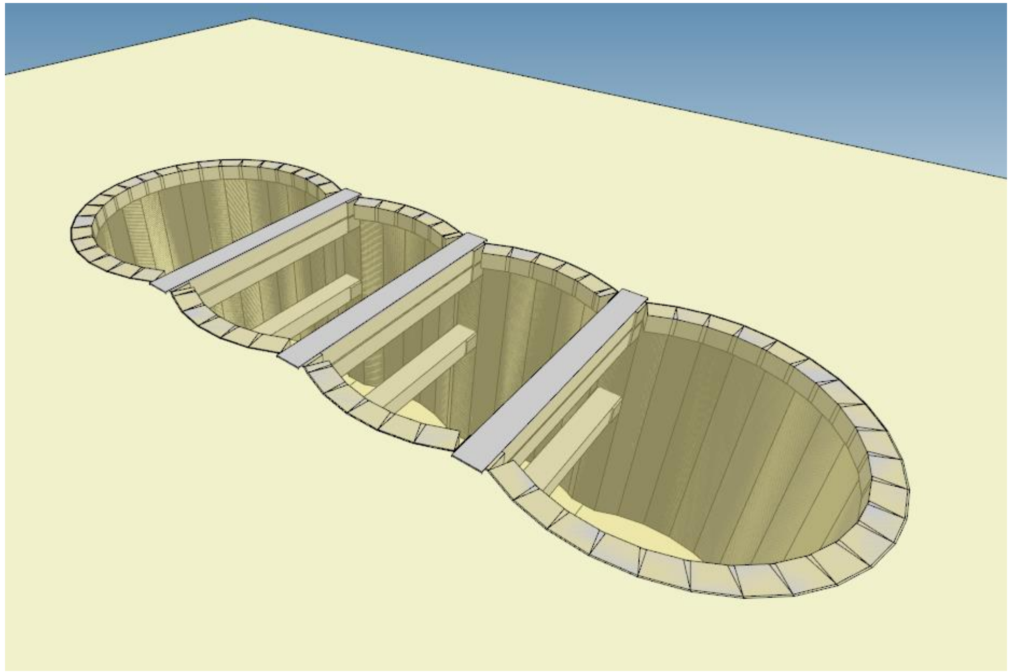




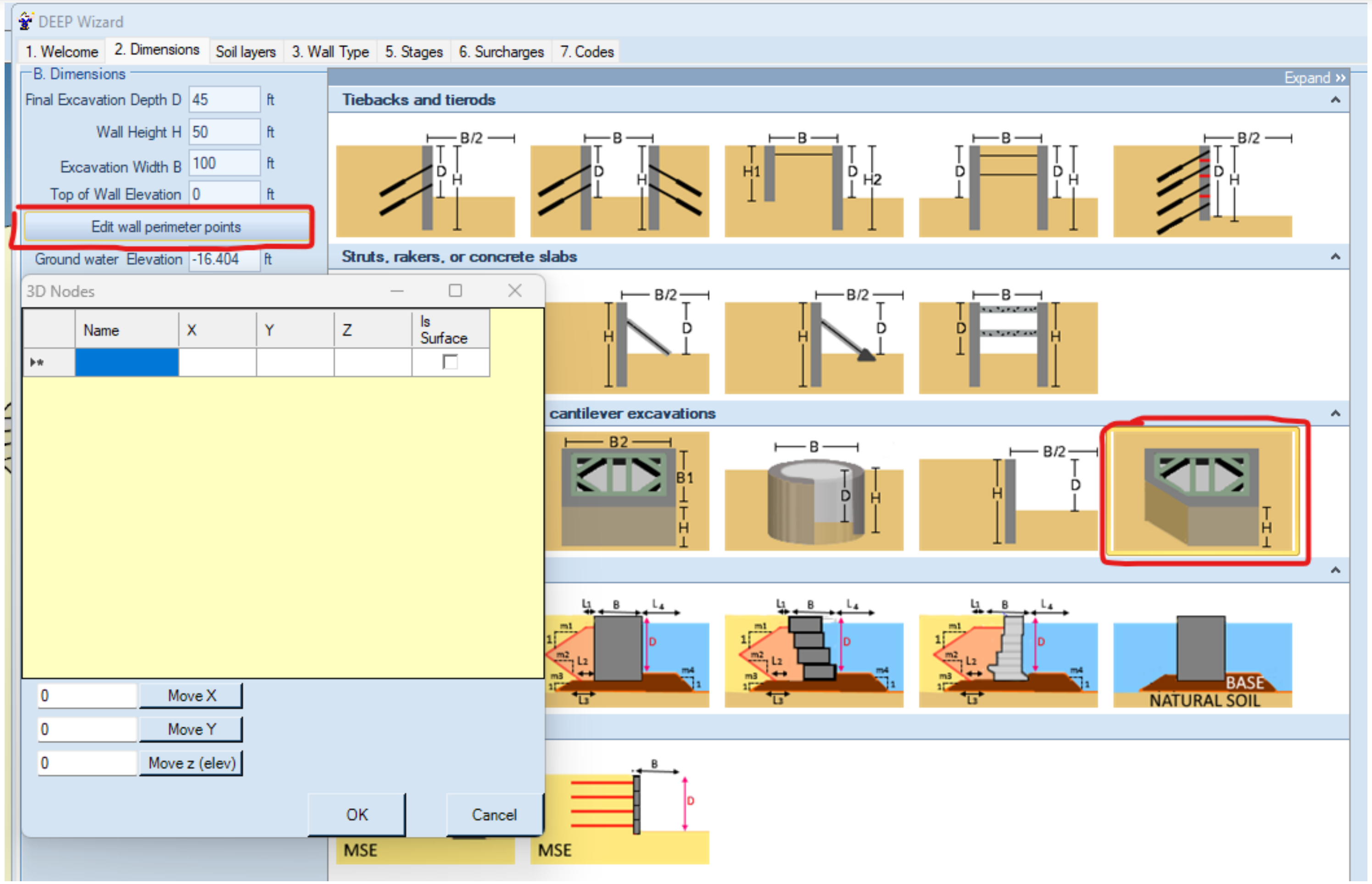
7.6.2 Caterpillar SOE

- 3D FEM only



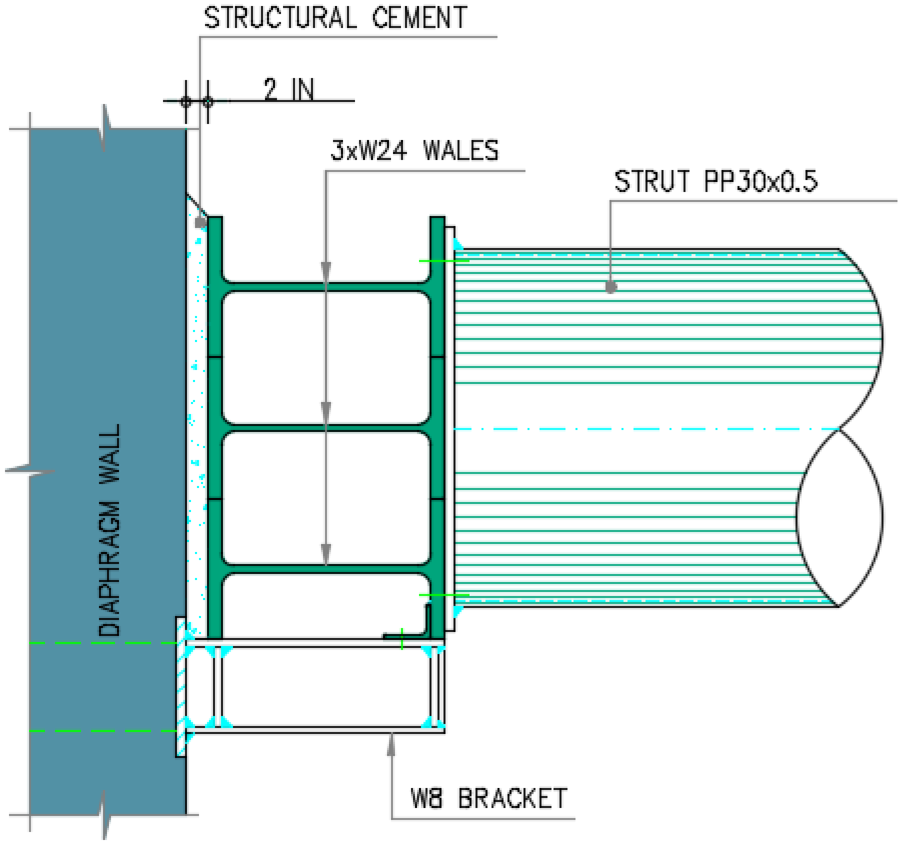


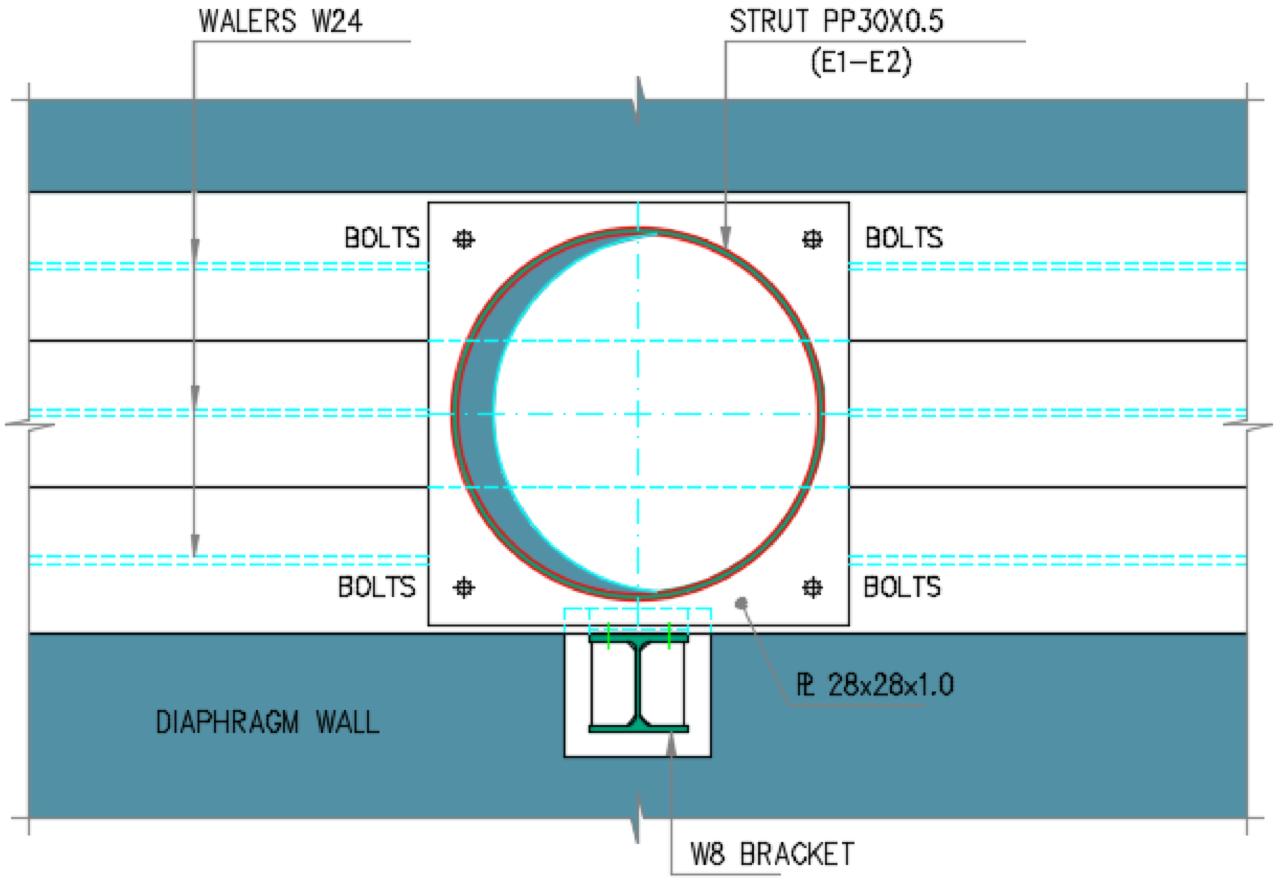
7.6.3 Arbitrary Excavation Shape



8. Steel connections

8.0.1 Waler connected to diaphragm wall





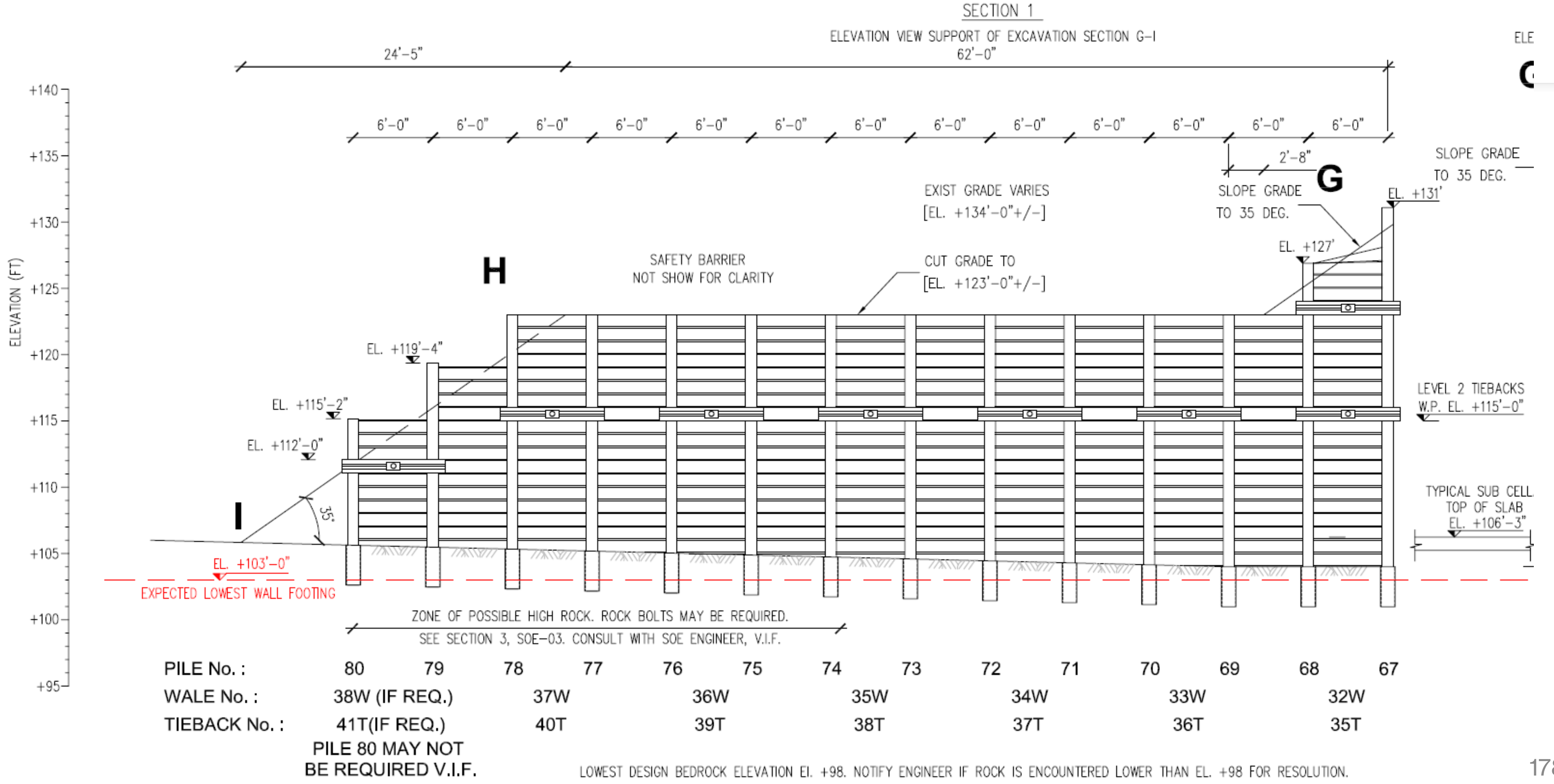
8.0.2 Double channel waler

- On soldier pile walls
- Typical to use one channel with one tieback per two piles
- Channels designed for moment

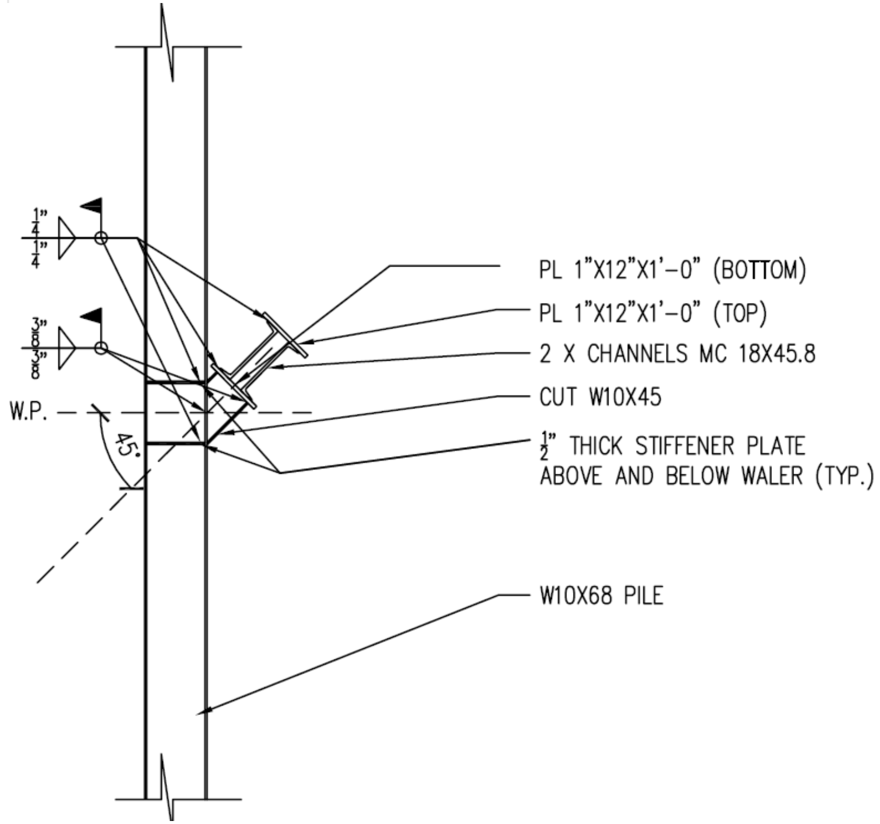
  $M = PL/4$ for both channels

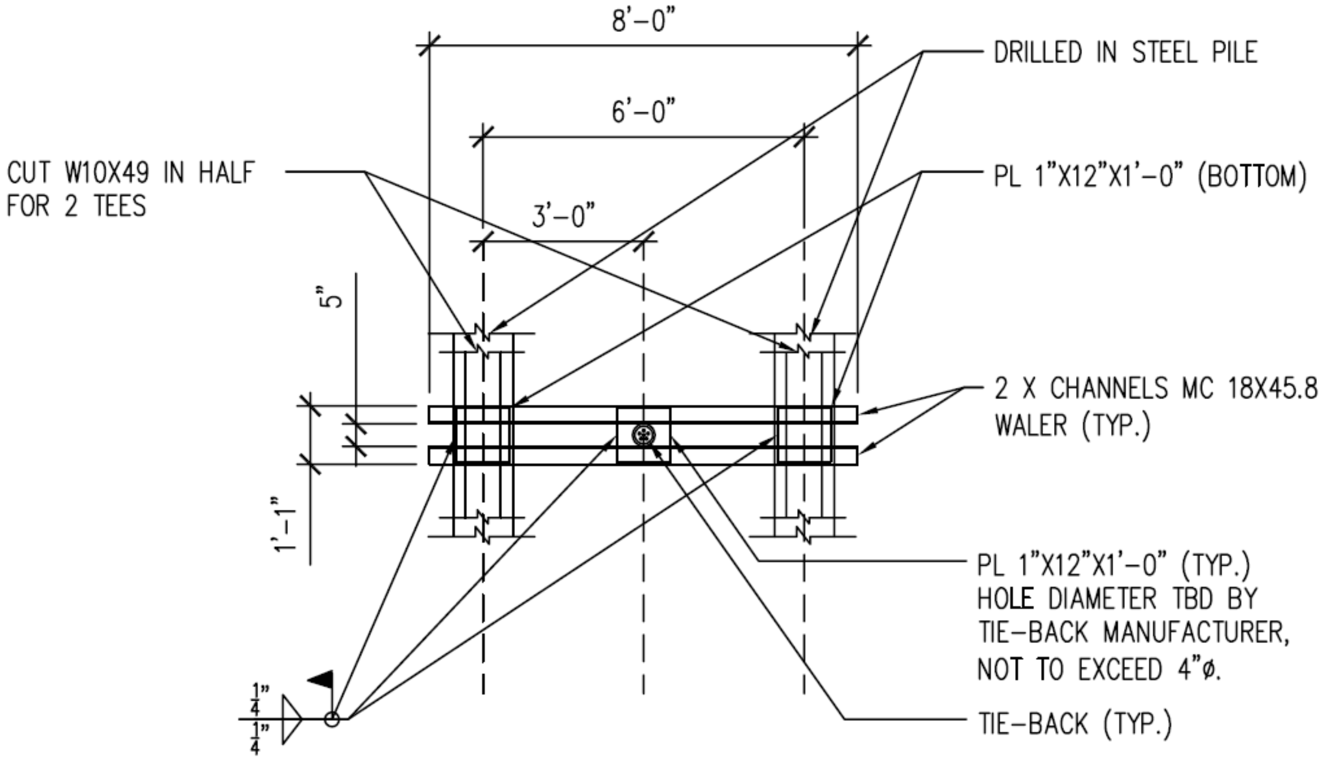
$$M_{DES} = \frac{\frac{PL}{4}}{2\ Channels}$$

8.0.3 Double channel walers



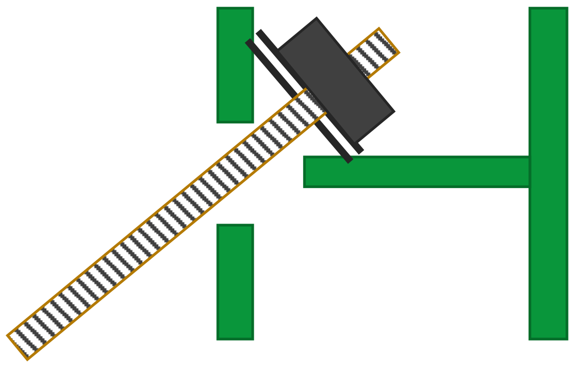
8.0.4 Double channel walers

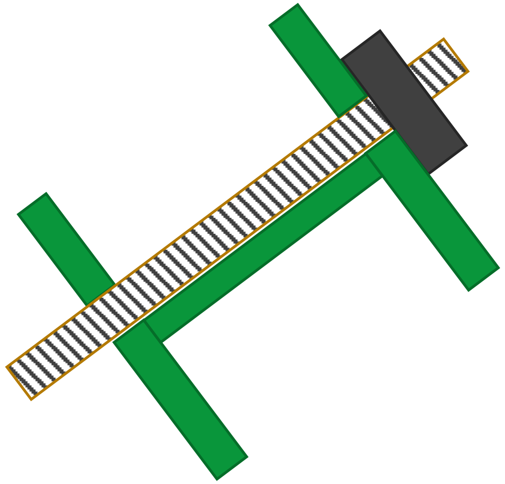




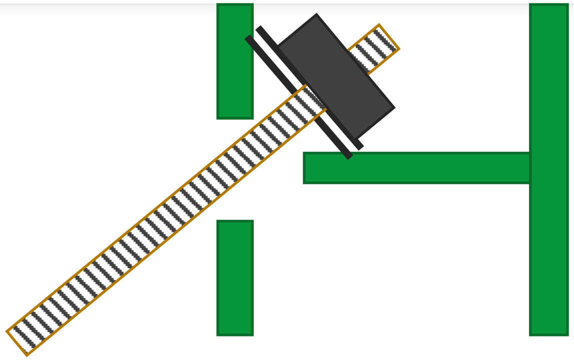
8.0.5 Tieback through W waler

- W walers may be used as walers for tiebacks
- Need to cut hole through web, flange, or both
- Cut hole reduces section modulus of waler

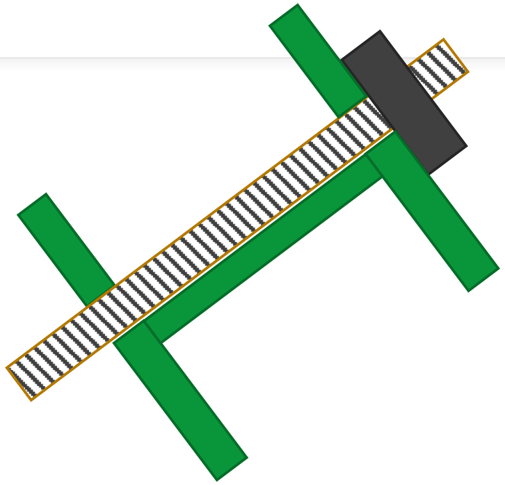




8.0.6 Tiebacks through W walers



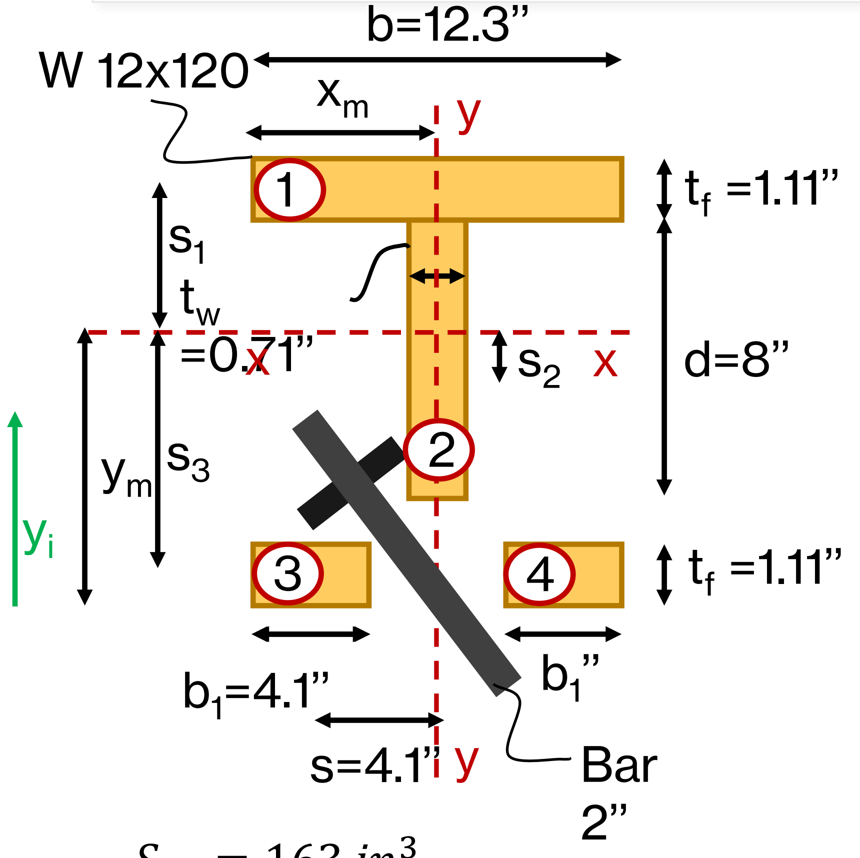
- Greater loss of section due to flange.
- Easier to connect to pile



- Smaller loss of section due to smaller holes at flange.
- Requires connection bracket to connect to pile

8.0.7 Example Tiebacks through W walers

Calculate Moment of Inertia and Section Modulus of the two W walers cases.



$S_{xx} = 163\ in^3$

$S_{yy} = 56\ in^3$

Bending y-y

Calculation of x_m (position of y-y axis)

$x_m = b/2 = 6.15''$ (symmetry)

Modulus of Elasticity

$$I_y = I_{y,1} + I_{y,2} + I_{y,3} + I_{y,4}$$

$$I_{y,1} = \frac{t_f \cdot b^3}{12} = 172.13\ in^4$$

$$I_{y,2} = \frac{d \cdot {t_w}^3}{12} = 0.239\ in^4$$

$$I_{y,3} = I_{y,4} = \frac{t_f \cdot {b_1}^3}{12} + t_f \cdot b_1 \cdot s^2 = 82.877\ in^4$$

So, $I_y = 338.123\ in^4$

Section Modulus:

$$S_{yy} = \frac{I_y}{b/2} = 54.98\ in^3$$

Bending x-x

Calculation of y_m (position of x-x axis)

$$y_m = \frac{A_1y_1 + A_2y_2 + +A_3y_3 + A_4y_4}{A_1 + A_2 + +A_3 + A_4} = 7.82''$$

Modulus of Elasticity

$$I_x = I_{x,1} + I_{x,2} + I_{x,3} + I_{x,4}$$

$$I_{x,1} = \frac{b \cdot {t_f}^3}{12} + t_f \cdot b \cdot {s_1}^2 = 304.071\ in^4$$

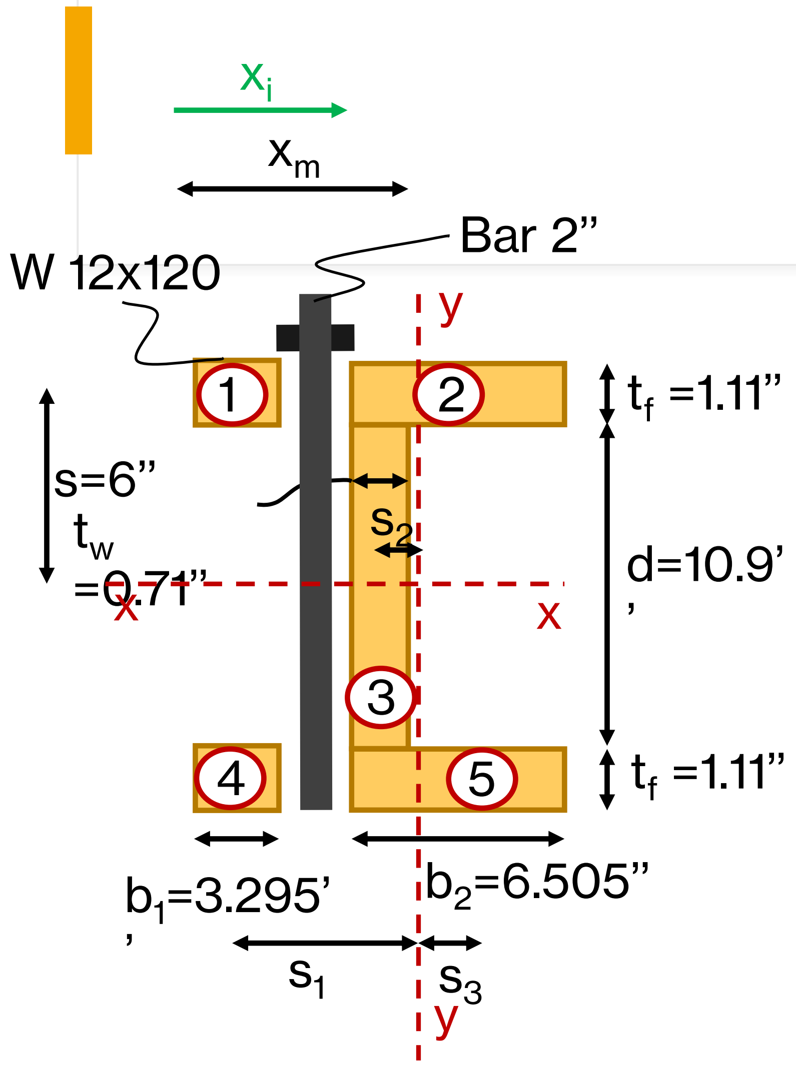
$$I_{x,2} = \frac{t_w \cdot d^3}{12} + t_w \cdot d \cdot {s_2}^2 = 113.178\ in^4$$

$$I_{x,3} = I_{x,4} = \frac{b_1 \cdot {t_f}^3}{12} + t_f \cdot b_1 \cdot {s_3}^2 = 238.821\ in^4$$

So, $I_x = 656.07\ in^4$

Section Modulus:

$$S_{xx} = \frac{I_x}{y_m} = 83.896\ in^3$$



$$S_{xx} = 163\ in^3$$
$$S_{yy} = 56\ in^3$$

Bending y-y

Calculation of x_m (position of y-y axis)

$$x_m = \frac{A_1 x_1 + A_2 x_2 + +A_3 x_3 + A_4 x_4}{A_1 + A_2 + +A_3 + A_4} = 6.284''$$

Modulus of Elasticity

$$I_y = I_{y,1} + I_{y,2} + I_{y,3} + I_{y,4} + I_{y,5}$$

$$I_{y,1} = I_{y,4} = \frac{t_f \cdot {b_1}^3}{12} + t_f \cdot b_1 \cdot {s_1}^2 = 81.934\ in^4$$

$$I_{y,3} = \frac{d \cdot {t_w}^3}{12} + d \cdot t_w \cdot {s_2}^2 = 0.464\ in^4$$

$$I_{y,2} = I_{y,5} = \frac{t_f \cdot {b_2}^3}{12} + t_f \cdot b_2 \cdot {s_3}^2 = 80.604\ in^4$$

So, $I_y = 325.54\ in^4$

Section Modulus:

$$S_{yy} = \frac{I_y}{x_m} = 51.8\ in^3$$

Bending x-x

Calculation of y_m (position of x-x axis)

$y_m = D/2 = 6.55''$ (symmetry)

Modulus of Elasticity

$$I_x = I_{x,1} + I_{x,2} + I_{x,3} + I_{x,4} + I_{x,5}$$

$$I_{x,1} = I_{x,4} = \frac{b_1 \cdot {t_f}^3}{12} + t_f \cdot b_1 \cdot s^2 = 132.044\ in^4$$

$$I_{x,3} = \frac{t_w \cdot d^3}{12} = 76.623\ in^4$$

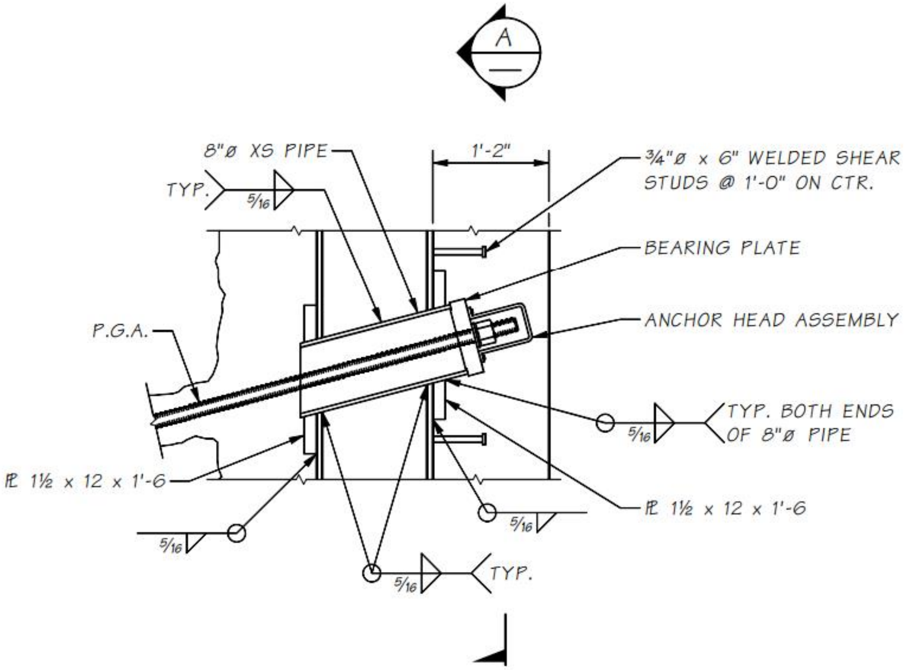
$$I_{x,2} = I_{x,5} = \frac{b_2 \cdot {t_f}^3}{12} + t_f \cdot b_2 \cdot s^2 = 260.681\ in^4$$

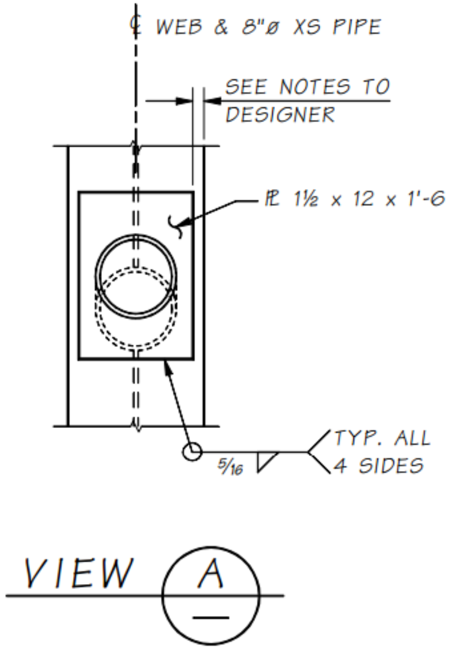
So, $I_x = 862.073\ in^4$

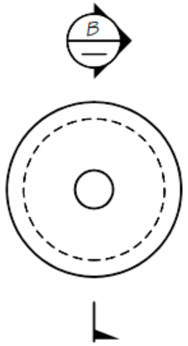
Section Modulus:

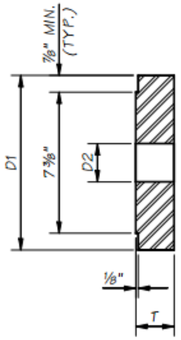
$$S_{xx} = \frac{I_x}{y_m} = 131.614\ in^3$$

8.0.10 Pipe hole through soldier pile



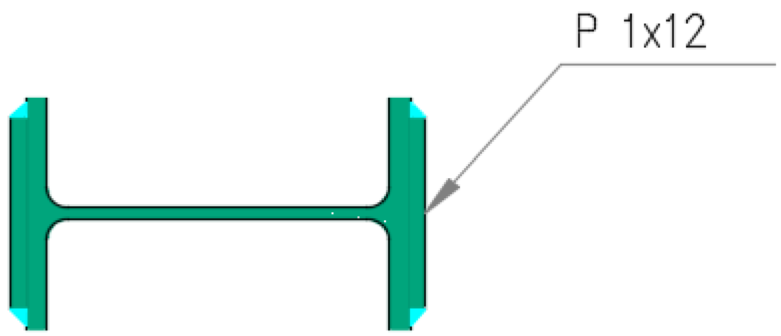


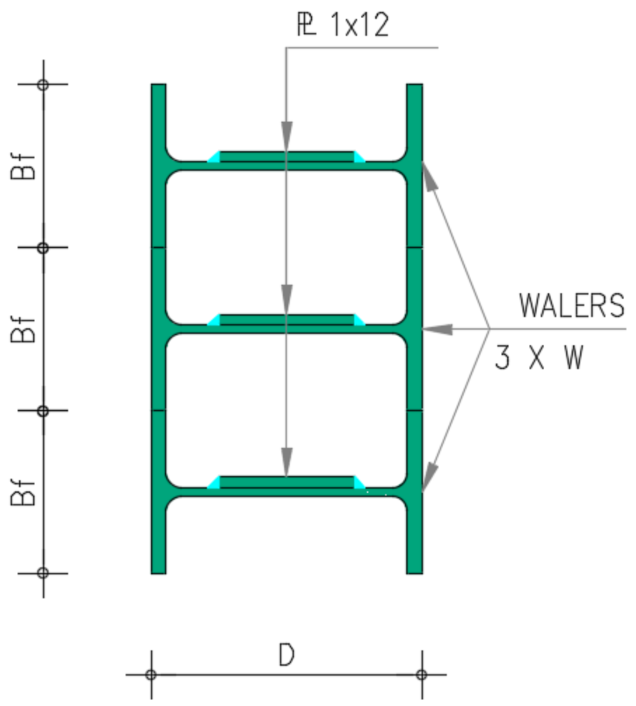




8.1 Trick of the trade (with walers and more)

- Need greater strength at specified location?
- Use web or flange plates





Quiz: Waler design - Torsion

You have a 70 ft wide sheet pile braced excavation supported by cross-lot struts, corner braces, and walers. The walers are welded on the sheet piles, and the struts and braces are welded on the walers without brackets. Should you consider torsion for the design of the walers?

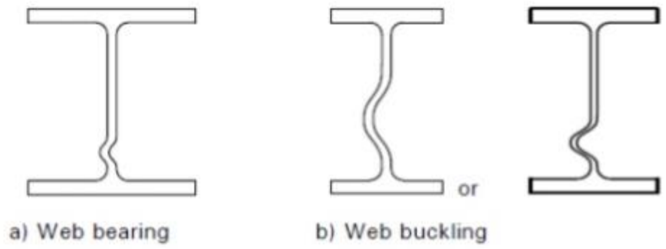
a) Yes, torsion should be considered

b) No, torsion can be ignored

8.2 Flanges and Webs with Concentrated Forces

This applies to single- and double-concentrated forces applied normal to the flange(s) of wide flange sections and similar built-up shapes. A single-concentrated force can be either tensile or compressive. Double-concentrated forces are one tensile and one compressive and form a couple on the same side of the loaded member.

When the required strength exceeds the available strength as determined for the limit states listed in this section, stiffeners and/or doublers shall be provided and shall be sized for the difference between the required strength and the available strength for the applicable limit state. Stiffeners are required at unframed ends of beams.



8.3.1 Flange Local Bending

This applies to tensile single-concentrated forces and the tensile component of double-concentrated forces.

The design strength, ϕR_n, and the allowable strength, R_n/Ω, for the limit state of flange local bending shall be determined as follows:

$$R_n = 6.25 \cdot t_f{}^2 \cdot F_{yf}$$

ϕ=0.90 (LRFD) Ω=1.67 (ASD)

where: F_{yf} = Specified minimum yield stress of the flange

t_f = Thickness of the loaded flange

If the length of loading across the member flange is less than $0.15 \cdot b_f$, where bf is the member flange width, the previous equation need not be checked.

When the concentrated force to be resisted is applied at a distance from the member end that is less than $10 \cdot t_f$, R_n shall be reduced by 50 percent.

When required, a pair of transverse stiffeners shall be provided.

8.3.2 Web Local Yielding

This applies to single-concentrated forces and both components of double-concentrated forces.

The available strength for the limit state of web local yielding shall be determined as follows:

ϕ=1.00 (LRFD) Ω=1.50 (ASD)

The nominal strength, R_n, shall be determined as follows:

i. When the concentrated force to be resisted is applied at a distance from the member end that is greater than the depth of the member d:

$$R_n = (5 \cdot k + N) \cdot t_w \cdot F_{yw}$$

ii. When the concentrated force to be resisted is applied at a distance from the member end that is less than or equal to the depth of the member d

$$R_n = (2.5 \cdot k + N) \cdot t_w \cdot F_{yw}$$

where: F_{yf} = Specified minimum yield stress of the web

t_w = Web thickness

k = distance from outer face of the flange to the web toe of the fillet

N = = length of bearing (not less than k for end beam reactions)

When required, a pair of transverse stiffeners or a doubler plate shall be provided.

9.3.3 Web Local Crippling

This applies to compressive single-concentrated forces or the compressive component of double-concentrated forces.

The available strength for the limit state of web local crippling shall be determined as follows:

ϕ=0.75 (LRFD) Ω=2.00 (ASD)

The nominal strength, R_n, shall be determined as follows:

i. When the concentrated compressive force to be resisted is applied at a distance from the member end that is greater than or equal to d/2:

$$R_n = 0.80 \cdot {t_w}^2 \cdot \left[1 + 3 \cdot \left(\frac{N}{d}\right) \cdot \left(\frac{t_w}{t_w}\right)^{1.5}\right] \cdot \sqrt{\frac{E \cdot t_f \cdot F_{yw}}{t_w}}$$

ii. When the concentrated compressive force to be resisted is applied at a distance from the member end that is less than d/2:

For $N/d \leq 0.2$

$$R_n = 0.40 \cdot {t_w}^2 \cdot \left[1 + 3 \cdot \left(\frac{N}{d}\right) \cdot \left(\frac{t_w}{t_w}\right)^{1.5}\right] \cdot \sqrt{\frac{E \cdot t_f \cdot F_{yw}}{t_w}}$$

For $N/d > 0.2$

$$R_n = 0.40 \cdot {t_w}^2 \cdot \left[1 + \left(\frac{4N}{d} - 0.2\right) \cdot \left(\frac{t_w}{t_w}\right)^{1.5}\right] \cdot \sqrt{\frac{E \cdot t_f \cdot F_{yw}}{t_w}}$$

where: d = Overall depth of the member

t_f = Flange thickness

When required, a transverse stiffener, or pair of transverse stiffeners, or a doubler plate extending at least one-half the depth of the web shall be provided.

8.3.4 Web Sideway Buckling

This applies only to compressive single-concentrated forces applied to members where relative lateral movement between the loaded compression flange and the tension flange is not restrained at the point of application of the concentrated force.

ϕ=0.85 (LRFD) Ω=1.76 (ASD)

The available strength of the web shall be determined as follows:

i. If the compression flange is restrained against rotation:

For $(h/t_w)/(l/b_f) \leq 2.3$:

$$R_n = \frac{C_r \cdot {t_w}^3 \cdot t_f}{h^2} \cdot \left[1 + 0.4 \cdot \left(\frac{h/t_w}{l/b_f}\right)^3\right]$$

For $(h/t_w)/(l/b_f) > 2.3$:

The limit state of web sideways buckling does not apply.

When the required strength of the web exceeds the available strength, local lateral bracing shall be provided at the tension flange or either a pair of transverse stiffeners or a doubler plate shall be provided.

ii. If the compression flange is not restrained against rotation:

For $(h/t_w)/(l/b_f) \leq 1.7$:

$$R_n = \frac{C_r \cdot {t_w}^3 \cdot t_f}{h^2} \cdot \left[0.4 \cdot \left(\frac{h/t_w}{l/b_f}\right)^3\right]$$

For $(h/t_w)/(l/b_f) > 1.7$:

The limit state of web sideways buckling does not apply.

When the required strength of the web exceeds the available strength, local lateral bracing shall be provided at both flanges at the point of application of the concentrated forces.

In the above equations:

b_f = Flange width

t_f = Flange thickness

t_w = Web thickness

l = largest laterally unbraced length along either flange at the point of load

h = clear distance between flanges less the fillet or corner radius for rolled shapes; distance between adjacent lines of fasteners or the clear distance between flanges when welds are used for built-up shapes

C_r = 960,000 ksi when Mu < My (LRFD) or 1.5Ma < My (ASD) at the location of the force
= 480,000 ksi when Mu ≥ My (LRFD) or 1.5Ma ≥ My (ASD) at the location of the force

8.3.5 Web Compression Buckling

This applies to a pair of compressive single-concentrated forces or the compressive components in a pair of double-concentrated forces, applied at both flanges of a member at the same location.

The available strength for the limit state of web local buckling shall be determined as follows:

ϕ=0.90 (LRFD) Ω=1.67 (ASD)

$$R_n = \frac{24 \cdot {t_w}^3 \cdot \sqrt{E \cdot F_{yw}}}{h}$$

When the pair of concentrated compressive forces to be resisted is applied at a distance from the member end that is less than d/2, R_n shall be reduced by 50 percent.

When required, a single transverse stiffener, a pair of transverse stiffeners, or a doubler plate extending the full depth of the web shall be provided.

8.3.6 Web Panel Zone Shear

This section applies to double-concentrated forces applied to one or both flanges of a member at the same location.

The available strength of the web panel zone for the limit state of shear yielding shall be determined as follows: ϕ=0.85 (LRFD) Ω=1.76 (ASD)

i. When the effect of panel-zone deformation on frame stability is not considered in the analysis:

For $P_r \leq 0.4 \cdot P_c$:

$$R_n = 0.60 \cdot F_y \cdot d_c \cdot t_w$$

For $P_r > 0.4 \cdot P_c$:

$$R_n = 0.60 \cdot F_y \cdot d_c \cdot t_w \cdot \left(1.4 - \frac{P_r}{P_c}\right)$$

ii. When frame stability, including plastic panel-zone deformation, is considered in the analysis:

For $P_r \leq 0.75 \cdot P_c$:

$$R_n = 0.60 \cdot F_y \cdot d_c \cdot t_w \cdot \left(1 + \frac{3 \cdot b_{cf} \cdot {t_{cf}}^2}{d_b \cdot d_c \cdot t_w}\right)$$

For $P_r > 0.75 \cdot P_c$:

$$R_n = 0.60 \cdot F_y \cdot d_c \cdot t_w \cdot \left(1 + \frac{3 \cdot b_{cf} \cdot {t_{cf}}^2}{d_b \cdot d_c \cdot t_w}\right) \cdot \left(1.9 - \frac{1.2 \cdot P_r}{P_c}\right)$$

When required, doubler plate(s) or a pair of diagonal stiffeners shall be provided within the boundaries of the rigid connection whose webs lie in a common plane.

In the above equations:

A= column cross-sectional area

b_{cf} = width of column flange

d_b = Beam depth

d_c = Column depth

F_c = specified minimum yield stress of the column web

$P_c = P_y$ $(LRFD)$

$P_c = 0.6 \cdot P_y$ (ASD)

P_r = Required strength

$P_y = F_y \cdot A$ = axial yield strength of the column

t_{cf} = thickness of the column flange

t_w = column web thickness

8.3.7 Unframed Ends of Beams and Girders

- At unframed ends of beams and girders not otherwise restrained against rotation about their longitudinal axes, a pair of transverse stiffeners, extending the full depth of the web, shall be provided.

8.3.8 Additional Stiffener Requirements for Concentrated Forces

Stiffeners required to resist tensile concentrated forces shall be designed in accordance with the requirements of Chapter D and welded to the loaded flange and the web. The welds to the flange shall be sized for the difference between the required strength and available limit state strength. The stiffener to web welds shall be sized to transfer to the web the algebraic difference in tensile force at the ends of the stiffener.

Stiffeners required to resist compressive concentrated forces either bear on or be welded to the loaded flange and welded to the web. The welds to the flange shall be sized for the difference between the required strength and the applicable limit state strength. The weld to the web shall be sized to transfer to the web the algebraic difference in compression force at the ends of the stiffener.

The member properties shall be determined using an effective length of $0.75 \cdot h$ and a cross section composed of two stiffeners and a strip of the web having a width of $25 \cdot t_w$ at interior stiffeners and $12 \cdot t_w$ at the ends of members. The weld connecting full depth bearing stiffeners to the web shall be sized to transmit the difference in compressive force at each of the stiffeners to the web.

Transverse and diagonal stiffeners shall comply with the following additional criteria:

i. The width of each stiffener plus one-half the thickness of the column web shall not be less than one-third of the width of the flange or moment connection plate delivering the concentrated force.

ii. The thickness of a stiffener shall not be less than one-half the thickness of the flange or moment connection plate delivering the concentrated load, and greater than or equal to the width divided by 15.

iii. Transverse stiffeners shall extend a minimum of one-half the depth of the member except as required in sections 3.8.5 and 3.8.7 above.

8.3.9 Additional Doubler Plate Requirements for Concentrated Forces

Doubler plates shall comply with the following criteria:

i. The thickness and extent of the doubler plate shall provide the additional material necessary to equal or exceed the strength requirements.

ii. The doubler plate shall be welded to develop the proportion of the total force transmitted to the doubler plate.

Quiz: Corner waler connections

You have a 90 degree corner wale to wale connection with W24x120 beams. Should you specify web stiffeners plates?

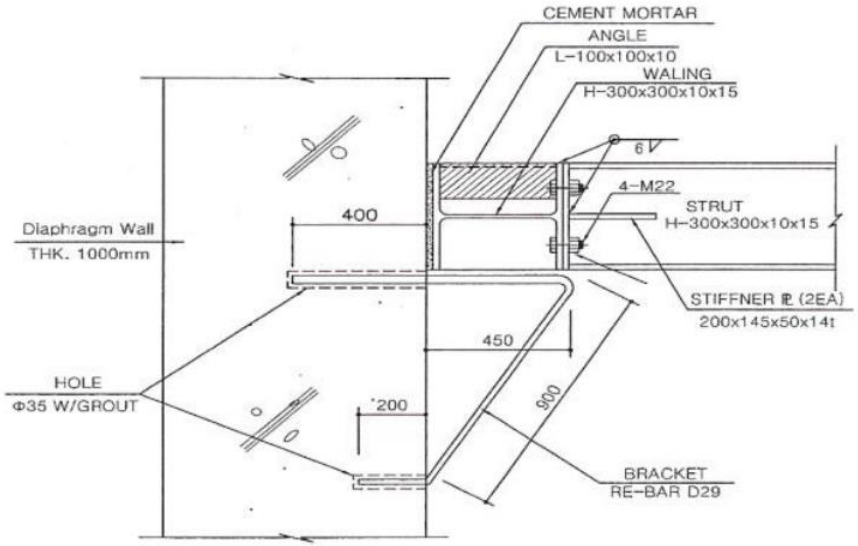
a) Yes

b) No

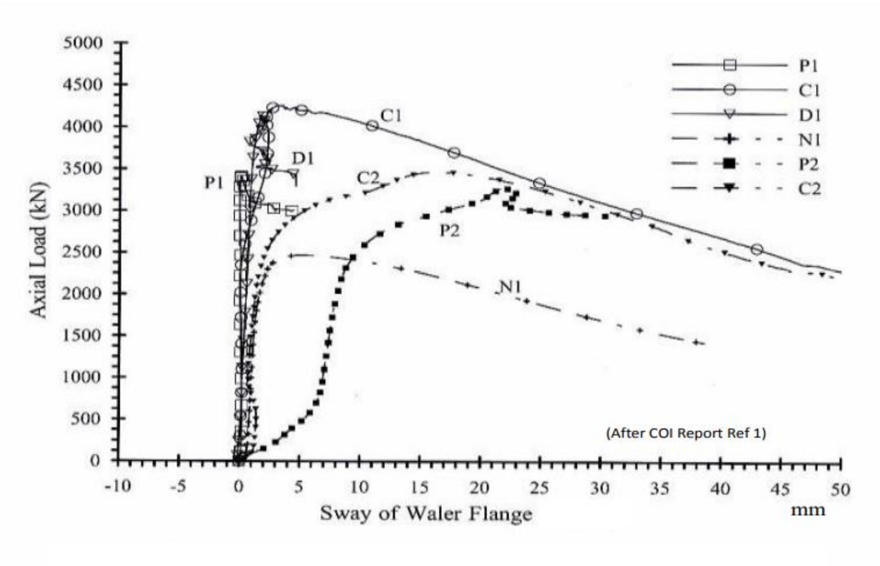
8.4.1 Steel Connections Done Wrong #1

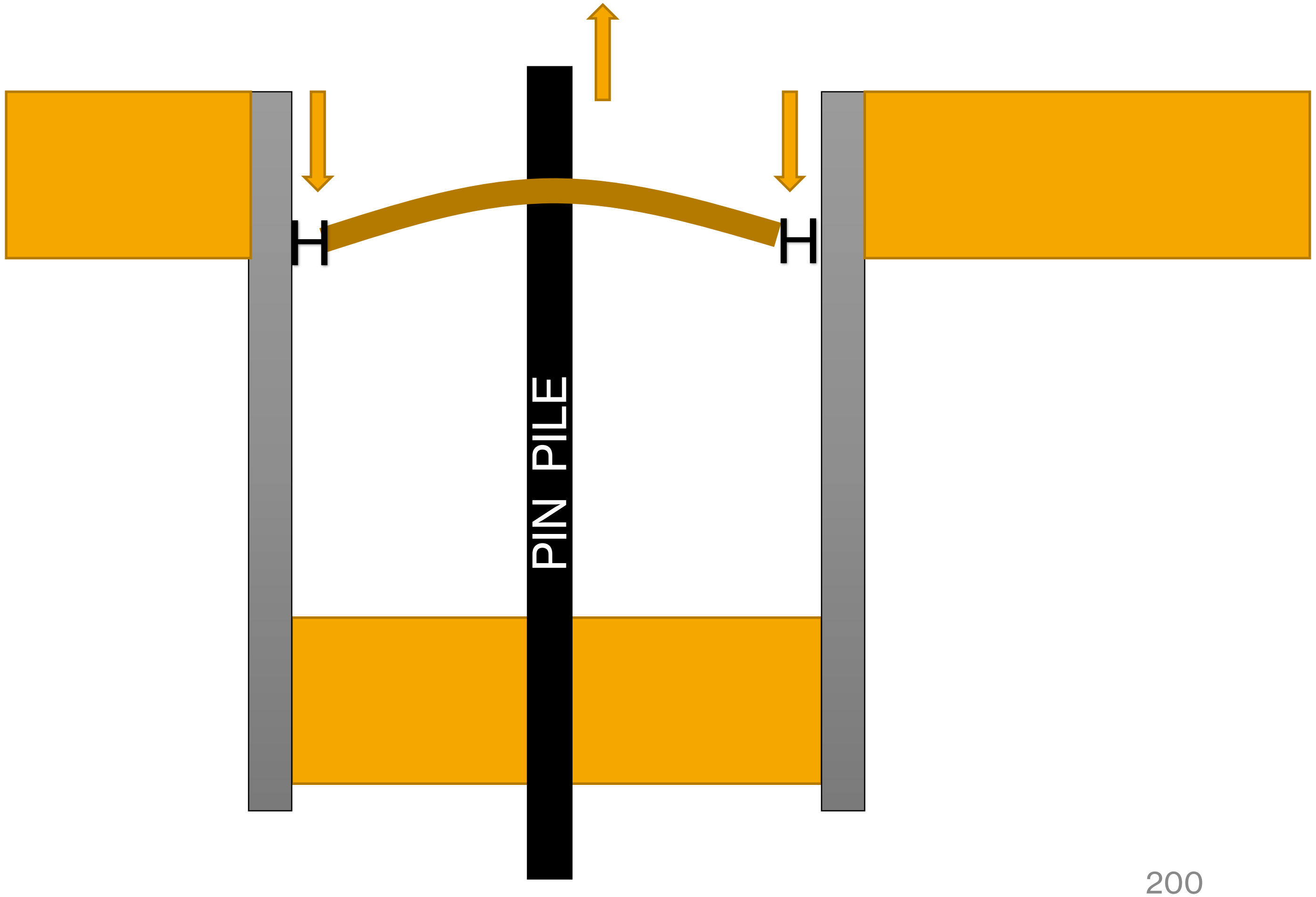
- Nicoll highway collapse

J. Endicott, 2004



8.4.2 Steel Connections Done Wrong #2





8.4.3 Steel Connection Gone Wrong #2

14th & H Streets, Washington DC, 1990

8.4.4 Steel Connections Gone Wrong #2

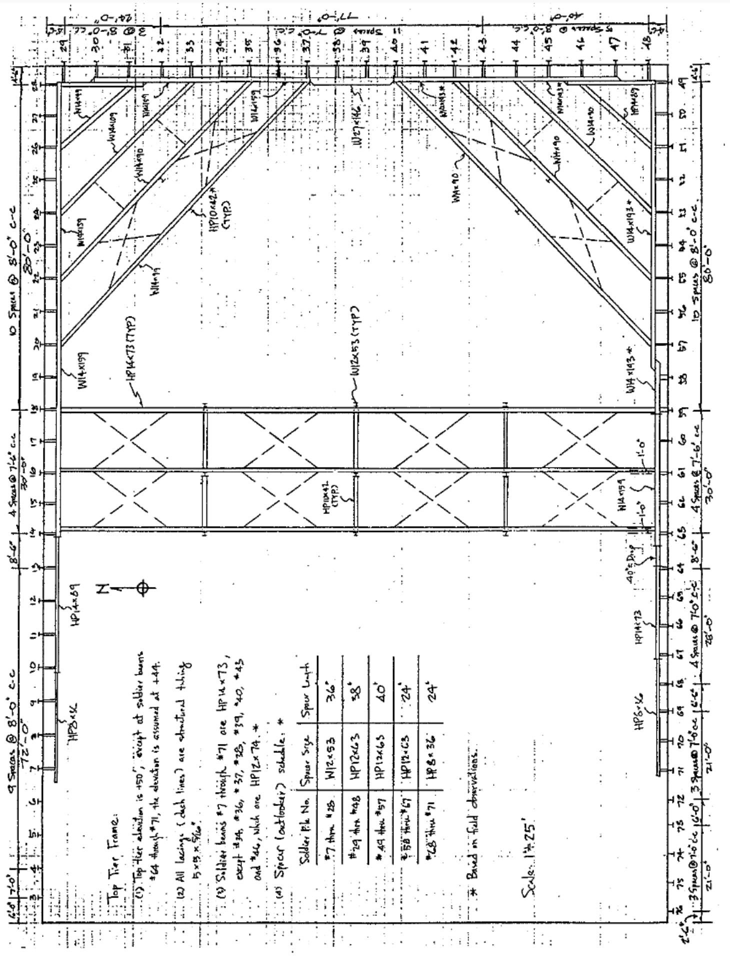
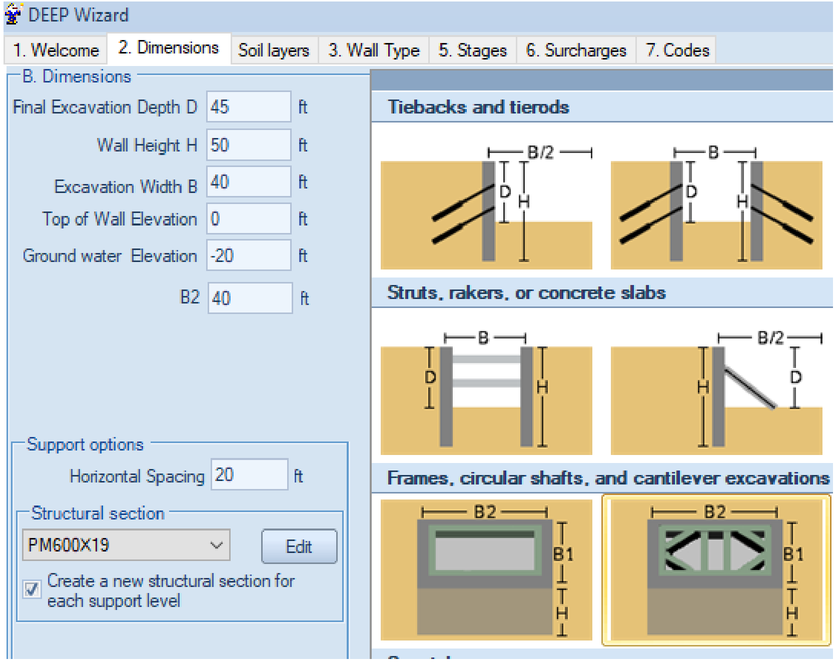
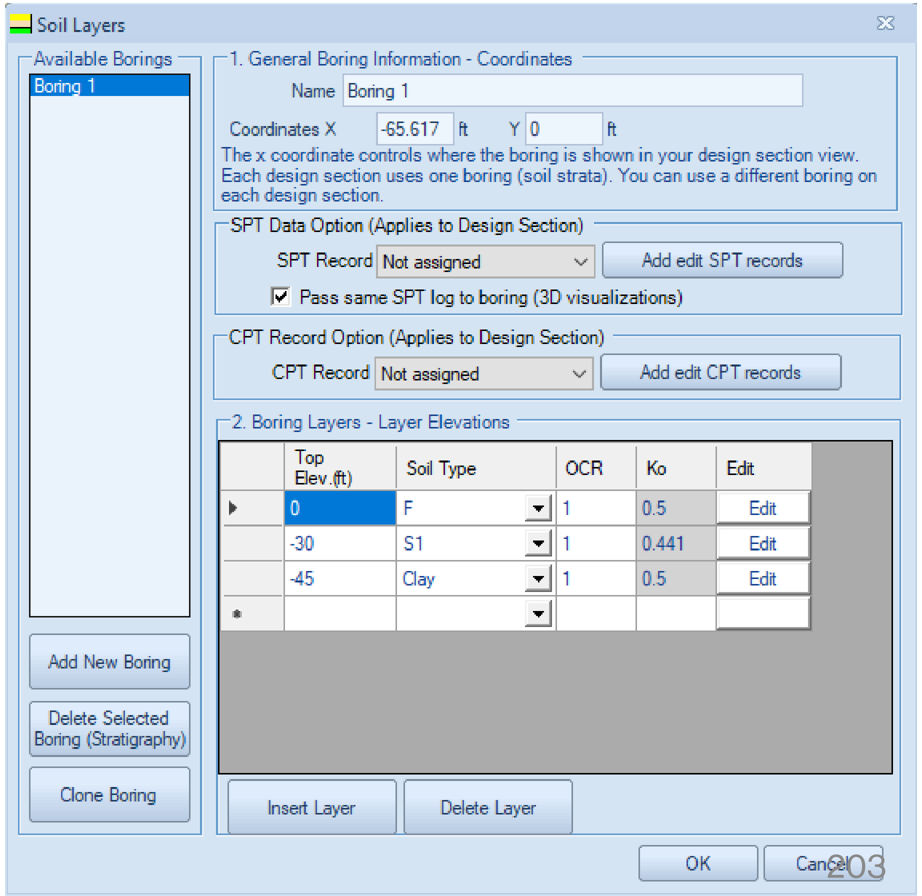


Figure 3.45 As-built Condition of the Top Tier Internal Support System.

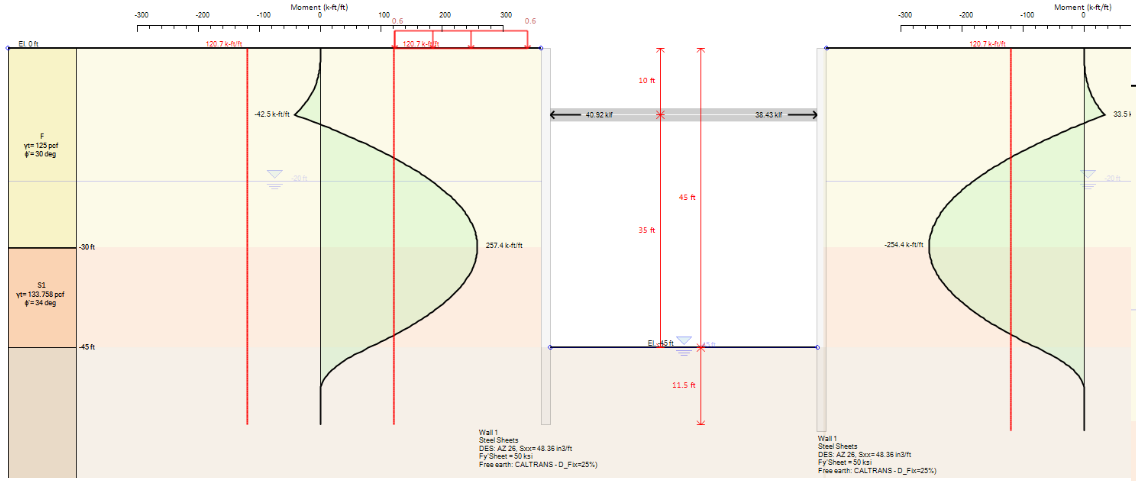
8.5.1 Steel Connection Example

- Create deep excavation box 40ft x 40ft, sheet piles
- 35 ft deep, one support at -10ft

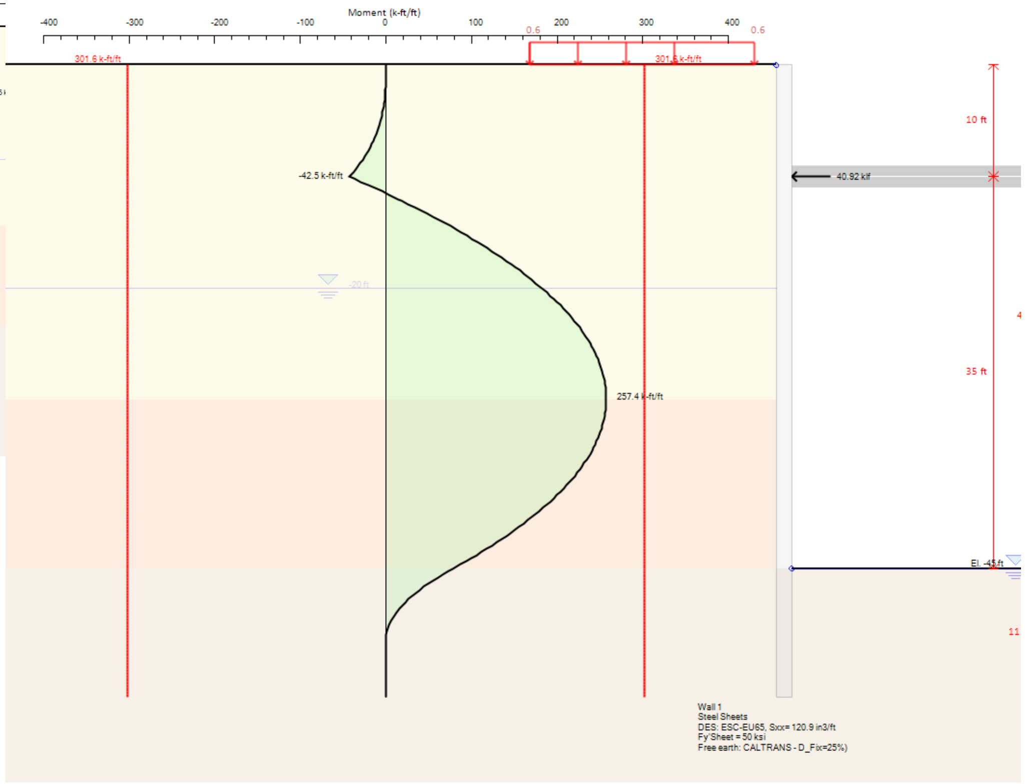




8.5.2 Steel Connection Example

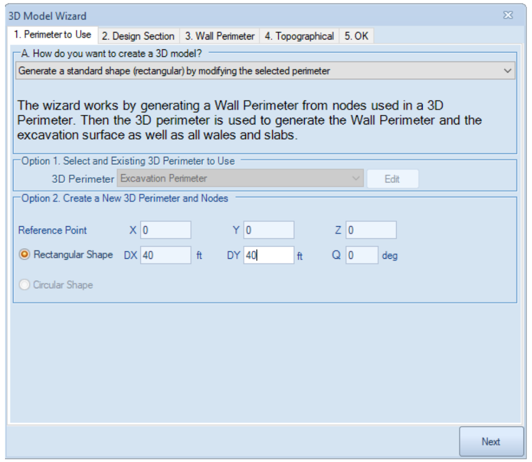


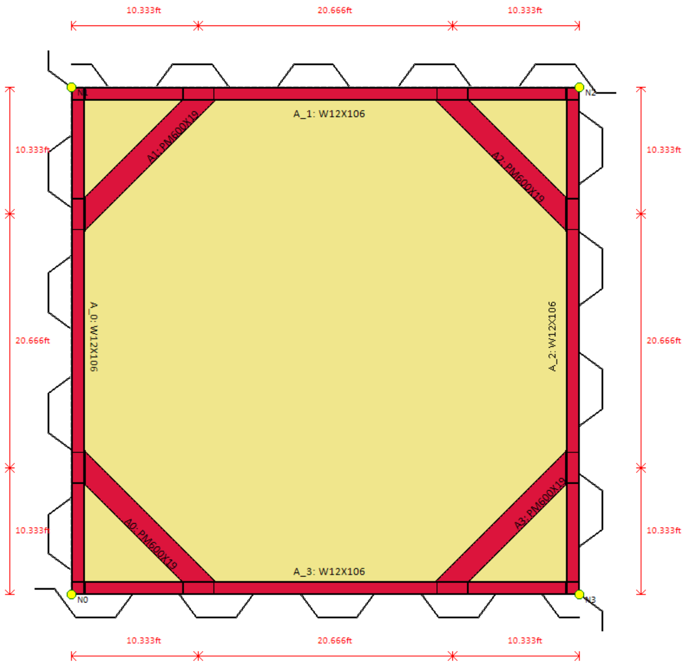
Optimize initial design section
Wall embedment FS >=1.3

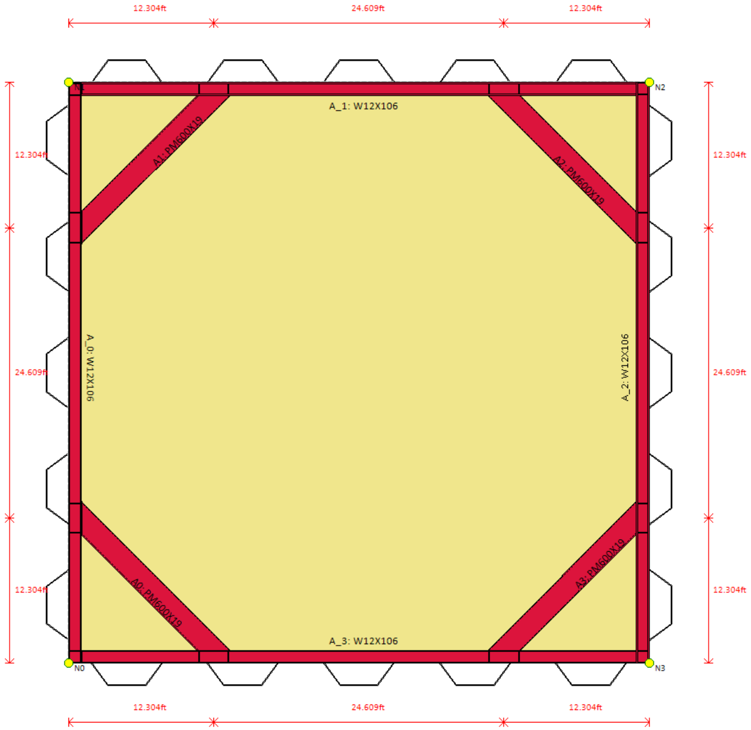


9.5.3 Steel Connections – Top View

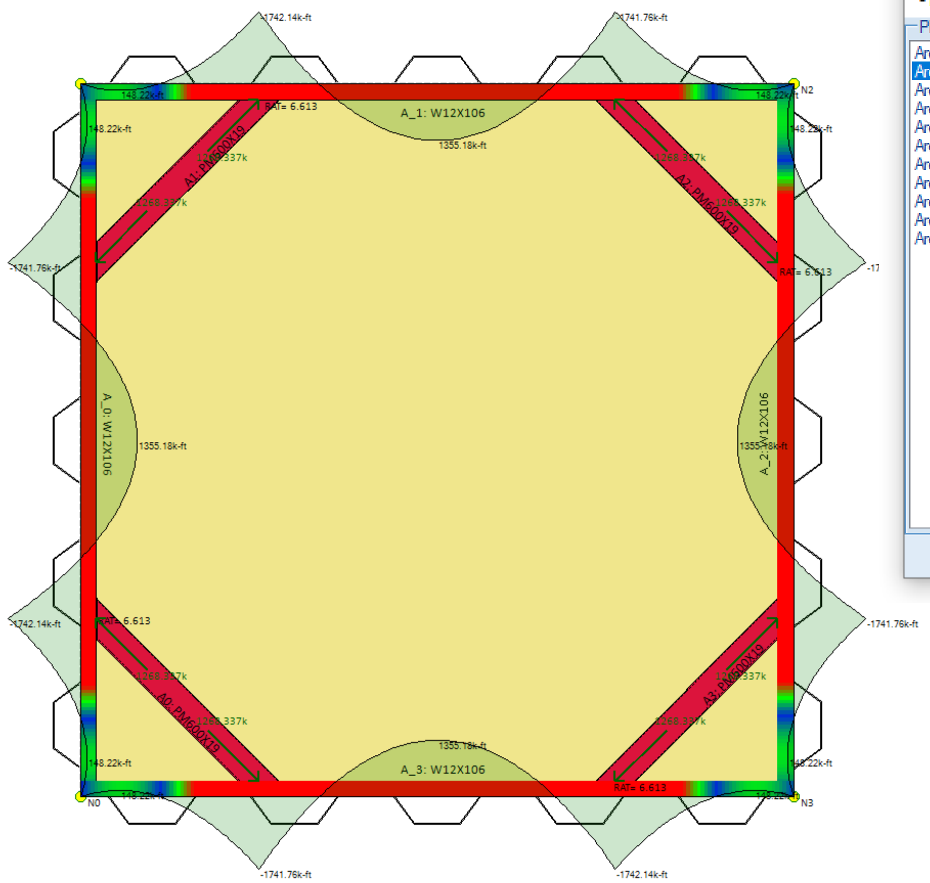
- Resize excavation for sheet pile sizes

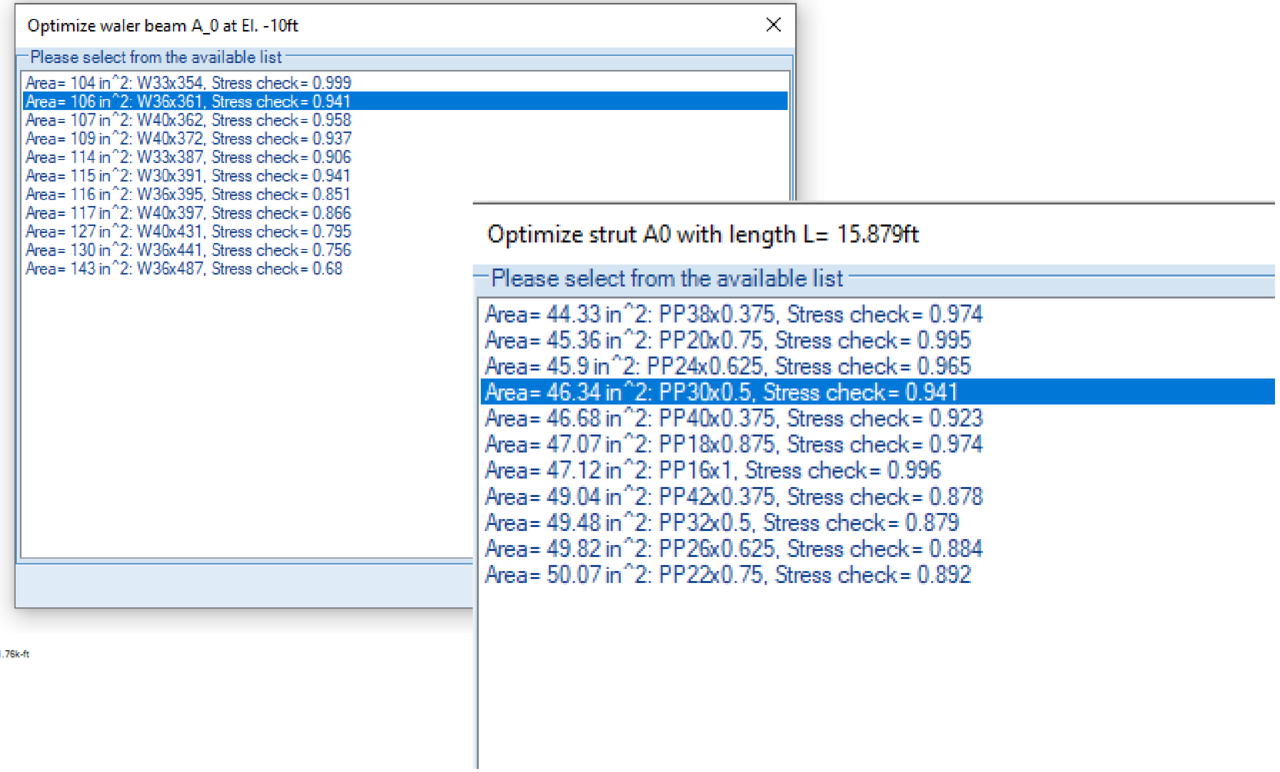




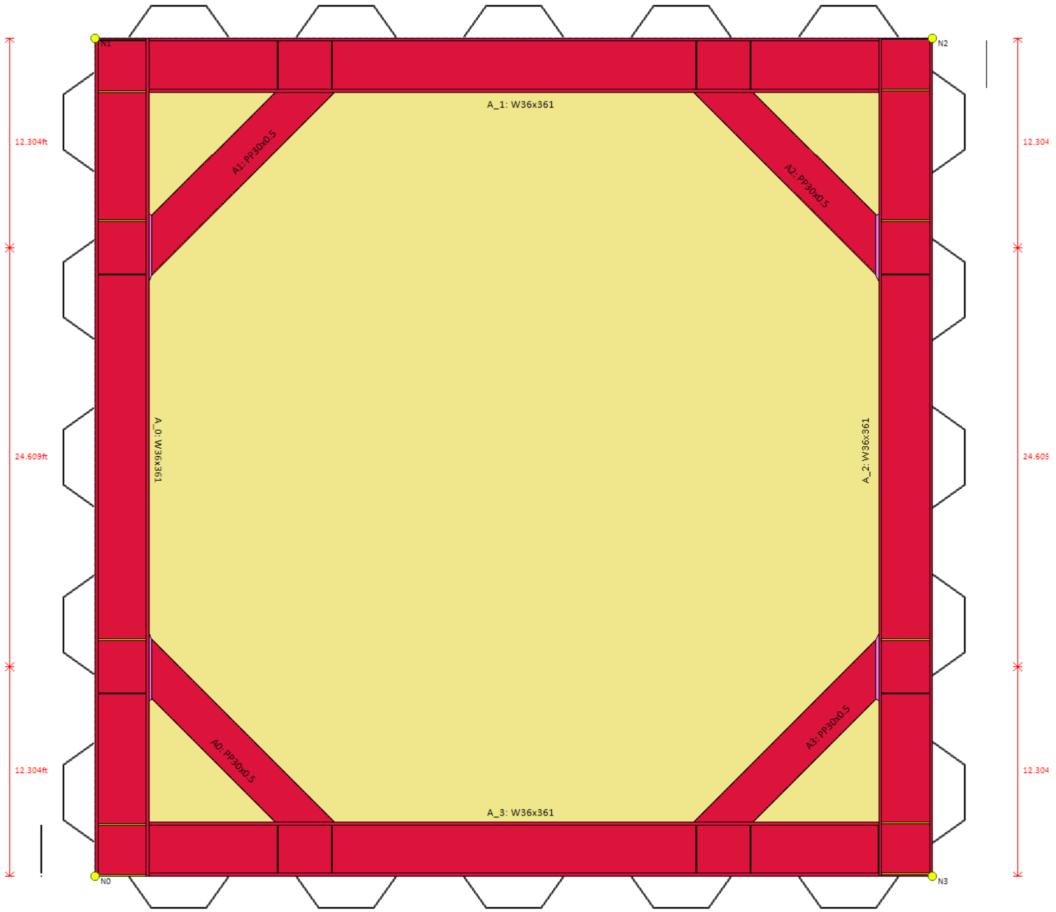


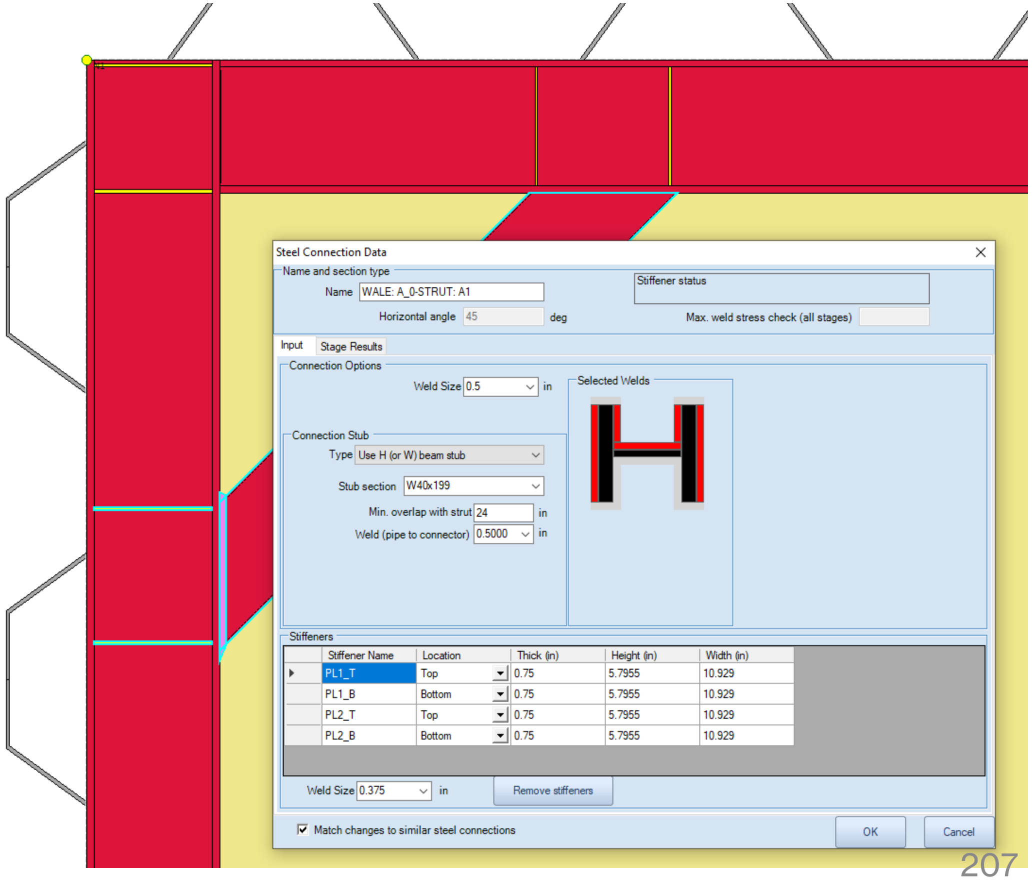
8.5.4 Optimize Walers and Braces



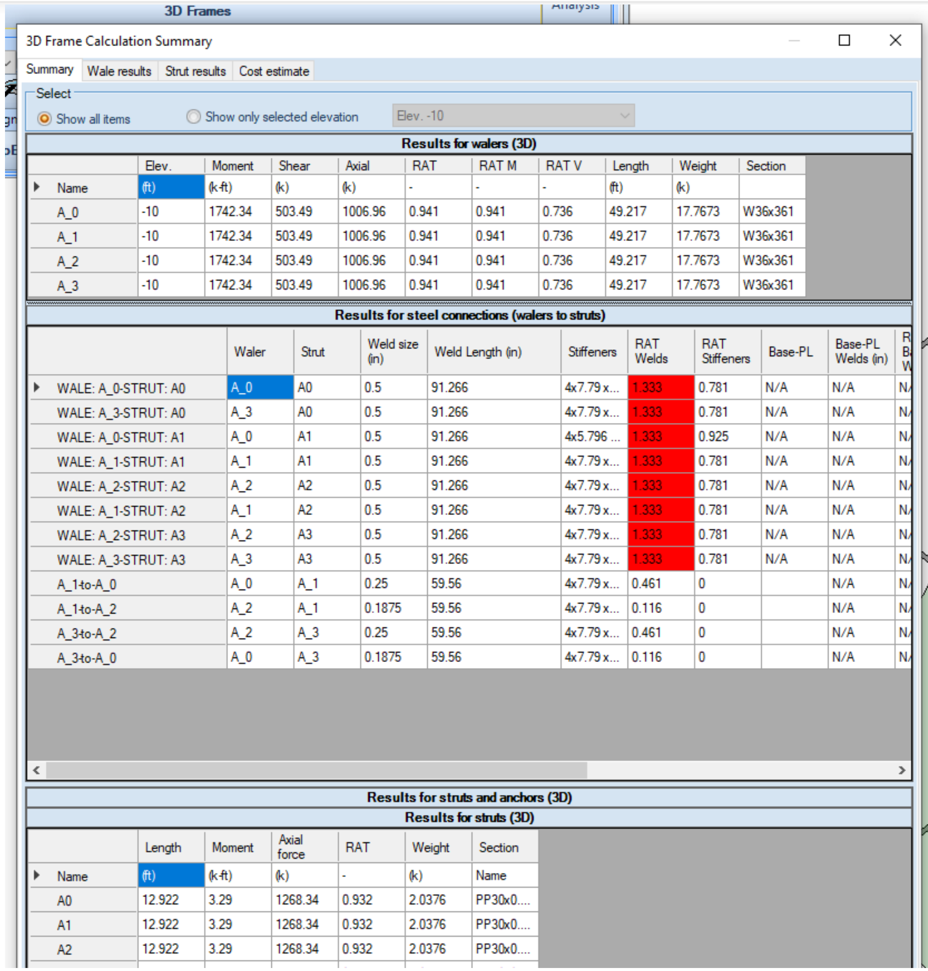


8.5.6 Optimized Bracing (not connections)





8.5.7 Steel Connection Results



3D Frames

3D Frame Calculation Summary

Summary | Wale results | Strut results | Cost estimate

Select

Show all items   Show only selected elevation   Elev. -10

Results for walers (3D)

Name	Elev. (ft)	Moment (k-ft)	Shear (k)	Axial (k)	RAT -	RAT M -	RAT V -	Length (ft)	Weight (k)	Section
A_0	-10	1742.34	503.49	1006.96	0.941	0.941	0.736	49.217	17.7673	W36x361
A_1	-10	1742.34	503.49	1006.96	0.941	0.941	0.736	49.217	17.7673	W36x361
A_2	-10	1742.34	503.49	1006.96	0.941	0.941	0.736	49.217	17.7673	W36x361
A_3	-10	1742.34	503.49	1006.96	0.941	0.941	0.736	49.217	17.7673	W36x361

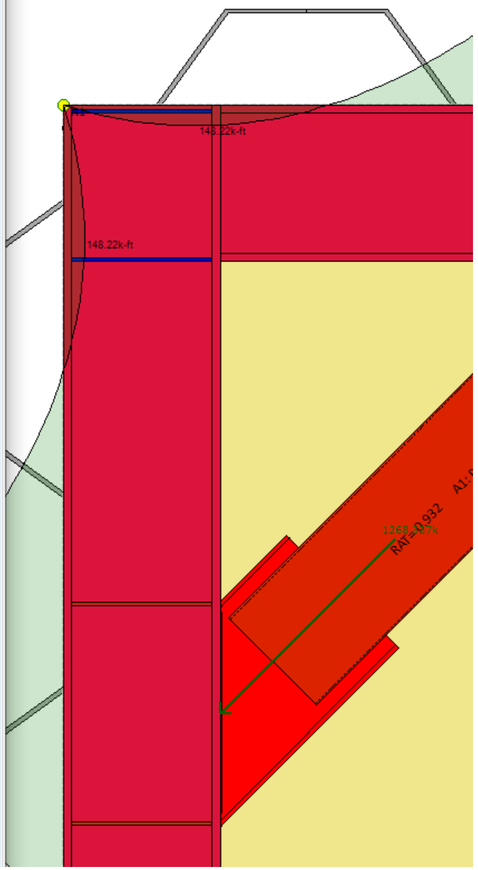
Results for steel connections (walers to struts)

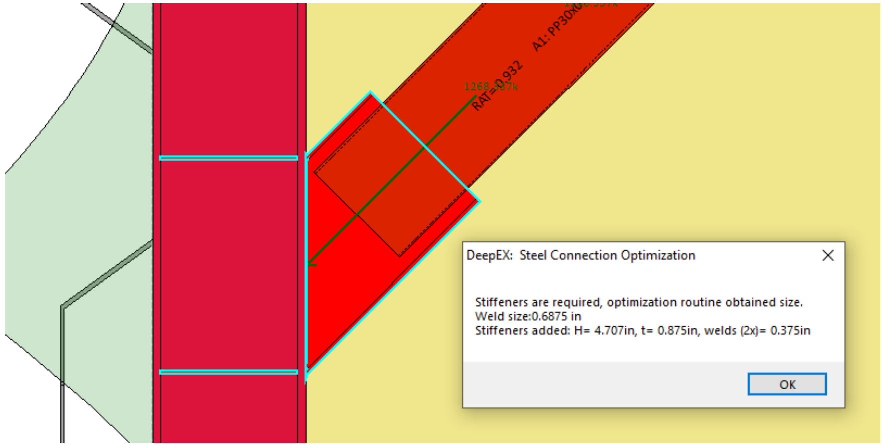
	Waler	Strut	Weld size (in)	Weld Length (in)	Stiffeners	RAT Welds	RAT Stiffeners	Base-PL	Base-PL Welds (in)	R B W
WALE: A_0-STRUT: A0	A_0	A0	0.5	91.266	4x7.79 x...	1.333	0.781	N/A	N/A	N/
WALE: A_3-STRUT: A0	A_3	A0	0.5	91.266	4x7.79 x...	1.333	0.781	N/A	N/A	N/
WALE: A_0-STRUT: A1	A_0	A1	0.5	91.266	4x5.796 ...	1.333	0.925	N/A	N/A	N/
WALE: A_1-STRUT: A1	A_1	A1	0.5	91.266	4x7.79 x...	1.333	0.781	N/A	N/A	N/
WALE: A_2-STRUT: A2	A_2	A2	0.5	91.266	4x7.79 x...	1.333	0.781	N/A	N/A	N/
WALE: A_1-STRUT: A2	A_1	A2	0.5	91.266	4x7.79 x...	1.333	0.781	N/A	N/A	N/
WALE: A_2-STRUT: A3	A_2	A3	0.5	91.266	4x7.79 x...	1.333	0.781	N/A	N/A	N/
WALE: A_3-STRUT: A3	A_3	A3	0.5	91.266	4x7.79 x...	1.333	0.781	N/A	N/A	N/
A_1-to-A_0	A_0	A_1	0.25	59.56	4x7.79 x...	0.461	0		N/A	N/
A_1-to-A_2	A_2	A_1	0.1875	59.56	4x7.79 x...	0.116	0		N/A	N/
A_3-to-A_2	A_2	A_3	0.25	59.56	4x7.79 x...	0.461	0		N/A	N/
A_3-to-A_0	A_0	A_3	0.1875	59.56	4x7.79 x...	0.116	0		N/A	N/

Results for struts and anchors (3D)

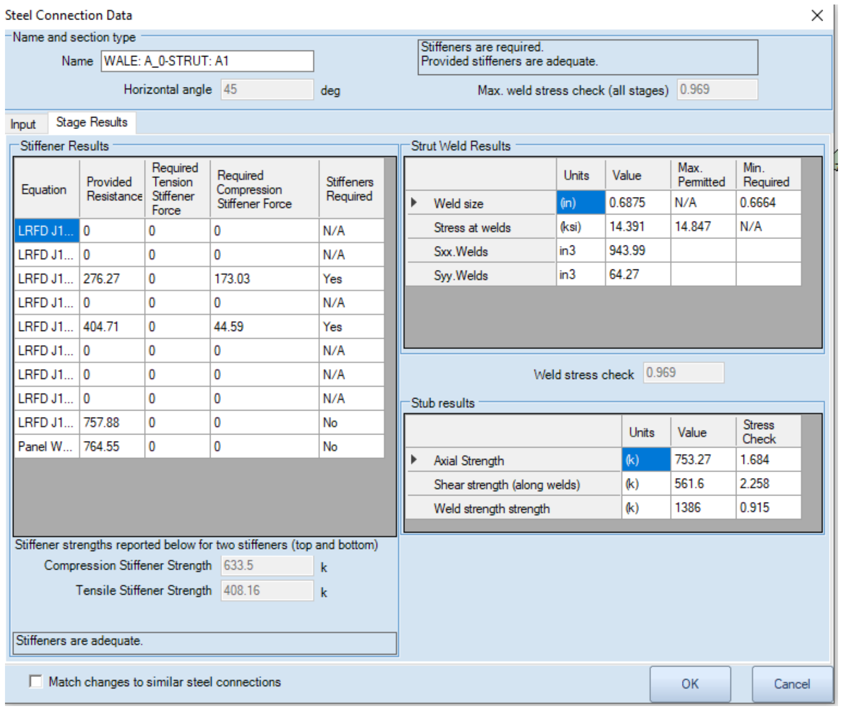
Results for struts (3D)

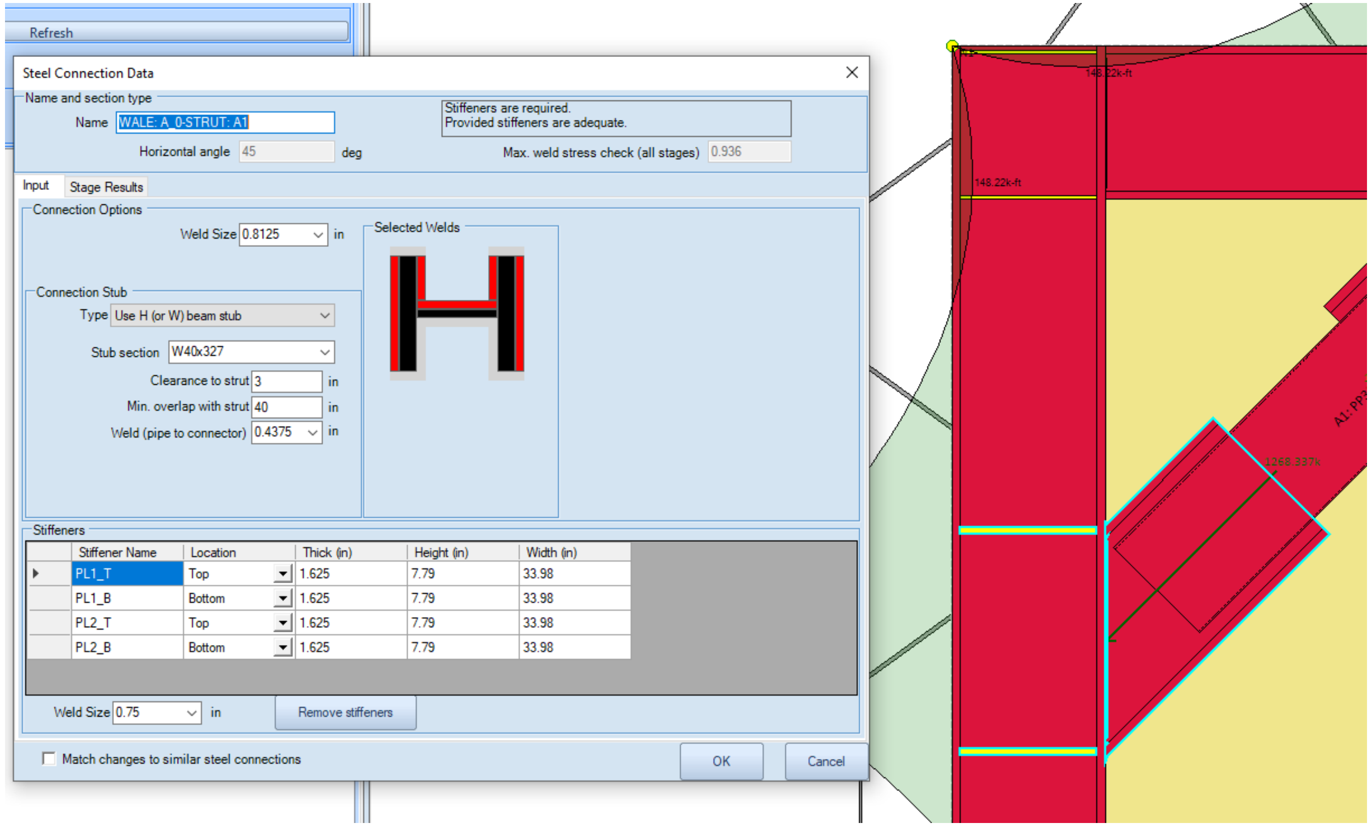
Name	Length (ft)	Moment (k-ft)	Axial force (k)	RAT -	Weight (k)	Section Name
A0	12.922	3.29	1268.34	0.932	2.0376	PP30x0....
A1	12.922	3.29	1268.34	0.932	2.0376	PP30x0....
A2	12.922	3.29	1268.34	0.932	2.0376	PP30x0....



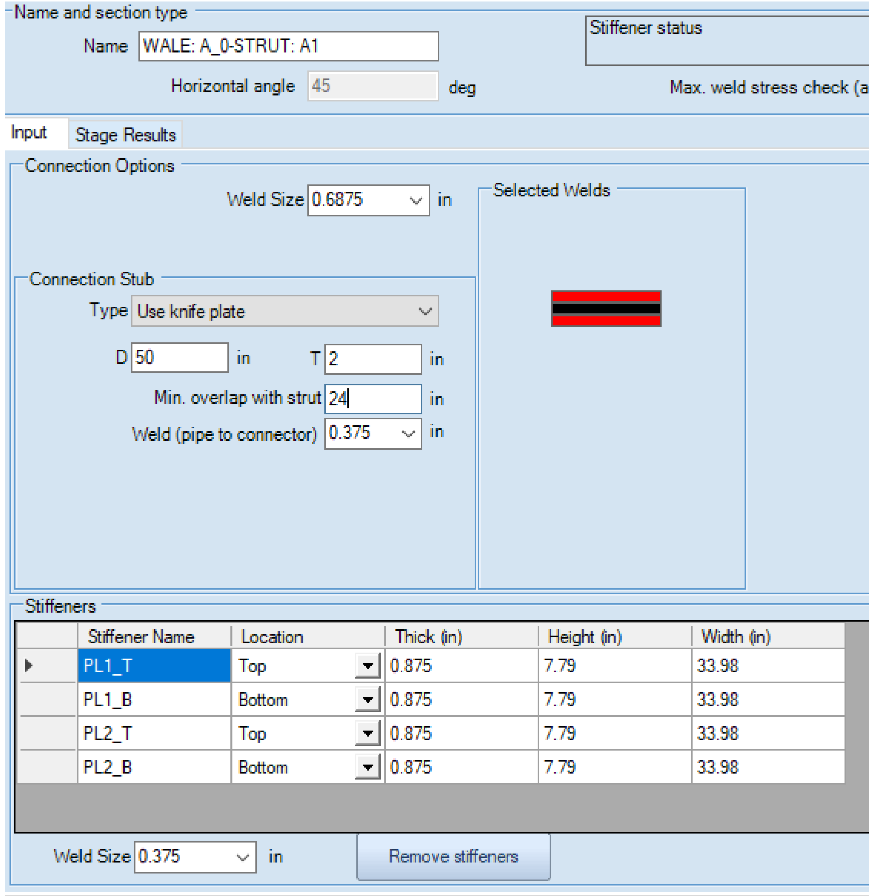


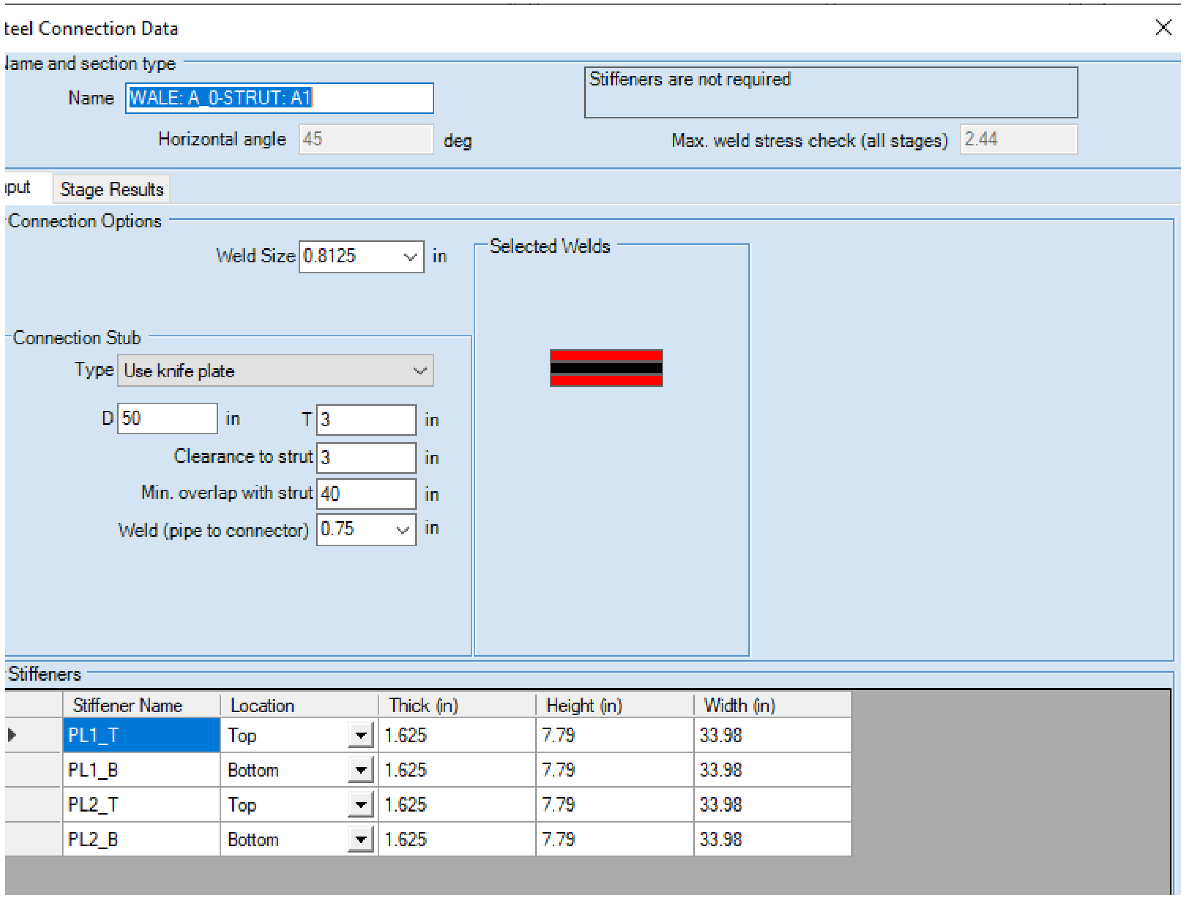
8.5.8 Sample Steel Connection Results



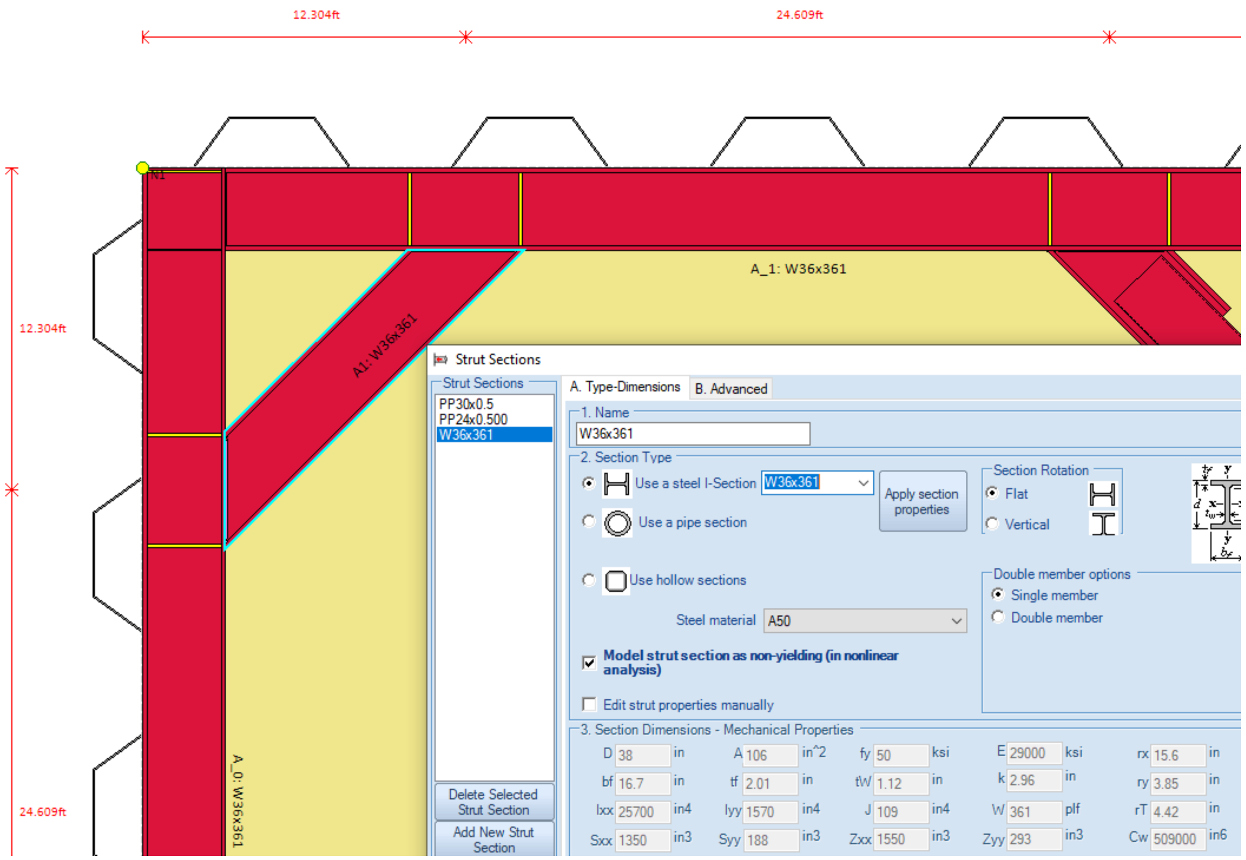


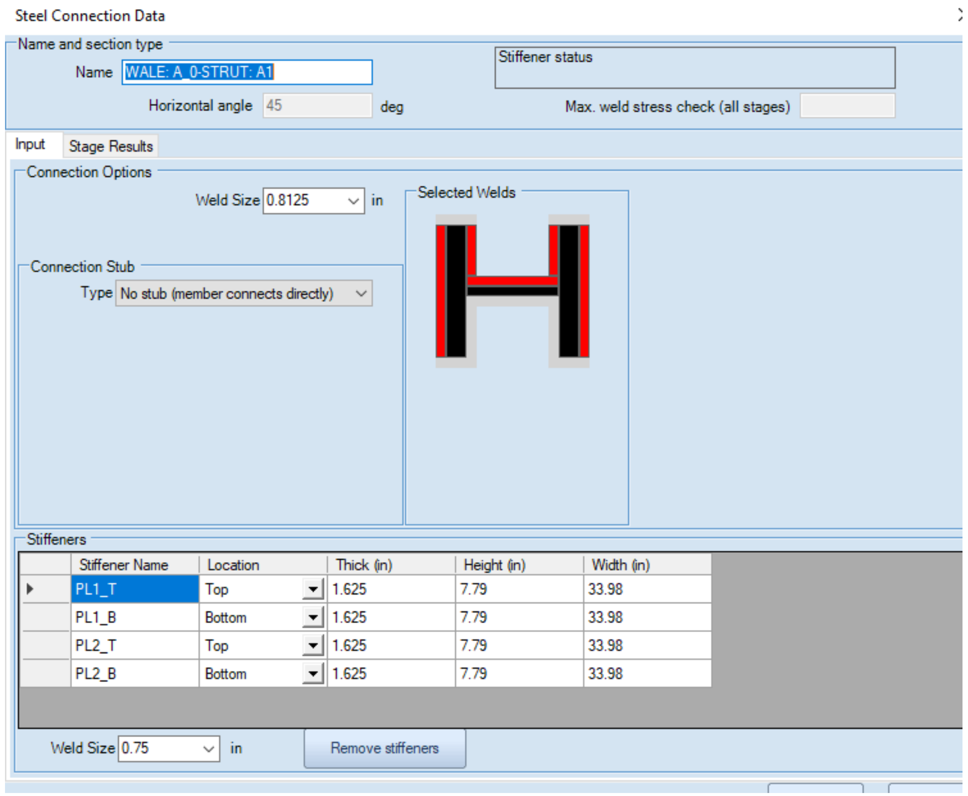
8.5.9 Knife Plate Connection





8.5.10 Keep it Simple



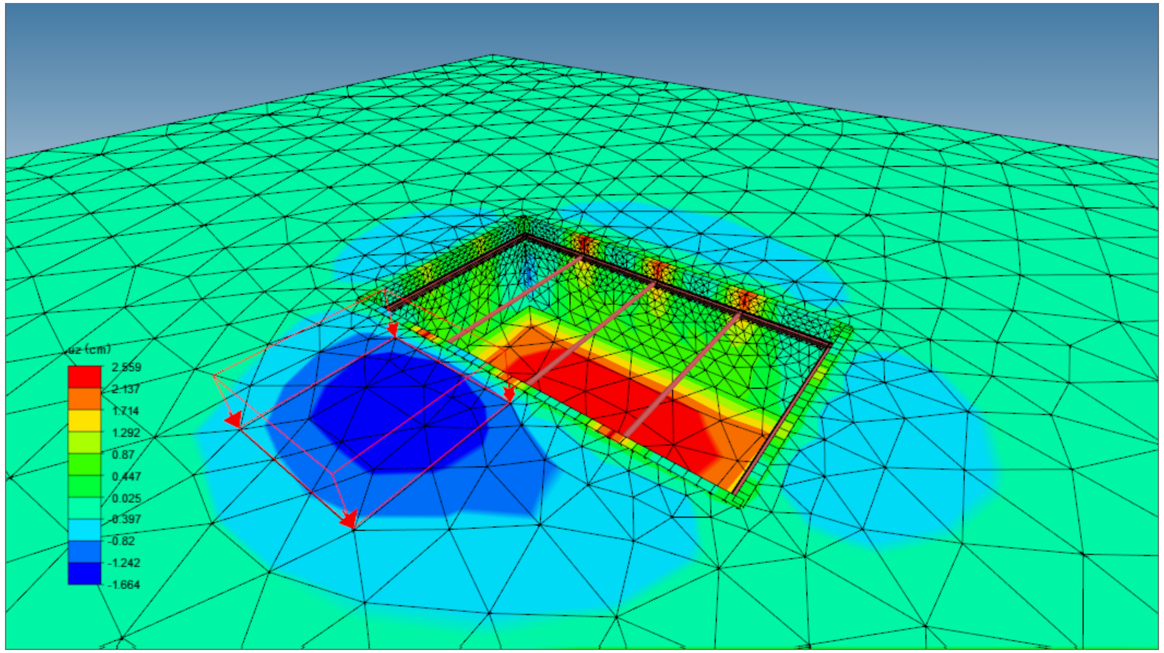


8.6 Frame Options to reduce moments

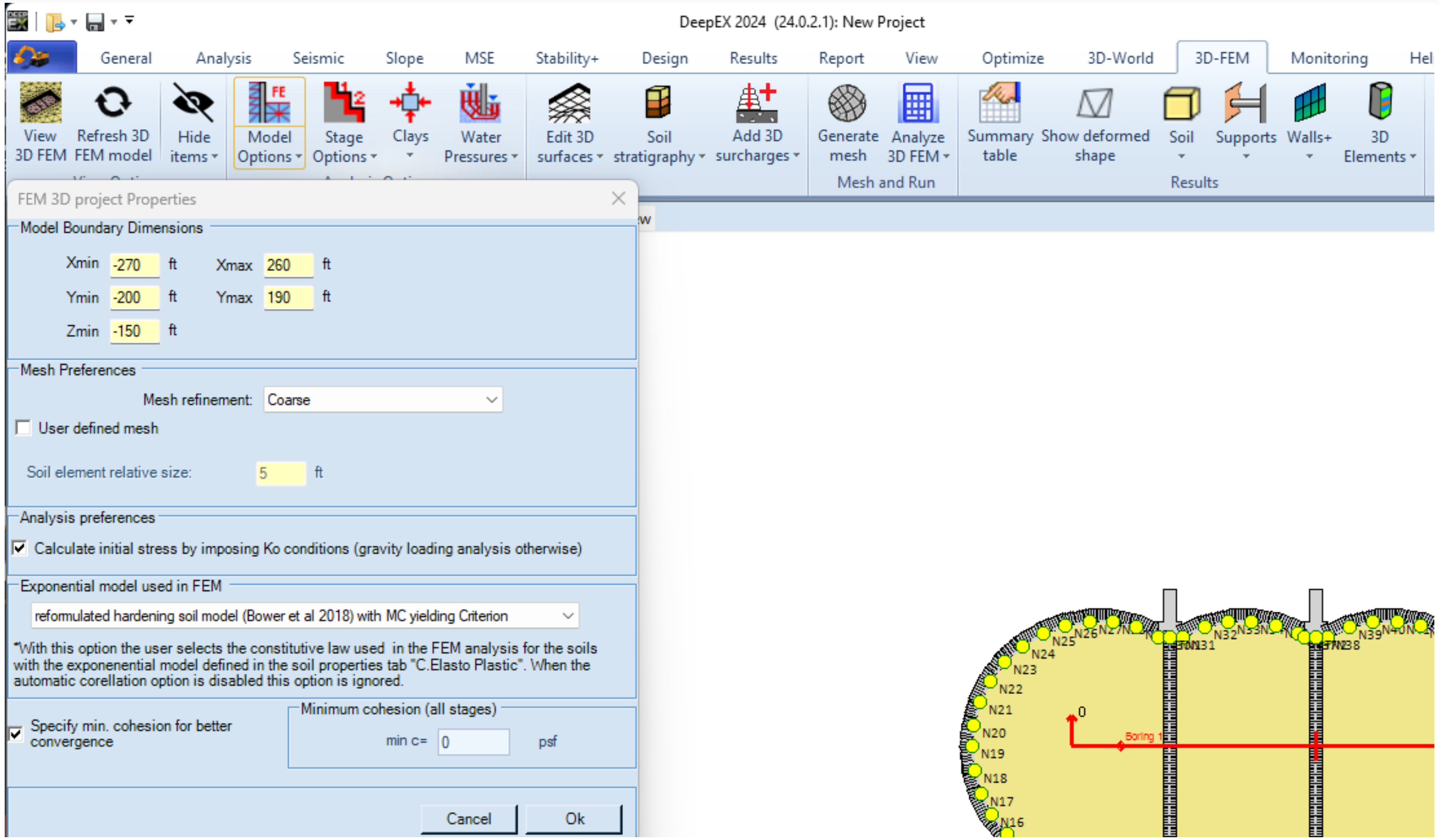
- Corner arching effects (EAB)
- Clear span points on walers at struts (mid point or edges)
- Waler points of rotation
- Splays

9.0 3D FEM Analysis

- Wizards generate quick models
- Wall perimeters – wall segments
- Walers
- Struts
- Anchors
- External 3D loads

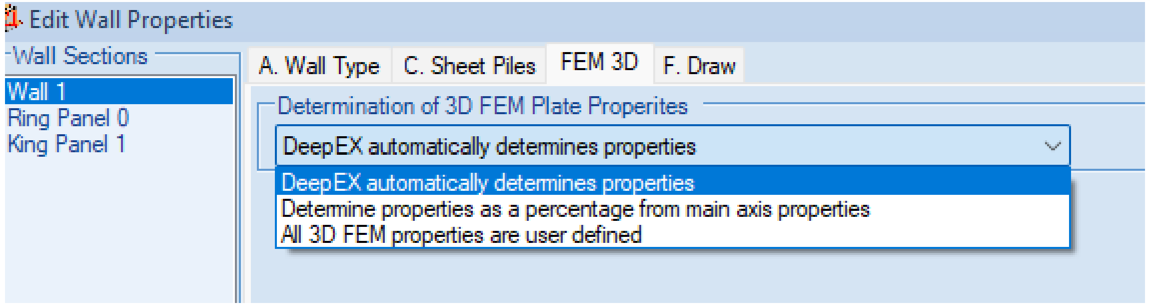


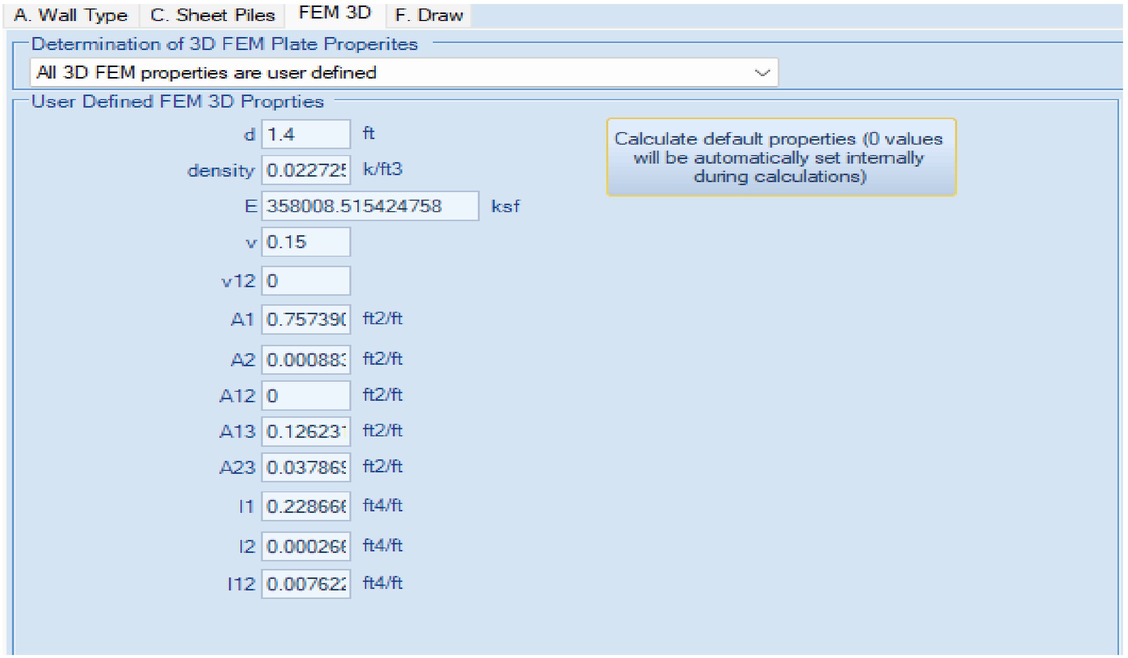
9.1.0 General FEM3D Settings



9.2.0 Wall Properties for 3D FEM

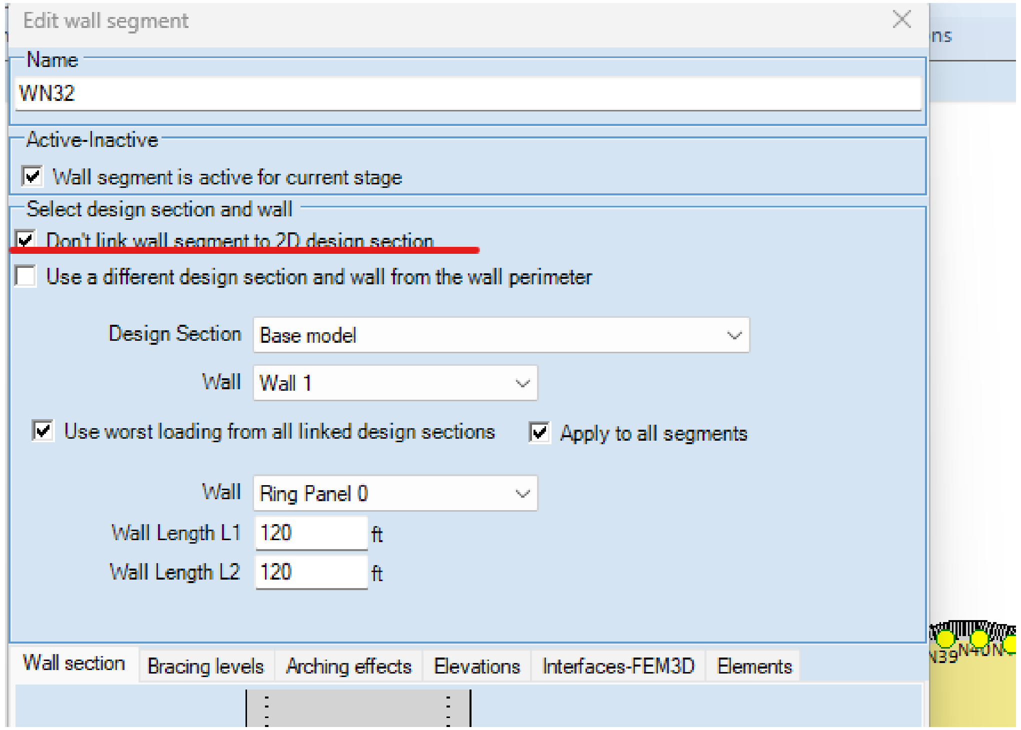
- FEM3D tab on wall sections



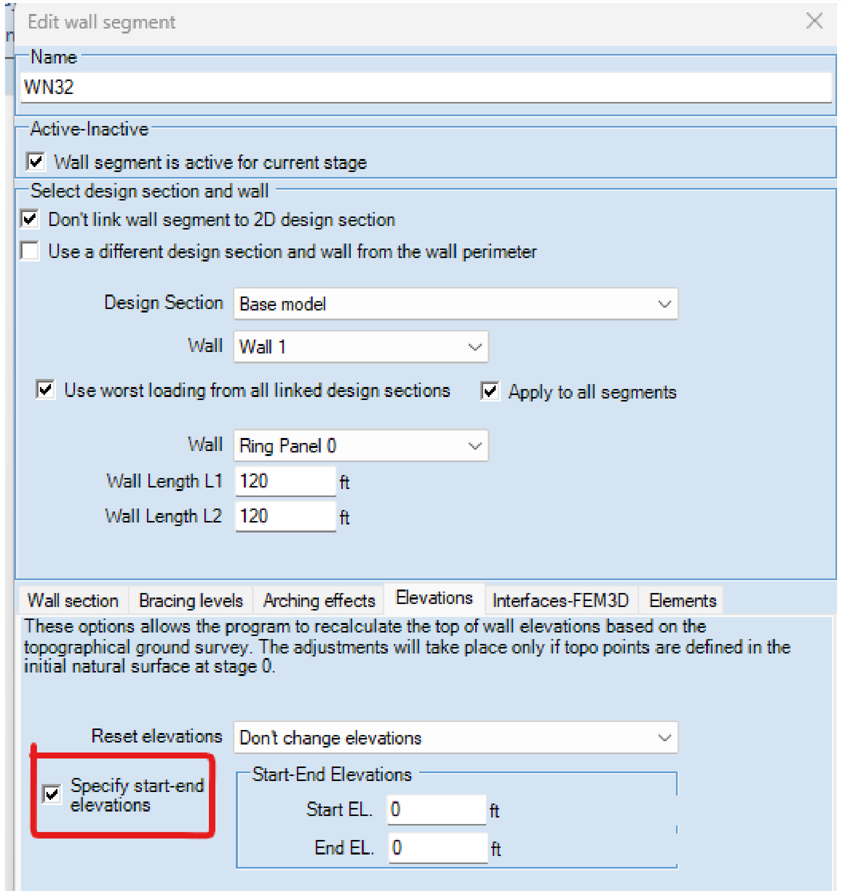


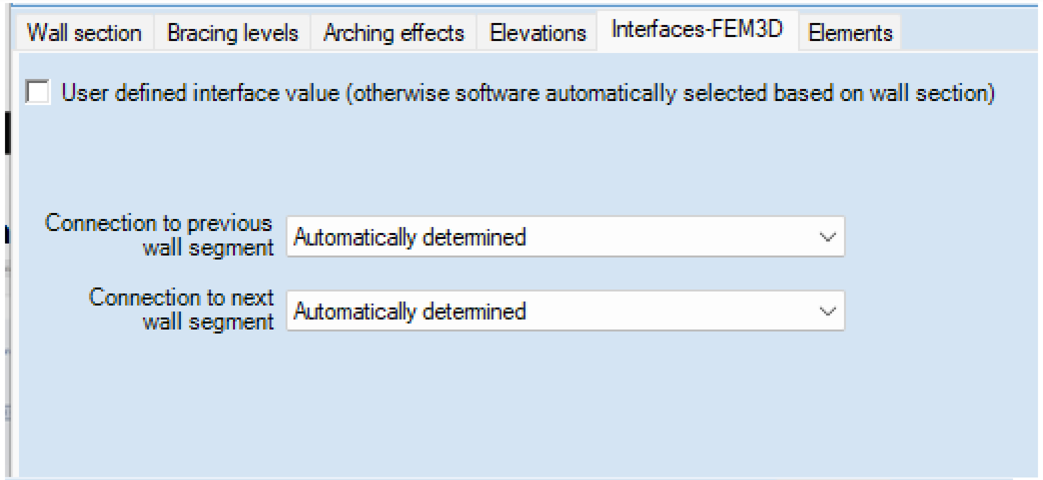
9.3.1 Wall Perimeter – Segment Options

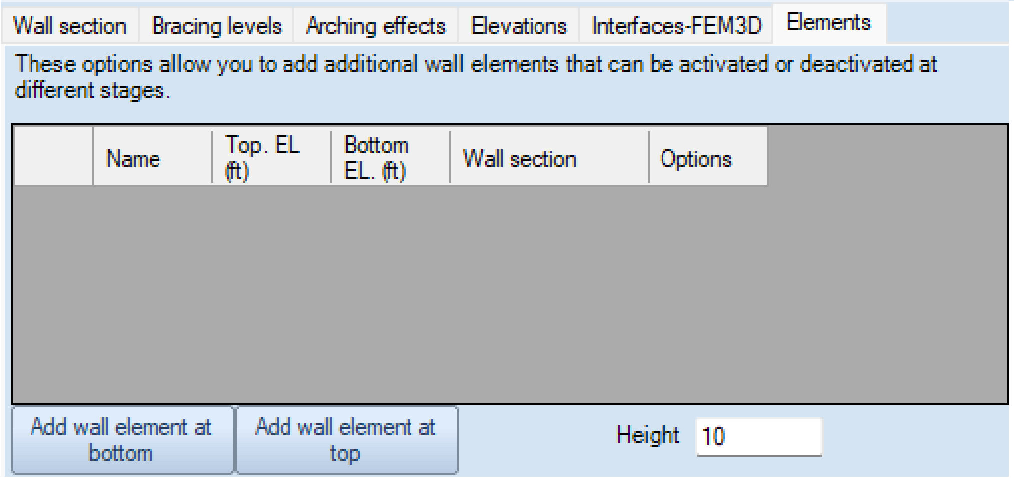
- Unlink from 2D design section (for structural properties)
- Length options (wall depth)



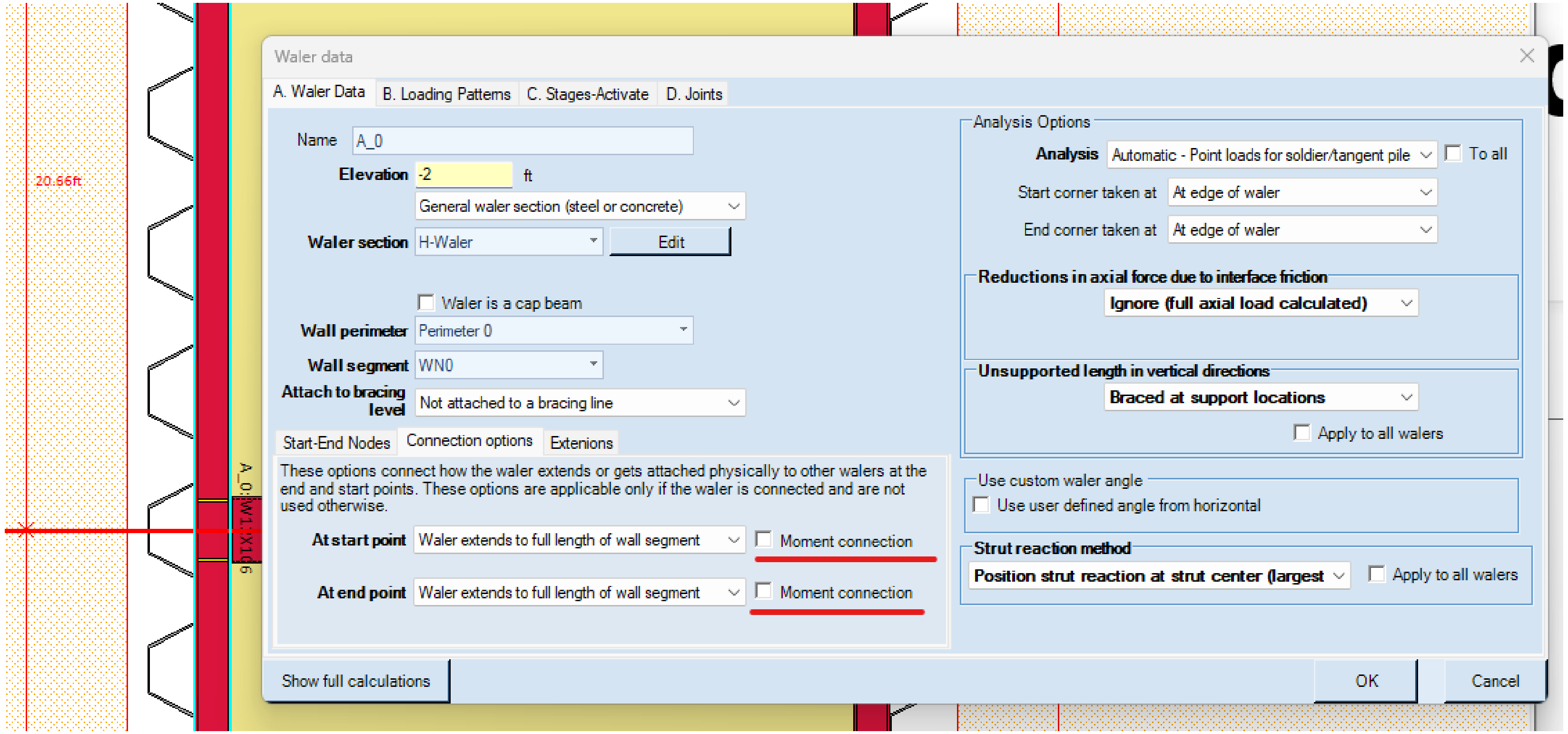
9.3.2 Elevations, Interfaces, Elements





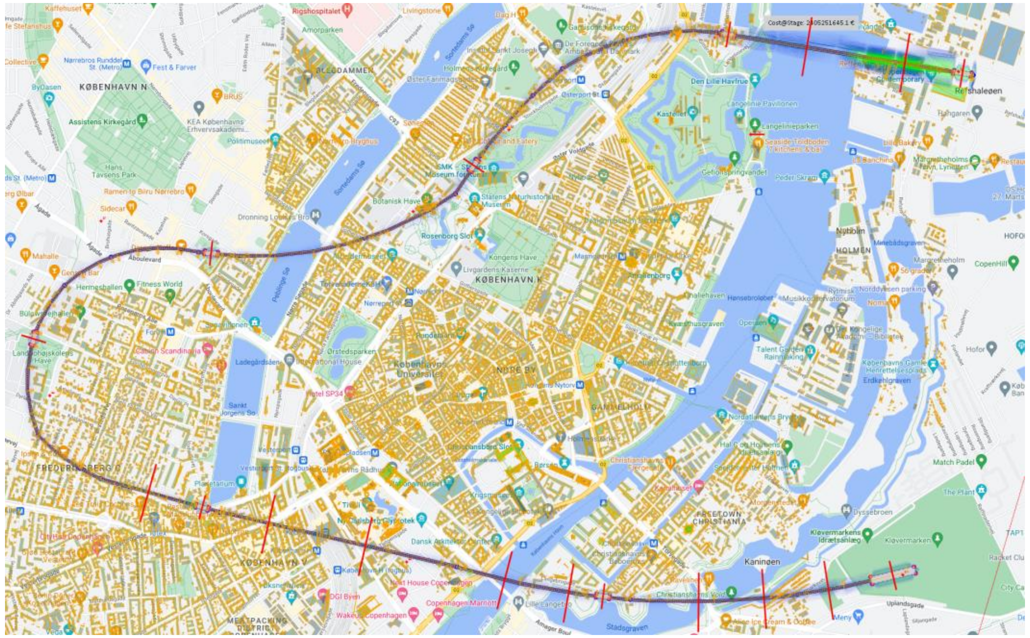


9.4.0 Wale Segment End Connections

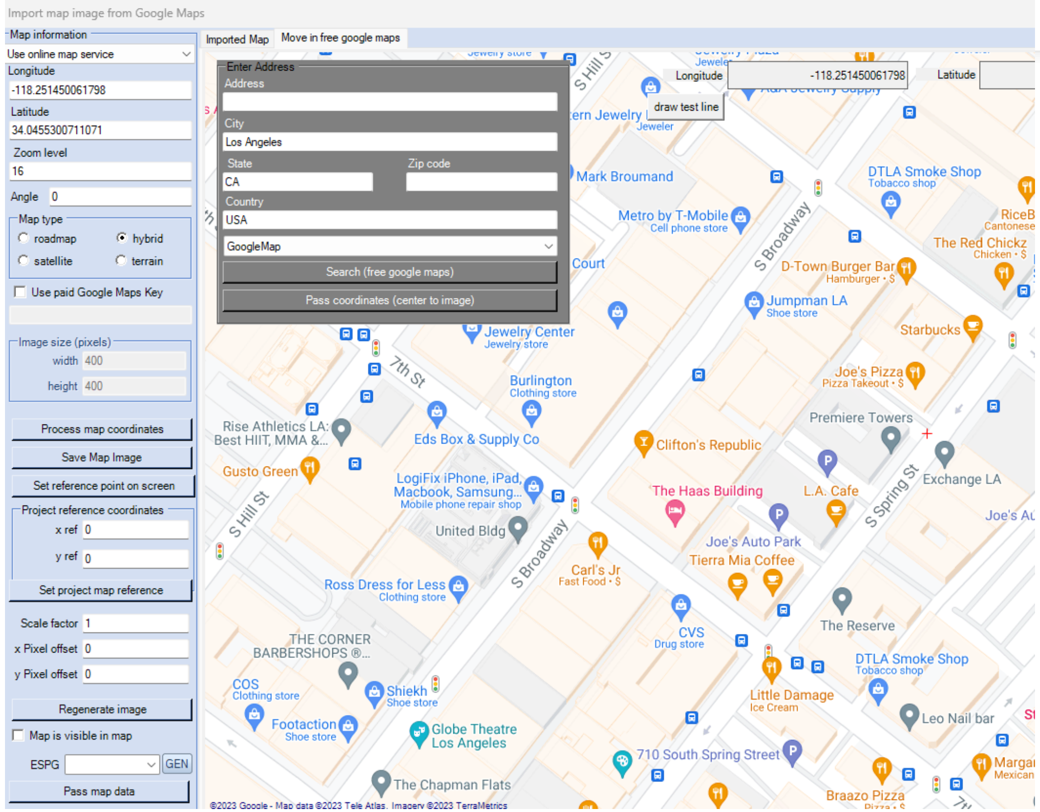


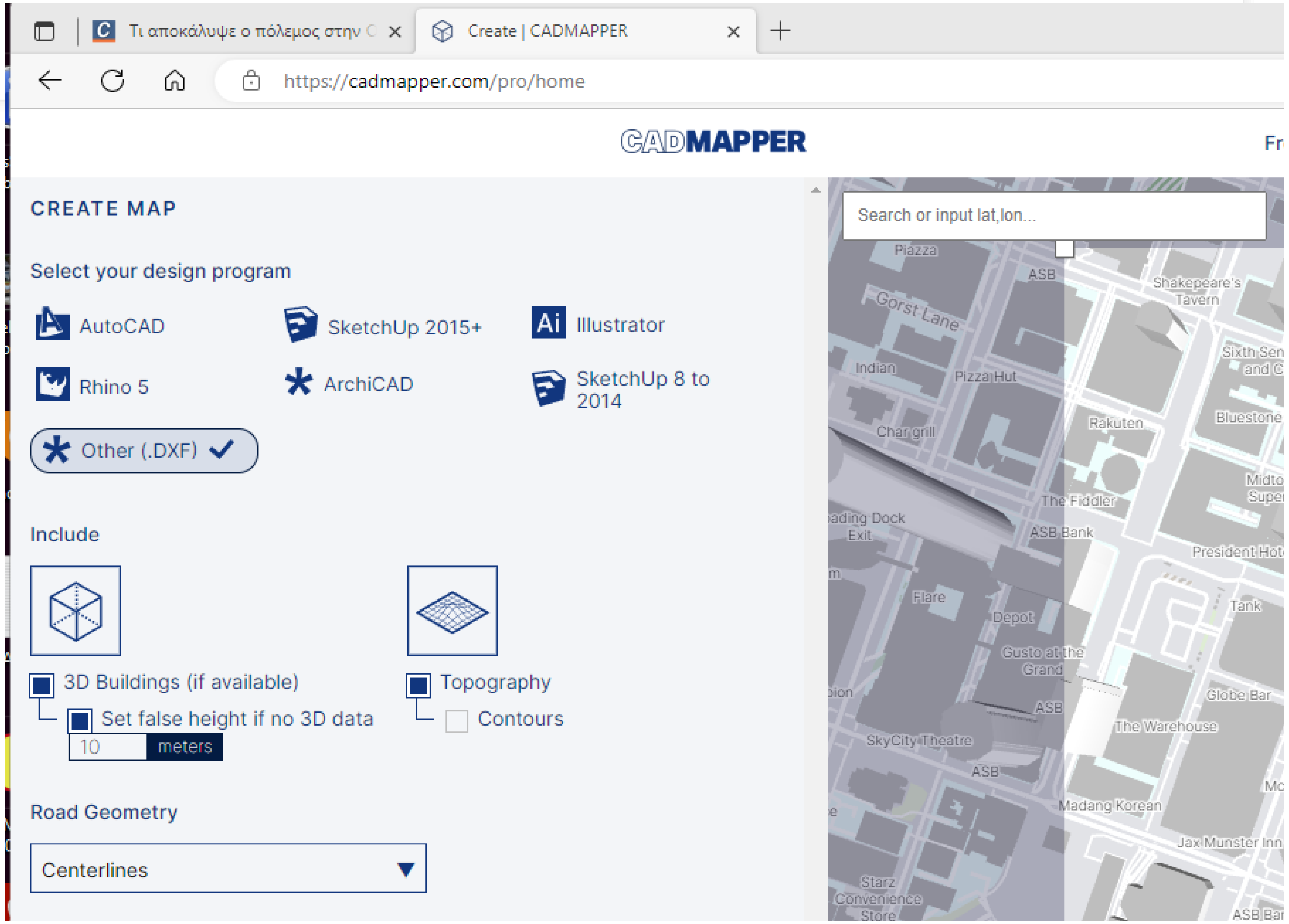
10. 3D City

- Import city buildings, elevations (cadmapper)
- Import utilities
- Generate new tunnel/subway line
- Position stations
- Estimate service level improvements
- Estimate costs & benefit analysis

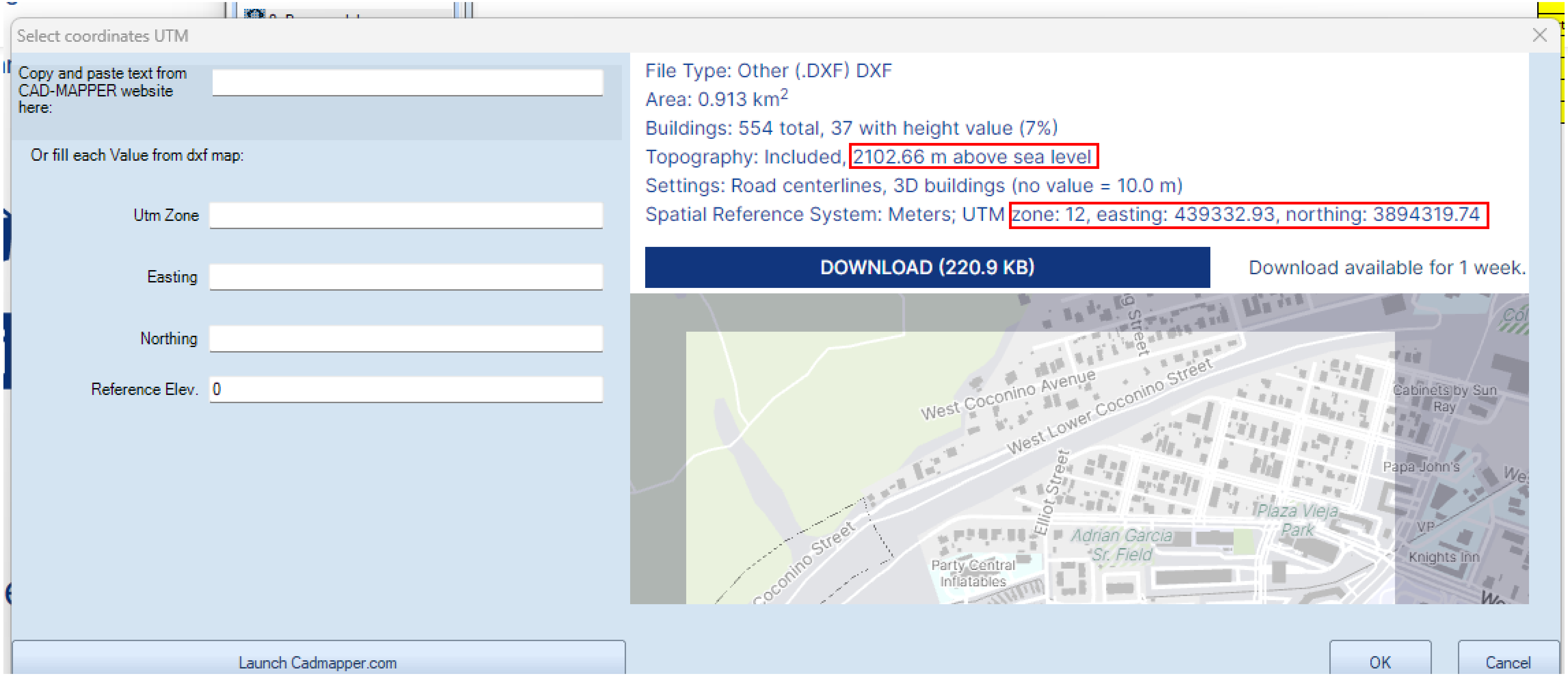


10.1 Set Map Location

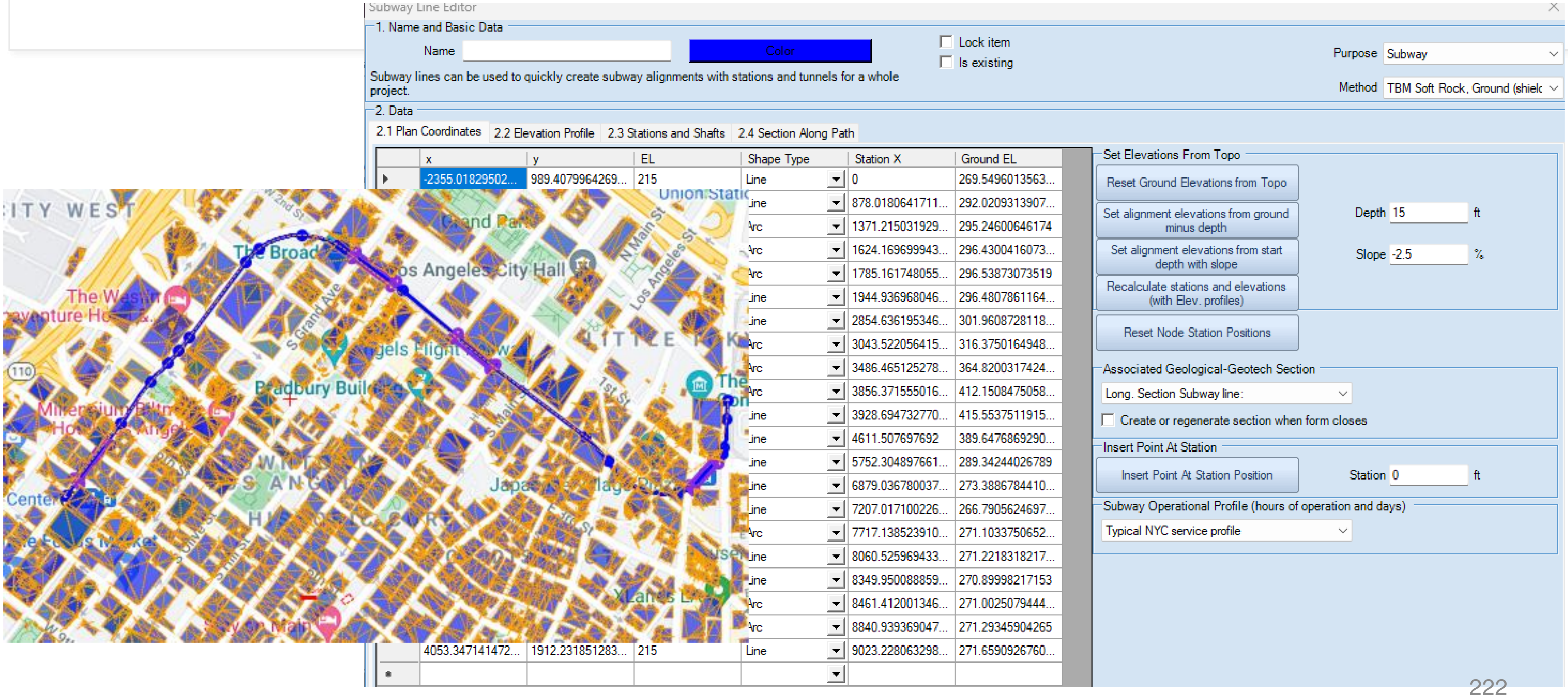




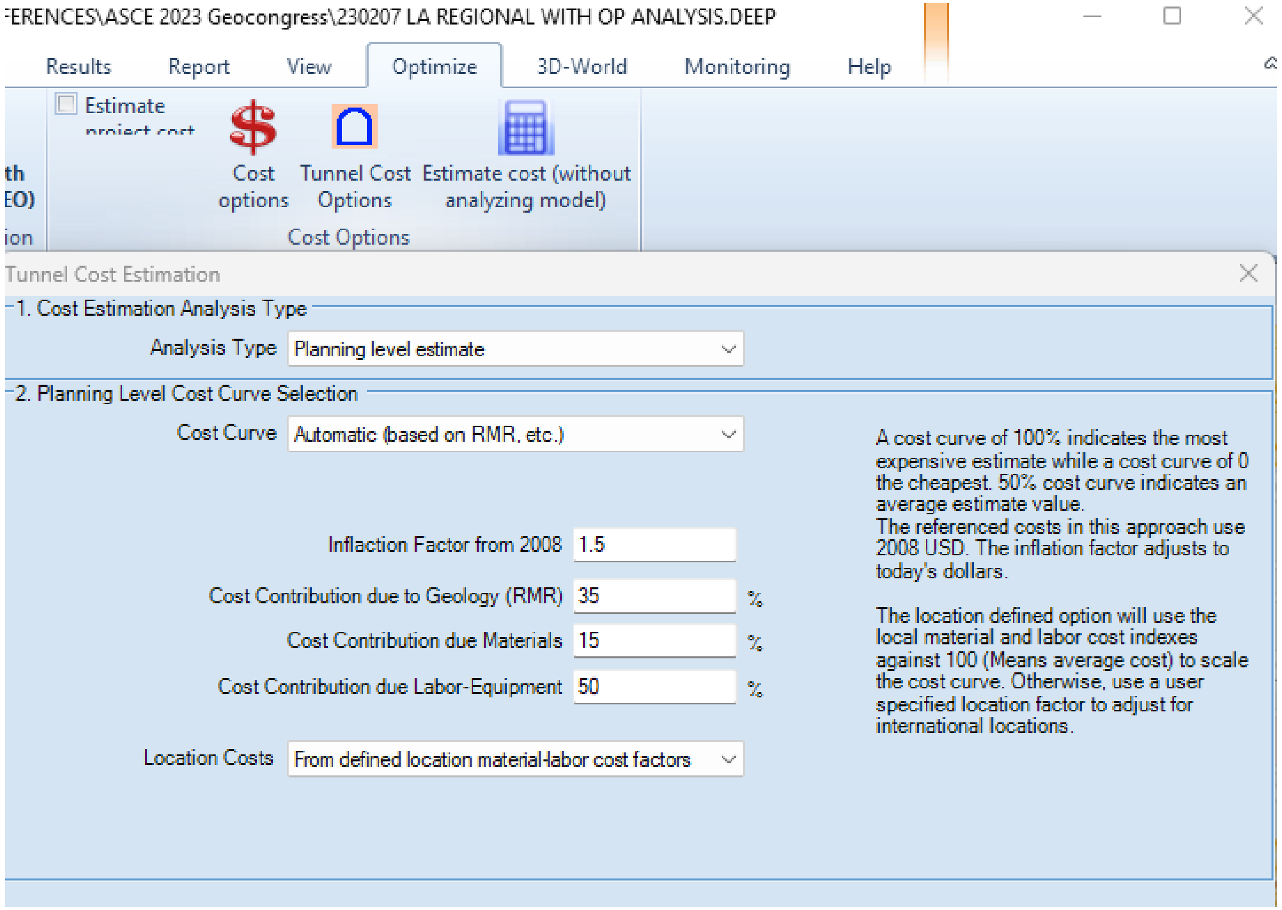
10.2 Import Cadmapper DXF



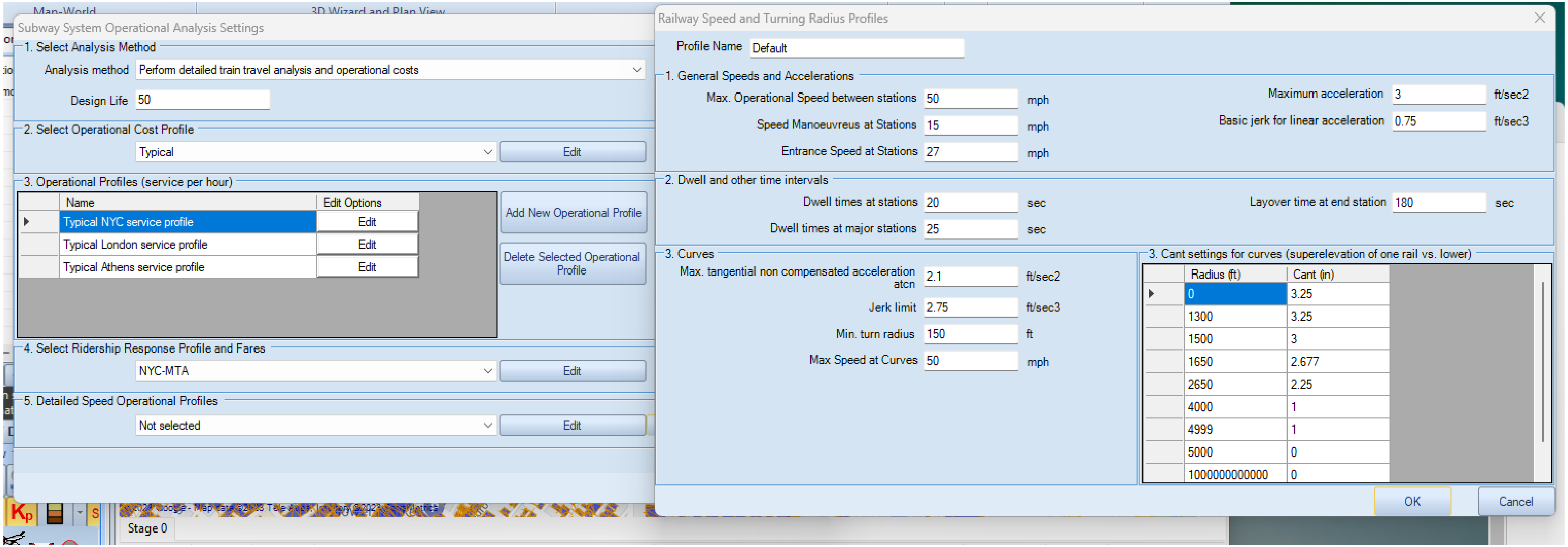
10.3 Draw Subway Lines & Stations



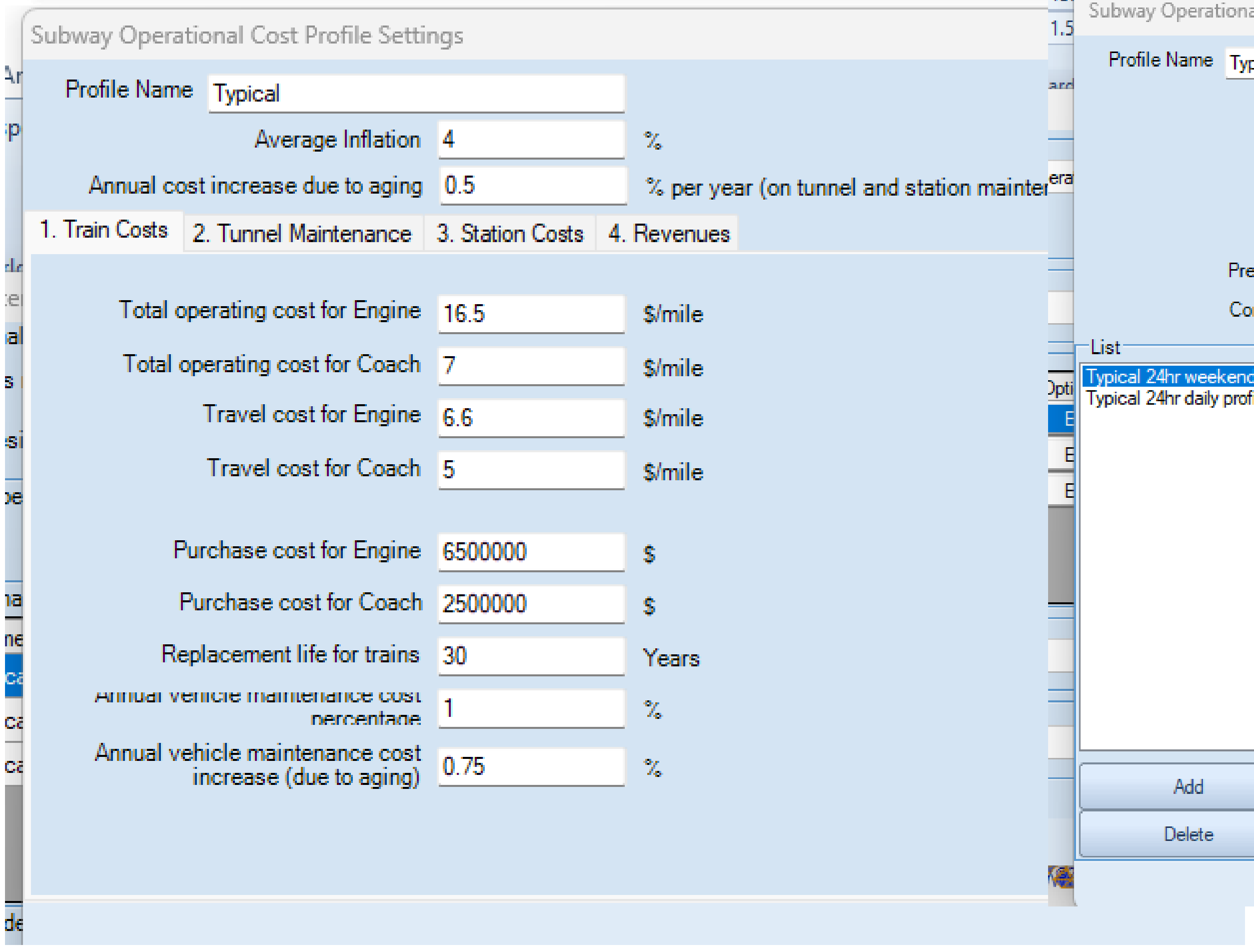
10.4 Tunnel Costs (Planning Level)

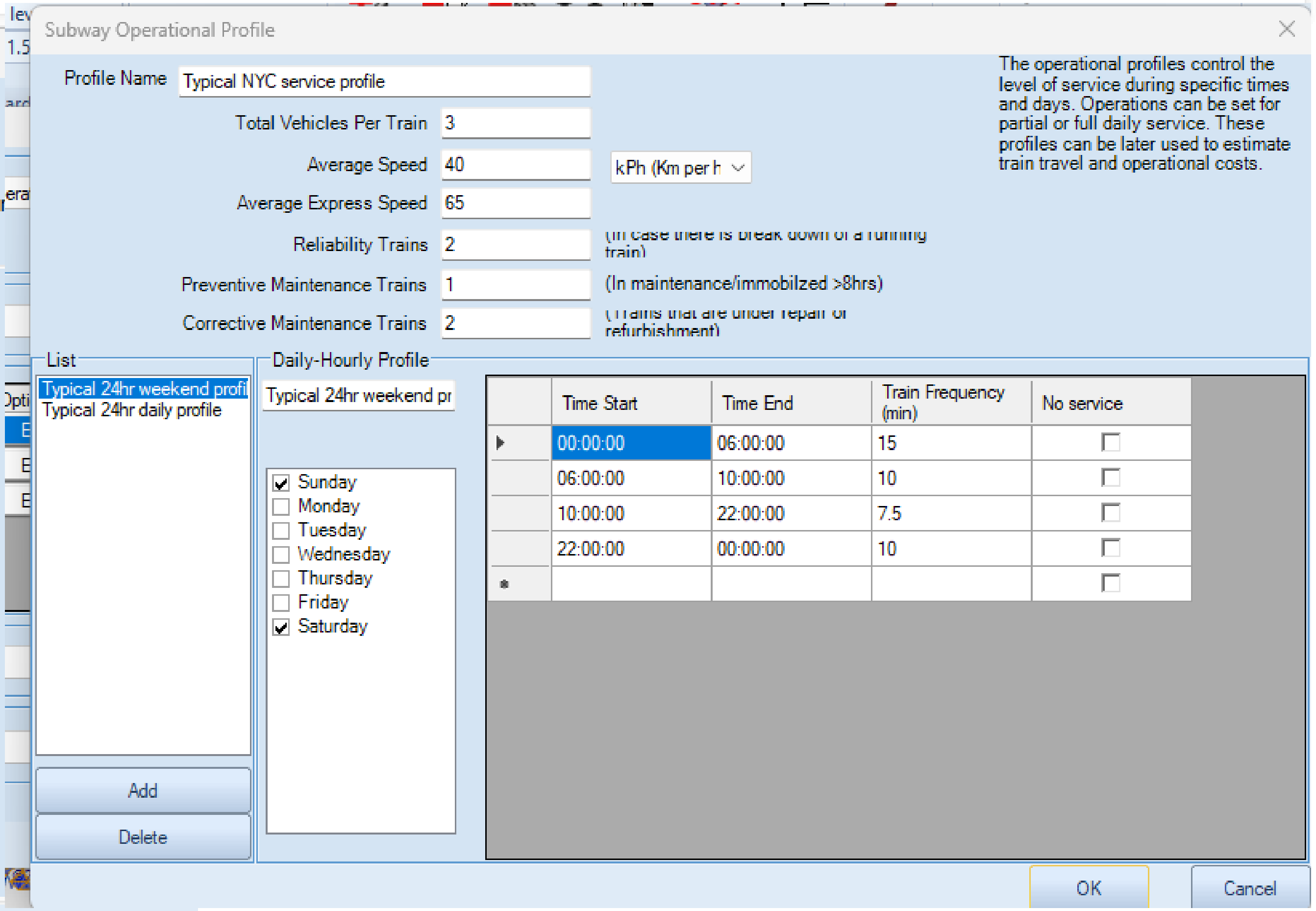


10.5 Operational Analysis Settings

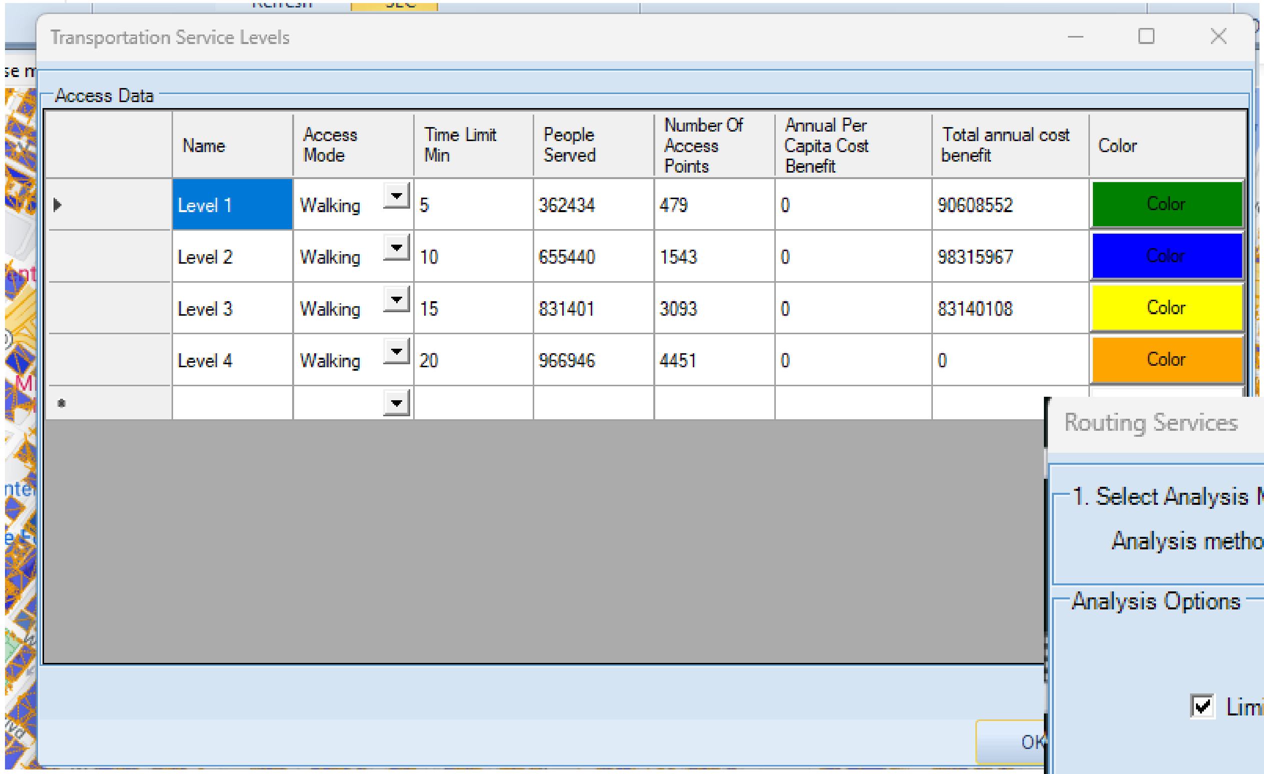


10.6 Subway Operations & Costs

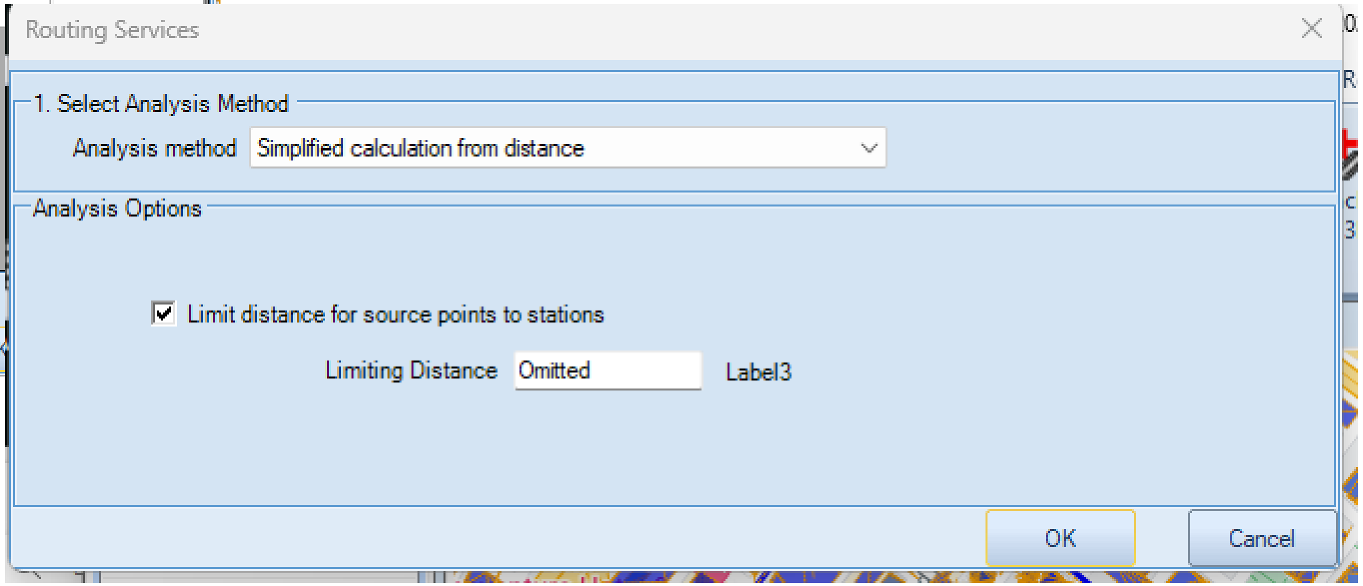




10.5 Service Level Improvements



Name	Access Mode	Time Limit Min	People Served	Number Of Access Points	Annual Per Capita Cost Benefit	Total annual cost benefit	Color
Level 1	Walking	5	362434	479	0	90608552	Color
Level 2	Walking	10	655440	1543	0	98315967	Color
Level 3	Walking	15	831401	3093	0	83140108	Color
Level 4	Walking	20	966946	4451	0	0	Color



10.7 Operational Cost Analysis

Subway Operational Cost Analysis Results : Total costs for new and existing lines

1. Table Results | 2. Graph Results | 3. Train Results | 4. Annual Ticket Sales

Year	Total cost $ mil.	Train Purchase $ mil.	Train Operationa $ mil.	Train Travel $ mil.	Train maintenanc $ mil.	Station Operationa $ mil.	Station Maintenan $ mil.	Tunnel Maintenan $ mil.	Other Costs $ mil.	Revenue $ mil.
From 1 t...	1644.1	331.55	489.44	187.15	135.25	304	357.7	26.16	0	7014.15
1	88.64	80.5	3.21	1.23	0.8	1.99	1.99	0.15	0	63.81
2	8.48	0	3.33	1.27	0.84	2.07	2.08	0.15	0	63.81
3	8.84	0	3.47	1.33	0.88	2.15	2.18	0.16	0	69.55
4	9.21	0	3.61	1.38	0.93	2.24	2.27	0.17	0	69.55
5	9.6	0	3.75	1.43	0.97	2.33	2.38	0.17	0	69.55
6	10.01	0	3.9	1.49	1.02	2.42	2.48	0.18	0	75.81
7	10.43	0	4.06	1.55	1.07	2.52	2.6	0.19	0	75.81
8	10.87	0	4.22	1.61	1.12	2.62	2.71	0.2	0	75.81
9	11.33	0	4.39	1.68	1.17	2.73	2.84	0.21	0	82.64
10	11.8	0	4.56	1.74	1.23	2.83	2.96	0.22	0	82.64
11	12.3	0	4.75	1.81	1.28	2.95	3.1	0.23	0	82.64
12	12.82	0	4.94	1.89	1.35	3.07	3.24	0.24	0	90.07
13	13.36	0	5.13	1.96	1.41	3.19	3.38	0.25	0	90.07
14	13.93	0	5.34	2.04	1.48	3.32	3.54	0.26	0	90.07
15	14.52	0	5.55	2.12	1.55	3.45	3.7	0.27	0	98.18
16	15.13	0	5.77	2.21	1.62	3.59	3.86	0.28	0	98.18
17	15.77	0	6	2.3	1.7	3.73	4.04	0.3	0	98.18
18	16.43	0	6.24	2.39	1.78	3.88	4.22	0.31	0	107.02
19	17.13	0	6.49	2.48	1.87	4.03	4.41	0.32	0	107.02
20	17.85	0	6.75	2.58	1.95	4.2	4.61	0.34	0	107.02
21	18.61	0	7.02	2.69	2.05	4.36	4.82	0.35	0	116.65
22	19.4	0	7.31	2.79	2.15	4.54	5.04	0.37	0	116.65
23	20.22	0	7.6	2.91	2.25	4.72	5.27	0.39	0	116.65
24	21.07	0	7.9	3.02	2.36	4.91	5.5	0.4	0	127.15
25	21.96	0	8.22	3.14	2.47	5.1	5.75	0.42	0	127.15

OK

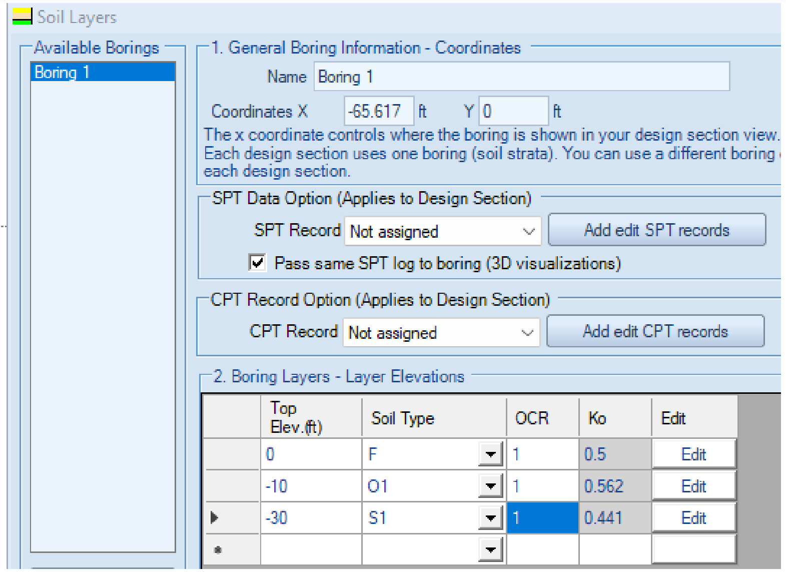
Subway Operational Cost Analysis Results : Total costs for new and existing lines

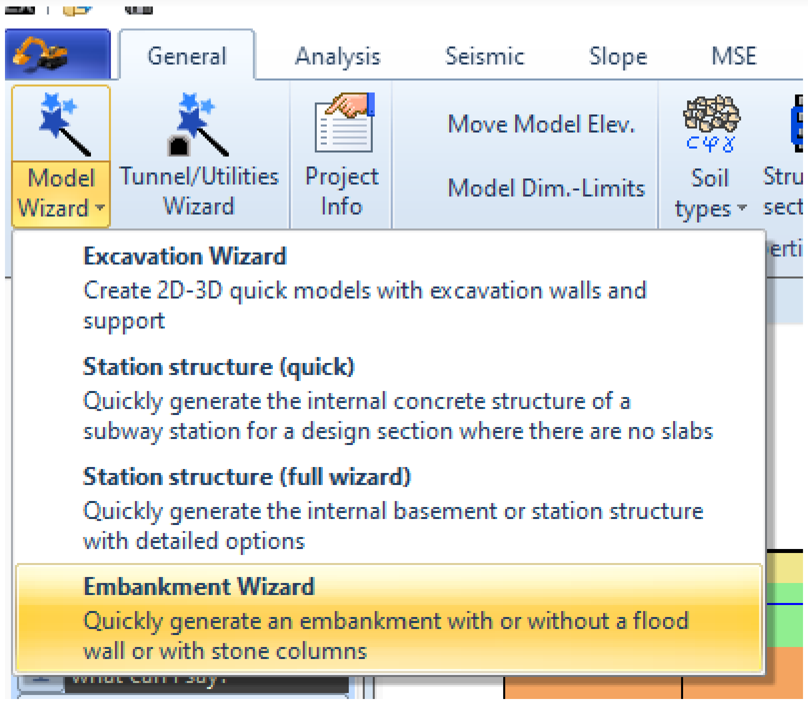
1. Table Results | 2. Graph Results | 3. Train Results | 4. Annual Ticket Sales

Year	Tickets Yearly	Tickets Monthly	Tickets Weekly	Tickets Daily	Tickets Single
1	89546	1480244	3179700	3314492	1611223
2	90441	1495046	3211497	3347636	1627336
3	91345	1509996	3243612	3381113	1643608
4	92258	1525096	3276049	3414924	1660045
5	93181	1540347	3308808	3449073	1676645
6	94113	1555750	3341897	3483565	1693411
7	95054	1571308	3375315	3518399	1710346
8	96004	1587021	3409069	3553583	1727449
9	96964	1602892	3443160	3589120	1744724
10	97934	1618920	3477590	3625011	1762170
11	98913	1635109	3512366	3661261	1779793
12	99902	1651460	3547491	3697874	1797591
13	100901	1667975	3582965	3734851	1815566
14	101911	1684655	3618795	3772200	1833722
15	102929	1701502	3654983	3809922	1852060
16	103958	1718517	3691532	3848021	1870580
17	104998	1735702	3728449	3886502	1889286
18	106049	1753058	3765733	3925368	1908178
19	107109	1770590	3803390	3964621	1927261
20	108179	1788295	3841424	4004268	1946533

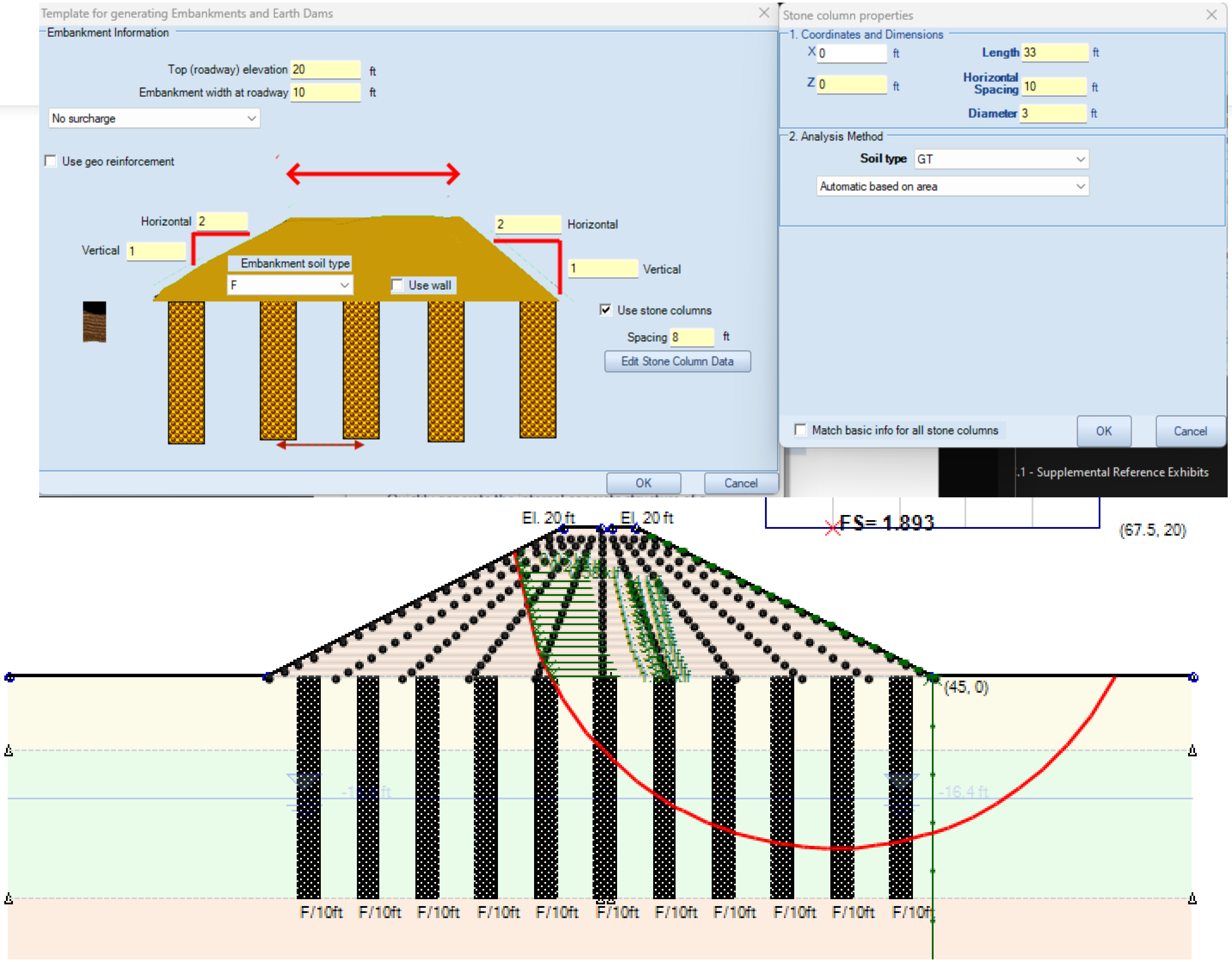
11.1 Embankment Wizard

- Embankments with MSE and stone columns





11.2 Embankment Wizard



11.0 Conclusion

- Attention to modelling choices in FEM
- Initial state of stress conditions
- Soil loss in tunnelling
- Wave pressures and corrosion for sea walls
- Import buildings from Google Maps and damage assessment
- Steel connections deserve attention