Seawall Failures

Typical Mistakes in Sea Wall Design and Construction

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Introduction:

Types of Seawalls

There are many different types of sea walls, and depending on the type of the sea wall construction, each can experience different modes of instability and failure.

Generally sea walls are built as bulkheads and can be described as Gravity Rigid and Flexible Bulkheads.

With advancements is available materials Gravity rigid walls are often considered economically unviable options. Therefore, this presentation will mostly concentrate on flexible bulkhead walls.

Gravity Rigid

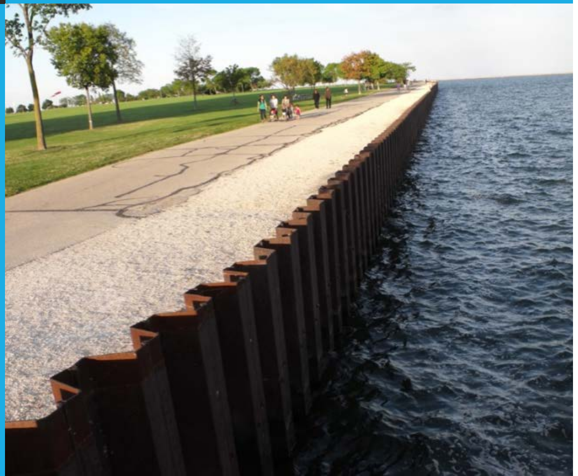
Concrete Panel

Flexible Anchored

Sheet Pile with Tieback System

Flexible Cantilevered

Sheet Pile with Deep Embedment



Most Common Types of

Seawall Failure

1. Excessive Deflection
2. Slip Circle Failure
3. Failure of Wall Anchorage
4. Down-drag (also #3)
5. Material Failure (rare)
6. Elastic Foundation Failure

A little about the Engineer…

Vitaly Feygin

Vitaly is the Principal Structural and Geotechnical engineer with Florida Geotechnical Engineering (FGE) and brings his 35+ years' experience of working on multiple challenging projects around the world (United States of America, Russian Federation, Ukraine, Germany, Trinidad and Tobago, Brazil, Indonesia, and Australia) to bear for our clients.

Vitaly's work around the world has earned him:

- Five engineering Awards (including three American Concrete Institute Grand Prize Awards for Design)
- Twelve publications on marine structures
- Two US patents on marine structural designs

Load Combinations Acting on Seawalls

In most design scenarios, there are two types of load combinations: Typical/Normal and Extreme/Abnormal

Normal

Active Soil Pressure

Earth pressure on the wall.

Surcharge Pressure

Pressure from permanent load from behind the wall (structure, etc.)

Lagging Water Table

Pressure from water drainage from behind the wall

Wave Load

Water pressure from wave action determined by wave climate at time of a storm.

Abnormal

Active Soil Pressure

Pressure when the wall is moving away from the retained soil.

Boussinesq Pressure

Pressure from transient load from behind the wall (equipment, etc.)

Down Drag

Suction on wall from reduction in pore-pressure (extreme low-tide)

Seepage Forces

High watertable on active-side and low watertable on passive side.

Loads Acting on Seawalls

Whilst we do not have seismic in Florida, very strong seismic is present in the Caribbean, and may constitute the life or death of a sea wall if dynamic loads were not considered in the sea wall design.

Proper Load application requires good understanding of the physics.

Active Pressure

Since vertical pressure is a function of soil density g and depth of the soil deposit, vertical pressure at depth will be $\gamma \times Z$, and Horizontal active pressure is

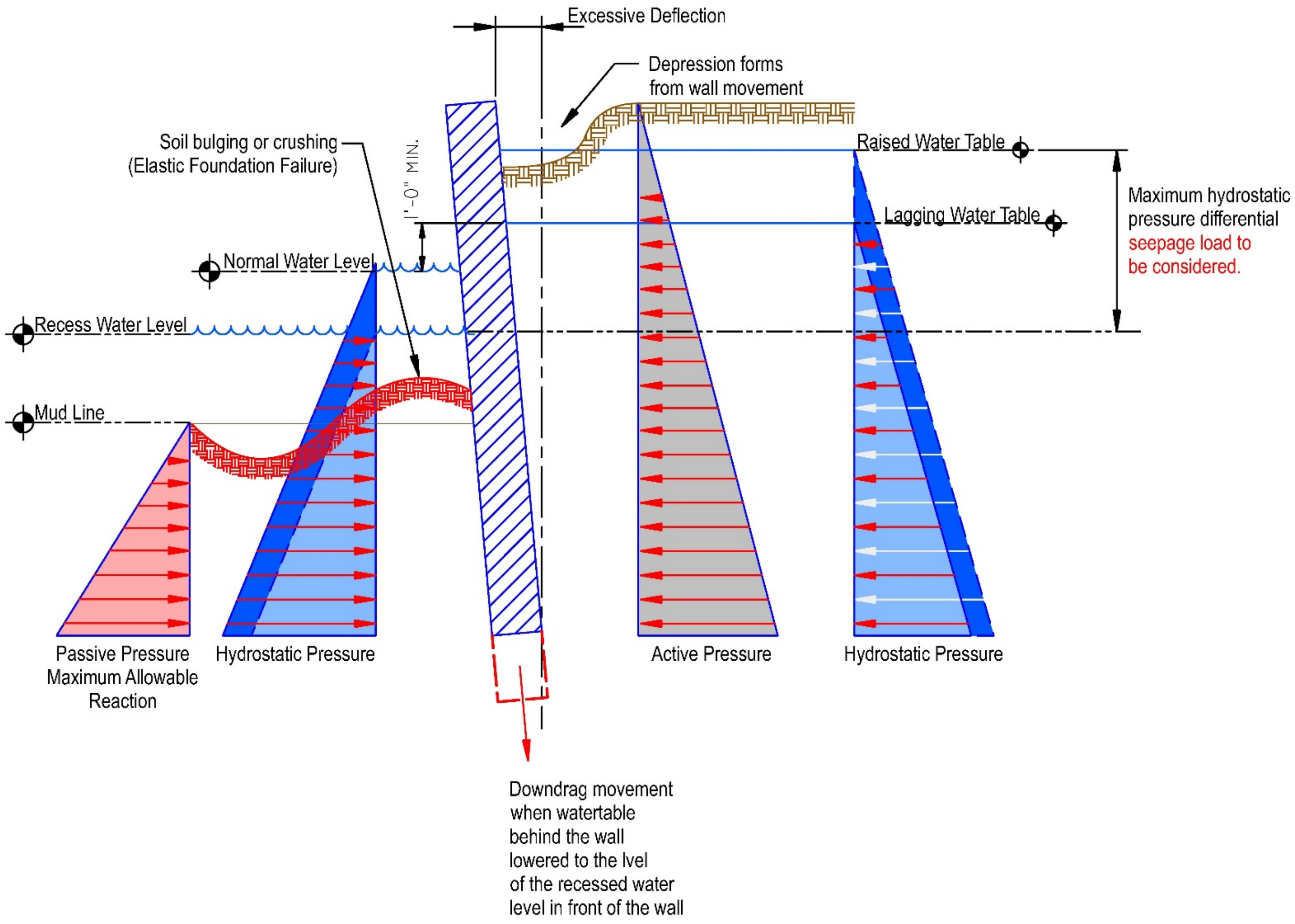
$$\boldsymbol{\gamma \times z \times K_a}$$

Where

$K_a = \tan\left(45 - \frac{\phi}{2}\right)$, and f = is angle of soil internal friction. Visually it can be presented as an angle of soil Repose.

K_a can be applied only to dry or normally saturated soil.

When soil is submerged active pressure coefficient is applied only to the buoyant weight of the soil, because Poisson ratio for water is equal to 1.0 and water hydrostatic pressure at depth will be equal $\gamma_w \times Z$

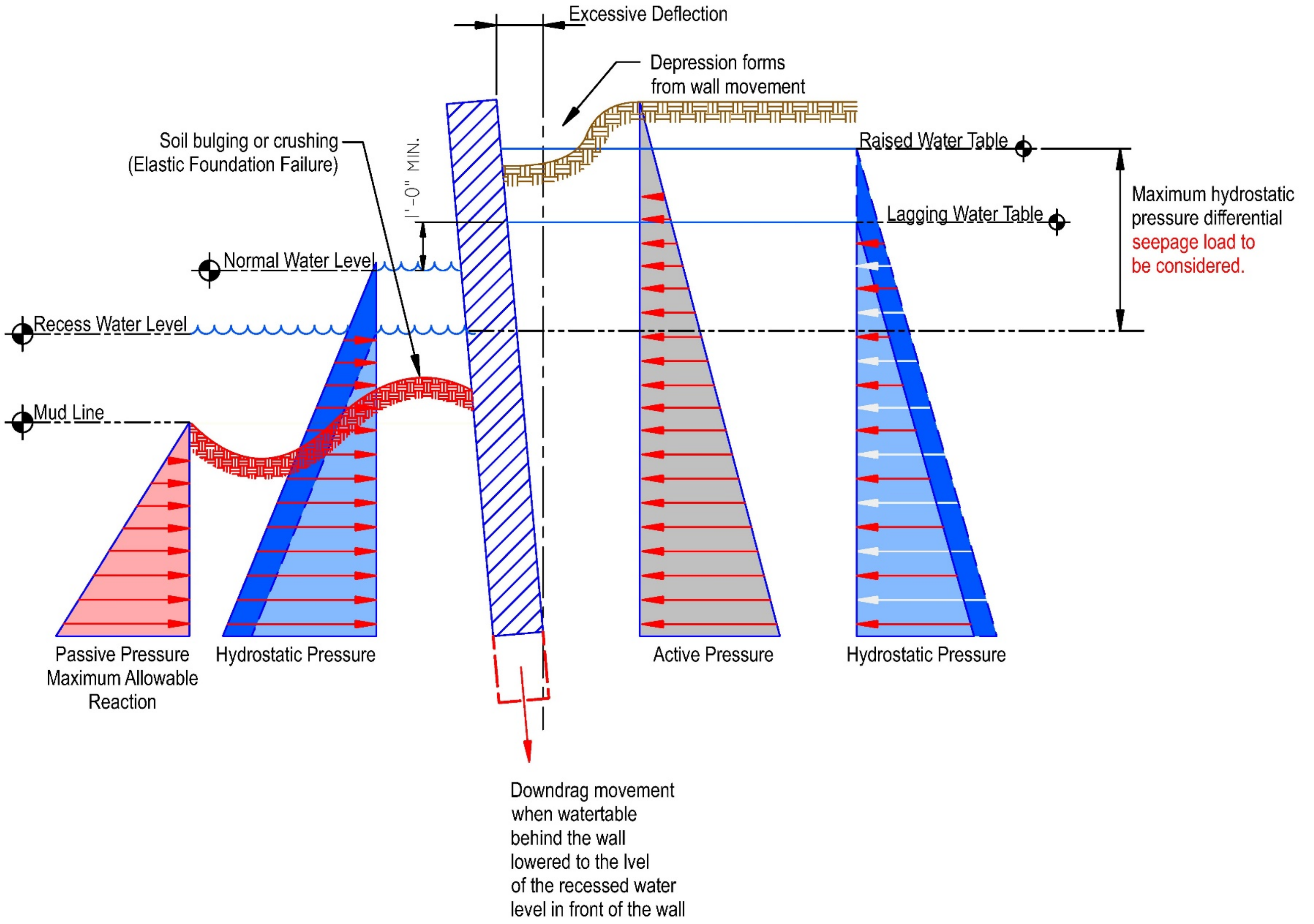


Loads Acting on Seawalls

Passive Pressure

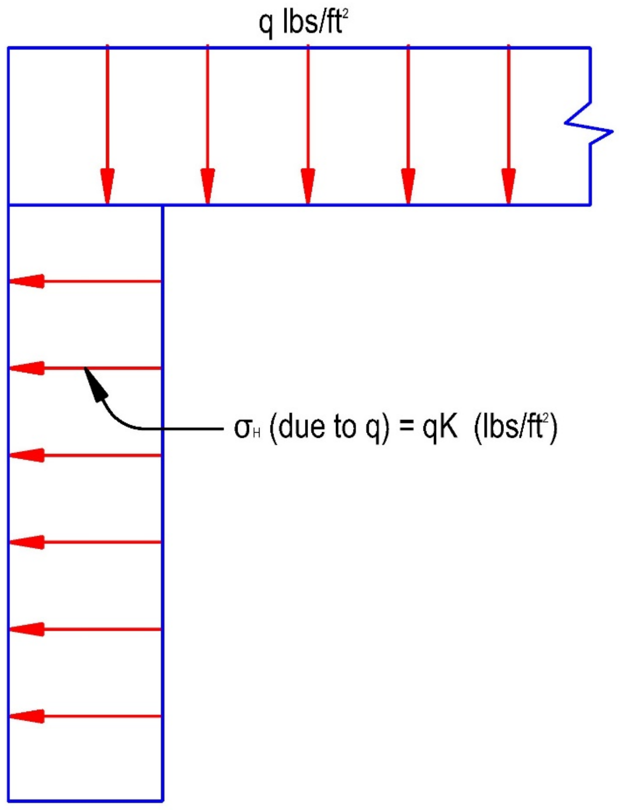
Shall be viewed as a maximum capacity of the soil to resist the pressure that retaining structure applies to the soil when wall moves against the soil. Elastic Foundation Reaction is compared against the Passive Pressure, and if it exceeds passive pressure, it gives a good indication of Elastic Foundation failure.

If active pressure is considered an action, the passive pressure is a maximum reaction that soil can provide prior to soil crushing (fully plastic failure).



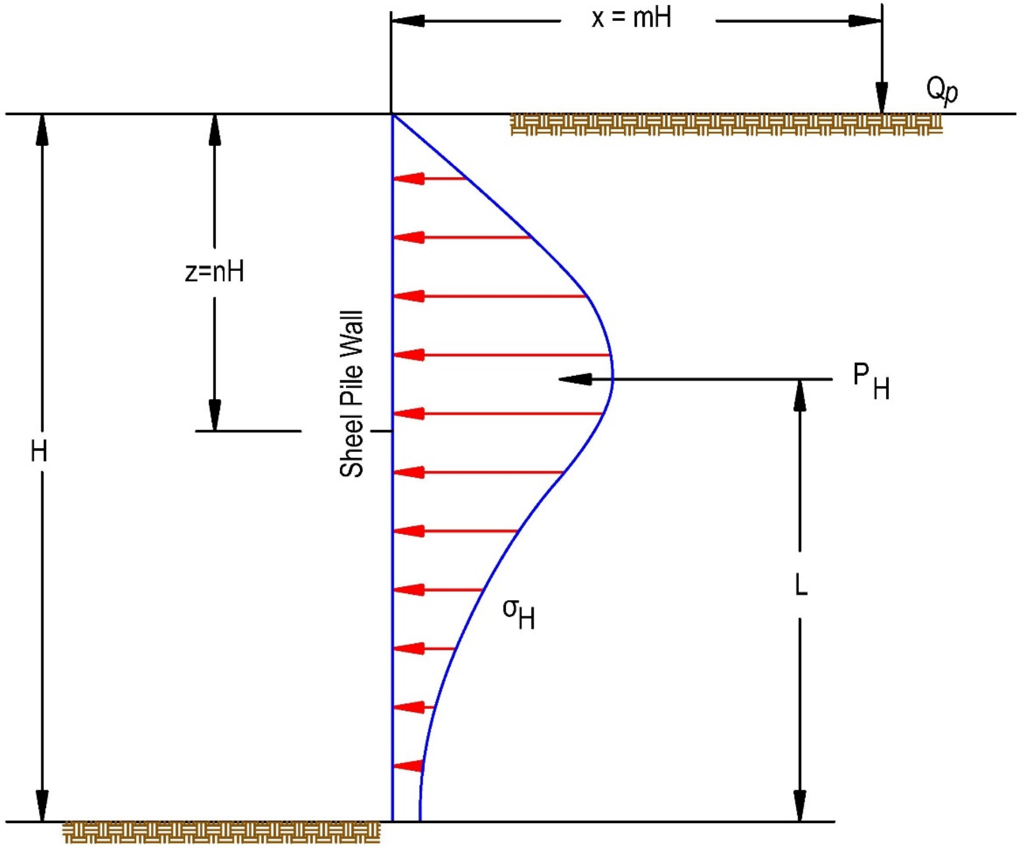
Boussinesq Pressure acting on Seawalls

Due to Blanket Load



Lateral pressure due to uniform surcharge

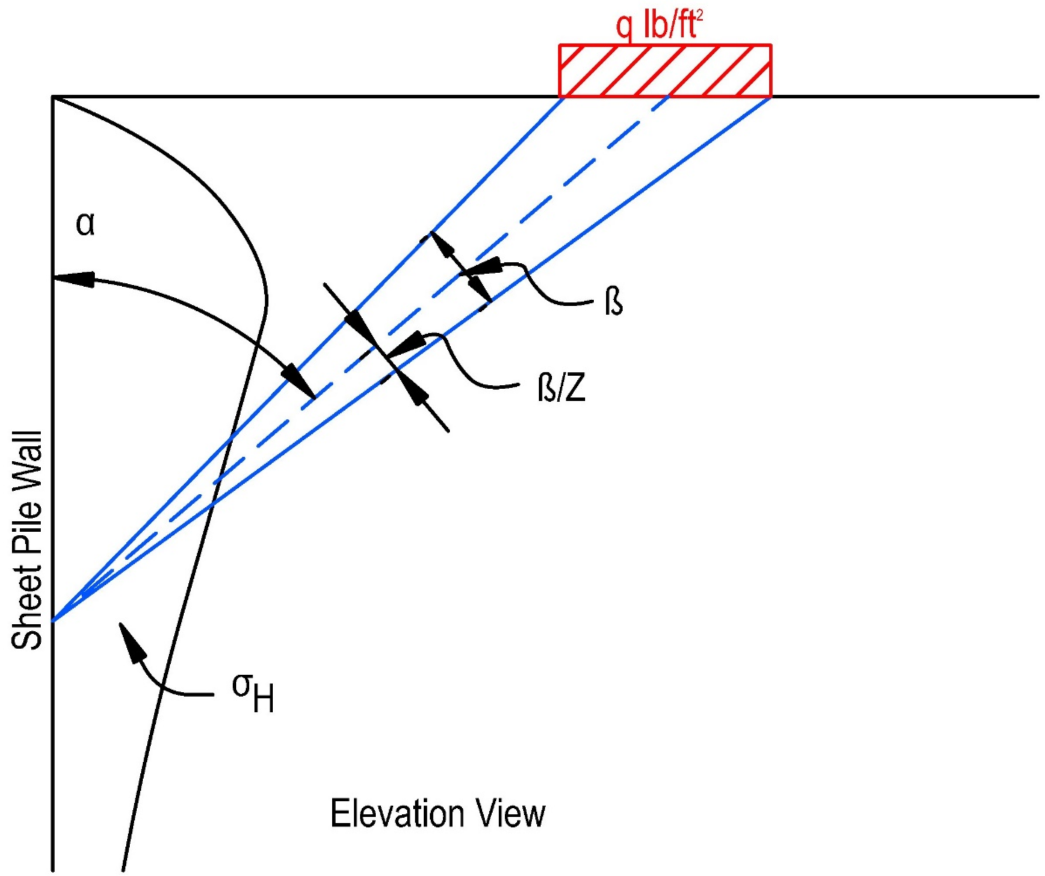
Due to Concentrated Load



Elevation View

Boussinesq Pressure acting on Seawalls

Due to Strip Load



Loads Acting on Seawalls

Hydrostatic and Seepage Pressure Interaction

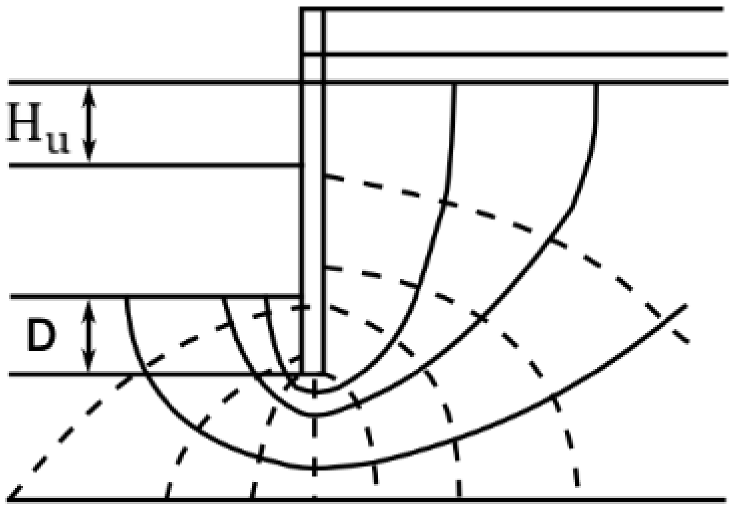
The difference in water table on either side of the wall creates additional hydrostatic pressure on the back side of the wall and reduction in the soil unit weight in front of the wall, reducing passive pressure. Reduction in the submerged unit weight of the soil in front of the wall is estimated as

$20 \times \frac{H_U}{D}$ where $H_u = Unbalanced\ Water\ Head$

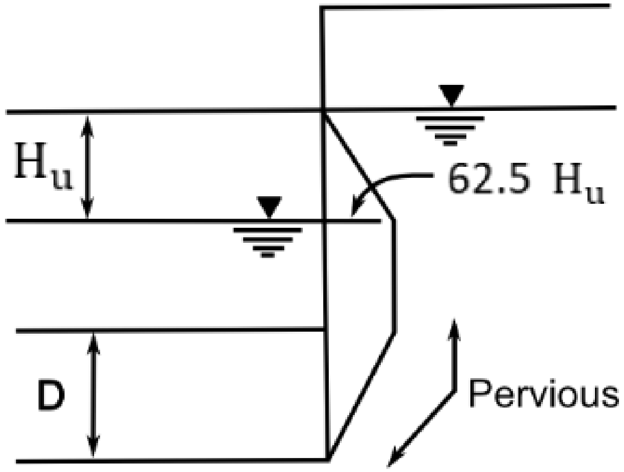
Effective unit weight that shall be used in the computation of the passive pressure limitation is $Y_{eff} = Y_s - \frac{20-H_u}{D}$ where $Y_s = is\ unit\ weight\ of\ saturated\ soil$

Neglecting effect of the passive pressure limit reduction due to the sudden drawback can be catastrophic.

Such sudden drawback happens during the hurricane when water in the basin is pulled away from the sea wall by a strong rotating eye wall of the hurricane wind.



(a)



(b)

Loads Acting on Seawalls

The picture shows breaking wave action and dynamic pressure distribution along the wall

Wave action shall be viewed as a two phase load application

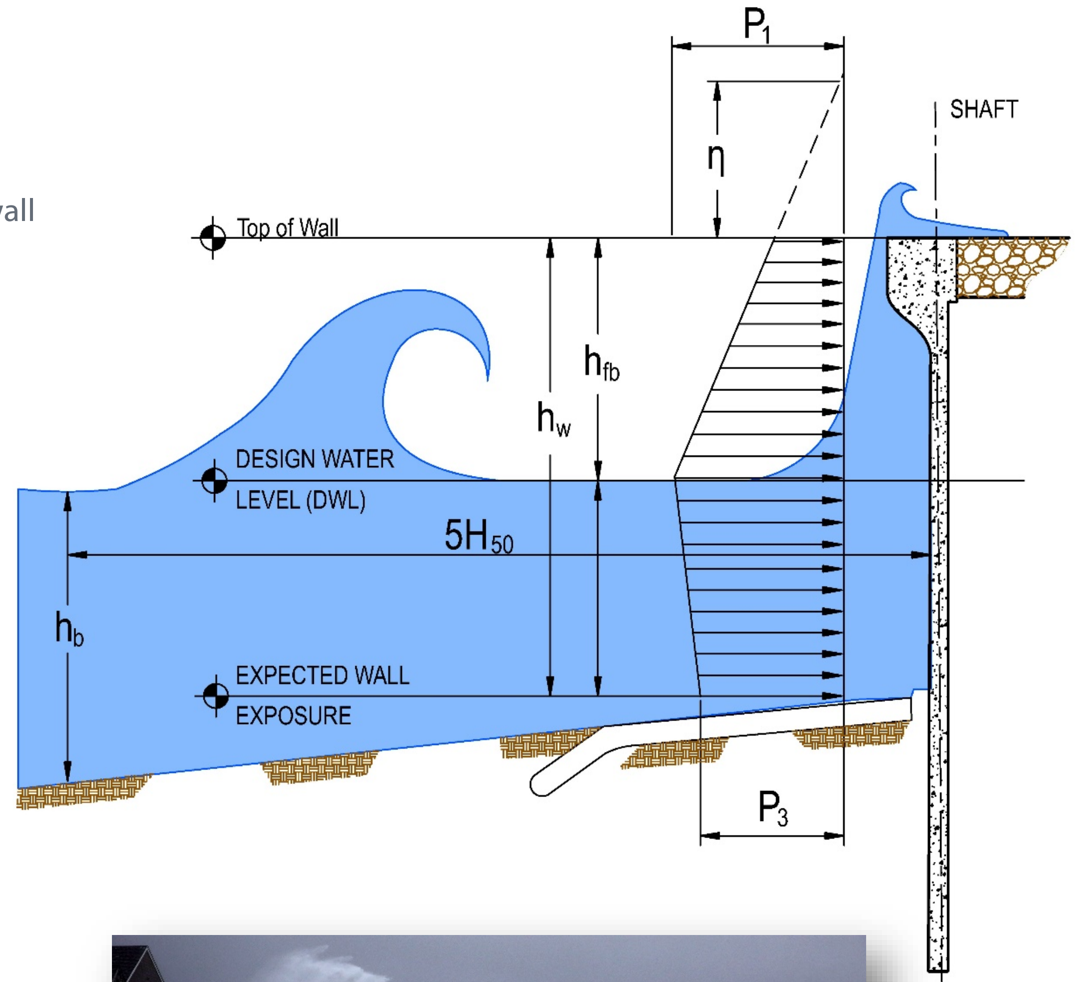
Direct Impact

Pressure Diagram is Shown

Overtopping

Water in front of the wall at the level of the wave trough. (reduced hydrostatic pressure in front of the wall), and simultaneous elevated hydrostatic pressure behind the wall, on the land side

For the purpose of analysis, wave trough shall be considered 1/2 wave height below the sea level during the storm surge. Overtopping of the wall is very typical for hurricane events. This condition is frequently neglected by designers



Sheet Pile Bulkhead Analysis

The are multiple methods for bulkhead analysis which were successfully used for sheet pile design.   However, Majority of these methods are highly imprecise.

This presentation will concentrate on Bulkhead analysis based on lateral springs (p-y curves). This type of analysis is generally called **Elastic Foundation Analysis**.

Nevertheless, even that method highly depends on our selection of the soil springs and stage of construction.

- Due to uncertainties associated with the development of the soil springs, sometimes engineer shall use a good judgement and check the wall utilizing Upper Bound and Lower Bound soil springs. This is particularly true for design at temporary loads during construction stage, when soil parameters closely resemble undrained soil condition

- At normal working condition soil will be in a normal drained state when pore pressure have already sufficiently subsided after construction.

- **Walls shall be designed for temporary loads at time of construction. Many walls fail at that time.**

Concept of Elastic Foundation

Soil is modeled as an elasto-plastic two point curvature. Prior to crushing soil behaves similarly to a linear spring. That is the basis of Elastic Foundation Analysis.

$$EF = k_{hz} \times B$$

Where

k_{hz} = *modulus of horizontal subgrade reaction*

B = *width of the pile*

Granular Soils

$$k_{hz} = \eta_h \times \frac{Z}{B}$$

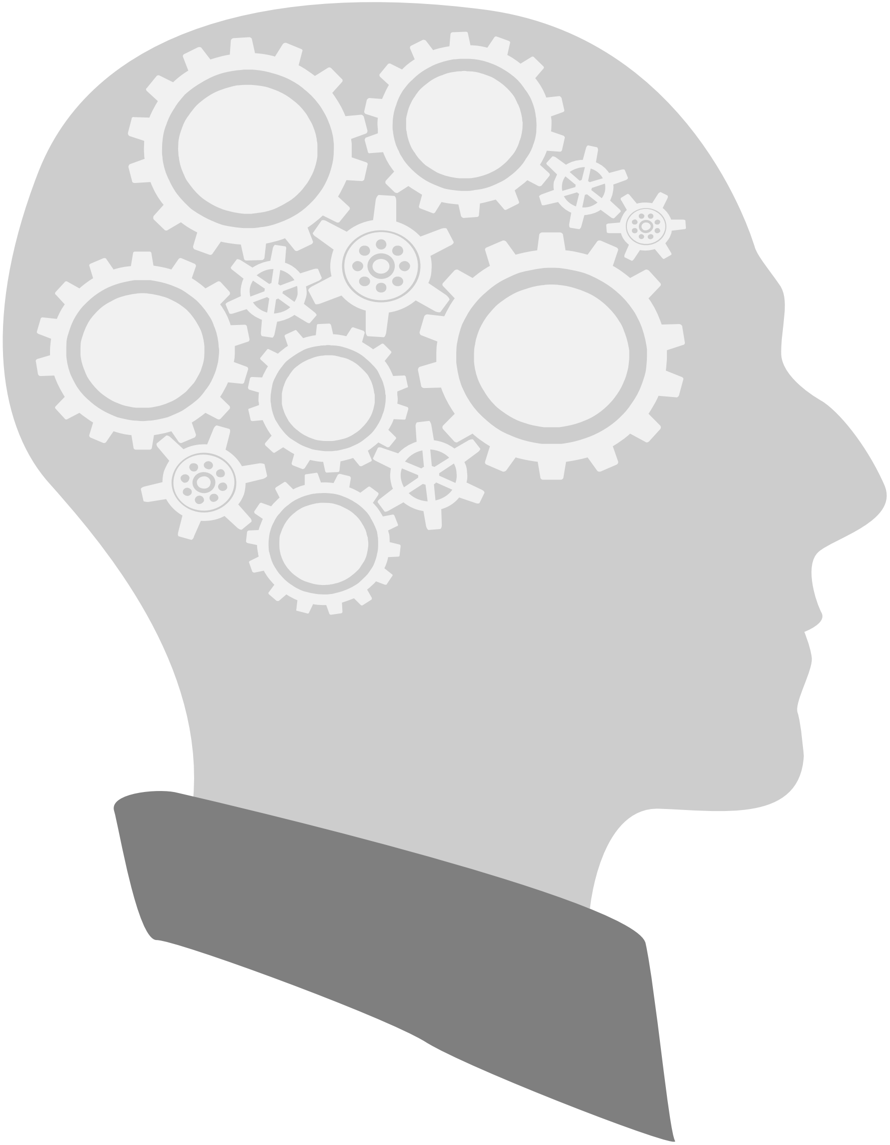
η_h = horizontal subgrade gradient or constant of Horizontal Subgrade

$$EF = \eta_h \times Z$$

Cohesive Soils

$$k_{hz} = \frac{64S_u}{B}$$

The following tables explain common mistakes in selection of the values for k_{hz}



Concept of Elastic Foundation

Soil is modeled as an elasto-plastic two point curvature. Prior to crushing soil behaves similarly to a linear spring. That is the basis of Elastic Foundation Analysis.

The gradients for discrete wall or single pile were factored by a factor 2.5
That factor is used instead of B eff for discrete piles, where B eff = 2.5 to 3 pile diameter.
The term "Discrete Wall" is a misnomer. The left table was created for a pile, not for wall analysis.

This should be remembered when engineer designs a king pile wall system where sheet piling stops short, and king pile tip point is deeper than the tip point of the sheeting. In this case king pile elastic foundation is based on B eff = 2.5*B, and designer can use direct values given in the left table.

Designer can use direct values in the top table for a king pile analysis; but use the same values divided by a factor 2.5 for continuous wall analysis.

Values of n_h for loose medium and dense sands are provided in Table

Estimated Values of the Constant of Horizontal Subgrade Reaction, Discrete Wall Systems in Moist and Submerged Sands (based on Table 3, Terzaghi 1955)

Soil Type - Sand	Constant of Horizontal Subgrade Reaction, n_h (range in pcl)		
Relative Density	**Loose**	**Medium**	**Dense**
"Dry" or moist sand (range)	4-13	13-43	43-86
"Dry" or moist sand (adopted)	8	25	64
Submerged sand (range)	3-8	8-27	27-54
Submerged sand (adopted)	5	16	40

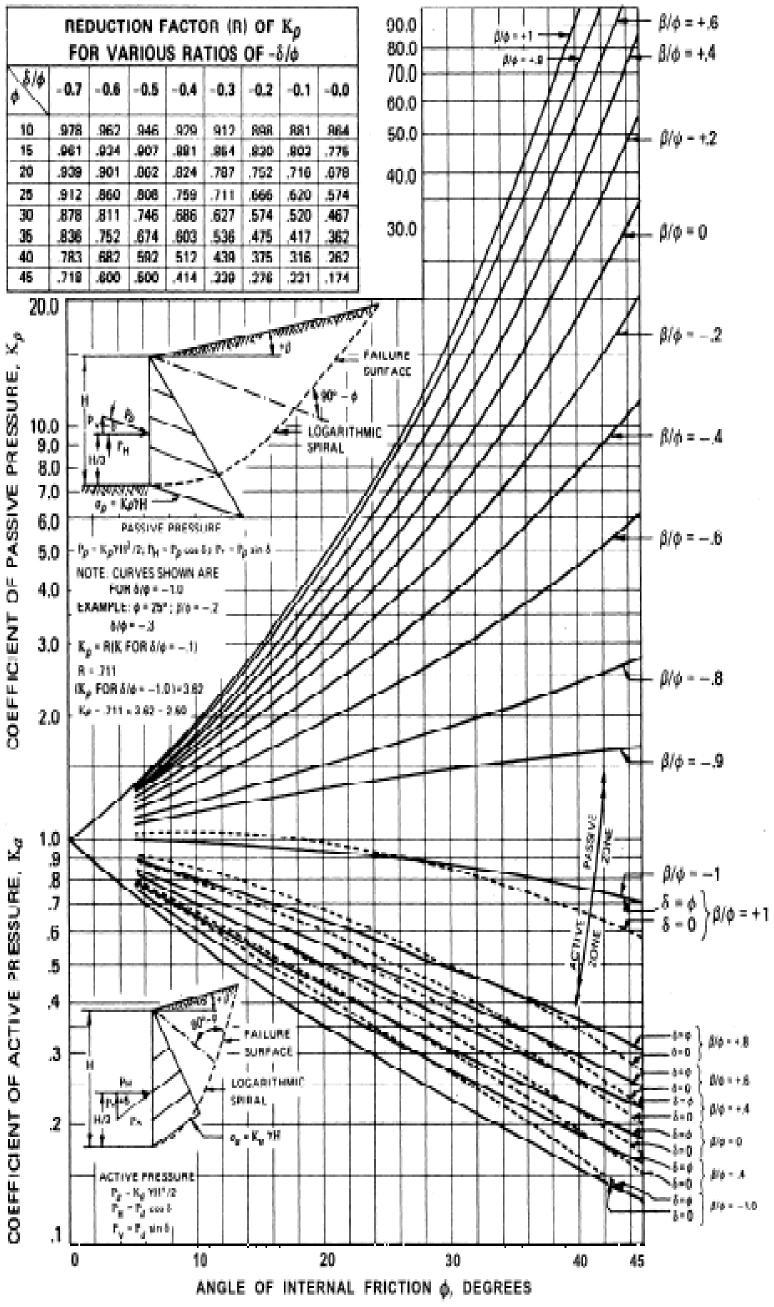
Estimated Values of the Subgrade Constant for Continuous Wall Systems in Moist and Submerged Sands (based on Table 4, Terzaghi 1955)

Soil Type - Sand	Subgrade Constant, l_h (pcl)		
Relative Density	**Loose**	**Medium**	**Dense**
"Dry" or moist sand (adopted)	3	9	23
Submerged sand (adopted)	2	6	15

Concept of Elastic Foundation

Elastic Foundation Reaction vs. Passive Pressure. Ultimate and Service Limit States.

Frequently designers use simplified Rankine theory for Kp.
Rankine theory is overly conservative and results based on that theory will be highly conservative and uneconomical.



ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	-0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.864	.830	.802	.776
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

Active and passive coefficients with wall friction (sloping backfill) (after Caquot and Kerisel[1])

Concept of Elastic Foundation

Two Soil Crush Limit States

Elastic Foundation Reaction vs. Passive Pressure. Understanding Concept of Passive Pressure Resistance.

First: Soil Crushing Limit State (Elastic Foundation vs. Passive Pressure)

- Elastic Foundation Reaction is a pressure that wall exerts on the soil and soil reaction to that pressure. It is equal but opposite in direction to the wall pressure.
- Elastic foundation for a king pile is based on B eff = 2.5*B, and has units of force per linear ft of the pile height. In order for calculating pressure exerted by the king pile on the soil, EFR shall be divided by B eff
- Calculated passive pressure is not a force, but an Ultimate Limit Capacity of the soil at the plastic limit.
- For conversion of soil Plastic Resistance Limit into Elastic Resistance Limit Kp shall be divided by a factor 1.5

Second: Deep Circle Slip Failure. This Limit State is related to Slope Stability Analysis

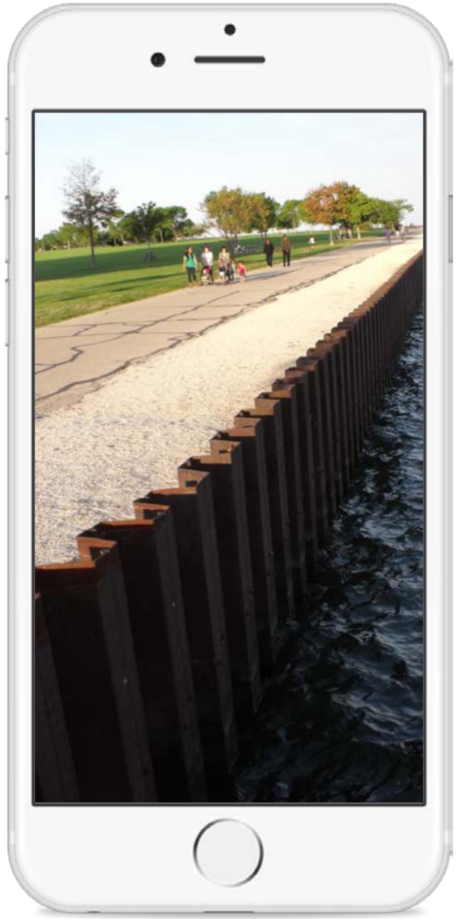
- This Limit State is frequently neglected by practicing engineers. <u>Physically that type of failure can be visualized as an action of the shovel blade in the soil.</u>

The following two Examples explain analysis of the sea sheet pile walls.

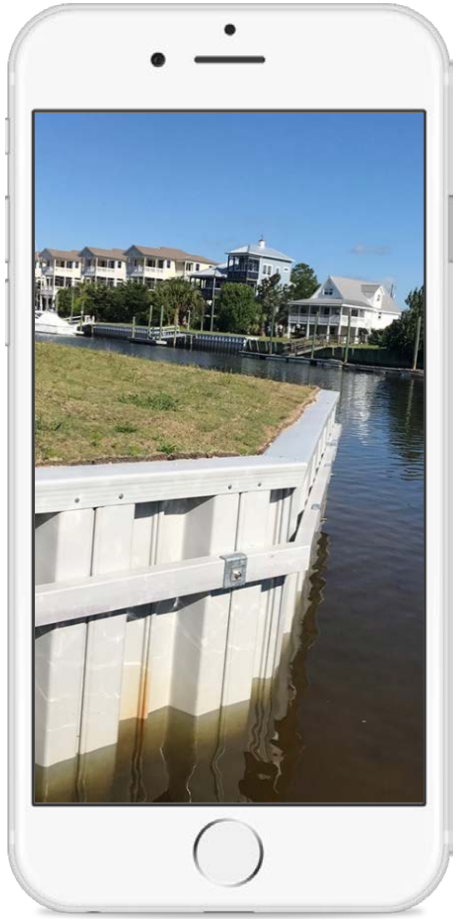
Cantilevered vs. Braced Flexible Bulkhead Walls

- Sometimes, cantilever walls present a better and more economical solution than walls with tiebacks and a dead man. Nevertheless, each solution has it pros and cons, and different failure modes.

- These failure modes and methods allowing to address them are discussed in the following Examples



VS



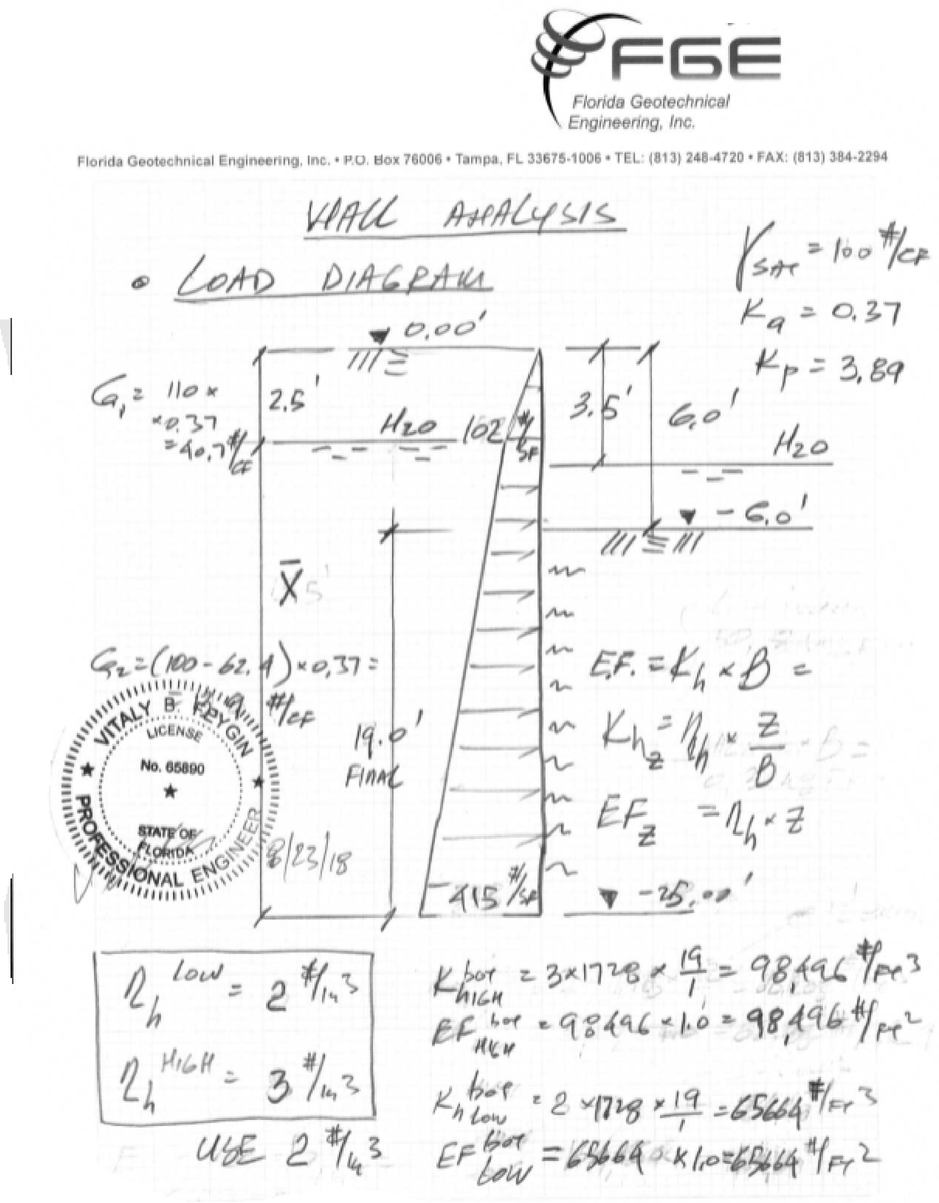
Concept of Elastic Foundation

Example 1

Example of the Cantilevered sheet pile wall

Load Diagram:

6′ -0 Cantilevered Sheet Pile Sea Wall

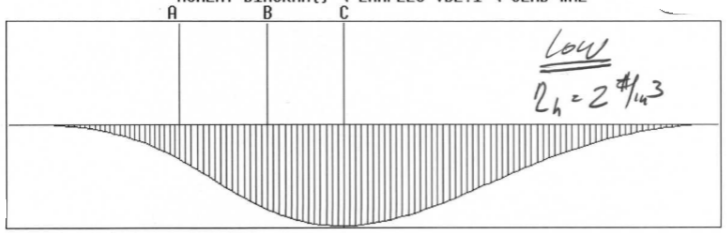


Concept of Elastic Foundation

Example 1 - Diagrams

Moment

MOMENT DIAGRAM() \ EXMPLES VB2.1 \ GLAD WAL



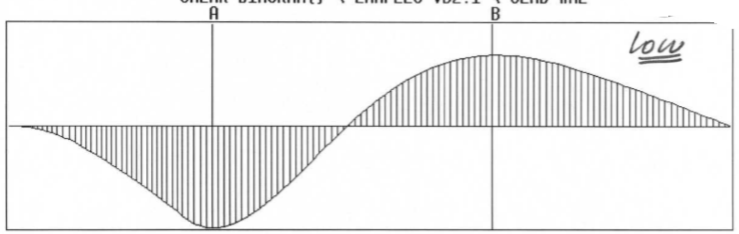
BEAM LENGTH = 25.0
MAX VALUE = 3735.48

POSITION =11.78
VALUE =-3735.48

	Position	Value		Position	Value		Position	Value
A	6.0	-1276.66	E			I		
B	9.1	-3119.49	F			J		
C	11.78	-3735.48	G			K		
D			H			L		

Shear

SHEAR DIAGRAM() \ EXMPLES VB2.1 \ GLAD WAL



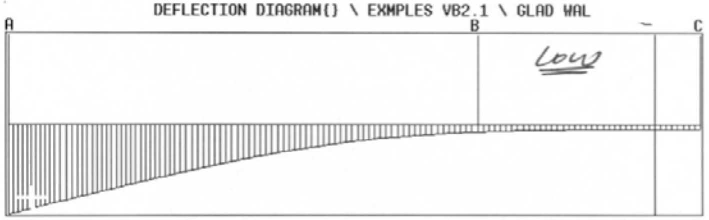
BEAM LENGTH = 25.0
MAX VALUE = 646.0794

POSITION =16.74
VALUE =451.3964

	Position	Value		Position	Value		Position	Value
A	7.03	-646.0794	E			I		
B	16.74	451.3964	F			J		
C			G			K		
D			H			L		

Deflection

DEFLECTION DIAGRAM() \ EXMPLES VB2.1 \ GLAD WAL



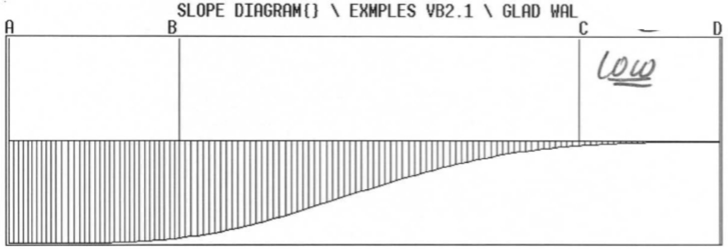
BEAM LENGTH = 25.0
MAX VALUE = 0.092881

POSITION =23.35
VALUE =0.005351

	Position	Value		Position	Value		Position	Value
A	0.0	0.092881	E			I		
B	16.95	0.007948	F			J		
C	25.0	0.005216	G			K		
D			H			L		

Slope

SLOPE DIAGRAM() \ EXMPLES VB2.1 \ GLAD WAL



BEAM LENGTH = 25.0
MAX VALUE = 0.00696

POSITION =25.0
VALUE =-0.00008

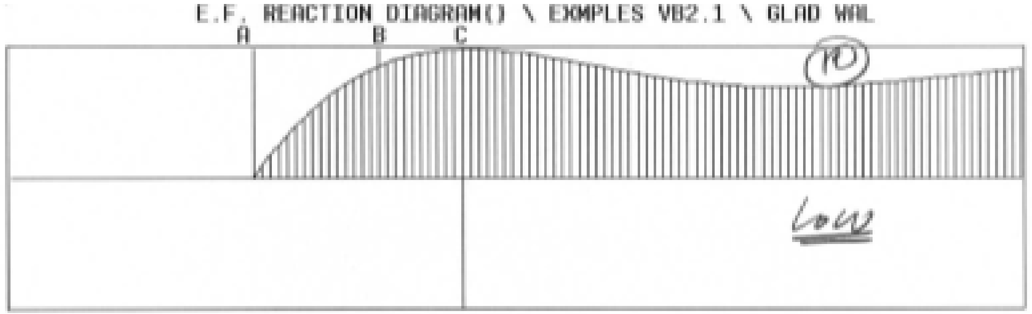
	Position	Value		Position	Value		Position	Value
A	0.0	-0.00696	E			I		
B	6.0	-0.00662	F			J		
C	20.04	-0.00032	G			K		
D	25.0	-0.00008	H			L		

Concept of Elastic Foundation

Example 1

Elastic Foundation Diagram

- What if EFR >P pass at service level.
- Then, designer needs to play with a pile length and stiffness.
- If that does not work, designer needs to change design concept and use braced wall solution.



BEAM LENGTH = 25.0
MAX VALUE = 402.7447

POSITION =11.16
VALUE =402.6003

	Position	Value		Position	Value		Position	Value
A	6.0	0.0	E			I		
B	9.1	348.9108	F			J		
C	11.16	402.6003	G			K		
D			H			L		

$K_{p_{ult}} = 3.89$ $K_{p_{serv}} = \frac{3.89}{1.5} = 2.59$

$EF_B = 349$ psf

$P_{pass_B} = (100 - 62.4) \times 2.59 \times 3.1 = 301$ psf

$F.S. = \frac{301}{349} \times 1.5 = 1.30$ v.s 1.5 O.K.

$EF_C = 402$ PSF

$P_{pass_C} = (100 - 62.4) \times 2.59 \times 5.16 = 502$ psf >

> 402 psf

O.K.

∴ soil does not crush

Concept of Elastic Foundation

Example 2

Let's solve the same problem using braced sea wall solution.

Load Diagram:

Braced sheet pile Sea wall

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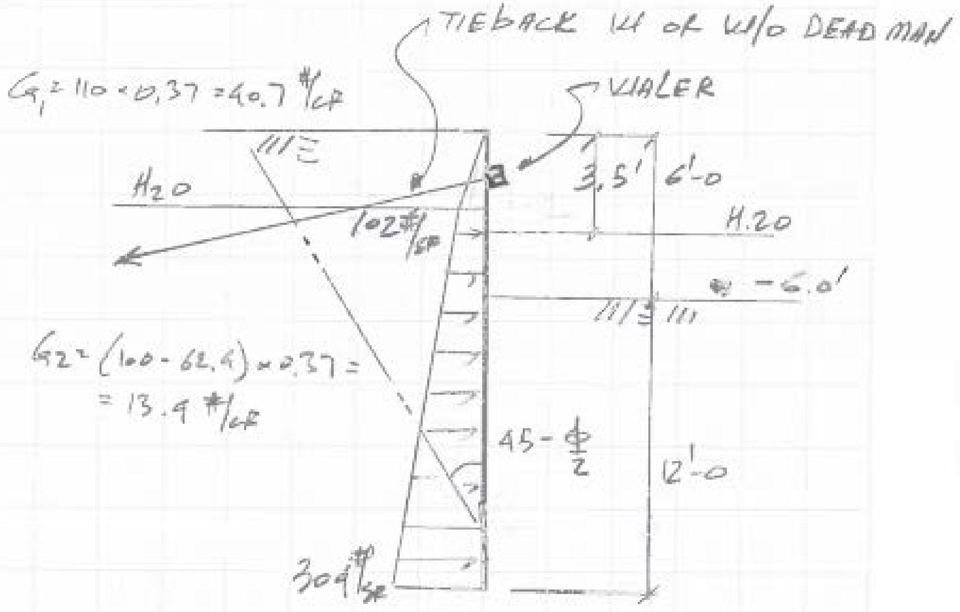


$$\boxed{\begin{aligned} \eta_h^{Low} &= 2\ \#/in^3 \\ \eta_h^{HIGH} &= 3\ \#/in^3 \end{aligned}}$$

USE $2\ \#/in^3$

$K_{HIGH}^{bot} = 3 \times 1728 \times \frac{12}{1} = 62{,}208\ \#/ft^3$

$EF_{HIGH}^{bot} = 62{,}208 \times 1.0 = 62{,}208\ \#/ft^2$

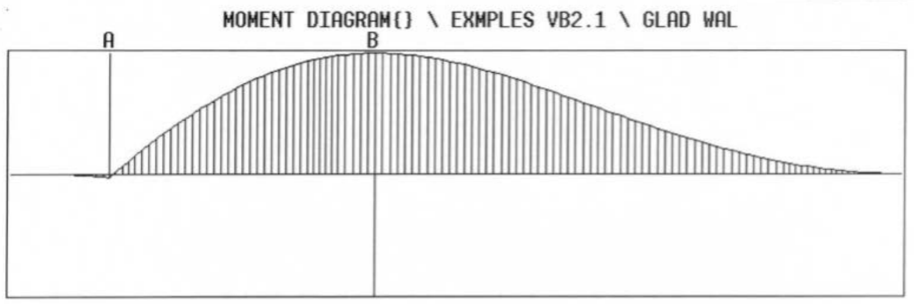
$K_{Low}^{bot} = 2 \times 1728 \times \frac{12}{1} = 41{,}472\ \#/ft^3$

$EF_{HIGH}^{bot} = 41{,}472 \times 1.0 = 41{,}472\ \#/ft^2$

Concept of Elastic Foundation

Example 2 - Diagrams

Moment

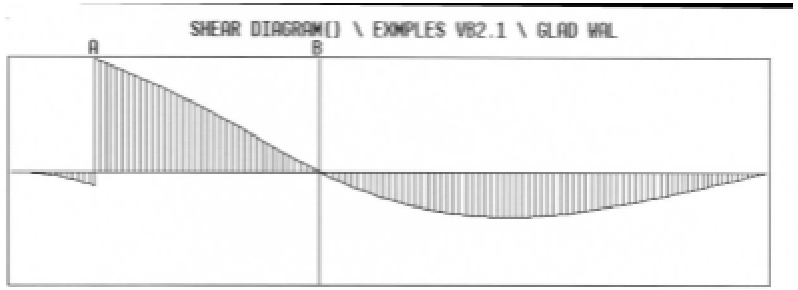


BEAM LENGTH = 18.0
MAX VALUE = 1832.93

POSITION =7.35
VALUE =1832.93

	Position	Value		Position	Value		Position	Value
A	2.0	-54.4	E			I		
B	7.35	1832.93	F			J		
C			G			K		
D			H			L		

Shear

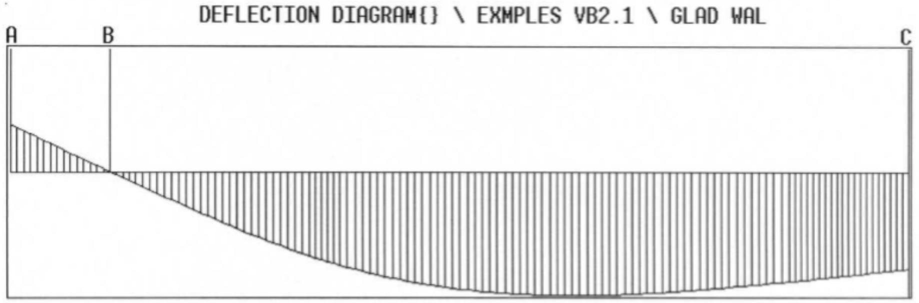


BEAM LENGTH = 18.0
MAX VALUE = 664.7484

POSITION =7.35
VALUE =-1.5299

	Position	Value		Position	Value		Position	Value
A	2.0	-81.6	E			I		
B	7.35	-1.5299	F			J		
C			G			K		
D			H			L		

Deflection



BEAM LENGTH = 18.0
MAX VALUE = 0.011176

POSITION =18.0
VALUE =0.008755

	Position	Value		Position	Value		Position	Value
A	0.0	-0.004256	E			I		
B	2.0	0.0	F			J		
C	18.0	0.008755	G			K		
D			H			L		

$R_s = 665 + 82 = 747$ #/'

ASSUME T/B @ 8'-0 c/c

$R = 0.75 \times 8 = 6.0^k$ /PER T.B.

∴ DEAD MAN IS REQ'D

OPTIONS:

- SHORT PIPE PILE; T/B; WALER
- SHORT SHEET PILE PANEL T/B; WALER

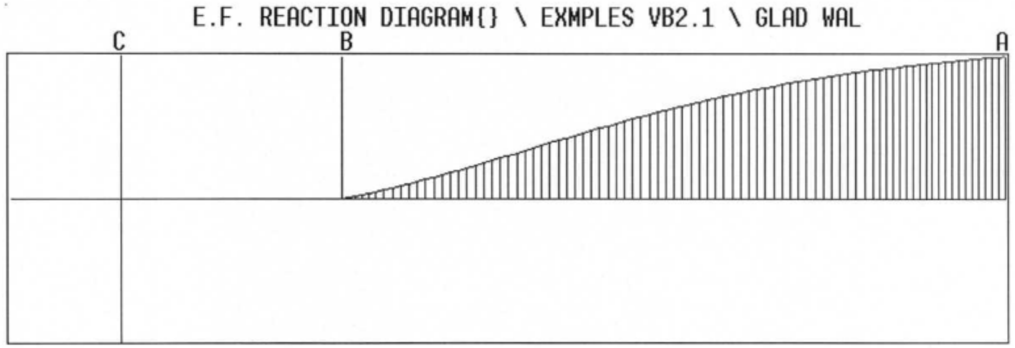
Concept of Elastic Foundation

Example 2

Elastic Foundation Diagram

Wall design is followed by the design of the Tie Back system with a Dead Man.

Such system coupled with wall itself, in some cases may become less economical than cantilevered wall option.



BEAM LENGTH = 18.0
MAX VALUE = 363.0954

POSITION =2.0
VALUE =0.0

	Position	Value		Position	Value		Position	Value
A	18.0	363.0954	E			I		
B	6.0	0.0	F			J		
C	2.0	0.0	G			K		
D			H			L		

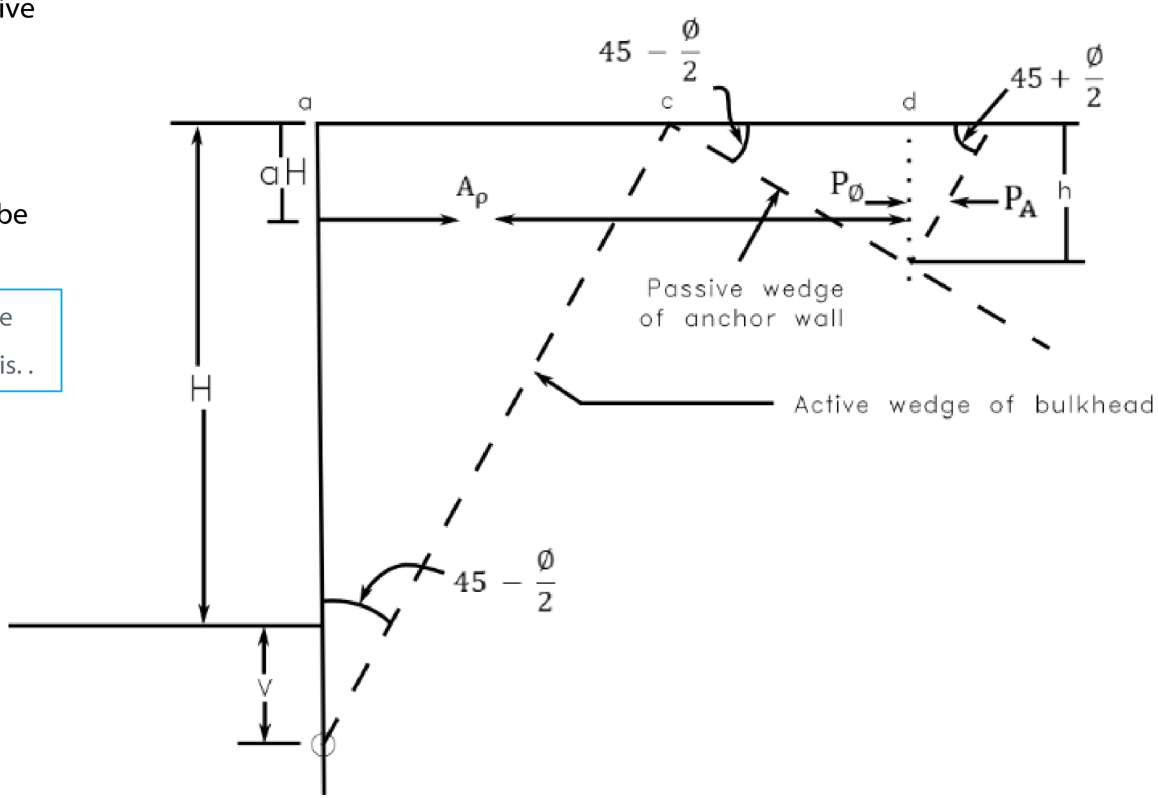
Dead Man Solutions and Dead Man Failure Modes

Rupture zone of the passive wedge of the dead man shall not intersect with an active wedge of the active pressure on the wall..

The point of the tie rod connection to the dead man shall be ideally placed at the resultant of the earth pressure acting on the anchorage. However, anchorage can be designed as a:

- Discrete Wall
- Monopile
- A-Frame

In case of mono-pile or discrete wall dead man can be designed using principles of Elastic Foundation Analysis. .



Common Design Mistakes

- **Designers forget to include active pressure acting on the dead man**
- **Sometimes dead man are placed into loose soil and settle**
- **For "dead man" placed into clayey soils, design shall be based on both drained (long term) and undrained (short term) soil conditions**

Concept of Circular Sliding Failure

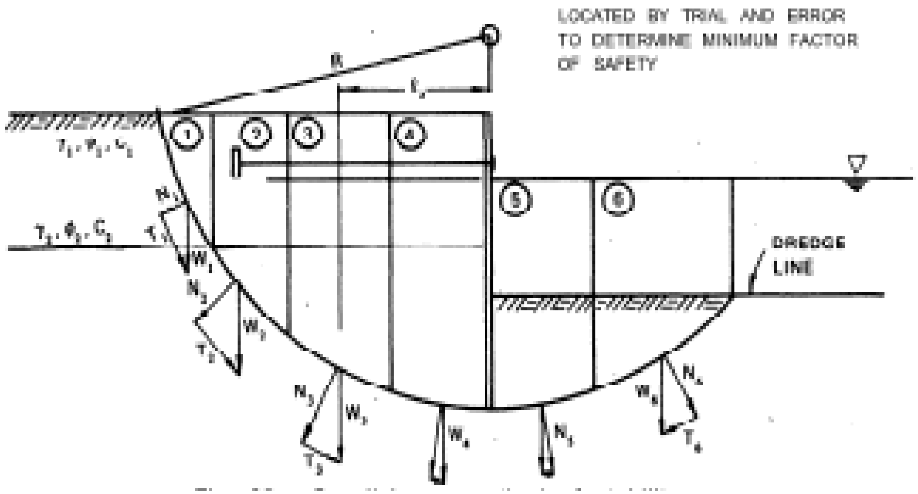
This check is a must for braced and un-braced (cantilevered) sheet pile walls

- This failure mode is easily solved by multiple Slope Stability programs. The program develops multiple sliding curves with different focal points and calculates slope stability Factor Safety against slip circle failure.
- That Factor Safety shall not be less than 1.2
  F.S. < 1.2 for Normal Load Combinations is considered unsafe.
- For Abnormal Load Combinations F.S. can be reduced to 1.1 to 1.15
- It should be understood that even though tieback is developed way beyond the rupture wedge, slope failure zone may encompass the whole development length of the tieback as it is evident from the diagram.

$$\text{Driving Moment} = \sum_{i=1}^{N} W_i \times \ell_i \text{ or } R\sum_{i=1}^{N} T_i$$

$$\text{Resisting Moment} = R\sum_{i=1}^{N} L_i c_i + R\sum_{i=1}^{N} N_i \tan\phi_i$$

where
W_i = weight of the i^{th} slice
ℓ_i = lever arm of the i^{th} slice about 0
L_i = length of circular arc at the base of the i^{th} slice.
c_i = cohesion at the base of the i^{th} slice.
N_i = normal component of the weight of the i^{th} slice.
R = radius of the circular arc.
$\tan\phi_i$ = angle of internal friction at the base of the i^{th} slice.



Thanks For Watching!

We hope we have made things a little easier for you to:

- Evaluate your next Sea Wall project
- Properly select engineer for your next project

We greatly appreciate Kelly White and the Florida Marine Contractor's Association for having us!