8. Seismic Analysis of Slopes

Αντισεισμικός Σχεδιασμός Πρανών Επιχωμάτων

με την πολύτιμη συμβολή του

Αχιλλέα Παπαδημητρίου

Λέκτορα στο Πανεπιστήμιο Θεσσαλίας

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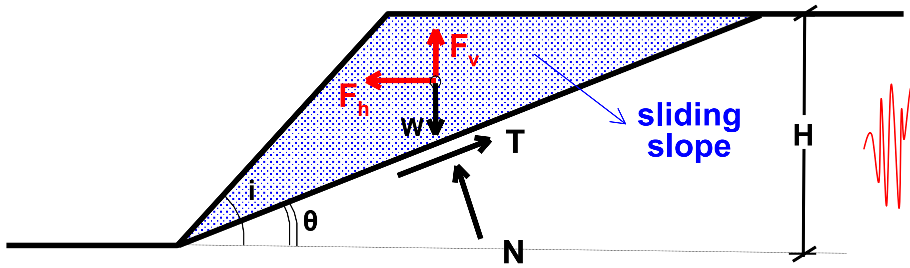
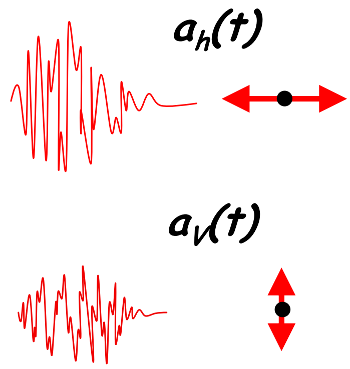
Πρόσθετο Διάβασμα:

- **Steven Kramer:**
  Chapter 10 (10.1 έως και 10.6.1)

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





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

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

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

- $FS_d > 1$ safe conditions ✓

  $FS_d < 1$ slope failure (dynamic) ?

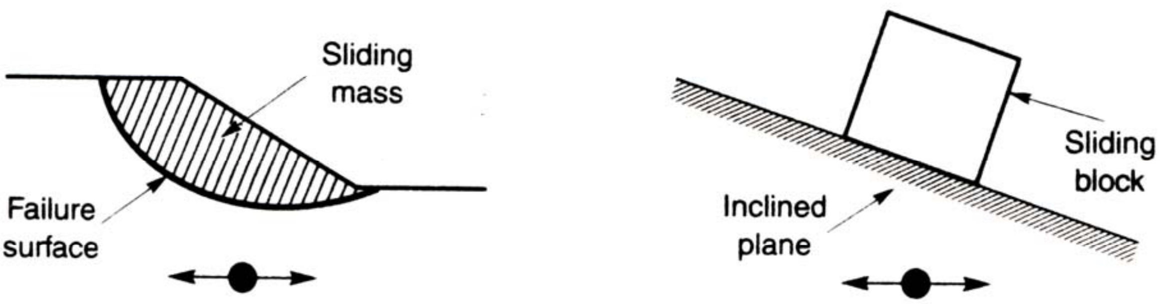
- Observe that when the horizontal inertia force F_h increases, then the factor of safety decreases drastically, as the nominator decreases and the denominator increases.

- On the other hand, the effect of the vertical inertia force F_v on the nominator and the denominator is similar, so that the overall effect on the factor of safety is much less. For this reason, as well as due to the fact that it is rather unlikely to have the peak F_h and F_v acting simultaneously, we often neglect F_v or consider a reduced value.

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"



(a) (b)

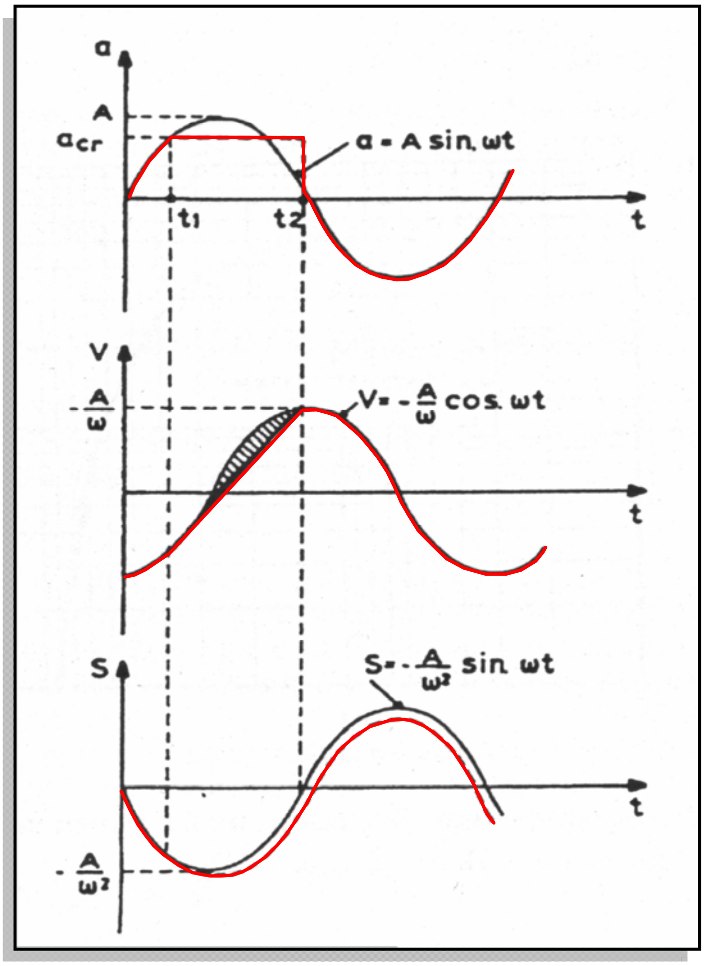
HOWEVER,

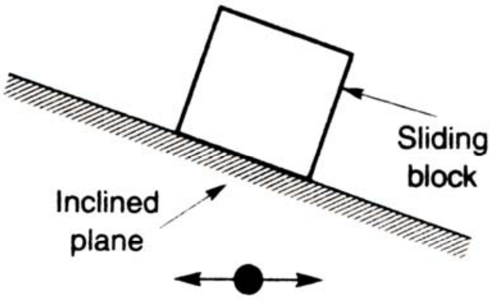
unlike STATIC FAILURE which lasts for ever,

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

"Sliding Block" kinematics

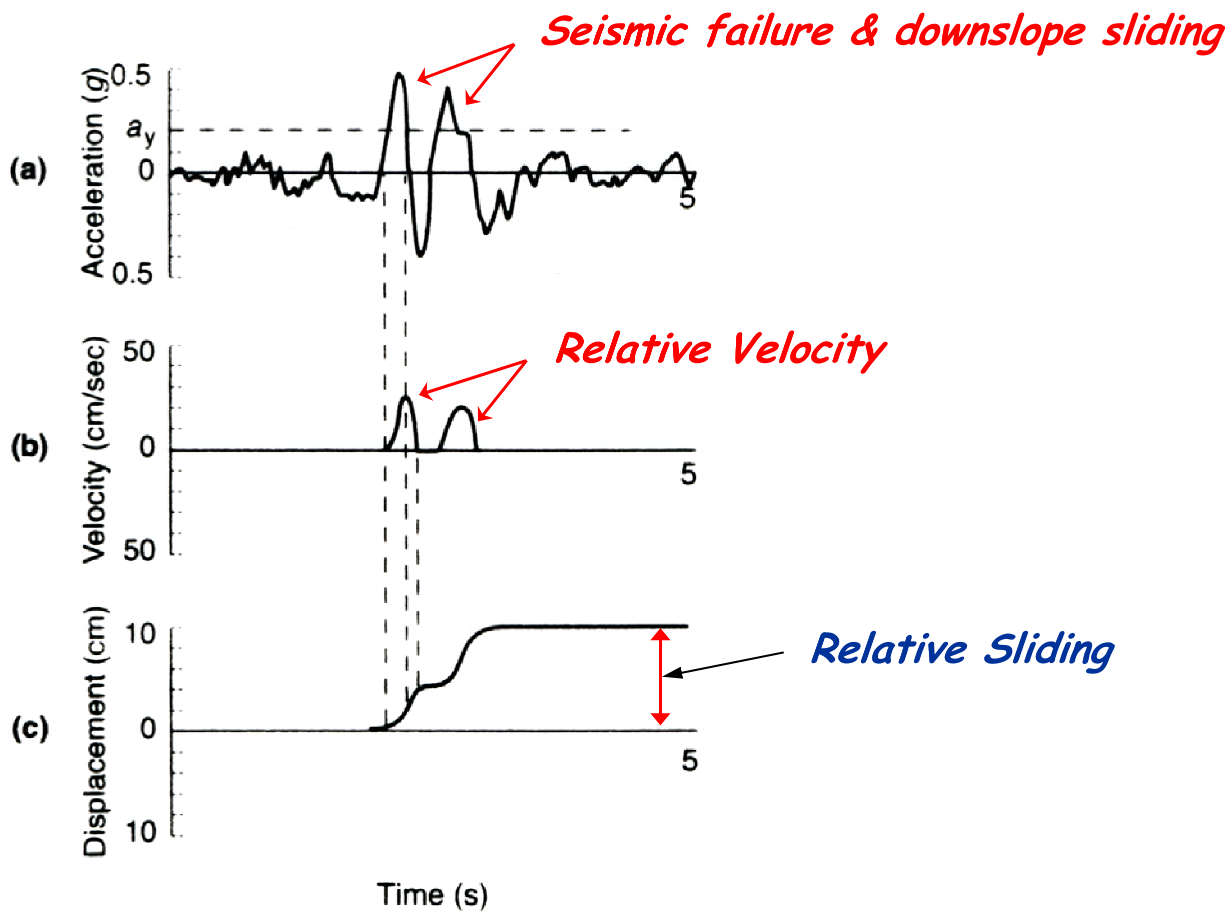
(for the simplified case of sinusoidal motion)





base motion

sliding block motion



Computation of Relative Sliding ....

NEWMARK (1965)

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

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

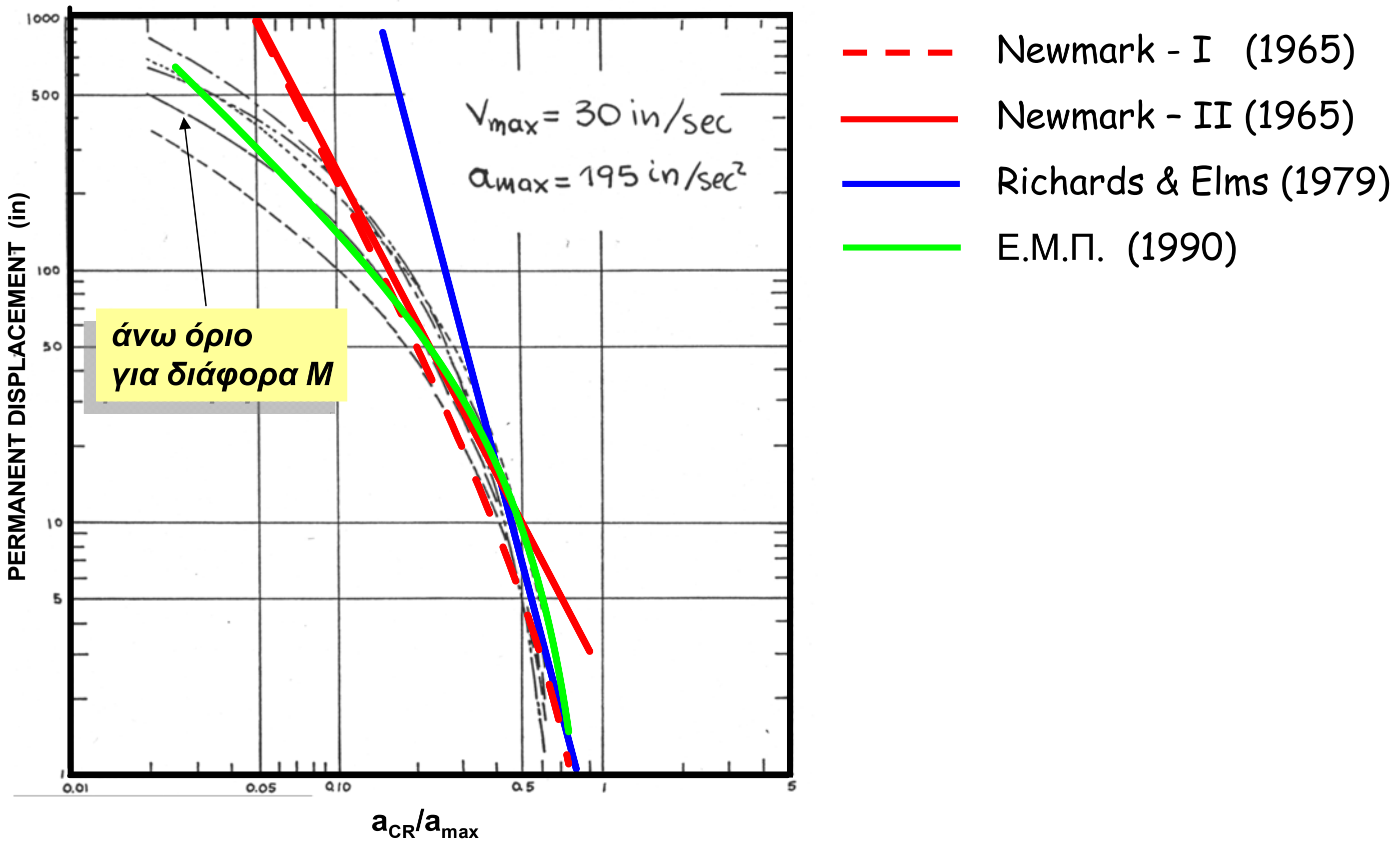
RICHARDS & ELMS (1979)

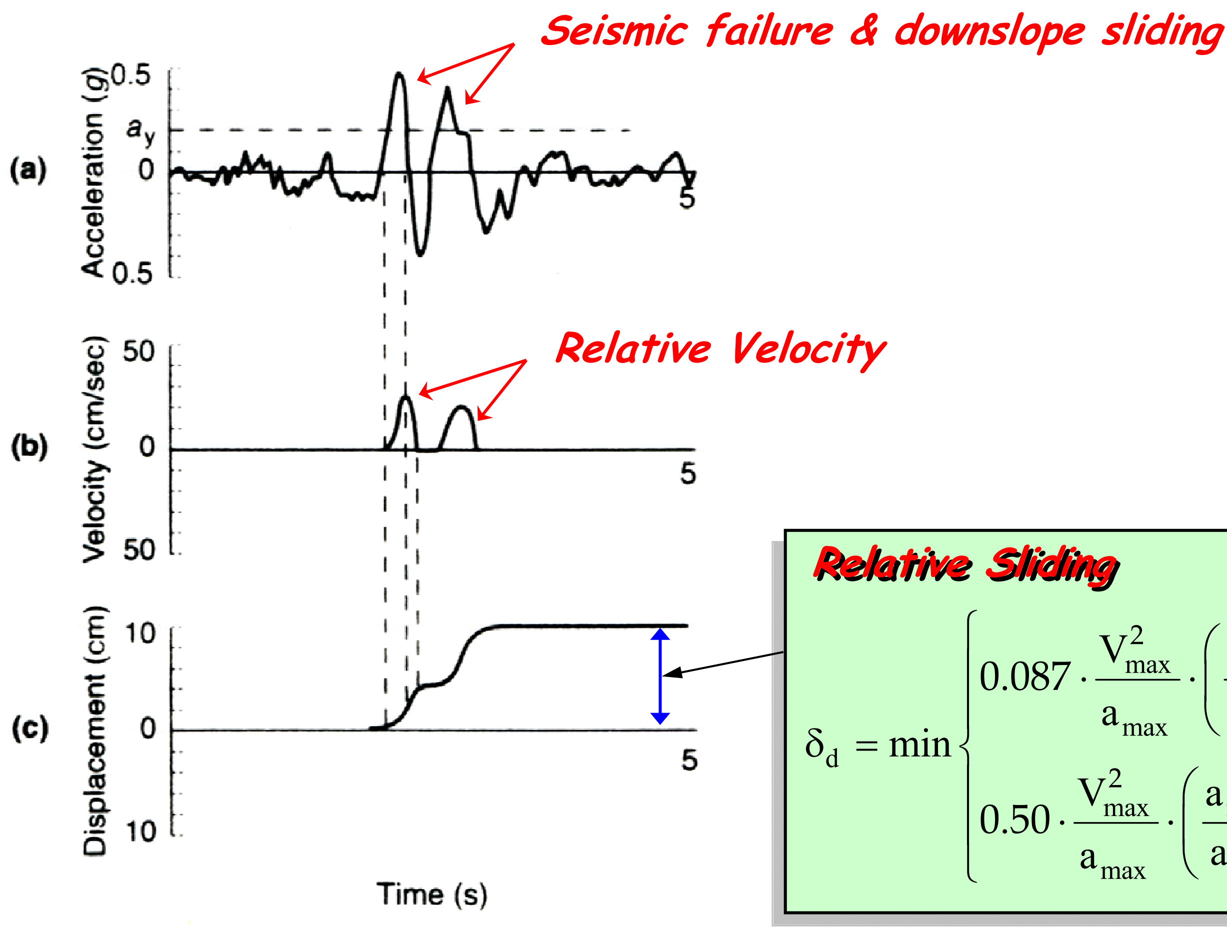
$$\delta \approx 0.087 \cdot \left(\frac{V_{max}^2}{a_{max}}\right) \cdot \frac{1}{\bar{a}_{CR}^{\,4}}$$

E.M.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

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

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

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

→ **9 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}

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.

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

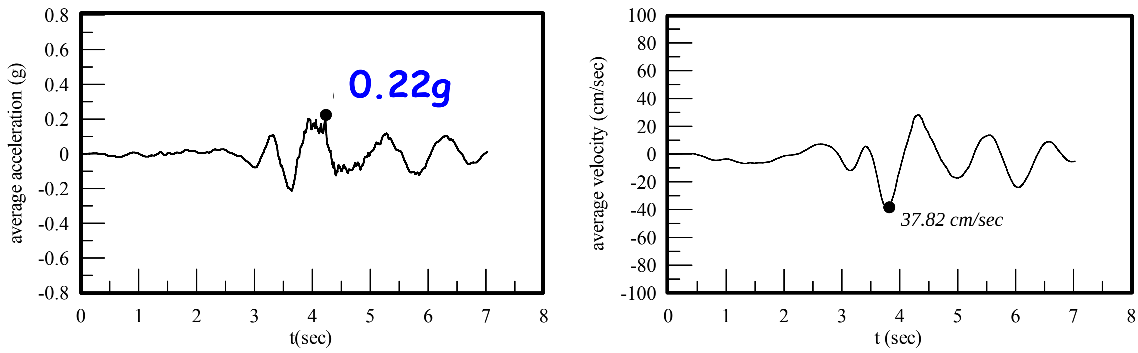
with $FS_d = 1.0 \div 1.10$

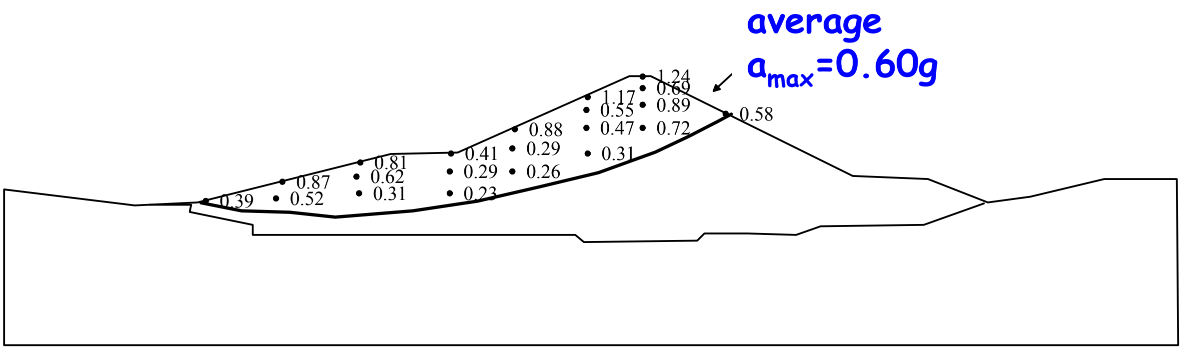
as this will usually lead to fairly small (< 10 cm) downslope displacements

SECONDLY . . . .

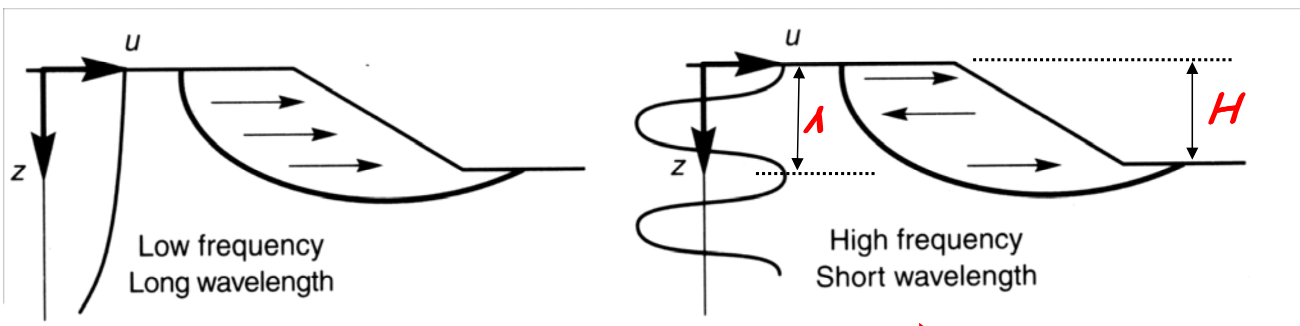
observe these numerical results ……

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:



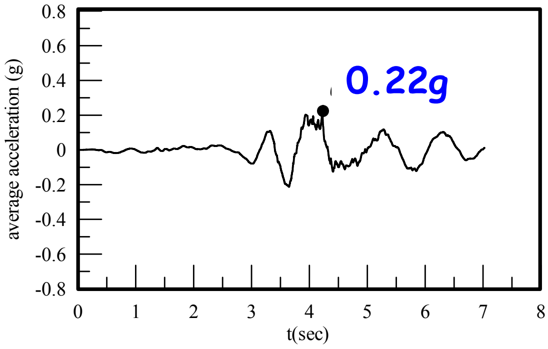
(a) (b)

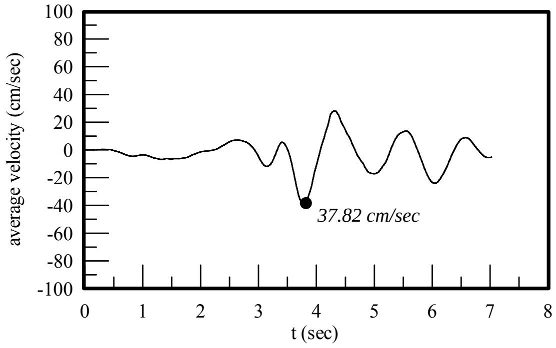
For common
earth dams H=(30 ÷ 120m) & earthquakes (T_e=0.30 ÷ 0.60s)

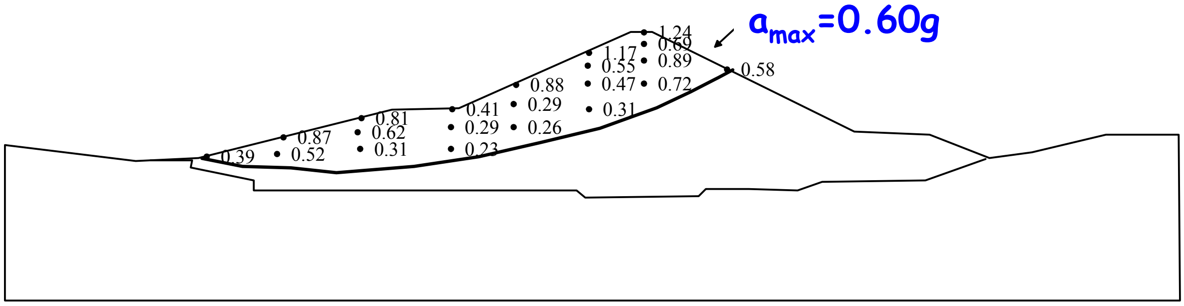
$\lambda \approx (1.00 \div 2.00)\, H$ i.e.

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. 2 Review & Evaluation of

SEISMIC COEFFICIENTS k_{hE}

proposed in the literature

REVIEW of k_{hE} values, proposed ……

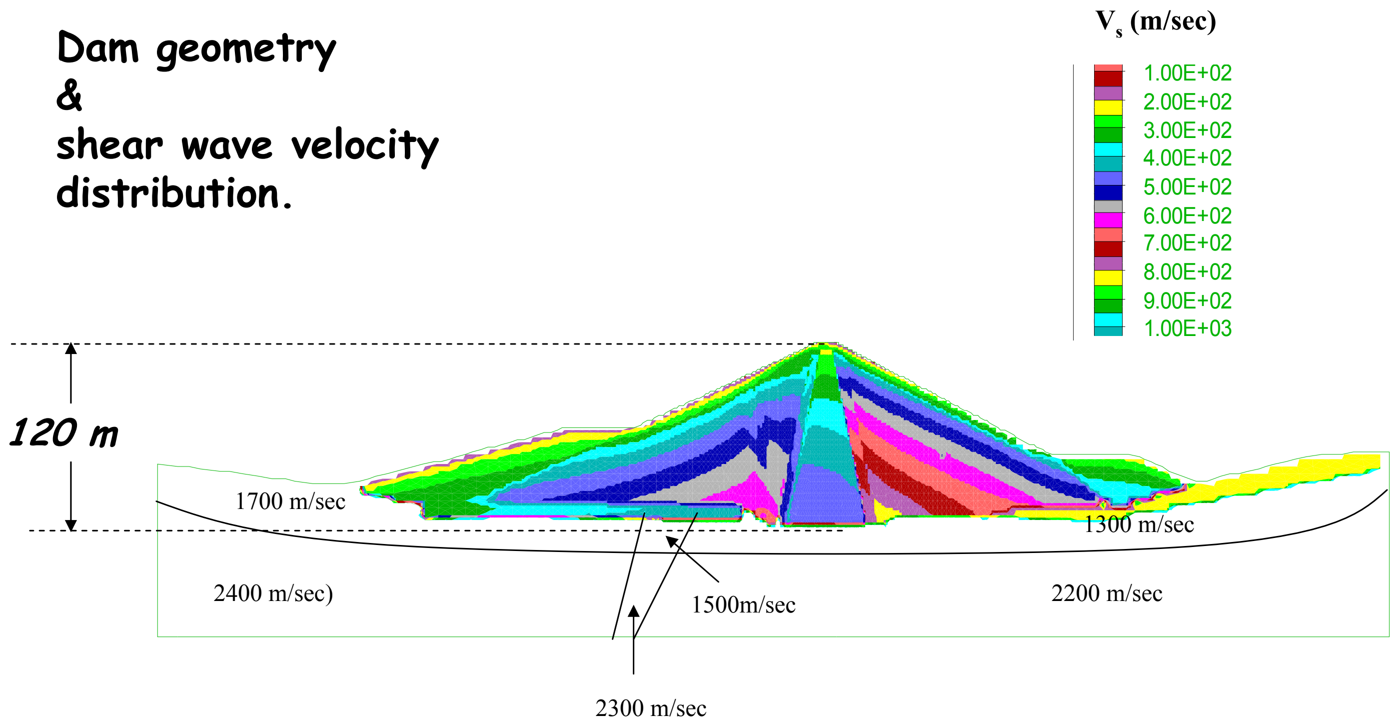
- on the basis of mere engineering . . . INTUITION
- in relation with the FREE FIELD peak ground acceleration (PGA)
- in relation with the peak seismic acceleration at the DAM CREST

EVALUATION in comparison with numerical analyses which take into account:

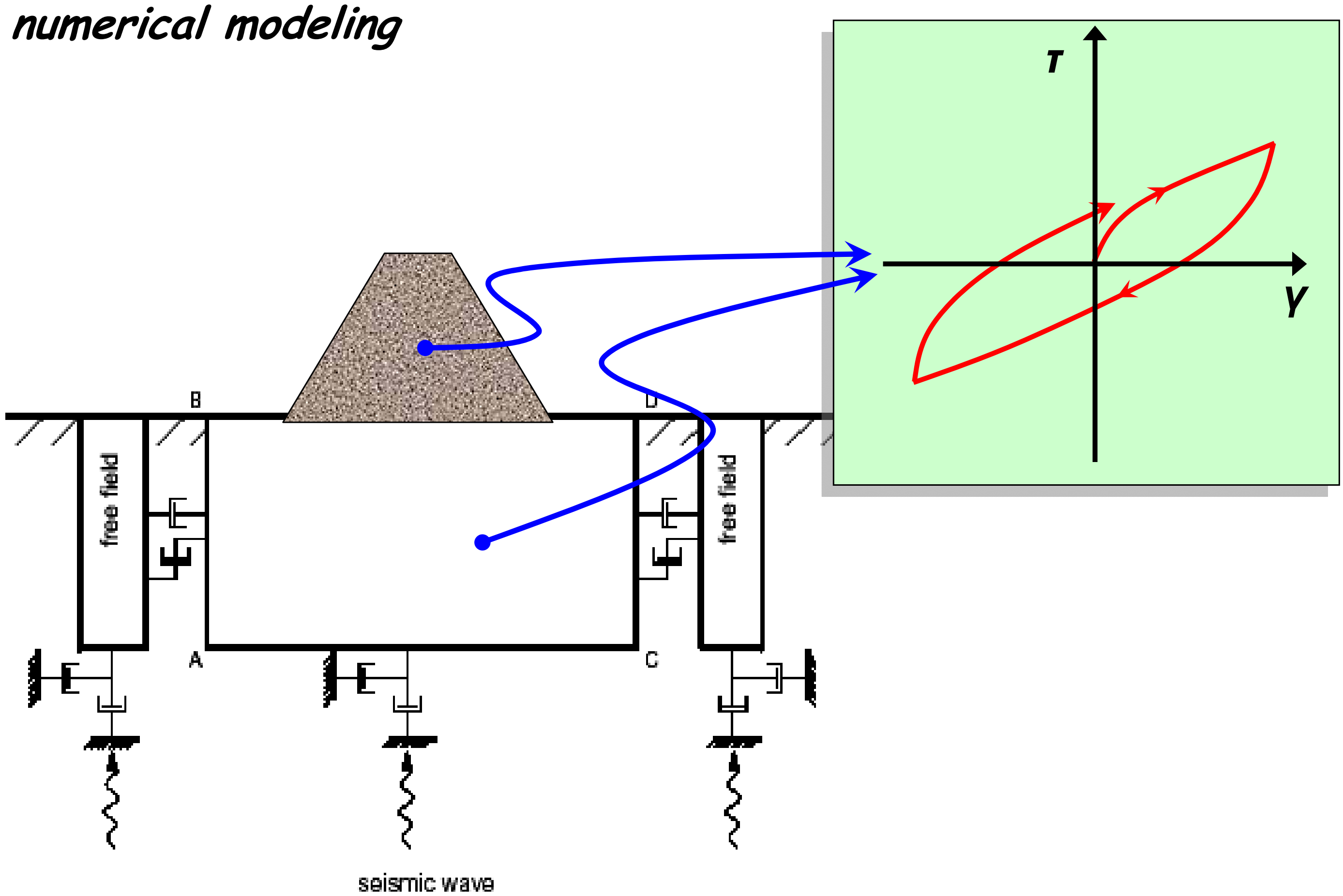
- foundation SOIL CONDITIONS
- dynamic DAM RESPONSE
- NON-LINEAR HYSTERETIC soil response to seismic excitation

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

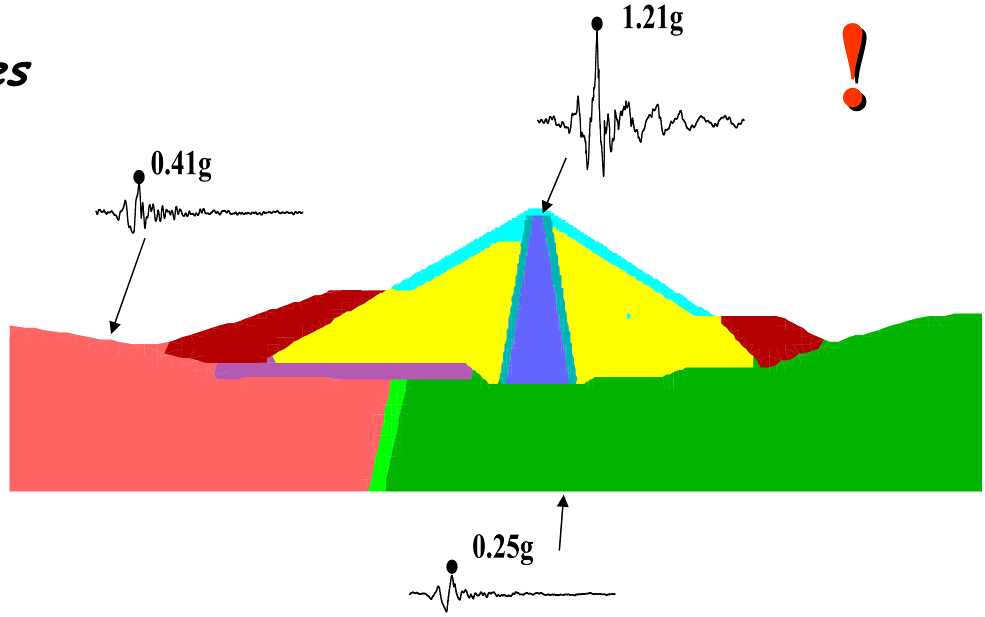
Dam geometry
&
shear wave velocity
distribution.



Basic aspects of numerical modeling



Typical acceleration time histories

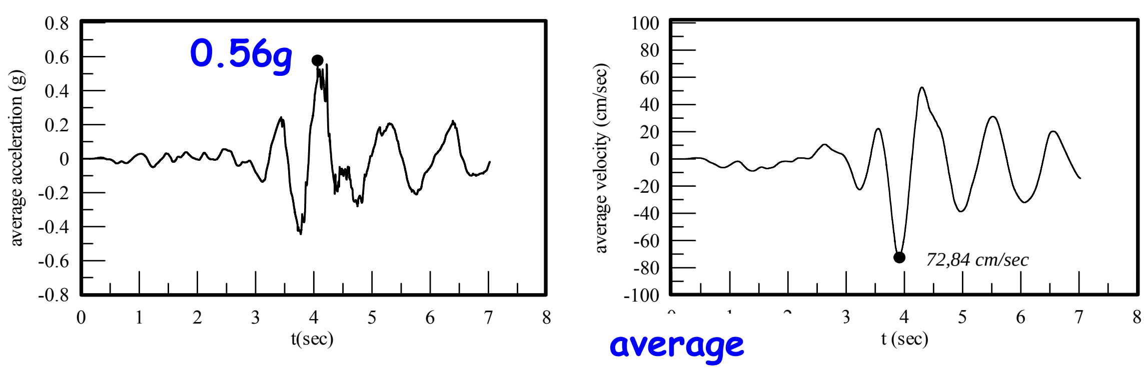


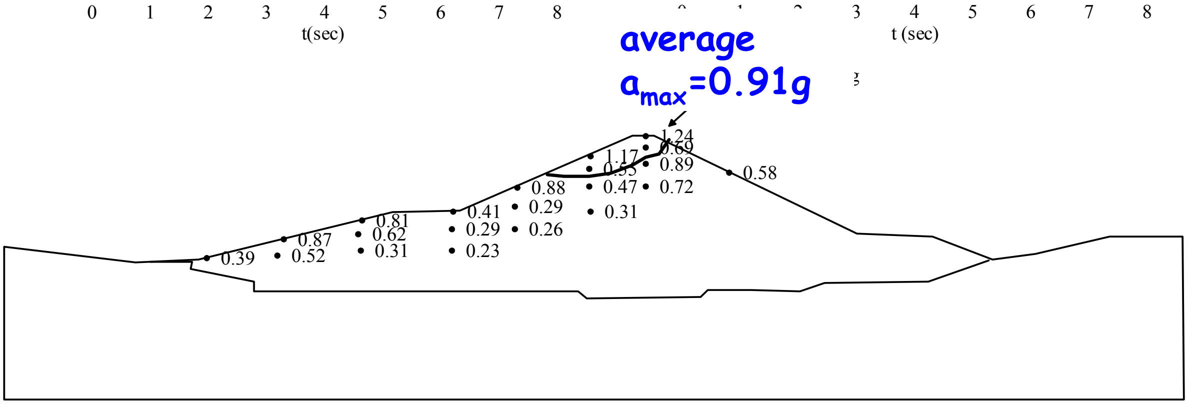
Shallow failure surface . . .

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



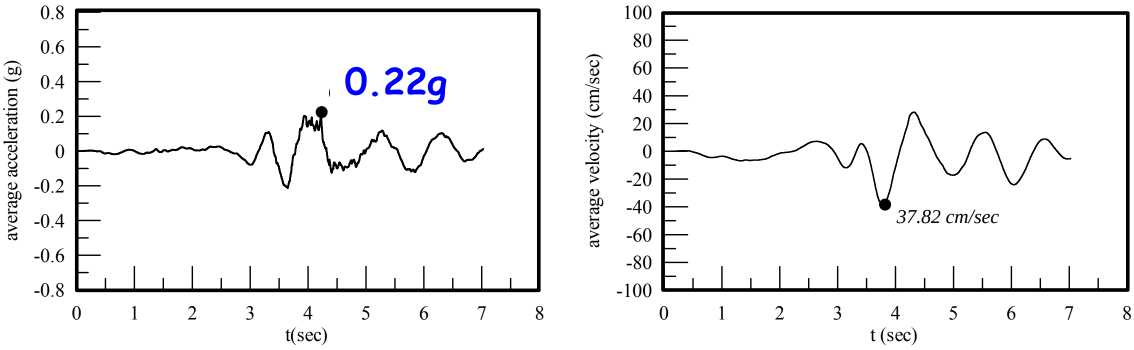


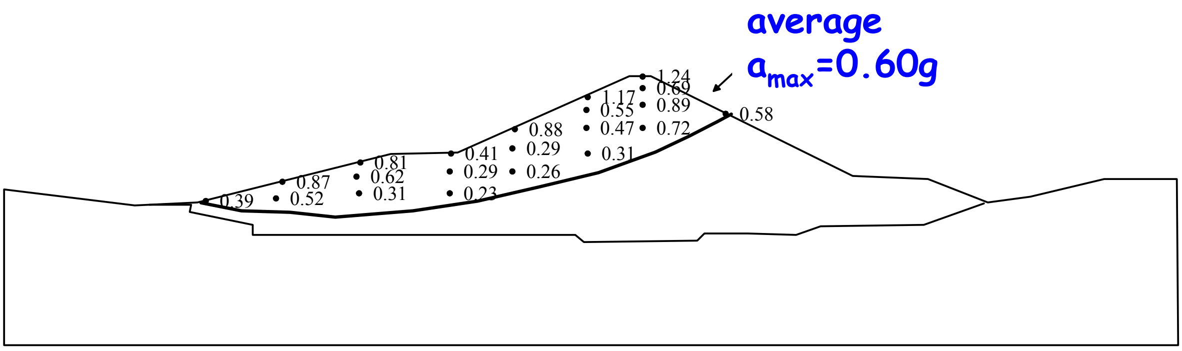
Deep failure surface . . .

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

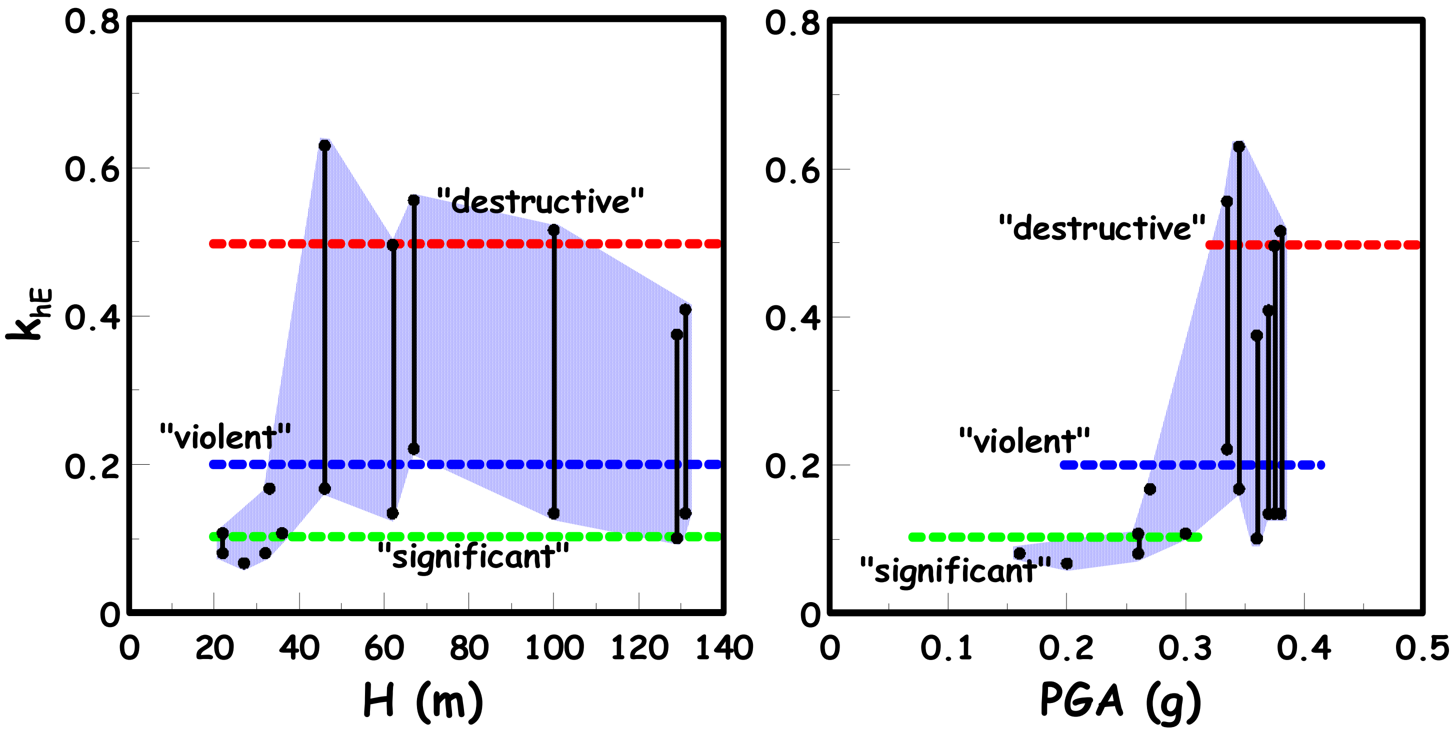




Ad-hoc values of k_{hE} based on ENGINEERING EXPERIENCE

- TERZAGHI (1950)

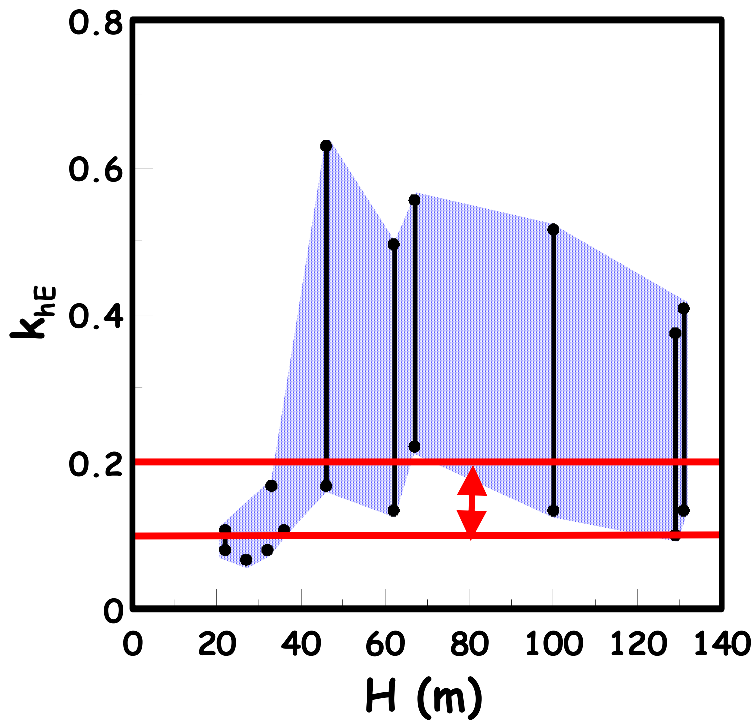
k_{hE} = { 0.10 "significant" earthquakes; 0.20 "violent" earthquakes; 0.50 "destructive" earthquakes }

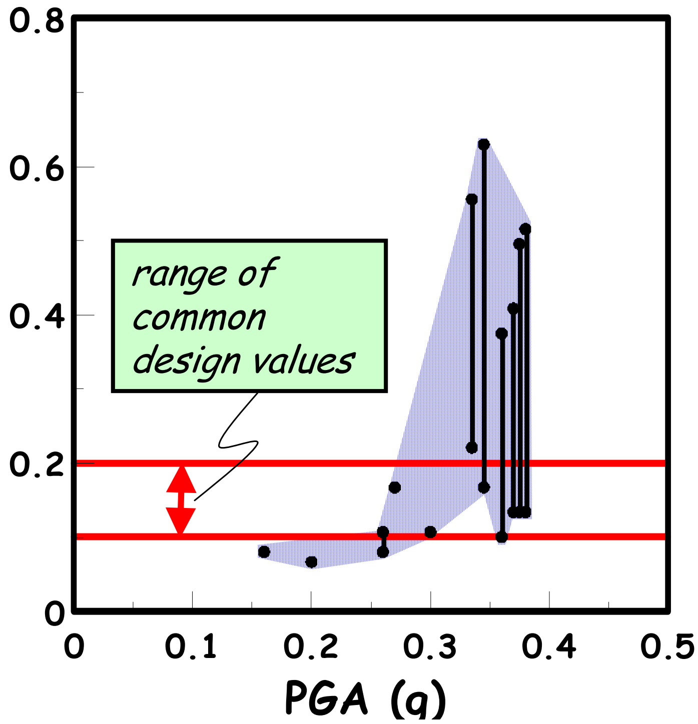


STANDARD PRACTICE of the 70's

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

$FS_d \geq$ 1.00 ÷ 1.15

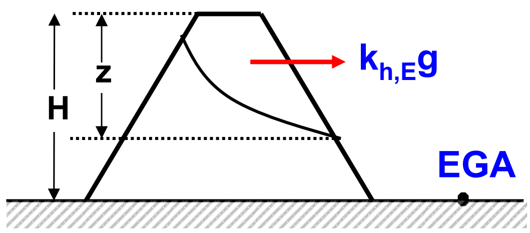


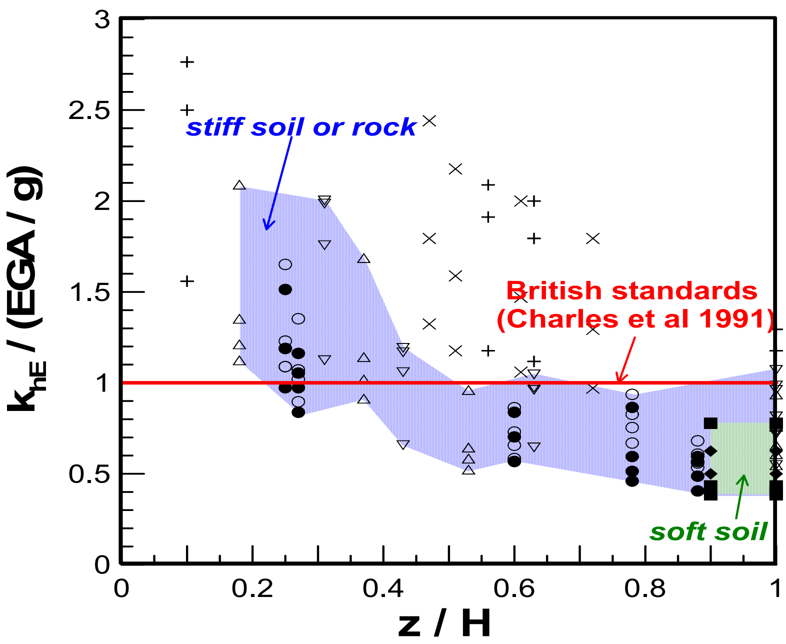


Correlation of k_{hE} with the EGA (effective FREE FIELD acceleration)

BRITISH STANDARDS (Charles et al 1991)

$k_{hE} \approx EGA/g$

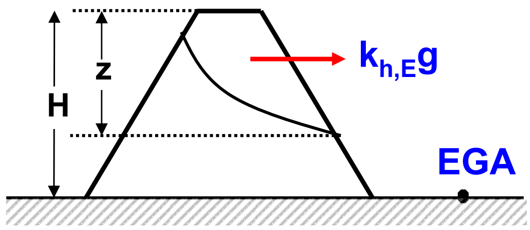




EUROCODE EC-8

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

S = Soil Factor

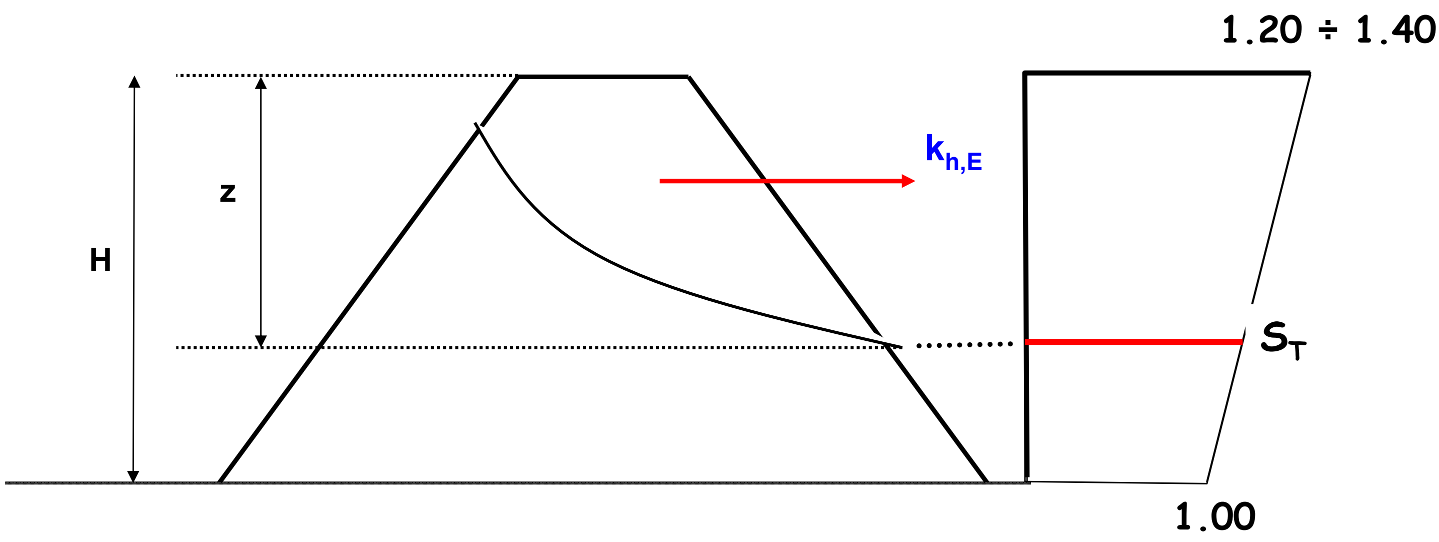


Ground Type	V_S (m/s)	N_{SPT}	C_U (kPa)	S	
				M≤5.5	M>5.5
A	> 800	-	-	1.00	1.00
B	360-800	> 50	> 250	1.35	1.20
C	180-360	15 - 50	70 - 250	1.50	1.20
D	< 180	< 15	< 70	1.80	1.35
E	SHALLOW C or D	< 50	< 250	1.60	1.40

EUROCODE EC-8

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

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



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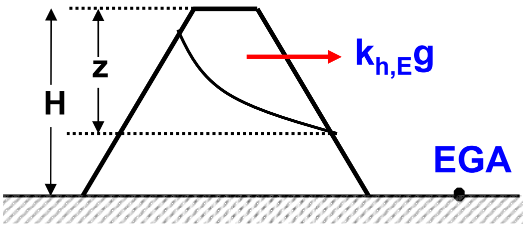
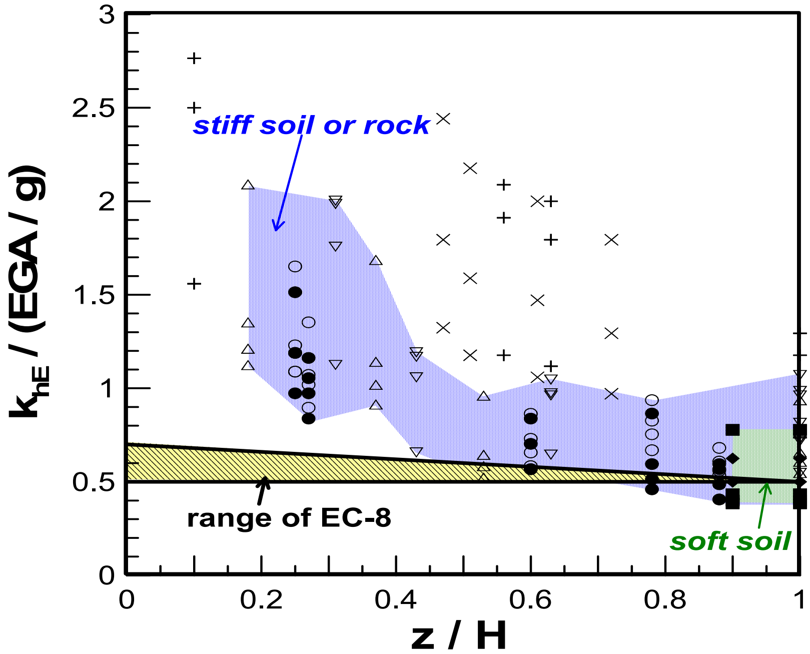
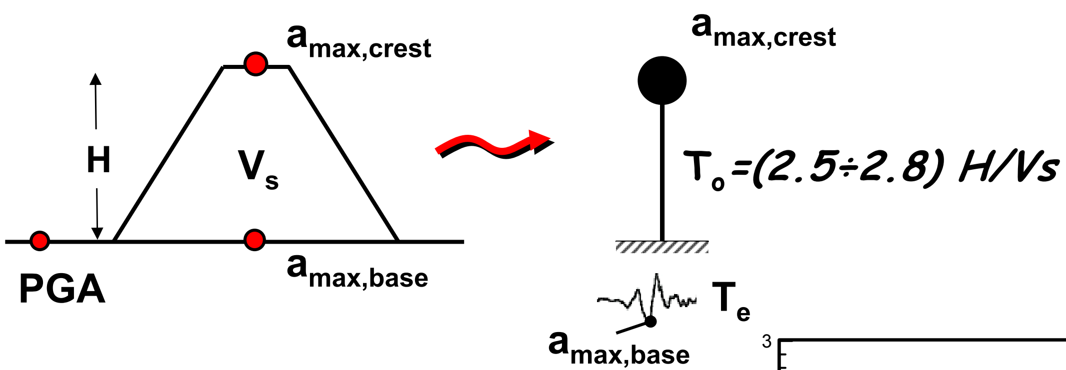
$$k_{hE} = 0.50\ S\ S_T\ (EGA/g)$$



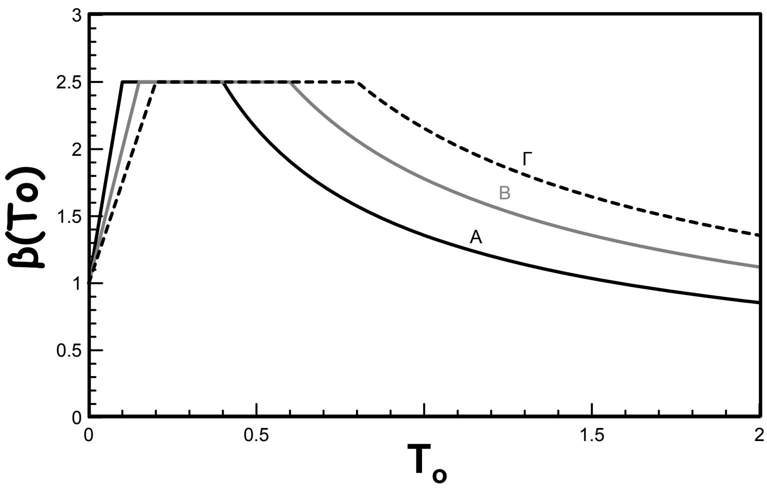


GREEK NATIONAL SEISMIC CODE EAK 2000



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

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



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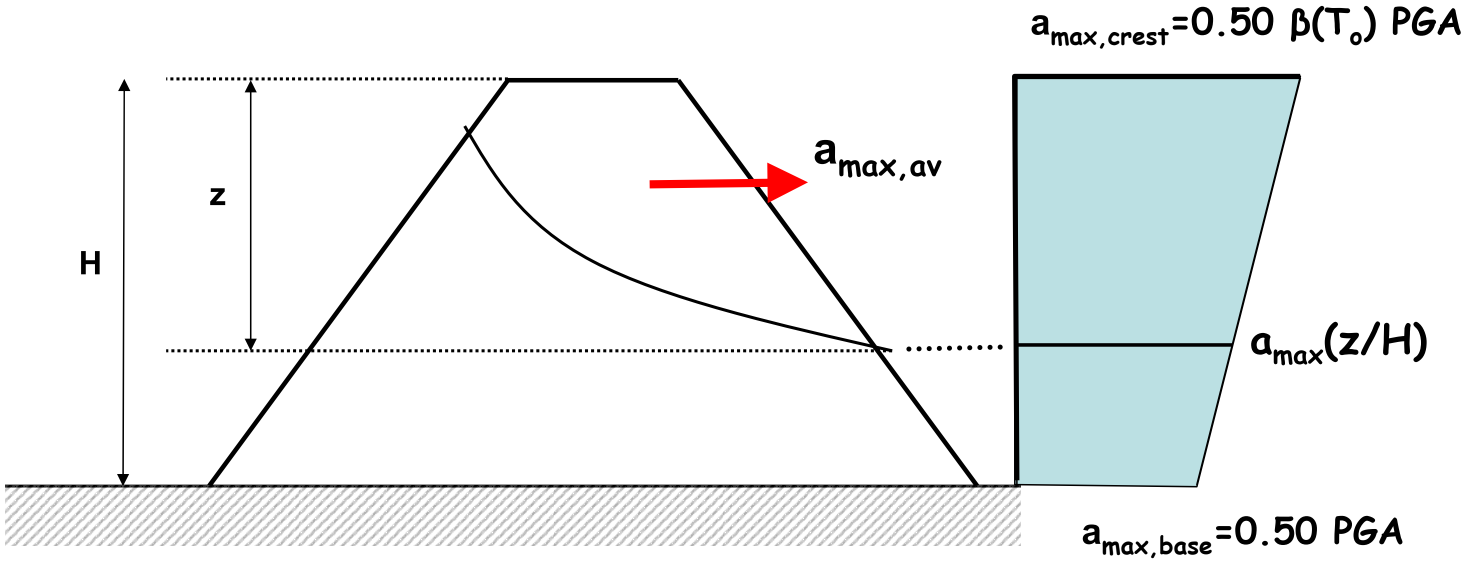


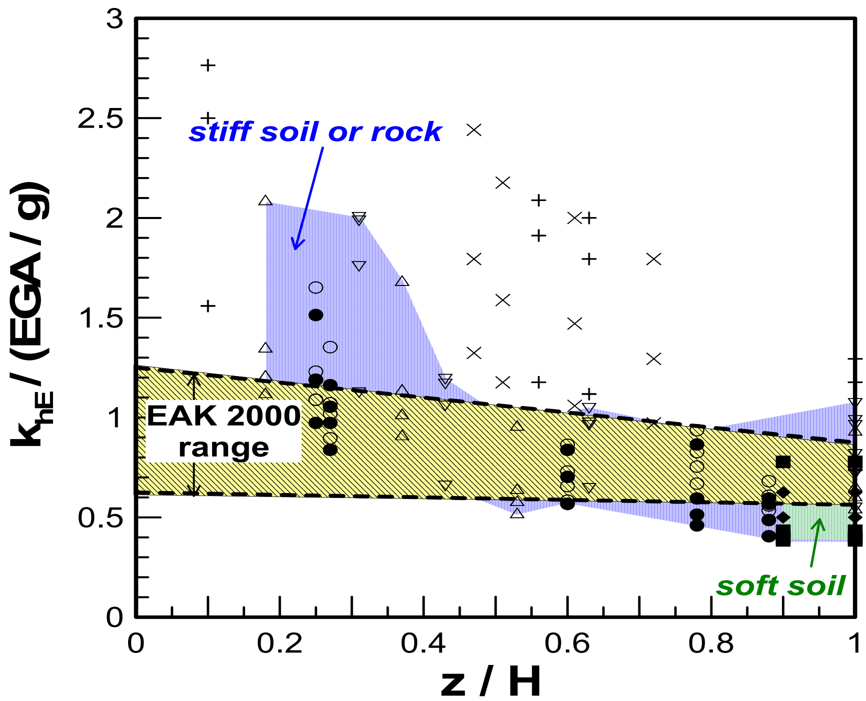
$$a_{max,av} = \frac{a_{max,crest} + a_{max}(z/H)}{2}$$

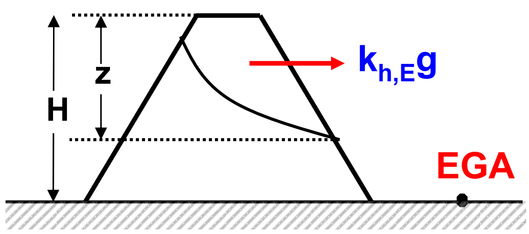
$$k_{hE} = 0.50 \cdot [(0.50 \div 0.80) \cdot a_{max,av} / g]$$

or

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

GREEK NATIONAL SEISMIC CODE EAK 2000





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

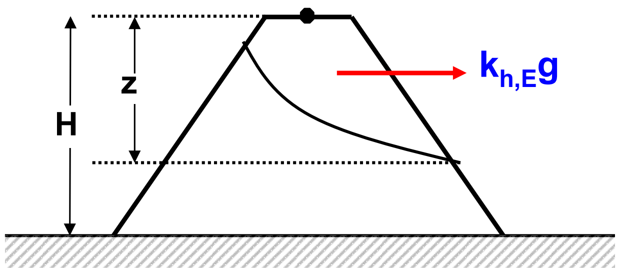
MARCUSON (1981)

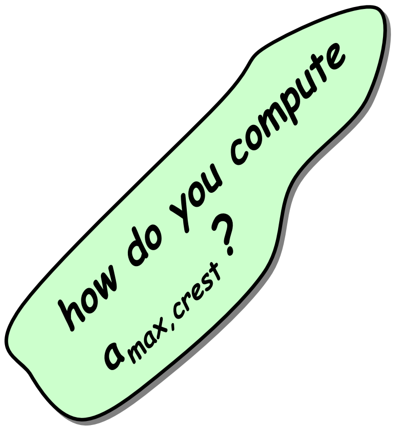
$$k_{hE} = 0.33 \div 0.50 \,(a_{max,crest}/g)$$

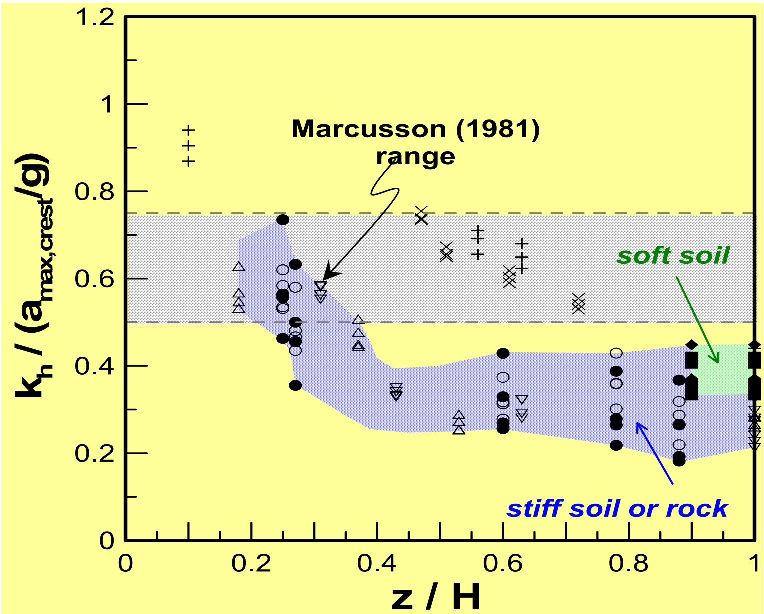
and

$$k_h = 0.50 \div 0.75 \,(a_{max,crest}/g)$$

$(k_h \approx 1.50\, k_{hE})$



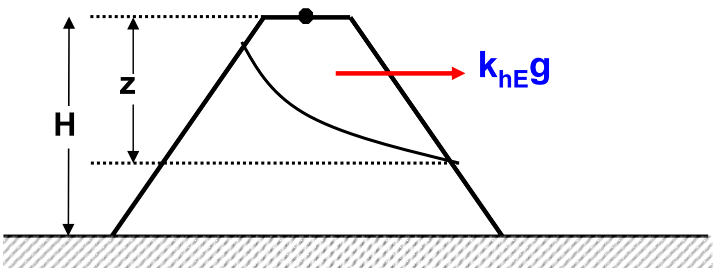


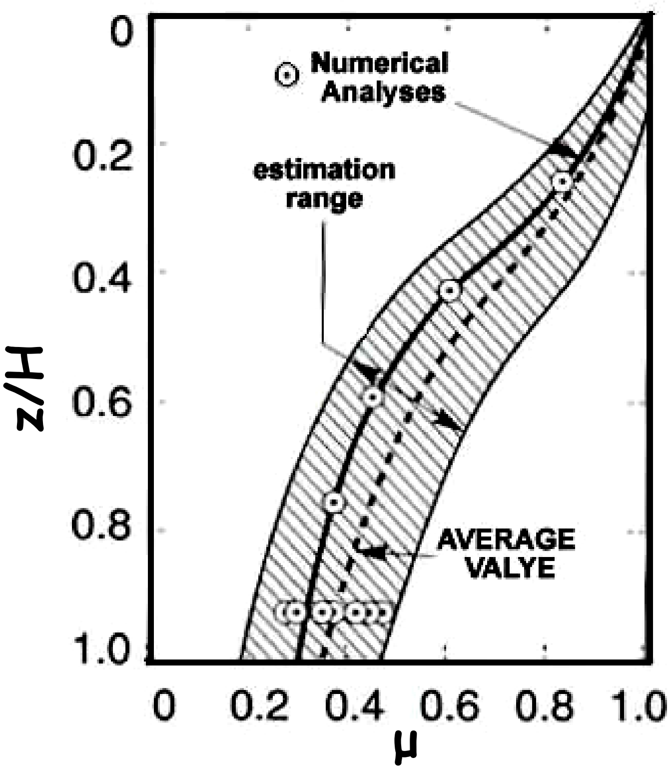


MAKDISI & SEED (1978)

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

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





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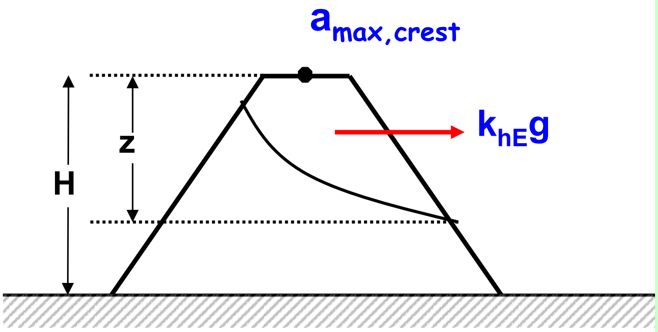
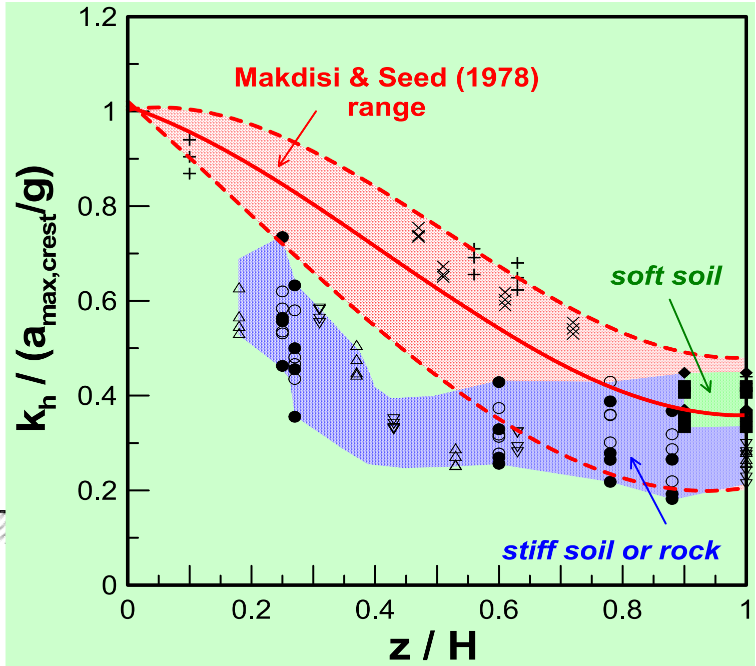
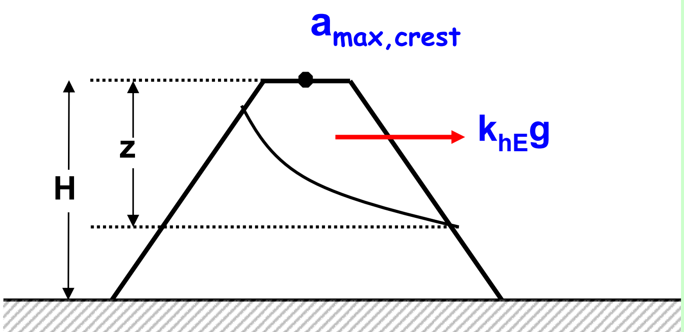
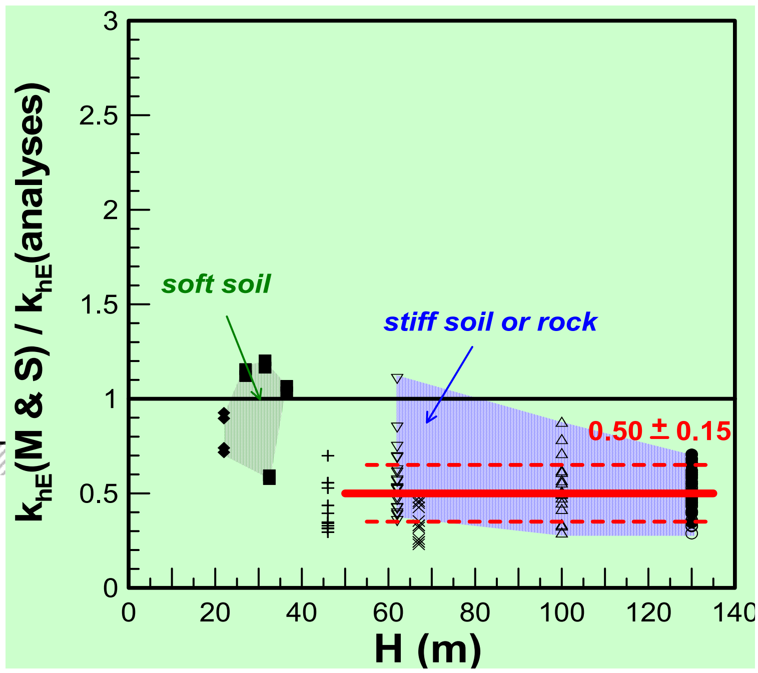




how do you compute $a_{max,crest}$?

e.g. from EAK 2000





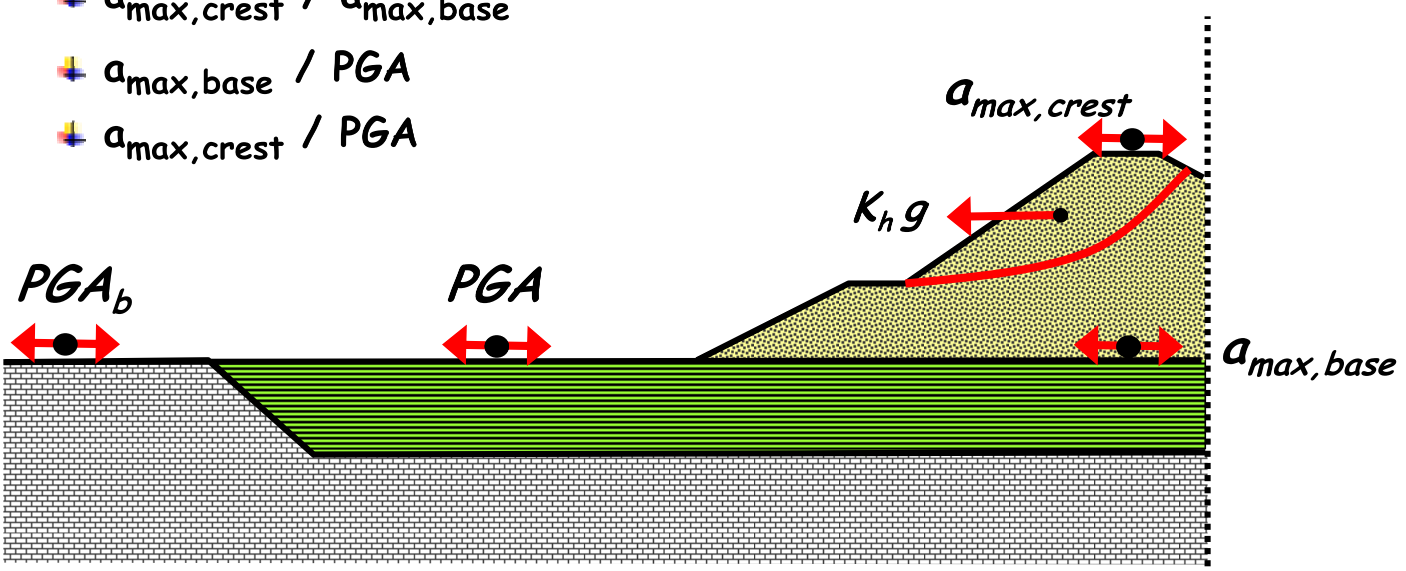
8.3 A New Integrated Approach
(Papadimitriou & Bouckovalas, 2007)

- Parametric Analysis of NUMERICAL RESULTS
- step-by-step METHODOLOGY OUTLINE
- EVALUATION OF PROPOSED METHODOLOGY
  error margins
- EXAMPLE APPLICATION
  the case of Ilarion dam in Northern Greece

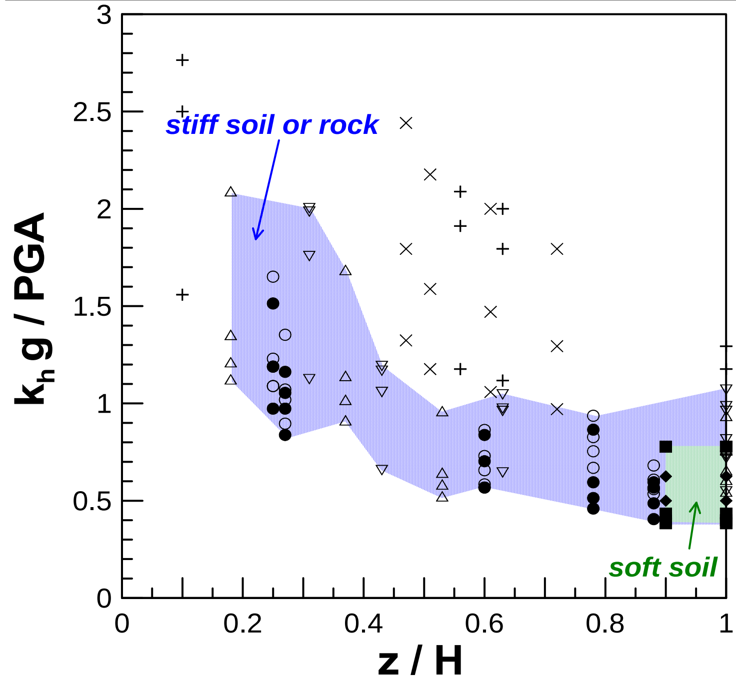
Towards to a new integrated approach . . . .

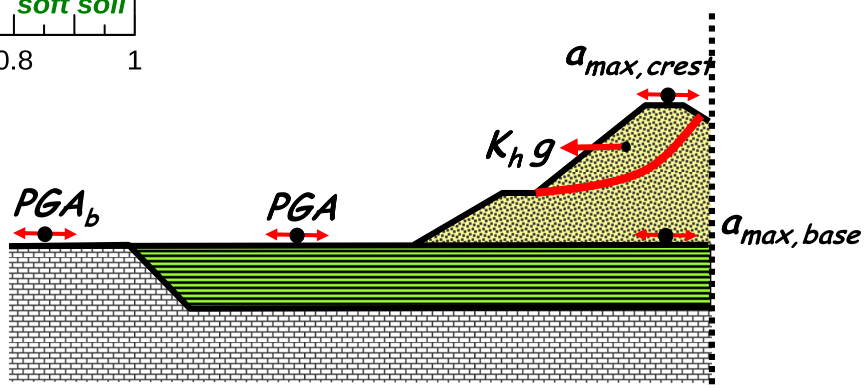
To refine k_h & k_{hE} predictions, we used the results from the numerical analyses to identify the factors affecting:

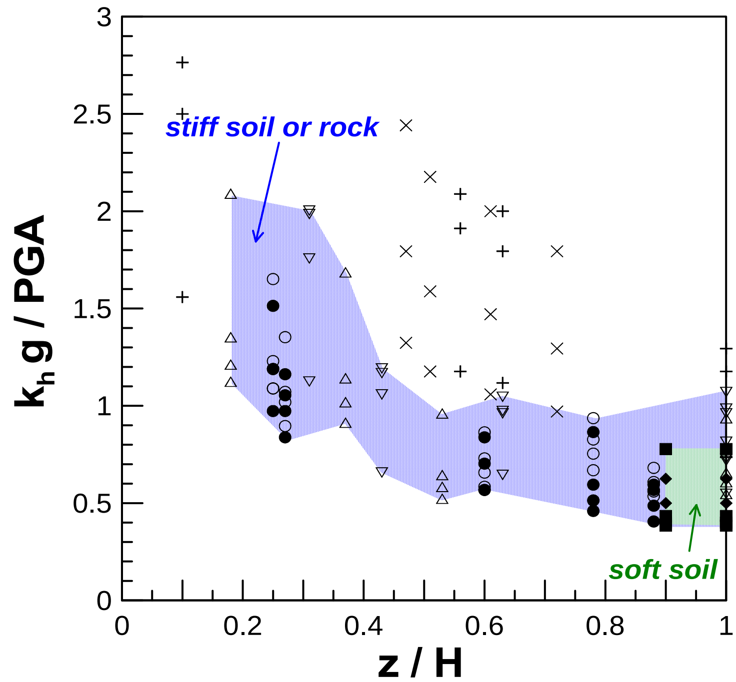
- k_h / PGA
- k_h / $a_{max,crest}$
- $a_{max,crest}$ / $a_{max,base}$
- $a_{max,base}$ / PGA
- $a_{max,crest}$ / PGA

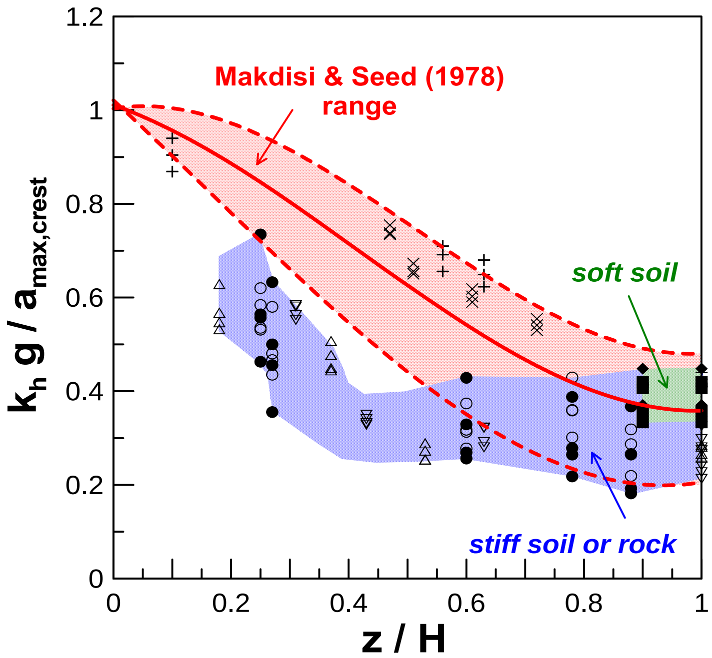


Factors affecting k_h

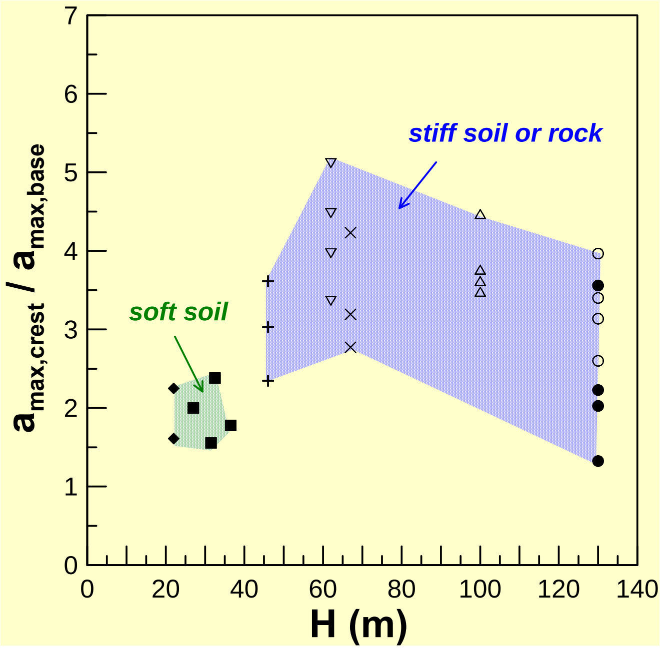


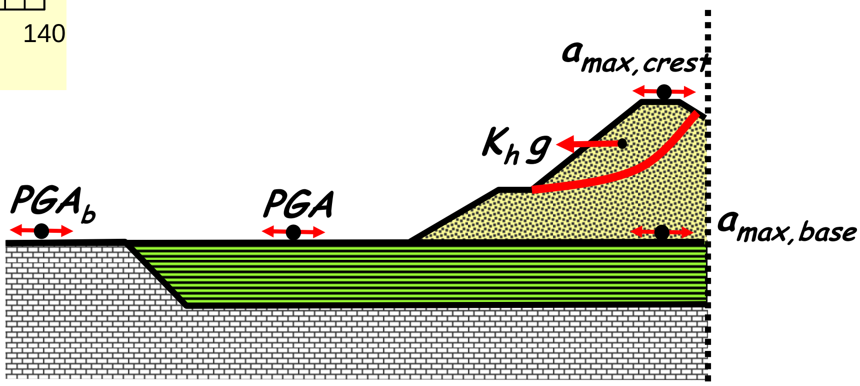


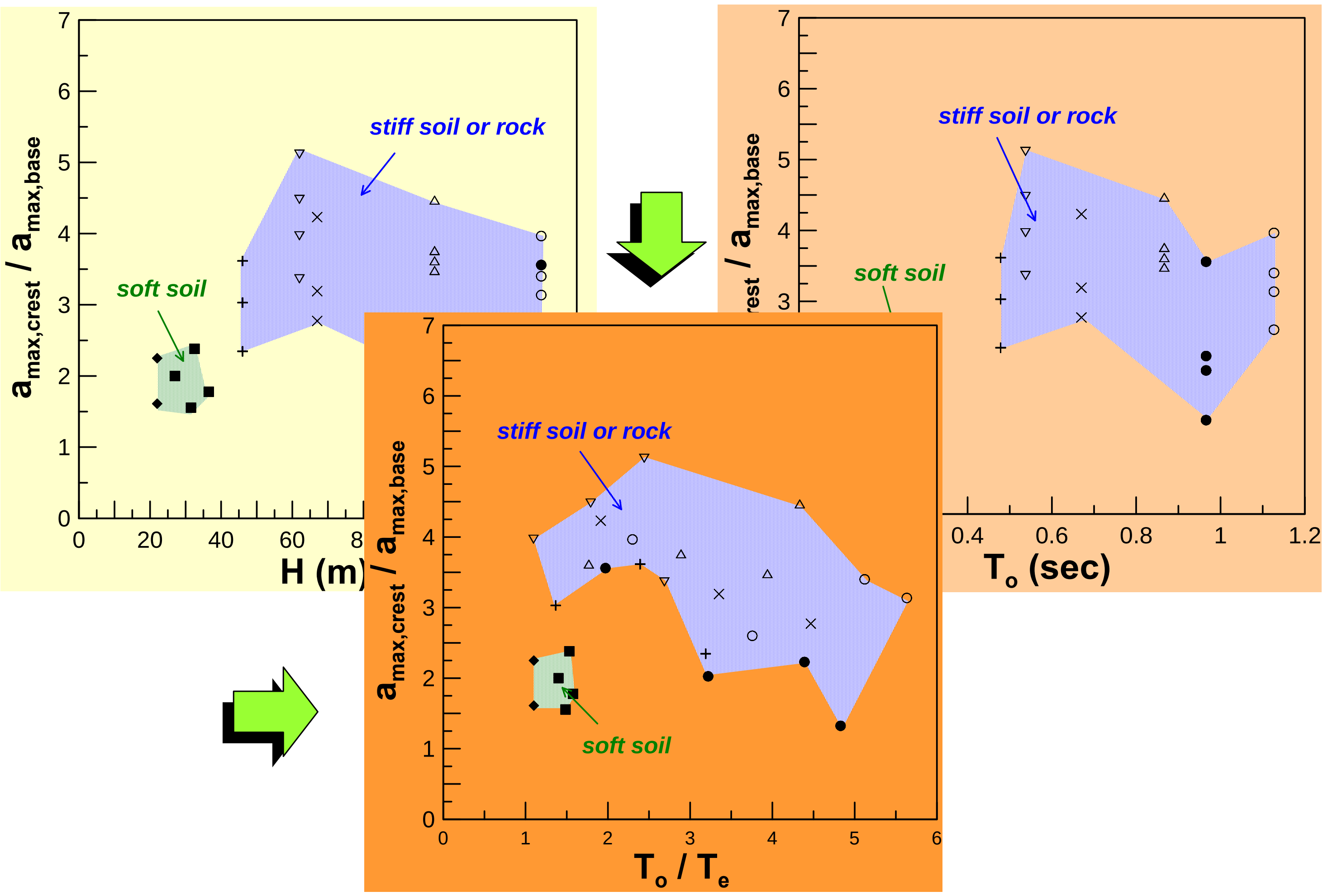




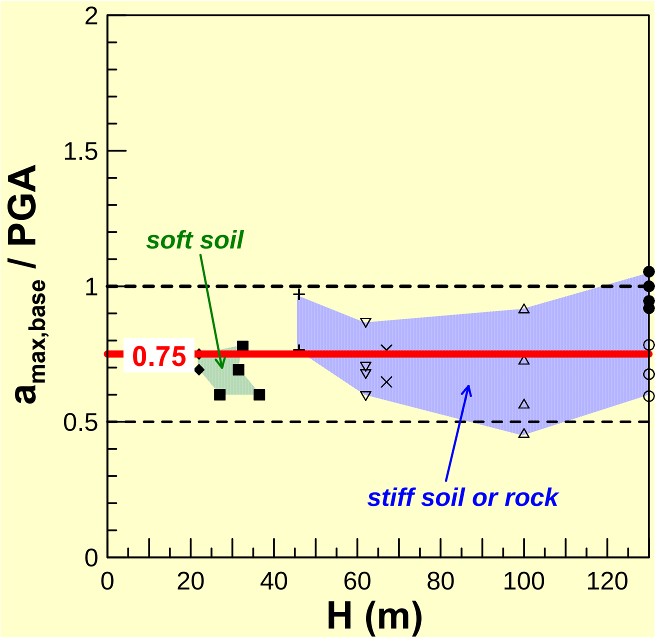
Factors affecting $a_{max,crest}$ / $a_{max,base}$

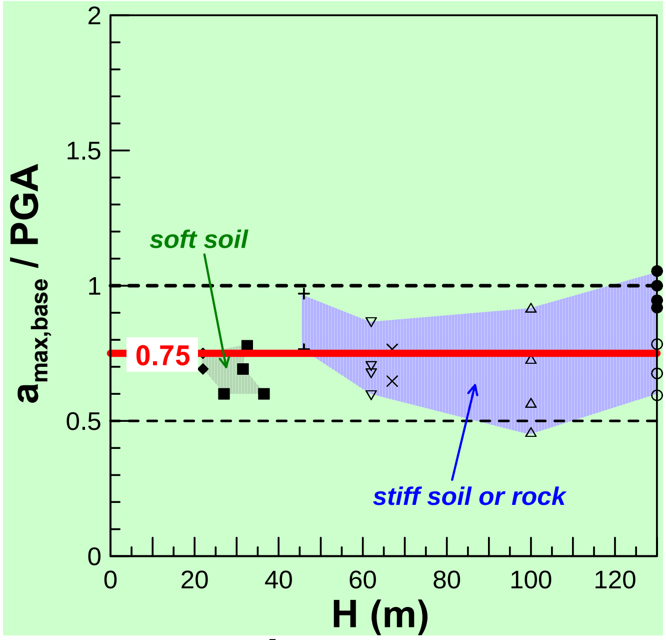


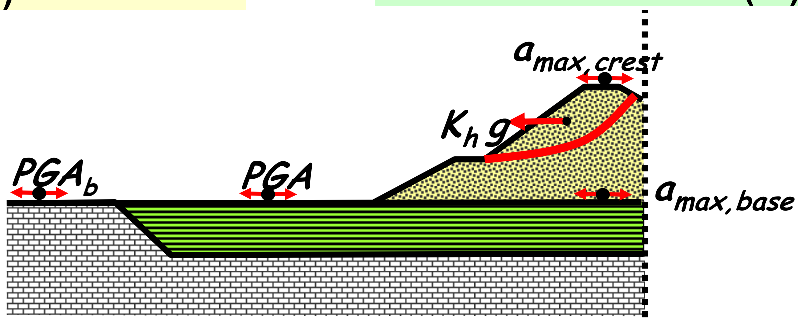


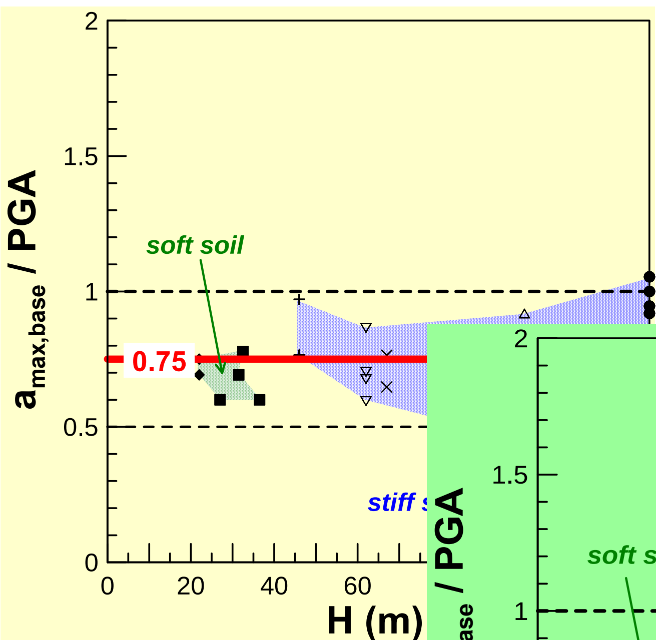


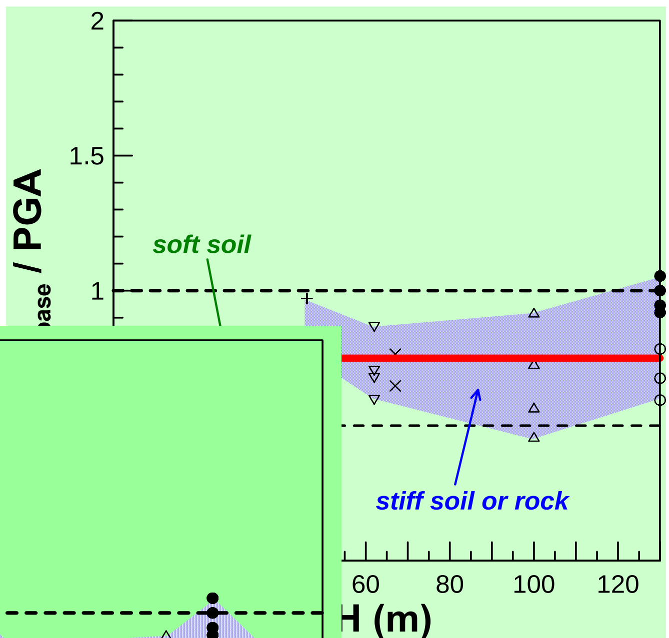
Factors affecting $a_{max,base}$ / PGA

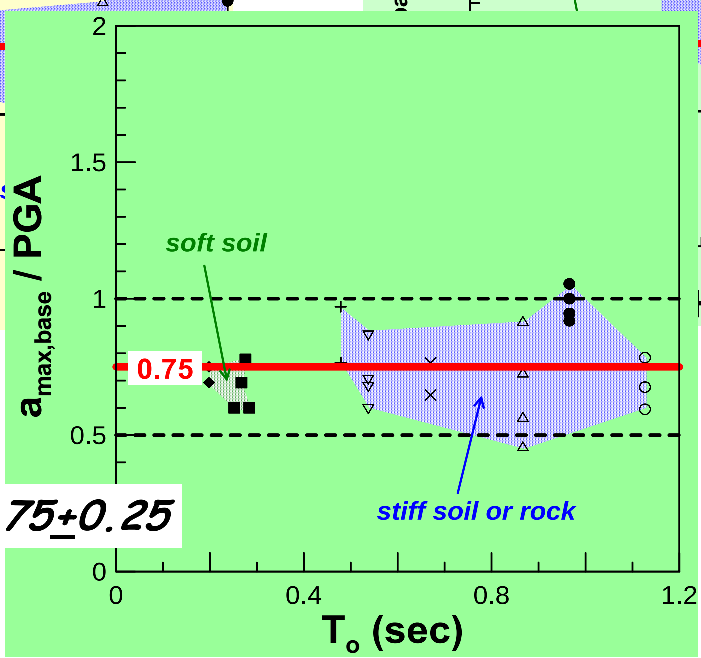






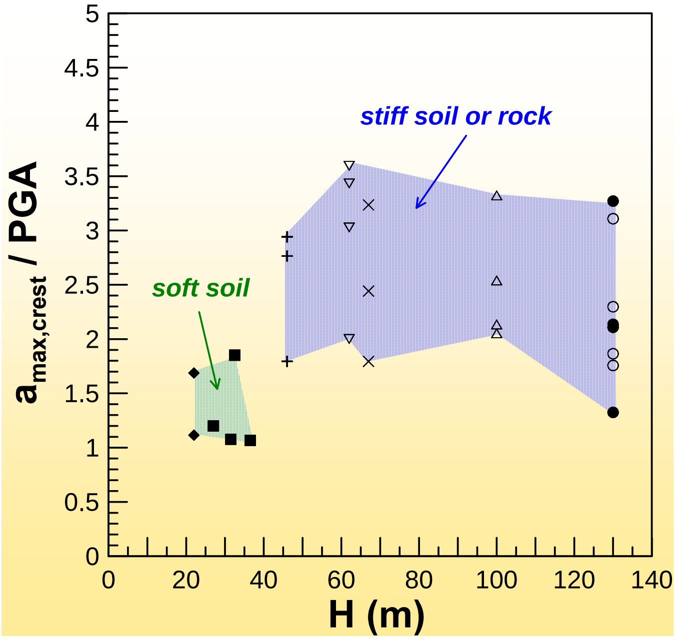


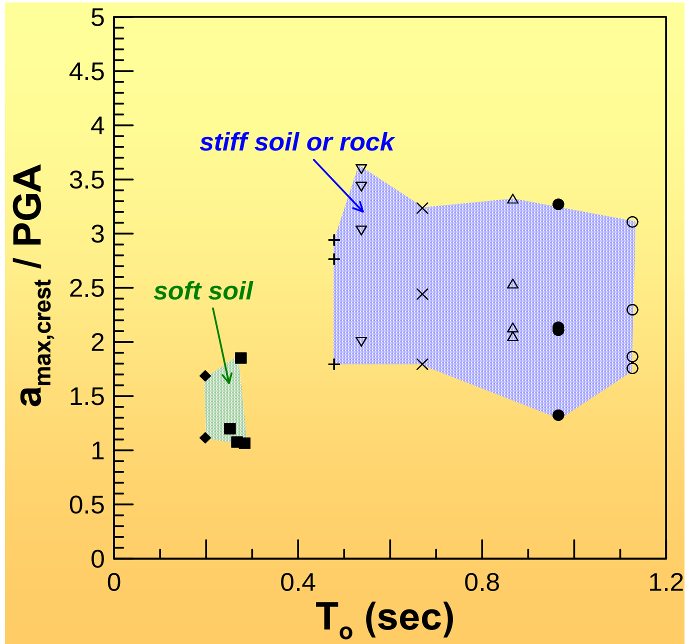


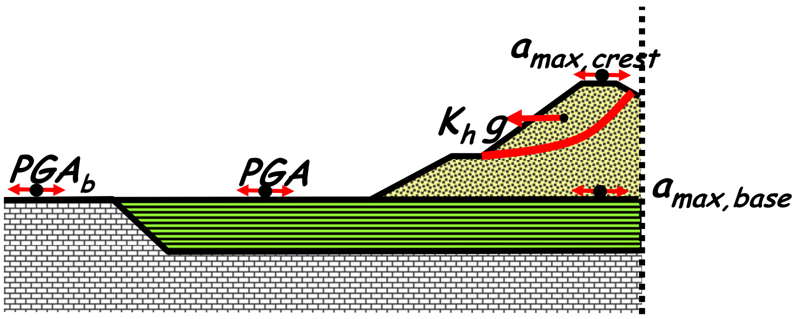


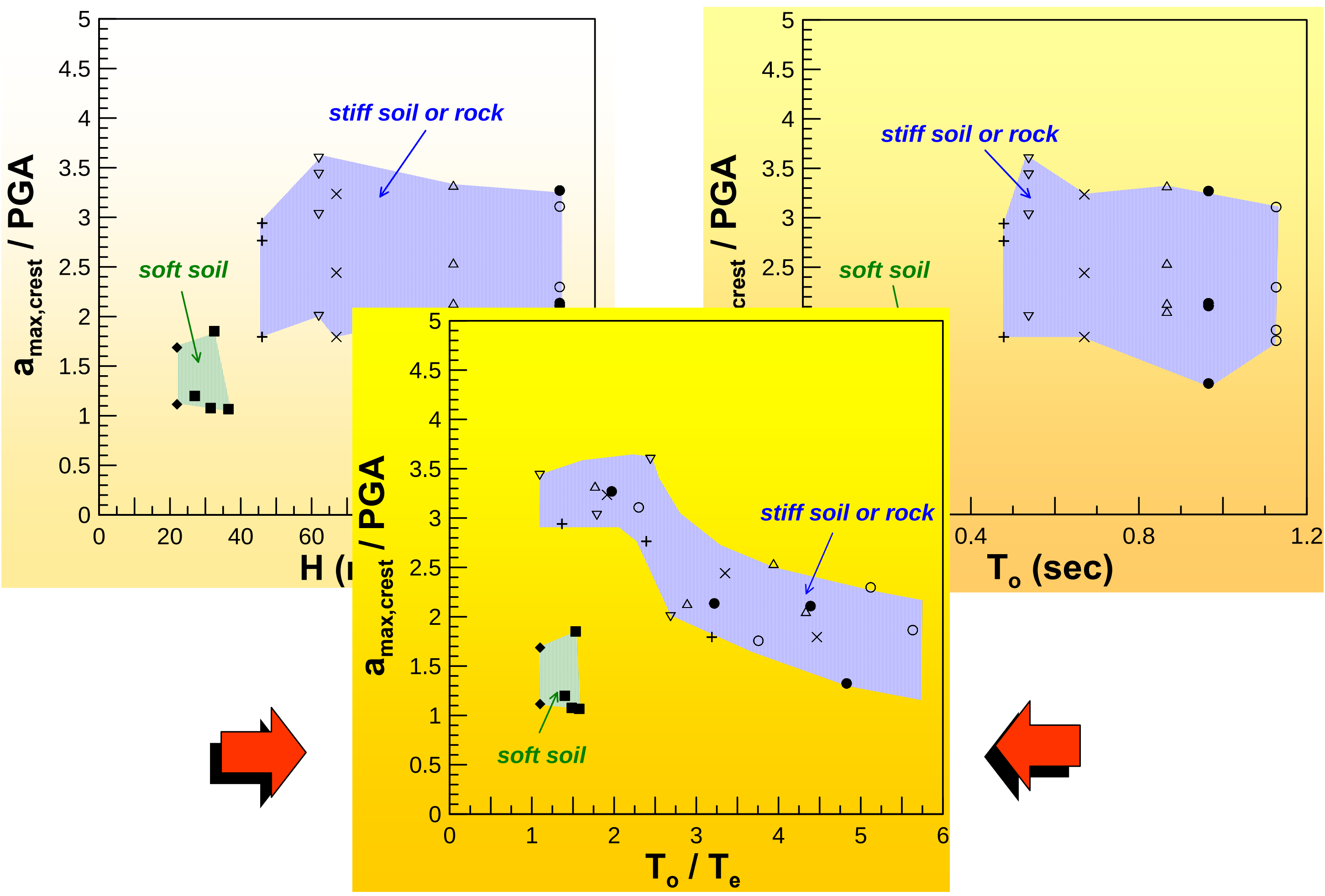
$a_{max,base}$ / PGA= 0.75 ± 0.25

Factors affecting $a_{max,crest}$ / PGA





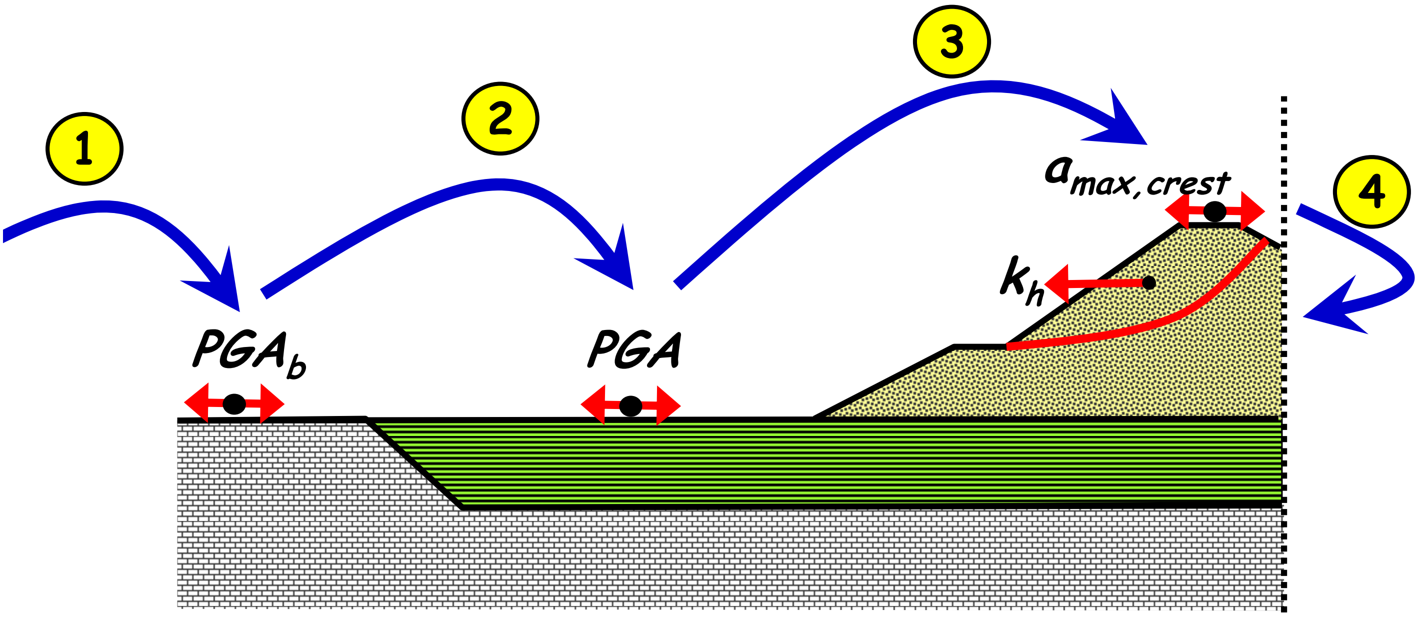




step-by-step

METHODOLOGY OUTLINE

- **STEP 1:** Define PGA_b and predominant shaking period T_e
- **STEP 2:** Compute PGA
- **STEP 3:** Compute predominant dam period T_o and $a_{max,crest}$
- **STEP 4:** Compute k_h and $k_{h,E}$

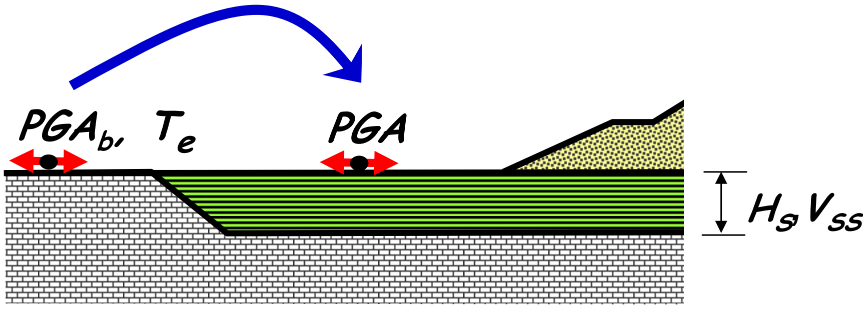


STEP 2: *Compute PGA . . . .*

NUMERICALLY (SHAKE, etc.)

OR

APPROXIMATELLY . . . .

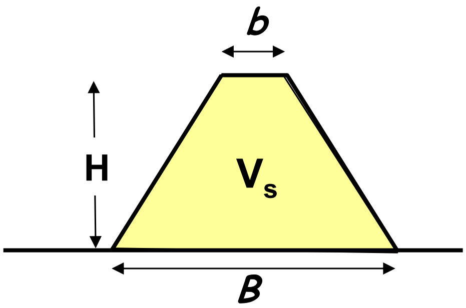


$$PGA = PGA_b \frac{1 + 0.85\left(\frac{PGA_b}{g}\right)^{-0.17}\left(\frac{T_s}{T_e}\right)^2}{\sqrt{\left(1-\left(\frac{T_s}{T_e}\right)^2\right)^2 + 1.78\left(\frac{T_s}{T_e}\right)^2}}$$

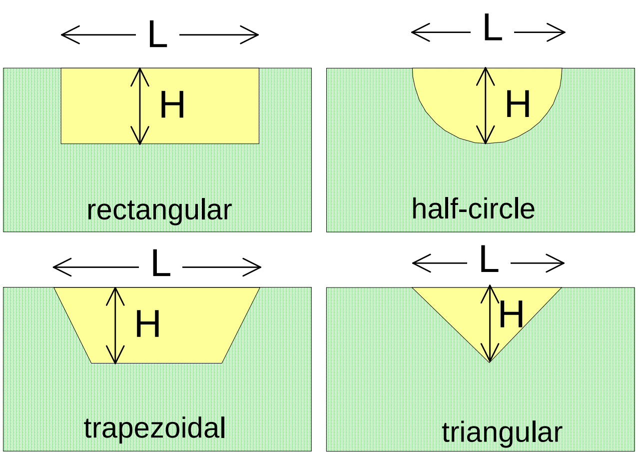
$$T_s = \left(\frac{4H_s}{V_{ss}}\right)\sqrt{1 + 5330\ V_{ss}^{-1.3}\left(\frac{PGA_b}{g}\right)^{1.04}}$$

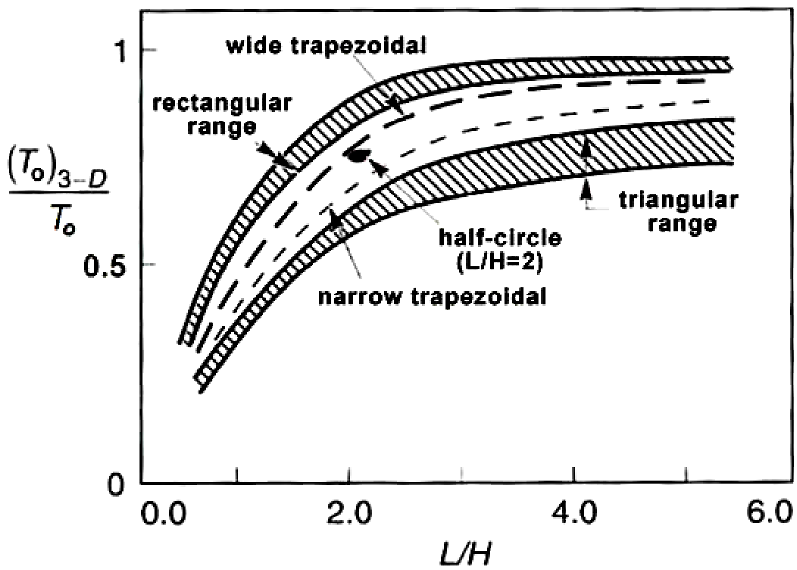
STEP 3: *Compute T_O & $a_{max,crest}$*

(A) FUNDAMENTAL DAM PERIOD OF VIBRATION (T_o) & $(T_o)_{3-D}$



$$T_o = (2.6 + 2r)\frac{H}{V_S}, \qquad r = \frac{b}{B} \leq 0.05$$





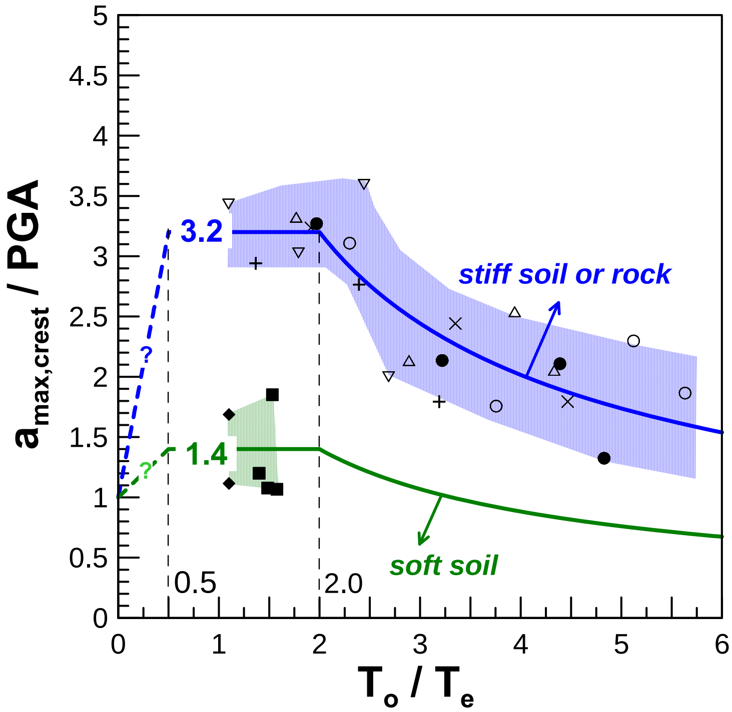
(B) PEAK SEISMIC ACCELERATION AT CREST $a_{max,crest}$

STIFF foundation soil or rock

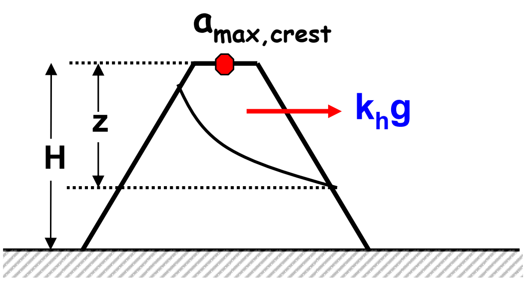
$$\frac{a_{max,crest}}{PGA} = \begin{cases} 1 + 4.4\left(\frac{T_o}{T_e}\right) & , \quad 0 \le \frac{T_o}{T_e} \le 0.5 \\ 3.2 & , \quad 0.5 \le \frac{T_o}{T_e} \le 2.0 \\ 3.2\left(\frac{2T_e}{T_o}\right)^{2/3} & , \quad 2.0 \le \frac{T_o}{T_e} \end{cases}$$

SOFT foundation soil

$$\frac{a_{max,crest}}{PGA} = \begin{cases} 1 + 0.8\left(\frac{T_o}{T_e}\right) & , \quad 0 \le \frac{T_o}{T_e} \le 0.5 \\ 1.4 & , \quad 0.5 \le \frac{T_o}{T_e} \le 2.0 \\ 1.4\left(\frac{2T_e}{T_o}\right)^{2/3} & , \quad 2.0 \le \frac{T_o}{T_e} \end{cases}$$



STEP 4: *Compute Seismic Coefficients k_h & $k_{h,E}$*

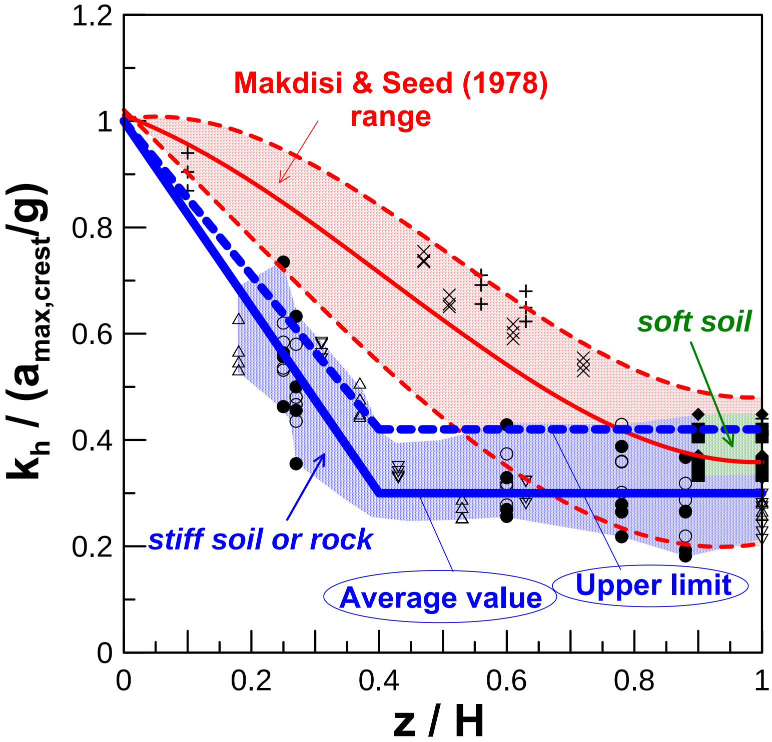


AVERAGE estimates

$$\frac{k_h \cdot g}{a_{max,crest}} = \begin{cases} 1 - 1.725\left(\frac{z}{H}\right) & , \quad \left(\frac{z}{H}\right) \le 0.4 \\ 0.31 & , \quad \left(\frac{z}{H}\right) > 0.4 \end{cases}$$

UPPER BOUND estimates

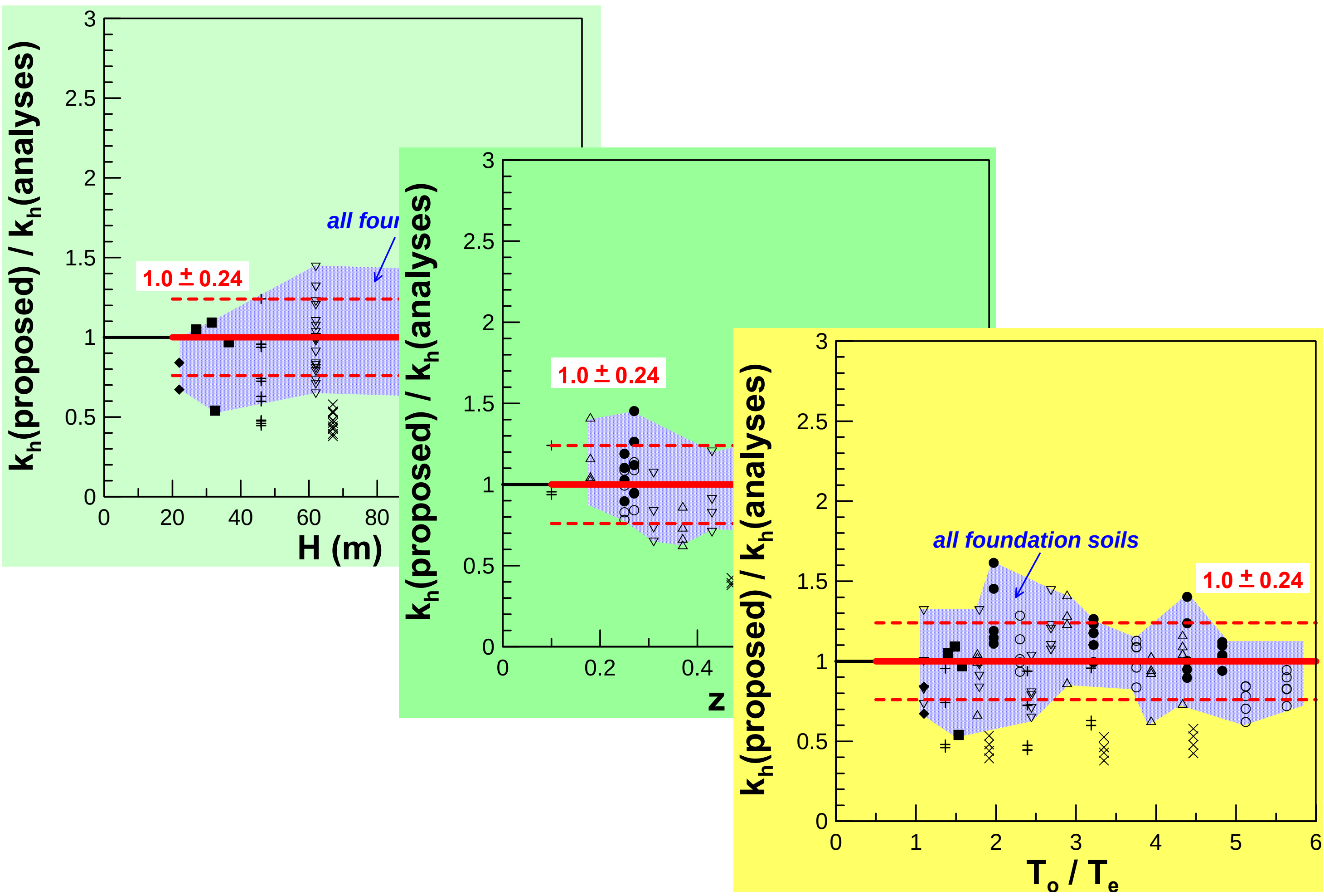
$$\frac{k_h \cdot g}{a_{max,crest}} = \begin{cases} 1 - 1.425\left(\frac{z}{H}\right) & , \quad \left(\frac{z}{H}\right) \le 0.4 \\ 0.43 & , \quad \left(\frac{z}{H}\right) > 0.4 \end{cases}$$



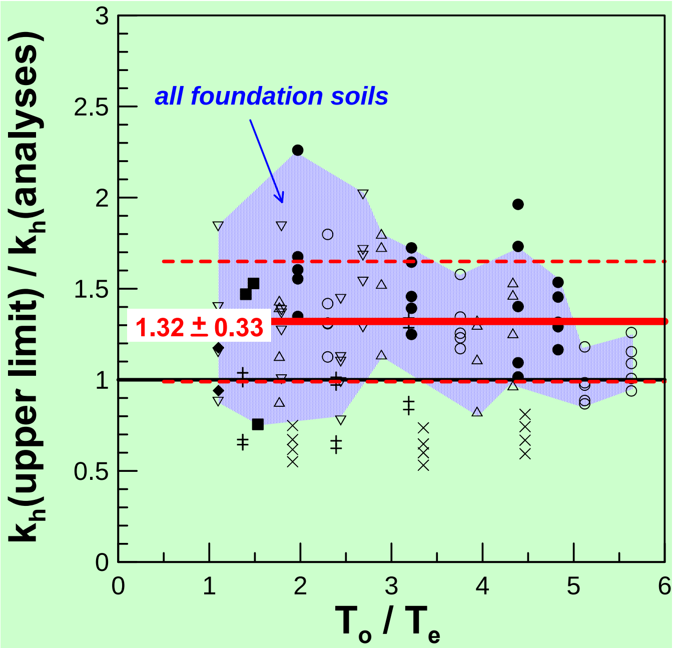
$K_{h,E}$ = (0.50÷0.80) k_h

EVALUATION OF THE NEW METHODOLOGY
error margins

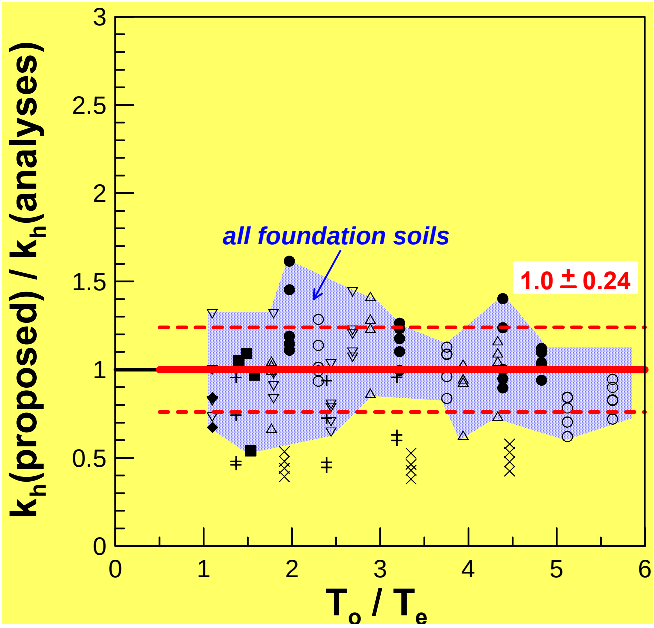
AVERAGE predictions



UPPER BOUND predictions

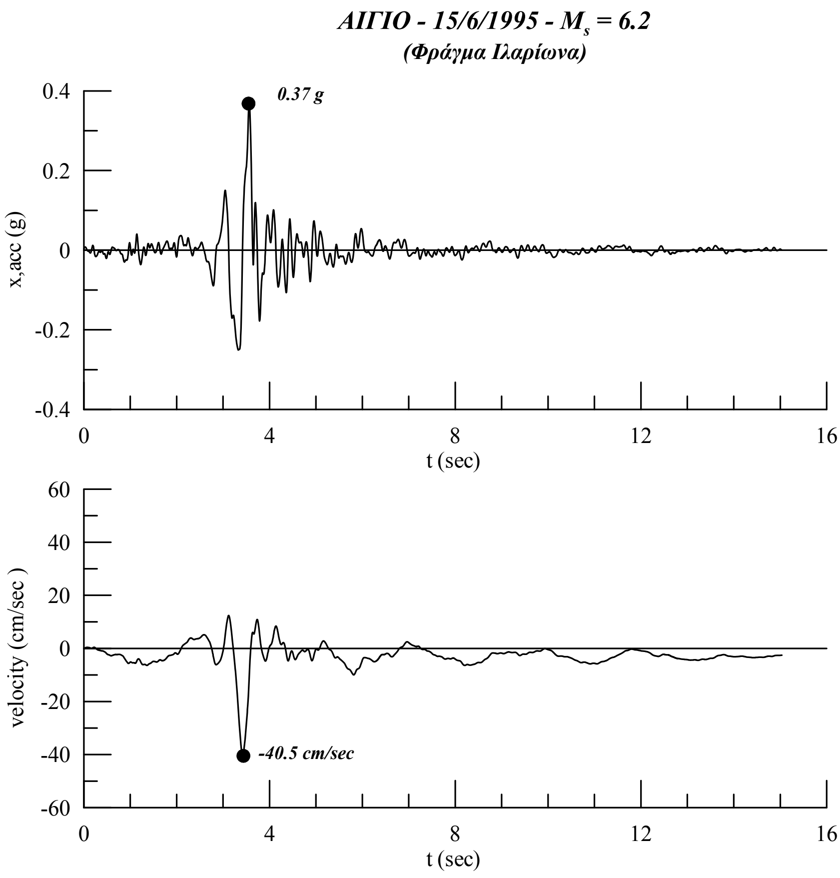


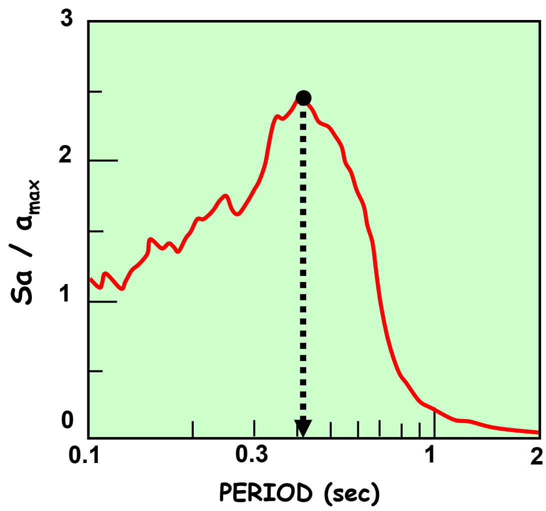
AVERAGE predictions



EXAMPLE APPLICATION
the case of Ilarion dam in Northern Greece

STEP 1: Define PGA_b and predominant shaking period T_e





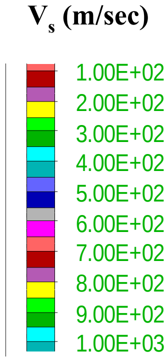
PGA_b=0.37g

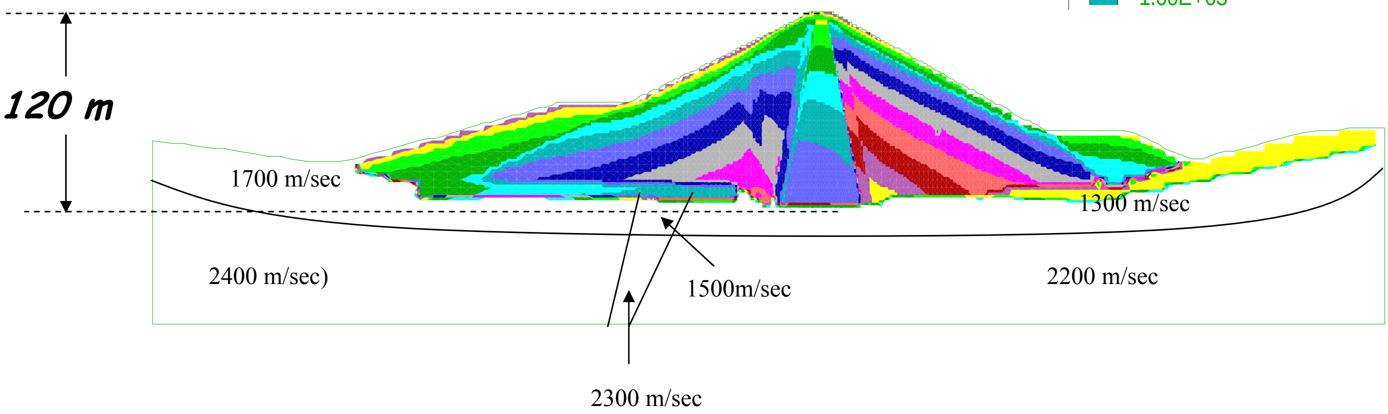
T_e ≈ 0.45s

STEP 2: Compute free field seismic ground acceleration PGA

The dam is constructed on weathered rock formations and consequently:

$PGA=PGA_b=0.37g$





STEP 3(A): Compute fundamental dam period T_O

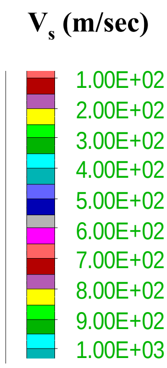
$V_{SD}=300\text{-}400$ m/s

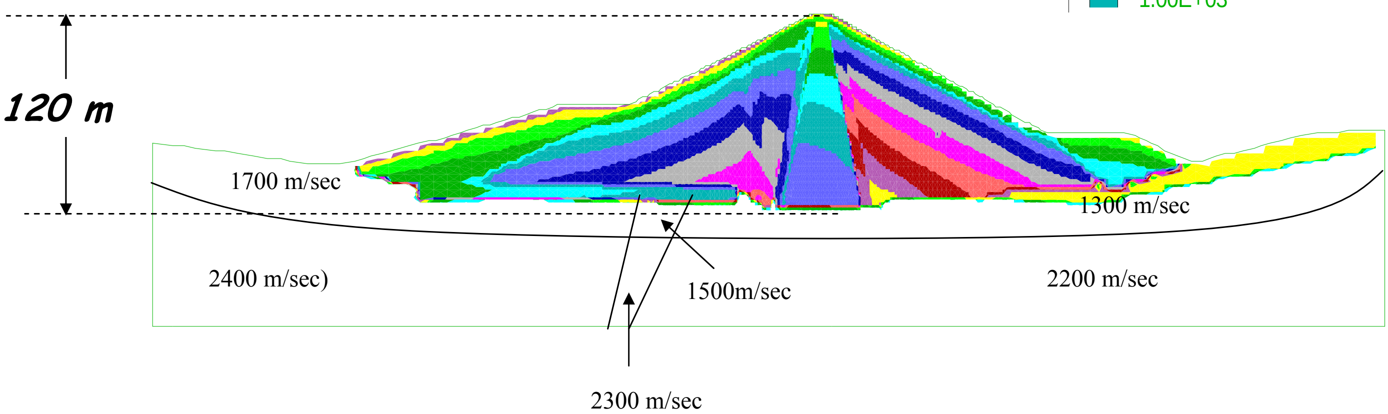
$H_D=120$ m

r=crest/base=0

$T_O=(2.6+2r)\ H_D/V_{SD}=0.78\text{-}1.04$ s

$T_{O,3\text{-}D} = 0.90\ T_O = 0.70\text{–}0.94$ s





STEP 3(B): Compute peak seismic acceleration at crest $a_{max,crest}$

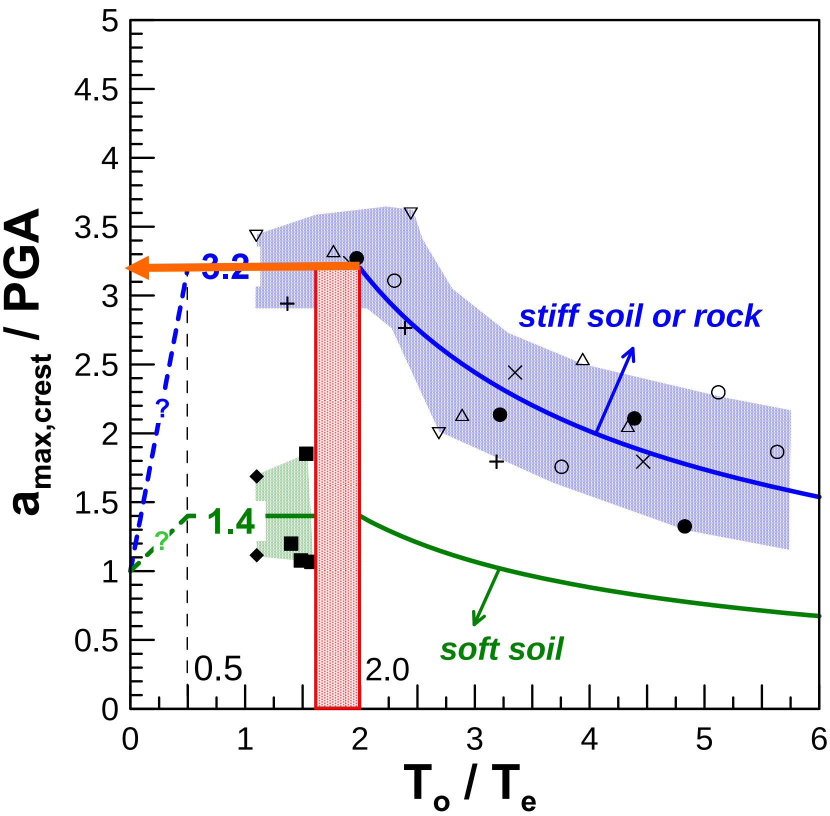
STIFF foundation soil

T_o/T_e = 1.50 ÷ 2.00

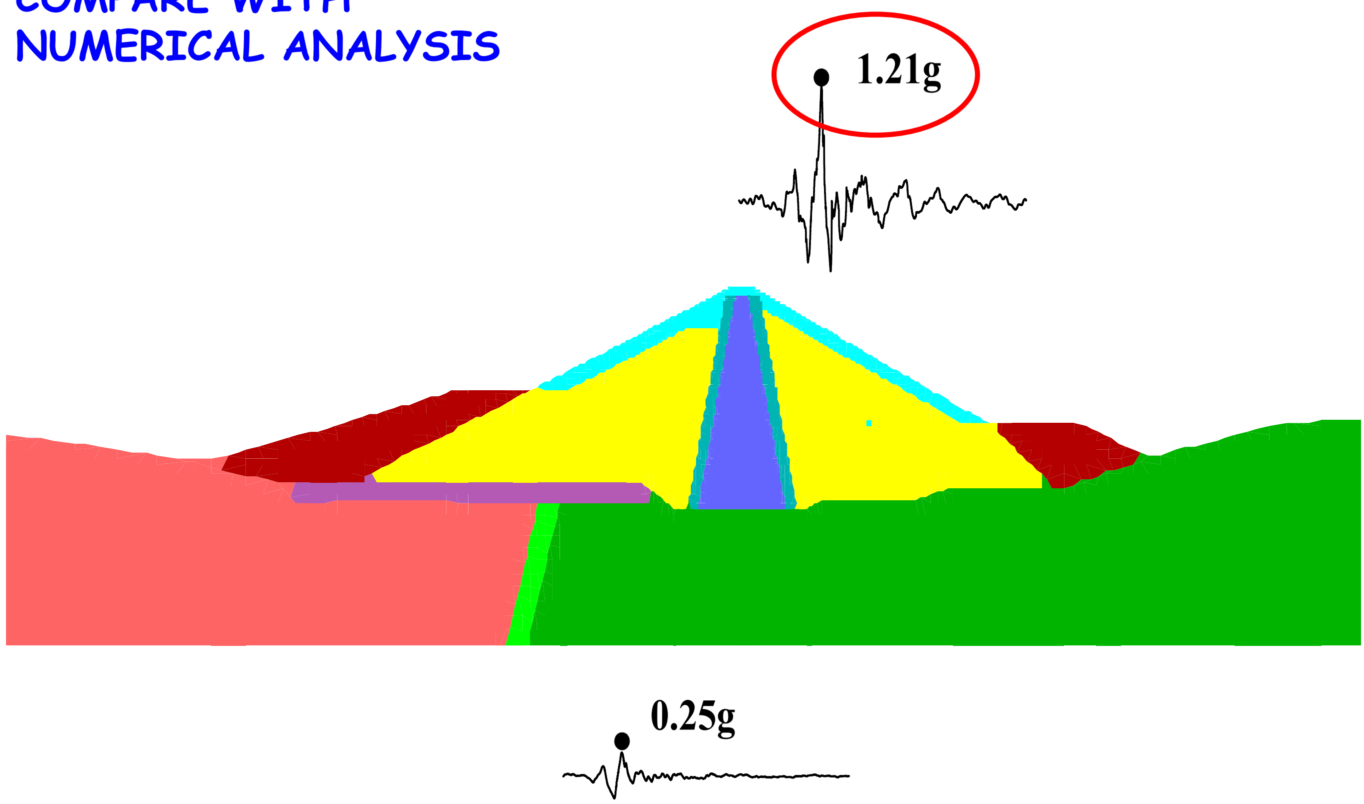
$a_{max,crest}$=3.2 PGA

or

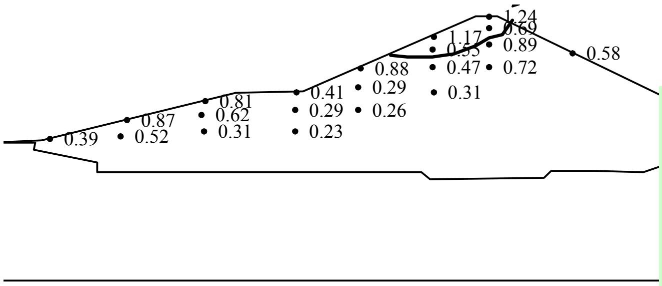
$a_{max,crest}$=3.2x0.37g=1.18g



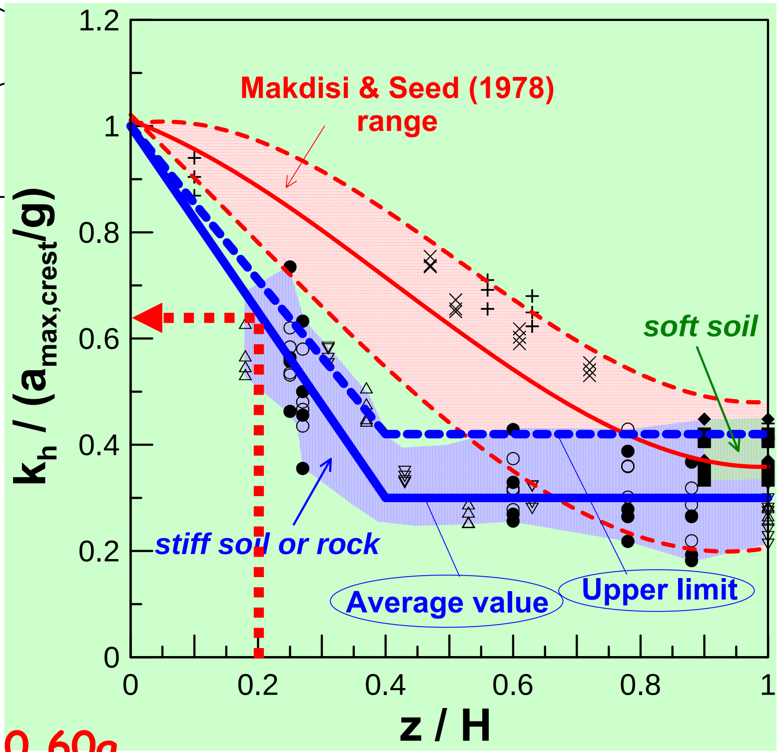
COMPARE WITH NUMERICAL ANALYSIS



STEP 4: Compute seismic coefficients k_h, k_{hE} (Shallow failure)



for z/H = 0.20



K_h = 0.64 $a_{max,crest}$

or

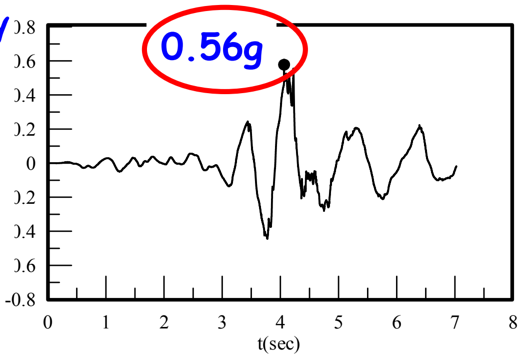
K_h = 0.64 x 1.18g=0.75g

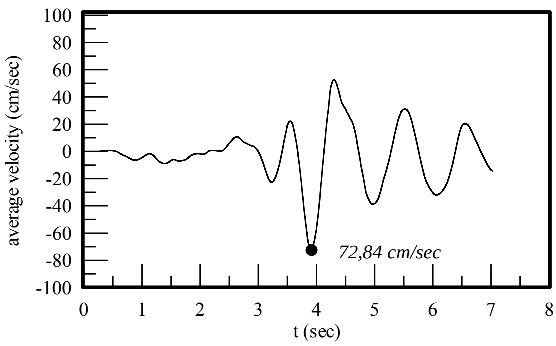
k_{hE}=(0.50-0.80) k_h = 0.38 - 0.60g

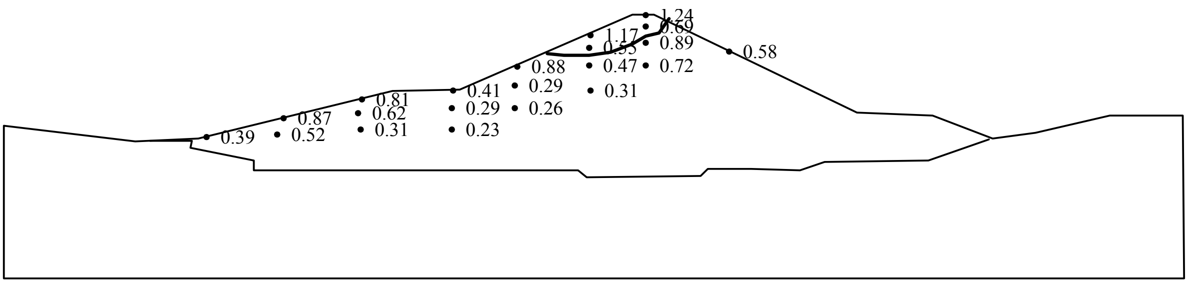
COMPARE WITH

NUMERICAL: RESULTS

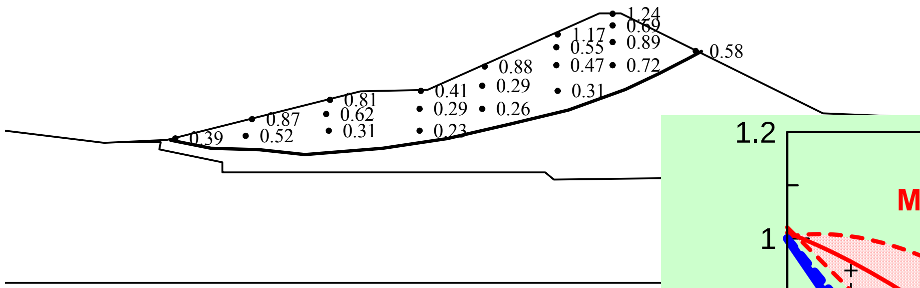
average time histories



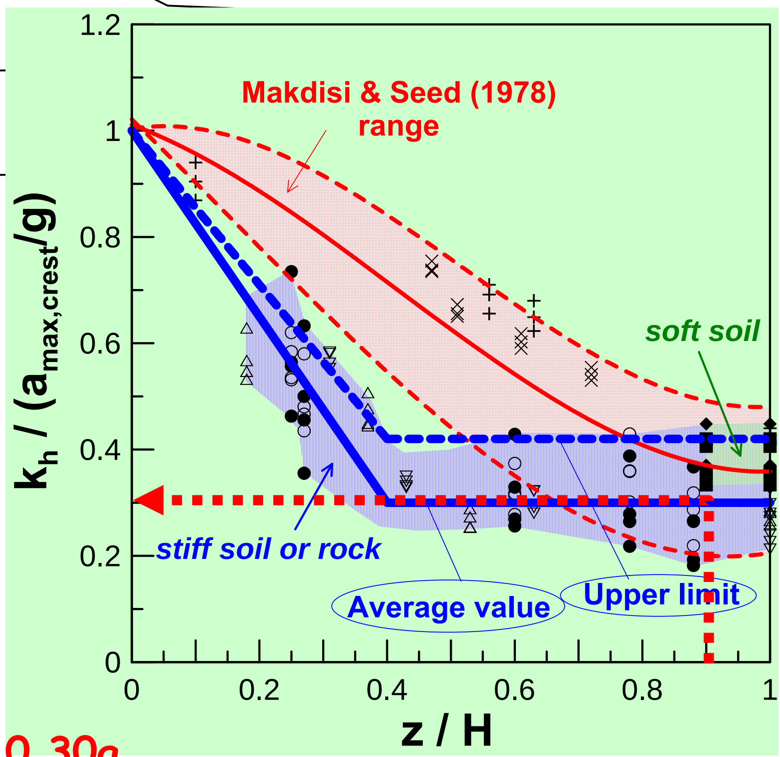




STEP 4: Compute seismic coefficients k_h, k_{hE} (Deep failure)



for z/H = 0.90

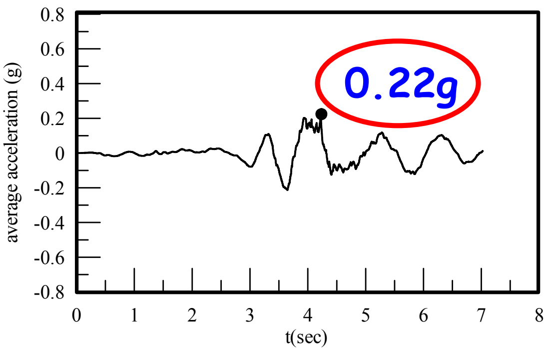


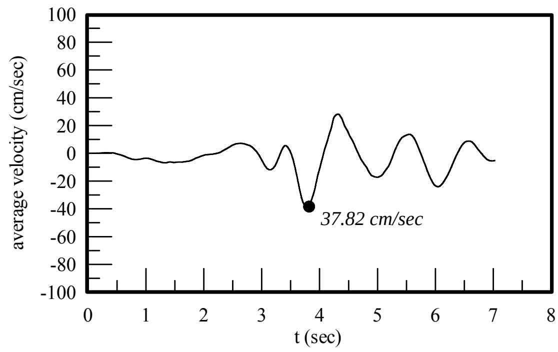
K_h = 0.31 $a_{max,crest}$
or
K_h = 0.31 x 1.18g=0.37g

k_{hE}=(0.50-0.80) k_h = 0.18 – 0.30g

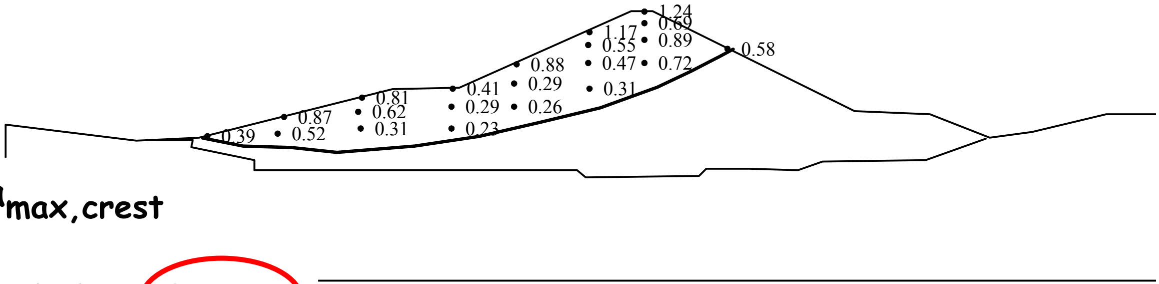
COMPARE WITH NUMERICAL PREDICTIONS

average time histories





for
z/H = 0.90



K_h = 0.31 $a_{max,crest}$
or
K_h = 0.31 x 1.18g=0.37g

8.4 CONCLUDING REMARKS

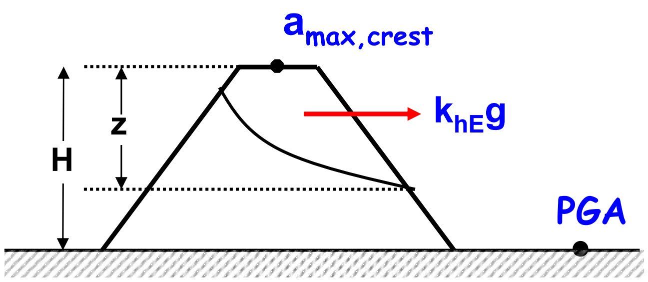
- The PSEUDO STATIC approach with FS_d >1.0 does not prevent slope stability failure.

  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:



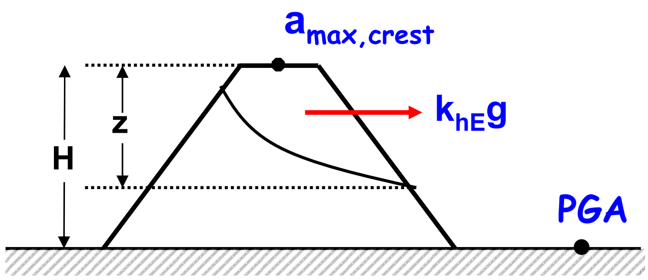
$$k_{hE} = (0.25 \div 1.60)\ PGA/g$$

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

* *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,

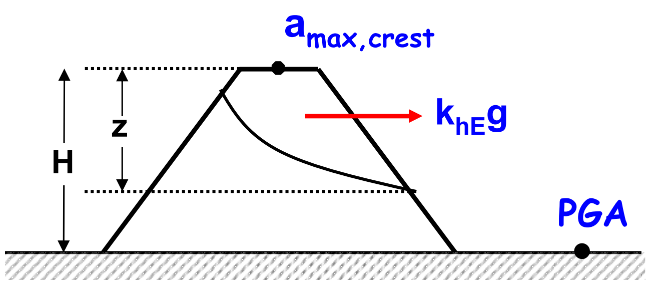


you may use:

- 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

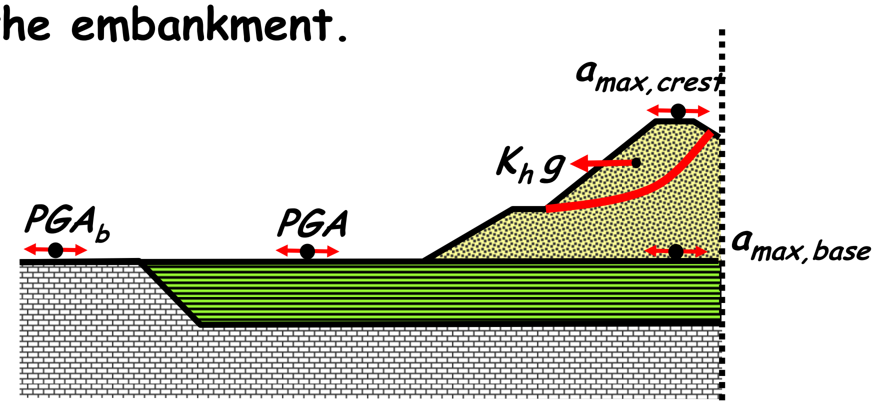
- When the maximum CREST ACCELERATION $a_{max,\ crest}$ is known (e.g. from seismic analysis of the dam),



you may use:

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

- The new INTEGRATED APPROACH provides a rational approach to the computation of the (peak or the effective) seismic coefficient, in terms of the free field seismic motion parameters, as well as the characteristic of the foundation soil and the embankment.



However, note that the method is still under development (*more analyses are performed for soft foundation soil and other than trapezoidal embankment sections*) and consequently it should be used in parallel with some other recognized approach.

1st HOMEWORK: 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=18kN/m^3, c=12.5 kPa and φ=28deg. No ground water is present. Assuming infinite slope conditions:

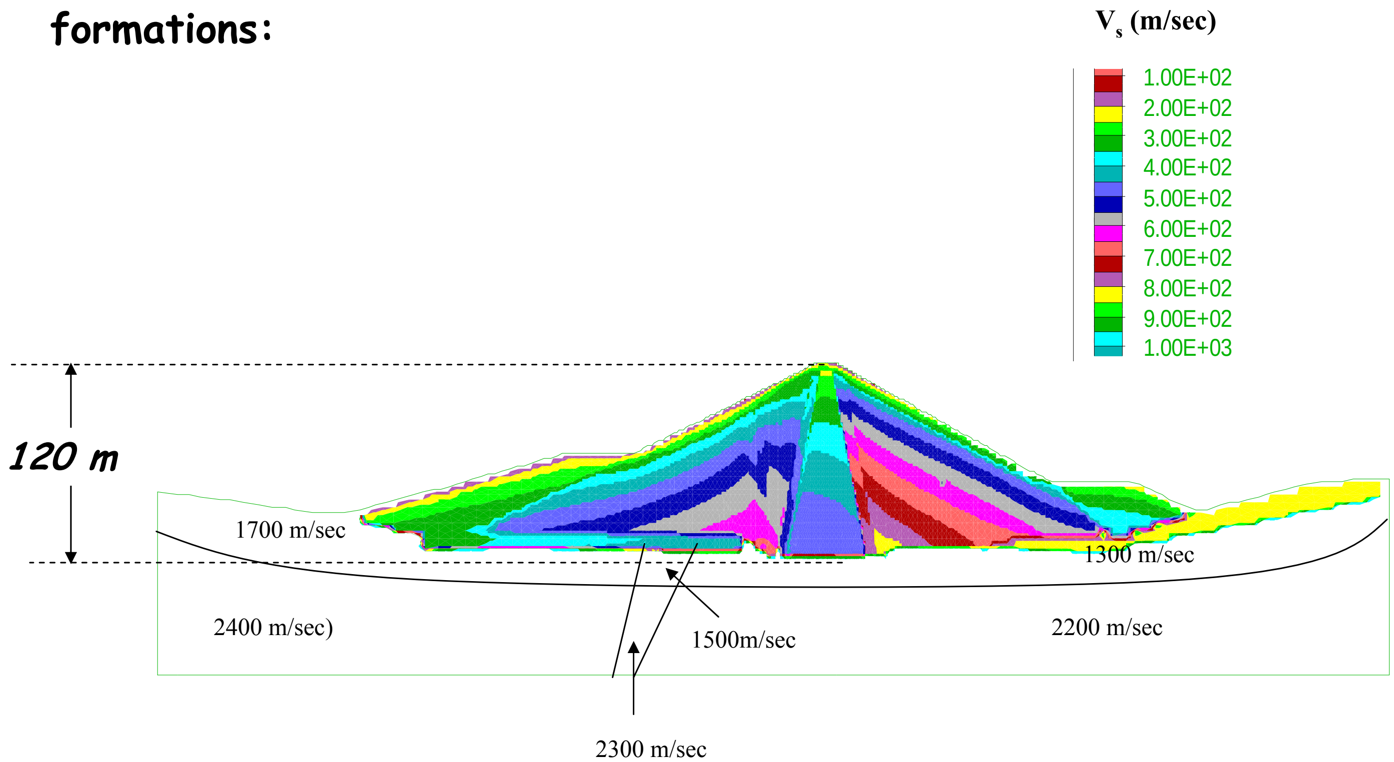
(a) Compute the static factor of safety FS_{ST}.

(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.

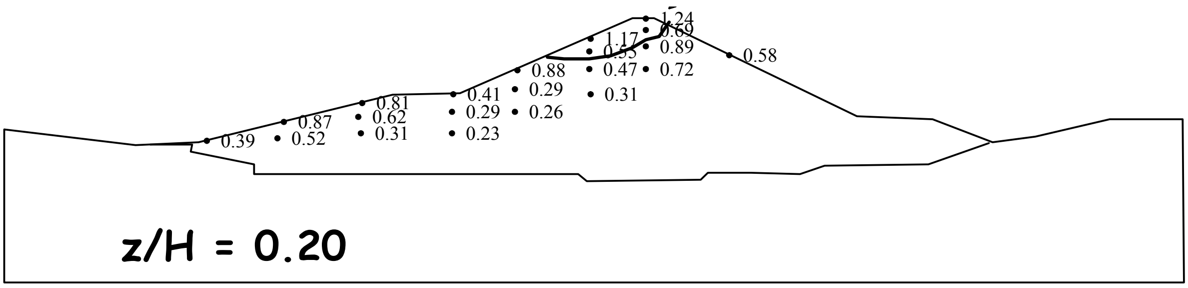
2nd HOMEWORK:
The case of Ilarion Dam in Northern Greece

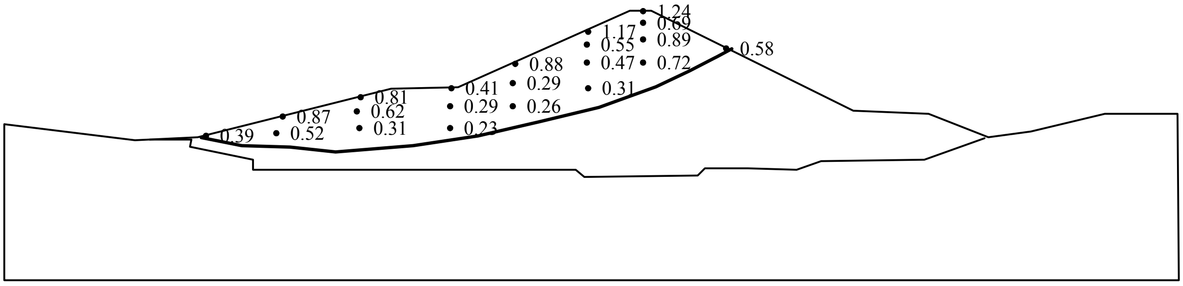
The dam is constructed on weathered rock formations:



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