9. Seismic Design of

RETAINING STRUCTURES

Part A:

GRAVITY WALLS

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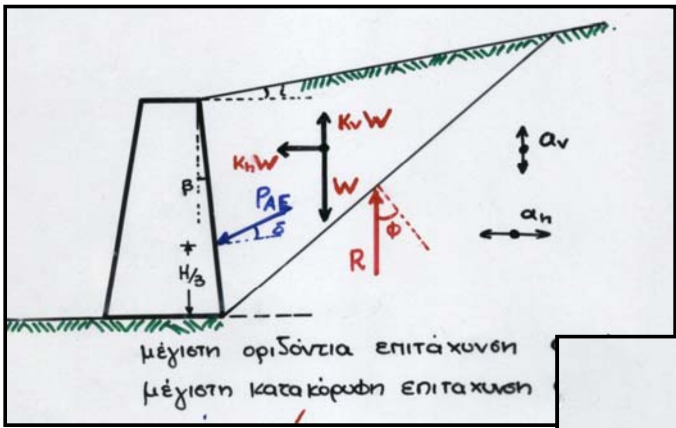
Professor of NTUA

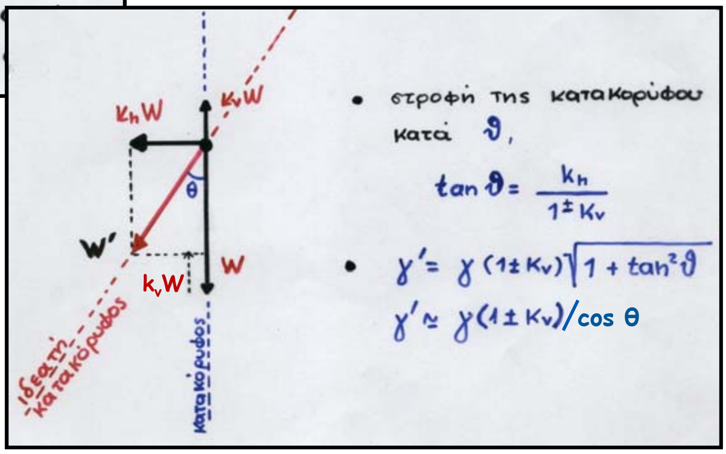
October 2016

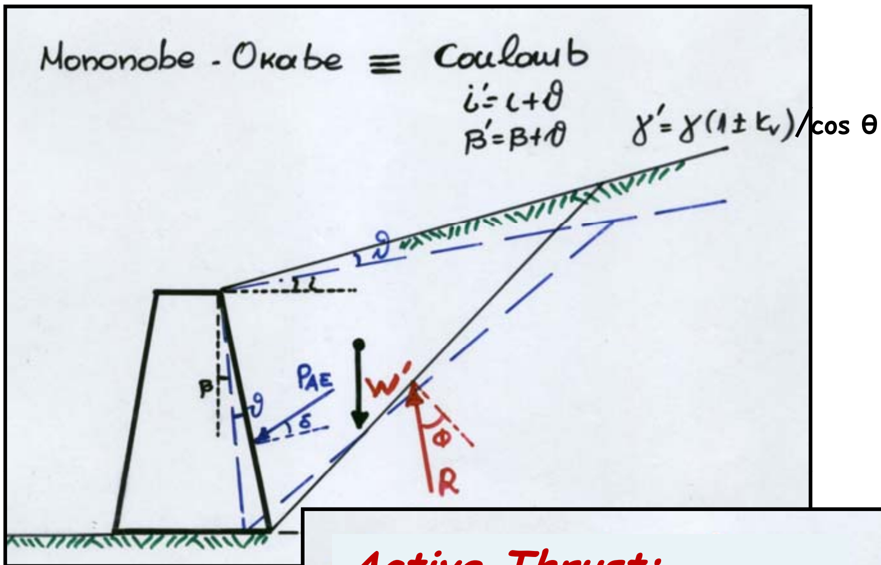
CONTENTS

9.1 DYNAMIC EARTH PRESSURES for DRY SOIL

The method of MONONOBE - OKABE



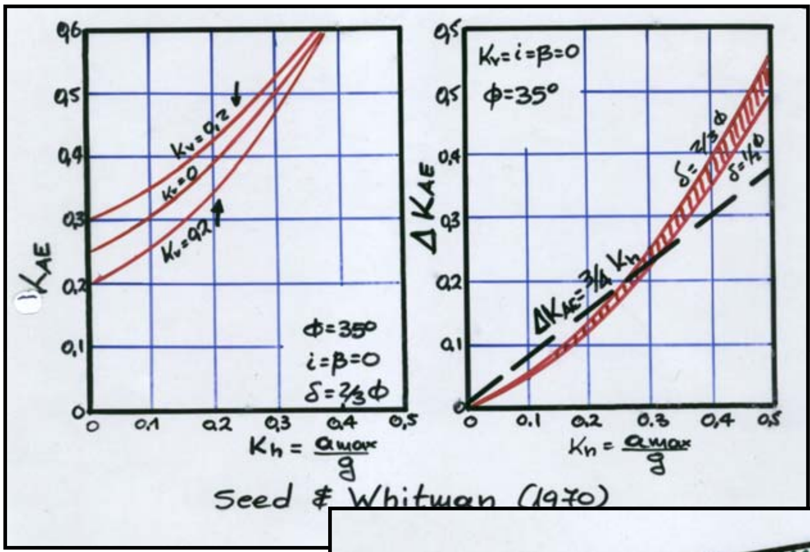




Active Thrust:

$$P_{AE} = \frac{1}{2}\gamma H^2 (1 \pm k_v)\, K_{AE}$$

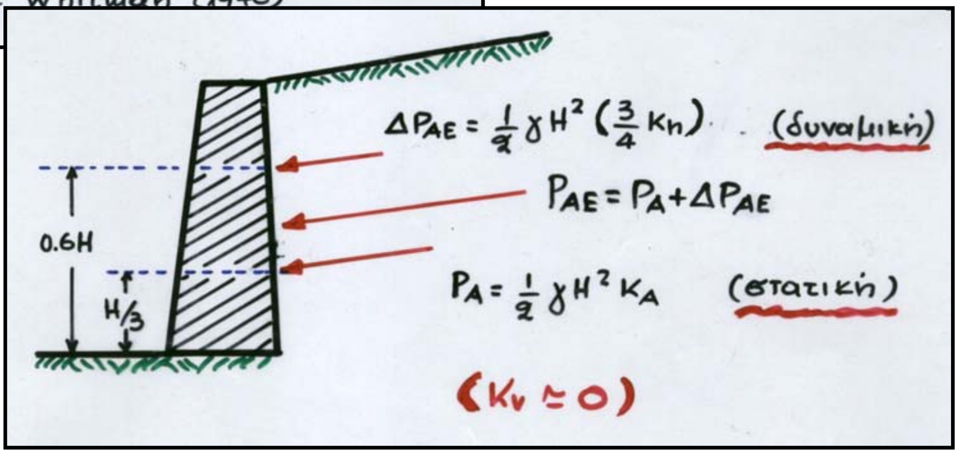
$$K_{AE} = \frac{\cos^2(\varphi - \beta - \theta)}{\cos\theta \cdot \cos^2\beta \cdot \cos(\delta + \beta + \theta)\left[1 + \sqrt{\frac{\sin(\varphi+\delta)\,\sin(\varphi-\theta-i)}{\cos(\delta+\beta+\theta)\,\cos(i-\beta)}}\right]^2}$$



why not …….

$$\Delta K_{AE} = 1.50\ k_h^{1.50}$$

G.B. 2014 !



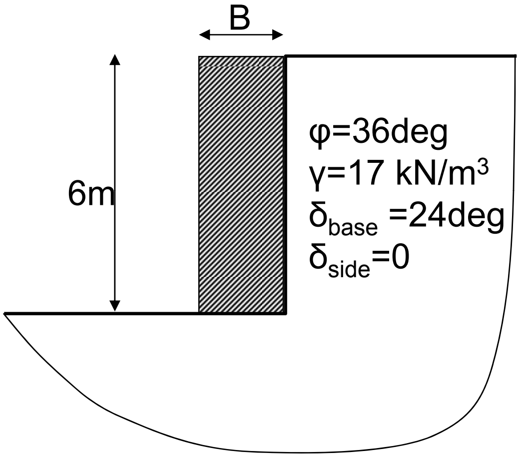
Homework 9.1

For the retaining wall shown in the figure,

(a) Compute B so that the factors of safety against sliding ($FS_{ολ.}$) and against overturning ($FS_{αν.}$) under static loading are equal to (or higher than) 1.50.

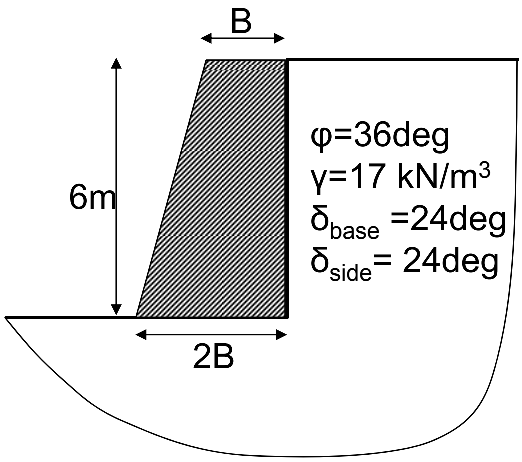
(b) In the sequel, compute $FS_{ολ.}$ και $FS_{αν}$ for seismic loading with k_h=0.15 and k_v=0.

(c) Plot the ($FS_{ολ}$÷ B) and ($FS_{αν}$÷ B) variation for static, as well as, for seismic loading. Comment and explain the effectiveness of B increase in the two loading cases.



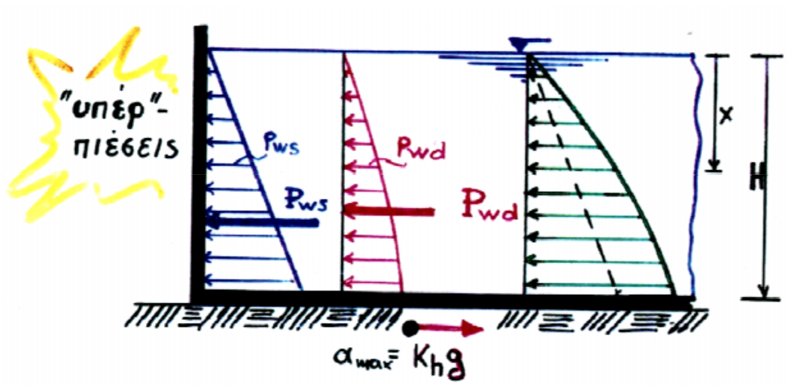
Homework 9.2

Repeat Homework 9.1 for the retaining wall shown in the figure.



9.2 HYDRO-DYNAMIC PRESSURES

WESTERGAARD (1933)

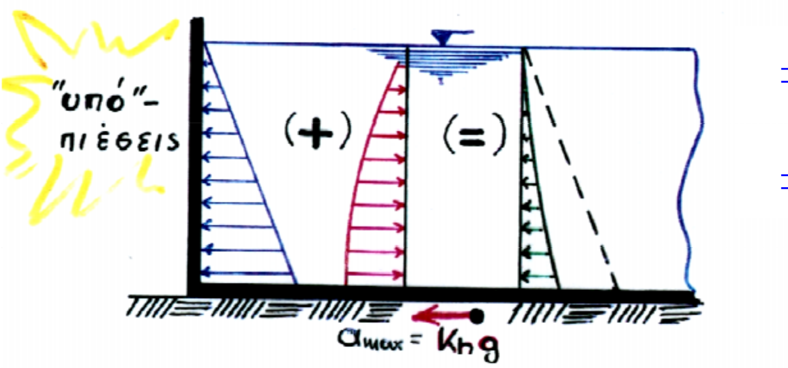


Hydro-STATIC pressures

$$p_{ws}(x) = \gamma_w \bullet x$$

$$P_{ws} = \int_0^H p_{ws}(x)dx = \frac{1}{2}\gamma_w H^2$$

application point: *H/3 from base*



Hydro-DYNAMIC pressures

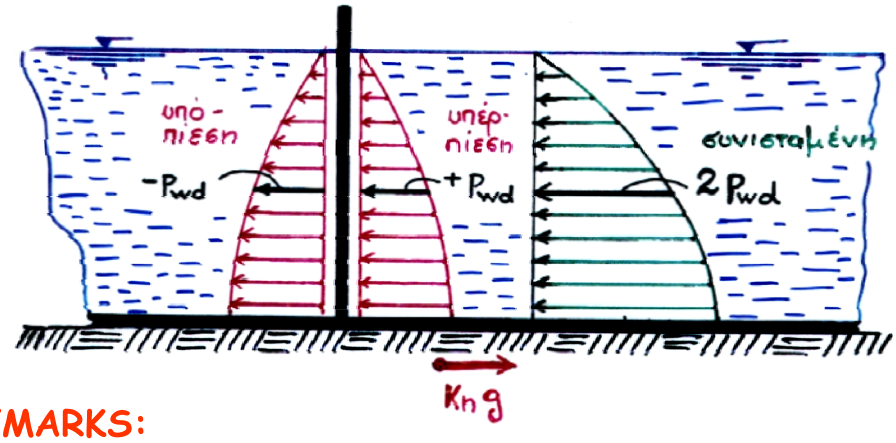
$$\pm p_{wd}(x) = \frac{7}{8}k_h\gamma_w H\sqrt{x/H}$$

$$\pm P_{wd} = \frac{7}{12}k_h\gamma_w H^2 \quad (= 1.17 k_h P_{ws})$$

application point: *0.40H from base*

ATTENTION !

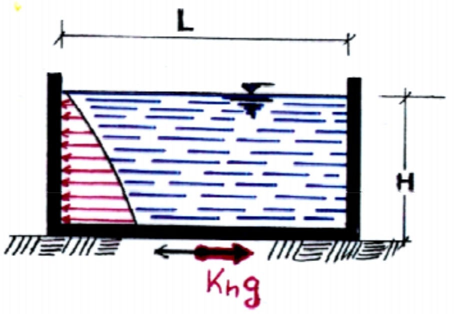
The excess pore pressures are positive in front of the wall and negative behind it. Thus the total hydro-dynamic pressure acting on a submerged wall is twice that given by the Westergard solution!



REMARKS:

Westergaard theory applies under the following assumptions:

- **free water (no backfill)**
- **vertical wall face**
- **very large (theoretically infinite) extent of water basin**



Effect of tank width

$$\pm p_{wd}(x) = \frac{7}{8} C_n k_h \gamma_w H \sqrt{x/H}$$

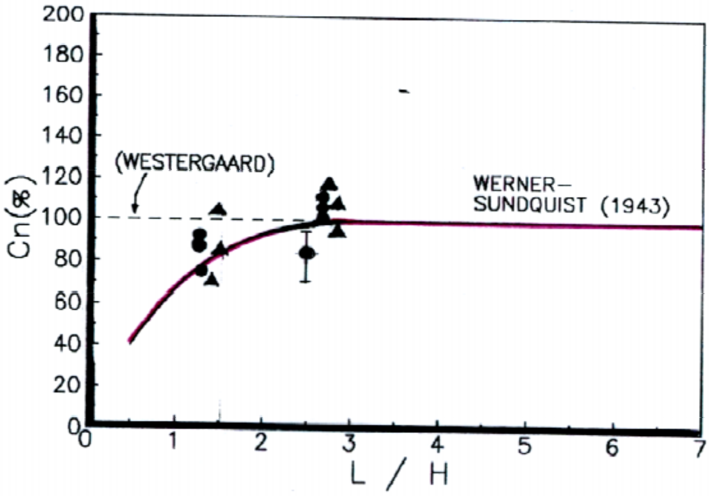
$$\pm P_{wd} = \frac{7}{12} C_n k_h \gamma_w H^2$$

$$\left(= 1.17 C_n k_h P_{ws}\right)$$

όπου

$$C_n = \frac{4}{3}\frac{L/H}{1 + L/H} < 1.0$$

$$(C_n = 1.00 \quad \gamma\iota\alpha \quad L/H > 2.70)$$



application point: ***0.40H from the base***

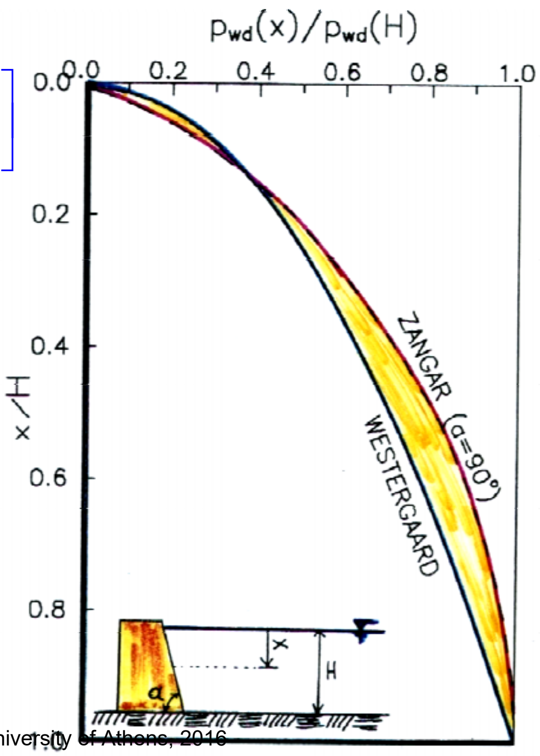
Effect of wall inclination

Zangar (1953) & Chwang (1978)

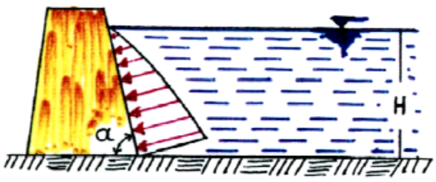
$$\pm p_{wd}(x,\alpha) = C_m(\alpha) k_h \gamma_w H \left[\frac{x}{H}\left(2 - \frac{x}{H}\right) + \sqrt{\frac{x}{H}\left(2 - \frac{x}{H}\right)}\right]$$

or, approximately

$$\pm p_{wd}(x,\alpha) = C_m(\alpha) \underbrace{\left[\frac{7}{8} k_h \gamma_w H \sqrt{\frac{x}{H}}\right]}_{\text{Westergaard}}$$



Effect of wall inclination



$$\pm p_{wd}(x,\alpha) = \frac{7}{8} C_m k_h \gamma_w H \sqrt{\frac{x}{H}}$$

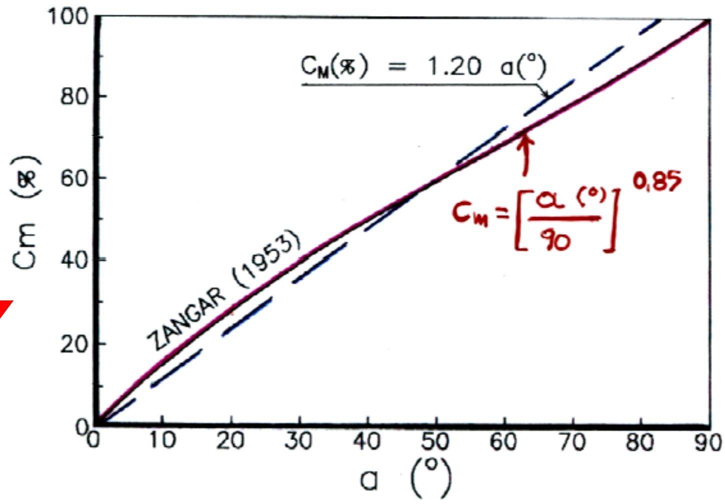
and

$$\pm P_{wd} = \frac{7}{12} C_m k_h \gamma_w H^2$$

$$(= 1.17 C_m k_h P_{ws})$$

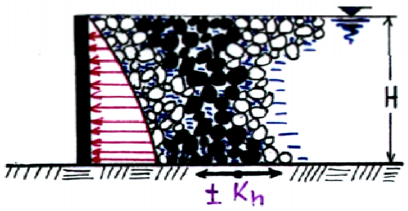
where

$$C_m \approx 0.012\alpha(^o) \approx 2.0 \frac{\alpha(rad)}{\pi}$$



application point: *0.40H από την βάση*

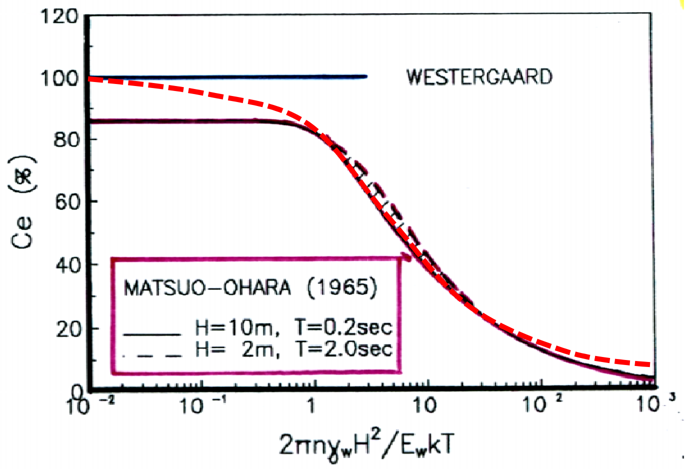
9.3 DYNAMIC PRESSURES for SATURATED FILL



$$\pm p_{wd}(x,e,..) = \frac{7}{8} C_e k_h \gamma_w H \sqrt{x/H}$$

$$\pm P_{wd}(e,..) = \frac{7}{12} C_e k_h \gamma_w H^2$$

$$(\approx 1.17 C_e k_h P_{ws})$$



Matsuzawa (1985)

$$C_e \approx 0.5 - 0.5\tanh\left[\log\frac{2\pi n\gamma_w H^2}{7E_w kT}\right]$$

με

n = πορώδες

γ_w = ειδικό βάρος νερού

H = βάθος νερού

E_w =Μέτρο συμπ. νερού ($\approx 2\cdot10^6$kPa)

k = συντελεστής διαπερατότητας

T = δεσπόζουσα περίοδος δόνησης

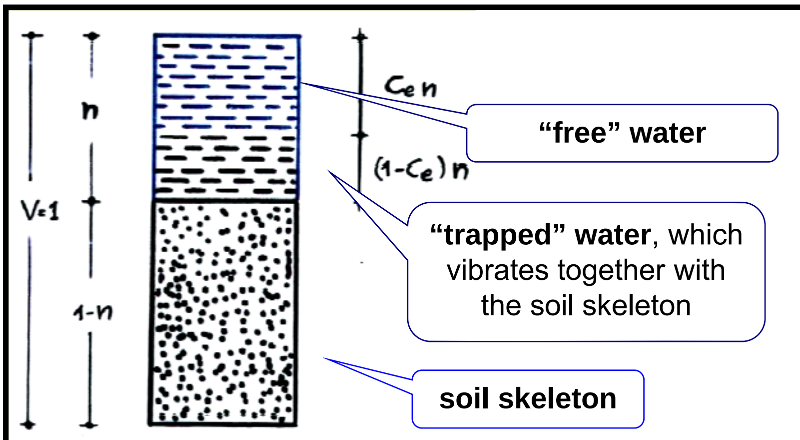
WATER + BACKFILL

Physical analog (Matsuzawa et al. 1985)

in other words….

Correction factor ***C_e*** expresses the portion of pore water which vibrates **FREELY**,

i.e. independently from the soil skeleton.



Hence, dynamic earth pressures are exerted by the **soil skeleton** AND **the "trapped water"** and consequently (you may prove it easily) the Mononobe-Okabe relationships apply for :

$$\gamma^* = \gamma_{DRY} C_e + \gamma_{SAT}(1 - C_e)$$

EXAMPLE:

n=40%, γ_w=10 kN/m^3
E_w=2x10^6 kPa, T=0.30 sec

$$C_e = 0.5 - 0.5\tanh\left[\log\left(6\cdot 10 - 6\frac{H^2}{k}\right)\right]$$

Fill Material	k (m/s)	Ce H=5m	Ce H=10m	Ce H=20m
well graded gravel	10^1	1.0	1.0	1.0
gravel	10^0	1.0	1.0	1.0
coarse sand	10^{-2}	1.0	0.95	0.80
fine sand	10^{-4}	0.42	0.16	0.04
silt	10^{-6}	0.0	0.0	0.0
Clayey sand & gravel	10^{-8}	0.0	0.0	0.0

C_e > 0.80 →
p_{wd} ≈ Westergaard

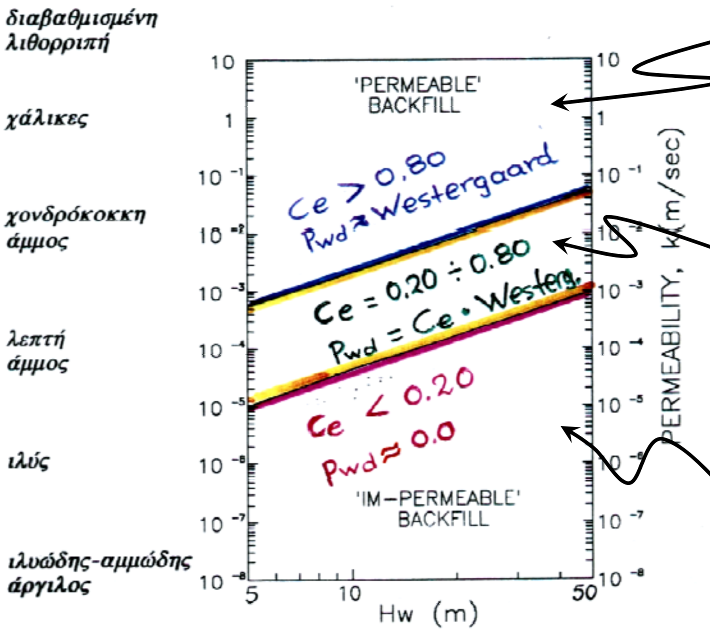
C_e = 0.20÷0.90 →
p_{wd} ≈ $C_e\cdot$Westergaard

C_e < 0.20 →
p_{wd} ≈ 0

EXAMPLE:

n=40%, γ_w=10 kN/m^3
E_w=2 106 kPa, T=0.30 sec

$$C_e = 0.5 - 0.5\tanh\left[\log\left(6\cdot 10 - 6\frac{H^2}{k}\right)\right]$$



"Permeable" fill:
Cobbles, gravel,
Coarse sand (H<20m)

"Semi-permeable» fill:
coarse sand (H > 20m),
fine sand (H < 20m)

"Impermeable" fill:
silt, clay, clayey or silty sand and gravel

SUMMARY of Hydrodynamic Pressures

Hydrodynamic pressures on the sea-side of the wall

$$p_{wd}(x) = \frac{7}{8} C_m C_n k_h \gamma_w H \sqrt{x/H}$$

$$P_{wd} = \frac{7}{12} C_m C_n k_h \gamma_w H^2$$

$$\left(= 1.17 C_m C_n k_h P_{ws}\right)$$

C_m = effect of inclined wall
C_n = effect of water basin length

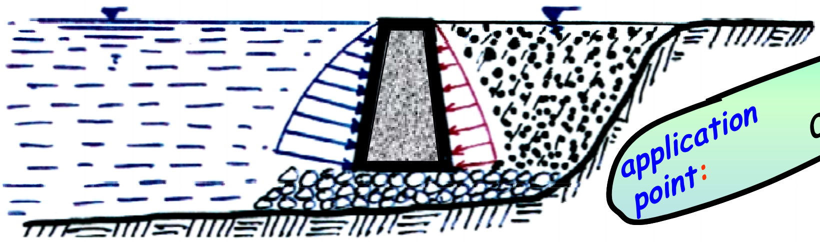
Hydrodynamic pressures on the fill-side of the wall

$$p_{wd}(x) = \frac{7}{8} C_m C_n C_e k_h \gamma_w H \sqrt{x/H}$$

$$P_{wd} = \frac{7}{12} C_m C_n C_e k_h \gamma_w H^2$$

$$\left(= 1.17 C_m C_n C_e k_h P_{ws}\right)$$

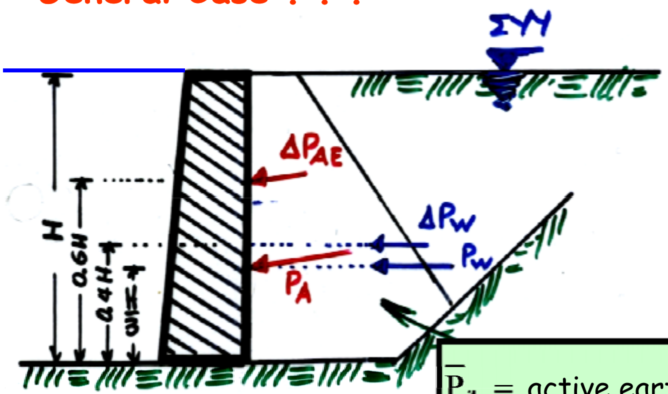
C_e = effect of filll



application point: 0.40H from base

9.4 PSEUDO – STATIC DESIGN

General Case . . .



$\bar{P}_a$ = active earth pressure $\left[= \frac{1}{2} k_a \, (\gamma_{SAT} - \gamma_W) H^2 \right]$

$P_W = \frac{1}{2} \gamma_W H^2$

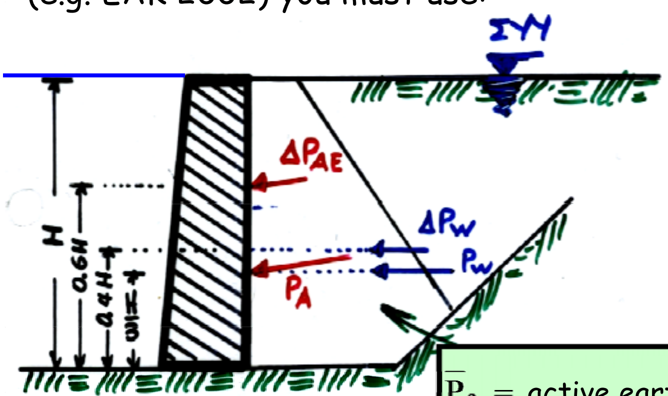
$\Delta P_W = P_{Wd}$ = hydrodynamic pressures

ΔP_{AE} = dynamic earth pressures $\left[= \frac{1}{2} (\frac{3}{4} k_h) \gamma^* H^2 \right]$

with $\gamma^* = C_e \, \gamma_{DRY} + (1 - C_e) \gamma_{SAT}$

ATTENTION !

P_a computation requires $(\gamma_{\kappa o \rho} - \gamma_w)$ while ΔP_{AE} computation requires γ^*. Thus, when it is necessary to compute both P_a and ΔP_{AE} with a common unit weight (e.g. EAK 2002) you must use:



- the buoyant unit weght $(\gamma_{SAT} - \gamma_w)$
- a modified seismic coefficient

$$k_h^* = k_h \frac{\gamma^*}{\gamma_{SAT} - \gamma_w}$$

$\bar{P}_a$ = active earth pressure $\left[= \frac{1}{2} k_a \, (\gamma_{SAT} - \gamma_W) H^2 \right]$

$P_W = \frac{1}{2} \gamma_W H^2$

$\Delta P_W = P_{Wd}$ = hydrodynamic pressures

ΔP_{AE} = dynamic earth pressures $\left[= \frac{1}{2} (\frac{3}{4} k_h) \gamma^* H^2 \right]$

with $\gamma^* = C_e \, \gamma_{DRY} + (1 - Ce) \gamma_{SAT}$

Special case: «IMPERMEABLE» fill (C_e = 0)

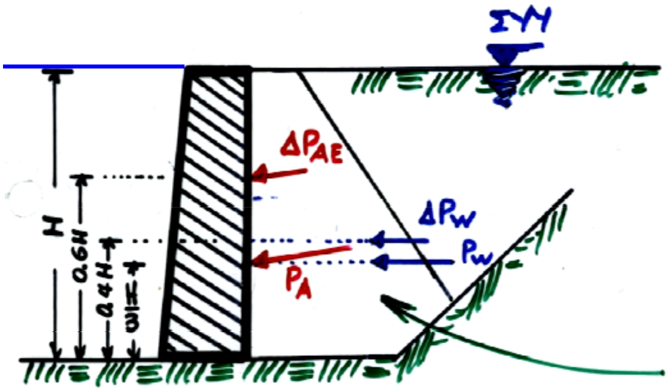
$\bar{P}_a$ = active earth pressure $\left[-\frac{1}{2}k_a\,(\gamma_{SAT}-\gamma_W)H^2\right]$

$P_W = \frac{1}{2}\gamma_W H^2$

$\Delta P_W = P_{Wd}$ = hydrodynamic pressures = 0 (C_e=0)

ΔP_{AE} =dynamic earth pressures $\left[-\frac{1}{2}(\frac{3}{4}k_h)\gamma^* H^2\right]$

with $\gamma^* = \gamma_{SAT}$ (Ce=0)



- Clayey sand
- Clayey silt
- Silty sand
- Clayey or silty gravel

Special case: «IMPERMEABLE» fill (C_e = 0)

$\bar{P}_a$ = active earth pressure $\left[=\frac{1}{2}k_a\,(\gamma_{SAT}-\gamma_W)H^2\right]$

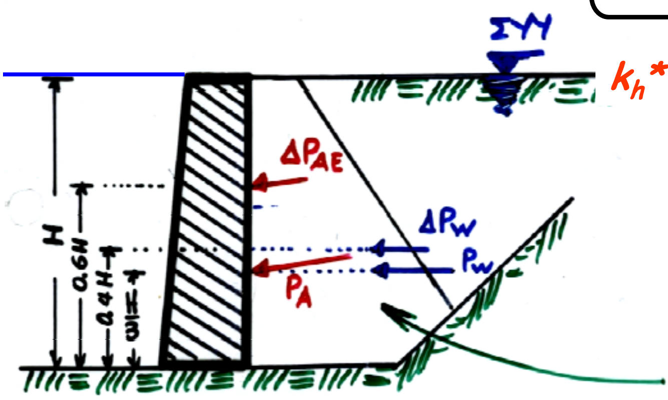
$P_W = \frac{1}{2}\gamma_W H^2$

$\Delta P_W = P_{Wd}$ = hydrodynamic pressures = 0 (Ce=0)

ΔP_{AE} =dynamic earth pressures $\left[=\frac{1}{2}(\frac{3}{4}k_h)\gamma^* H^2\right]$

$$= \left[\frac{3}{8}(k_h\frac{\gamma_{SAT}}{\gamma_{SAT}-\gamma_W})(\gamma_{SAT}-\gamma_W)H^2\right]$$

$k_h^* \approx 2.2\ k_h$



- Clayey sand
- Clayey silt
- Silty sand
- Clayey or silty gravel

Special case: «PERMEABLE» fill C_e=1

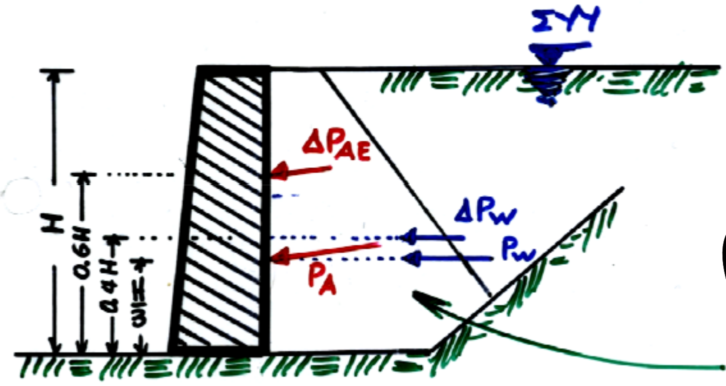
$\overline{P}_a$ = active earth pressure $\left[=\frac{1}{2}k_a(\gamma_{SAT}-\gamma_w)H^2\right]$

$P_W = \frac{1}{2}\gamma_W H^2$

$\Delta P_W = P_{Wd}$ = hydrodynamic pressure ≠ 0

ΔP_{AE} = dynamic earth pressure $\left[=\frac{1}{2}(\frac{3}{4}k_h)\gamma^* H^2\right]$

with $\gamma^* = \gamma_{DRY}$ $(C_e = 1)$



- sand
- sand & gravel
- cobbles
- ballast

Special case: «PERMEABLE» fill C_e=1

$\overline{P}_a$ = active earth pressure $\left[=\frac{1}{2}k_a(\gamma_{SAT}-\gamma_w)H^2\right]$

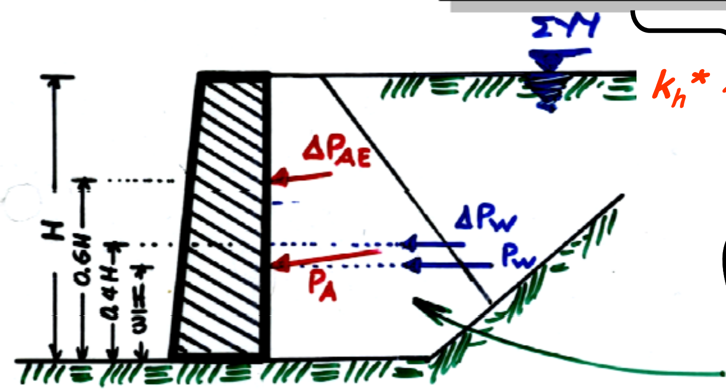
$P_W = \frac{1}{2}\gamma_W H^2$

$\Delta P_W = P_{Wd}$ = hydrodynamic pressure ≠ 0

ΔP_{AE} = dynamic earth pressure $\left[=\frac{1}{2}(\frac{3}{4}k_h)\gamma^* H^2\right]$

$$= \left[\frac{3}{8}k_h \frac{\gamma_{DRY}}{(\gamma_{SAT}-\gamma_w)}(\gamma_{SAT}-\gamma_w)H^2\right]$$

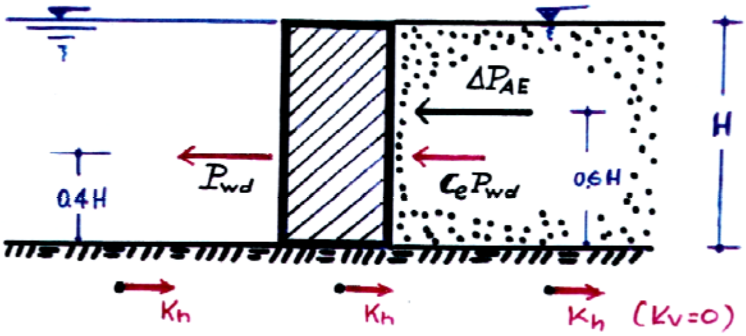
$k_h^* \approx 1.6\ k_h$



- sand
- sand & gravel
- cobbles
- ballast

EXAMPLE: What do I do when I am not sure about the permeability of the fill material?

Vertical & smooth wall
Basin of infinite length } $C_m = C_n = 0$



Fill:

γ_{DRY}=16 kN/m^3

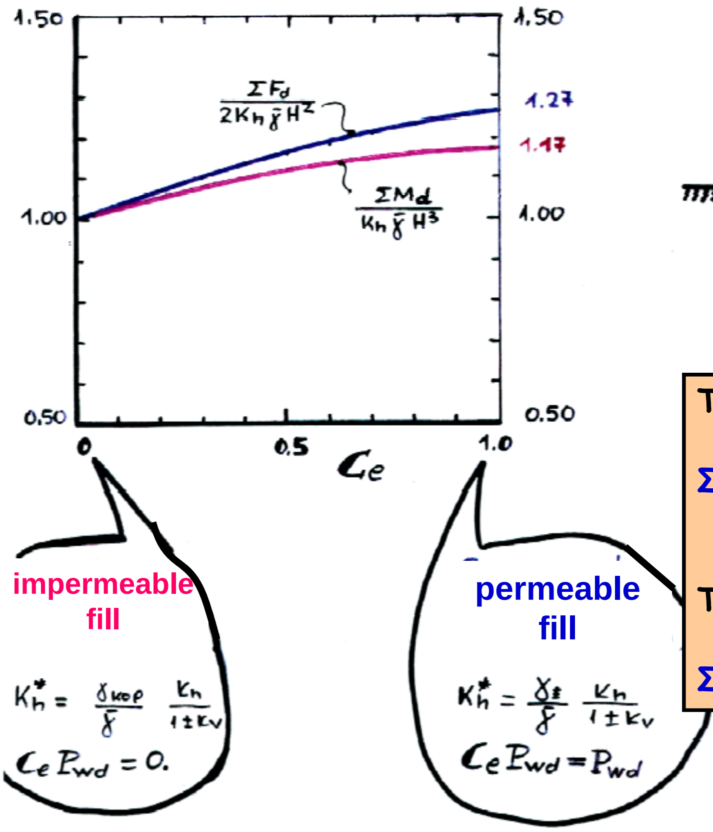
γ_{SAT}= 20 kN/m^3

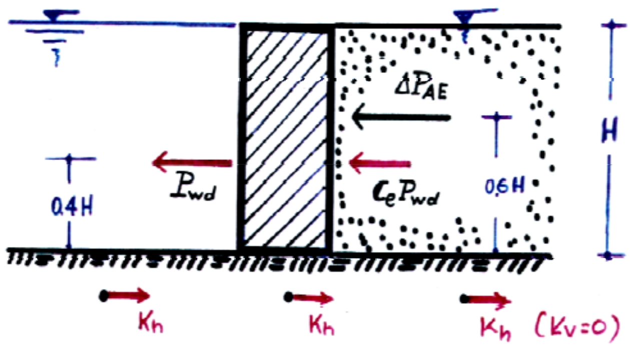
C_e = 0 ÷ 1.0

$$\Delta P_W = P_{Wd} = \frac{7}{12} k_h \gamma_w H^2$$

$$\Delta P_{AE} = \frac{1}{2}\left(\frac{3}{4} k_h^*\right)(\gamma_{SAT} - \gamma_w) H^2$$

με $$k_h^* = \frac{C_e \gamma_{DRY} + (1 - C_e)\gamma_{SAT}}{\gamma_{SAT} - \gamma_w} k_h$$





Total horizontal thrust:

$\Sigma F_d = \Delta P_{AE} + P_{wd} + C_e P_{wd}$

Total overturning moment:

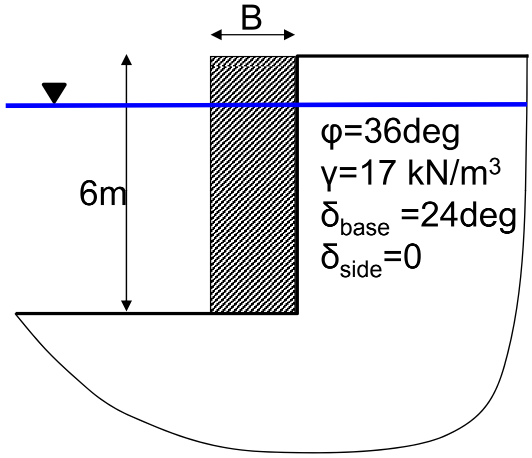
$\Sigma M_d = 0.60H\ \Delta P_{AE} + 0.40H\ (1 + C_e)\ P_{wd}$

Homework 9.3

For the quay wall shown in the figure,

(a) Compute B so that the factors of safety against sliding ($FS_{ολ.}$) and against overturning ($FS_{αν.}$) under static loading are equal to (or higher than) 1.50.

(b) In the sequel, compute $FS_{ολ.}$ και $FS_{αν}$ for seismic loading with k_h=0.15 and k_v=0.

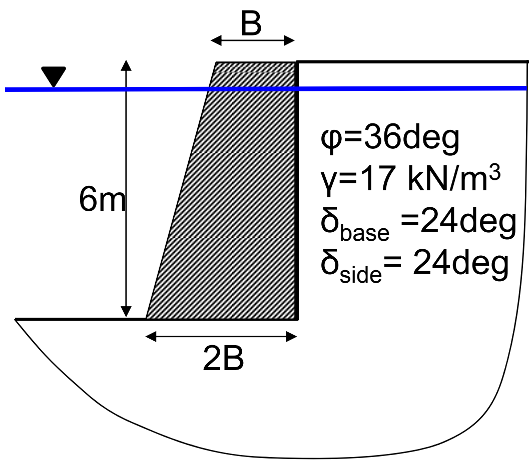


Notes:

- Solve for two backfill options: medium-coarse sand ($k=10^{-3}$m/s) or silty sand ($k=10^{-5}$m/s).
- For simplicity, assume that the sea level coincides with the ground surface.
- Compare with Hwk 9.1

Homework 9.4

Repeat Homework 9.3 for the quay wall shown in the figure.



9. Seismic Design of

RETAINING STRUCTURES

Part B:

WALLS
WITH LIMITED DISPLACEMENET

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October 2016

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Problem outline …..

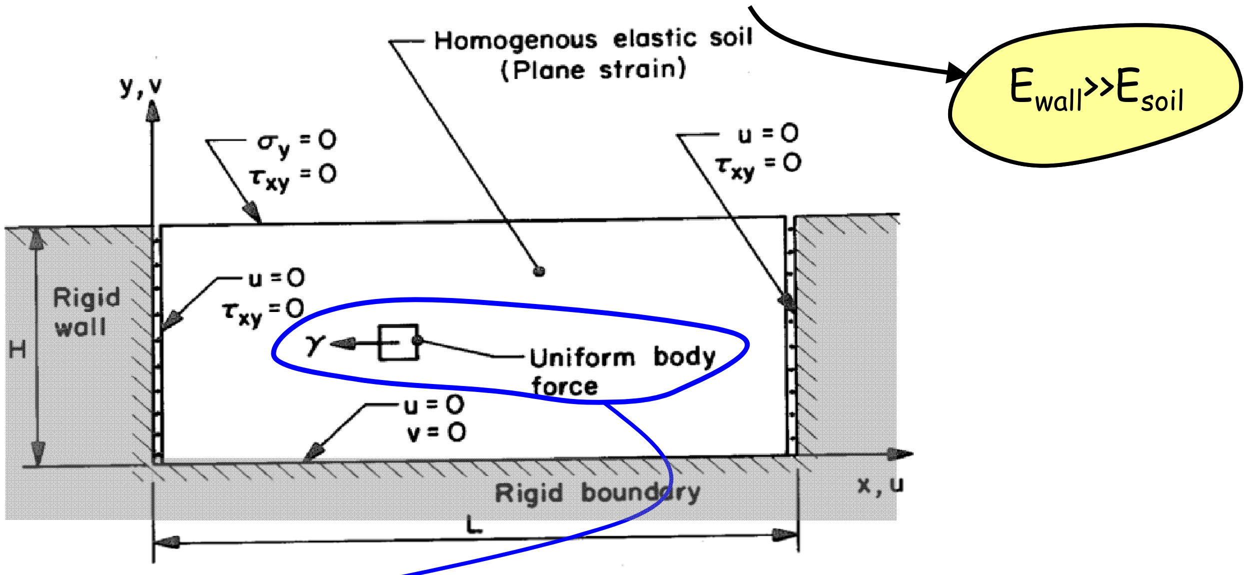
The Mononobe-Okabe method requires that the retaining wall can move freely (slide or rotate) so that active earth pressures develop behind the wall.

Nevertheless, there are cases where the free movement of the wall is totally or partially restrained (e.g. basement walls, braced walls, massive walls embeded in rock like formations) .

Solutions for "perfectly rigid" or "semi-rigid" walls

9.5 PERFECTLY RIGID WALLS (Wood 1973)

Elastic soil between two **rigid** walls

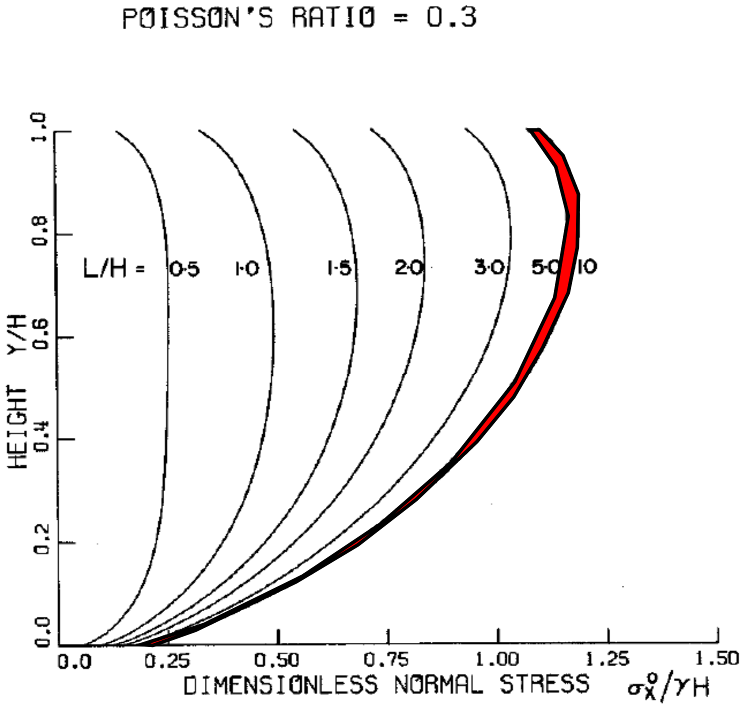


Assumptions

1. **Pseudo static** conditions ($T_{διεγερ} >> 4H/V_s$) – *quite usual case* ***(why?)***
2. plane strain
3. Elastic soil
4. **Smooth** & **rigid** walls

Analytical Solutions for ……..

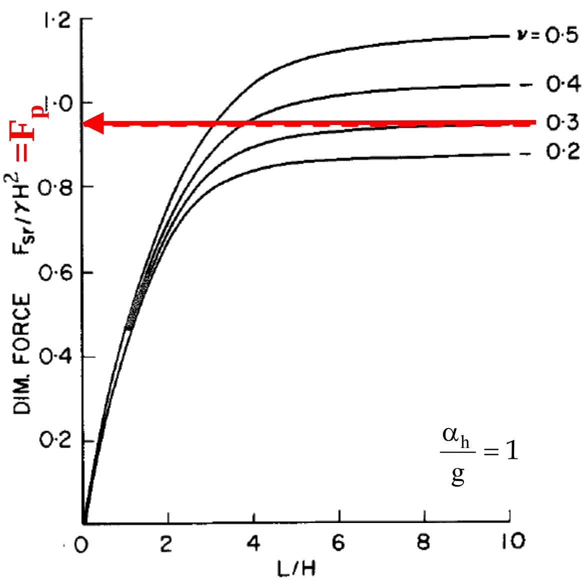
dynamic earth pressures

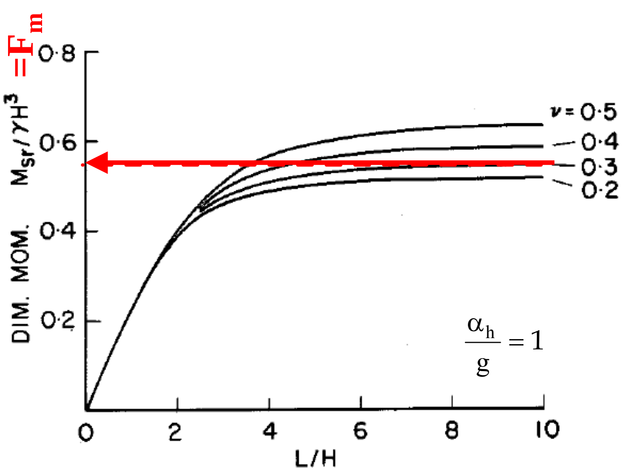


$$\text{για}\ \frac{\alpha_h}{g} = 1$$

Analytical Solutions for ……..

Overturning moment and base shear



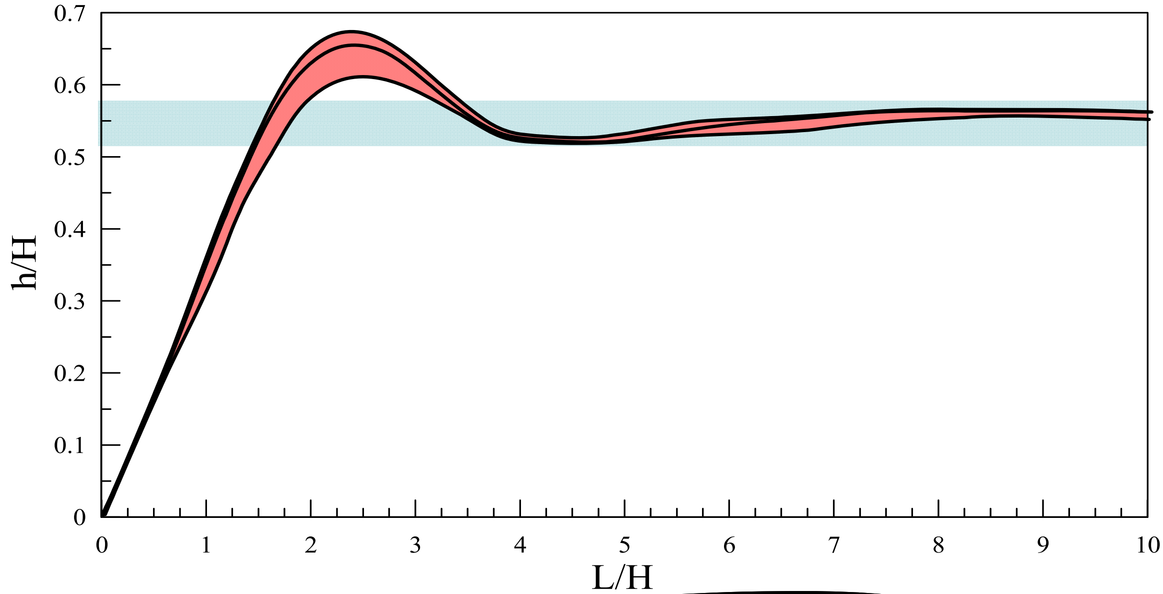


$$\Delta F_{eq} = F_P \gamma H^2 \frac{\alpha_h}{g}$$

$$\Delta M_{eq} = F_m \gamma H^3 \frac{\alpha_h}{g}$$

Analytical Solutions for ……..

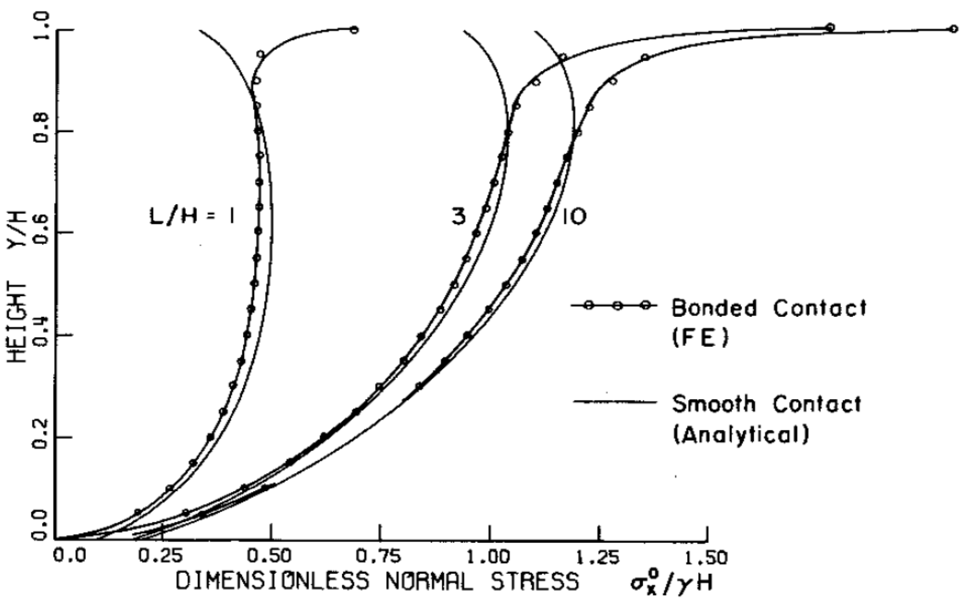
Application point of the resultant seismic thrust $h = \frac{\Delta M_{eq}}{\Delta F_{eq}}$



$$\frac{h}{H} \approx 0.55 \quad \left(\gamma\iota\alpha \ \frac{L}{H} > 4\right)$$

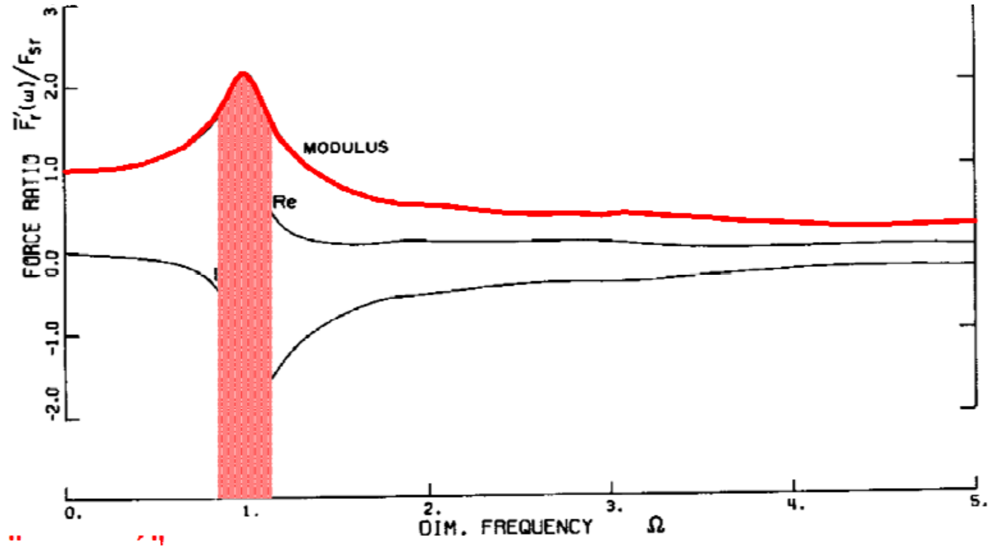
Analytical Solutions for ……..

Smooth vs. bonded (rough) wall side



Analytical Solutions for ……..

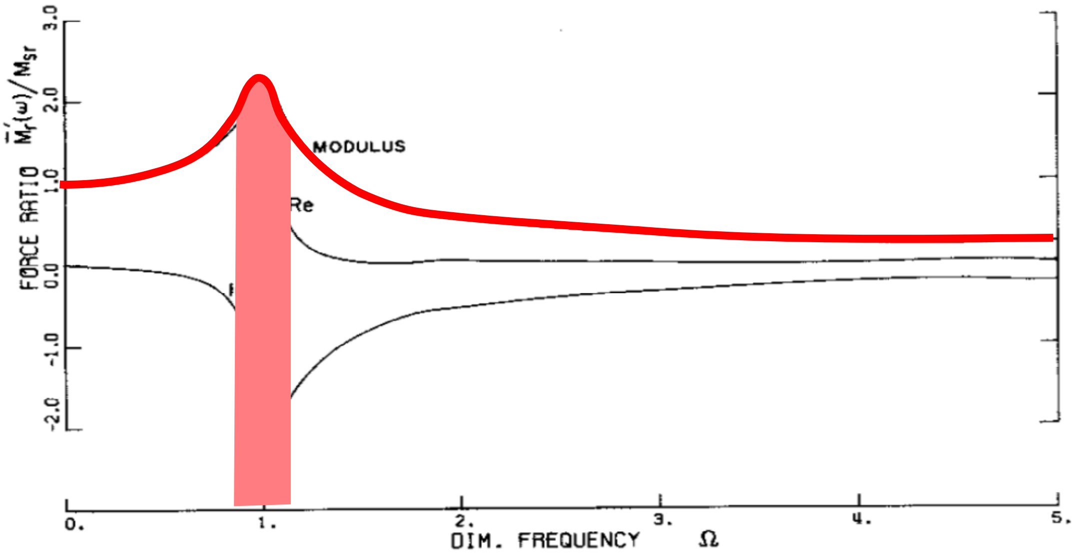
Extension to harmonic base excitation – **Base shear**



$$\Omega = \frac{\omega_{excit}}{\omega_{soil}} = \frac{T_{soil}}{T_{excit}}$$

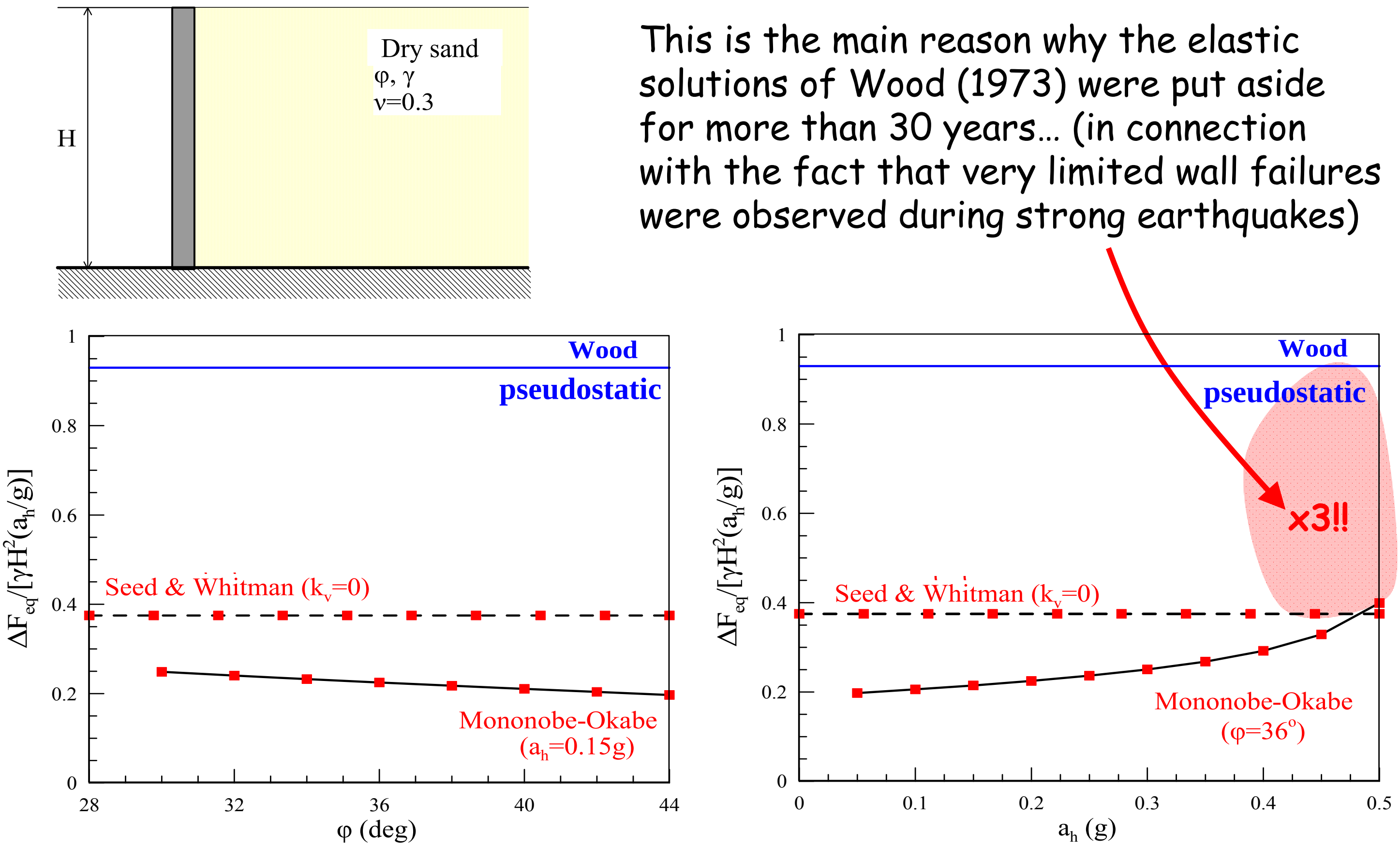
Analytical Solutions for ……..

Extension to harmonic base excitation – **Overturning Moment**



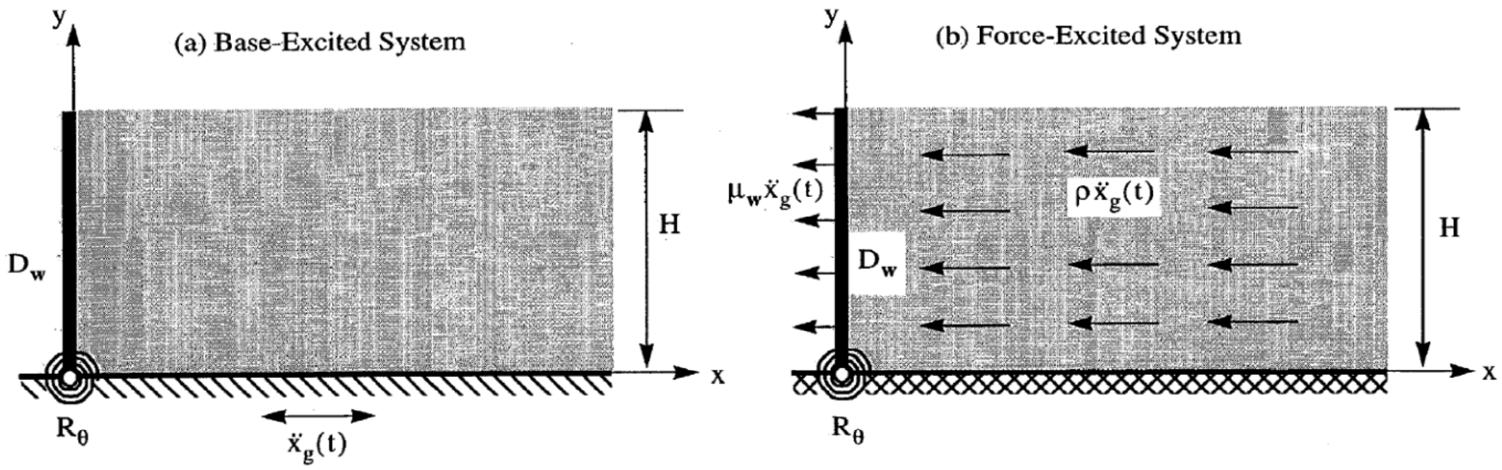
$$\Omega = \frac{\omega_{excit}}{\omega_{soil}} = \frac{T_{soil}}{T_{excit}}$$

Comparison with Mononobe - Okabe



This is the main reason why the elastic solutions of Wood (1973) were put aside for more than 30 years… (in connection with the fact that very limited wall failures were observed during strong earthquakes)

9.6 WALLS WITH LIMITED DISPLACEMENT (displacement & rotation, Veletsos & Yunan, 1996)



$$d_w = \frac{GH^3}{D_w}$$ Relative translational rigidity of the wall-fill system

$$D_w = \frac{E_w t_w^3}{12\left(1-\nu_w^2\right)}$$

$$d_\theta = \frac{GH^2}{R_\theta}$$ Relative rotational rigidity of the wall-fill system

Assumptions

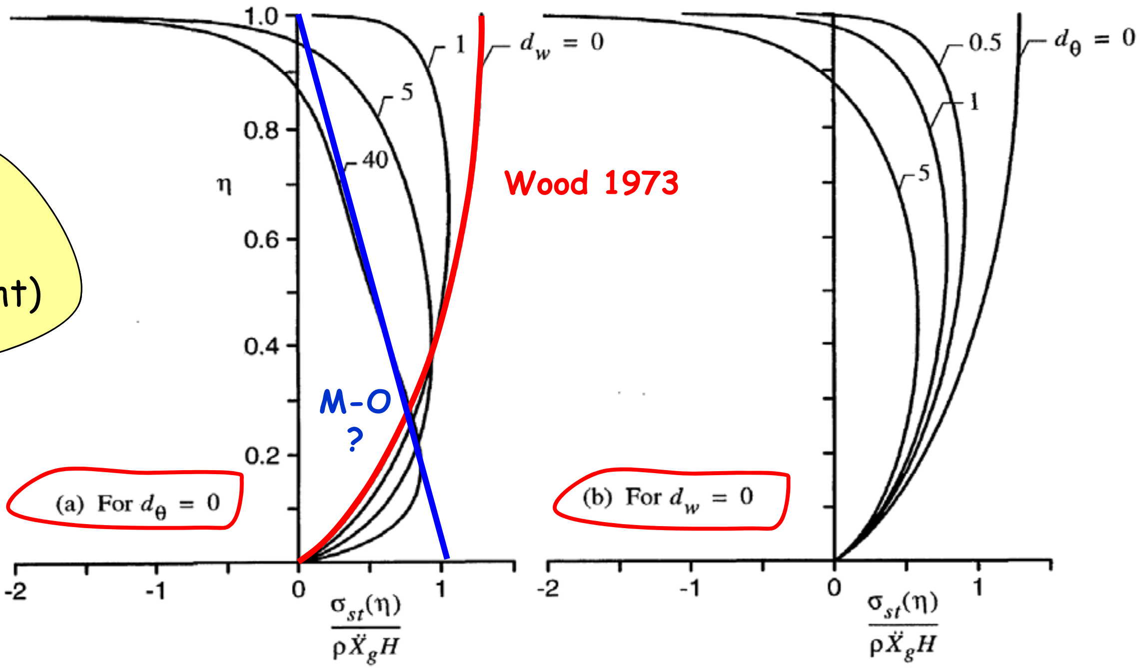
- bonded wall-soil
- mass-less wall
- 5% soil damping
- 2% wall damping

Analytical solutions for …….

Pseudo-static **earth pressures**

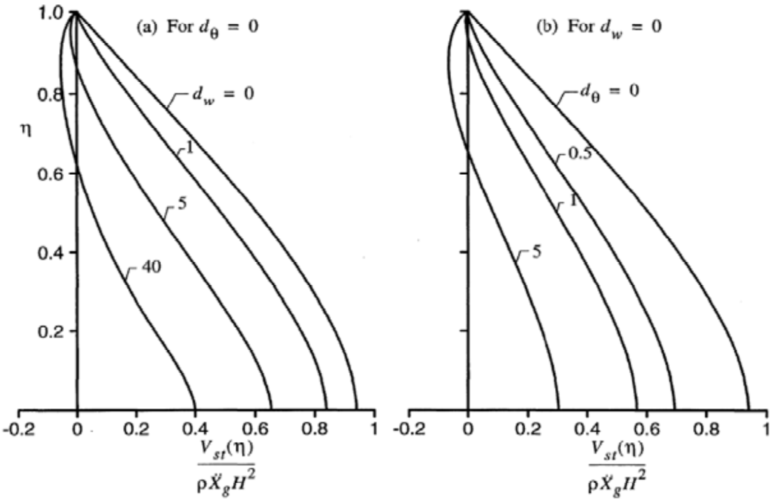
$$\eta = \frac{h}{H}$$

(normal. height)

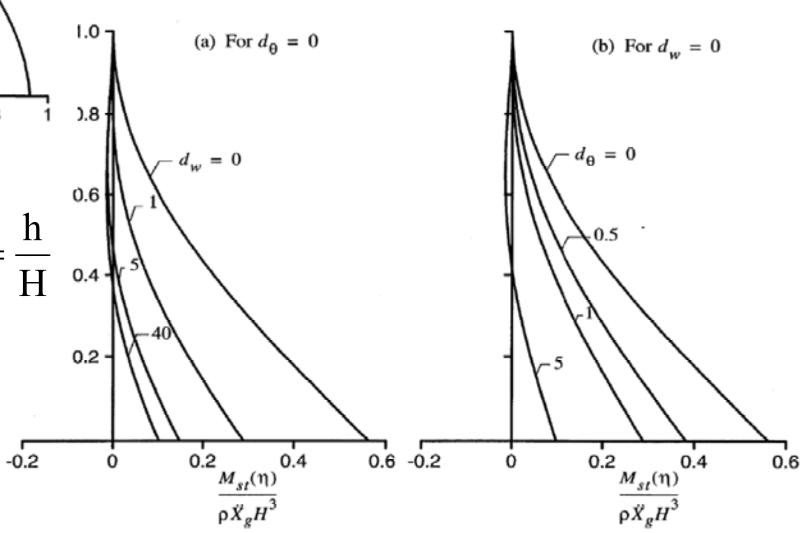


Analytical solutions for …….

Pseudo-static shear forces & bending moments

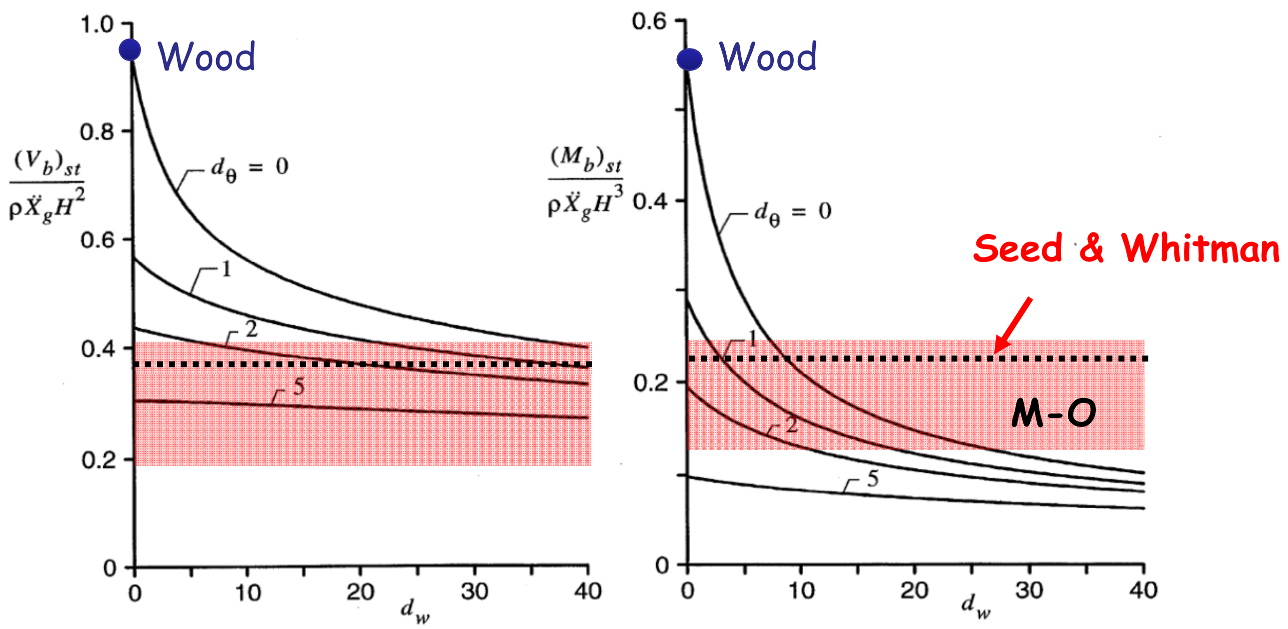


$$\eta = \frac{h}{H}$$



Analytical solutions for …….

Pseudo static
base shear** & **overturning moment



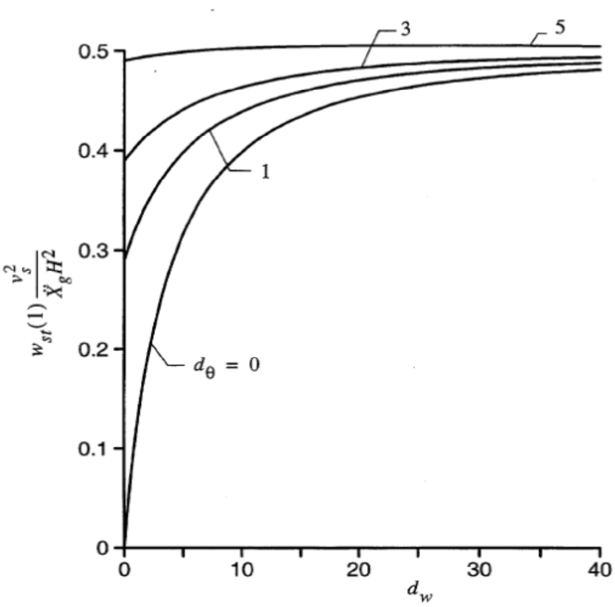
HWK 9.5:

(a) Based on the diagrams of the previous slide, draw the diagrams h/H – d_W, where h is the distance from the base of the resultant dynamic pressure.

(b) Compare with the solutions of Wood, M-O kai Seed & Whitman

Analytical solutions for …….

Pseudo static displacements…

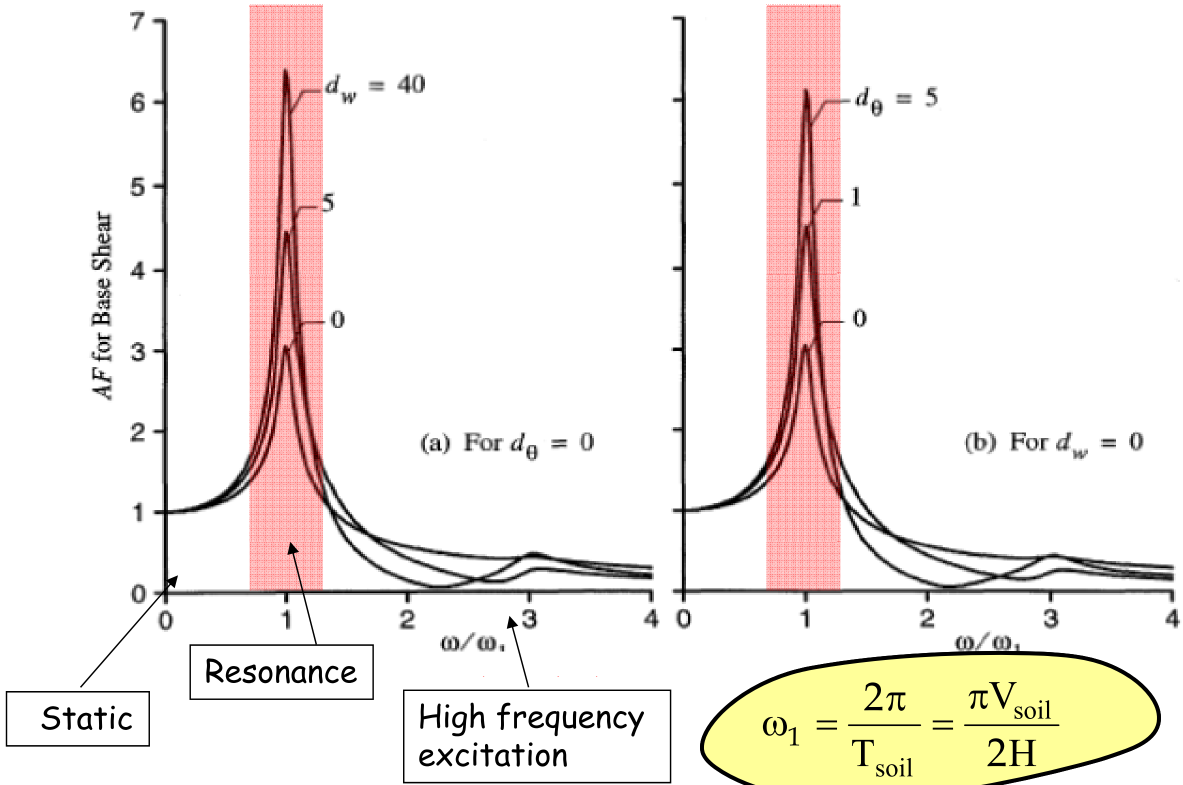


For a flexible (compared to the fill) concrete wall (d_w=20) and a seismic excitation with a_{max}=0.3g, the resulting displacement is U/H=0.13%...

[U=0.1%-0.4%H for active "failure" → M-O]

Analytical solutions for …….

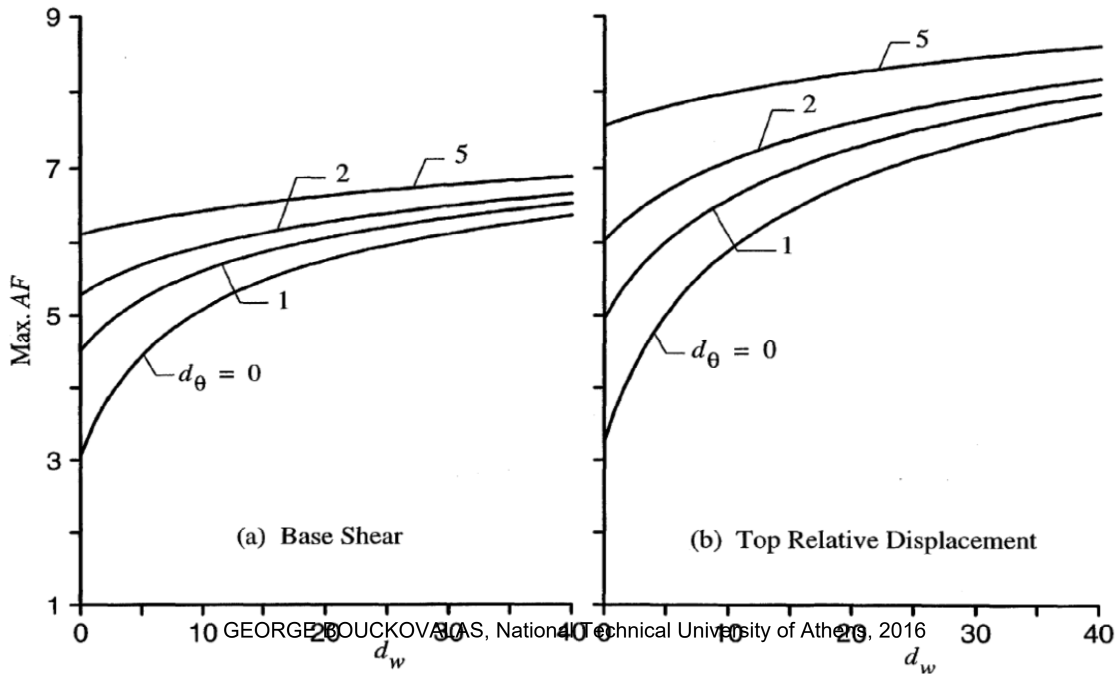
the effect of harmonic excitation frequency on **base shear** *(AF coefficient)*



$$\omega_1 = \frac{2\pi}{T_{soil}} = \frac{\pi V_{soil}}{2H}$$

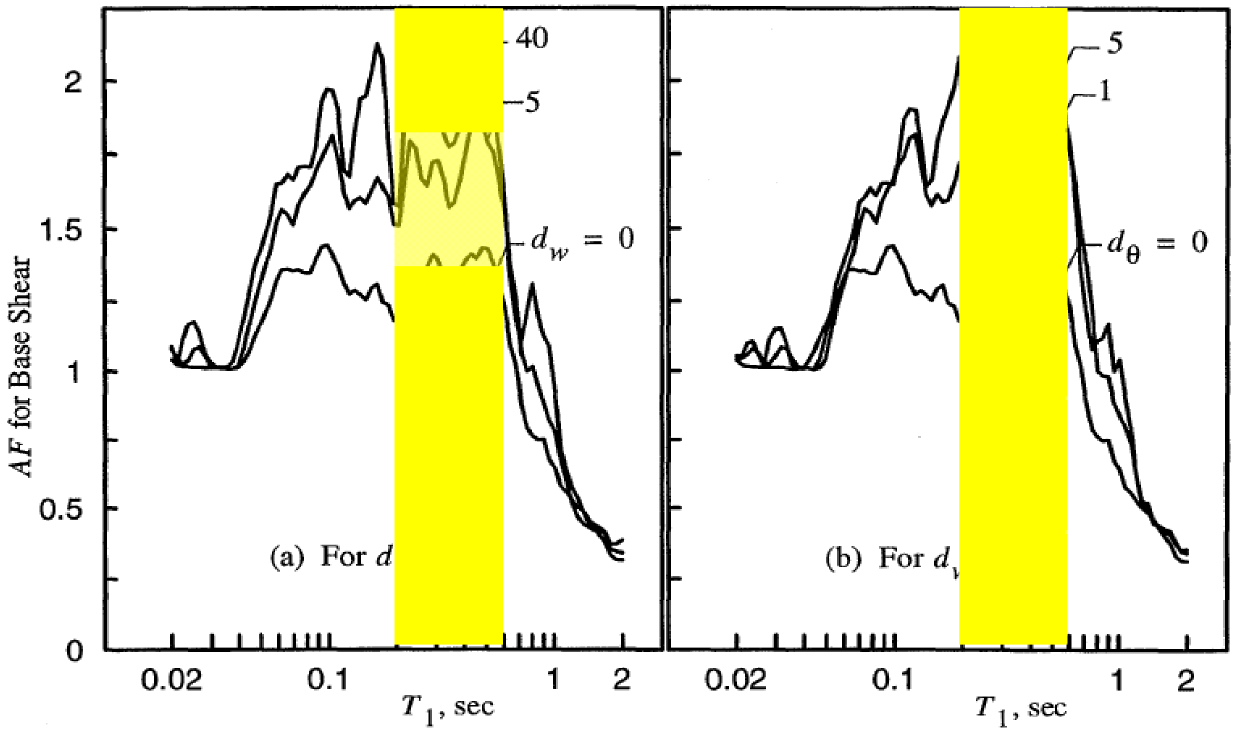
Analytical solutions for …….

resonance… (Max AF)



Numerical solution - El Centro (1940) earthquake

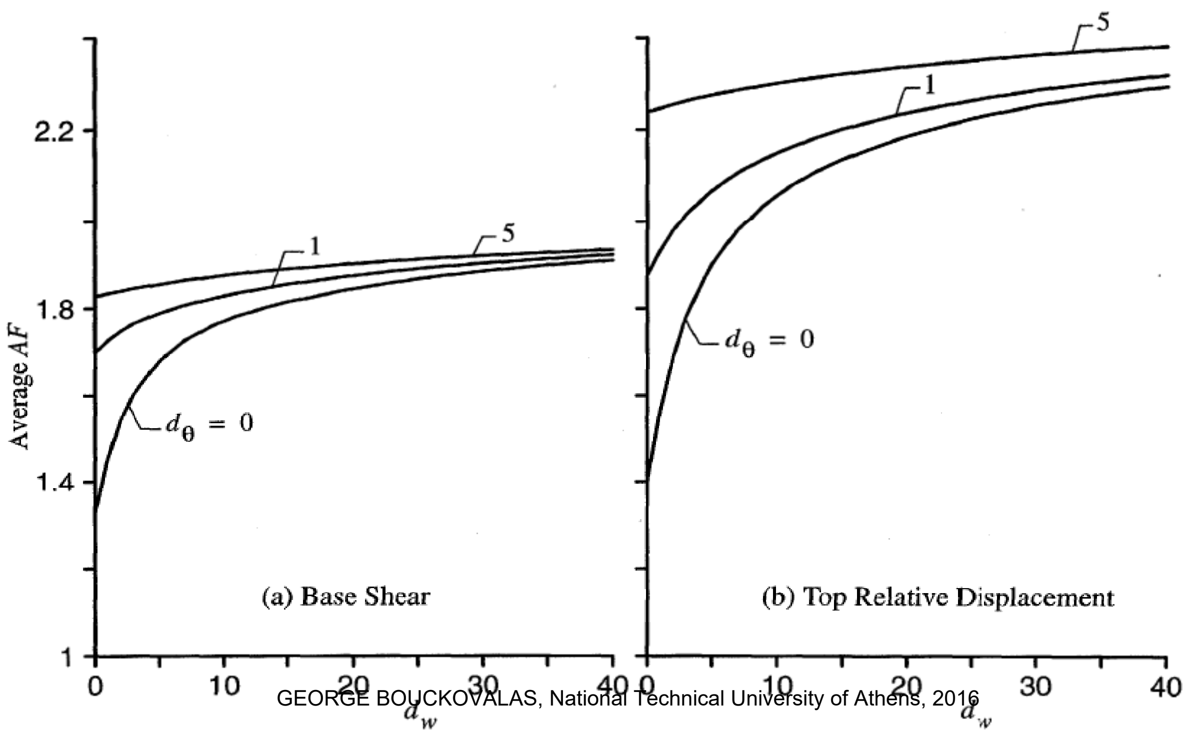
Variation of amplification factor AF for base shear versus the fundamental soil period)



(does this remind something to you?)

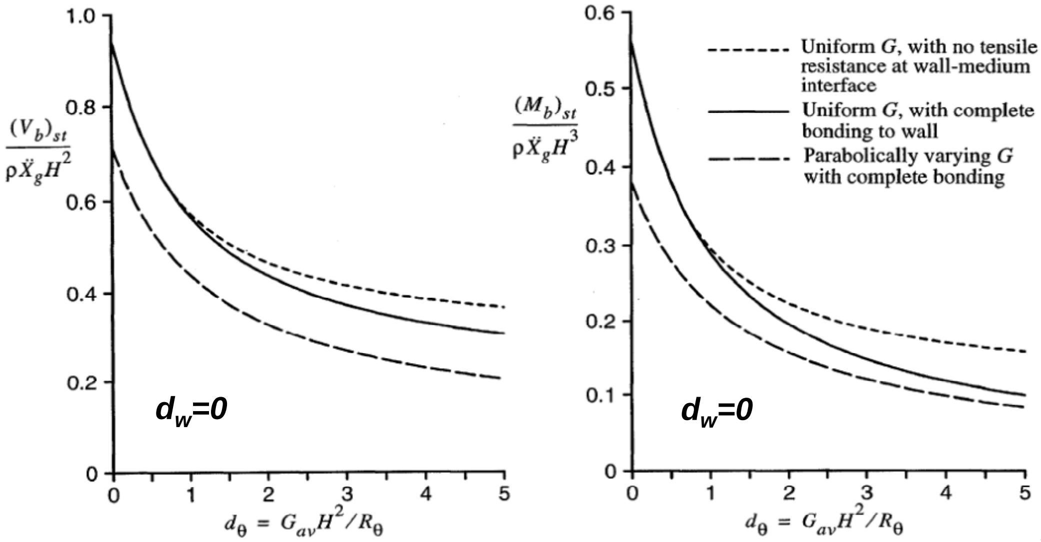
Numerical solution - El Centro (1940) earthquake

Average values of the amplification factor AF for base shear and relative displacement

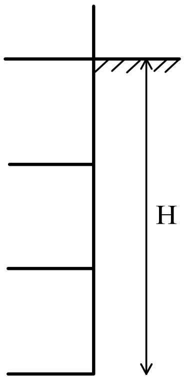


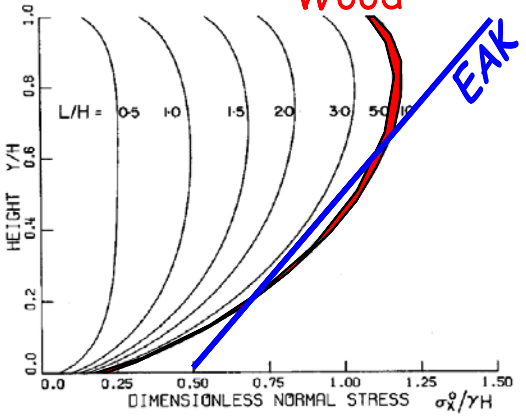
Limitations . . . .

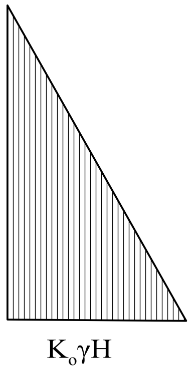
1. Tensile cracks, at the top of the wall, are not taken into account
   (→shear forces and bending moments are under-estimated)
2. Uniform soil is assumed
   (→shear forces and bending moments are over-estimated)



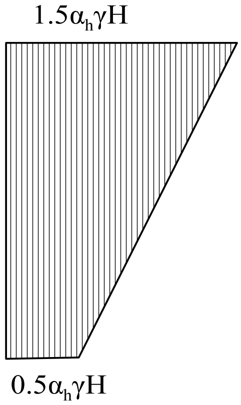
EAK 2002 – Rigid walls







Earth press. at rest



Dynamic earth press.

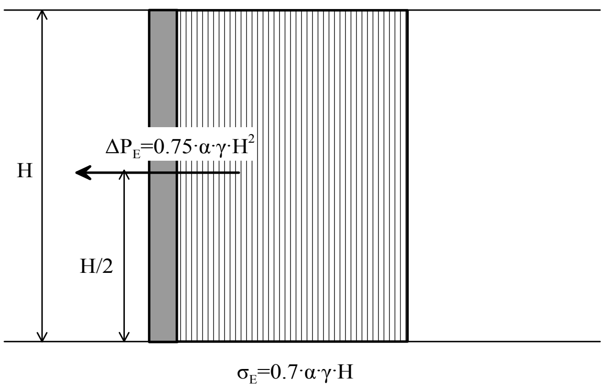
$$\Delta P_{eq} = \gamma H^3 \frac{\alpha_h}{g}$$

$$\Delta M_{eq} = 0.58\gamma H^3 \frac{\alpha_h}{g}$$

} Wood

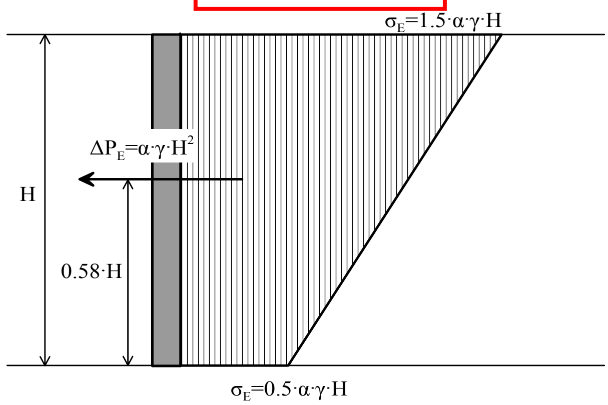
ΥΠΕΧΩΔΕ-εγκ.39/99 «Guidelines for the design of bridges»

Walls with limited displacement
$0.1\% > U/H \geq 0.05\%$



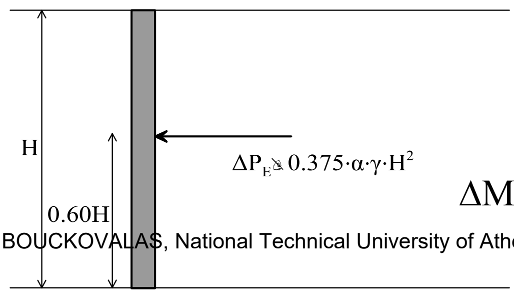
$$\Delta M_{eq} = 0.375\gamma H^3 \frac{\alpha_h}{g}$$

Rigid walls
$0.05\% > U/H$



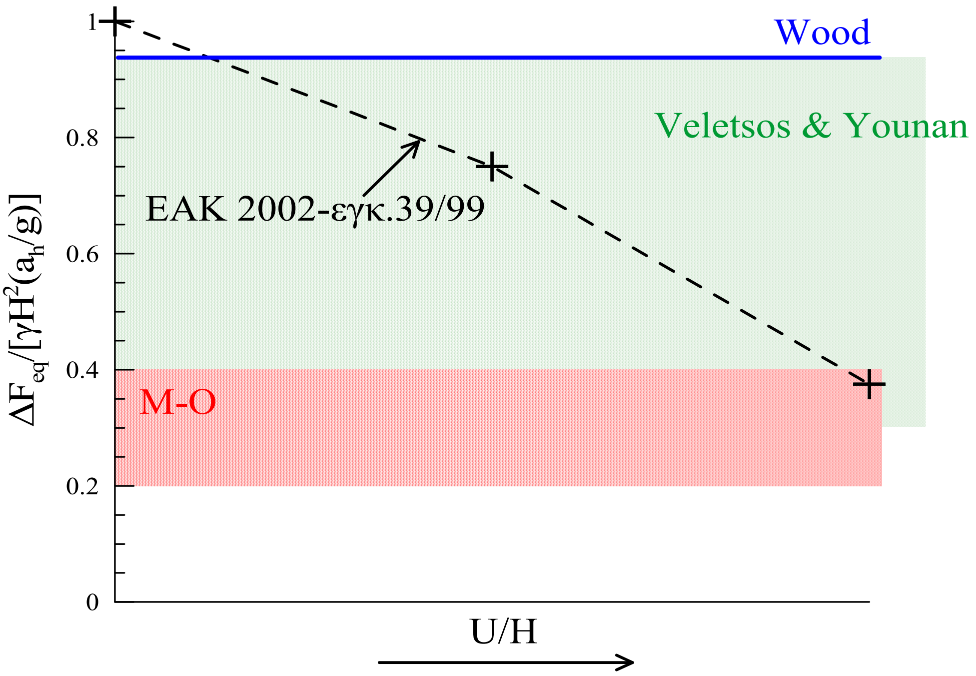
$$\Delta M_{eq} = 0.58\gamma H^3 \frac{\alpha_h}{g}$$

reminder: M-O (U/H>0.1%)



$$\Delta M_{eq} = 0.225\gamma H^3 \frac{\alpha_h}{g}$$

COMPARISON OF DIFFERENT METHODS ………..…



DIFFERENCES CAN BECOME SIGNIFICANT !

HWK 9.6:

Compute the total base shear force and overturning moment which develops at the base of a 5m high retaining wall during seismic excitation with a_{max}=0.15g. The wall is vertical and smooth, while the fill consists of sandy gravel with c=0, φ=36°, γ_{Ξ}=17kN/m³ and V_S=100m/s. The computations will be performed:

(α) for rigid wall,

(β) for a wall with limited deformation (d_w=10, d_θ=1), using the V&Y methodology,

(γ) for a wall with limited deformation (d_w=10, d_θ=1), using the seismic code provisions,

Note: *assume pseudo static conditions and neglect the wall mass.*

HWK 9.7

Repeat HWK 9.6 for the extreme case of resonance between soil and excitation.

9. Seismic Design of

RETAINING STRUCTURES

Part C:

DISPLACEMENT COMPUTATION
(& PERFORMANCE BASED DESIGN)

G. BOUCKOVALAS

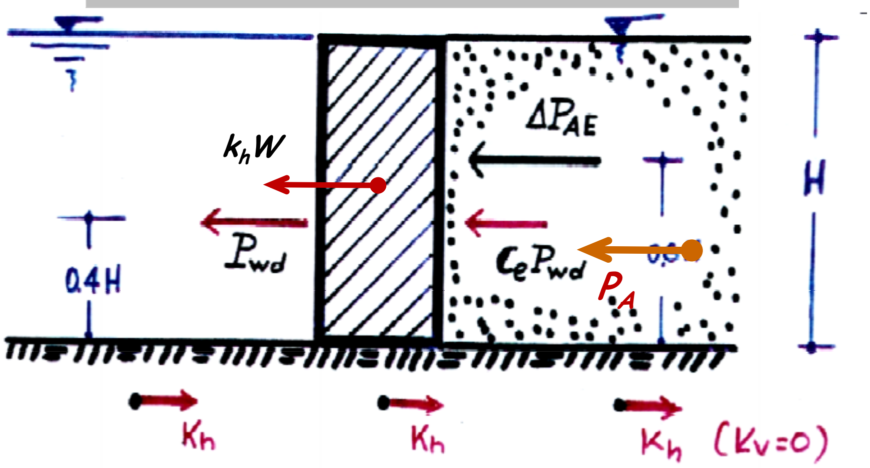
Professor of NTUA

October 2016

Problem Outline ….

During a pseudo-static analysis, it is very common to obtain FS>1.0.

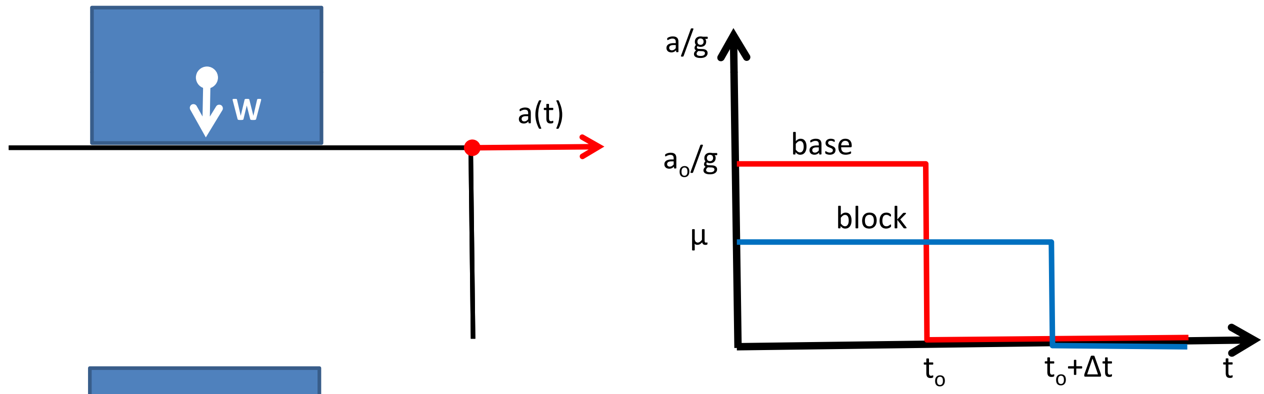
$$FS_{sl} = \frac{\mu \cdot W_b}{k_h W + (1+C_e)P_{wd} + P_A + P_{AE}} << 1.0$$

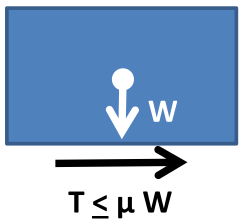


However, this does not necessarily mean "failure" of the wall, but permanent outward displacements (and rotations). In such cases, the performance of the wall is evaluated using the famous "Newmark Sliding Block" analysis (follows).

NEWMARK (1965): Rankine Lecture on Seismic Slope Displacements

SLIDING BLOCK subjected to pulse acceleration

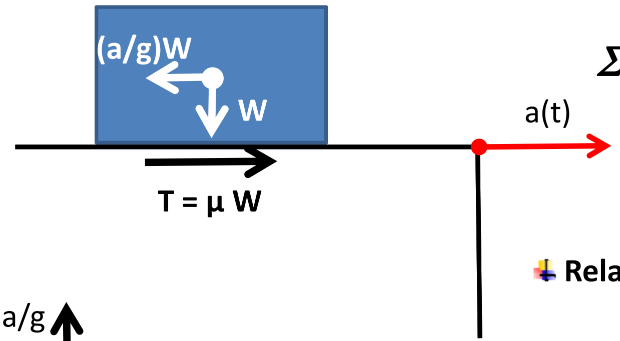




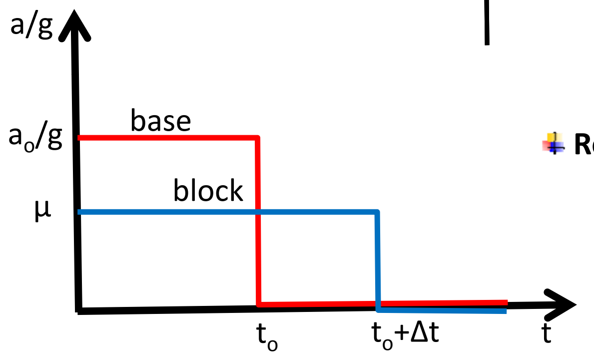
$$\Sigma F \le \mu W$$

$$a_{block} = \frac{\Sigma F}{W/g} \le \mu g$$

A. RELATIVE DISPLACEMENT OF BLOCK for $t \le t_o$



$$\Sigma F = \frac{a_o}{g} W - \mu W = \left(\frac{a_o}{g} - \mu \right) W$$



- **Relative SB acceleration:**

$$a_{SB} = \frac{\Sigma F}{W/g} = a_o - \mu g$$

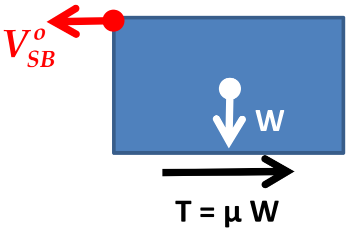
- **Relative SB velocity at $t < t_o$:**

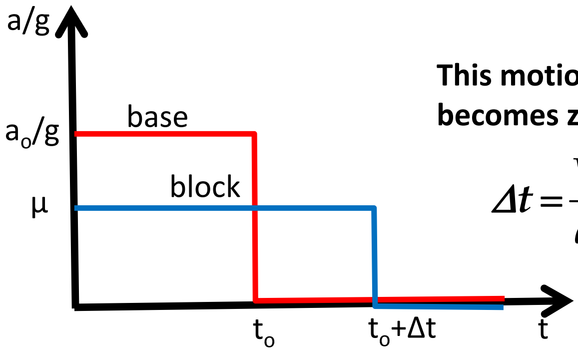
$$V_{SB}^o = a_{SB} t_o = (a_o - \mu g) t_o$$

- **Relative SB displacement at $t < t_o$:**

$$S_{SB}^o = \frac{1}{2} a_{SB} t_o^{\ 2} = \frac{1}{2} (a_o - \mu g) t_o^{\ 2}$$

B. RELATIVE DISPLACEMENT OF BLOCK for $t \geq t_o$





Steadily de-celerating motion with initial velocity:

$$V_{SB}^{o} = a_{SB} t_o = (a_o - \mu g) t_o$$

and de-celeration:

$$a'_{SB} = \frac{\Sigma F}{W/g} = \mu g$$

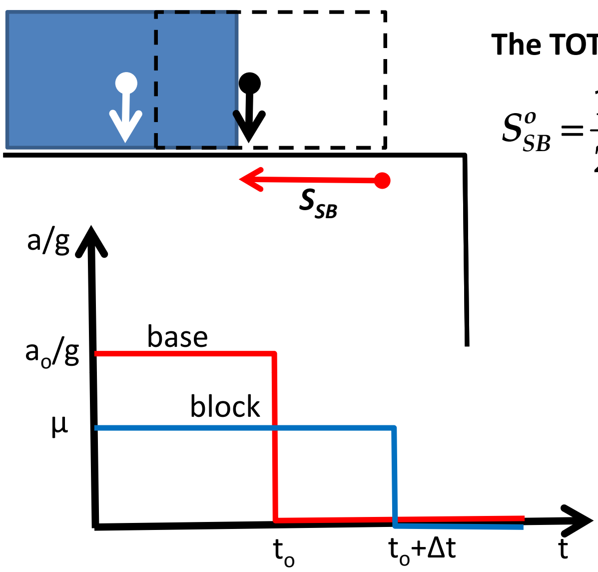
This motion will last until the SB relative velocity becomes zero, i.e. ……

$$\Delta t = \frac{V_{SB}^{o}}{a'_{SB}} = \frac{(a_o - \mu g) t_o}{\mu g} = \left(\frac{a_o}{\mu g} - 1\right) t_o$$

During Δt we will have additional relative displacement:

$$\Delta S_{SB} = V_{SB}^{o} \Delta t - \frac{1}{2} a'_{SB} \Delta t^2 = \ldots\ldots = \frac{1}{2} \frac{(a_o - \mu g)^2}{\mu g} t_o^{\,2}$$

C. At the end of RELATIVE block-base sliding, i.e. at $t \geq t_o + \Delta t$



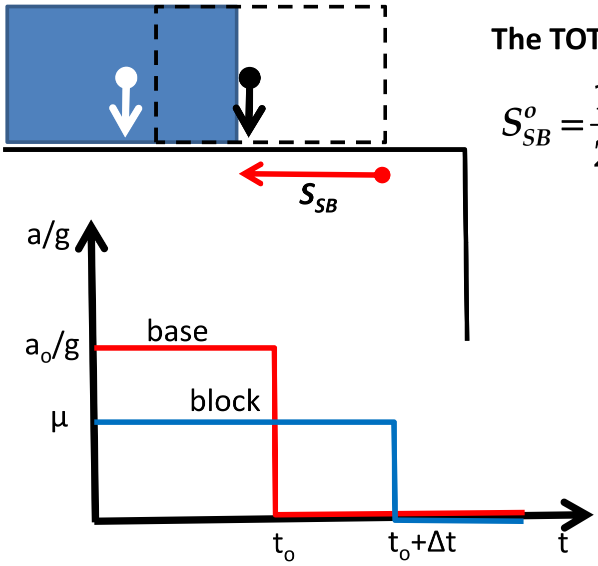
The TOTAL relative displacement will be:

$$S_{SB}^{o} = \frac{1}{2} (a_o - \mu g) t_o^{\,2} \; +$$

$$\Delta S_{SB} = \frac{1}{2} \frac{(a_o - \mu g)^2}{\mu g} t_o^{\,2} \; = \ldots\ldots$$

$$S_{SB} = \frac{1}{2} (a_o - \mu g) \frac{a_o}{\mu g} t_o^{\,2}$$

C. At the end of RELATIVE block-base sliding, i.e. at $t \geq t_o + \Delta t$



The TOTAL relative displacement will be:

$$S^o_{SB} = \frac{1}{2}(a_o - \mu g)t_o^2 \; +$$

$$\Delta S_{SB} = \frac{1}{2}\frac{(a_o - \mu g)^2}{\mu g} t_o^2 = \ldots\ldots$$

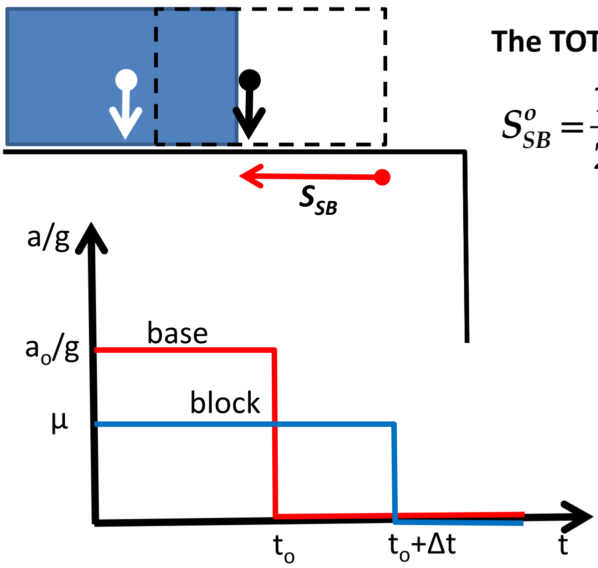
$$S_{SB} = \frac{1}{2}(a_o - \mu g)\frac{a_o}{\mu g} t_o^2$$

Assuming further that:

$a_o = a_{max}$ (peak seismic acceleration)
$\mu g = a_{CR}$ (critical seismic acceleration required to triger sliding, i.e. FS_{slide}=1.0)
V_{max} (peak seismic velocity), and
$t_o = V_{max}/a_{max}$

$$S_{SB} = \frac{1}{2}\frac{V^2_{max}}{a_{max}}\left(1 - \frac{a_{CR}}{a_{max}}\right)\left(\frac{a_{CR}}{a_{max}}\right)^{-1}$$

C. At the end of RELATIVE block-base sliding, i.e. at $t \geq t_o + \Delta t$



The TOTAL relative displacement will be:

$$S^o_{SB} = \frac{1}{2}(a_o - \mu g)t_o^2 \; +$$

$$\Delta S_{SB} = \frac{1}{2}\frac{(a_o - \mu g)^2}{\mu g} t_o^2 = \ldots\ldots$$

$$S_{SB} = \frac{1}{2}(a_o - \mu g)\frac{a_o}{\mu g} t_o^2$$

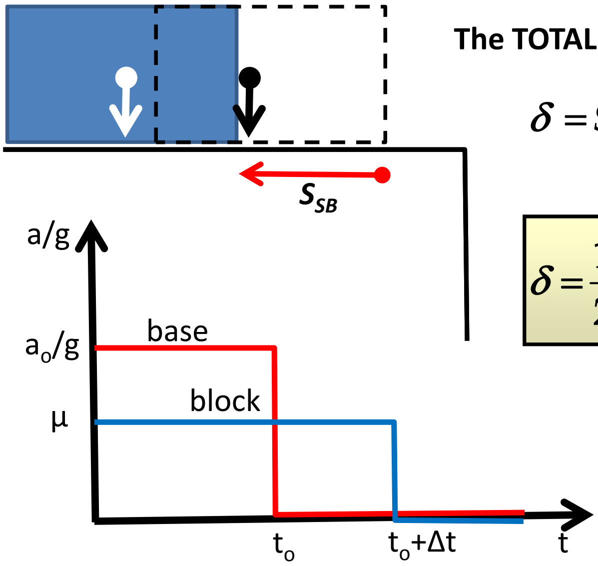
Assuming further that:

$a_o = a_{max}$ (peak seismic acceleration)
$\mu g = a_{CR}$ (critical seismic acceleration required to triger sliding, i.e. FS_{slide}=1.0)
V_{max} (peak seismic velocity), and
$t_o = V_{max}/a_{max}$

$$S_{SB} = \frac{1}{2}\frac{V^2_{max}}{a_{max}}(1 - a^*_{CR})(a^*_{CR})^{-1}$$

$$\text{with} \quad a^*_{CR} = a_{CR} / a_{max}$$

D. For N similar pulses of base motion ………………



The TOTAL relative displacement will be:

$$\delta = S_{SB} * N \quad with \quad N \approx \frac{a_{max}}{a_{CR}} = \left(a^*_{CR}\right)^{-1}$$

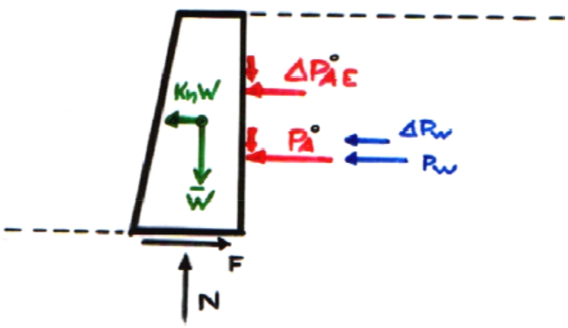
$$\delta = \frac{1}{2}\frac{V^2_{max}}{a_{max}}(1 - a^*_{CR})\left(a^*_{CR}\right)^{-2}$$ **NEWMARK I**

or, for very small a^*_{CR} (<0.30) ……

$$\delta \approx \frac{1}{2}\frac{V^2_{max}}{a_{max}}\left(a^*_{CR}\right)^{-2}$$ **NEWMARK II**

$$with \quad a^*_{CR} = a_{CR} / a_{max}$$

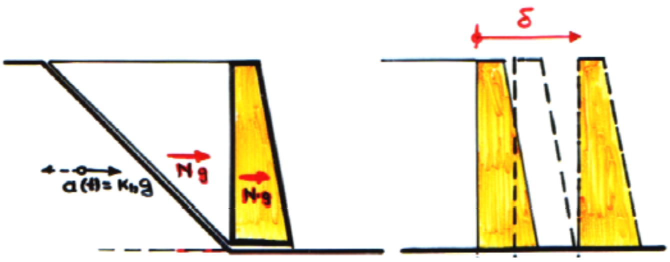
RICHARDS & ELMS (1979):
Gravity walls under real seismic excitation



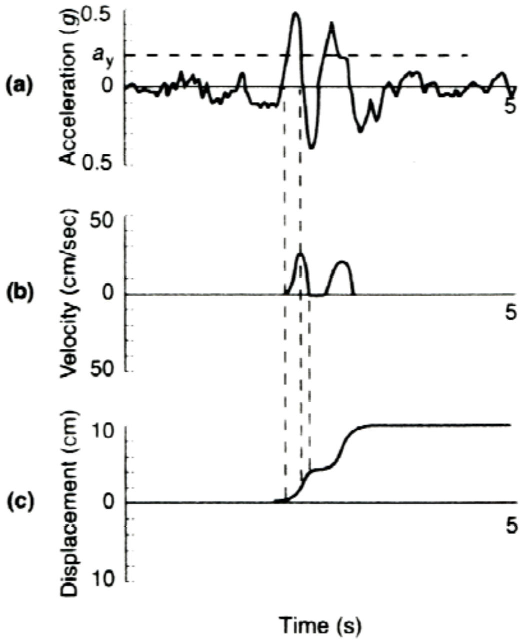
δ: friction angle between wall side and fill
φ_o: friction angle between wall base and ground

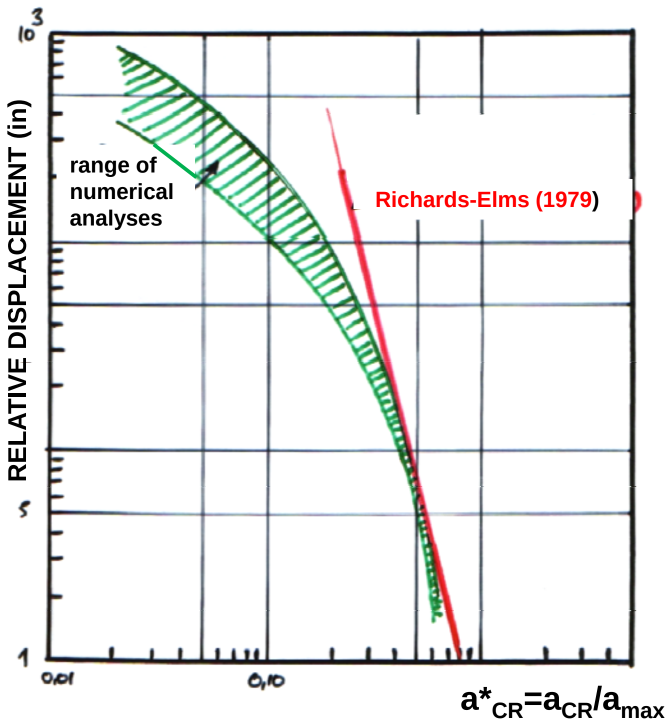
$$N = \bar{W} + \left(\Delta P^o_{AE} + P^o_A\right)\tan\delta$$
$$F = N\tan\varphi_o$$
$$F.S.^{o\lambda} = \frac{N\tan\varphi_o}{P^o_A + \Delta P^o_{AE} + P_w + \Delta P_w + k_h W}$$

Even though $F.S.^{o\lambda}$< 1.0 (sliding failure)
<u>there is no collapse</u> of the wall (!!),
but development of limited displacements, which may be tolerable ……..



a_{cr}=Ng :
critical seismic acceleration
leading to F.S.^{ολ}=1.00





Richards & Elms (1997)

$$\delta \approx 0.087 \frac{V_{max}^2}{a_{max}} \left(a^*_{CR}\right)^{-4}$$

Computation of Relative Sliding ….

NEWMARK (1965)

$$\delta = 0.50 \cdot \left(\frac{V_{max}^2}{a_{max}}\right) \cdot \frac{\left(1 - a^*_{CR}\right)}{a^{*\,2}_{CR}}$$

.
.
.

$$\delta \approx 0.50 \cdot \left(\frac{V_{max}^2}{a_{max}}\right) \cdot \frac{1}{a^{*\,2}_{CR}}$$

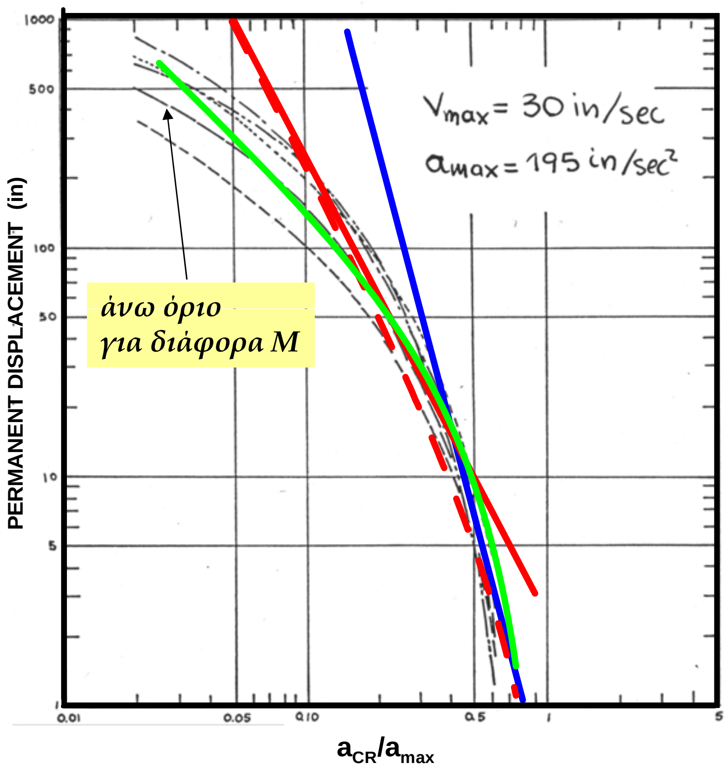
RICHARDS & ELMS (1979)

$$\delta \approx 0.087 \cdot \left(\frac{V_{max}^2}{a_{max}}\right) \cdot \frac{1}{a^{*\,4}_{CR}}$$

NTUA (1990)

$$\delta \approx 0.080 \cdot t^{1.15} \cdot \left(\frac{V_{max}^2}{a_{max}}\right) \cdot \left[1 - a^{*\,(1-a^*_{CR})}_{CR}\right] \cdot \frac{1}{a^*_{CR}}$$

Comparison with numerical predictions for actual earthquakes by Franklin & Chang (1977) . . . .



Newmark - I (1965)
Newmark - II (1965)
Richards & Elms (1979)
NTUA (1990)

Relative Sliding

$$\delta_d = min\begin{Bmatrix} 0.087 \cdot \dfrac{V_{max}^2}{a_{max}} \cdot \left(a^*_{CR}\right)^{-4} \\ 0.50 \cdot \dfrac{V_{max}^2}{a_{max}} \cdot \left(a^*_{CR}\right)^{-2} \end{Bmatrix}$$

………… or NTUA (1990)

for EXAMPLE . . . . . .

- PEAK SEISMIC ACCELERATION a_{max} = 0.50g
- PEAK SEISMIC VELOCITY V_{max} = 1.00 m/s ($T_e \approx 0.80$ sec)
- "CRITICAL" or "YIELD" ACCELERATION a_{CR} = 0.33g (=2/3 a_{max})

Relative Sliding

$$\delta_d = min\left\{ \begin{array}{l} 0.087 \cdot \dfrac{V_{max}^2}{a_{max}} \cdot \left(a_{CR}^*\right)^{-4} \\ 0.50 \cdot \dfrac{V_{max}^2}{a_{max}} \cdot \left(a_{CR}^*\right)^{-2} \end{array} \right\}$$

→ **9 cm !**

for EXAMPLE . . . . . .

- PEAK SEISMIC ACCELERATION a_{max} = 0.50g
- PEAK SEISMIC VELOCITY V_{max} = 0.50 m/s ($T_e \approx 0.40$ sec)
- "CRITICAL" or "YIELD" ACCELERATION a_{CR} = 0.33g (=2/3 a_{max})

Relative Sliding

$$\delta_d = min\left\{ \begin{array}{l} 0.087 \cdot \dfrac{V_{max}^2}{a_{max}} \cdot \left(a_{CR}^*\right)^{-4} \\ 0.50 \cdot \dfrac{V_{max}^2}{a_{max}} \cdot \left(a_{CR}^*\right)^{-2} \end{array} \right\}$$

→ **2 cm !**

THUS, if we can tolerate some small outwards displacements, the pseudo static analysis is NOT performed for the peak seismic acceleration a_{max}, but for the critical acceleration a_{CR} (& FS_{slide}=1.0)

In other words. . . . Displacement Based Design (PBD)

DISPLACEMENT BASED DESIGN

New design philosophy:

$$\delta = 0.087 \frac{V_{max}^2}{\alpha_{max}} \left(\frac{a_{CR}}{a_{max}} \right)^{-4}$$

$$k_h^* = \frac{a_{CR}}{g} = a_{max} \left[0.087 \frac{V_{max}^2}{a_{max}\delta} \right]^{1/4}$$

Instead of designing the wall for $k_h = a_{max}/g$, I choose a lower k_h^* (< k_h) which is a function of the allowable wall displacement δ. In that case, the required factor of safety is F.S.=1.00

alternatively:

$$k_h^* = \frac{k_h}{q_w} \qquad \text{with: } q_w = \frac{1}{\left[0.087 \frac{V_{max}^2}{\alpha_{max}\delta} \right]^{1/4}}$$

DISPLACEMENT BASED DESIGN

In more detail ….

$$\delta = \min \left\{ \begin{matrix} 0.087 \cdot \frac{V_{max}^2}{a_{max}} \cdot \left(a_{CR}^* \right)^{-4} \\ 0.50 \cdot \frac{V_{max}^2}{a_{max}} \cdot \left(a_{CR}^* \right)^{-2} \end{matrix} \right\}$$

$$k_h^* = \frac{k_h}{q_w} \qquad \text{with:}$$

$$q_w = \max \left\{ \begin{matrix} \left[0.087 \cdot \frac{V_{max}^2}{a_{max}} \cdot \frac{1}{\delta} \right]^{-1/4} \\ \left[0.50 \cdot \frac{V_{max}^2}{a_{max}} \cdot \frac{1}{\delta} \right]^{-1/2} \end{matrix} \right\}$$

In accordance with this design philosophy, **EAK** requests that:

$$k_h = \frac{\alpha \cdot \gamma_n}{q_w}$$

$$\alpha = \frac{\alpha_{max}}{g}$$

γ_n=importance coefficient

q_w =

q_w =	**2.00**	**δ(mm)=300a**
	1.50	**δ(mm)=200a**
	1.25	**δ(mm)=100a (τοίχοι από Ο.Σ.)**
	1.00	**anchored flexible walls**
	0.75	**basement walls, etc**

HWK 9.5:

For the gravity retaining wall of HWKs 9.1 and/or 9.2:

(a) Compute the critical horizontal acceleration a_{CR}, required to trigger sliding

(b) Compute FS_{slide} and the corresponding outward displacement of the wall for a_{max} = 1.50 to 4.0 a_{CR} and predominant excitation period T_{exc}=0.40s.