National Technical University of Athens
School of Civil Engineering
Geotechnical Division

Seismic Analysis of Slopes
Current Design Practice

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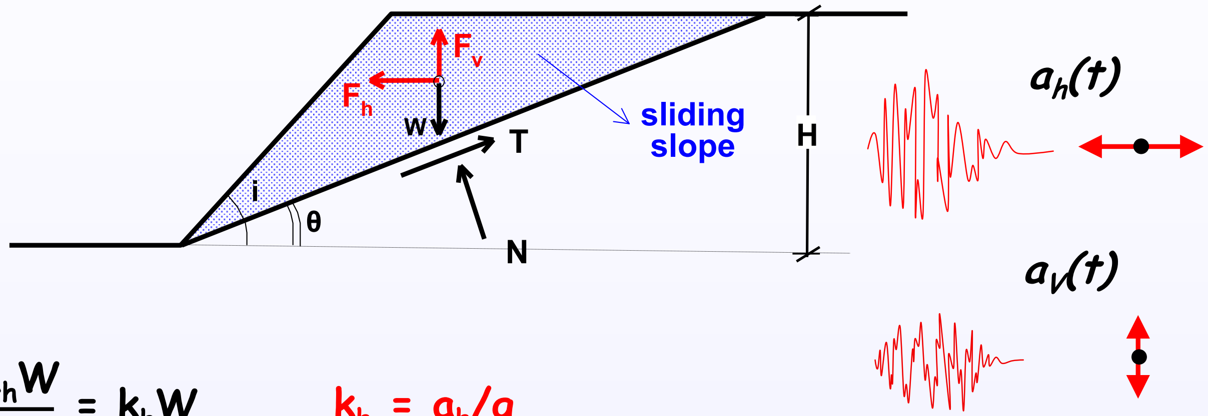
8.1 The "Pseudo Static" approach: *BASIC CONCEPTS*

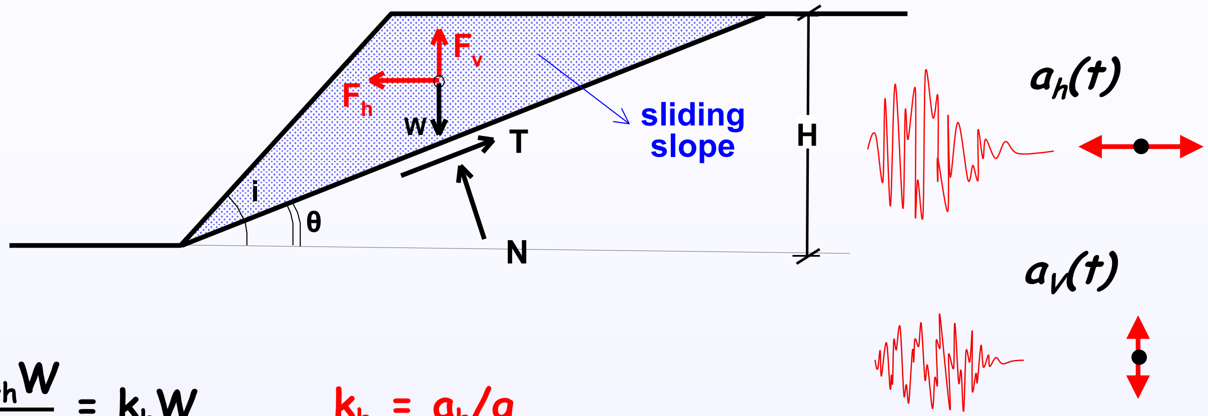


$$F_h = \frac{a_h W}{g} = k_h W \qquad k_h = a_h/g$$

$$F_v = \pm \frac{a_v W}{g} = k_v W \qquad k_v = a_v/g$$

$$FS_d = \frac{cL + [(W - F_v)\cos\theta - F_h \sin\theta]\tan\varphi}{(W - F_v)\sin\theta + F_h \cos\theta}$$

some times we tend to …. forget F_V



$$F_h = \frac{a_h W}{g} = k_h W \qquad k_h = a_h/g$$

$$F_v = \pm \frac{a_v W}{g} = k_v W \qquad k_v = a_v/g$$

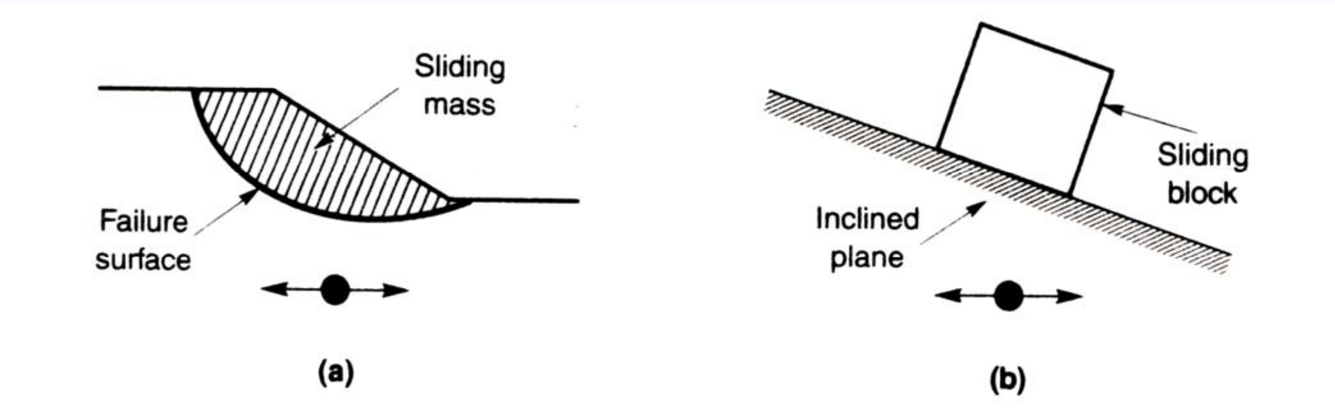
$FS_d > 1$	safe conditions	✓
$FS_d < 1$	slope failure (dynamic)	?

8.2 Dynamic Slope Failure *(FS_d<1.0)*:

. . . . and so what?

When FS_d becomes less than 1.0,

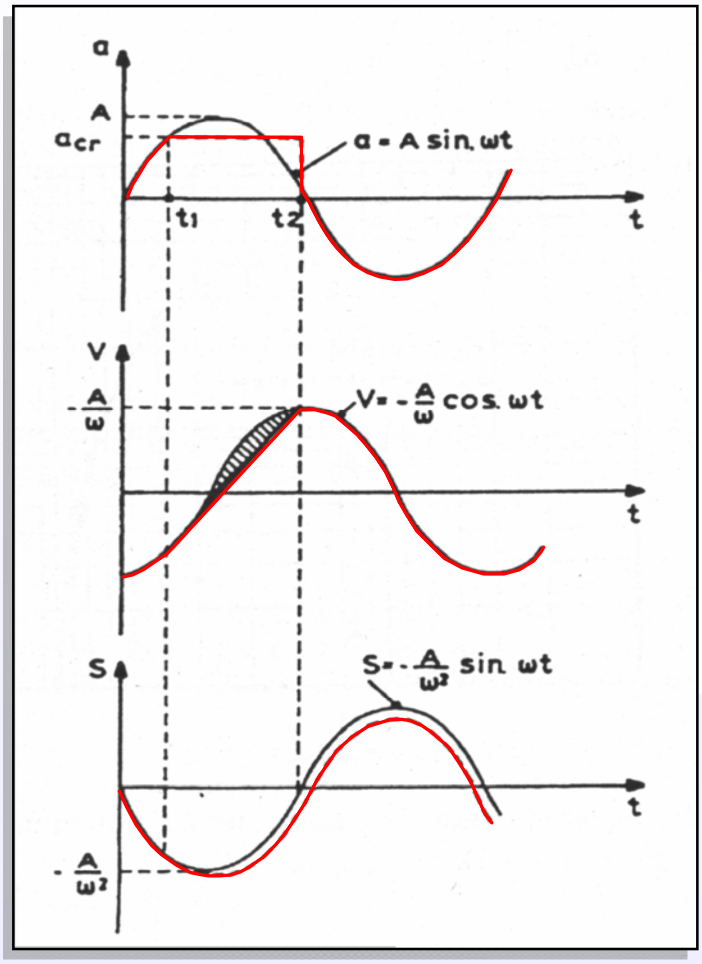
the soil mass above the failure surface will slide downslope as in the case of a "sliding block on an inclined plane"



HOWEVER,

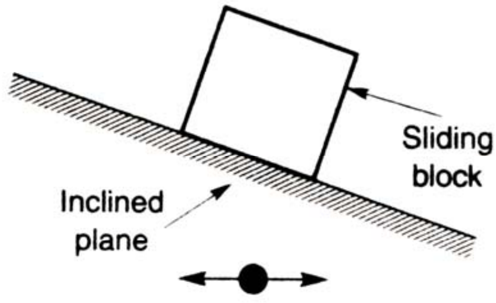
unlike STATIC FAILURE which lasts for ever,

SEISMIC FAILURE lasts only for a very short period (fraction of a second), as . . . . . .



"Sliding Block" kinematics

(for the simplified case of sinusoidal motion)



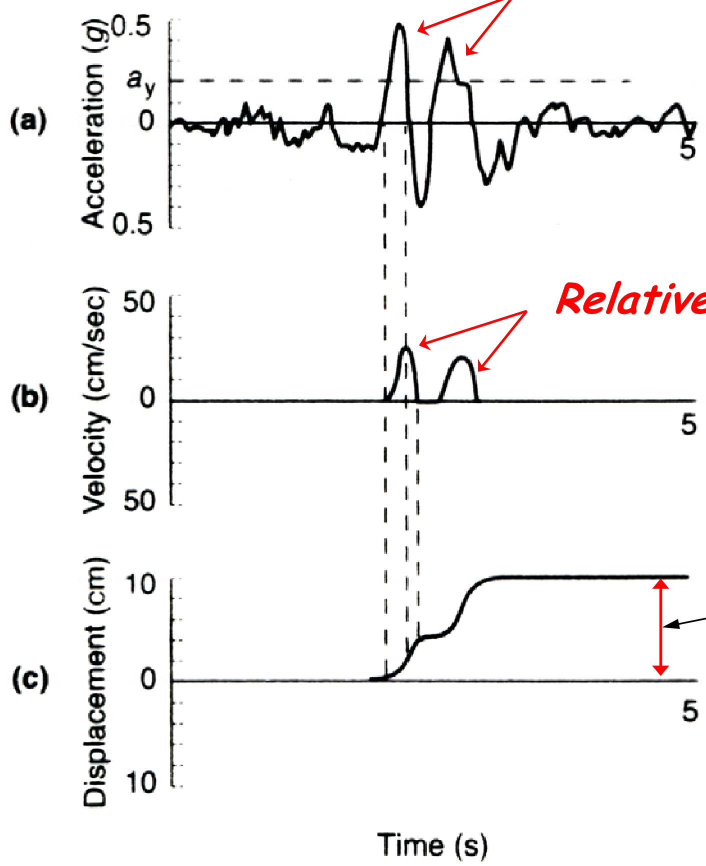
base motion

sliding block motion

Seismic failure & downslope sliding

Relative Velocity

Relative Sliding



Computation of Relative Sliding ….

NEWMARK (1965)

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

.
.
.

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

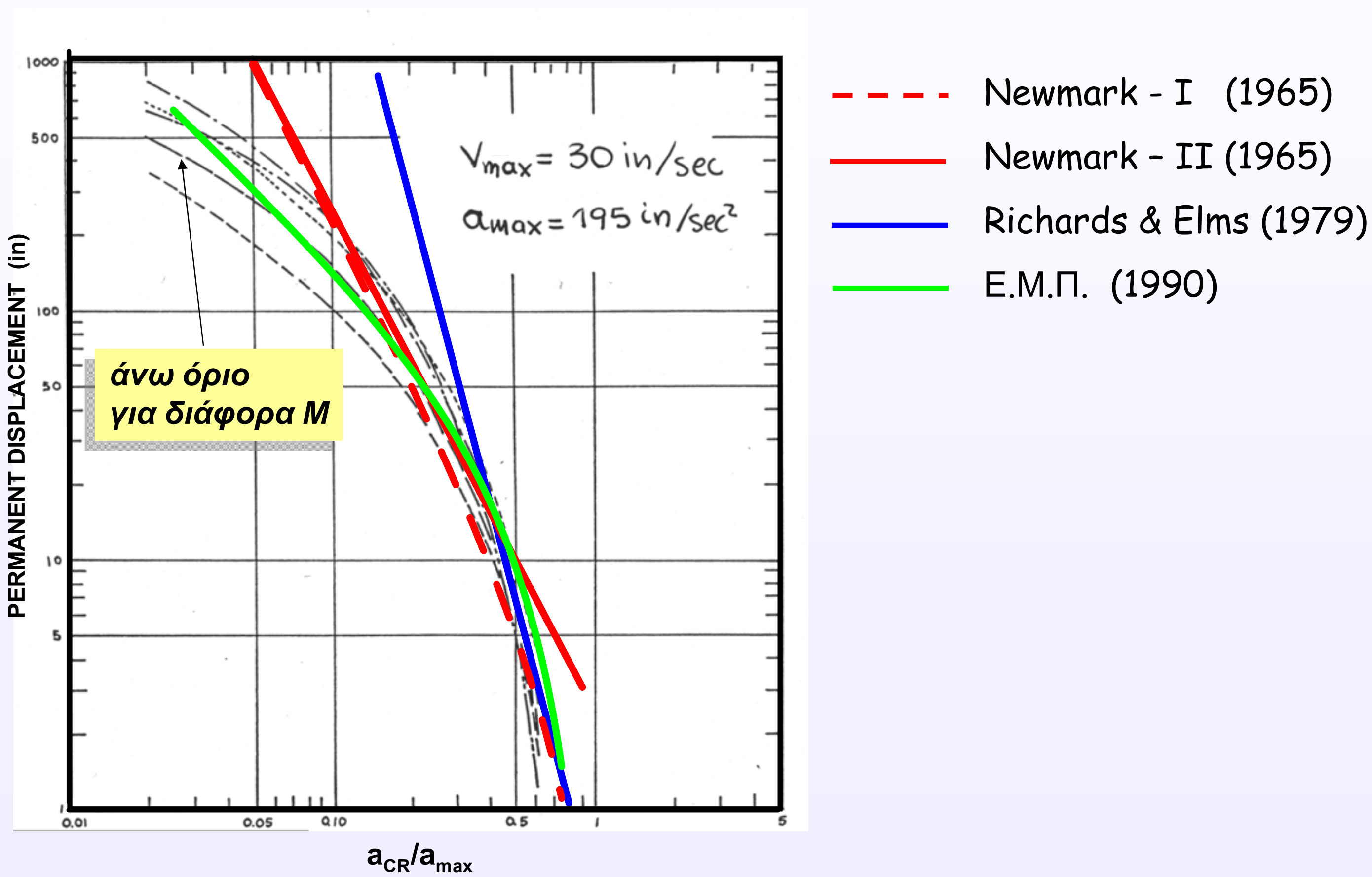
RICHARDS & ELMS (1979)

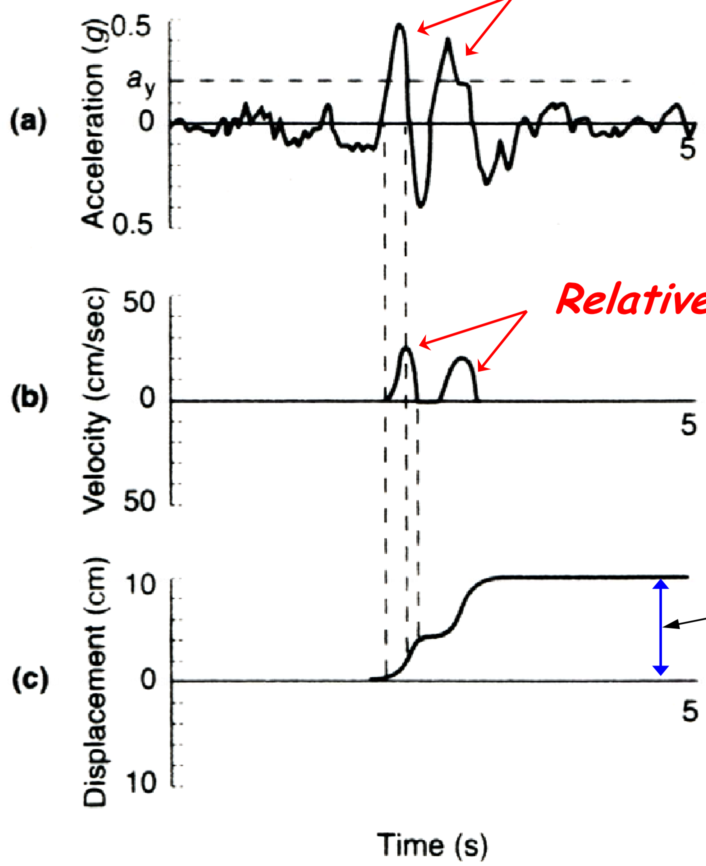
$$\delta \approx 0.087 \cdot \left(\frac{V_{max}^2}{a_{max}} \right) \cdot \frac{1}{\bar{a}_{CR}^{\,4}}$$

Ε.Μ.Π. (1990)

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

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





Relative Sliding

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

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

$$\delta_d = \min\left\{\begin{array}{l} 0.087 \cdot \dfrac{V_{max}^2}{a_{max}} \cdot \left(\dfrac{a_{max}}{a_{CR}}\right)^4 \\ 0.50 \cdot \dfrac{V_{max}^2}{a_{max}} \cdot \left(\dfrac{a_{max}}{a_{CR}}\right)^2 \end{array}\right\} \rightarrow 9 \text{ cm } !$$

THUS, if we can tolerate some small down-slope displacements, the pseudo static analysis is NOT performed for the peak seismic acceleration a_{max}, but for the . . . .

EFFECTIVE seismic acceleration a_E = (0.50 ÷ 0.80) a_{max}

8.3 The pseudo static SEISMIC COEFFICIENT k_{hE}

why is it much lower than the peak seismic acceleration a_{max} ?

- **FIRST. . . .**

Using the PEAK seismic acceleration (i.e. $k_h = a_{max}/g$) is TOO conservative.

Instead we use the EFFECTIVE seismic acceleration, i.e.

$$k_{h,E} = (0.50 \div 0.80)\, a_{max}/g$$

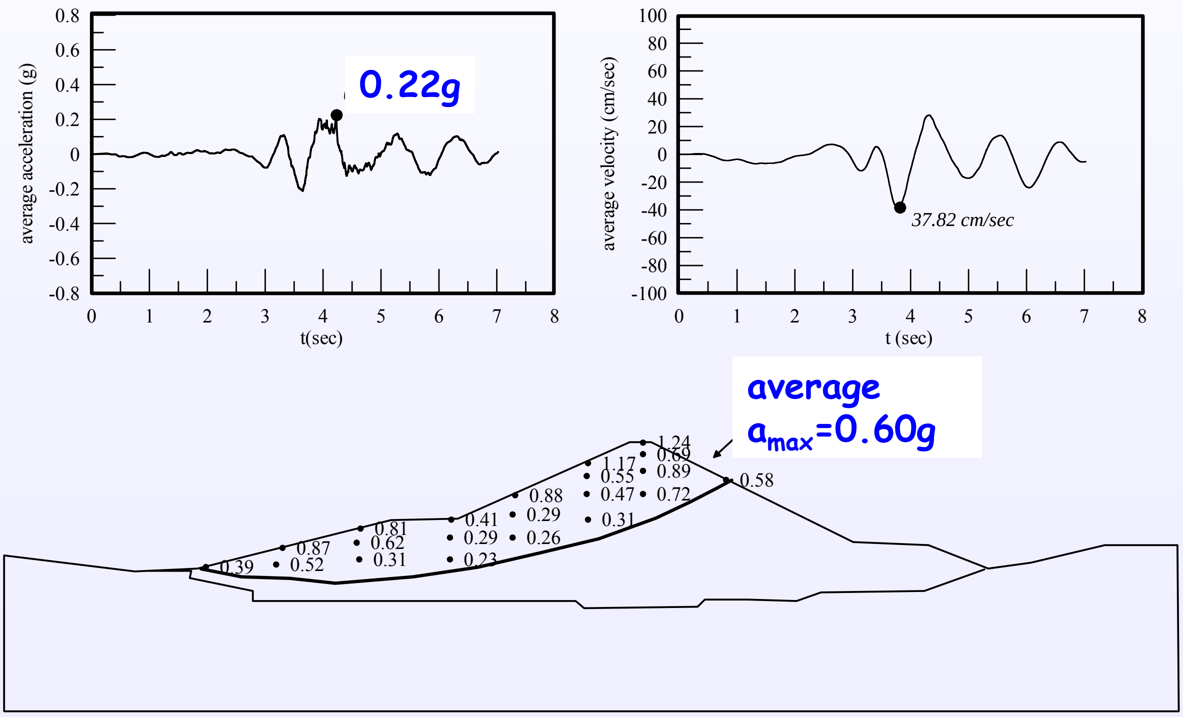
with $FS_d = 1.0 \div 1.10$

as this will usually lead to fairly small (< 10 cm) downslope displacements

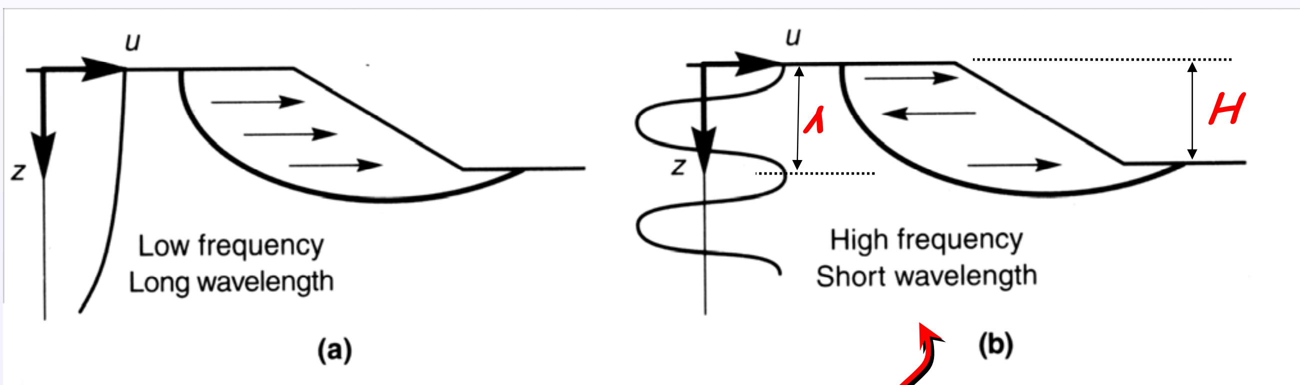
SECONDLY . . . .

observe these numerical results ……

average time histories



This is because tall earth dams (i.e. H > 30m) are flexible and consequently the seismic motion is NOT synchronous all over the the sliding mass:

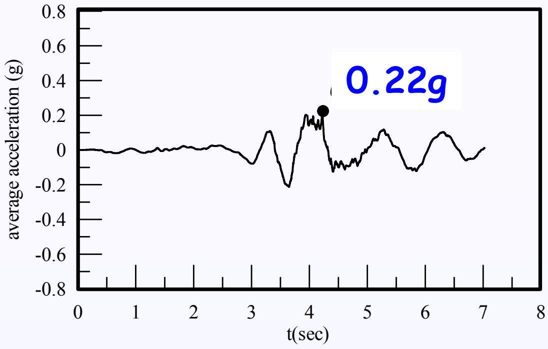


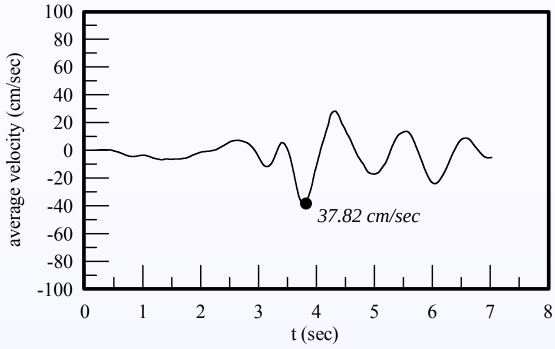
For common
earth dams H=(30 ÷ 120m) & earthquakes (T_e=0.30 ÷ 0.60s)

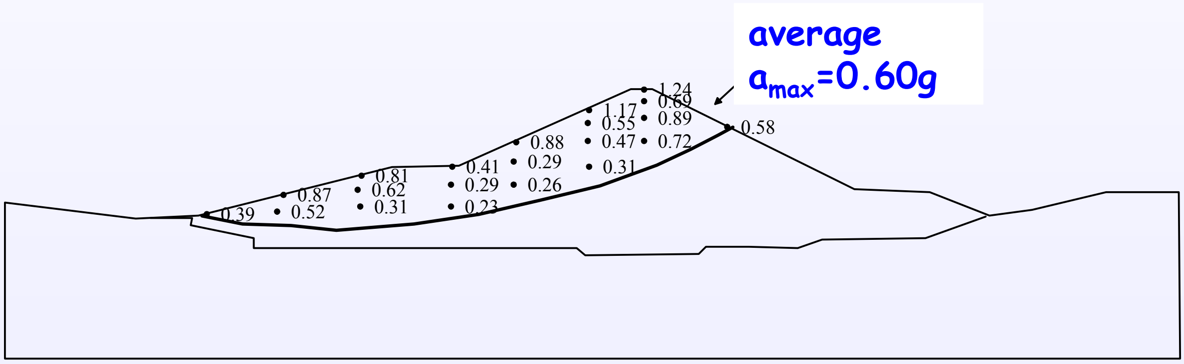
$\lambda \approx (1.00 \div 2.00)\, H$ i.e.

AS A RESULT OF THESE EFFECTS . . . .

average time histories







K_h = 0.22 *(from the average acceleration time history)*

k_{hE}=(0.50÷0.80) k_h = 0.11 ÷ 0.18

8.4 Review & Evaluation of

SEISMIC COEFFICIENTS k_{hE}

proposed in the literature

REVIEW of k_{hE} values, proposed .....

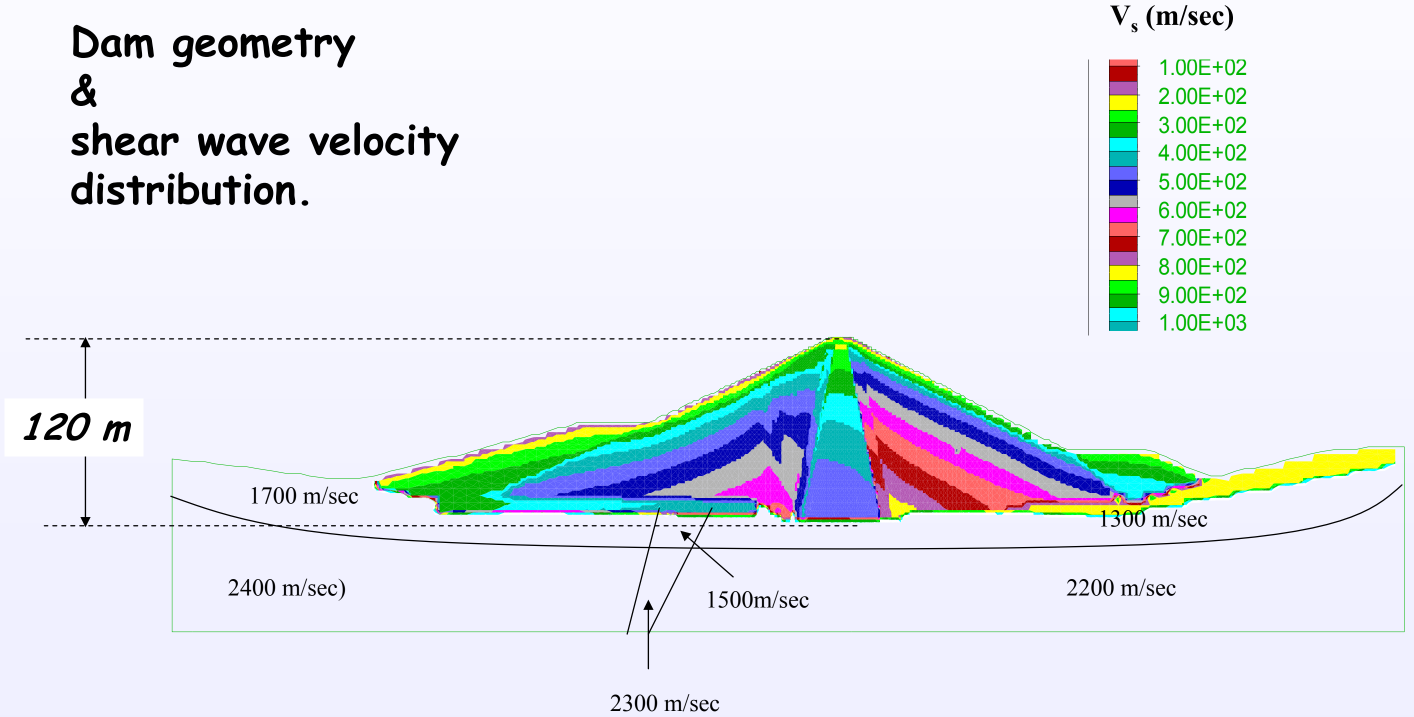
- on the basis of mere engineering . . . INTUITION
- in relation with the FREE FIELD peak ground acceleration (PGA)
- in relation with the peak seismic acceleration at the DAM CREST

EVALUATION in comparison with numerical analyses which take into account:

- foundation SOIL CONDITIONS
- dynamic DAM RESPONSE
- NON-LINEAR HYSTERETIC soil response to seismic excitation

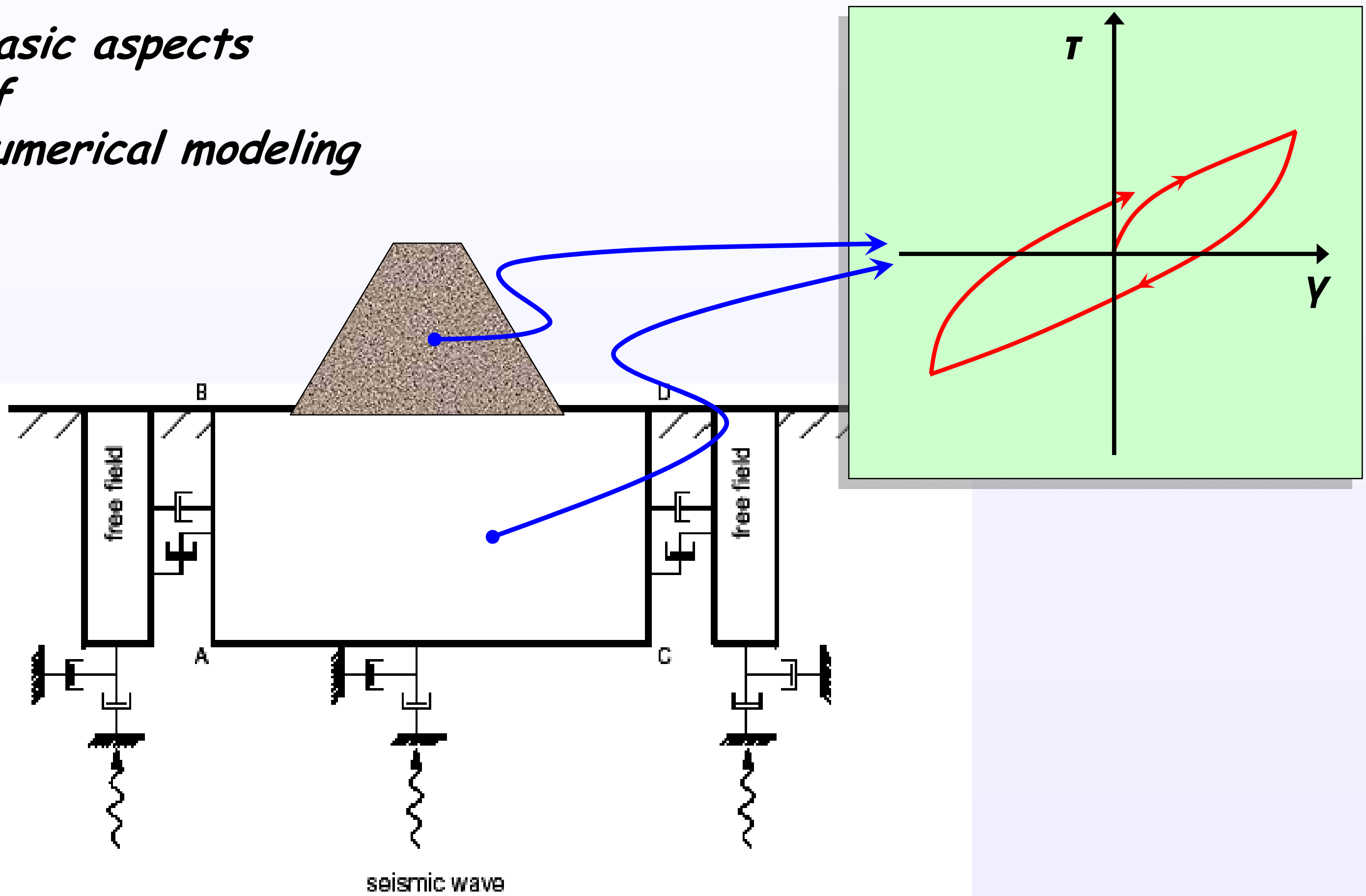
Numerical Evaluation of k_{hE} :
The case of Ilarion Dam in Northern Greece

Dam geometry
&
shear wave velocity
distribution.



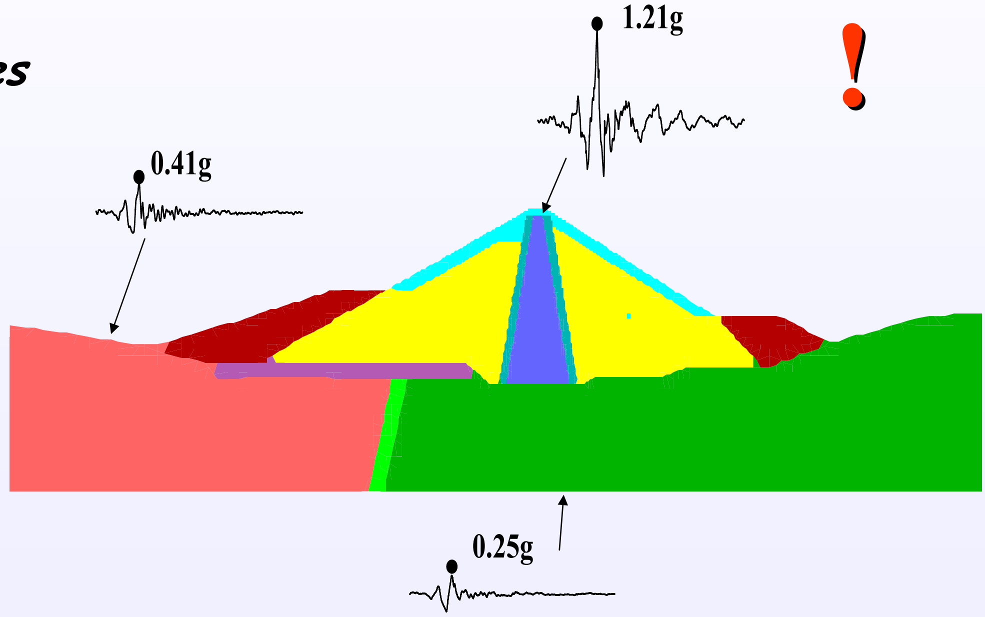
Numerical Evaluation of k_{hE} :
The case of Ilarion Dam in Northern Greece

Basic aspects
of
numerical modeling



Numerical Evaluation of k_{hE} : The case of Ilarion Dam in Northern Greece

Typical acceleration time histories

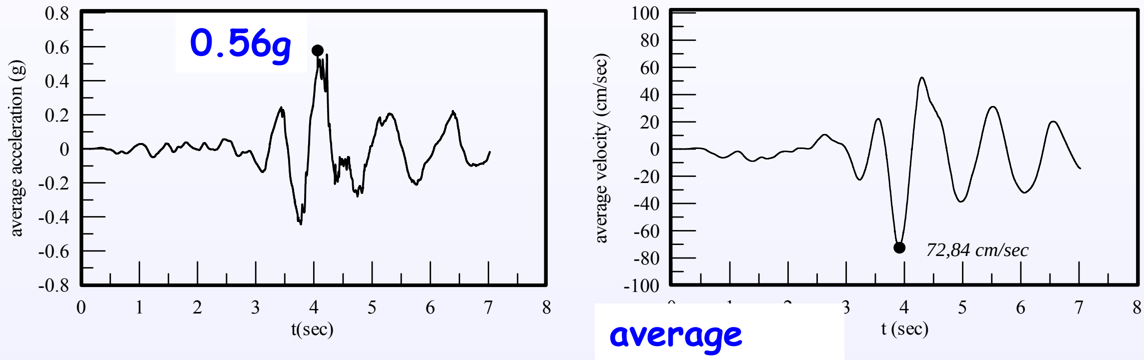


Numerical Evaluation of k_{hE} : the case of Ilarion Dam in Northern Greece

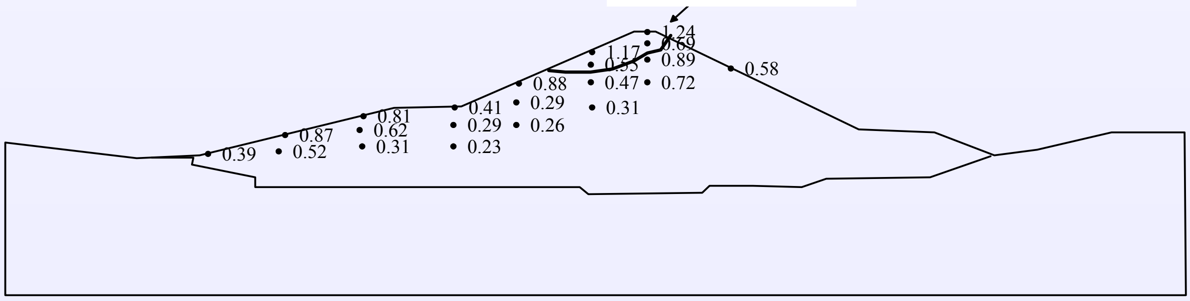
K_h = 0.56 *(from the average acceleration time history)*

k_{hE}=(0.50÷0.80) k_h = 0.28 ÷ 0.45

average time histories



average a_{max}=0.91g

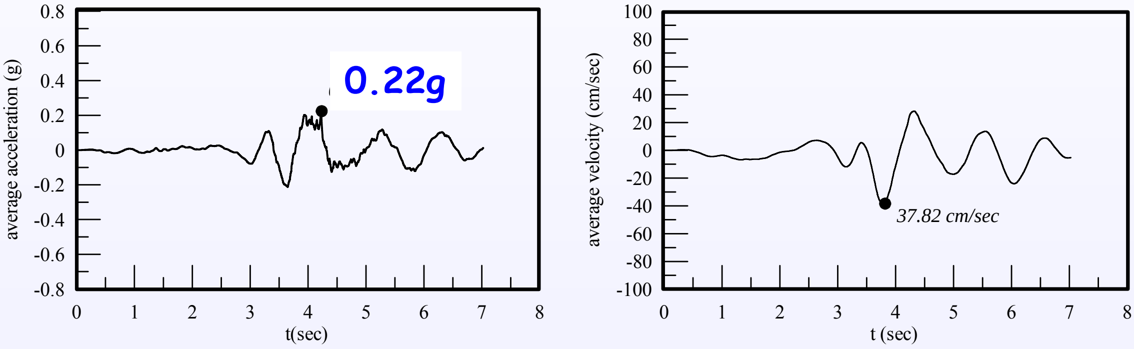


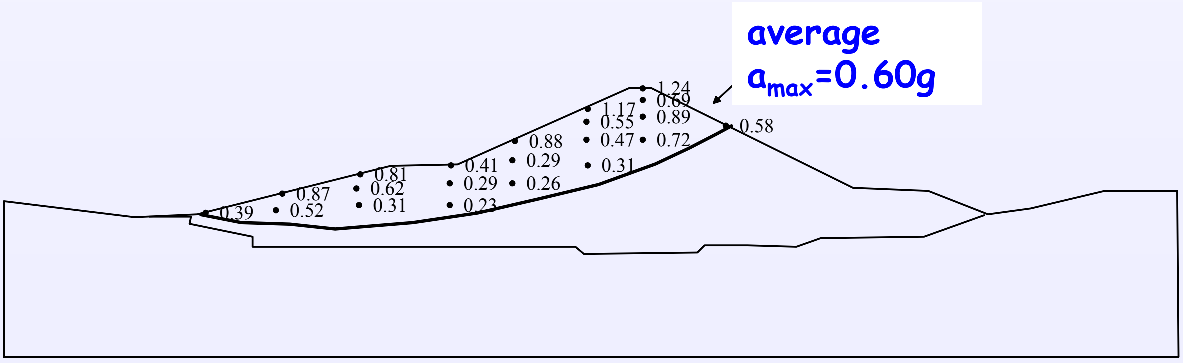
Numerical Evaluation of k_{hE} : The case of Ilarion Dam in Northern Greece

K_h = 0.22 *(from the average acceleration time history)*

k_{hE}=(0.50÷0.80) k_h = 0.11 ÷ 0.18

average time histories

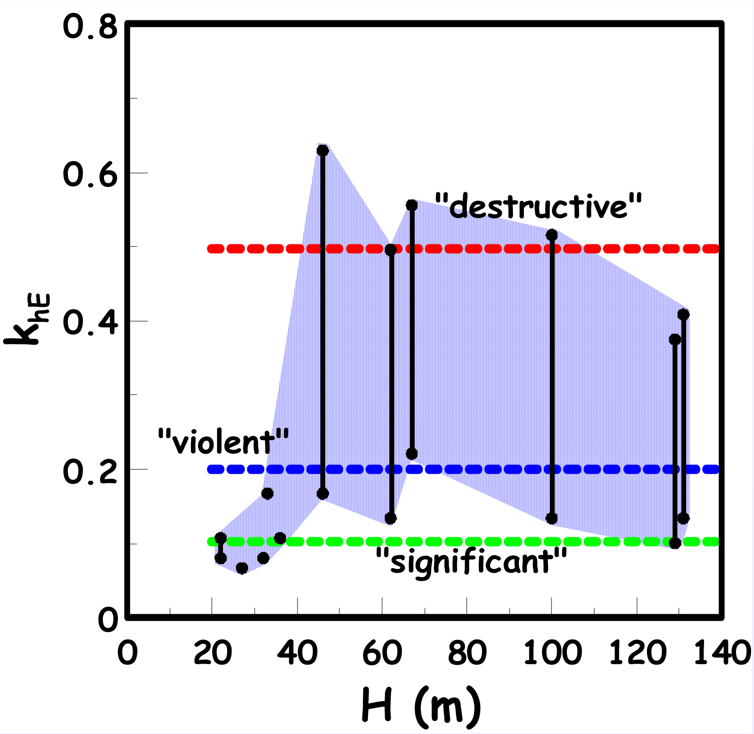


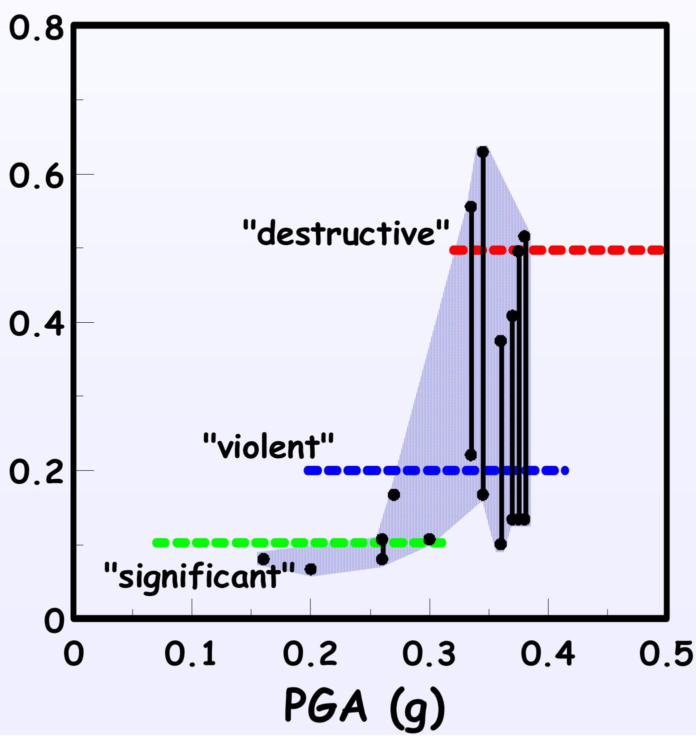


Ad-hoc values of k_{hE} based on ENGINEERING EXPERIENCE

- TERZAGHI (1950)

$$k_{hE} = \begin{cases} 0.10 & \text{“significant” earthquakes} \\ 0.20 & \text{“violent” earthquakes} \\ 0.50 & \text{“destructive” earthquakes} \end{cases}$$

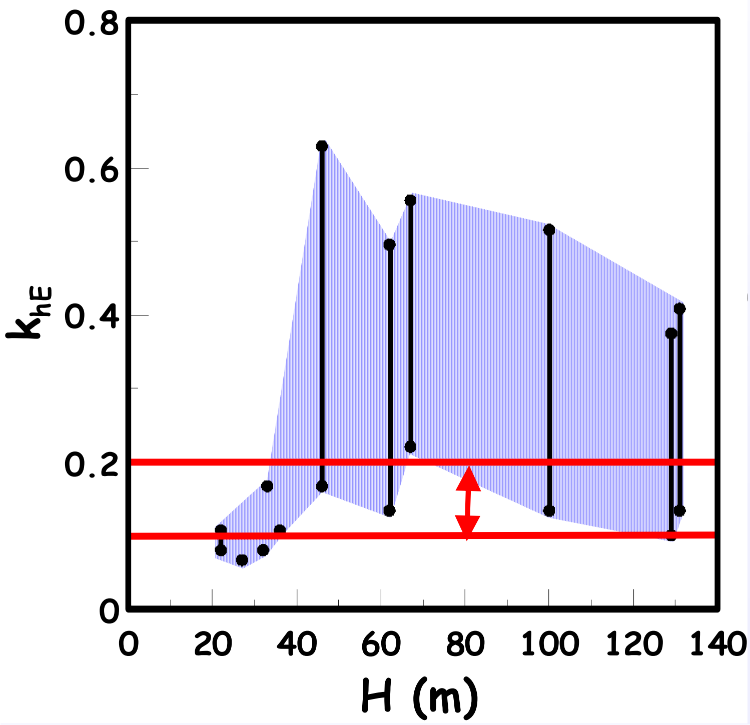


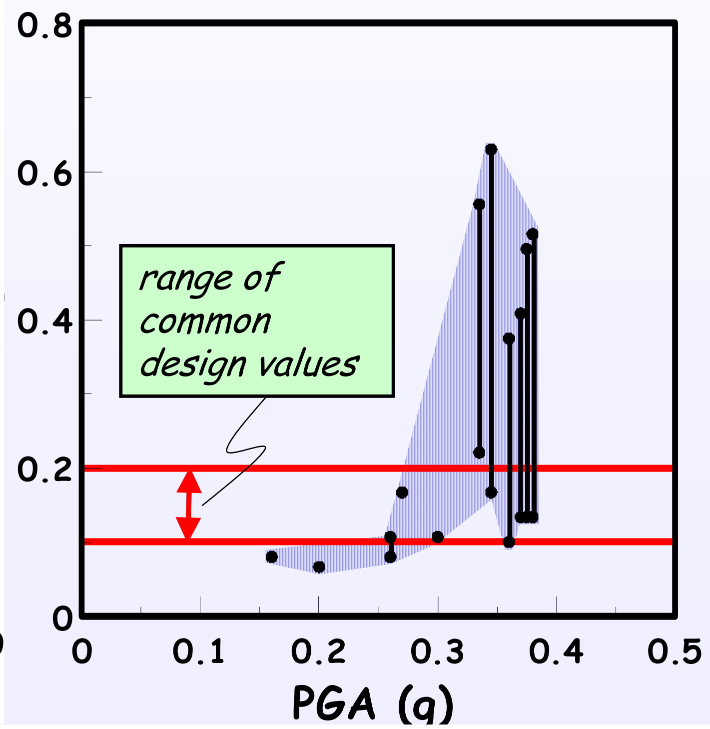


STANDARD PRACTICE of the 70's

k_{hE} = 0.10 ÷ 0.20 (*depending on M*)

$FS_d \geq$ 1.00 ÷ 1.15

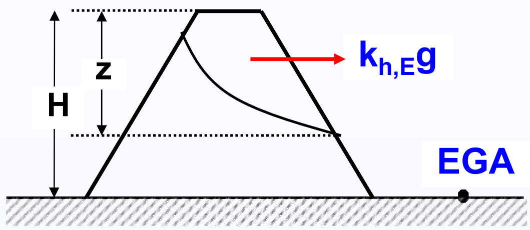


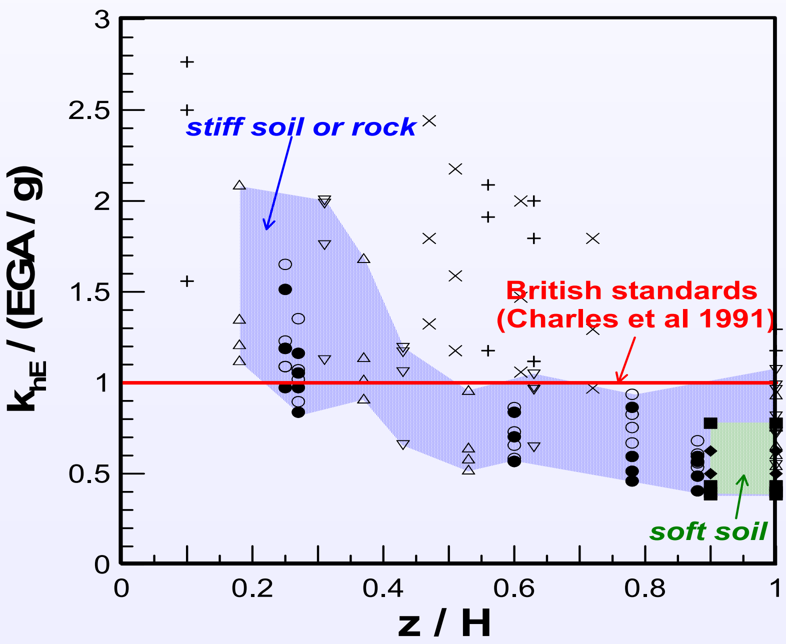


Correlation of k_{hE} with the EGA (effective FREE FIELD acceleration)

BRITISH STANDARDS (Charles et al 1991)

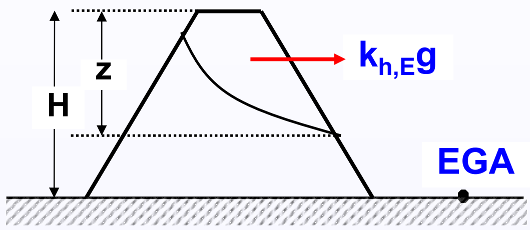
$k_{hE} \approx EGA/g$





EUROCODE EC-8

$k_{hE} = 0.50\ S\ S_T\ (EGA/g)$



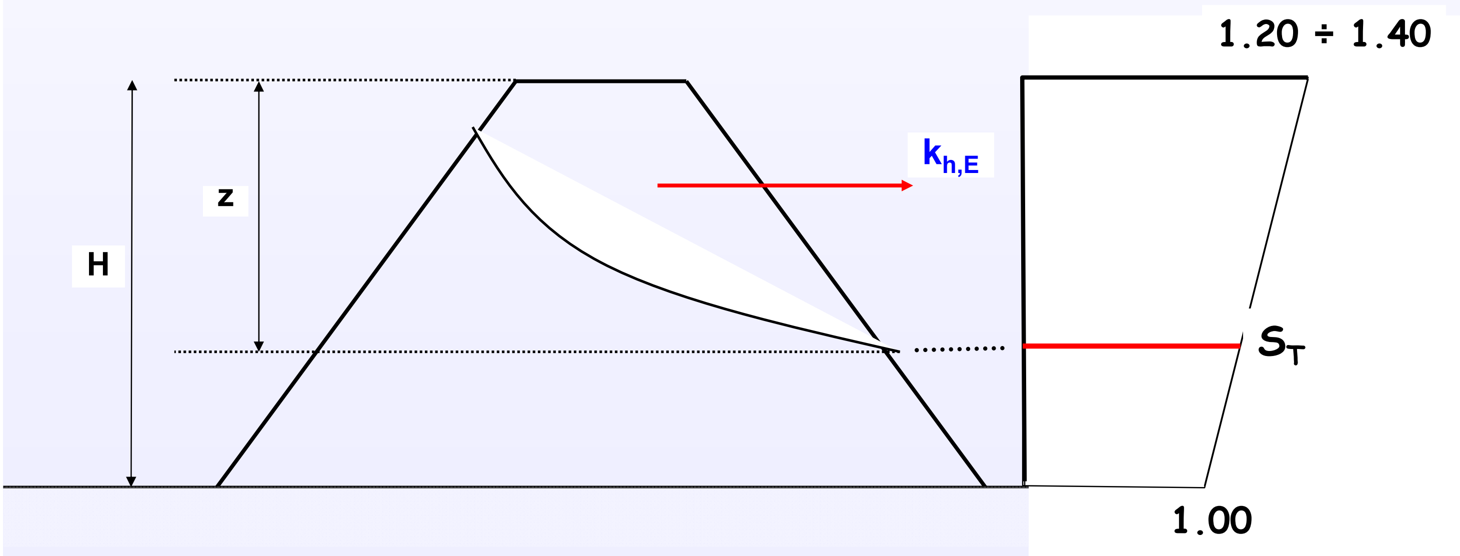
S = Soil Factor

Ground Type	V_S (m/s)	N_{SPT}	c_U (kPa)	S	
				M≤5.5	M>5.5
A	> 800	-	-	1.00	1.00
B	360-800	> 50	> 250	1.35	1.20
C	180-360	15 - 50	70 - 250	1.50	1.20
D	< 180	< 15	< 70	1.80	1.35
E	SHALLOW C or D	< 50	< 250	1.60	1.40

EUROCODE EC-8

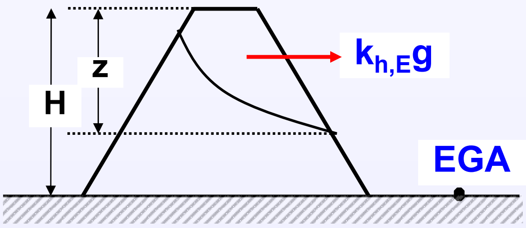
$k_{hE} = 0.50\ S\ S_T\ (EGA/g)$

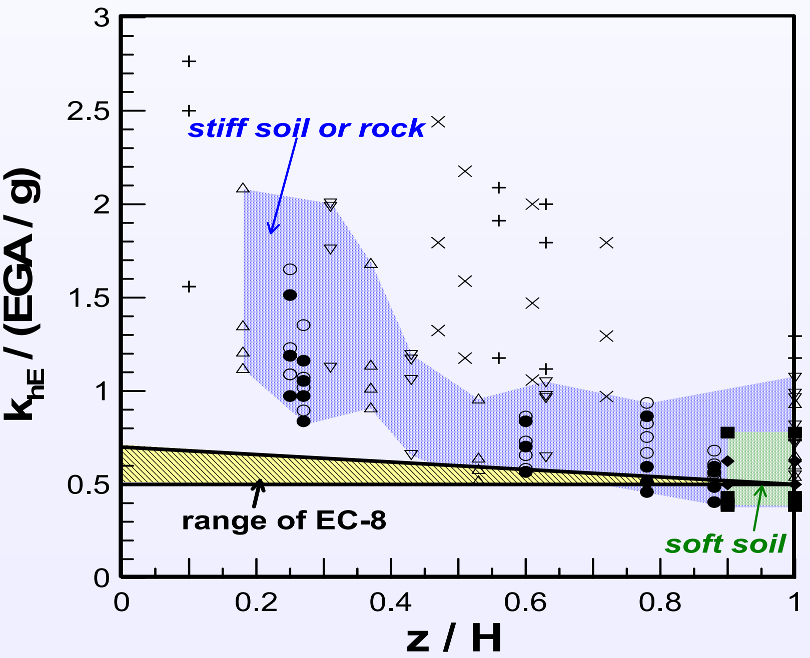
S_T = Topography Factor (only for H>30m and i>15°)



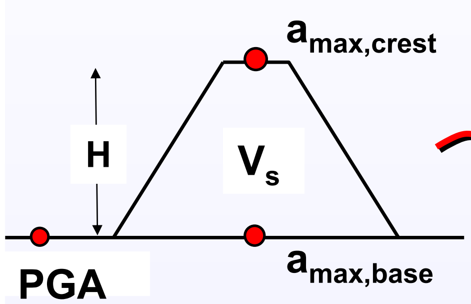
EUROCODE EC-8

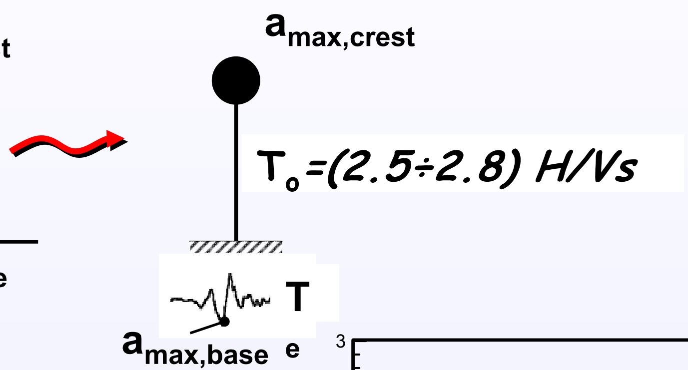
$$k_{hE} = 0.50\ S\ S_T\ (EGA/g)$$





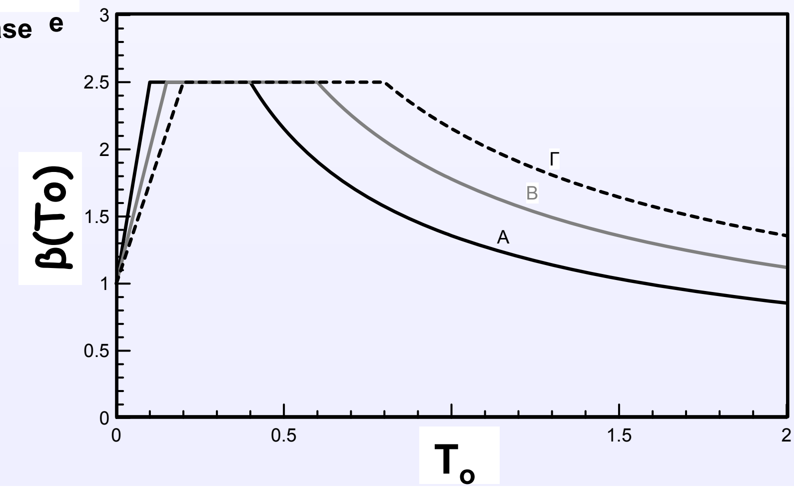
GREEK NATIONAL SEISMIC CODE EAK 2000





$$a_{max,\ base} = 0.50\ PGA$$

$$a_{max,\ crest} = \beta(To)\ a_{max,\ base}$$



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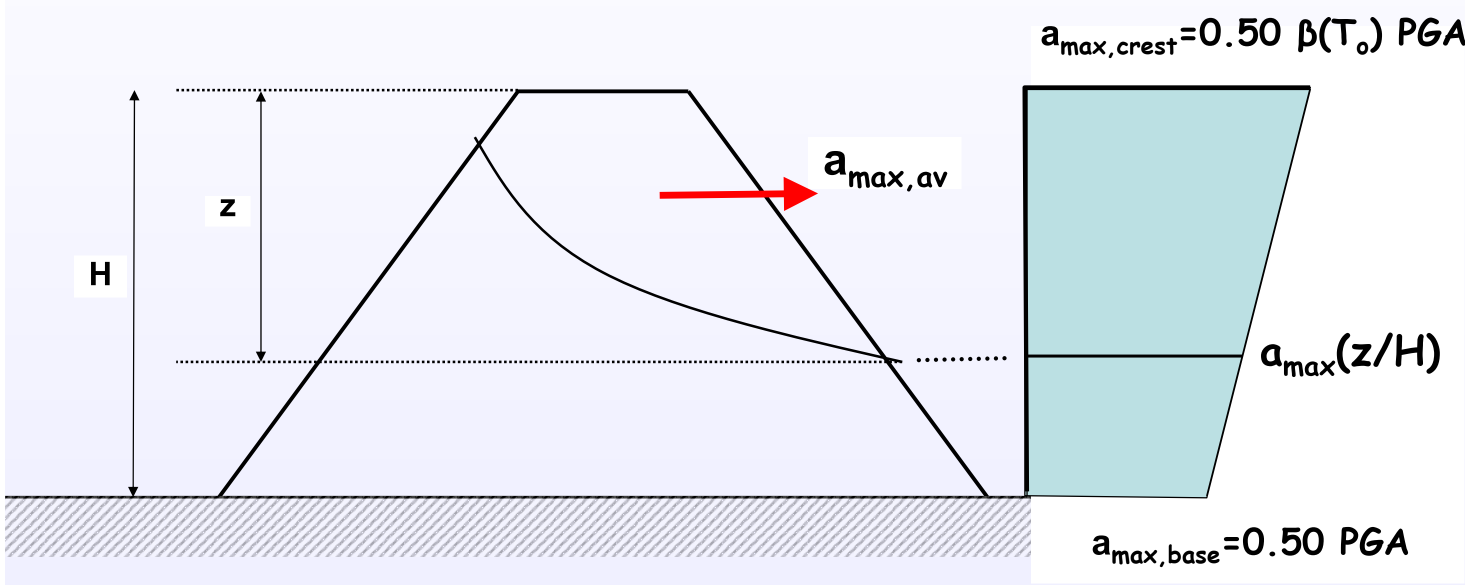
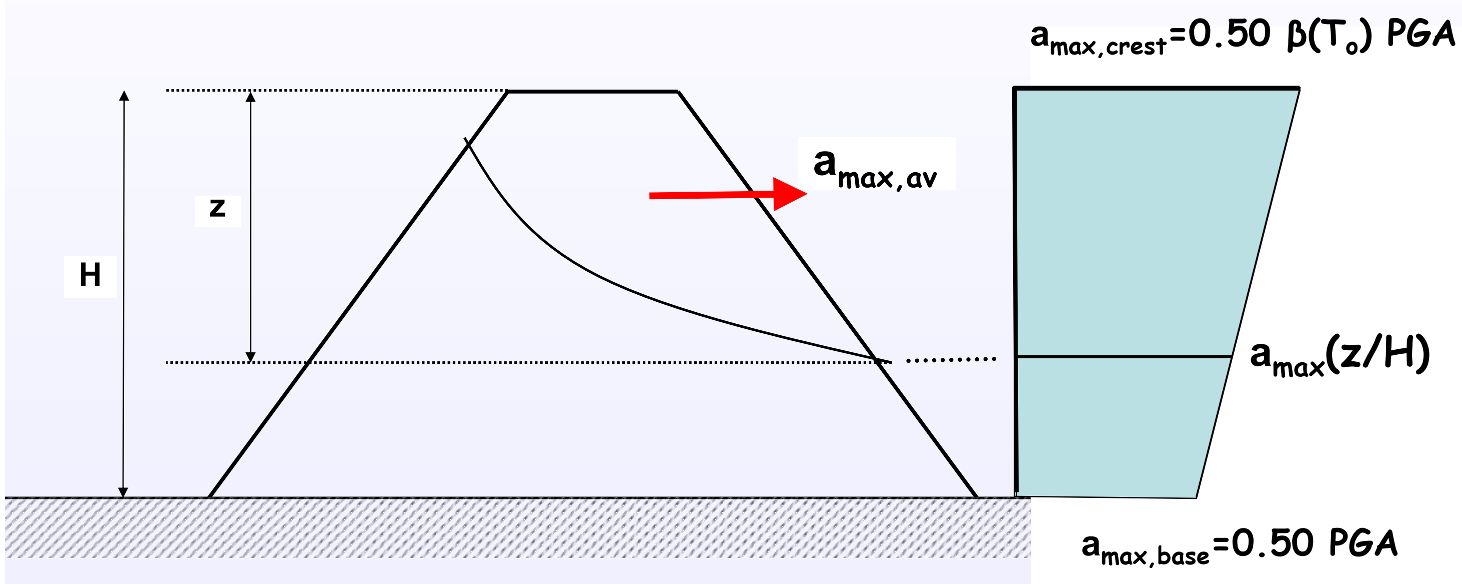


$$a_{max,av} = \frac{a_{max,crest} + a_{max}(z/H)}{2}$$

GREEK NATIONAL SEISMIC CODE EAK 2000



$$k_{hE} = 0.50 \cdot [(0.50 \div 0.80) \cdot a_{max,av} / g]$$

or

$$\boxed{k_{hE} = (0.25 \div 0.40) \cdot a_{max,av} / g}$$

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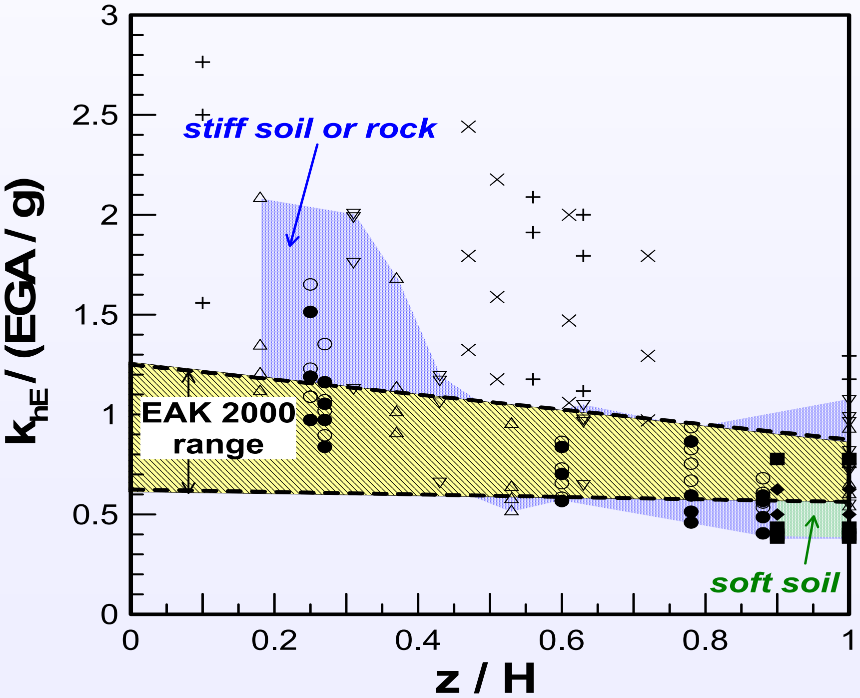

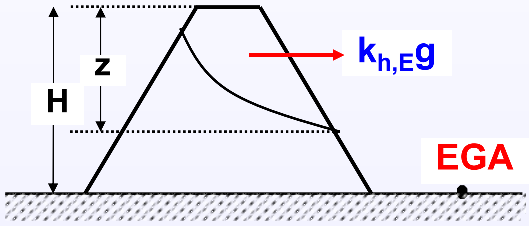

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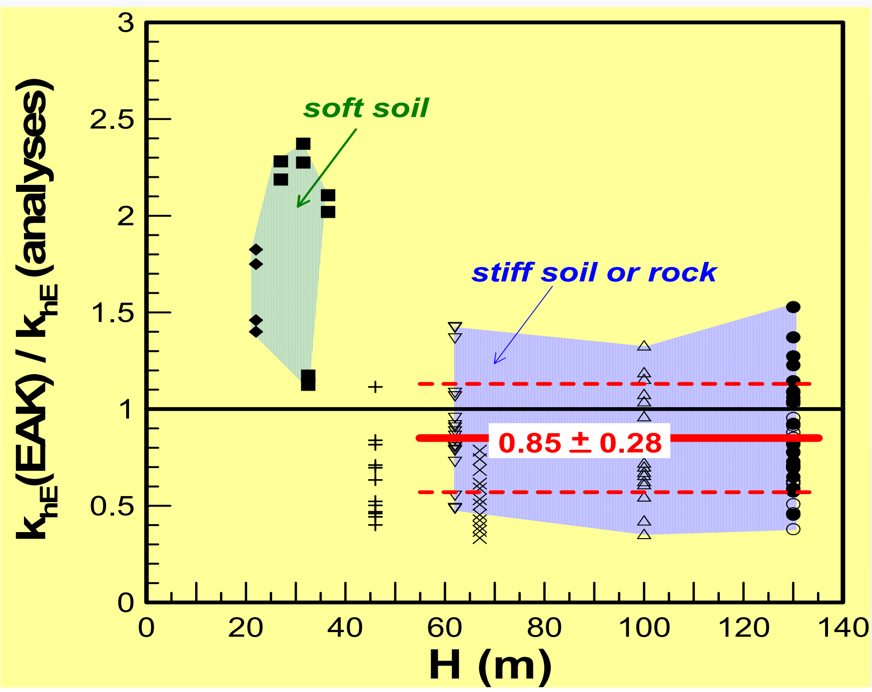

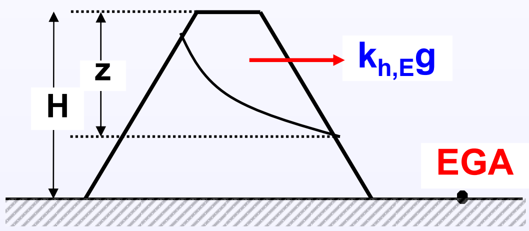

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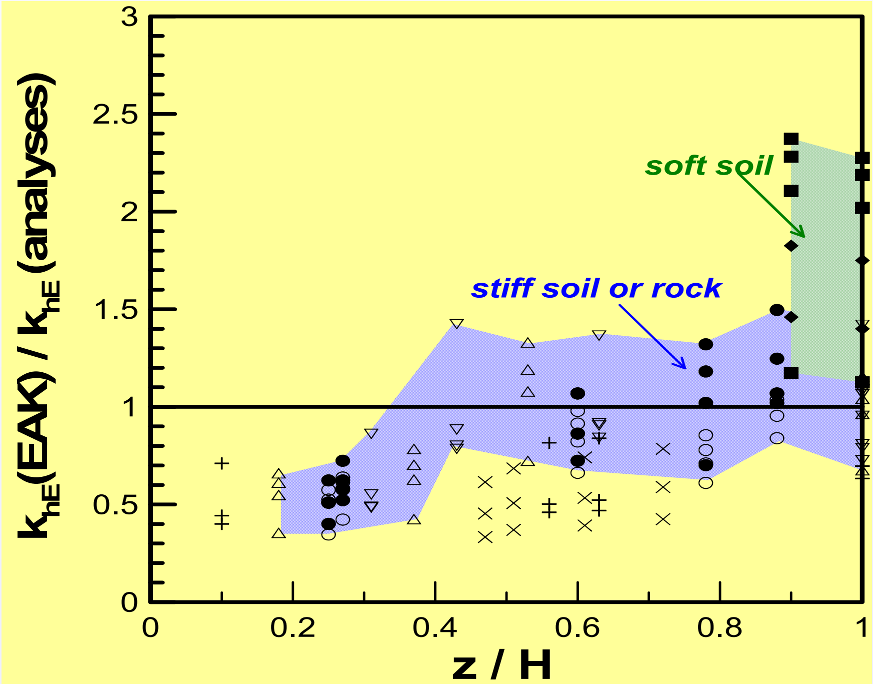

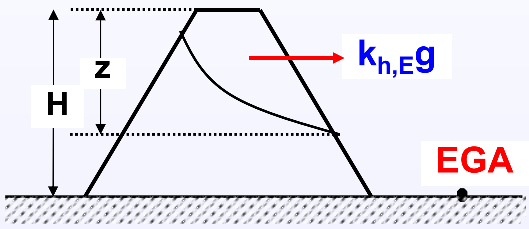

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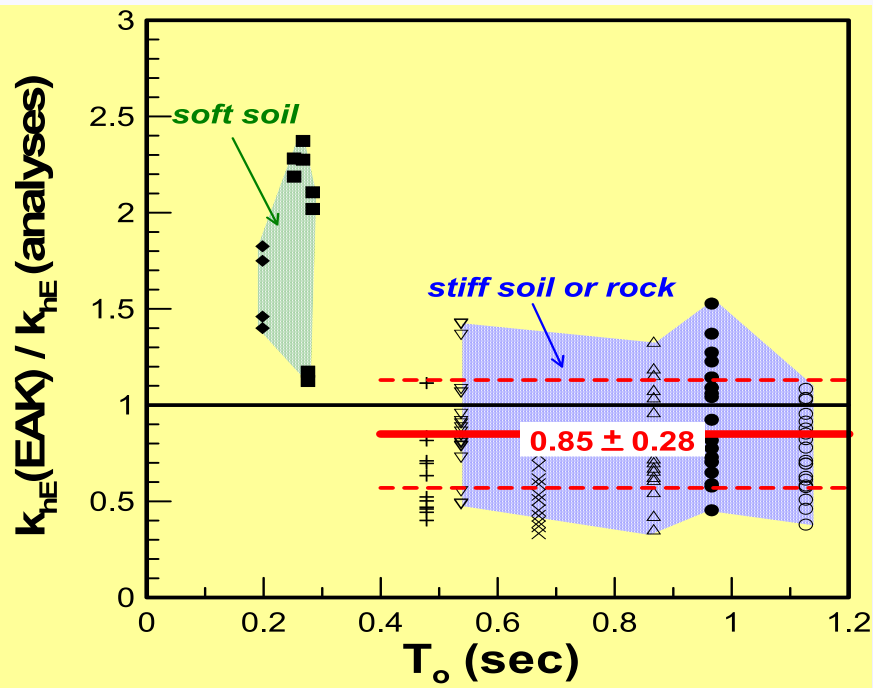

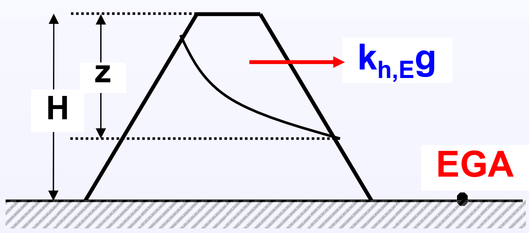

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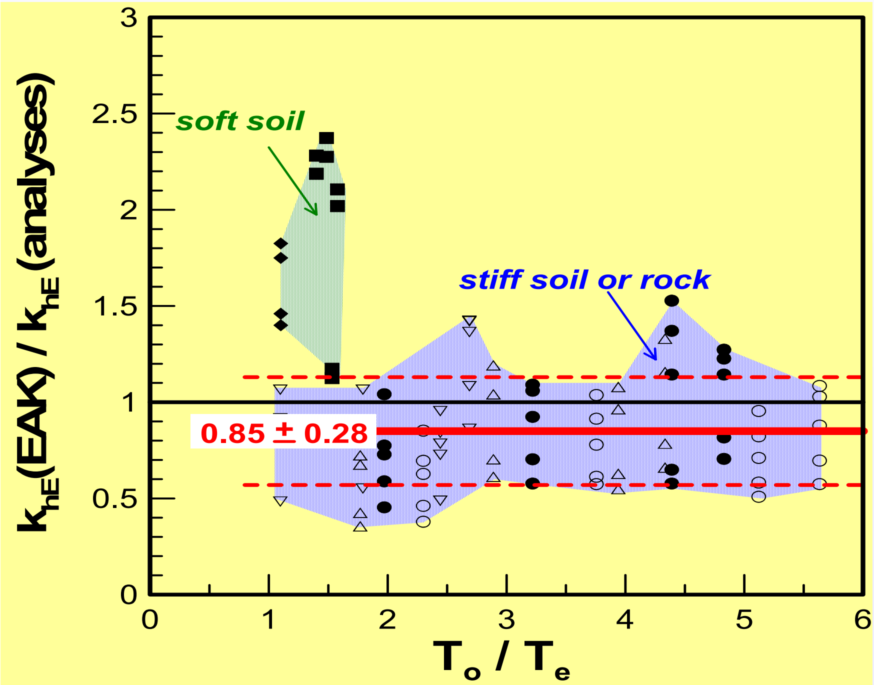
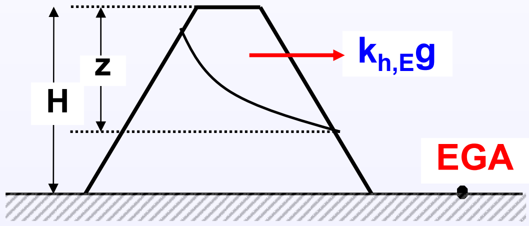




Correlation of k_{hE} with the acceleration at the CREST

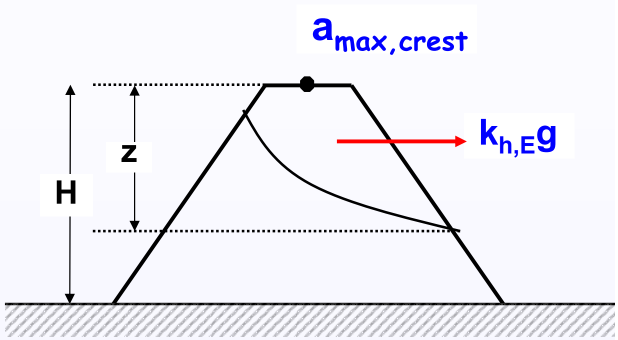
- **MARCUSON (1981)**

k_{hE} = 0.33÷0.50 ($a_{max,crest}$/g)

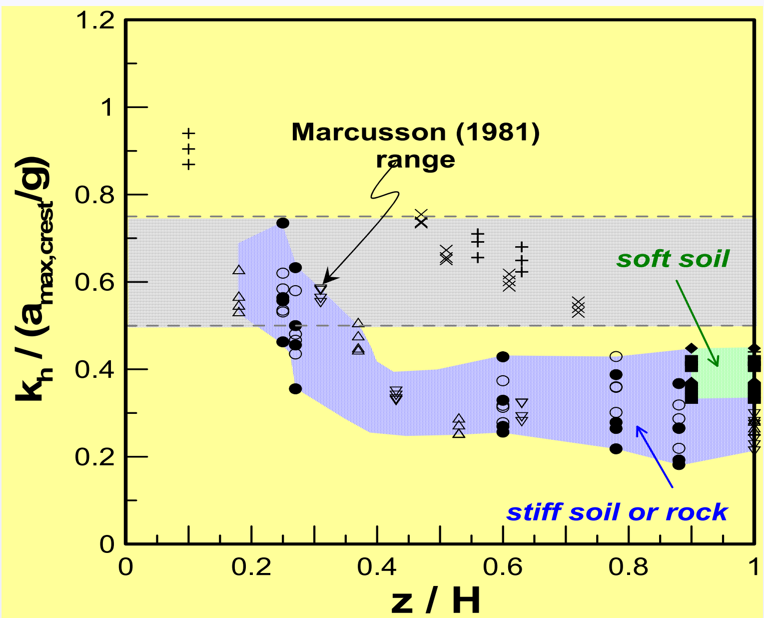
and

k_h = 0.50÷0.75 ($a_{max,crest}$/g)

(k_h ≈ 1.50 k_{hE})



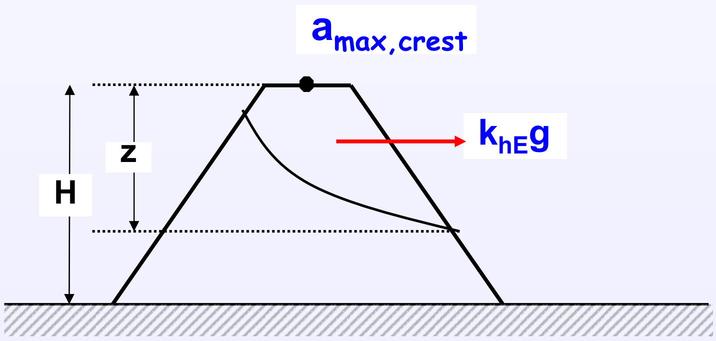
how do you compute $a_{max,crest}$?

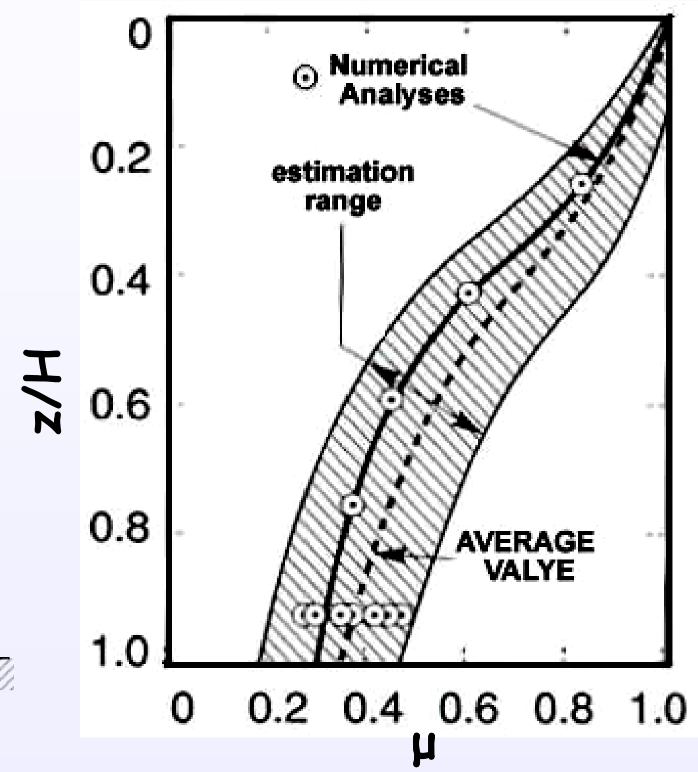


MAKDISI & SEED (1978)

$k_{hE} \approx 2/3\ k_h$

$k_h = \mu\ (a_{max,crest}/g)$





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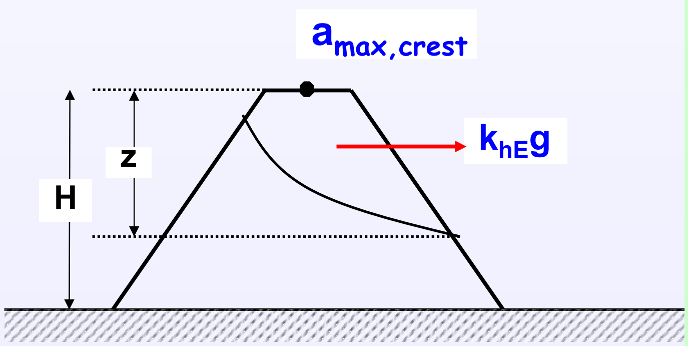
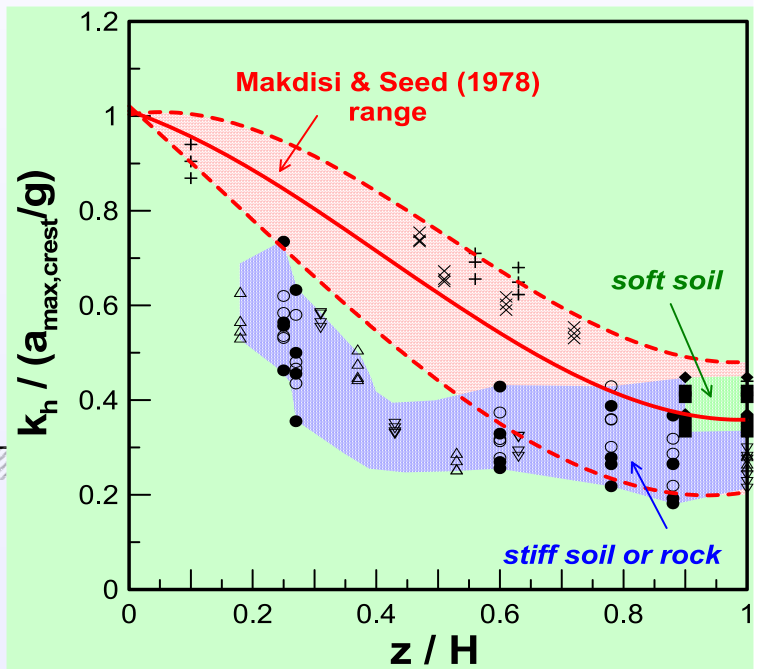
how do you compute $a_{max,crest}$?





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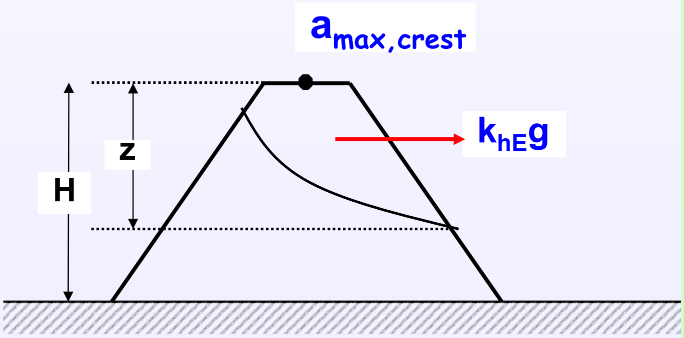
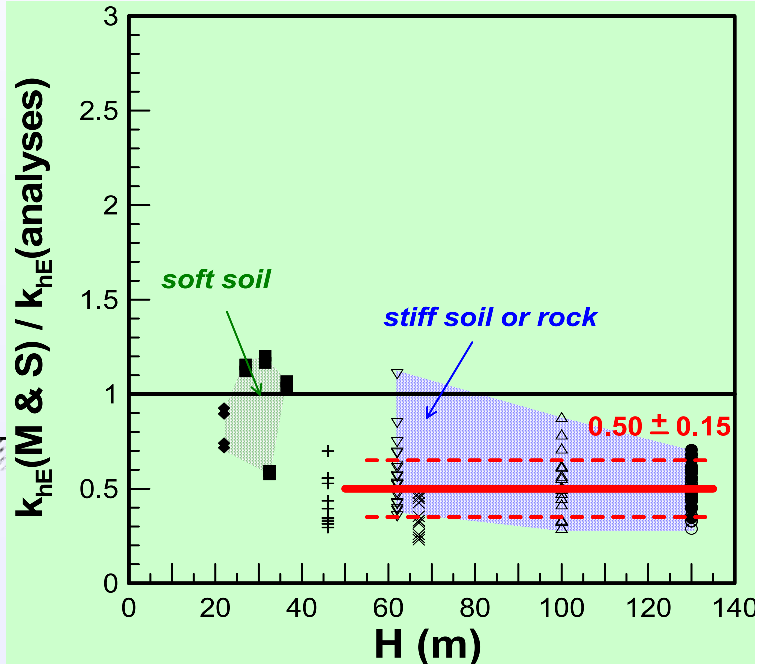
$k_{hE} \approx 2/3\ k_h$

$k_h = \mu\ (a_{max,crest}/g)$

e.g. from EAK 2000



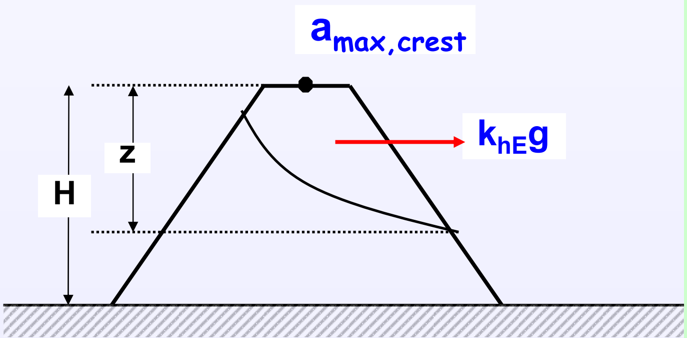


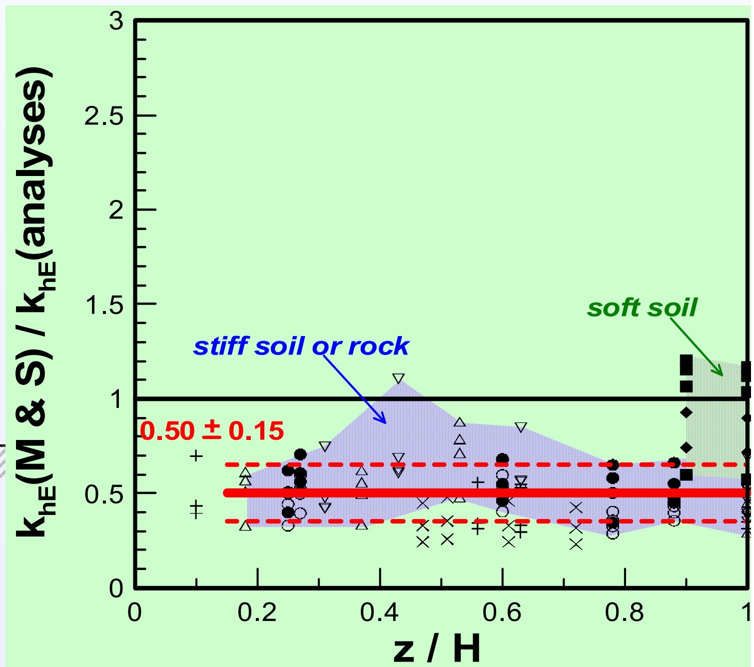
MAKDISI & SEED (1978)

$k_{hE} \approx 2/3\ k_h$

$k_h = \mu\ (a_{max,crest}/g)$

e.g. from EAK 2000





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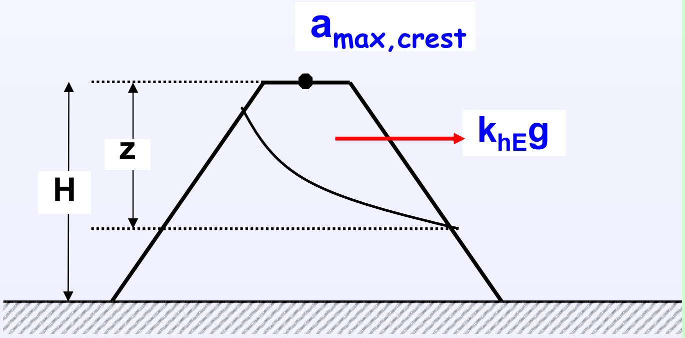

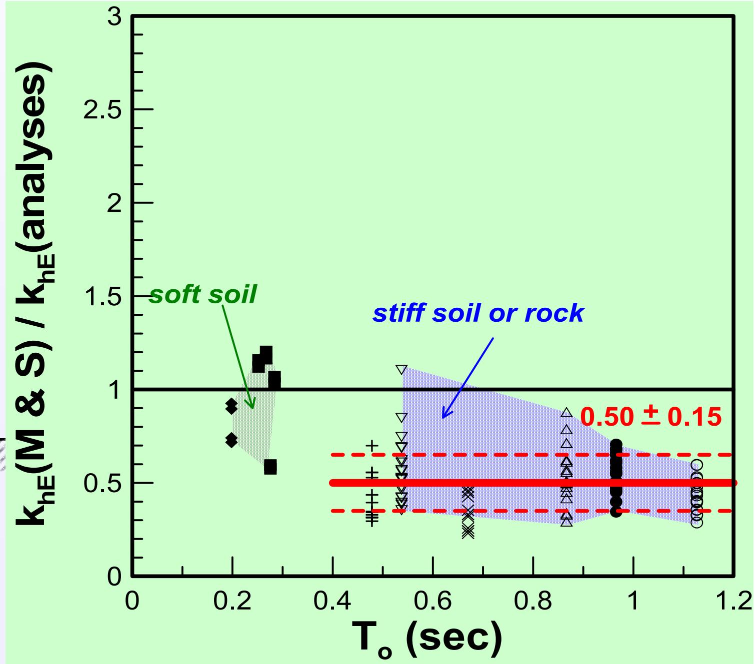

MAKDISI & SEED (1978)

$k_{hE} \approx 2/3\ k_h$

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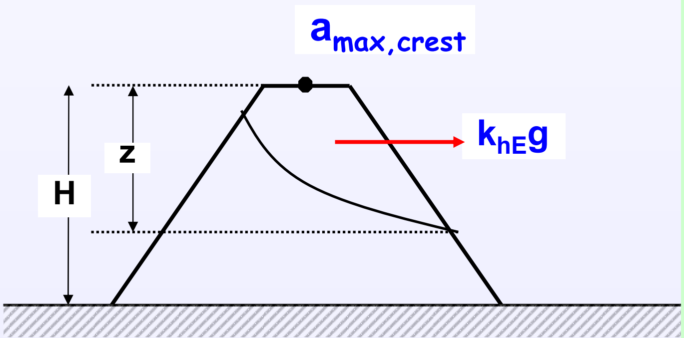
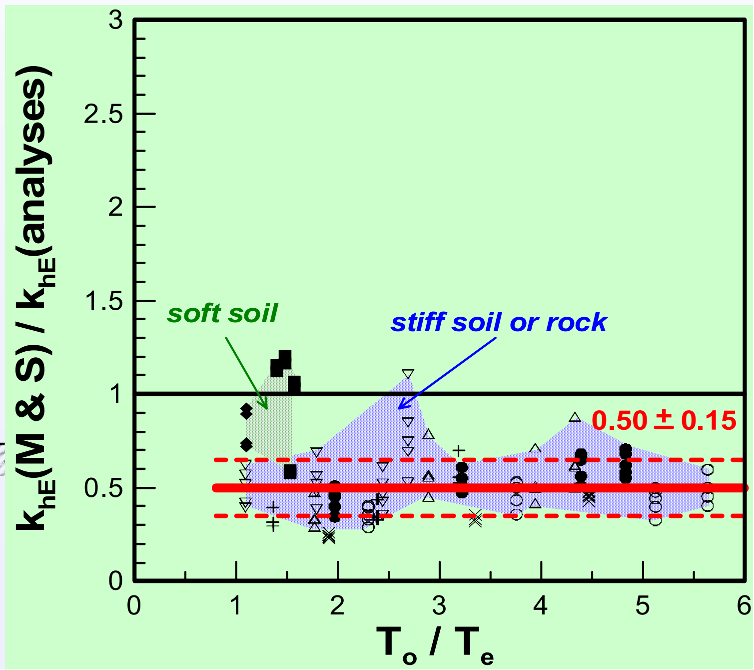
e.g. from EAK 2000





8.5 CONCLUDING REMARKS.....

CONCLUDING REMARKS . . . .

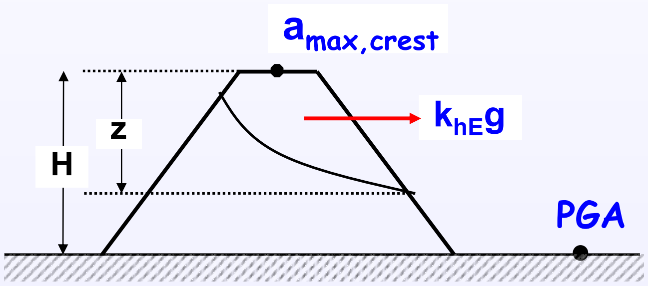
- The PSEUDO STATIC approach with FS_d >1.0 does not prevent slope stability failure.

  However,

  for STABLE SOILS, it ensures that only very small (e.g.<10 cm) downslope displacements will occur.

  for UNSTABLE SOILS (e.g. liquefiable): NEVER USE IT !

- If we can tolerate these displacements, the SEISMIC COEFFICIENT k_{hE} may become much smaller than the corresponding peak seismic acceleration:

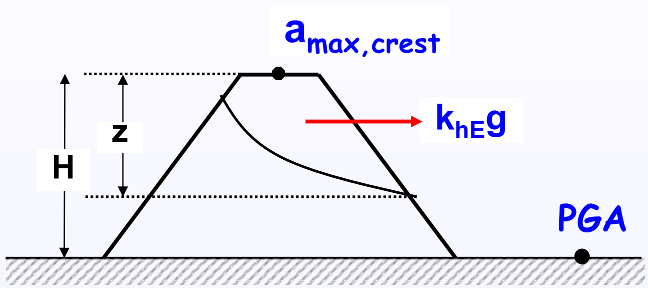


$$k_{hE} = (0.25 \div 1.60)\ PGA/g$$

$$k_{hE} = (0.15 \div 0.56)\ a_{max,crest}/g$$

* *In general, the higher values are associated with*
- *Tall Embankments (H > 30m) &*
- *Shallow failure surfaces (z/H < 0.40)*

- When ONLY the PGA is known,



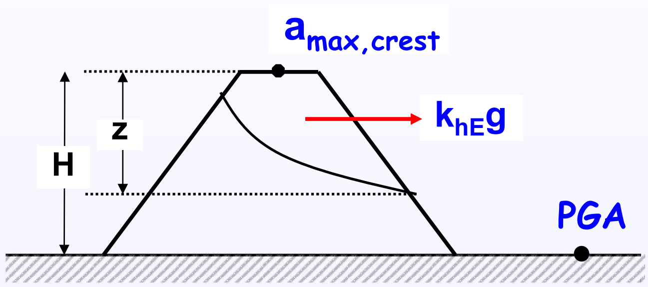
you may use:

- the BRITISH STANDARDS
(conservative for z/H > 0.40)

- EAK 2000
(O.K. for z/H > 0.40)

The EC-8 is rather UN-conservative

- When the maximum CREST ACCELERATION $a_{max,\ crest}$ is known (e.g. from seismic analysis of the dam),



you may use:

- MARCUSON (1981)
  [only for very shallow z/H < 0.30]
- MAKDISHI & SEED (1978)
  (overconservative for z/H = 0.15 ÷ 0.60)

HOMEWORK 8.1: Earthquake – induced Permanent displacements of an infinite slope

This HWK concerns an idealized geotechnical natural slope, where 5m of weathered (soil-like) rock rests on the top of intact rock. The inclination of the slope relative to the horizontal plane is i=25deg, while the mechanical properties of the weathered rock are γ=18κN/m^3, c=12.5 kPa and φ=28deg. No ground water is present. Assuming infinite slope conditions:

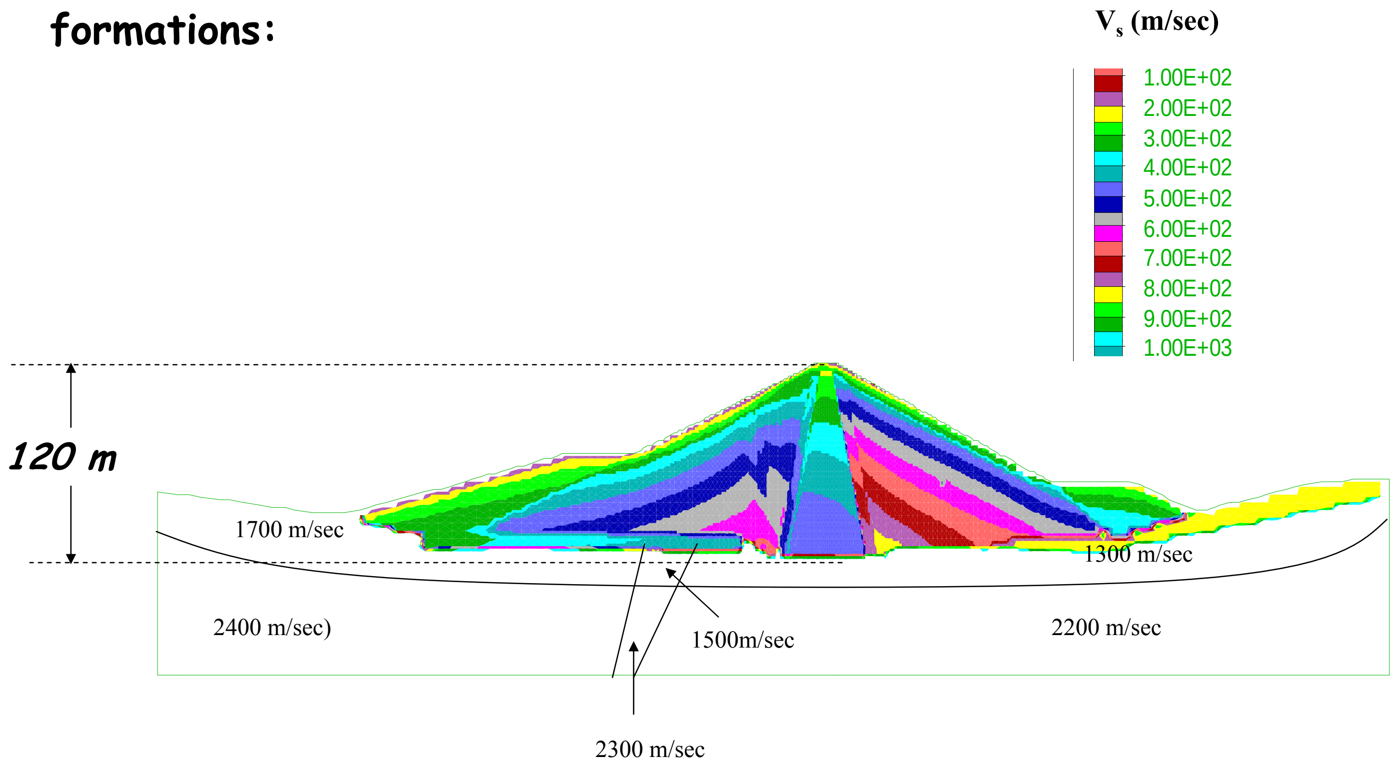
(a) Compute the static factor of safety FS_{ST}.

(b) Compute the seismic factor of safety $FS_{EQ.}$, for a maximum horizontal acceleration $a_{H,max}$=0.45g accompanied by a maximum vertical acceleration $a_{V,max}$=0.15g.

(c) In case that $FS_{EQ.}$, comes out less than 1.00, compute the associated downslope displacements, for an estimated predominant excitation period T_e=0.50 sec.

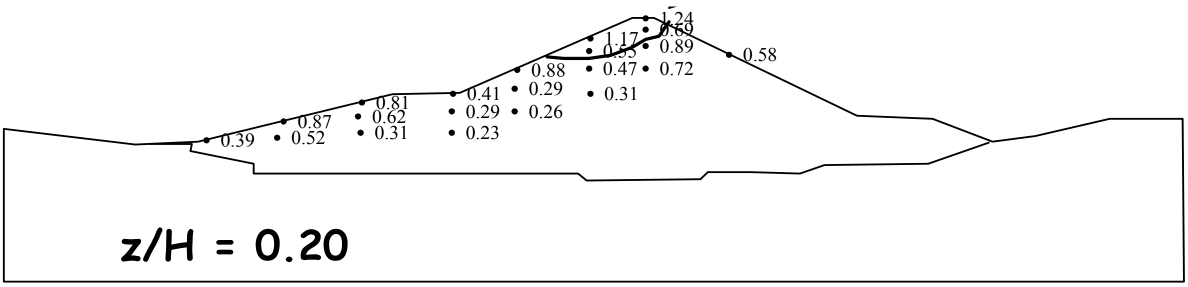
HOMEWORK 8.2: The case of Ilarion Dam in Northern Greece

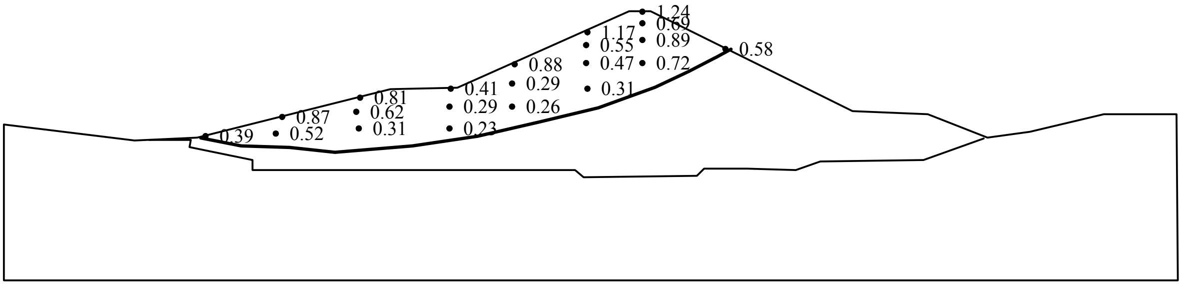
The dam is constructed on weathered rock formations:



Compute seismic coefficients k_h, k_{hE}

- for the following potential failure surfaces





z/H = 0.90

- and the following peak seismic accelerations and predominant periods for the (horizontal) seismic excitation:
  - Low frequency excitation a_{max}=0.37g, T_e=0.20 s
  - High frequency excitation a_{max}=0.20g, T_e=0.65 s