7 th COULOMB Lecture, Paris 26 June 2009

Seismic Soil–Foundation Interaction on the Verge of "Failure"

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Topics of Presentation

PART 1

(a) Why Going to the Limit and "Beyond" in Seismic Foundation Analysis / Design

(b) Conventional versus New Design Concept: Bridge Pier Foundation

PART 2

The Causes of Overturning of Buildings in Adapazari (1999)

Seismic Foundation Practice

- *Analysis in terms of FORCES*
- *Safety through SAFETY FACTORS*

But now TIME has come for CHANGE :

Analysis–Design in terms of

DISPLACEMENTS, ROTATIONS

" Performance–Based Design "

Current Seismic Approach : "Capacity" Design

(a) Plastic Deformation Allowed Only in the (Super)Structure

(b) NO "Plastic Hinging" Below Ground:

- *Piles, Cap, Footings : Structurally Elastic*
- *NO Mobilization of Bearing Capacity Failure Mechanisms*
- *NO Slippage , LIMITED Uplift*

Why we need to consider Soil–Foundation *Nonlinearity* + *Inelasticity:*

(a) Records in last 20 years: have revealed very strong seismic shaking

Examples:

1994 Northridge	:	0.98 g , 1.40 m/s
1995 Kobe	:	0.85 g , 1.50 m/s
1986 San Salvador	:	0.75 g , 0.84 m/s

and SA values reaching 2 g

Foundation "Plastic Hinging": UNAVOIDABLE

(b) Retrofitting Existing/Damaged Structures

Usually Impossible

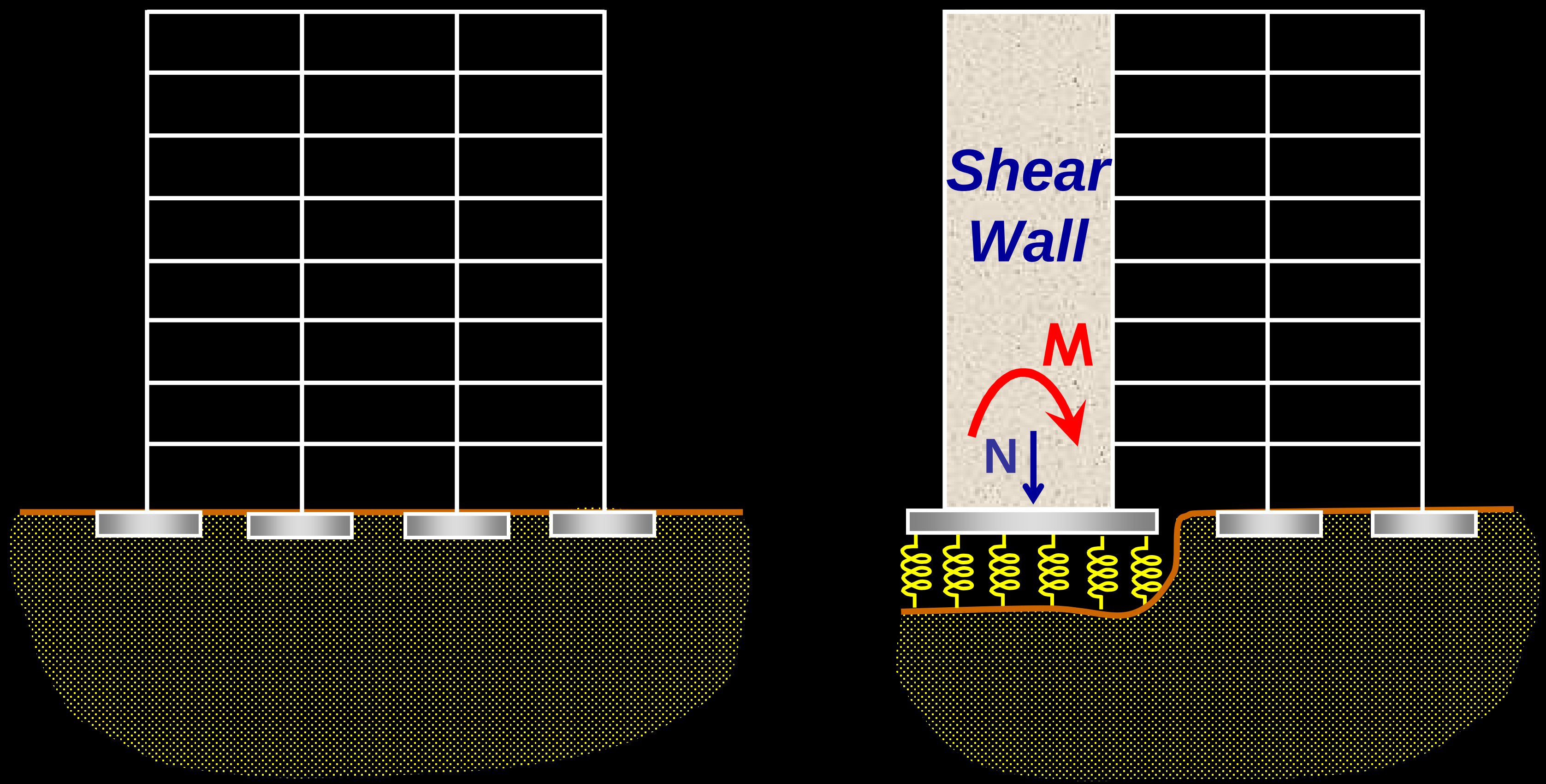
to Accomplish Elastically

(even if very conservative design required)

Must Consider Inelastic Action in

Soil + Foundation

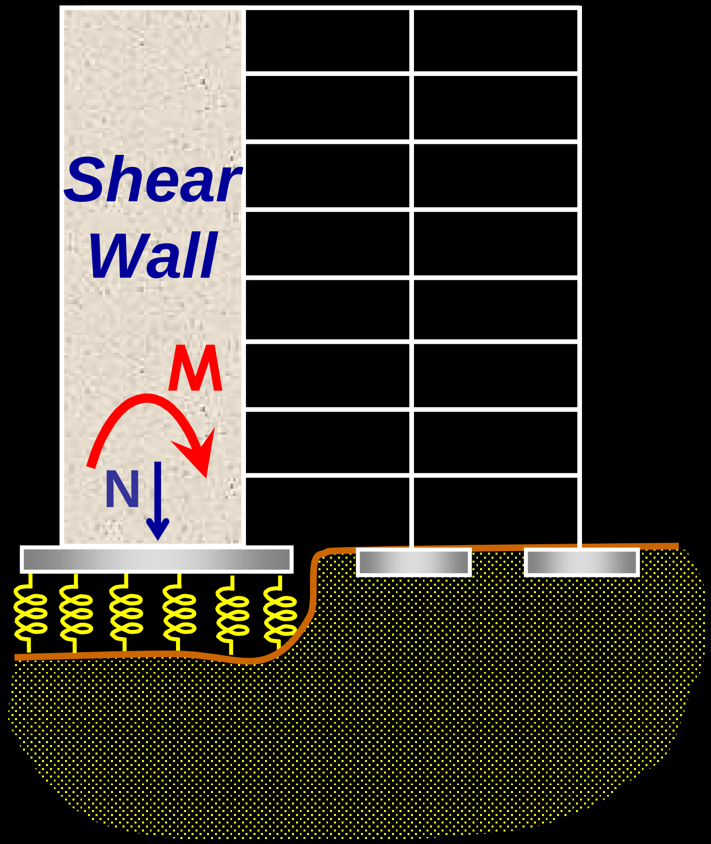
Existing **Retrofitted**



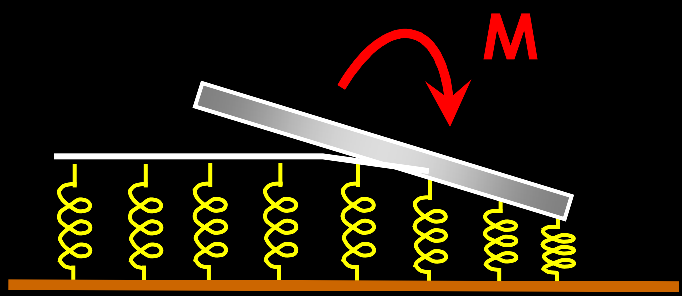
⇨ ***N small***

⇨ ***M very large***

Retrofitted



Uplifting, Nonlinearity:



Significantly affect the sharing of lateral force among shear-wall and frames

(c) Need : Determine Collapse Motion

- *for Compatibility with Structural Design*

(Push-over analysis, ductility–based design)

- *for Insurance Purposes*

(special projects demanding estimate of LOSS in worst case)

Can we move Beyond this Conventional "Capacity" Design ?

Major Contribution of Alain Pecker (1998)

"Capacity design principles for shallow foundations in seismic areas"

Previous Research /Applications

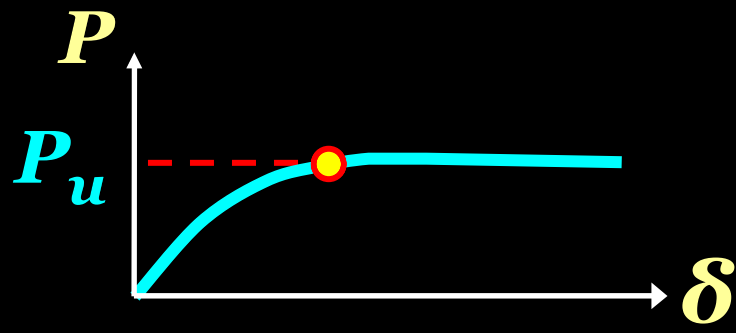
- Pecker (1998): Capacity Design for Foundations
- Paolluci (1998): Inelastic-soil SSI
- FEMA 356 (2000): Rehabilitation Code
- Kutter et al (2001): Centrifuge Experiments
- Martin & Lam (2000): Retrofit of Bridges
- El Naggar et al (2000): Elasto-plastic Winkler
- Pecker et al (2009): Inelastic Macro-element

Factors of Safety

- Static : $FS > 1$

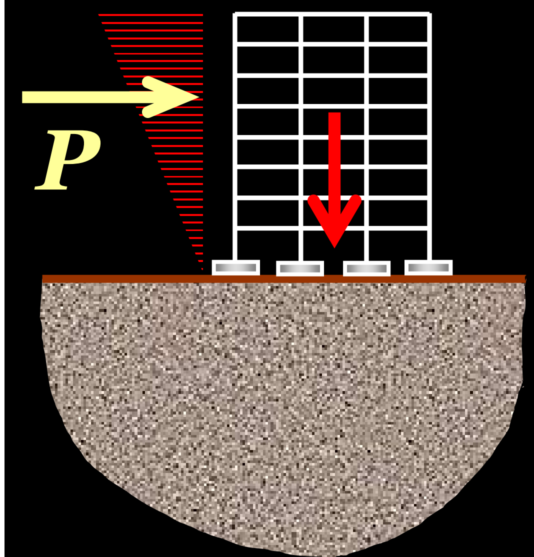
- Seismic : $\min_{t} FS(t) < 1$

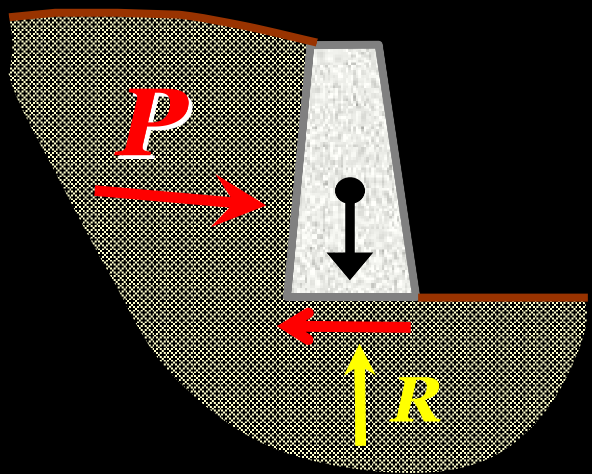
Elasto-Plastic Systems :

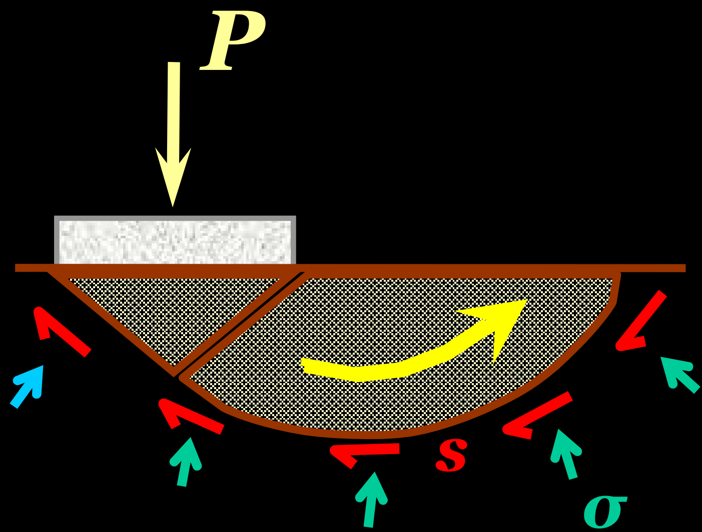


External Force $P > P_u$

(a) if STATIC : Failure







(b) if SEISMIC :

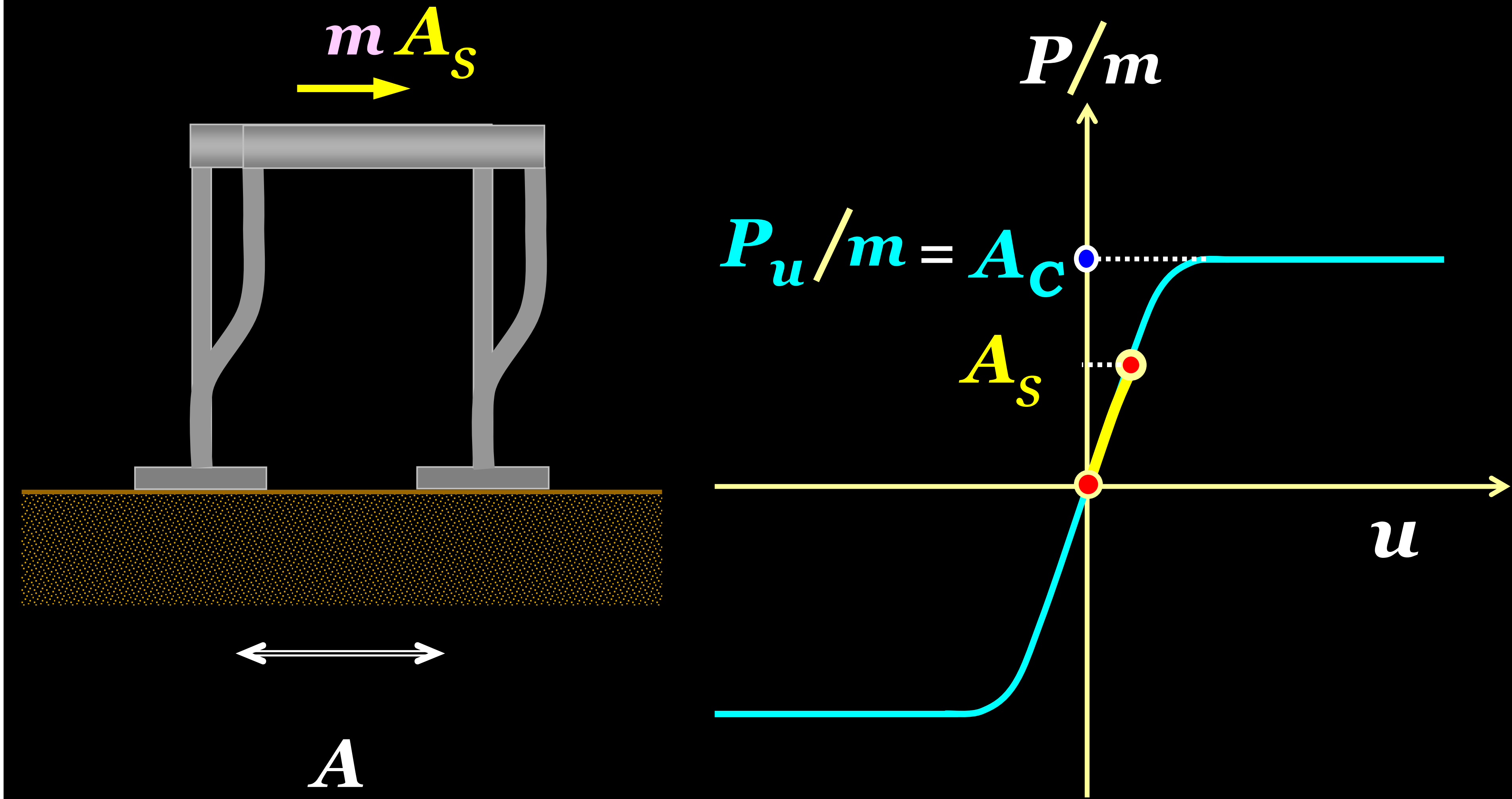
Inelastic Deformations (only)

Thanks to the Nature of Seismic Excitation

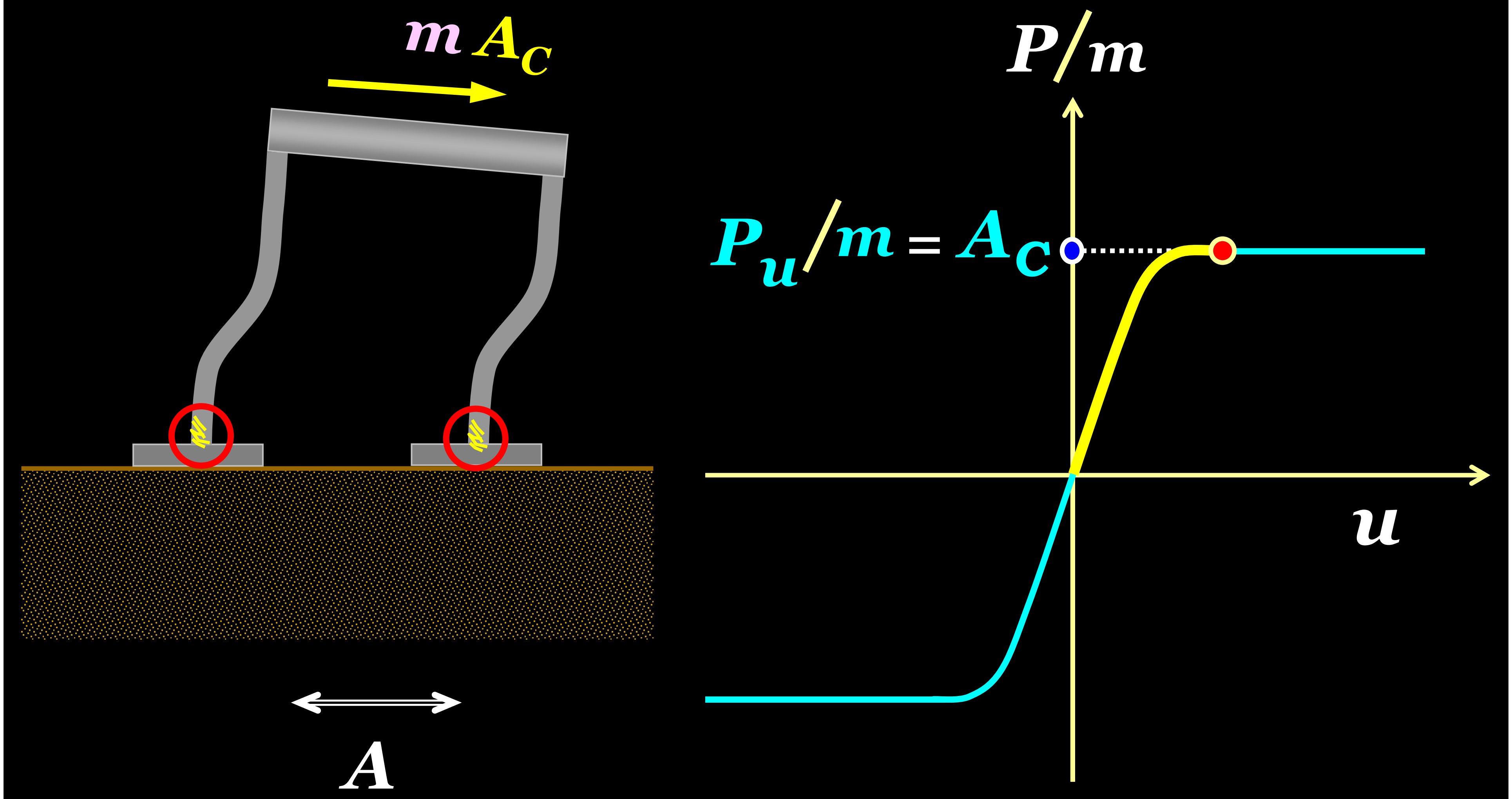
⇒ **CYCLIC**

⇒ **KINEMATIC**

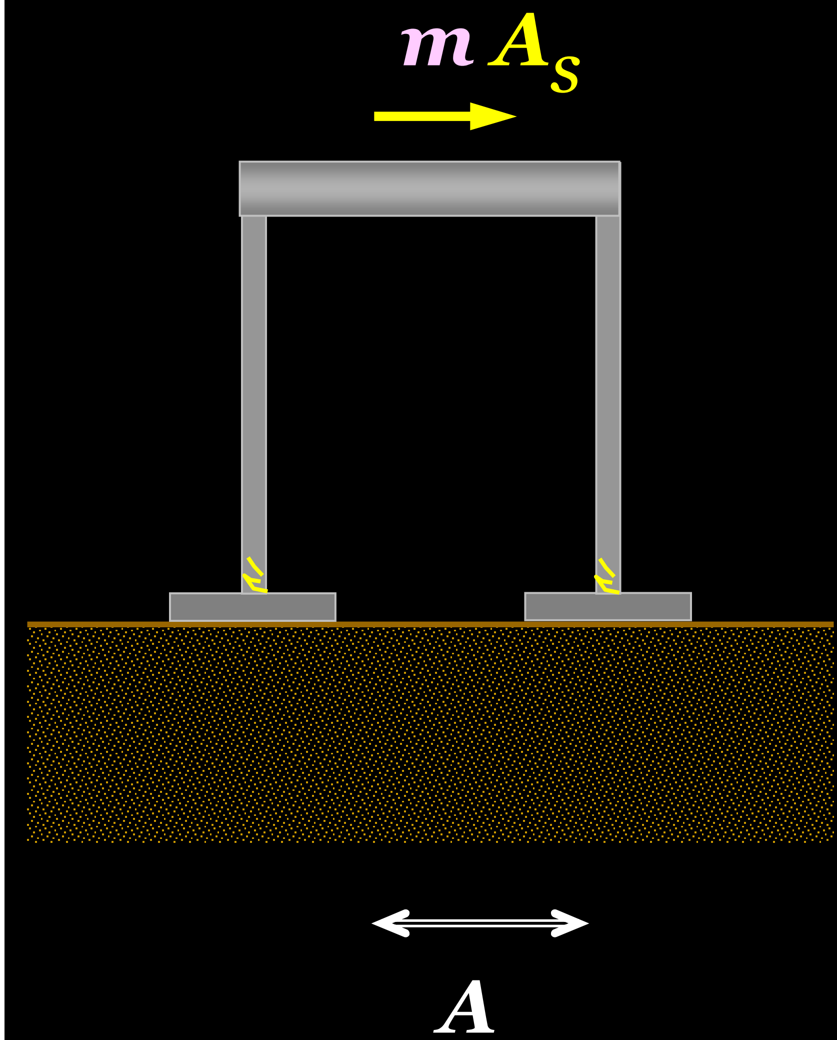
Elastic Response : $A_S < A_C$

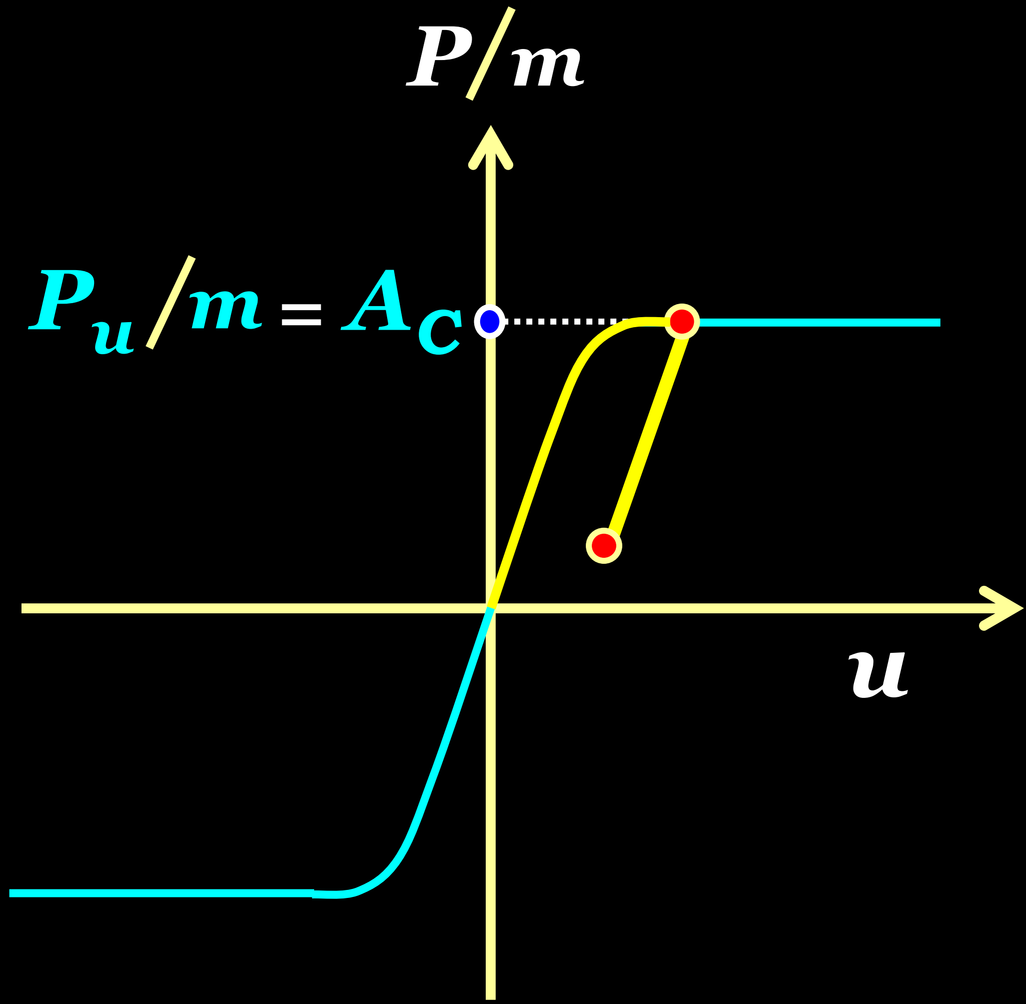


Plastic Response: $A_S = A_C \Rightarrow P_S = P_u$

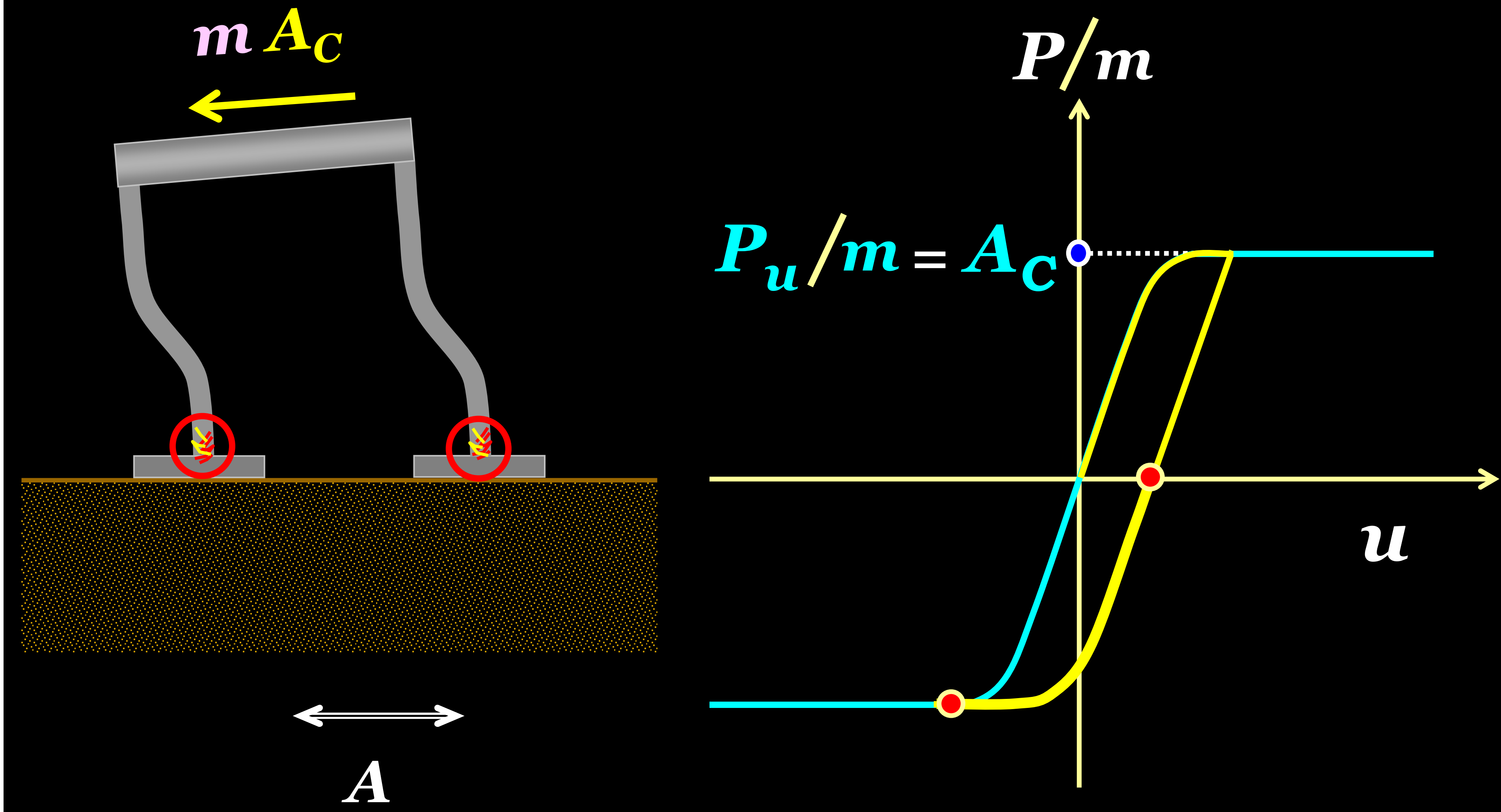


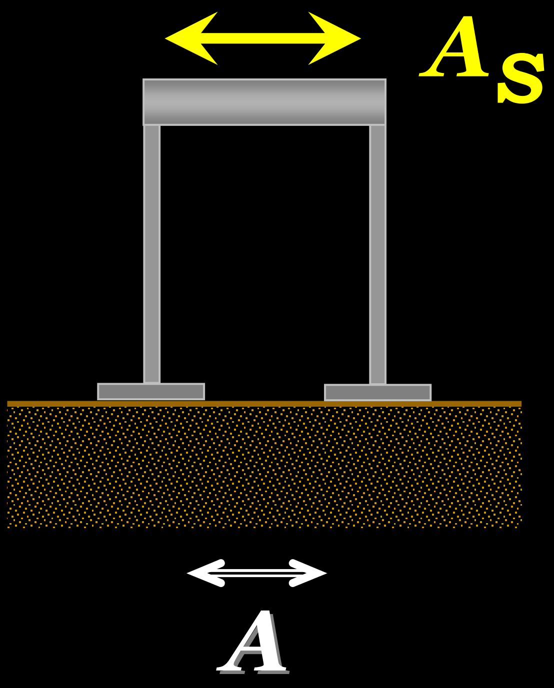
Unloading: $0 < A_S < A_C$

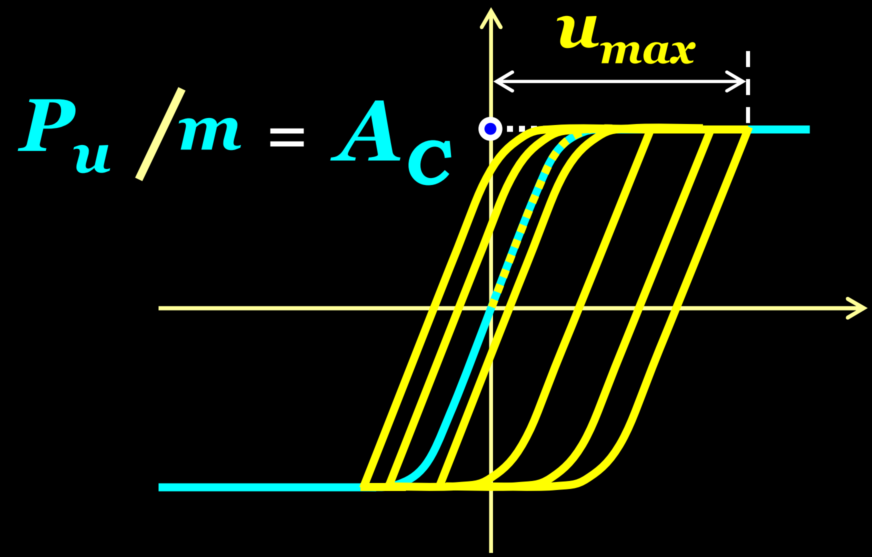




Unloading: $A_s = -A_C$







$A > A_C$

$max\, A_S\, (structure) = A_C$

$u_{max} > u_{elast}$

But NO Failure

in Geotechnical Engineering
the implications of
Dynamic Safety Factor
FS < 1 :

(a) Sliding (symmetric, asymmetric)

(b) Uplifting , Overturning

(c) Bearing Capacity “Failure” ??

N. Newmark:
1965 Rankine Lecture

Whitman 1964

Ambraseys & Sarma 1967

Seed et al 1967

Richards & Elms 1979

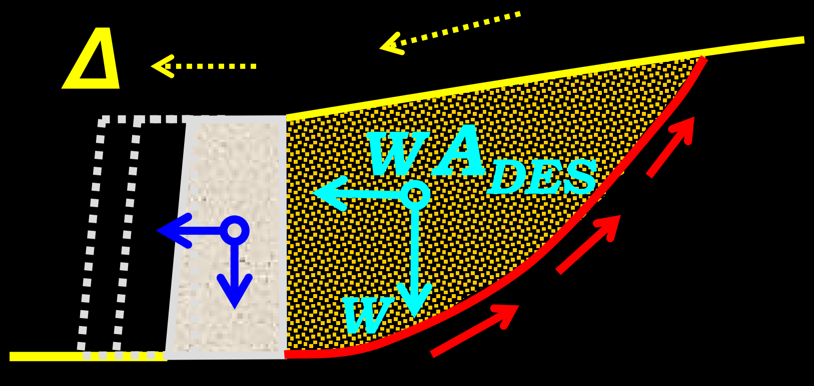
Pecker 1998

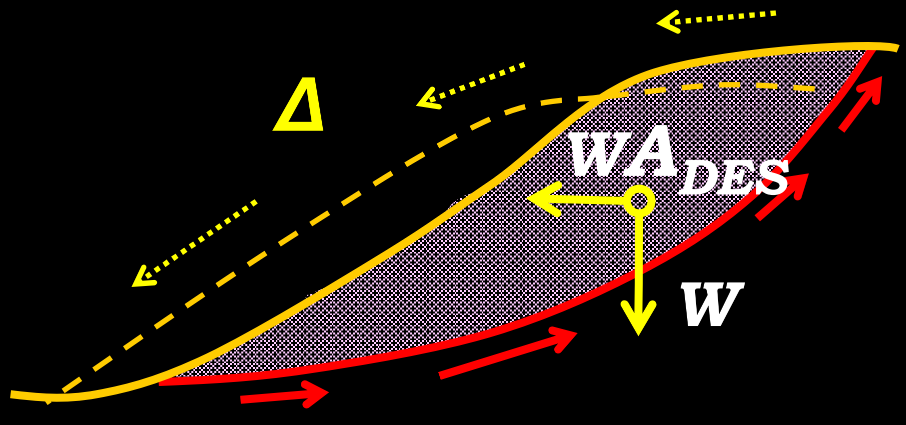
Current Seismic Codes

- *(Gravity) Retaining walls*
- *Embankments / Natural Slopes*

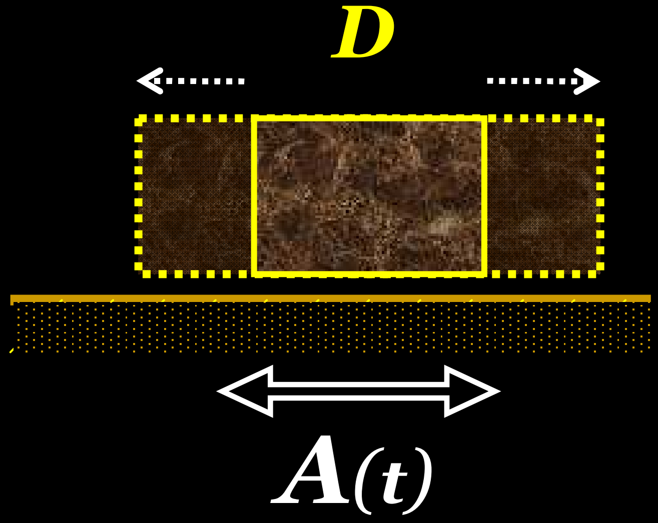
Designed (indirectly) for inelastic deformation Δ ~ 10–30 cm :

$$A_{DESIGN} = {}^{1}/_{2}\ A$$



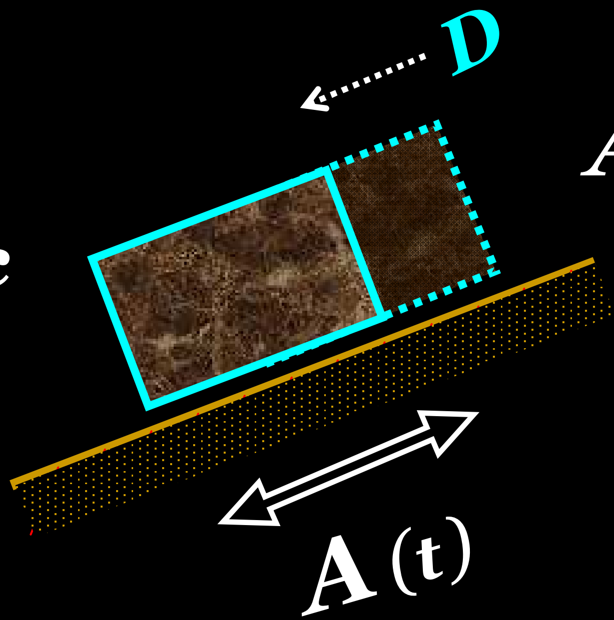


Symmetric sliding

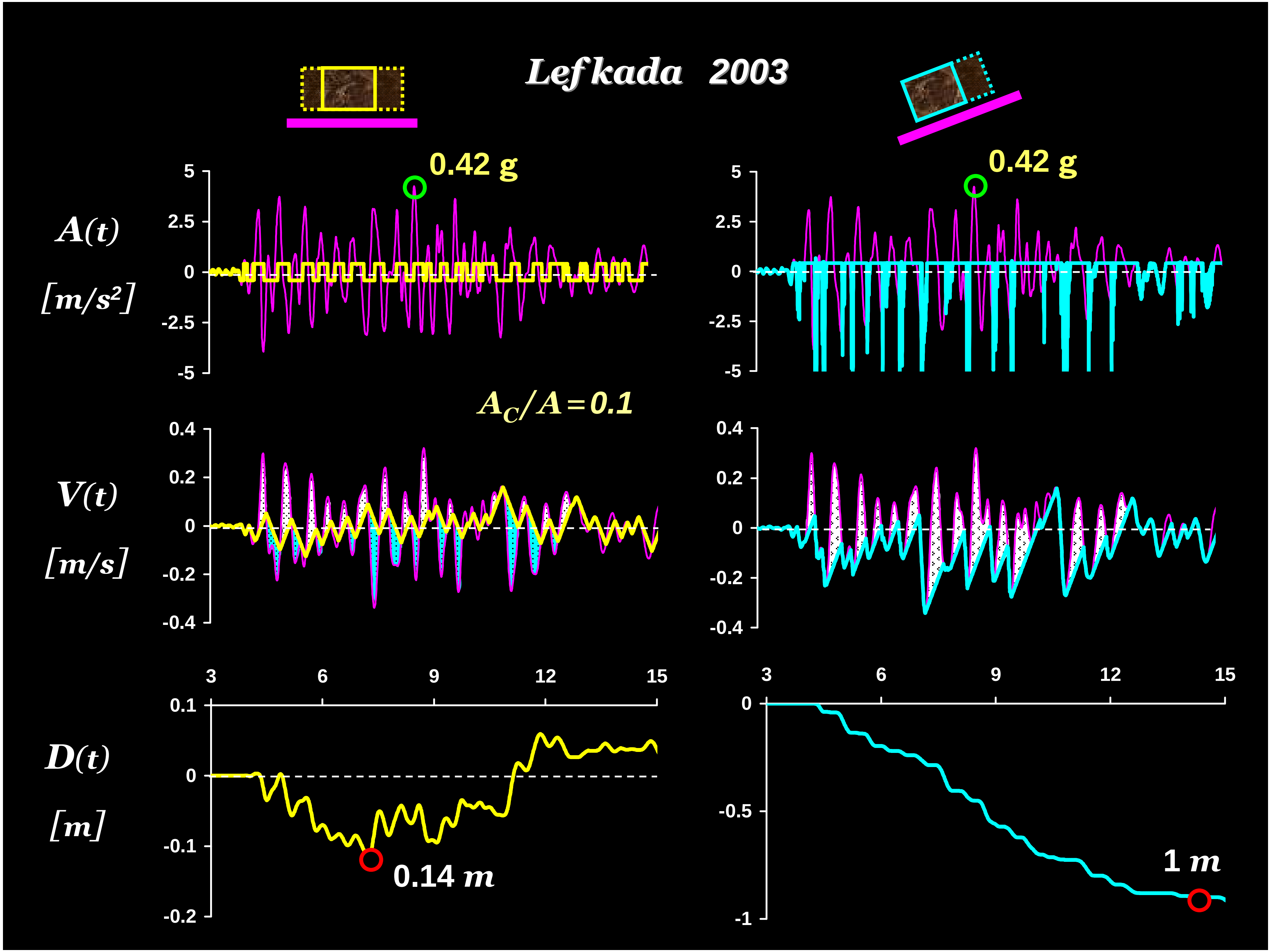


$$A_C = \mu g$$

Asymmetric sliding

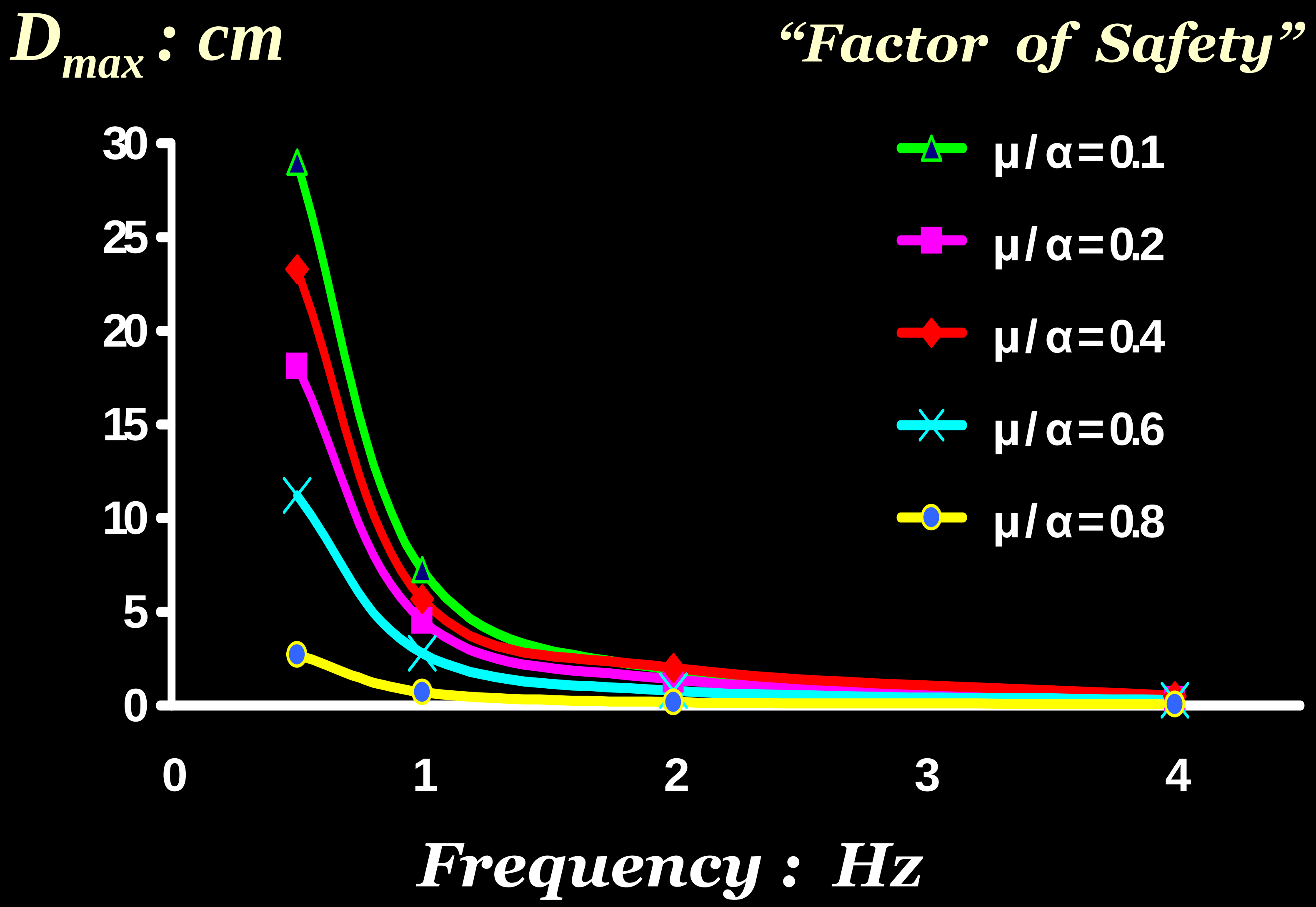


$$A_C = \mu\, g \cos\beta - g \sin\beta$$

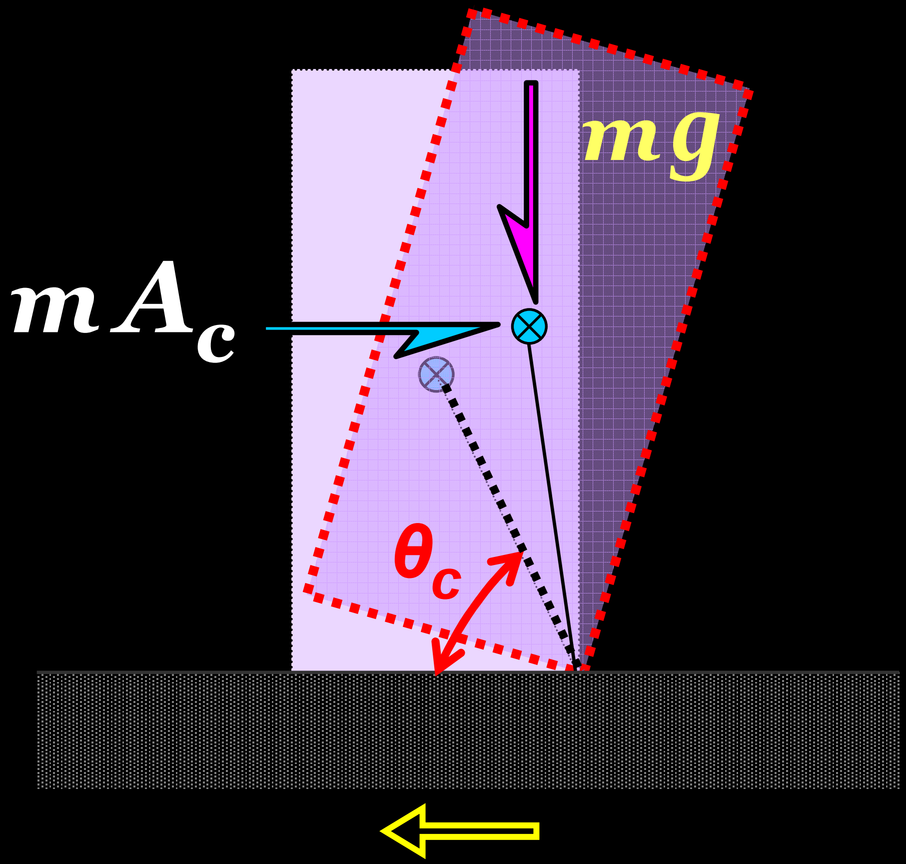


Lefkada 2003
0.42 g
0.42 g
A(t)
[m/s²]
A_C/A = 0.1
V(t)
[m/s]
D(t)
[m]
0.14 m
1 m

Effect of Excitation Frequency



Rigid Block on a Rigid Base



Uplifting Acceleration

$$A_c = (b/h)\,g$$

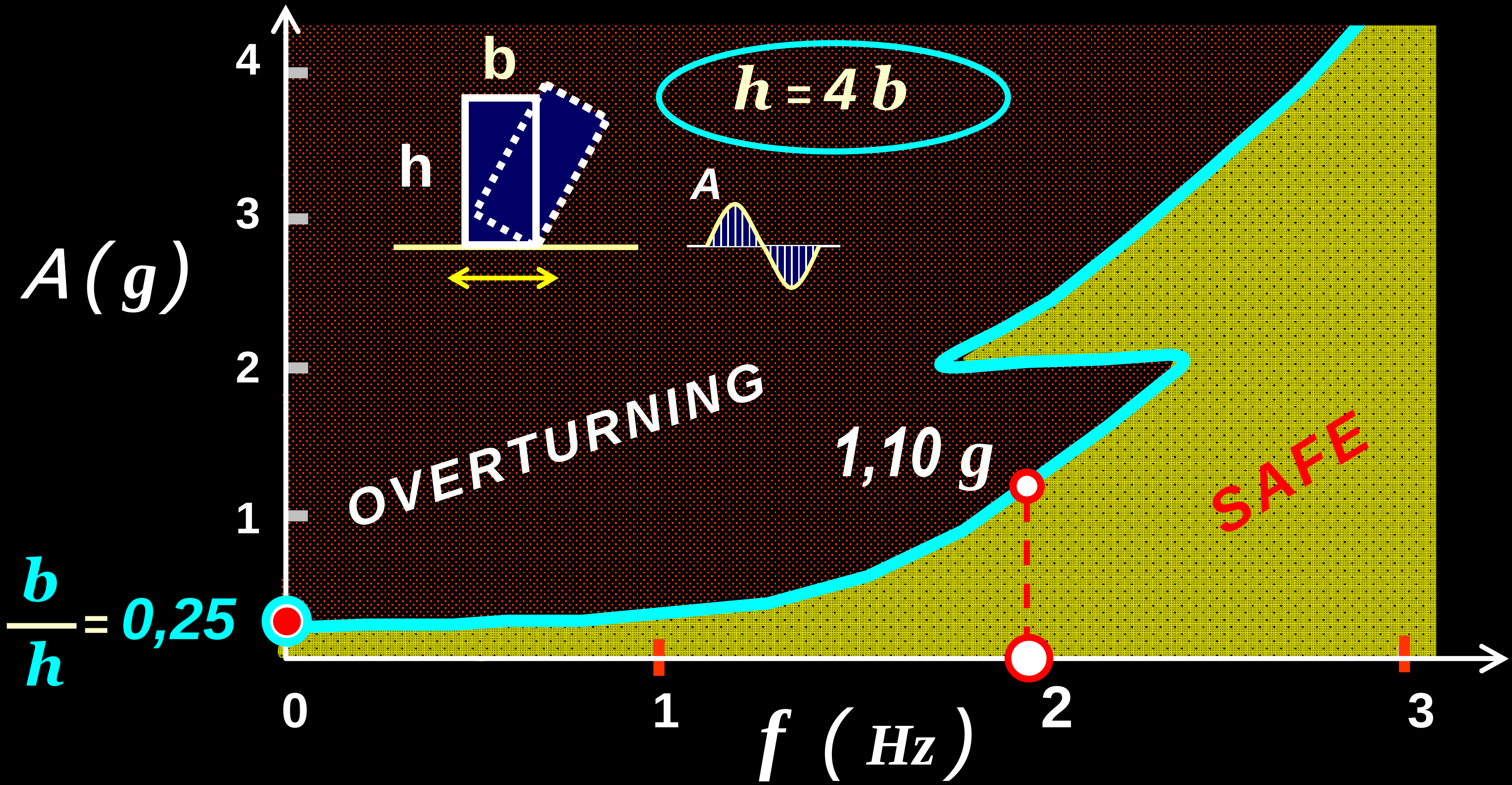
⇩

Toppling

(under **Static** Conditions)

Rocking of Slender Block on Rigid Base

(undergoing a one-cycle sinusoidal shaking)

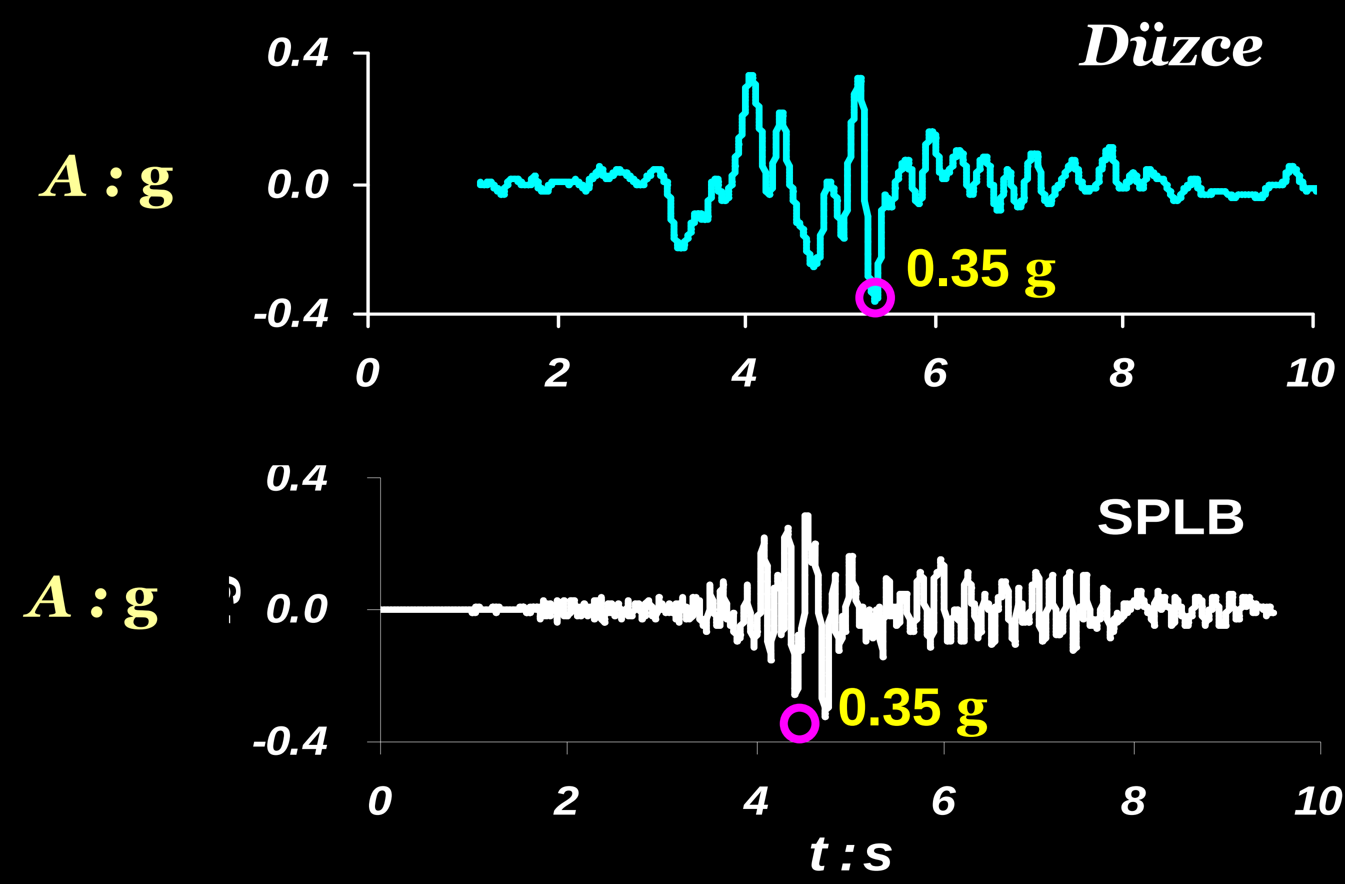


Static Failure ⟹ $A > 0{,}25\ g$

Seismic Failure (with $f = 2$ Hz) ⟹ $A > 1{,}10\ g$

Overturning of a Slender Tombstone
in the Athens Earthquake : 7 - 9 - 99

Two Hypothetical Base Excitations :



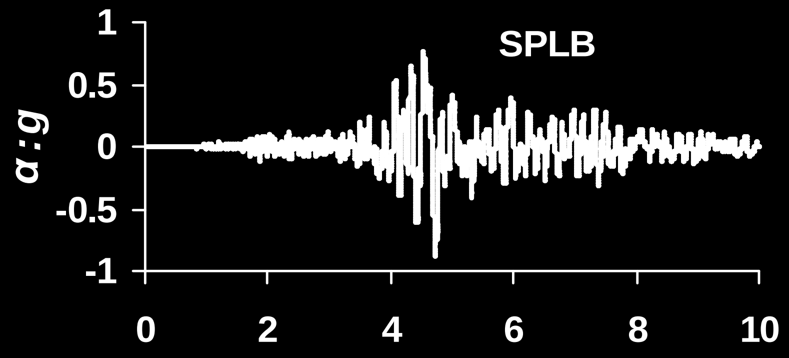
Overturning of Tombstone

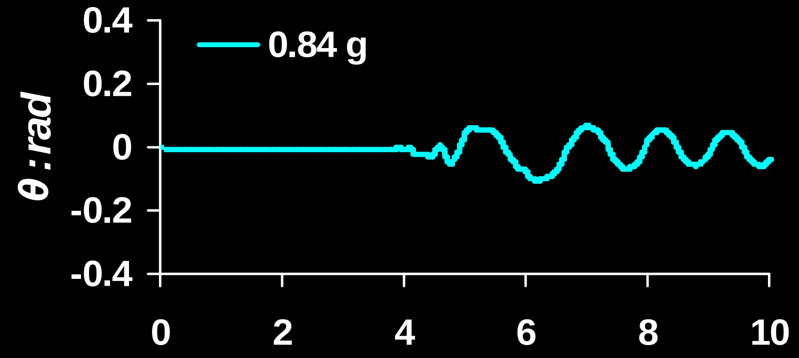
$2h = 1.27\ m$, $2b = 0.20\ m$, $h / b = 6.35$, $A_c \approx 0.16\ \mathrm{g}$

Overturning of Tombstone

$2\,h = 1.27\ m$, $2\,b = 0.20\ m$, $h\,/\,b = 6.35$, $A_c \approx 0.16$ g

Scaling of the Records needed to Overturn the Tombstone



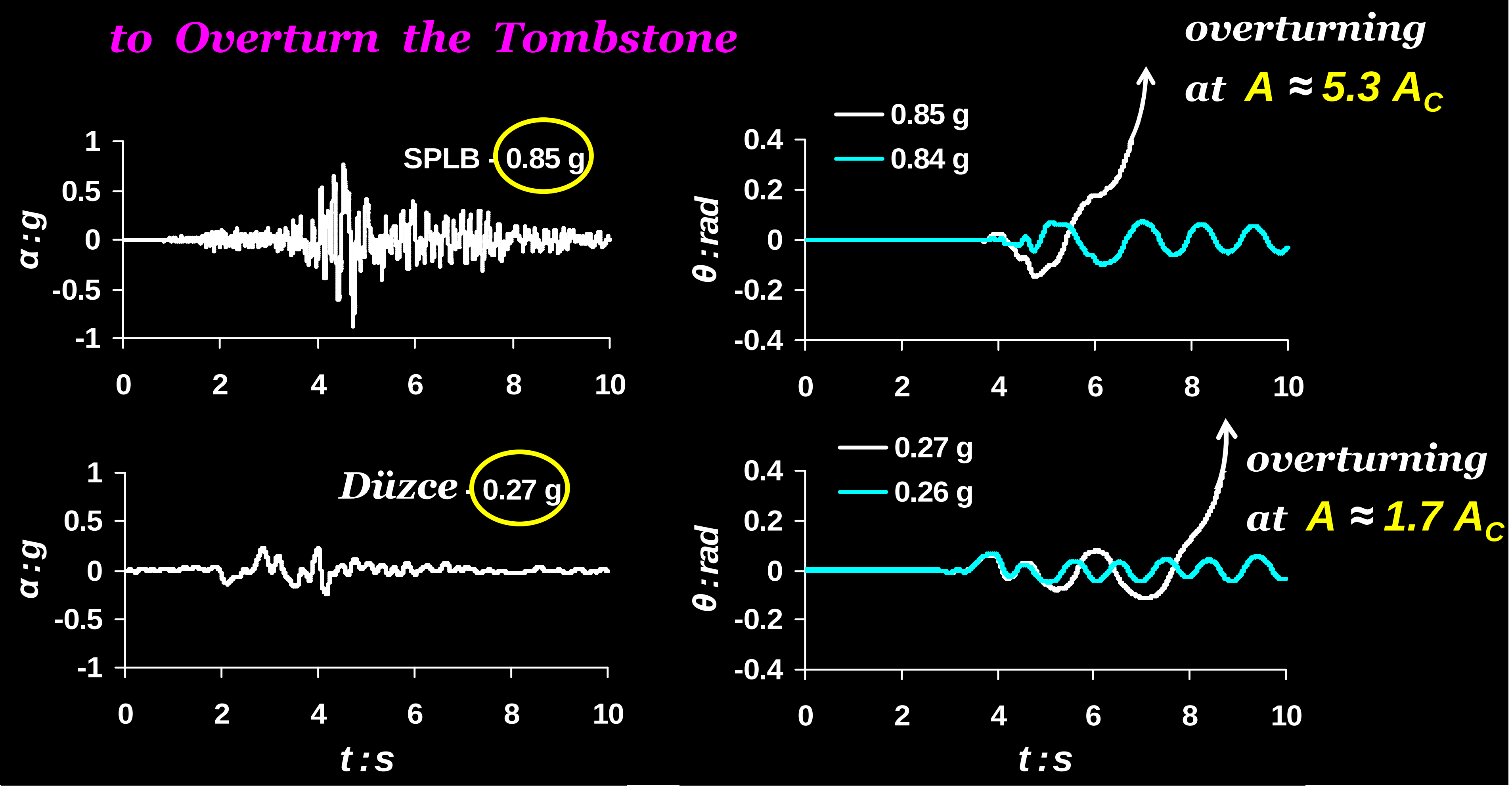


Overturning of Tombstone

$2\,h = 1.27\ m\,,\quad 2\,b = 0.20\ m\,,\quad h\,/\,b = 6.35\,,\ A_c \approx 0.16\ g$

Scaling of the Records needed

to Overturn the Tombstone



Dynamic Safety Factor
FS << 1 :

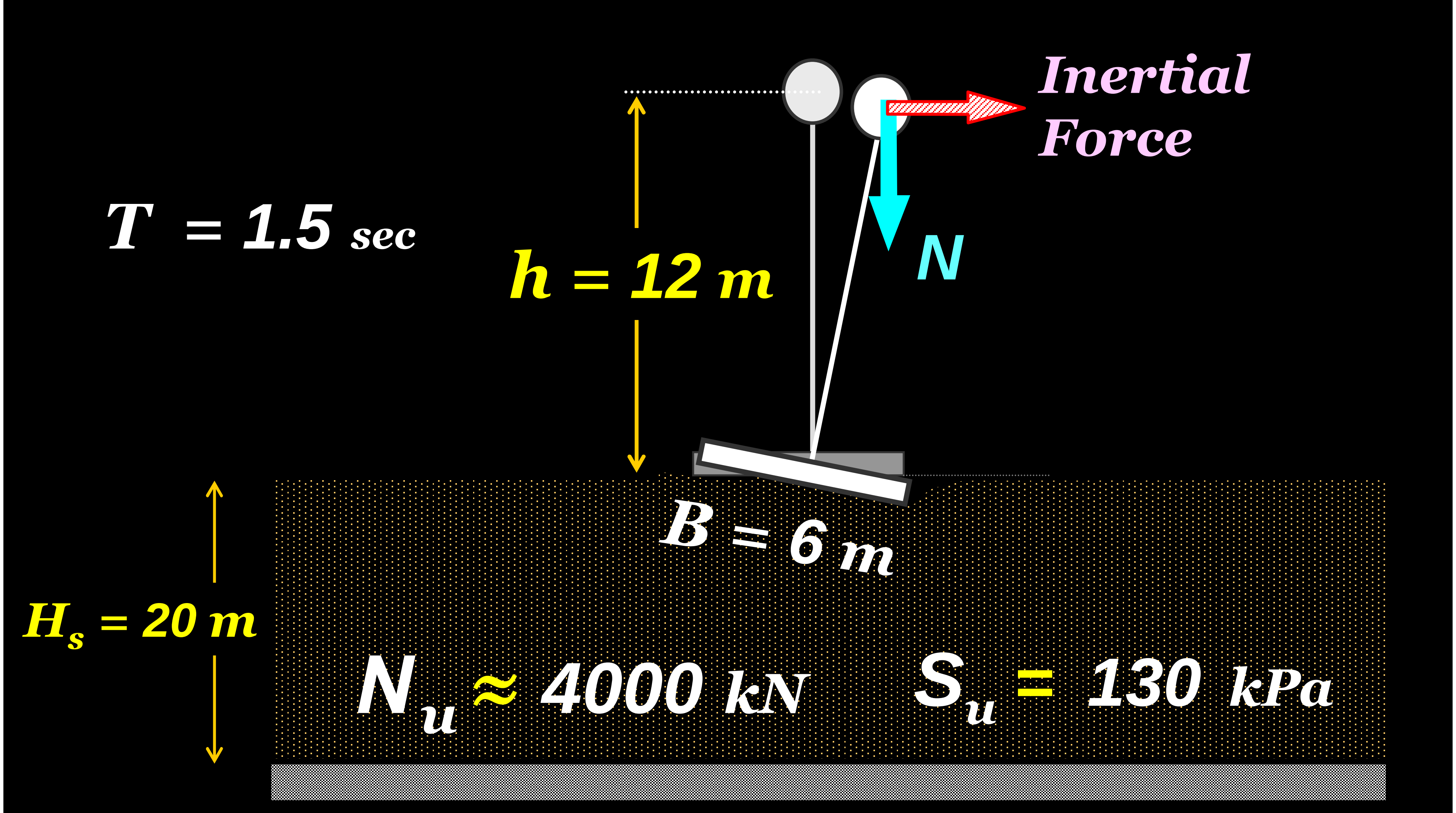
Consequences

(α) *Sliding (symmetric, asymmetric)*

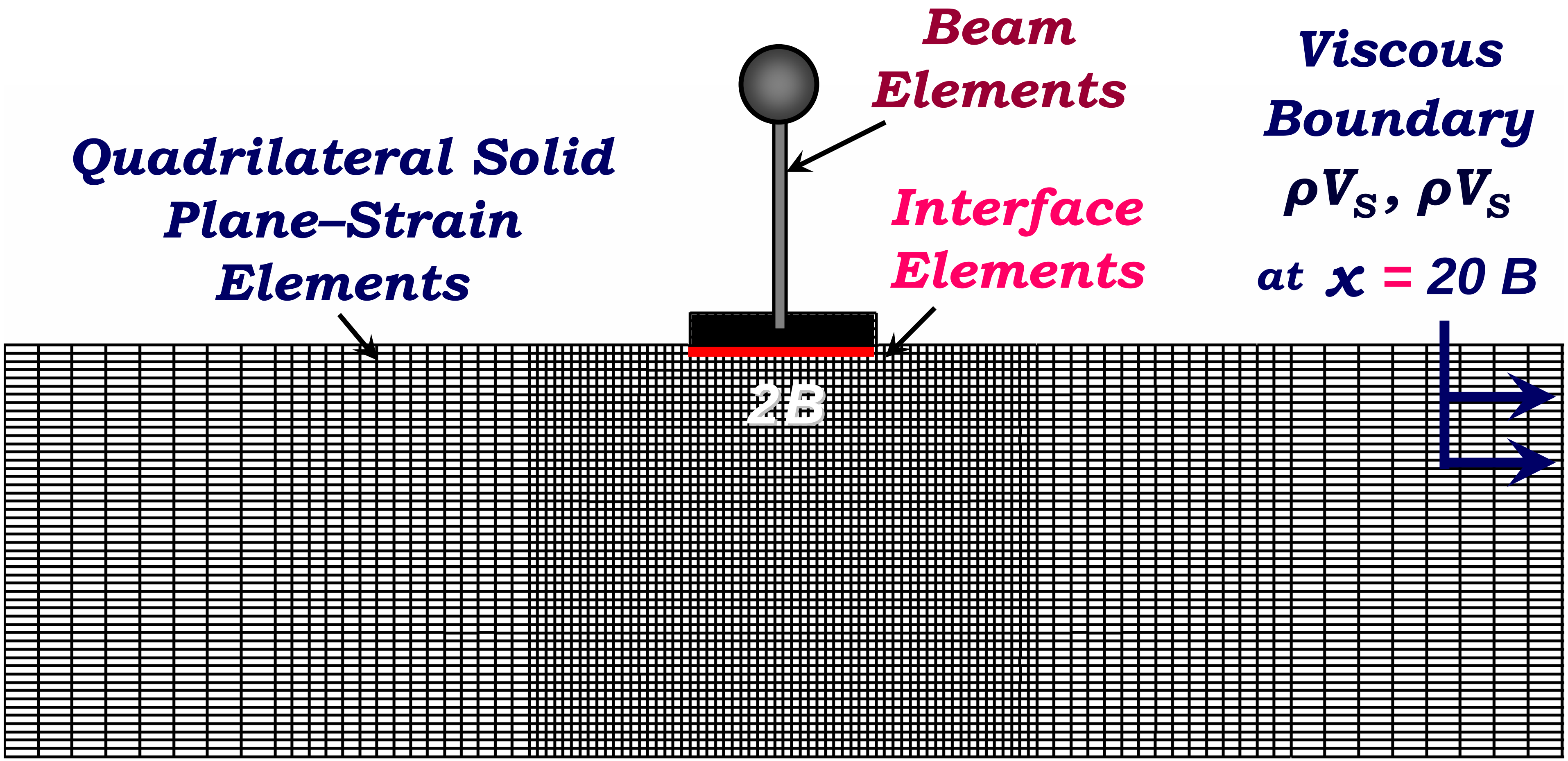
(b) *Uplifting , Overturning*

(c) ***Bearing Capacity "Failure"***

Simple Model of Bridge Pier



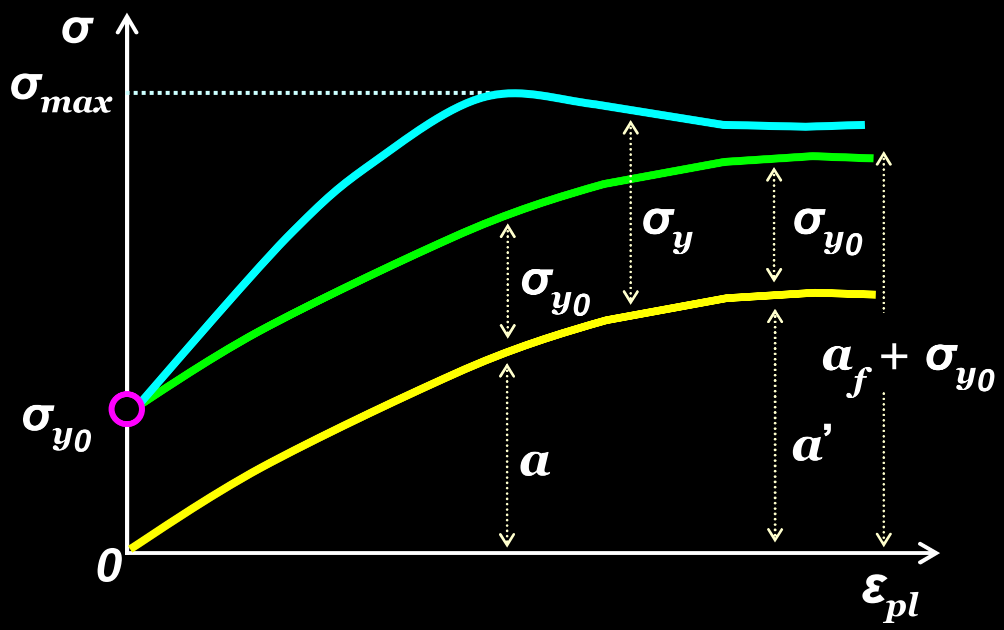
Modelling with Finite Elements



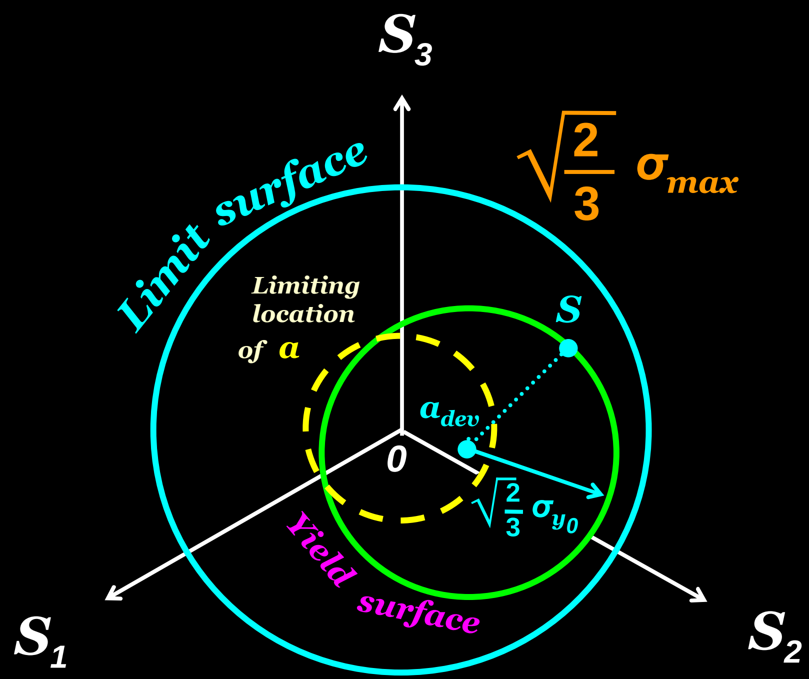
Soil and Column: Inelastic

Constitutive Cyclic Model for Soil

Nonlinear, Von-Mises Yielding,
Isotropic/Kinematic Hardening ,
, Associative Flow Rule

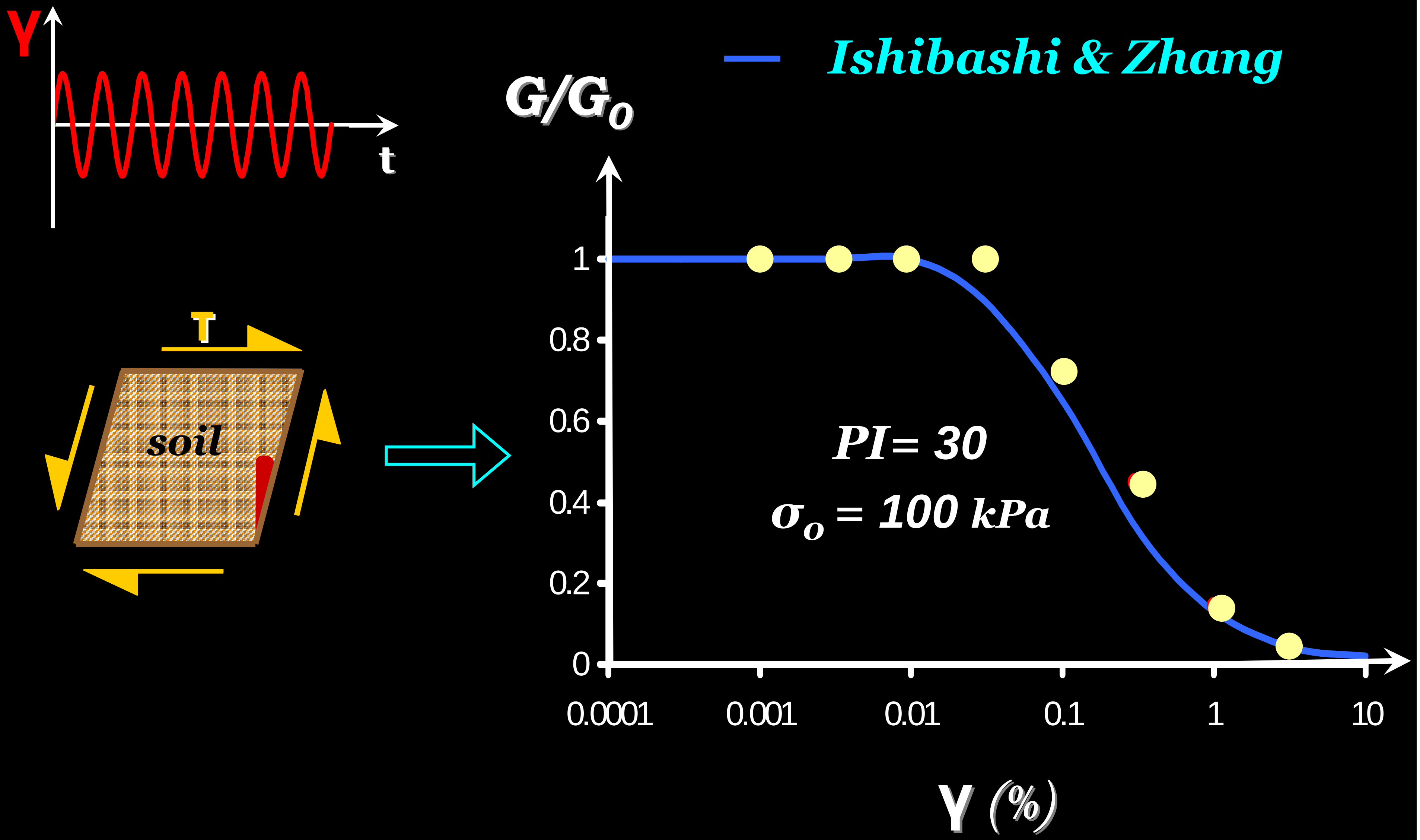


1-D Representation

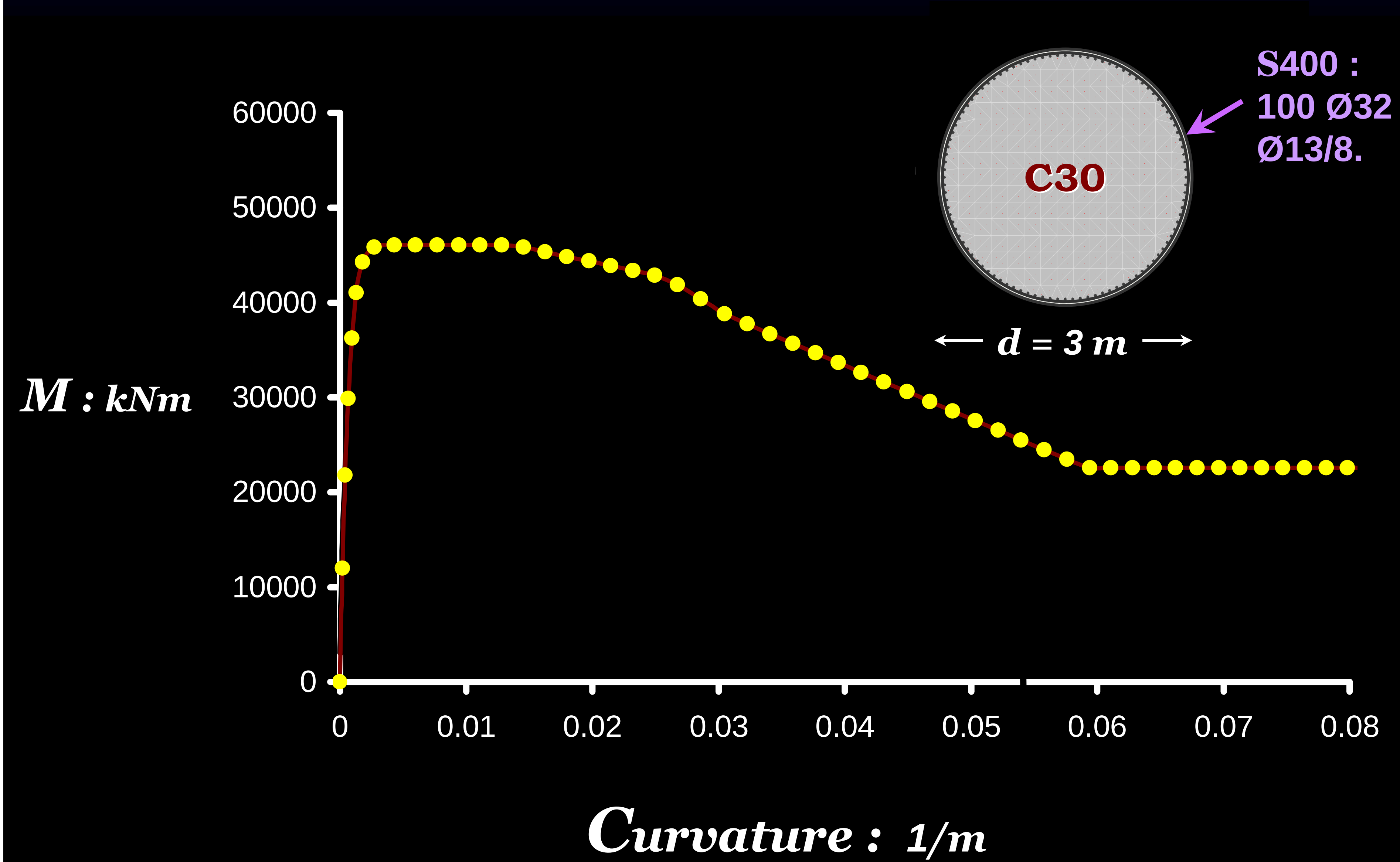


3-D Representation

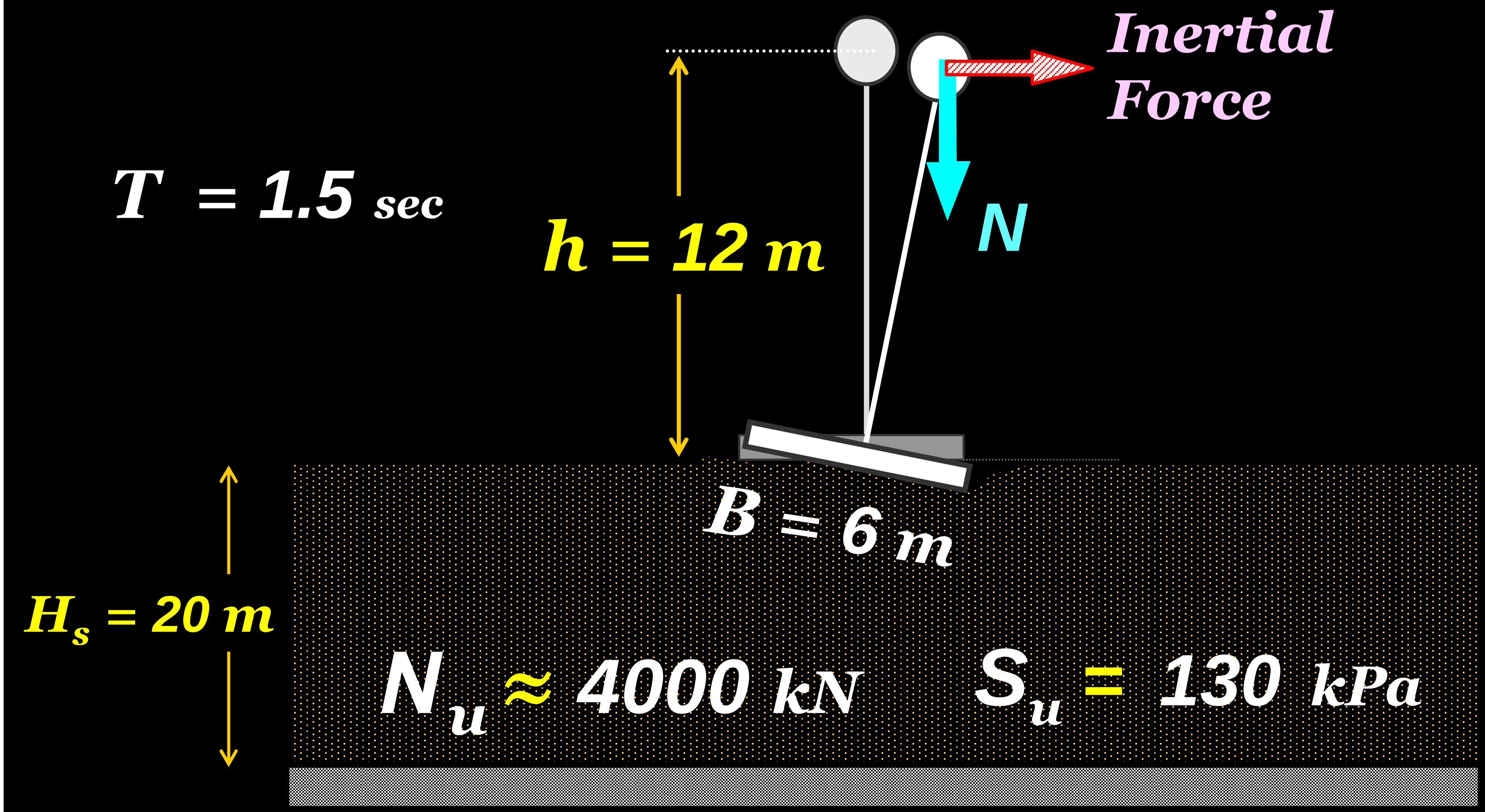
Calibration against $G-\gamma$ curves

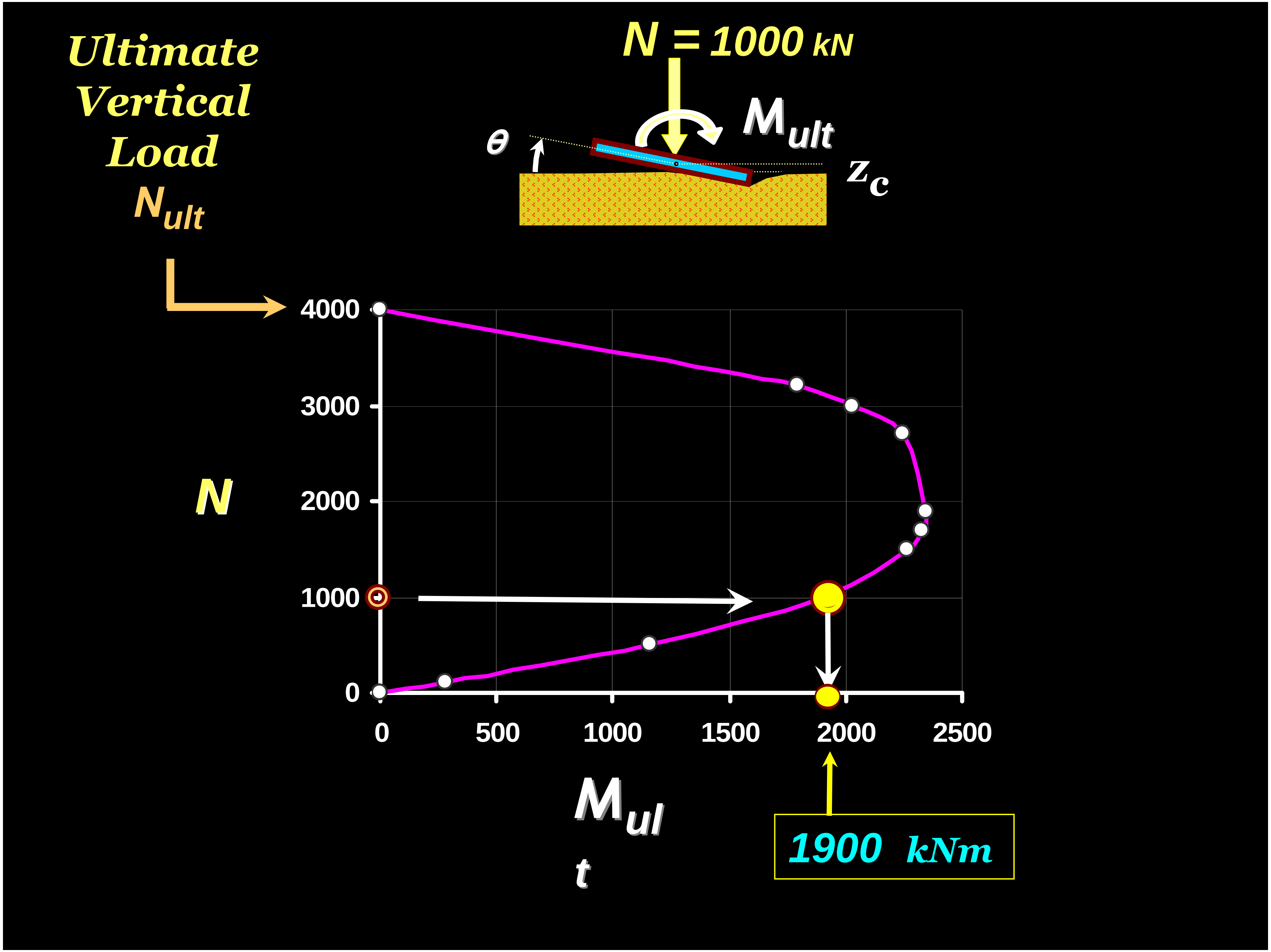


Reinforced Concrete



Simple Model of Bridge Pier

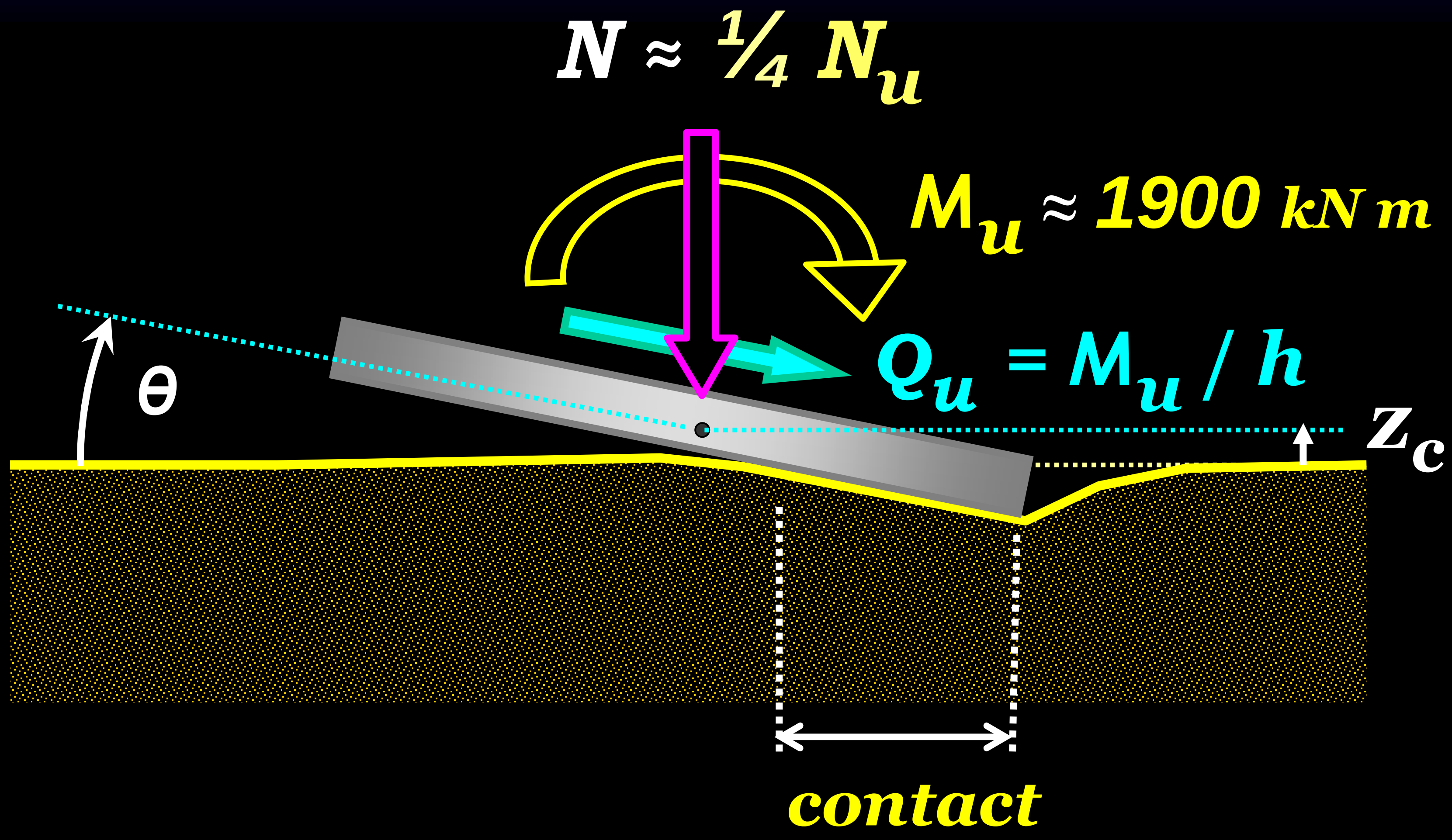


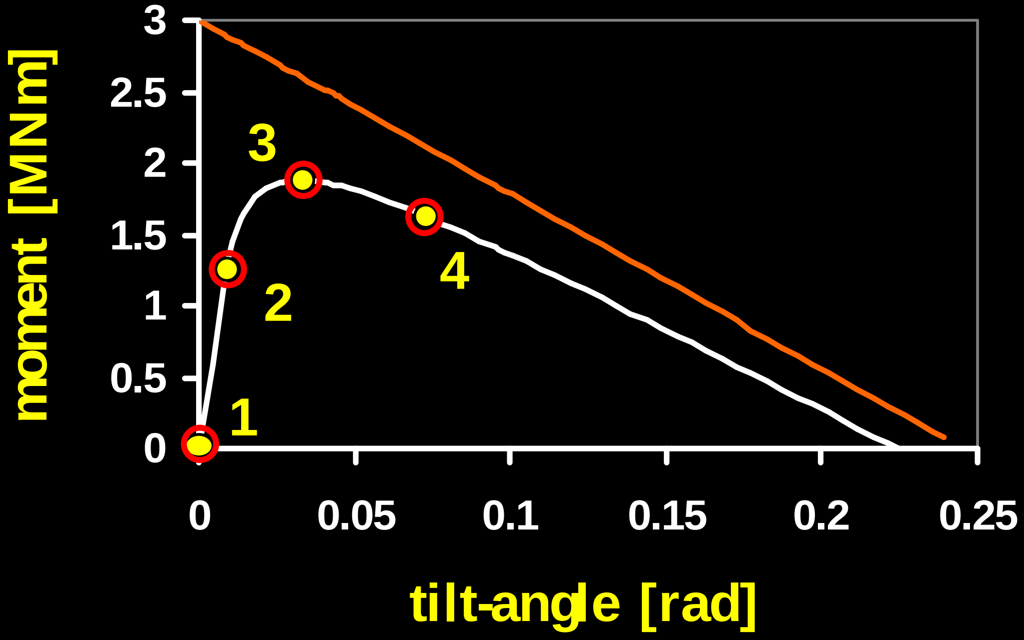


Ultimate
Vertical
Load
N ult
N = 1000 kN
M ult
θ
z c
4000
3000
2000
1000
0
N
0
500
1000
1500
2000
2500
M ult
1900 kNm

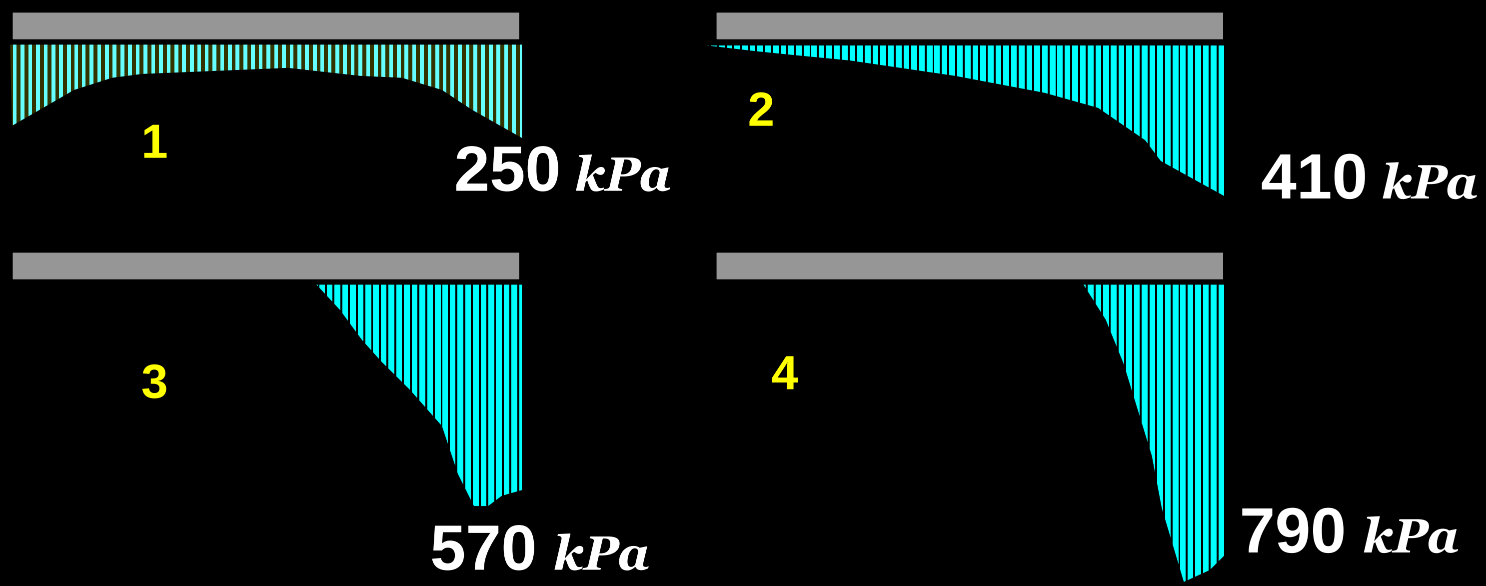
Pseudo–Static Analysis

(constant $N = 1000\ kN$*)*

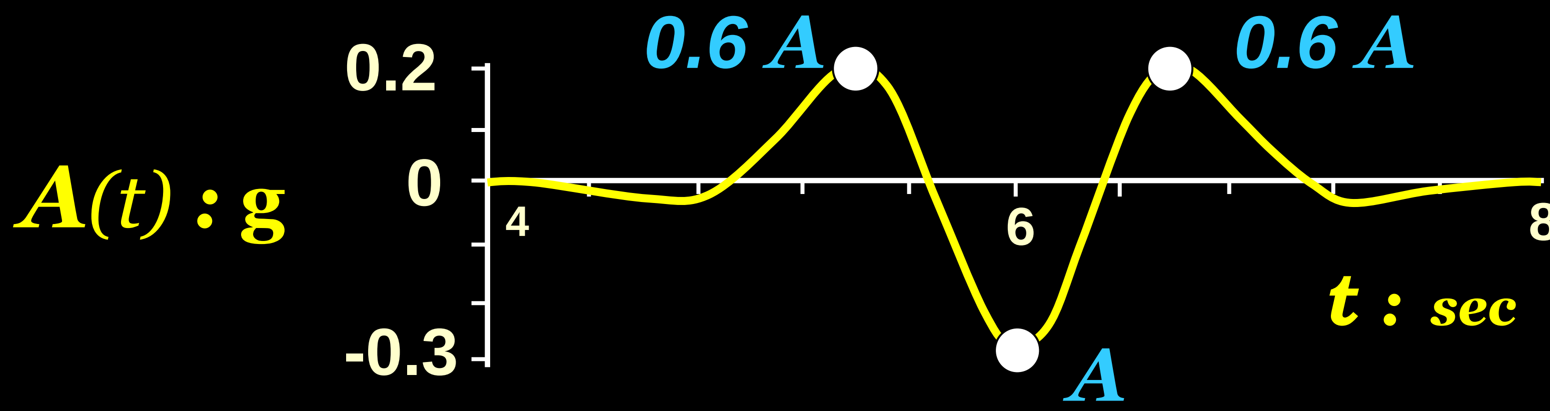


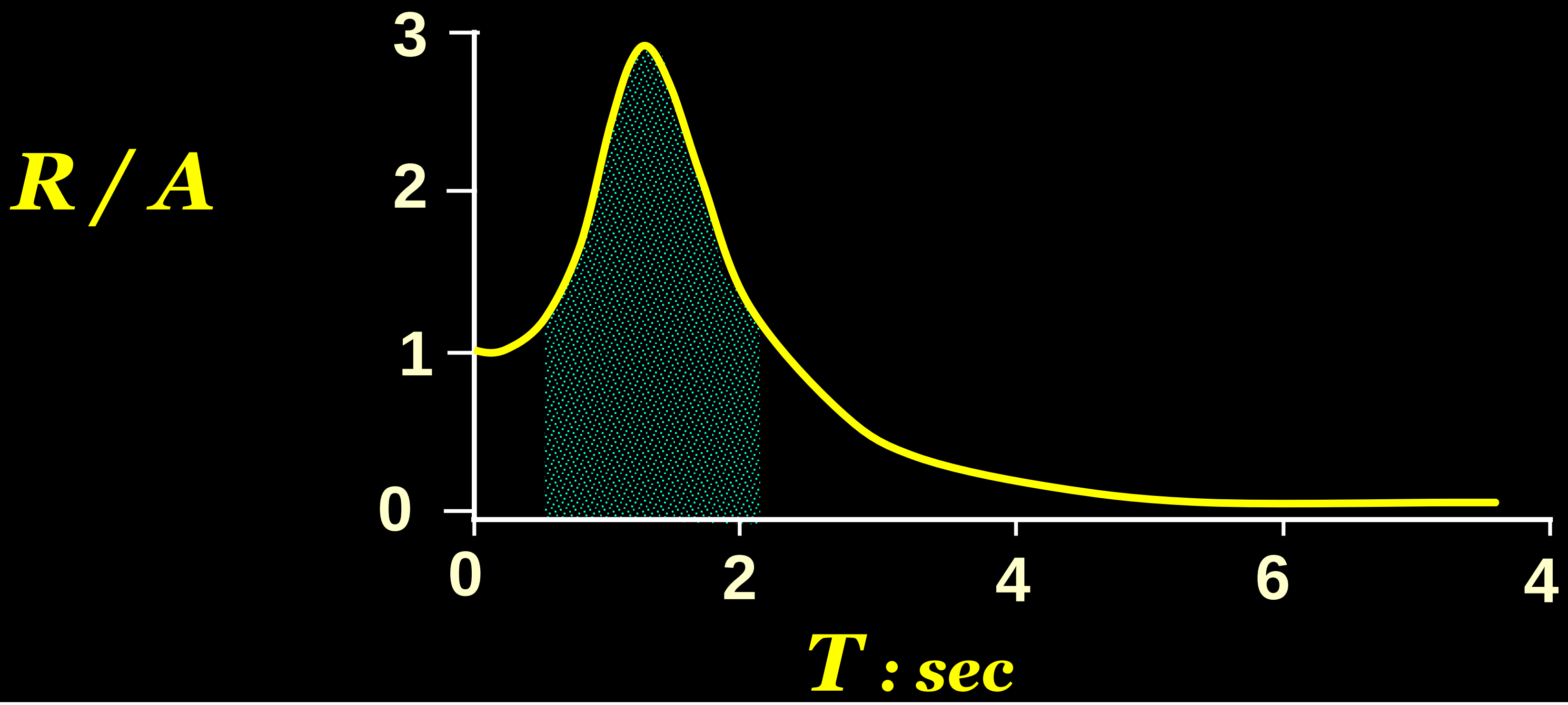


$$(\pi + 2)\, S_u \approx 670\ kPa$$



Ricker Wavelet : f_o = 0.5 Hz and A = 0.30 g



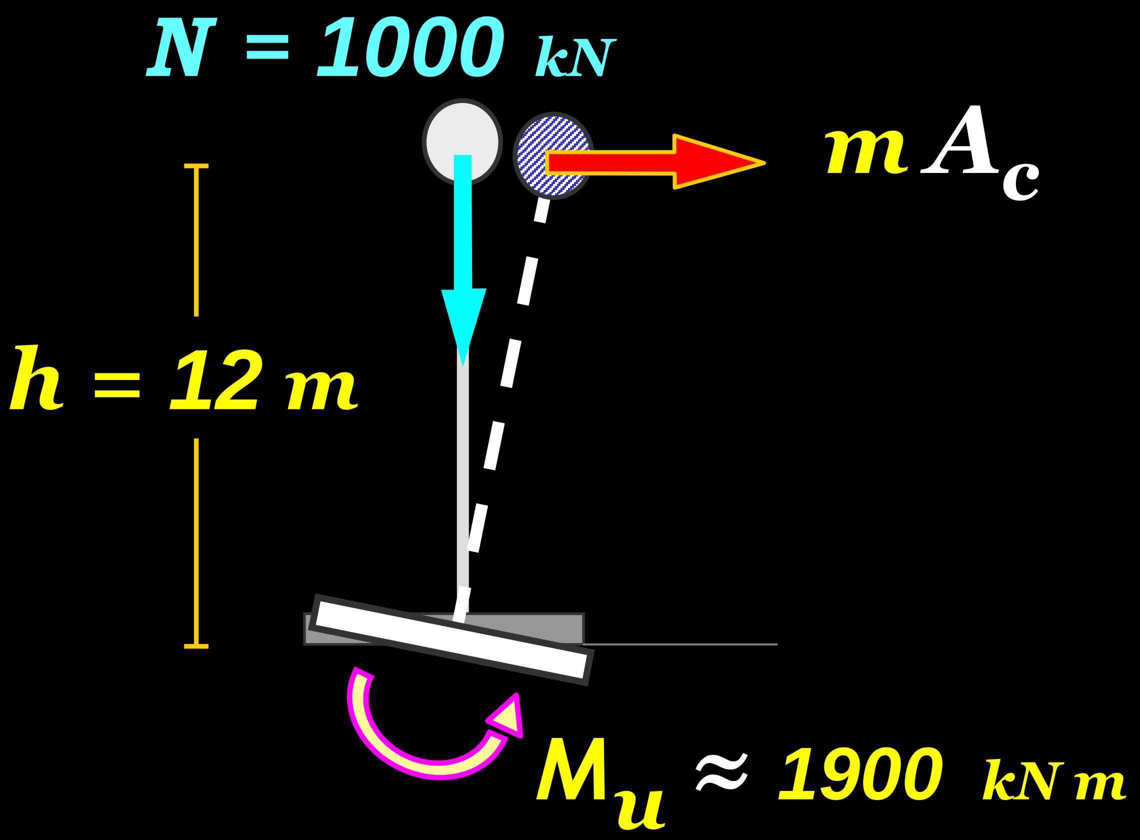


Pseudo – Static Analysis:

Determine the Yield Acceleration

$$A_c$$

(i.e., the maximum possible acceleration of the mass)

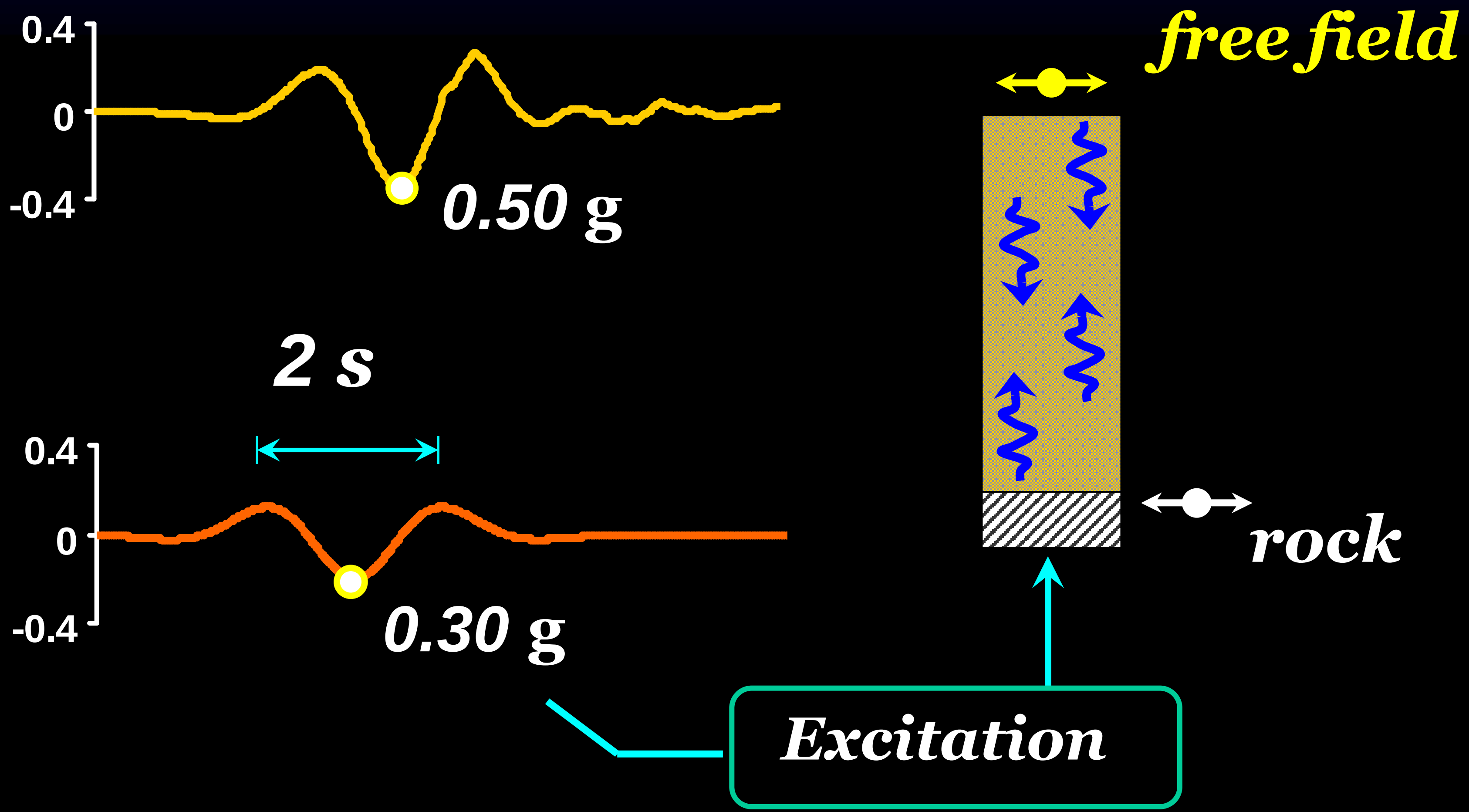


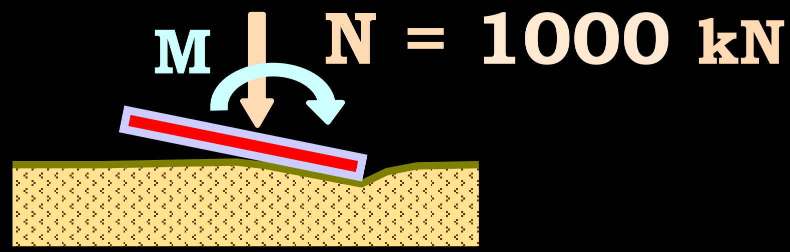
$$M_u = m\,A_c\,h \;\Rightarrow$$

$$A_c = 1900 \,/\, [(1000 : g) \times 12] = 0.16\,g$$

Seismic Analysis

Excitation: Ricker Wavelet





Pseudo–Static Failure : $M = M_u$

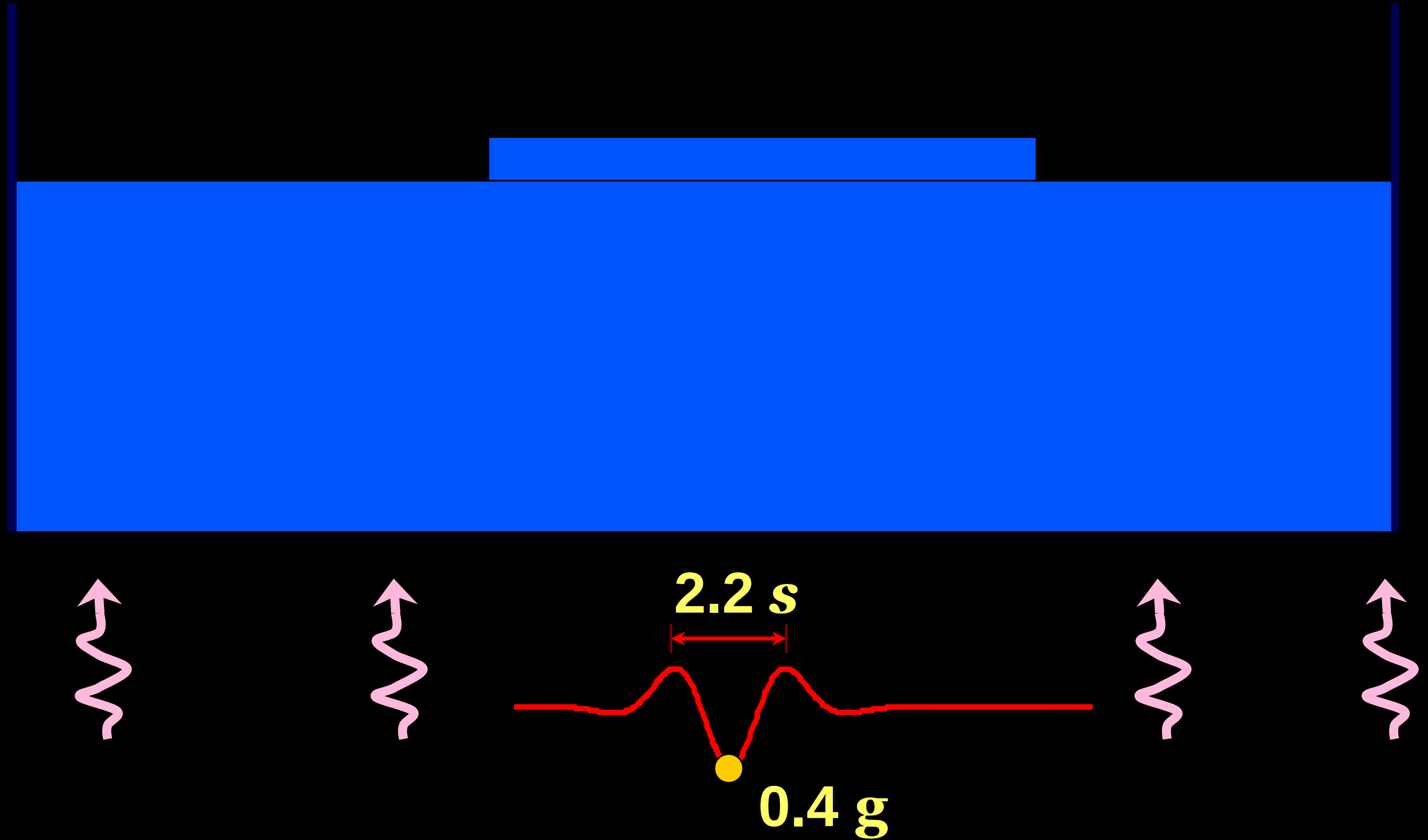
Critical Acceleration : $A_c = 0.16\,g$

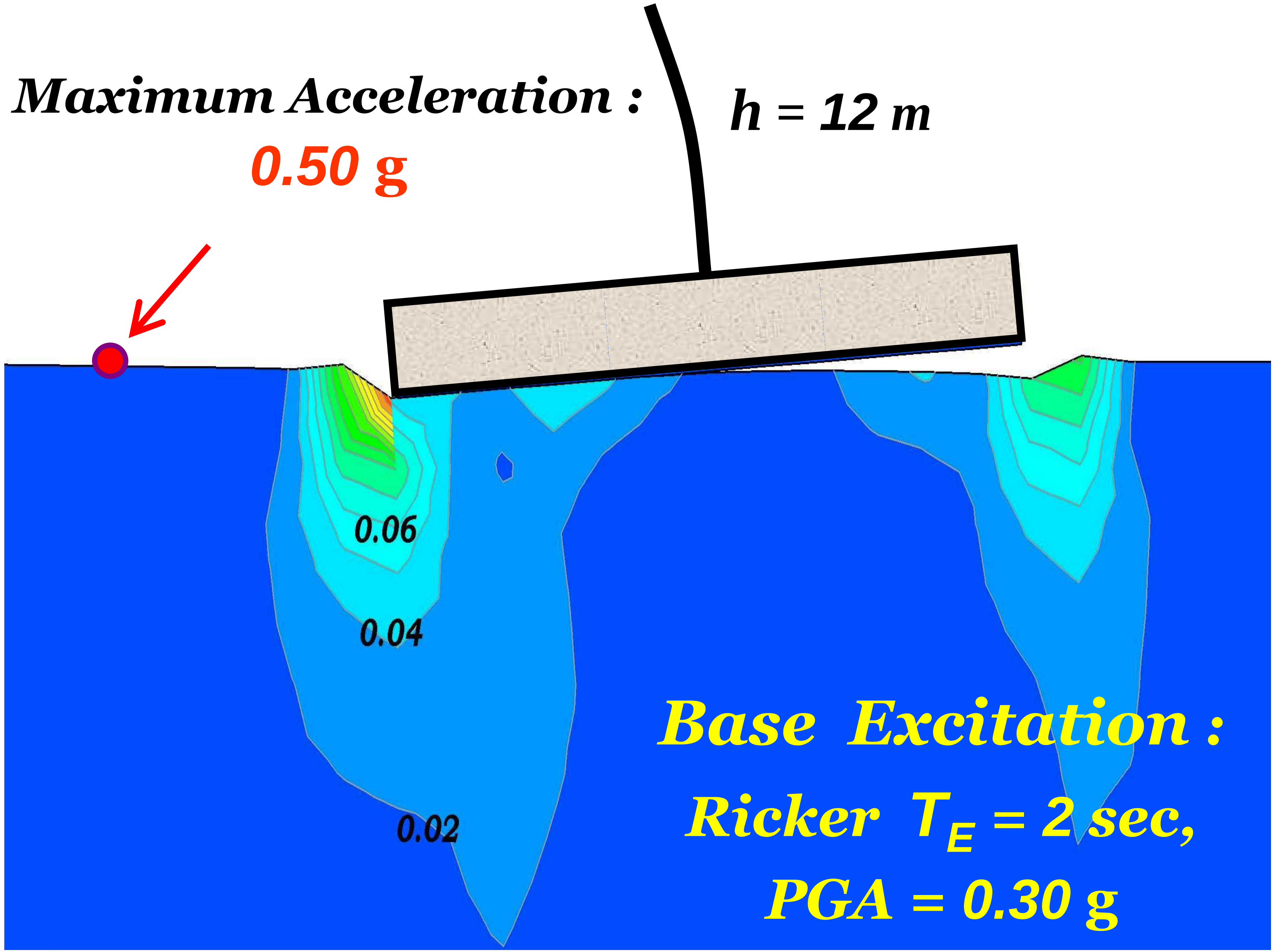
Seismic ("Apparent") Safety Factor :

$$A_c / A = 0.16 / 0.50 = 0.32 \ll 1$$

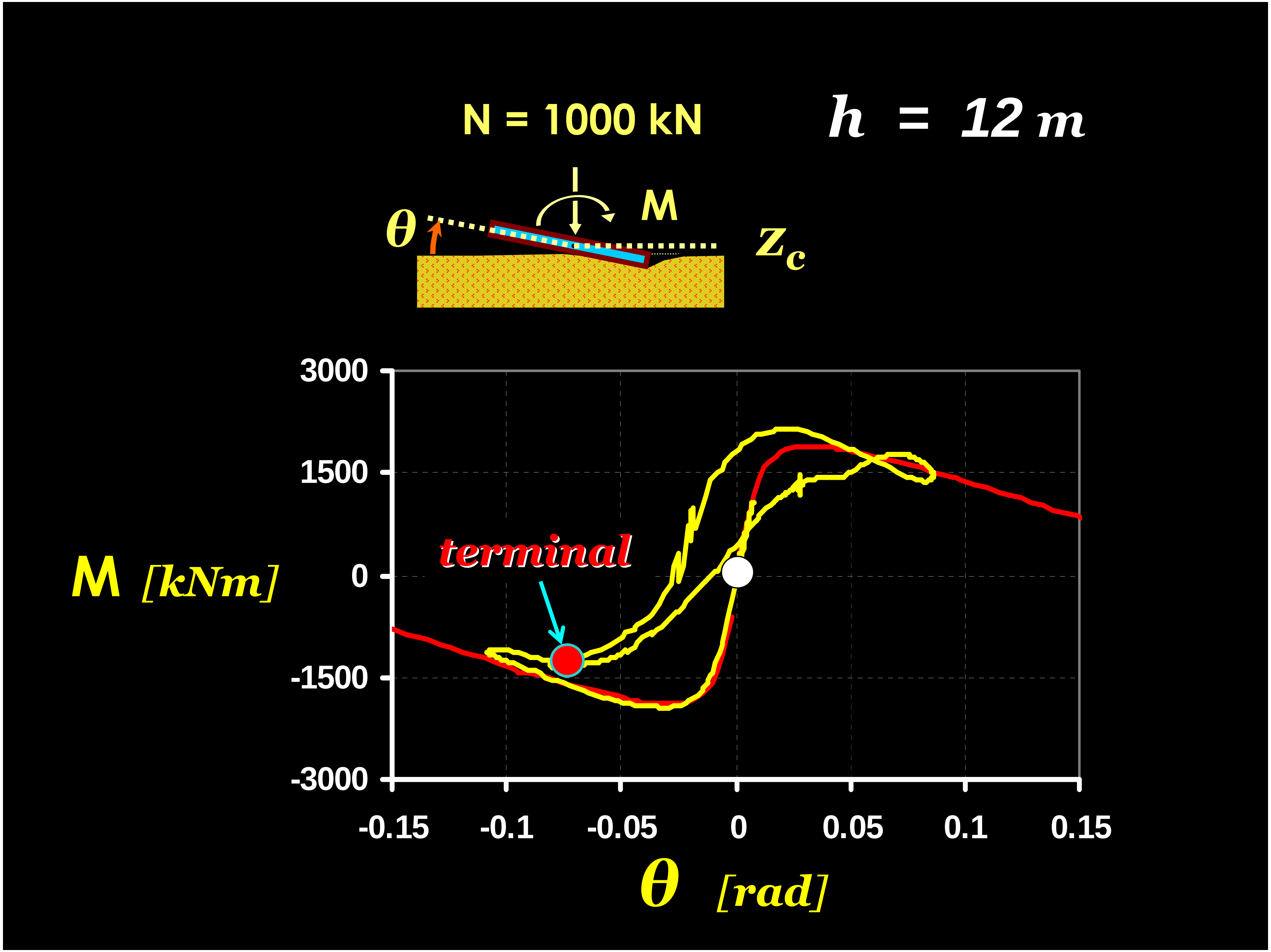
Will there be Failure ??

Contours of Plastic Strains





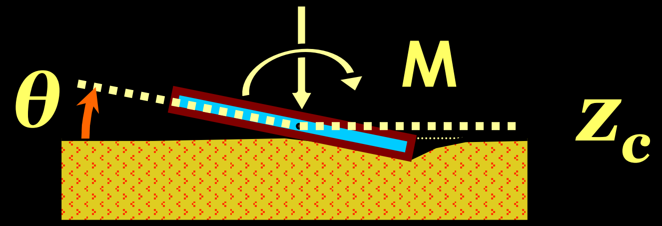
Maximum Acceleration :
0.50 g
h = 12 m
0.06
0.04
0.02
Base Excitation :
Ricker T_E = 2 sec,
PGA = 0.30 g

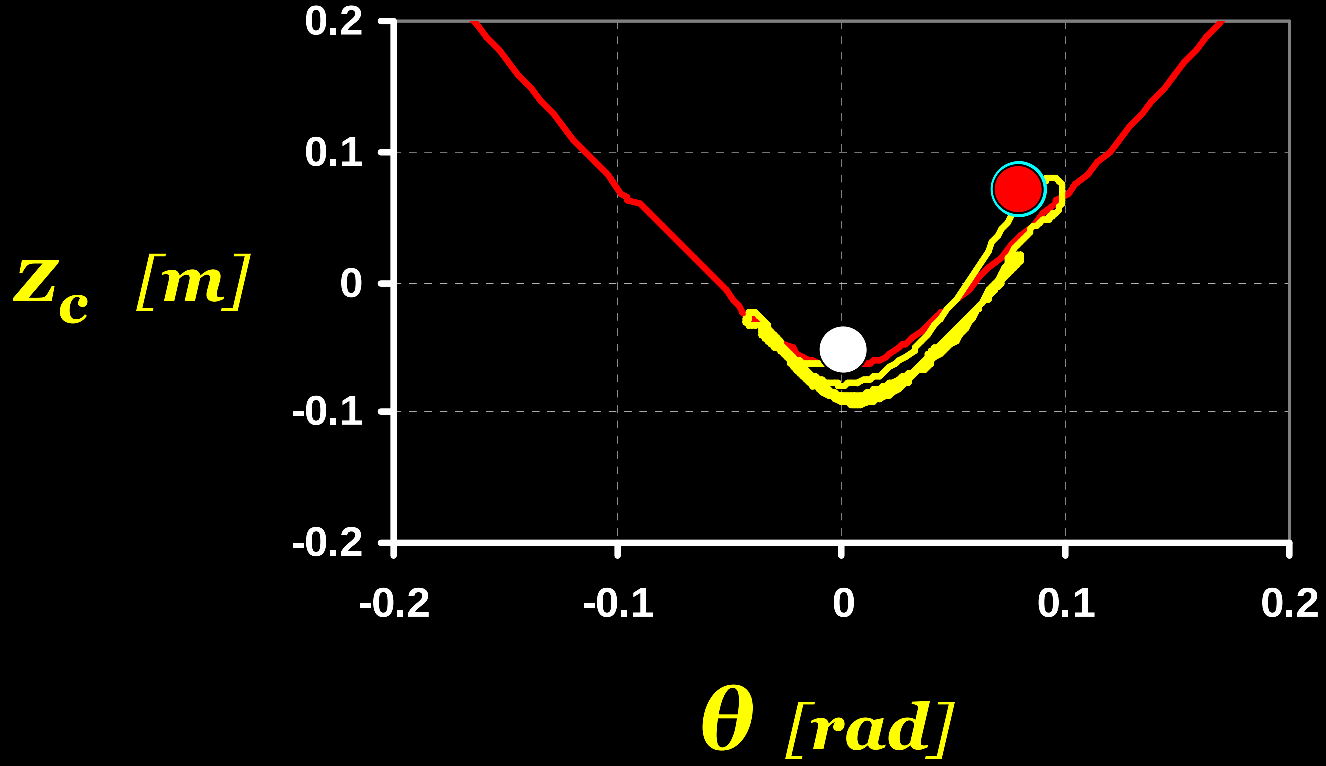


N = 1000 kN
h = 12 m
M
θ
z_c
3000
1500
0
-1500
-3000
M [kNm]
terminal
-0.15
-0.1
-0.05
0
0.05
0.1
0.15
θ [rad]

N = 1000 kN

$h = 12\ m$





Topics of Presentation

PART 1

(a) Why Going to the Limit and "Beyond" in Seismic Foundation Analysis / Design

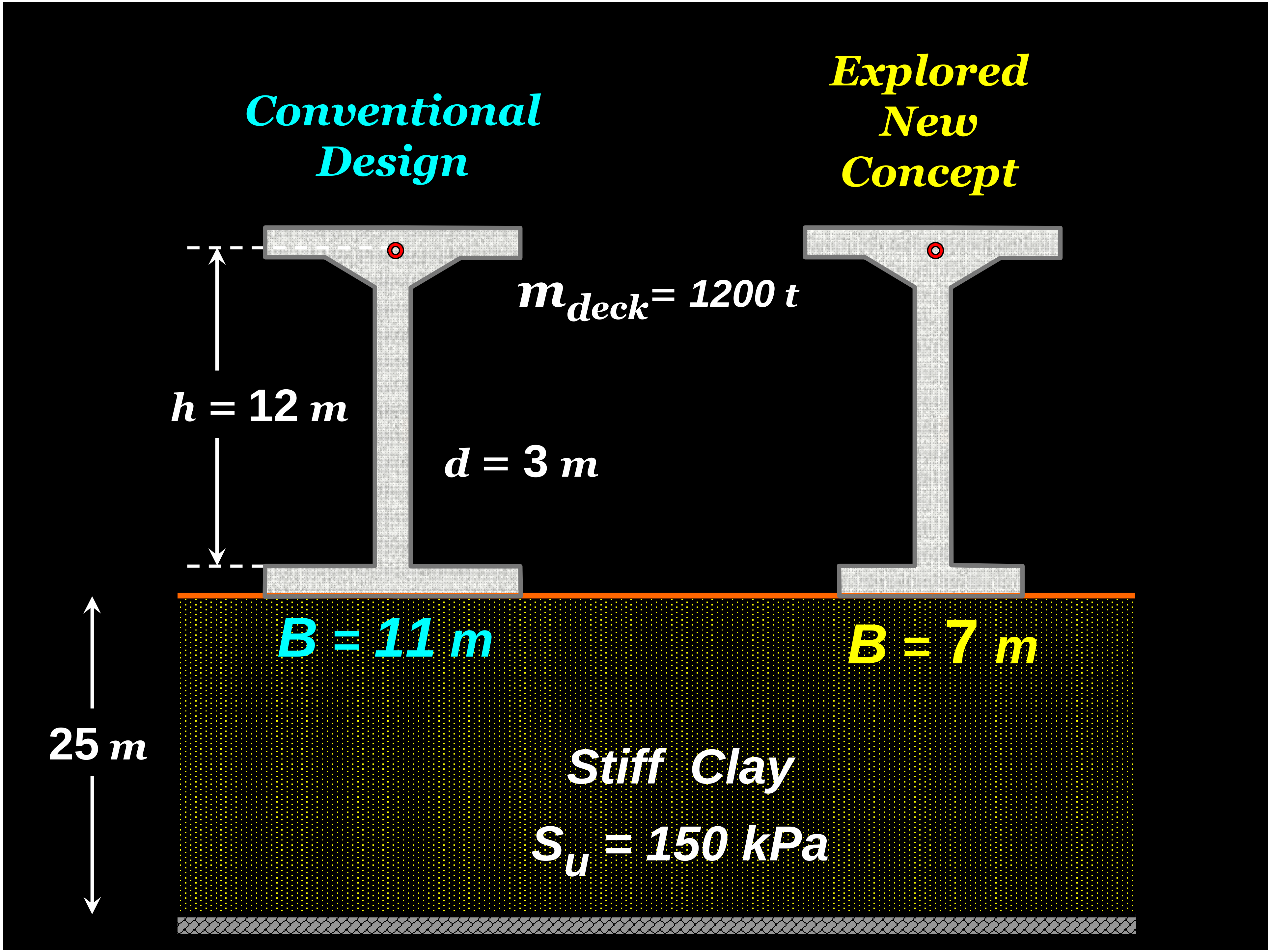
(b) Conventional versus New Design Concept: Bridge Pier Foundation

PART 2

The Causes of Overturning of Buildings in Adapazari (1999)

a Comparative Example:

*To demonstrate the feasibility of designing a foundation to **undergo large deformations**, beyond the conventional wisdom*



Conventional Design
Explored New Concept
m_{deck} = 1200 t
h = 12 m
d = 3 m
B = 11 m
B = 7 m
25 m
Stiff Clay
S_u = 150 kPa

Pseudo-Static Safety Factors for a Seismic Coefficient $C_s = R_d/q = 0.30$

$B = 11\ m \Rightarrow$

$FS_V = 5.8$

$FS_E = 2.0$

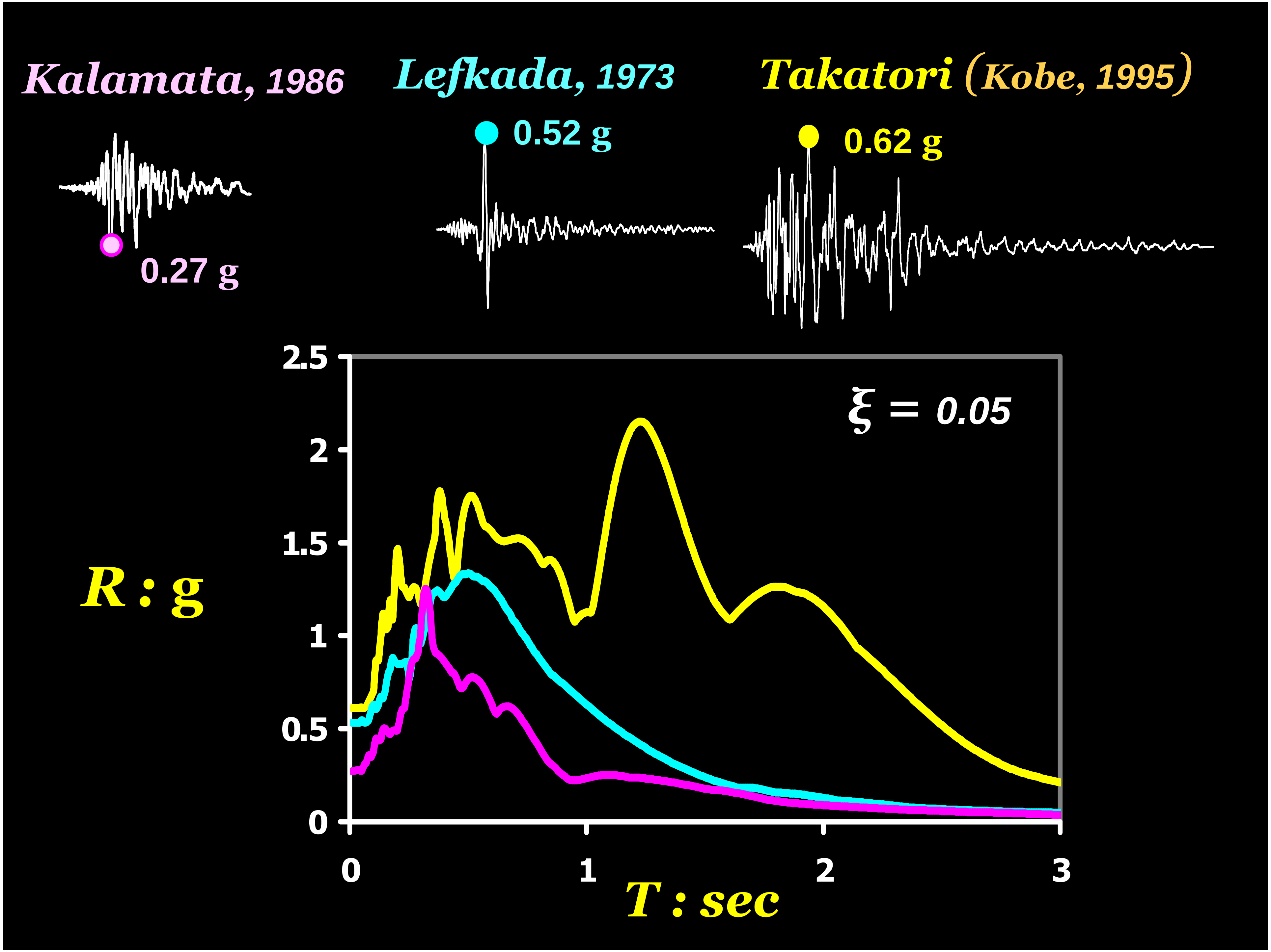
$e < B/3$

$B = 7\ m \Rightarrow$

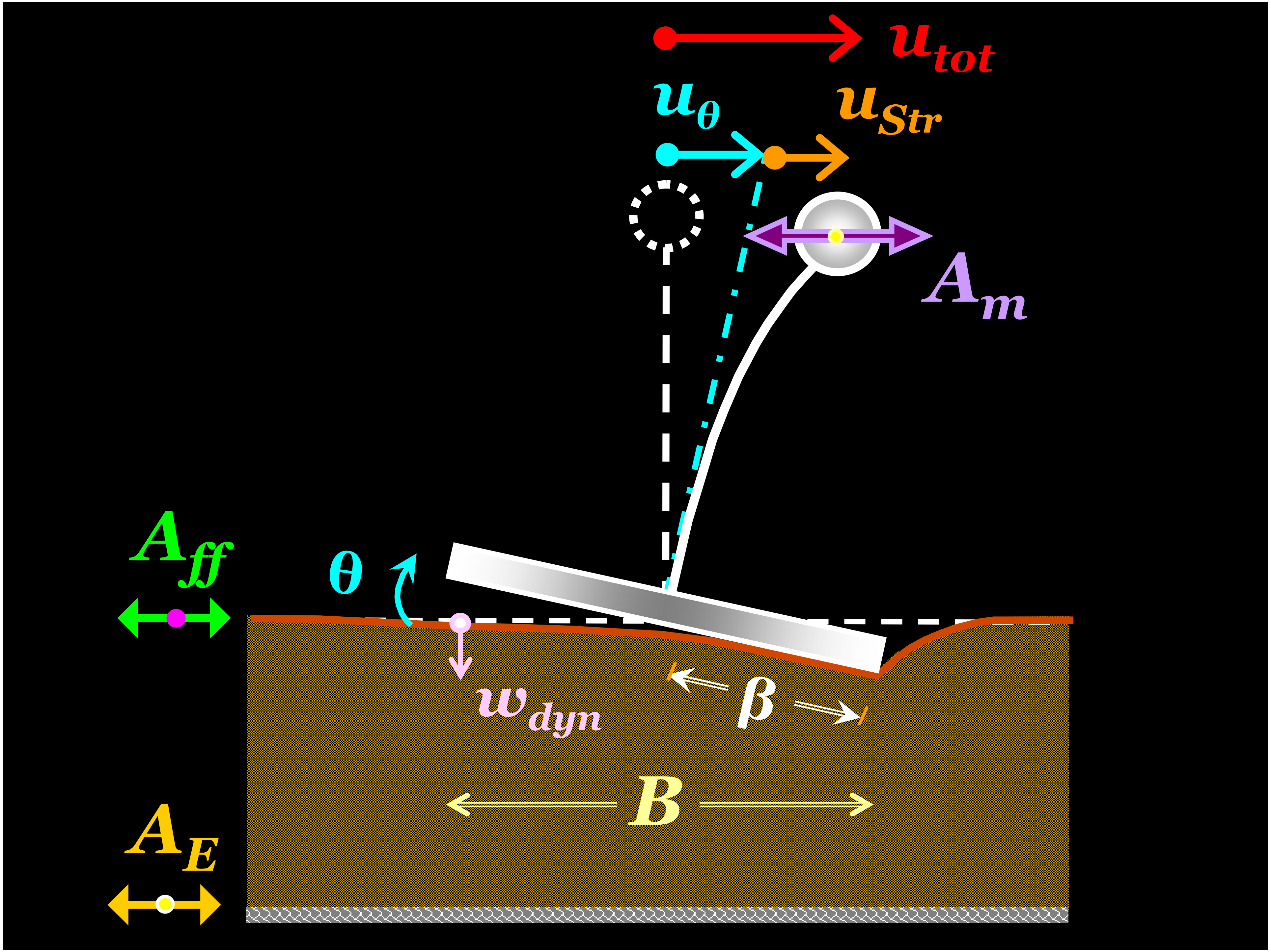
$FS_V = 2.8$

$FS_E = 0.5 < 1$

$e > B/3$



Kalamata, 1986
Lefkada, 1973
Takatori (Kobe, 1995)
0.27 g
0.52 g
0.62 g
ξ = 0.05
R : g
T : sec
2.5
2
1.5
1
0.5
0
0
1
2
3



u_{tot}
u_θ
u_{Str}
A_m
A_{ff}
θ
w_{dyn}
β
B
A_E

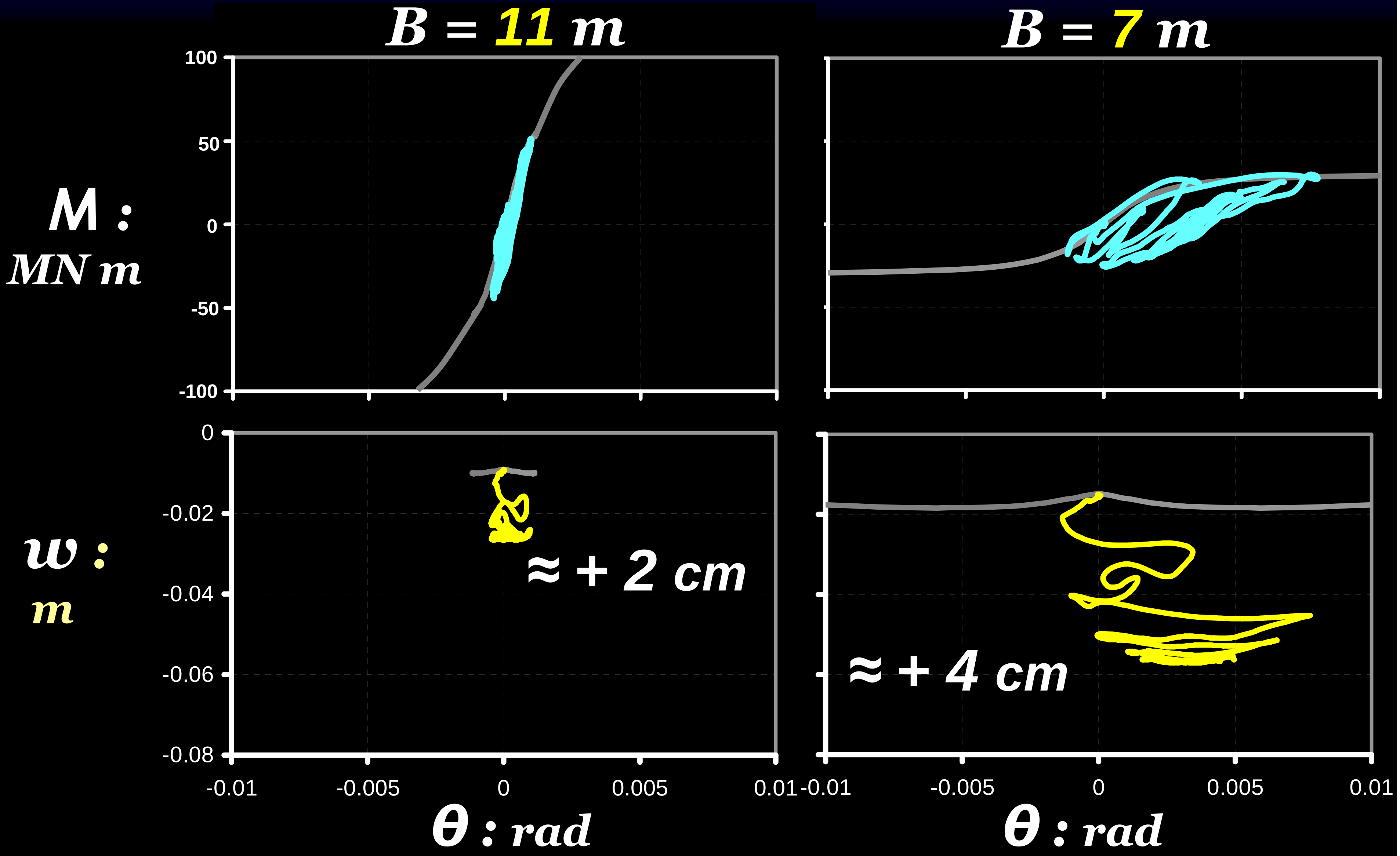
Low-Intensity Shaking: Kalamata 1986

RESULTS for the RESPONSE of the:

(a) FOUNDATION

(b) STRUCTURE – FOUNDATION System

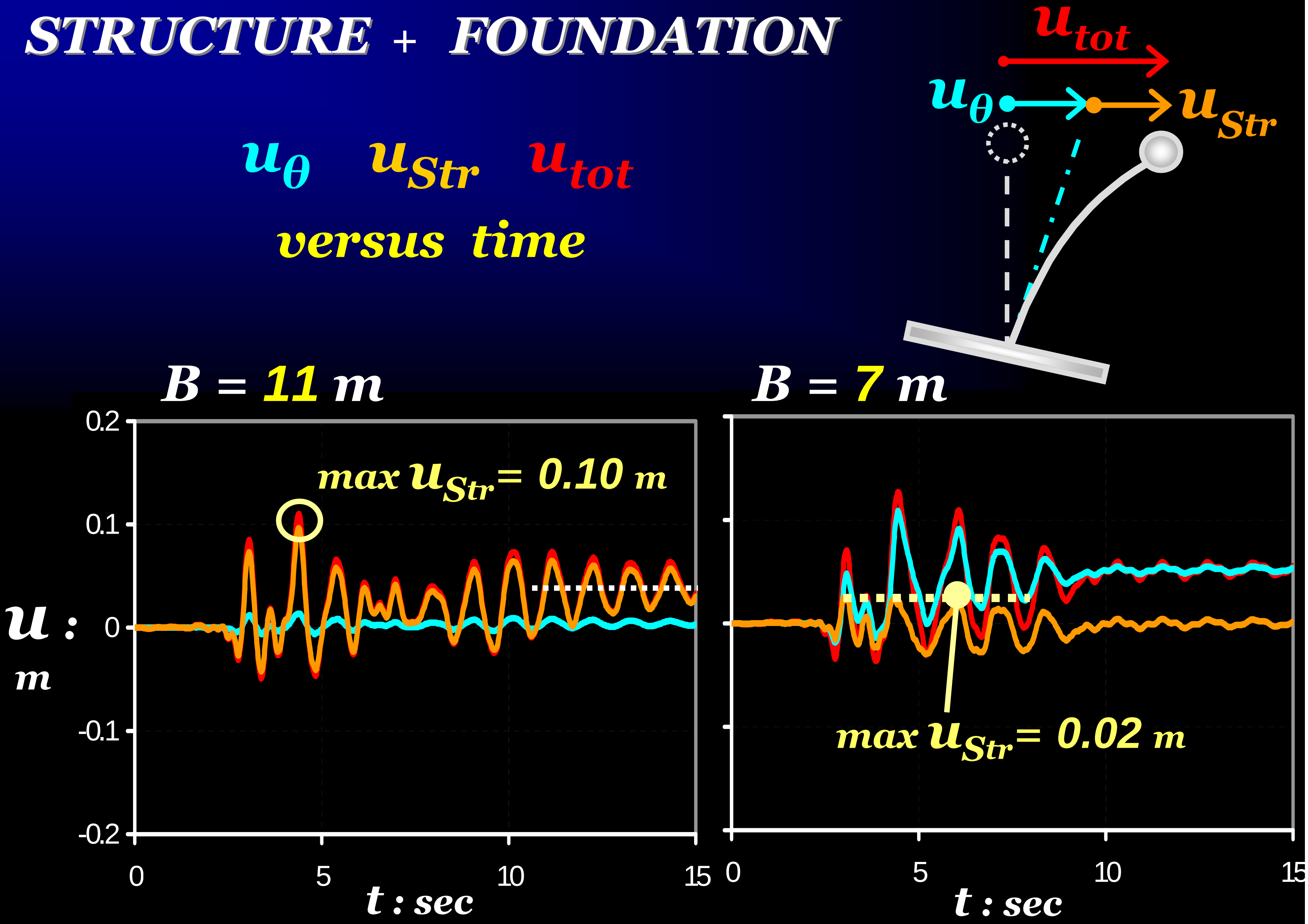
FOUNDATION ($M - \theta$ and $w - \theta$)



STRUCTURE + FOUNDATION

u_θ u_{Str} u_{tot}

versus time



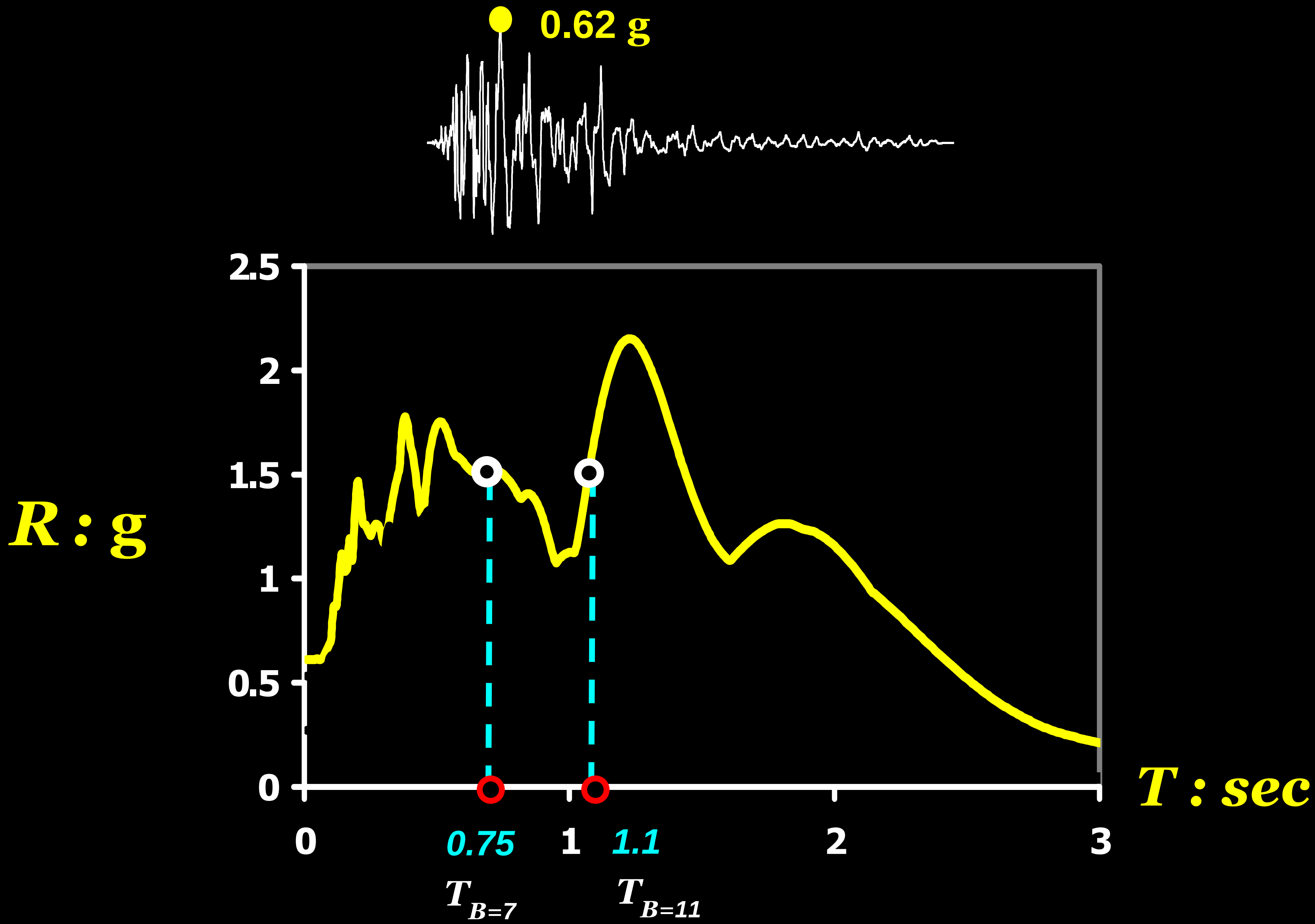
Very Strong Shaking:
Takatori (Kobe 1995)

RESULTS for the RESPONSE of the:

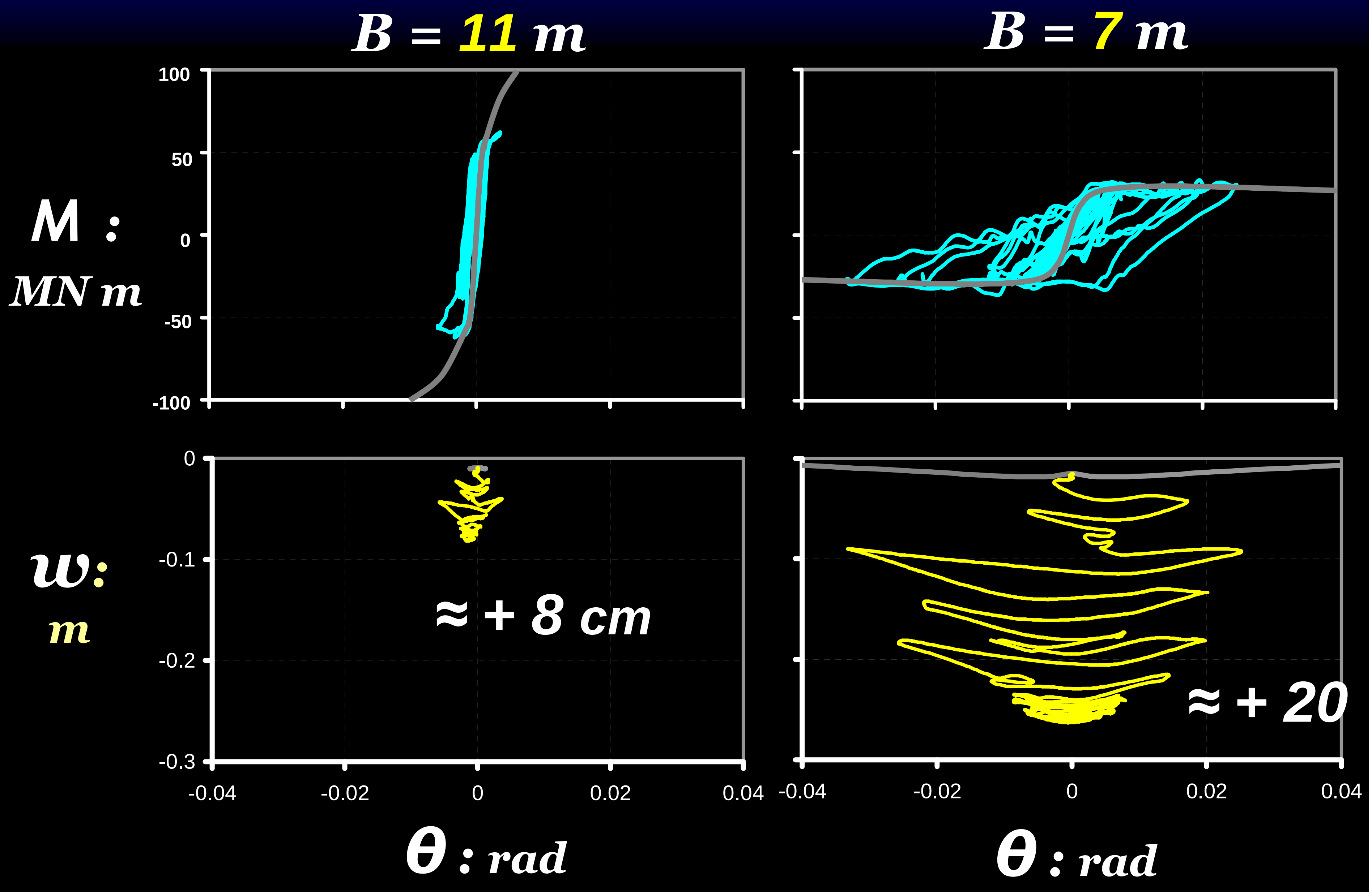
(a) FOUNDATION

(b) STRUCTURE – FOUNDATION System

Takatori (Kobe, 1995)



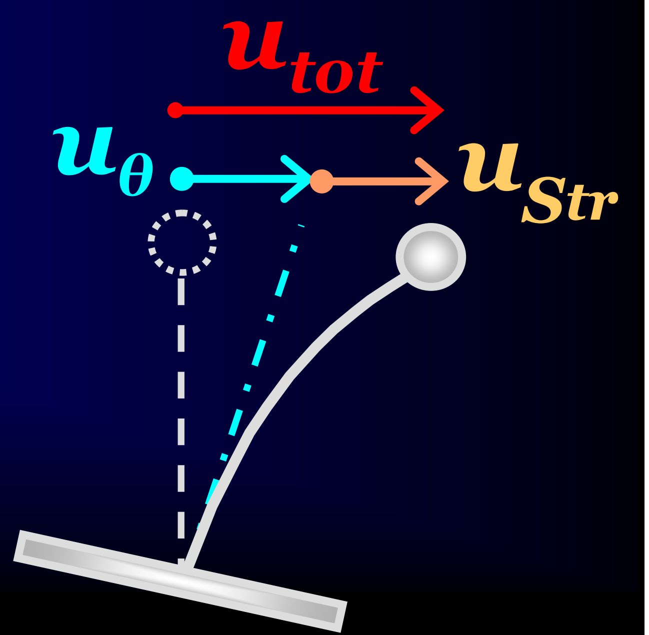
FOUNDATION ($M - \theta$ and $w - \theta$)

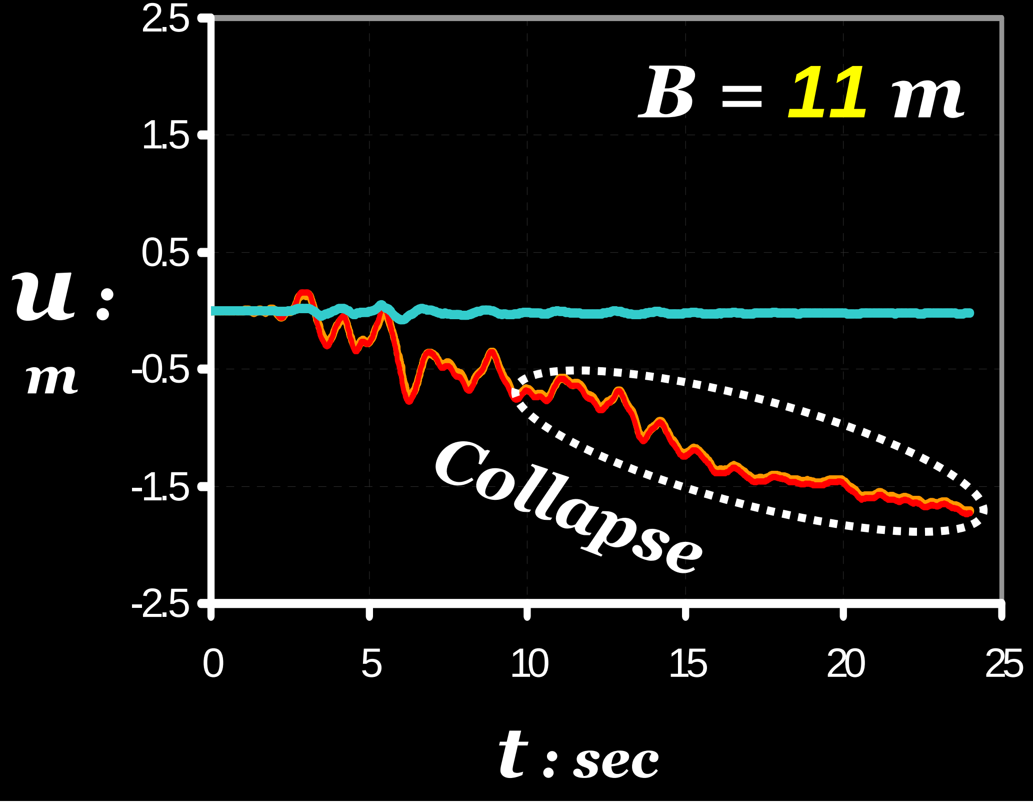


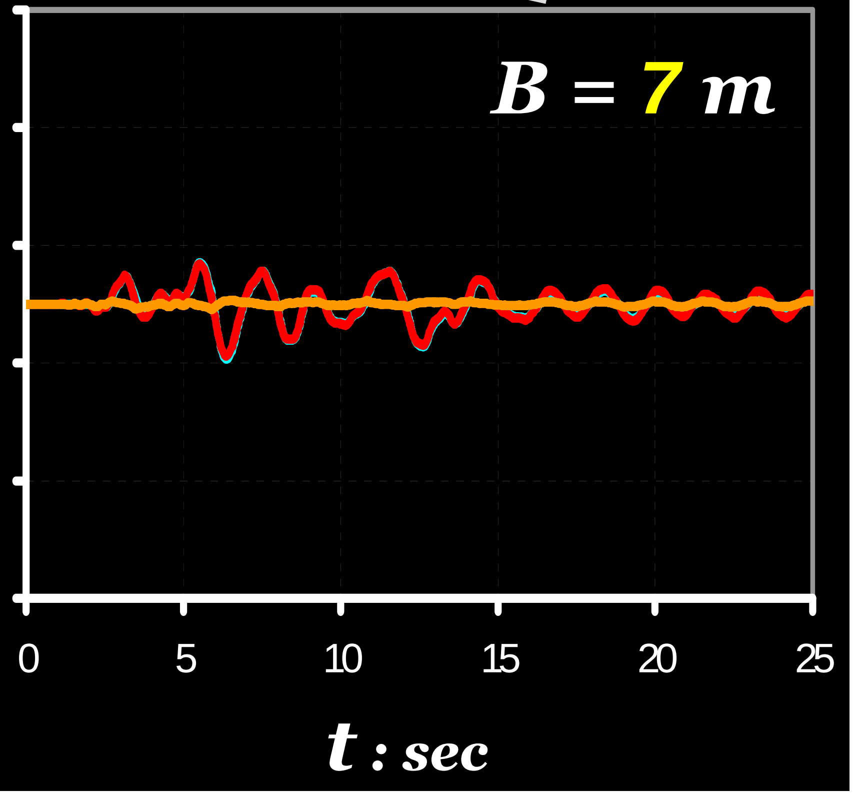
STRUCTURE + FOUNDATION

u_θ u_{Str} u_{tot}

versus time



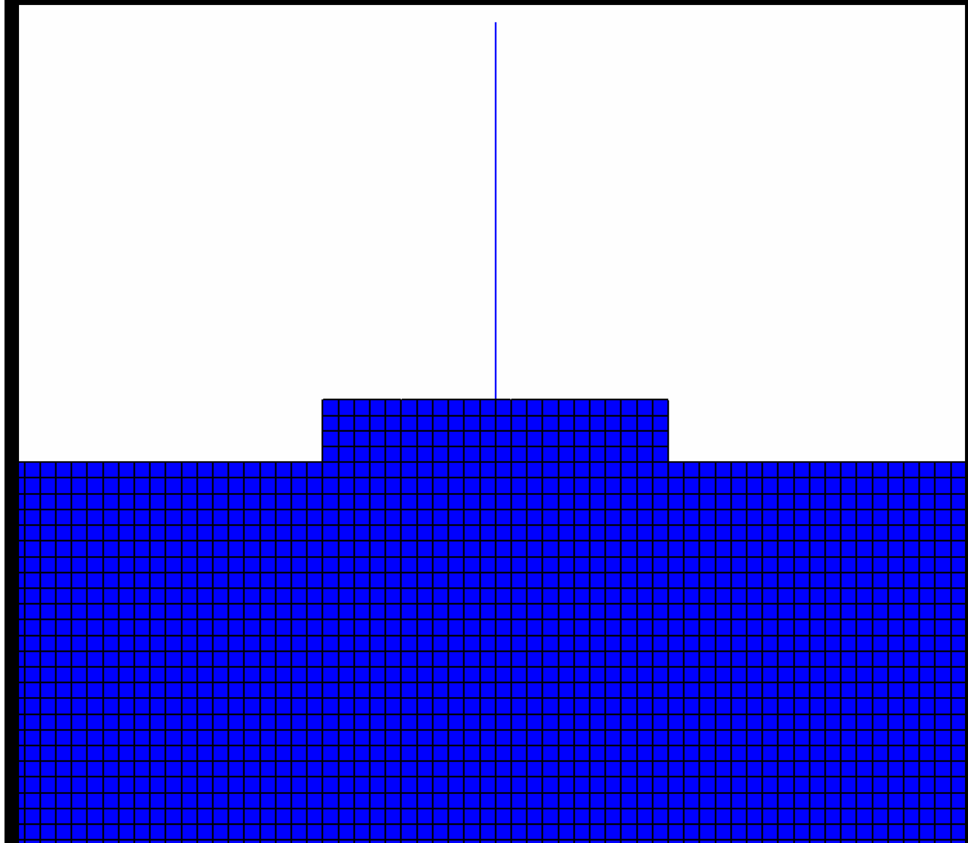


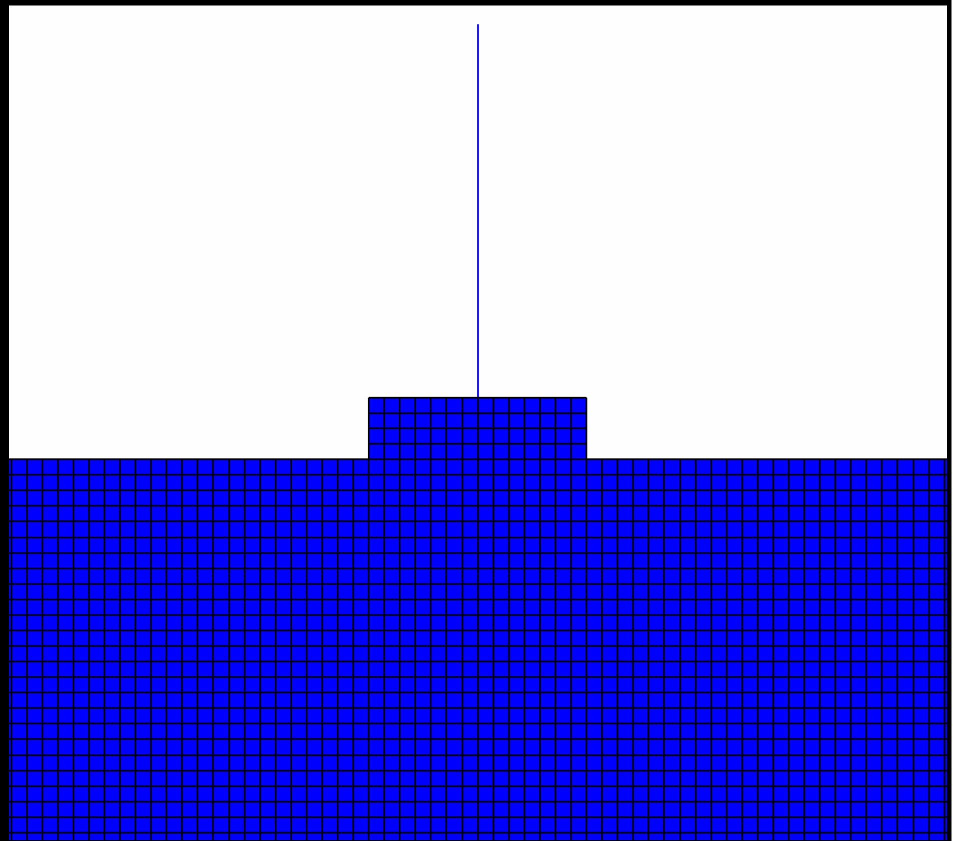


Comparison of the two Foundation Schemes

Takatori (Kobe, 1995)

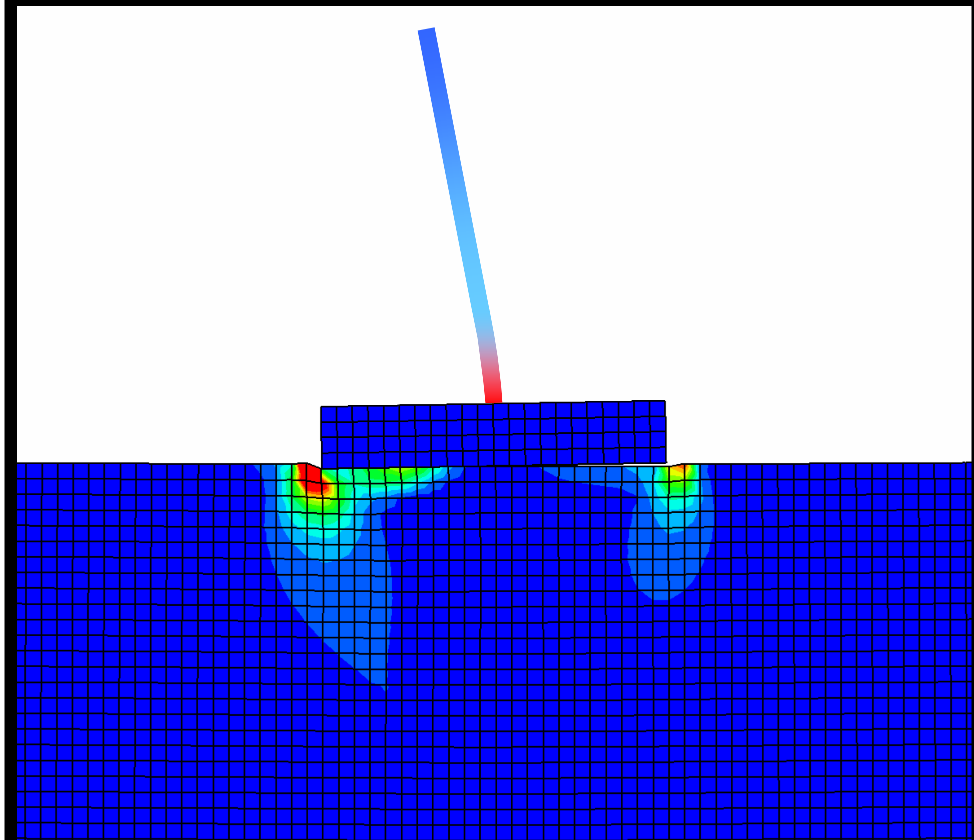
B = 11 m **B = 7 m**



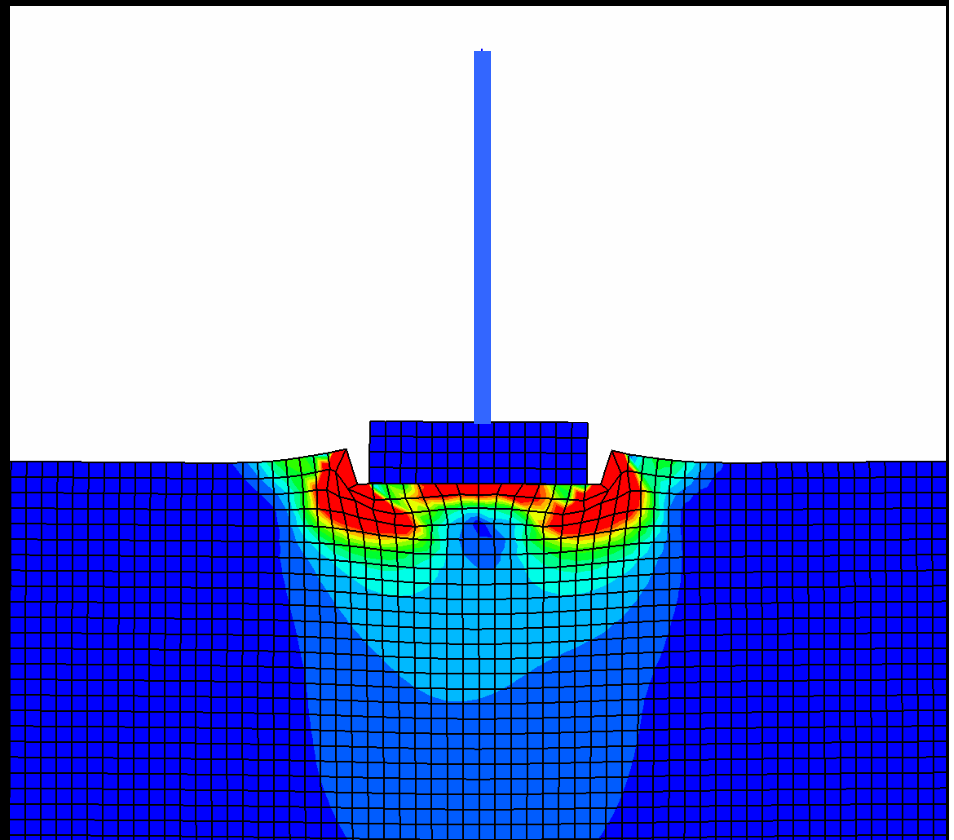


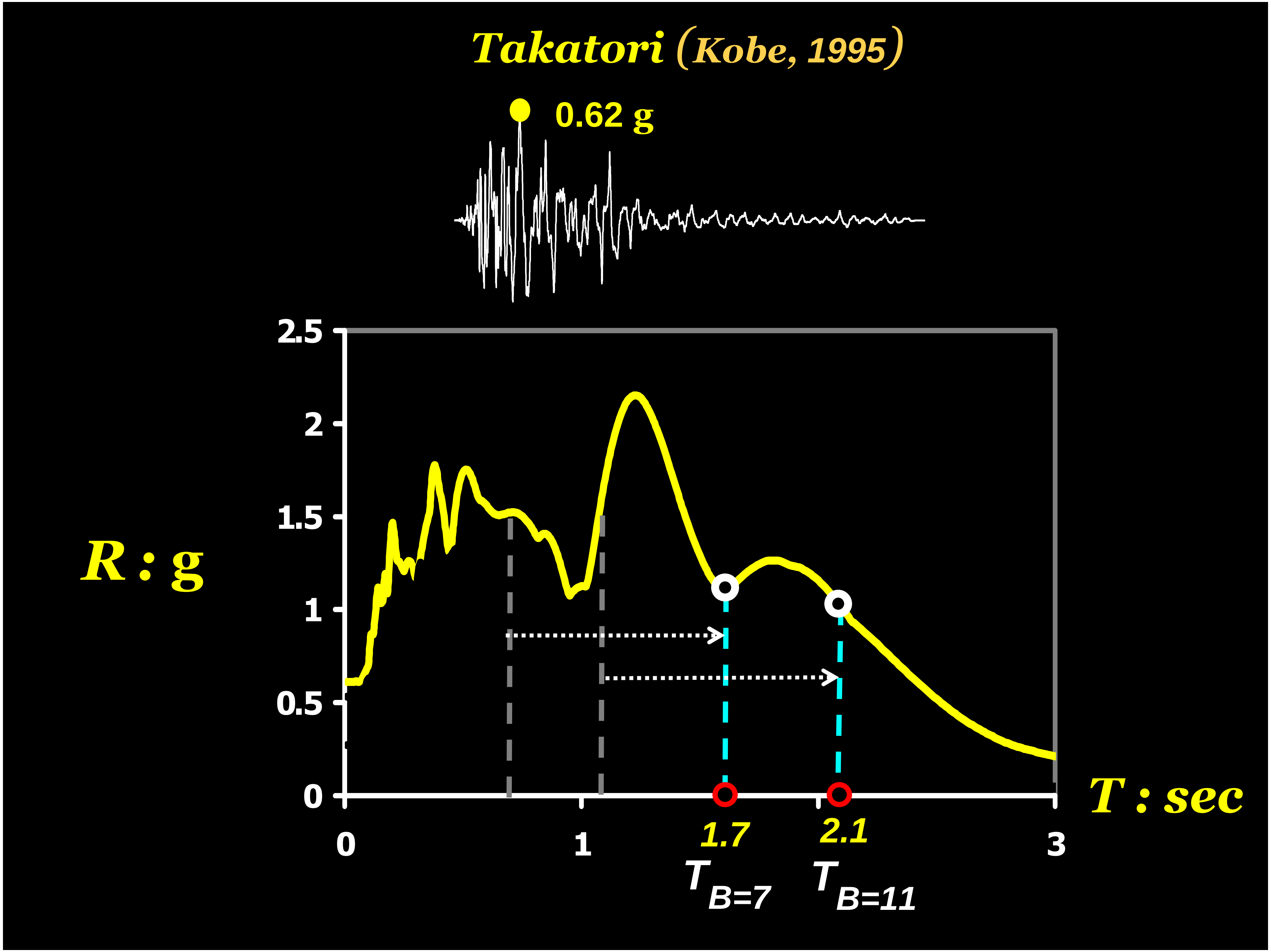
The Final Stage

B = 11 m



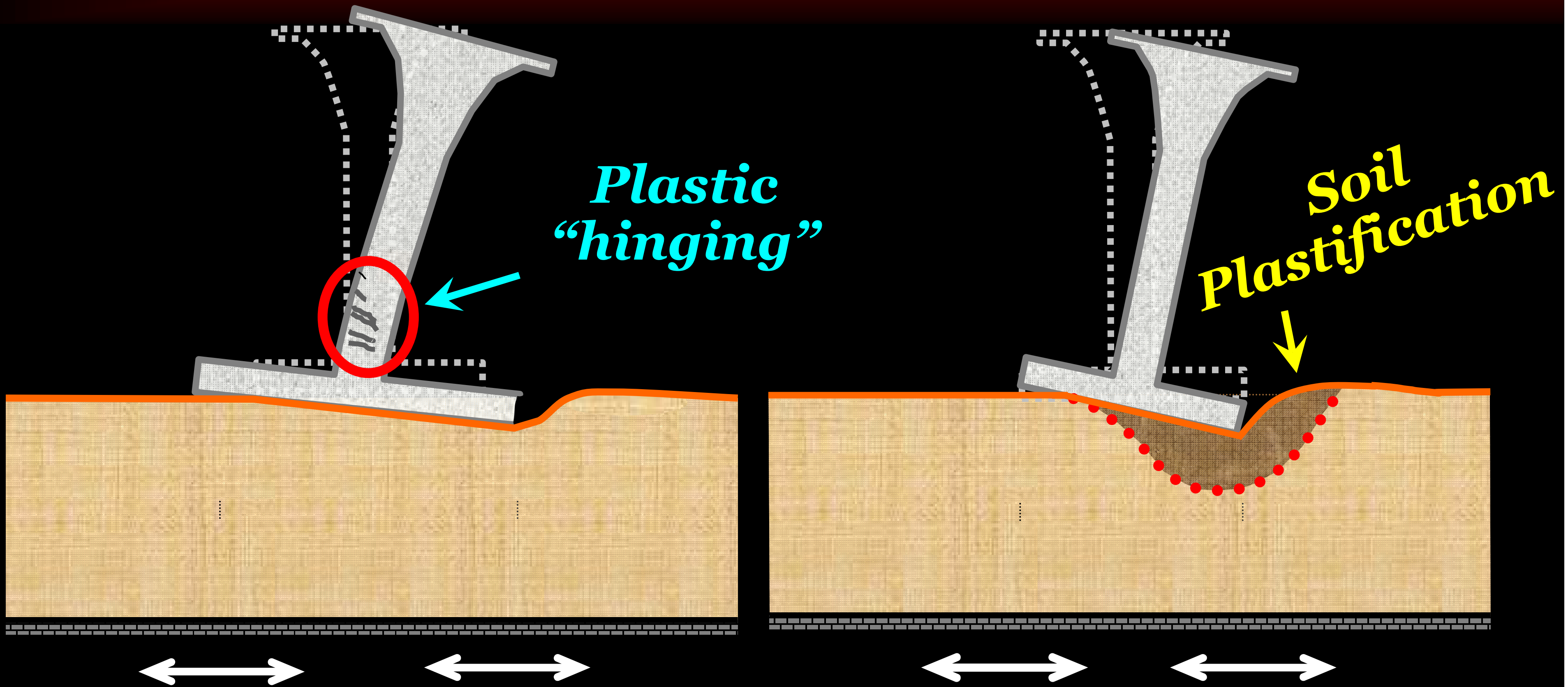
B = 7 m





Takatori (Kobe, 1995)
0.62 g
R : g
T : sec
2.5
2
1.5
1
0.5
0
0
1
1.7
2.1
3
T B=7
T B=11

Conclusion



Conventional Solution:
"Capacity" Design

New Concept:
Beyond "Capacity" Design

The Main Conclusions of Part 1

(α) The Dynamic behaviour of inelastic soil–foundation systems differs from the Pseudo-Static response.

Sometimes dramatically

(β) New Design Approach ("Philosophy") :

Plastification

not only in the structure

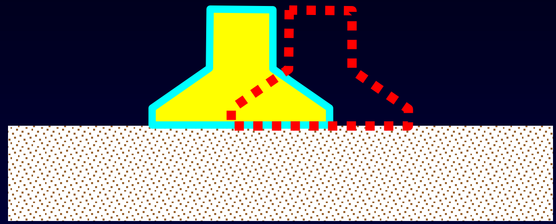
but also :

⇒ ***at the soil–footing interface***
(sliding, detachment + uplifting)

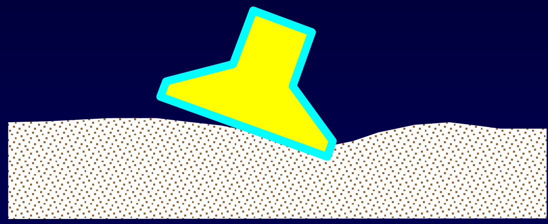
⇒ ***in the supporting soil***
(bearing–capacity mechanism)

(c) The following should NOT be a priori "forbidden" modes

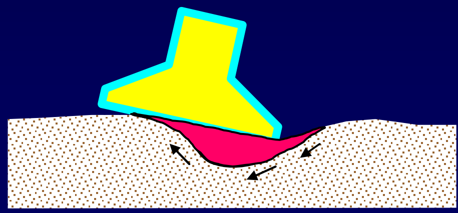
- *Sliding at the Interface*



- *Detachment–Uplifting*



- *Mobilisation of "Bearing–Capacity" Mechanisms*



FIN

Partie 1

The Overturning of Buildings in Adapazari

1999 Izmit [Kocaeli] Earthquake

*“ Even the **most refined theories** (before they can be established) must be **VALIDATED** by **COMPARISONS** against the **REALITY** that these theories describe … ”*

The Geography of the 1999 Earthquakes

ADAPAZARI : 17 – 8 – 1999

(α) Seismology; Damage distribution

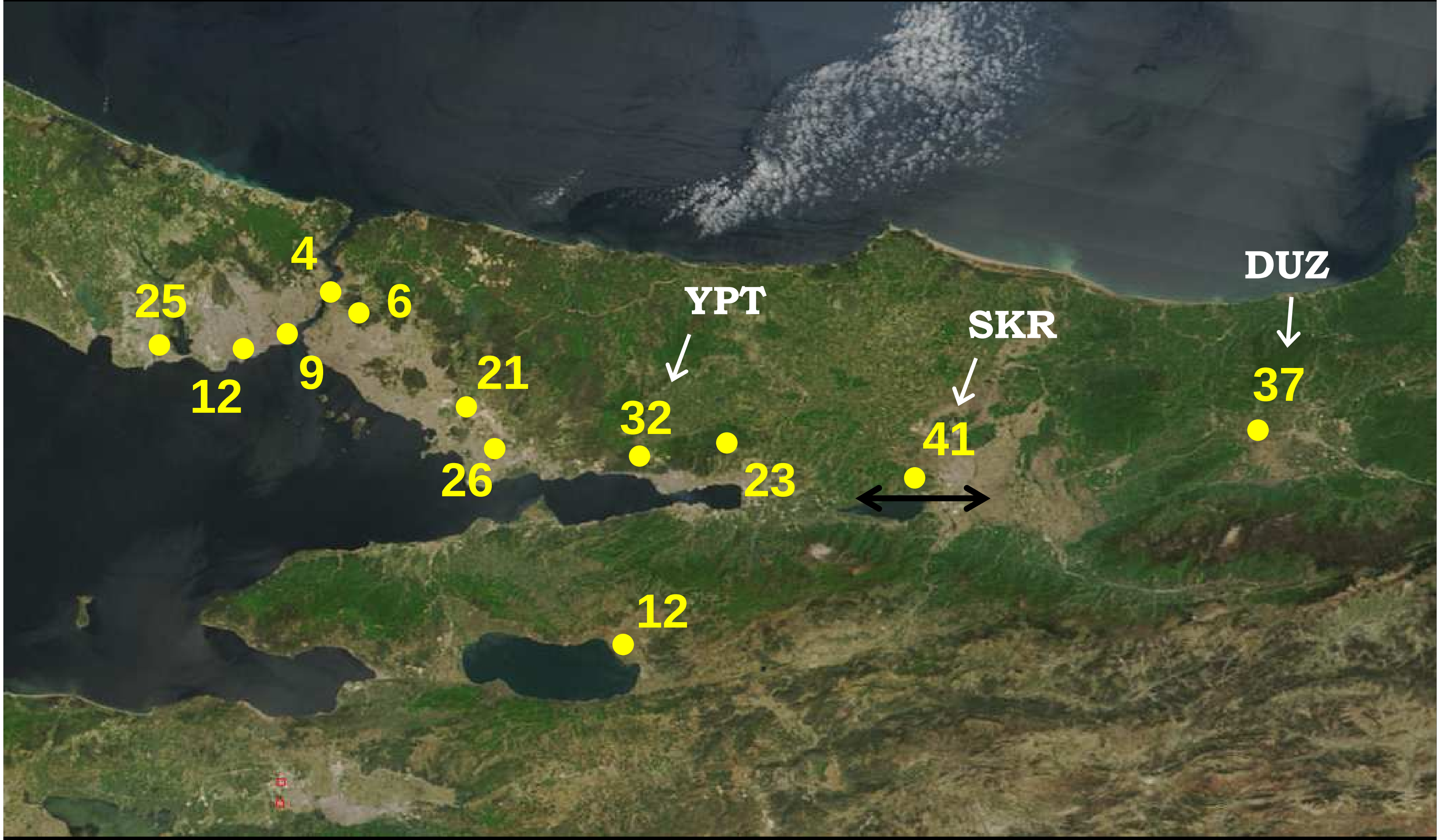
(b) Soil: the disputed role of fines

(c) Soil: wave propagation analysis

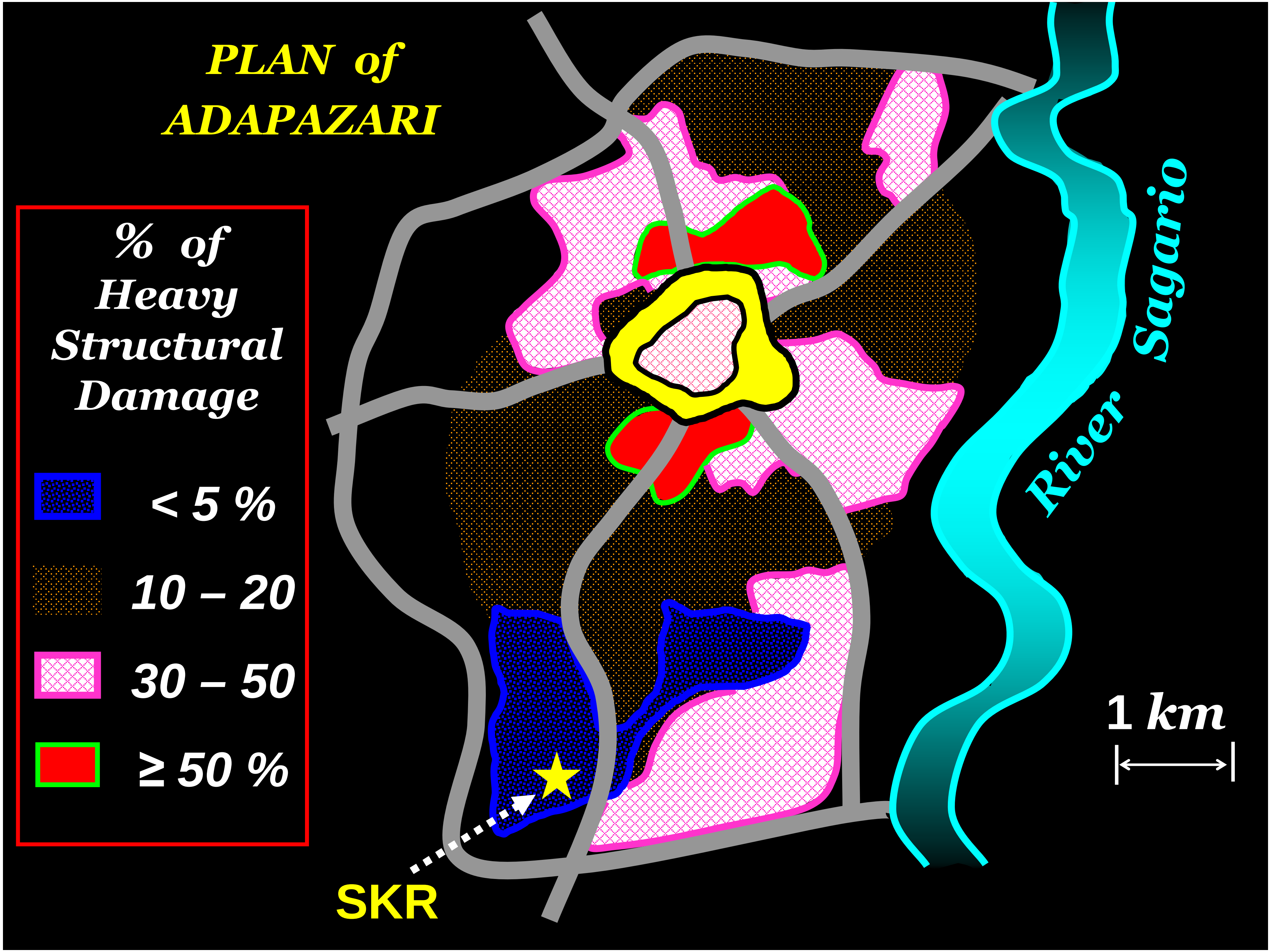
(d) Seismic Analysis of Building Response
Explanation of Toppling, Settlement

PGA (% g)

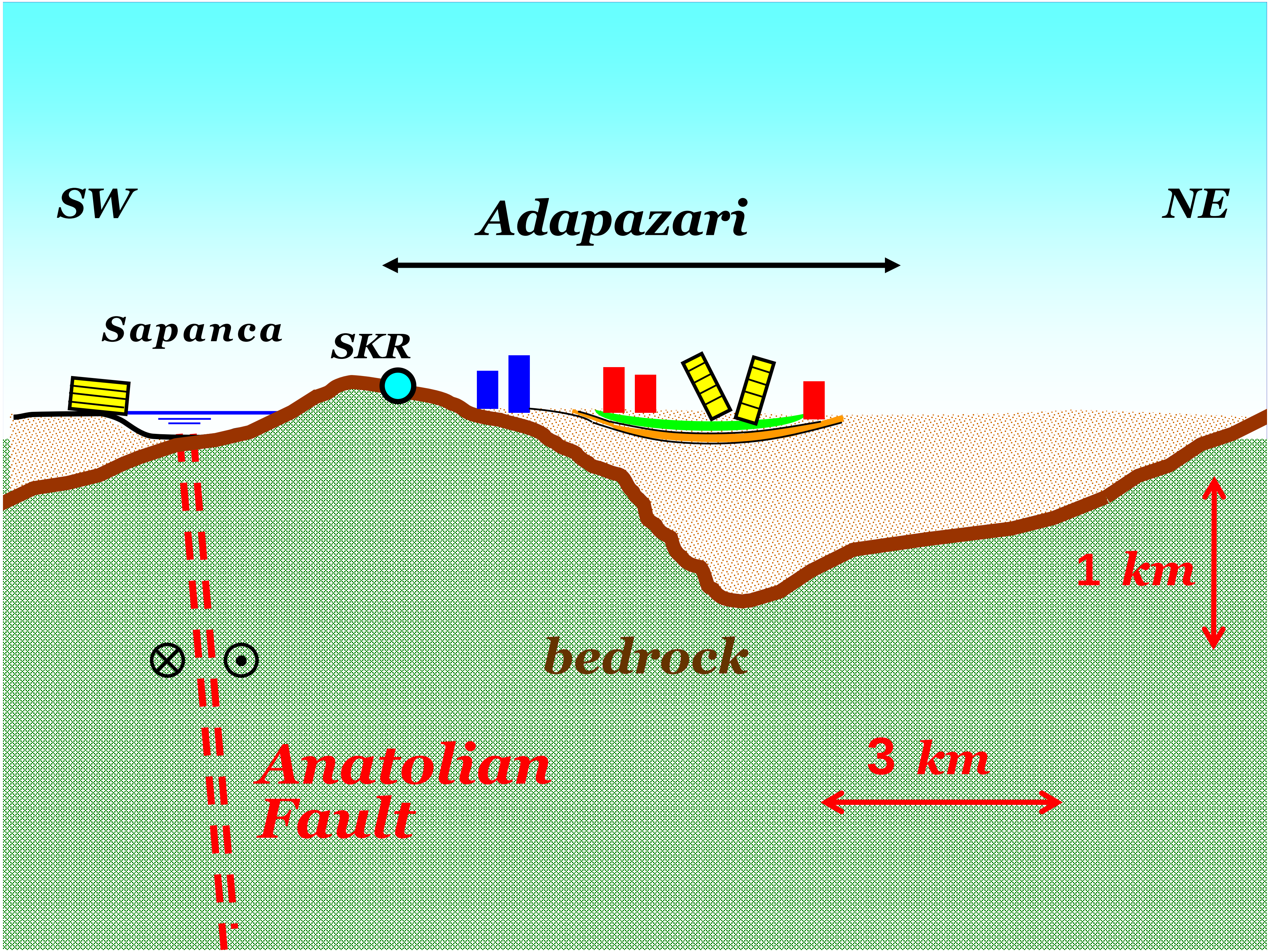
Izmit (Kocaeli) 17–8–99



$41 \;=\; 0{,}41\ \mathrm{g} \;\approx\; 4{,}1\ m/s^2$



PLAN of
ADAPAZARI
% of
Heavy
Structural
Damage
< 5 %
10 – 20
30 – 50
≥ 50 %
River Sagario
1 km
SKR



SW
Adapazari
NE
Sapanca
SKR
1 km
bedrock
Anatolian
Fault
3 km


SİTESİ
BULVAR SİTESİ


ATASOY
TAŞINMIŞTIR

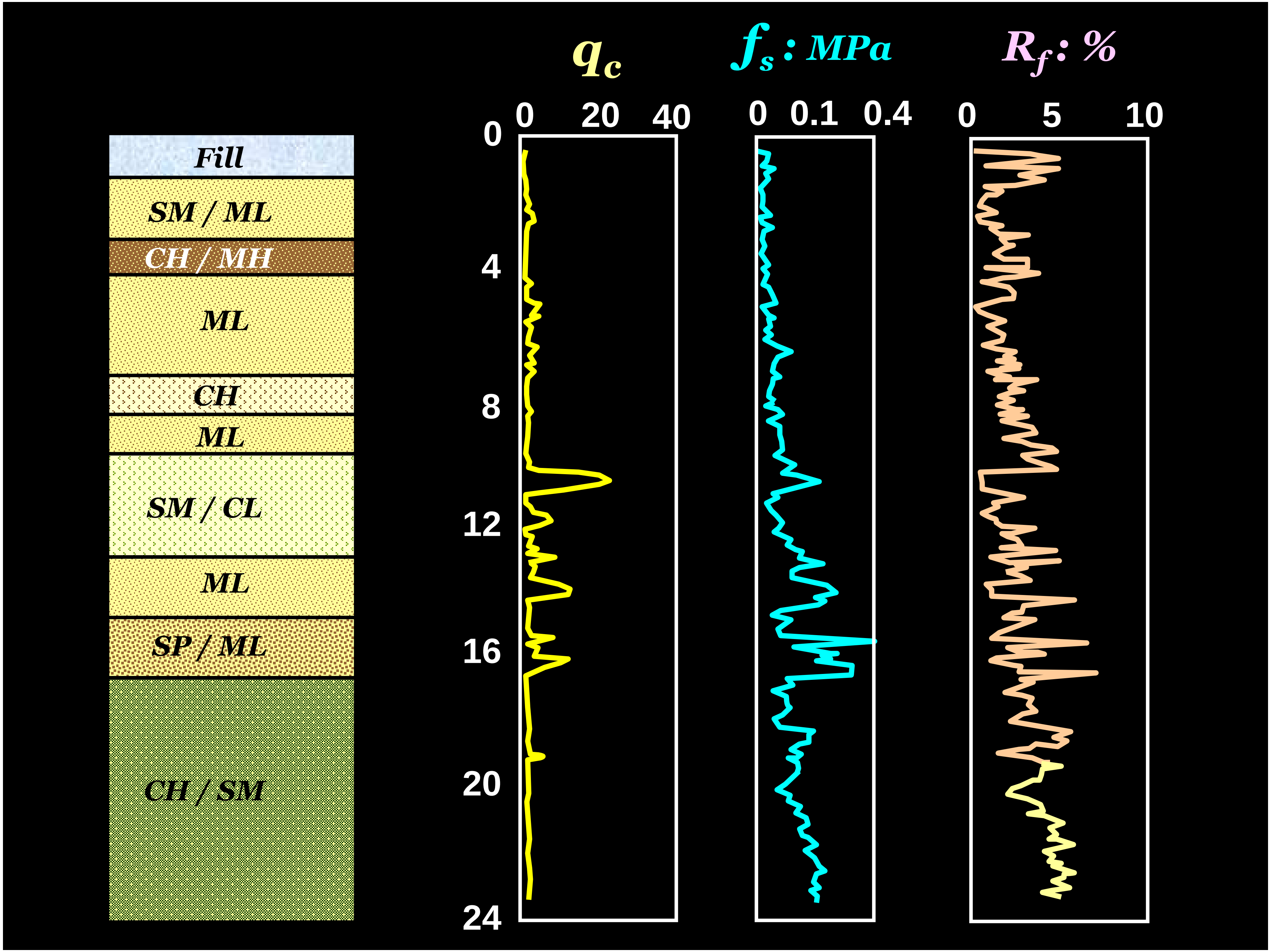
Typical Soil Profile: Very Soft/Loose Soils

Alternating : → Sandy Silt

→ Sandy Clay

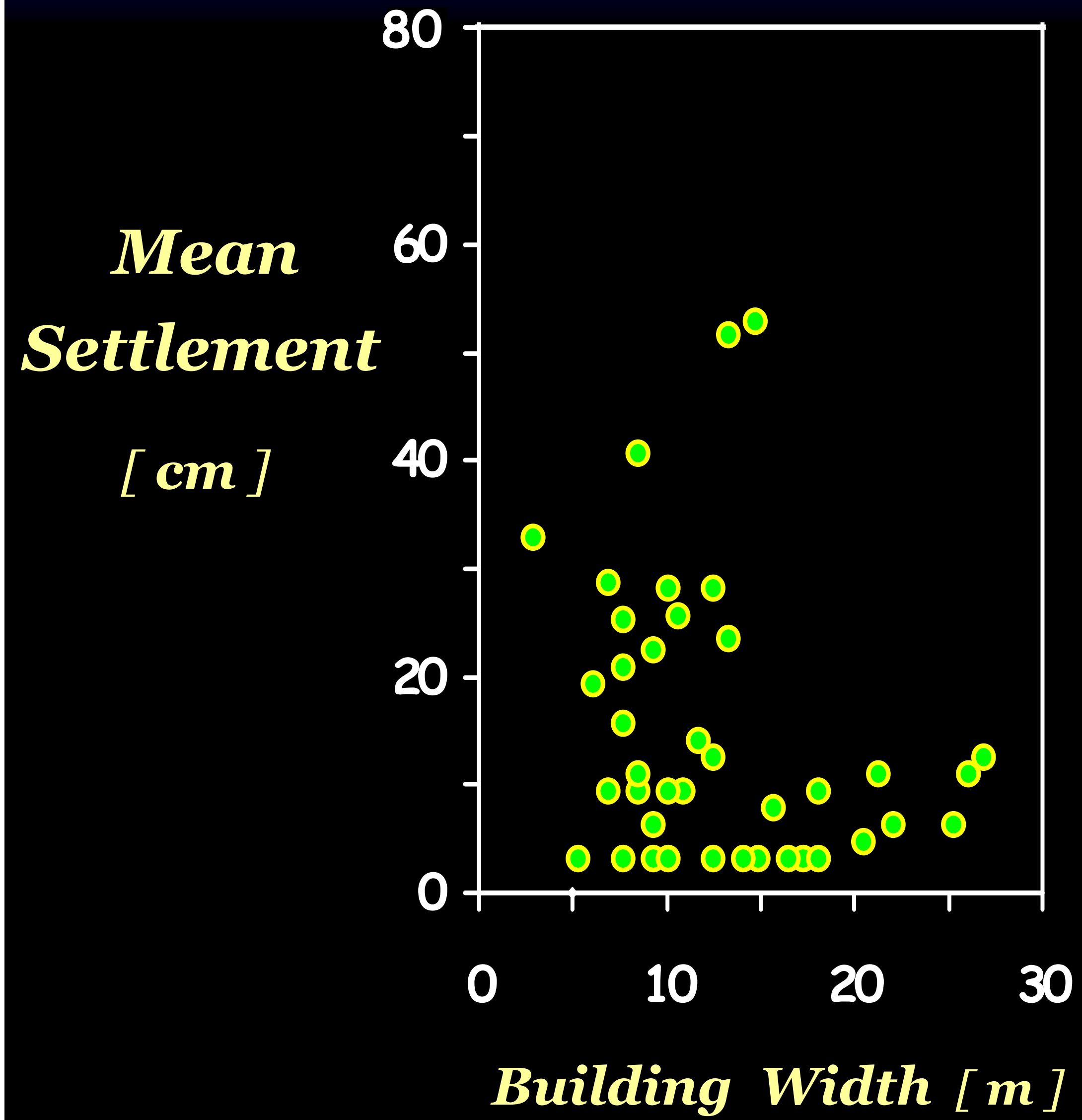
Small total thickness

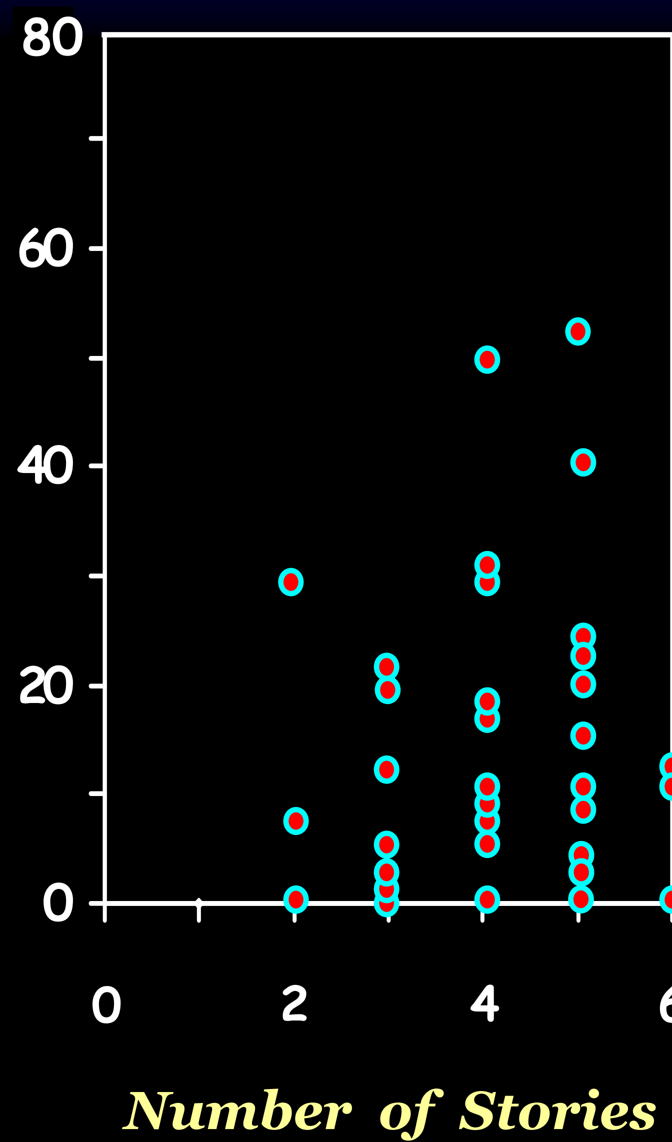
of any liquefiable layers

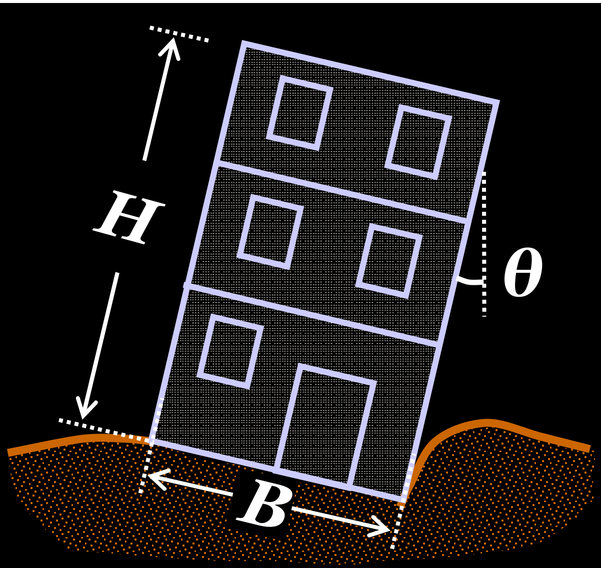


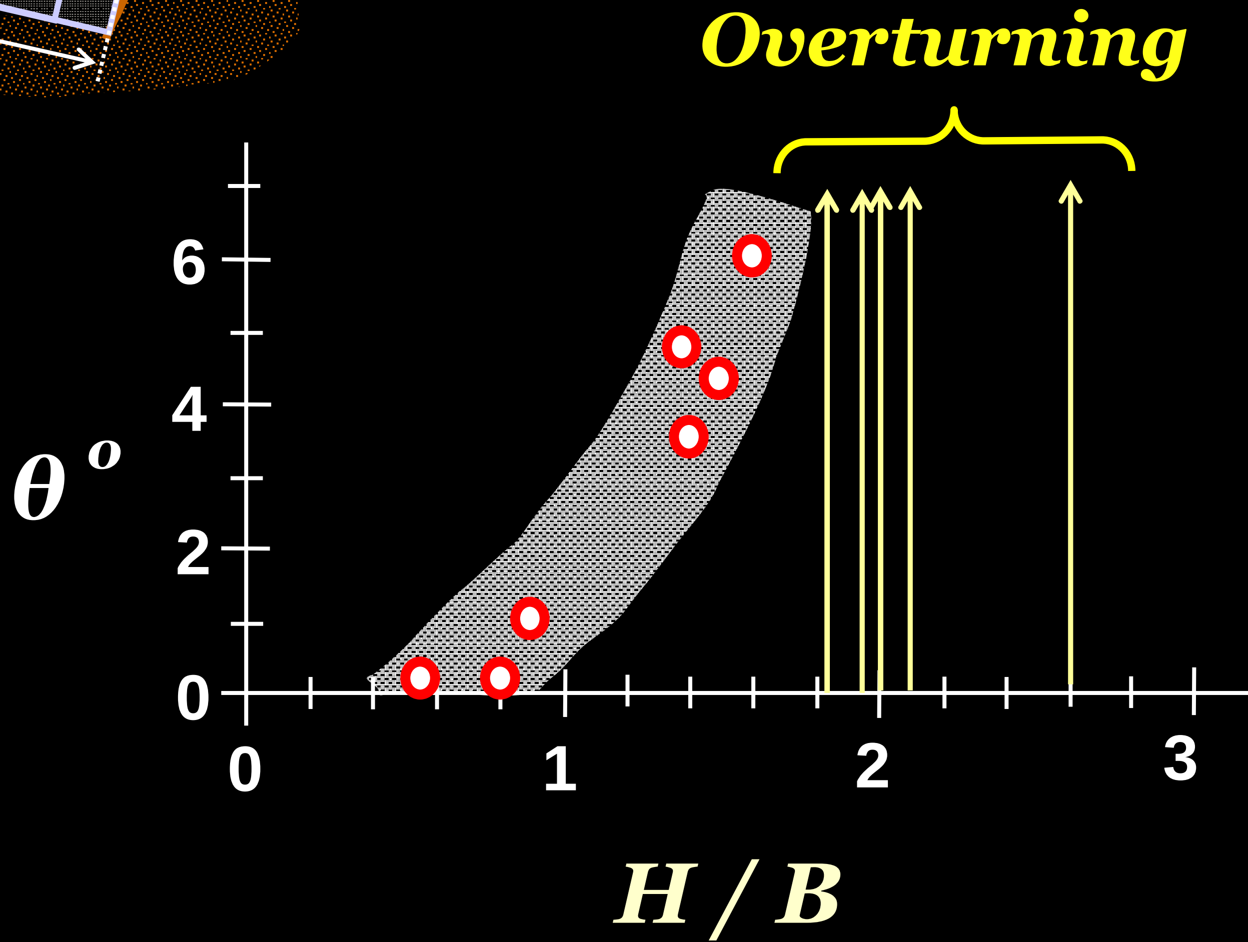
q_c
f_s : MPa
R_f : %
0
20
40
0
0.1
0.4
0
5
10
0
4
8
12
16
20
24
Fill
SM / ML
CH / MH
ML
CH
ML
SM / CL
ML
SP / ML
CH / SM

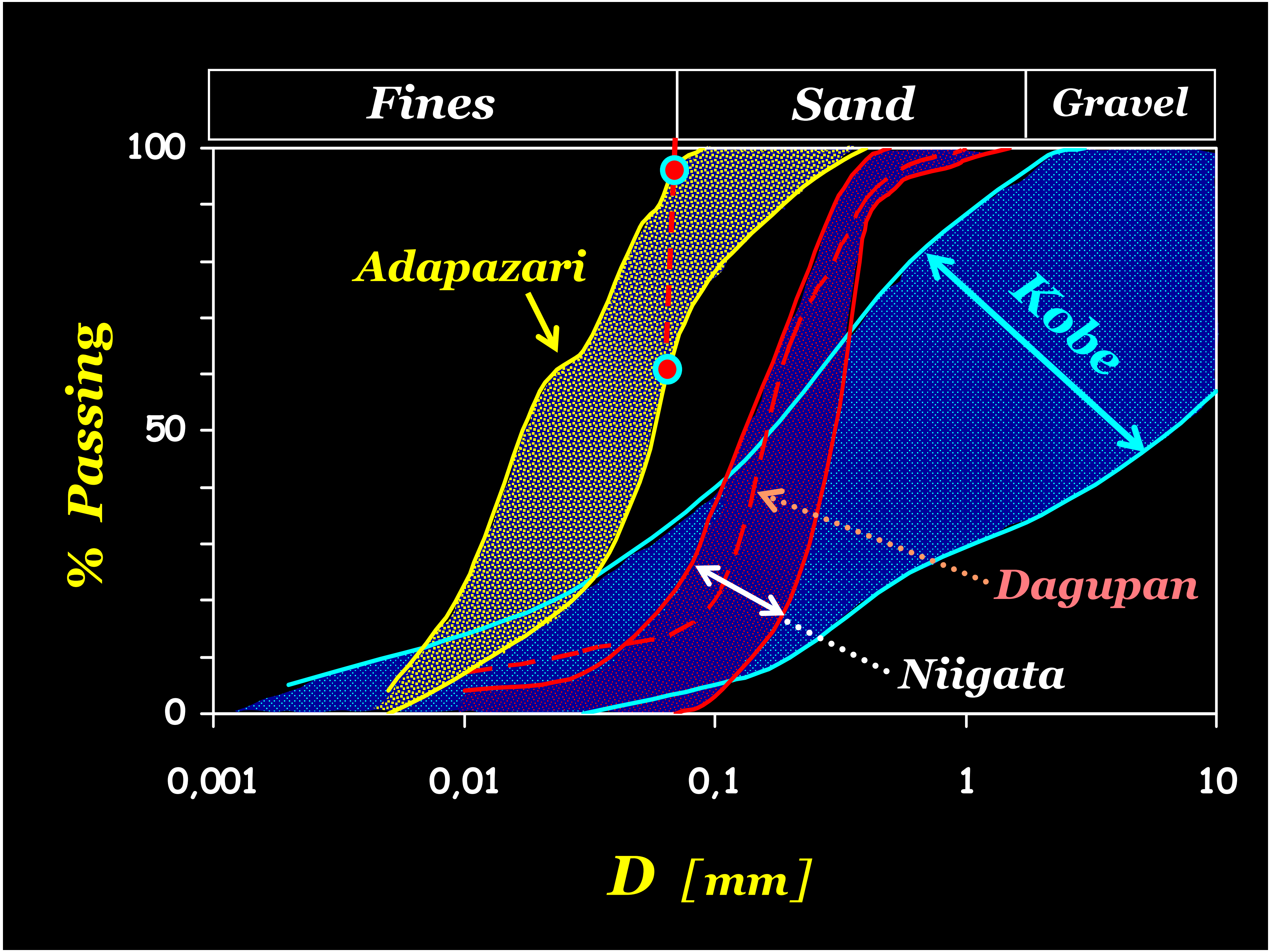
Attempts for Empirical Correlations by other researchers: Yasuda et al, 2002



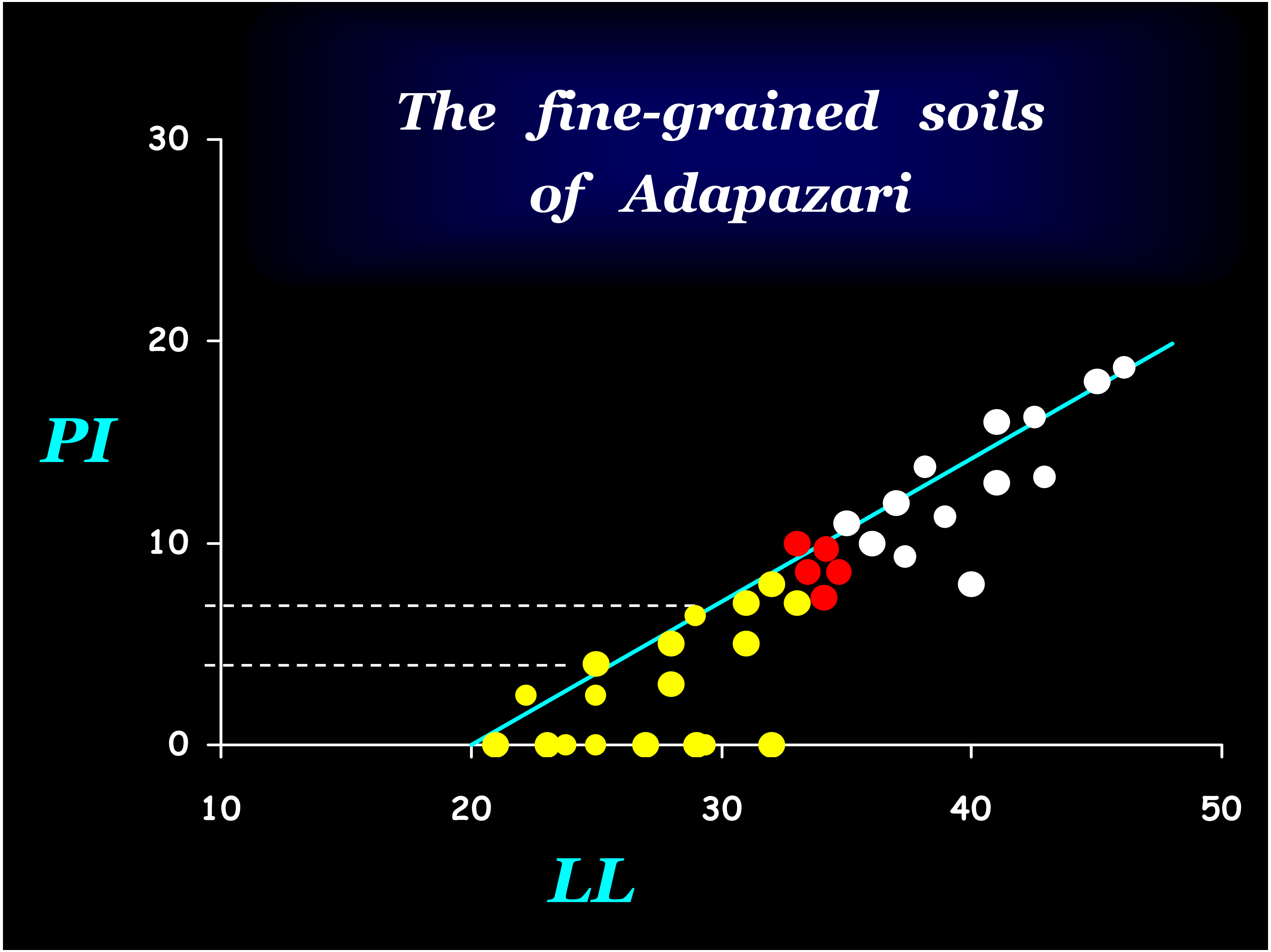


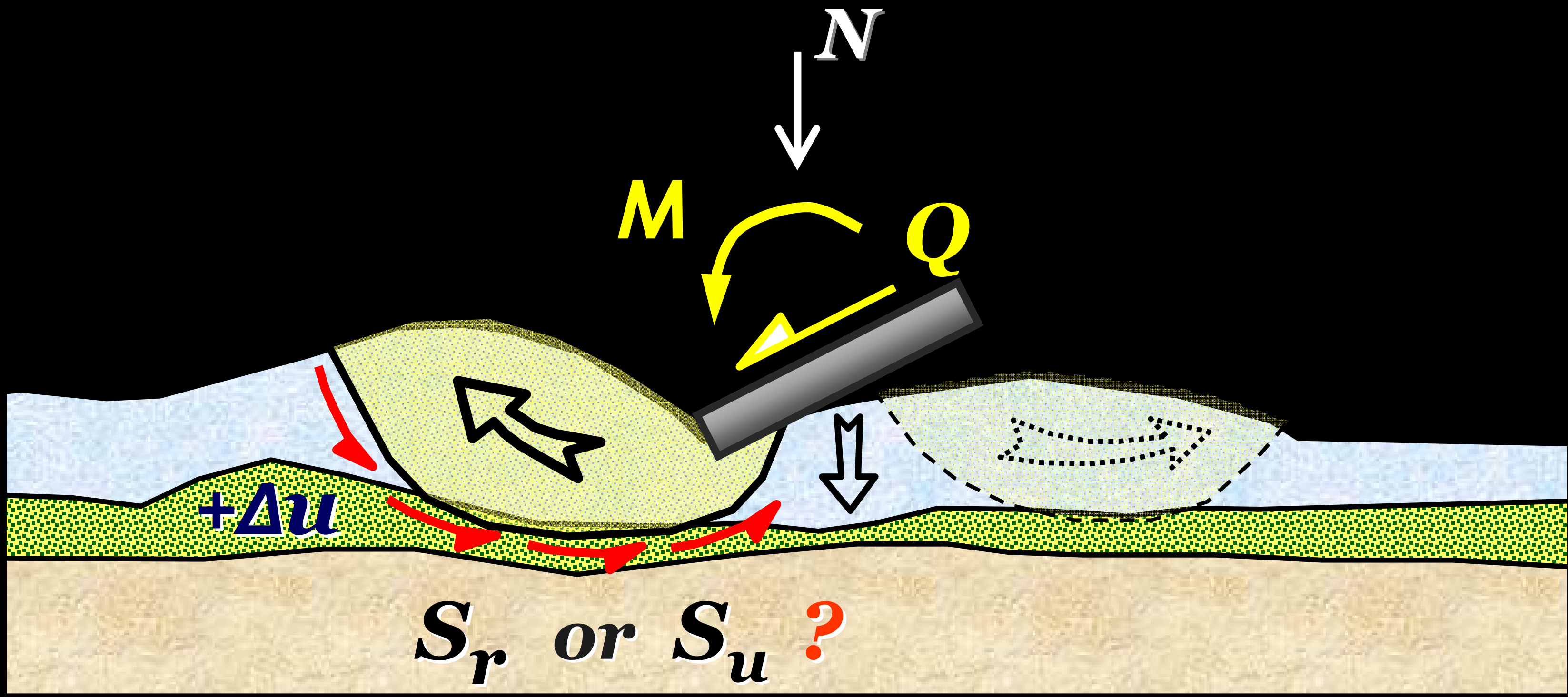

H
θ
B

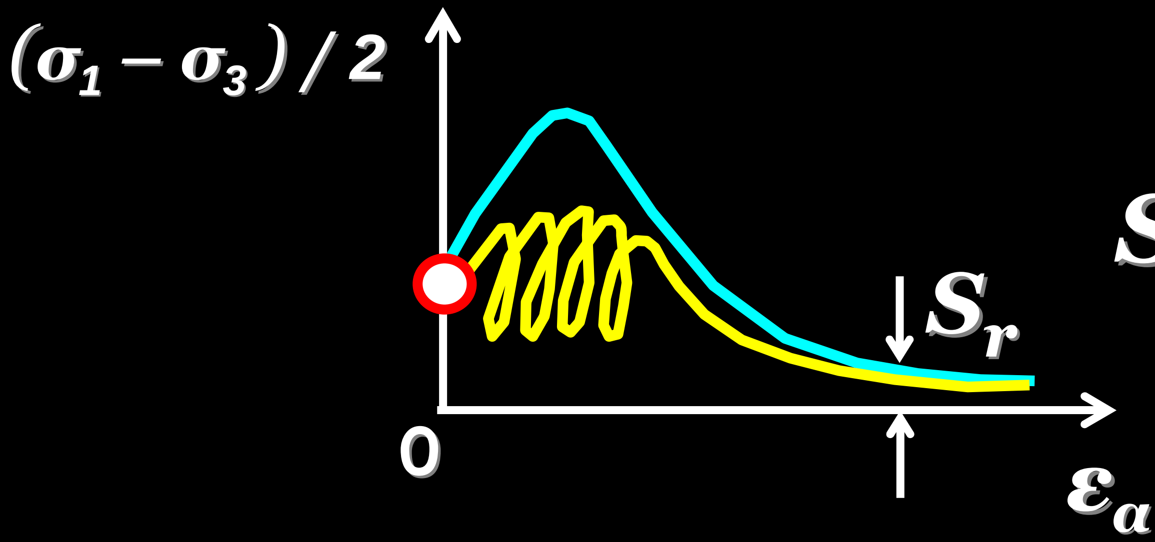

Overturning
θ°
6
4
2
0
0
1
2
3
H / B



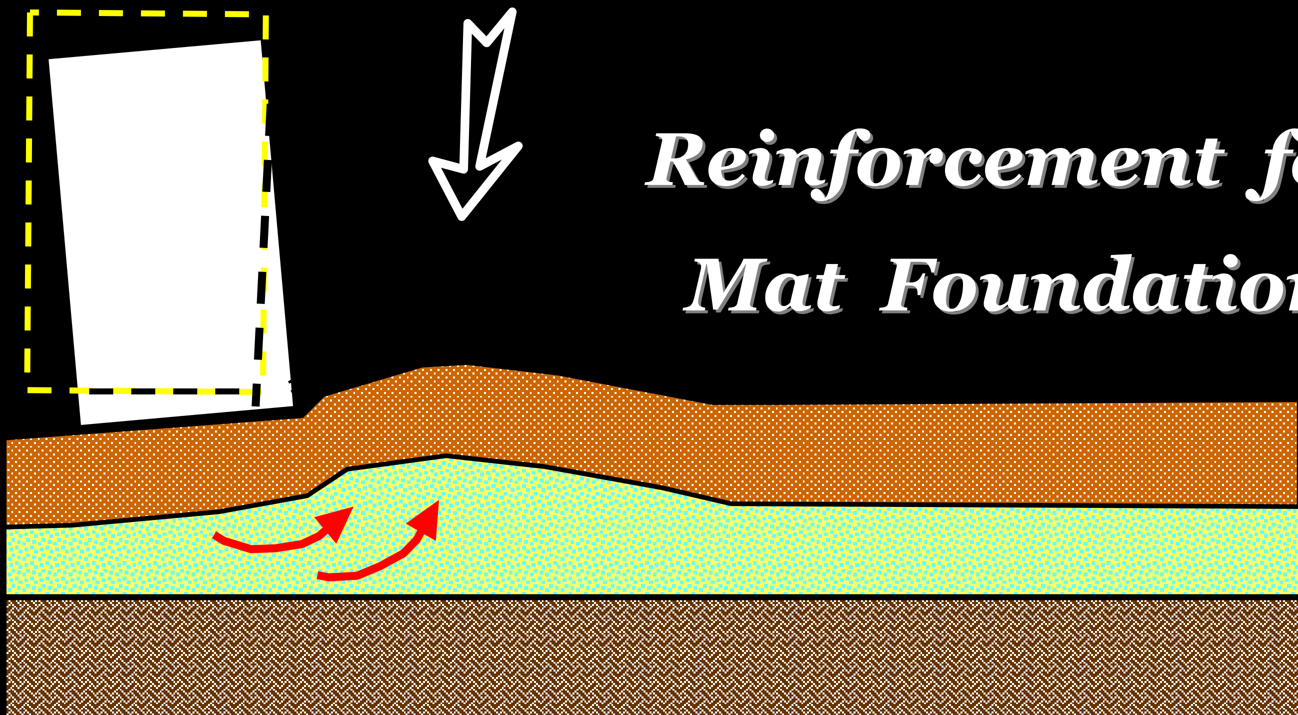
Fines
Sand
Gravel
100
50
0
% Passing
Adapazari
Kobe
Dagupan
Niigata
0,001
0,01
0,1
1
10
D [mm]


The fine-grained soils
of Adapazari
PI
LL
30
20
10
0
10
20
30
40
50



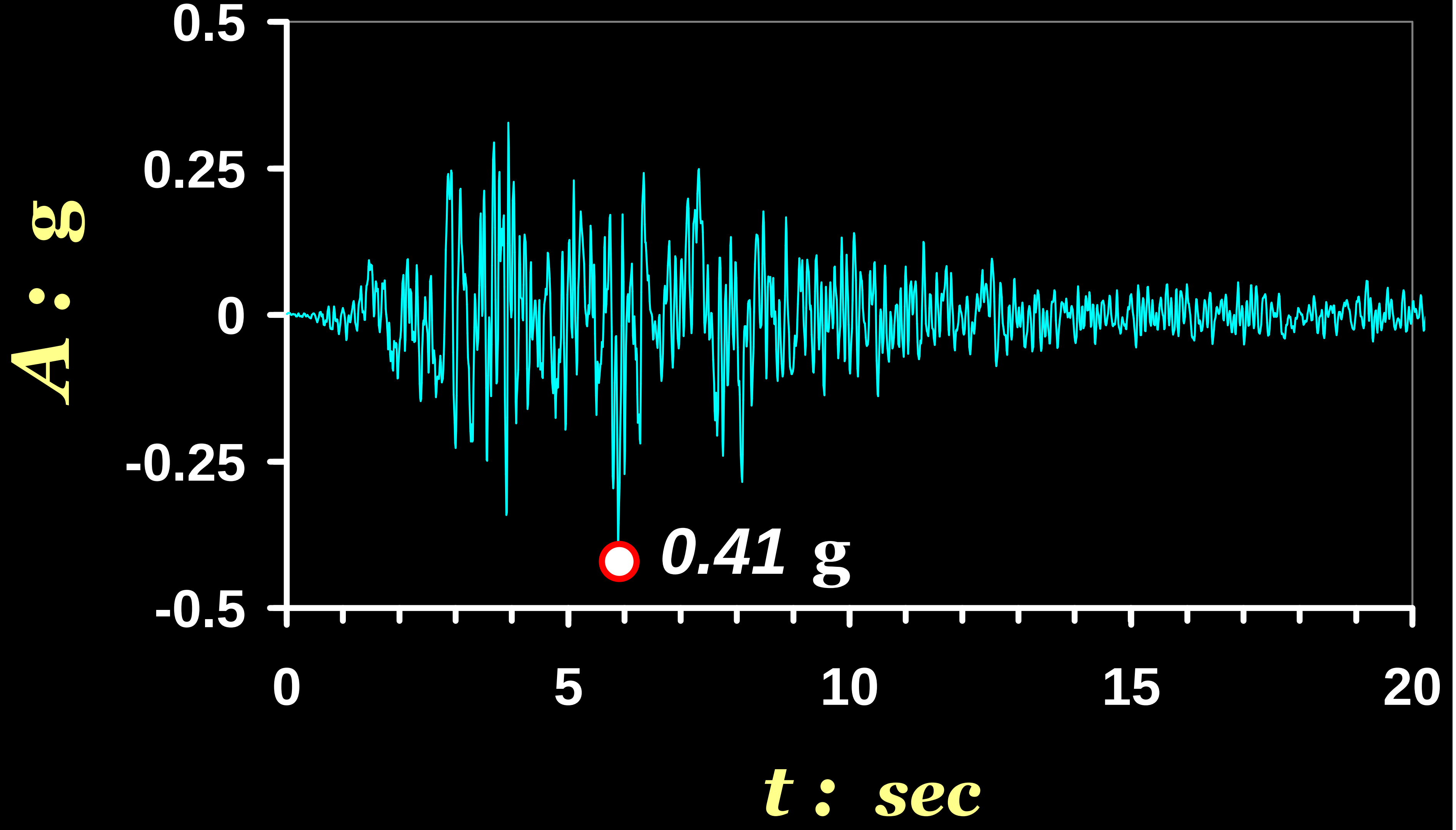


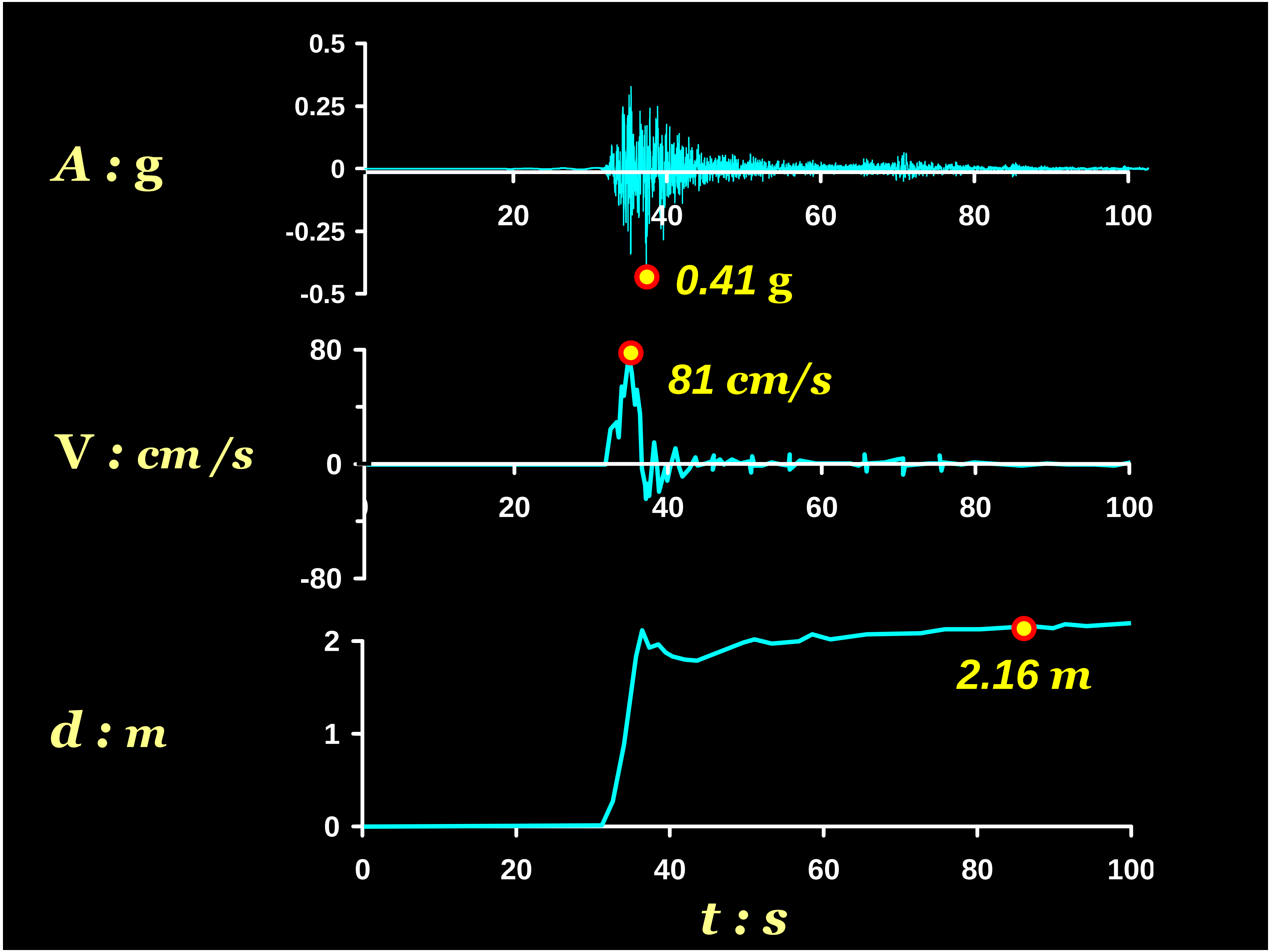
$$S_r \approx 0.09\,\bar{\sigma}_v$$



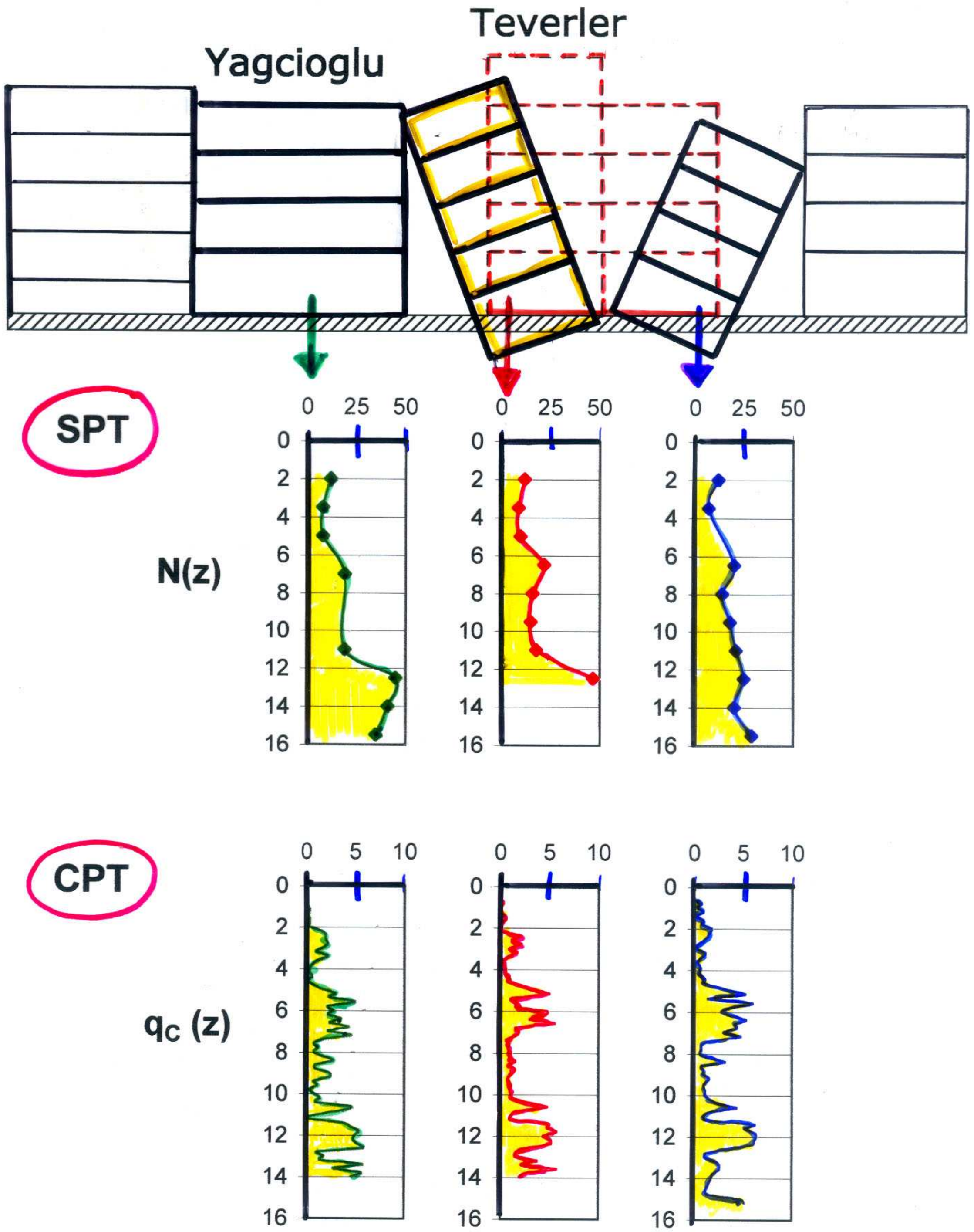
Reinforcement for Mat Foundation

The Sakarya Record



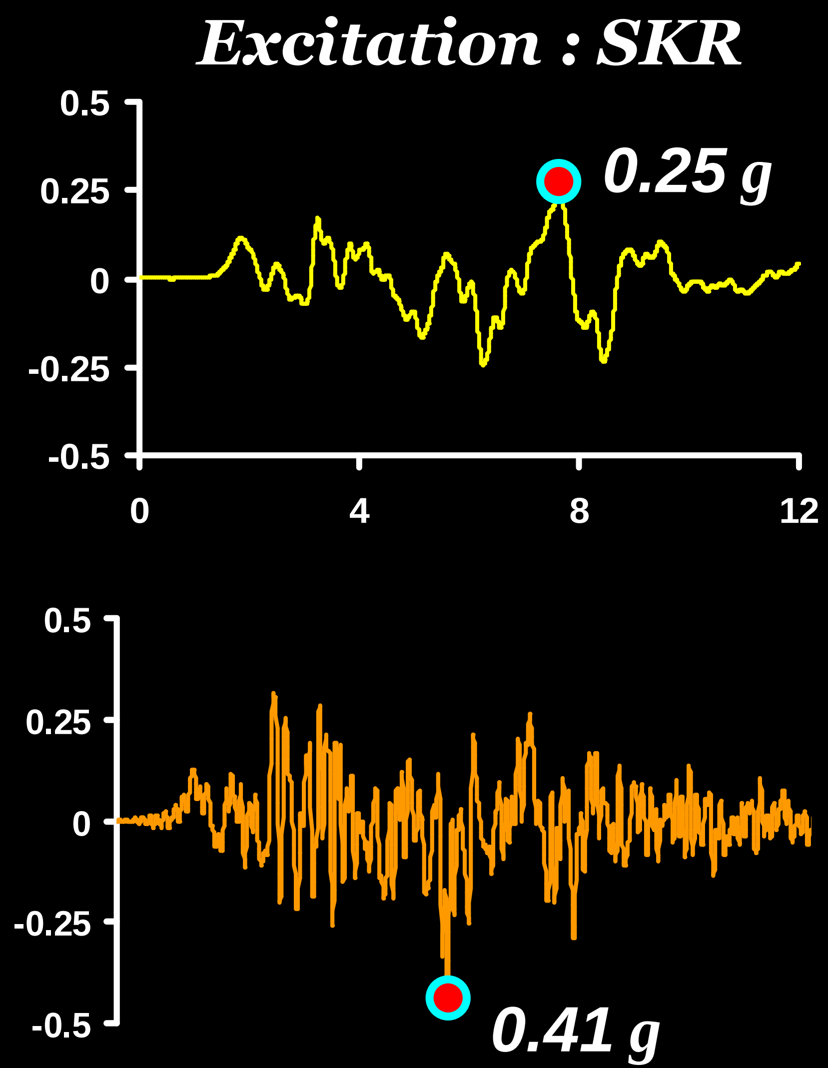

A : g
0.5
0.25
0
-0.25
-0.5
20
40
60
80
100
0.41 g
V : cm /s
80
0
-80
81 cm/s
d : m
2
1
0
2.16 m
t : s

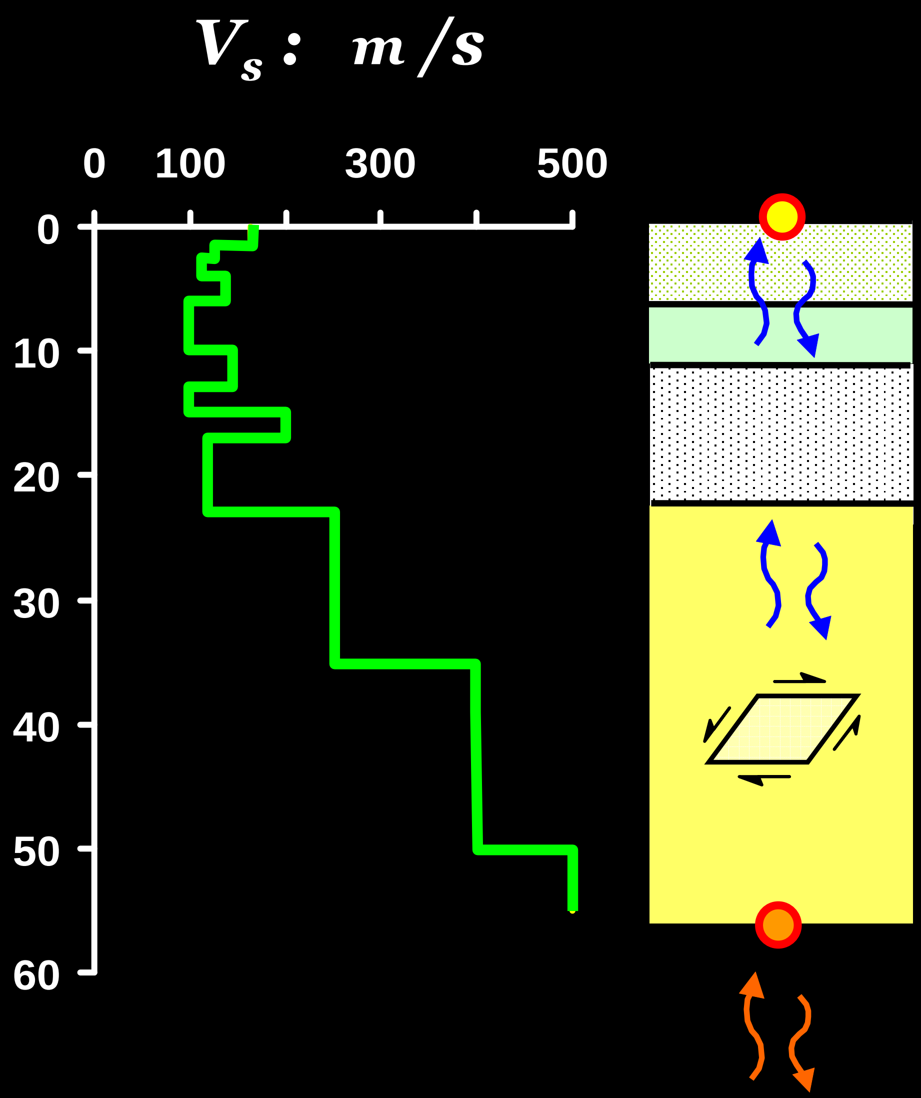
Teverler (Appartment Building)



Teverler
Yagcioglu
SPT
N(z)
CPT
qc (z)
0 25 50
0 5 10
0 2 4 6 8 10 12 14 16

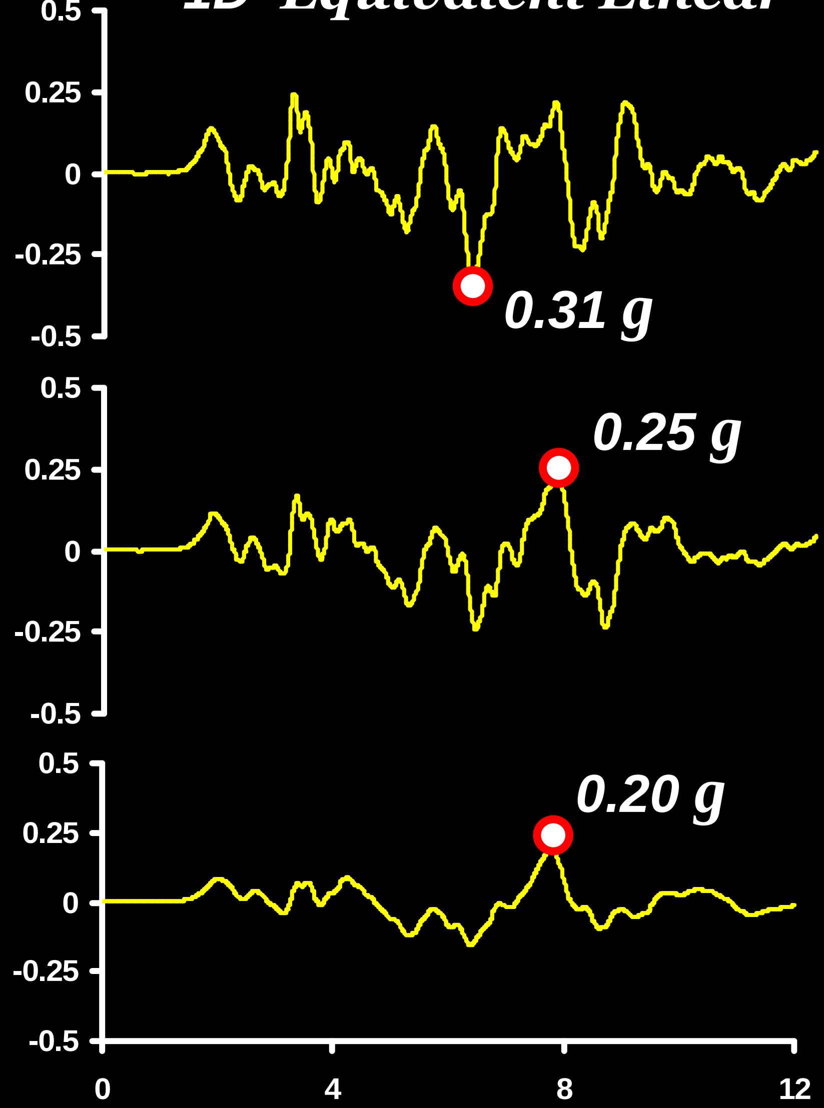
Soil Response : Teverler





Possible Ground Motions

1D Equivalent Linear

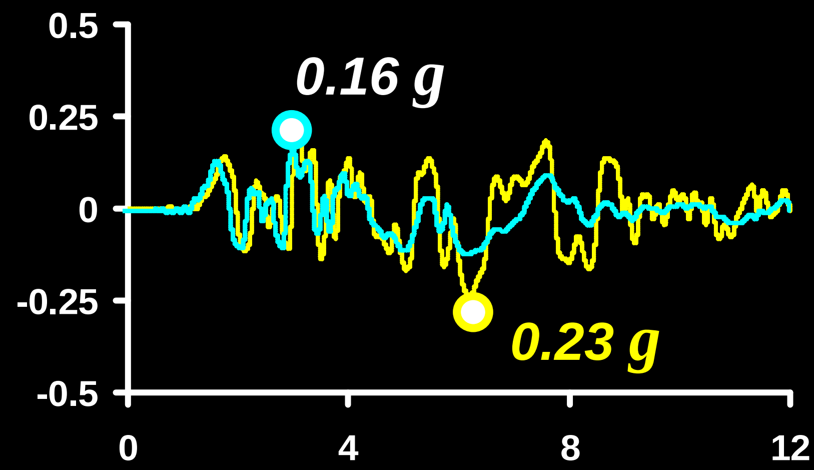


1D Inelastic Analysis

BWGG

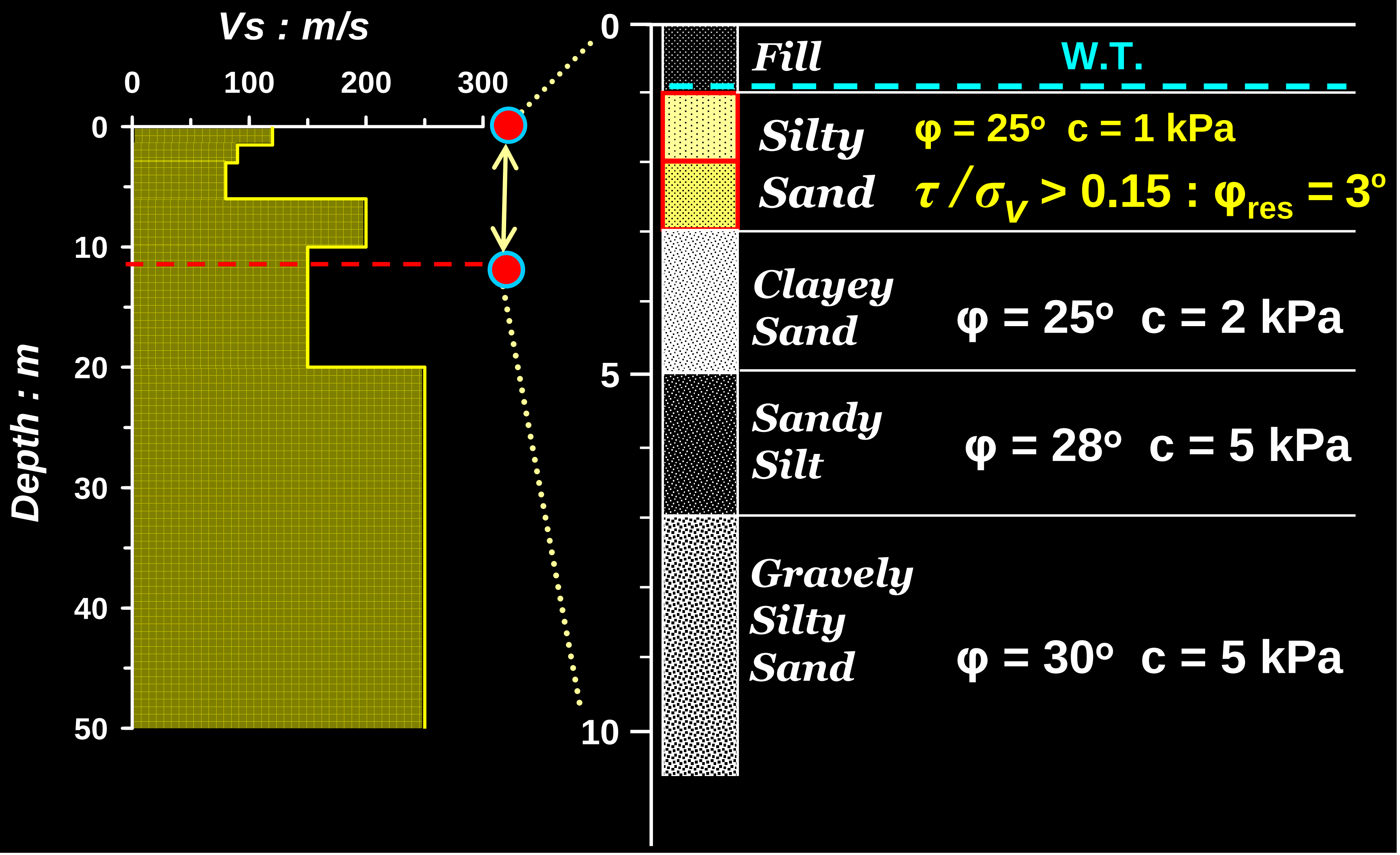
Without Pore pressure rise

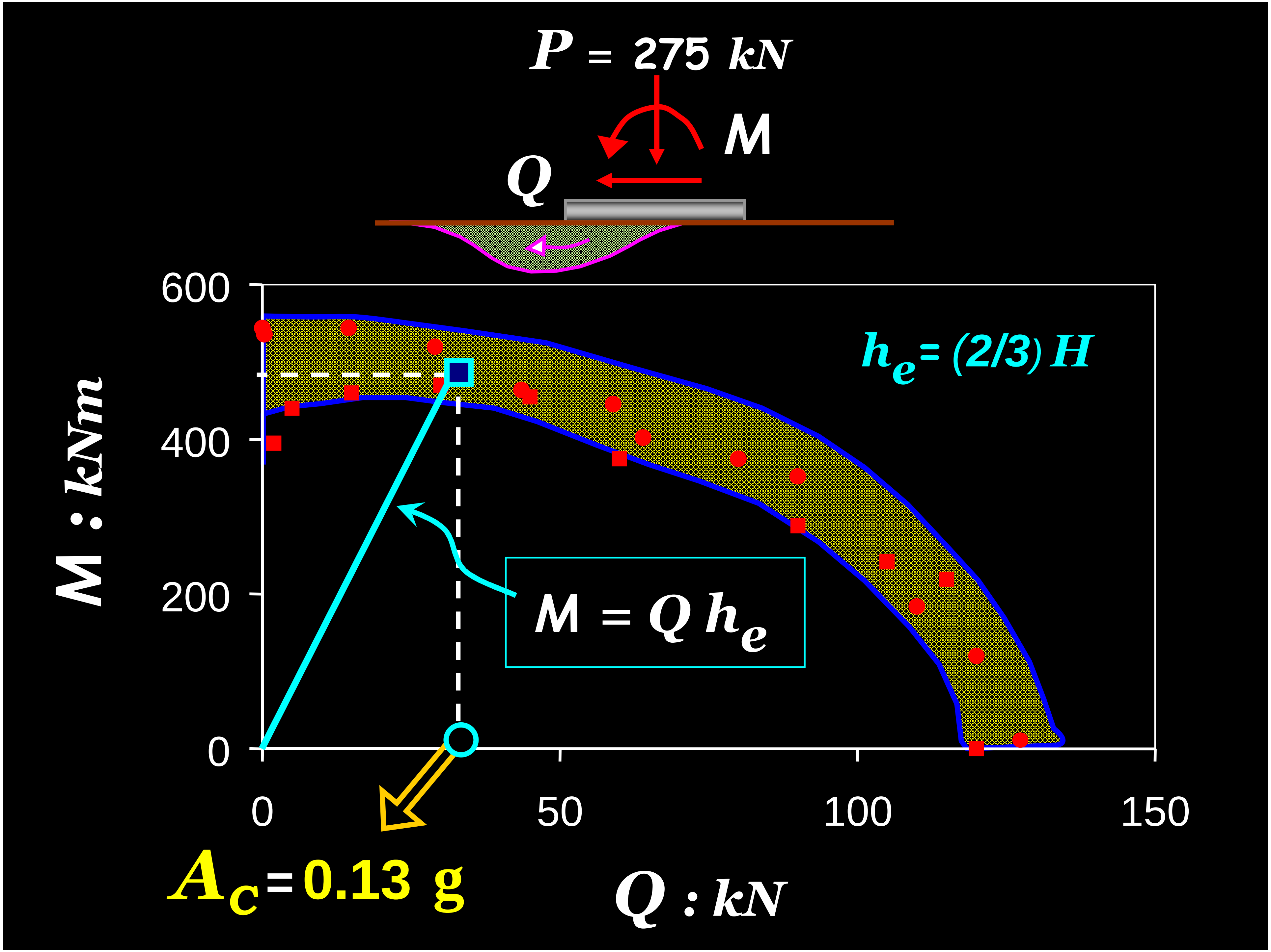
With Pore pressure rise



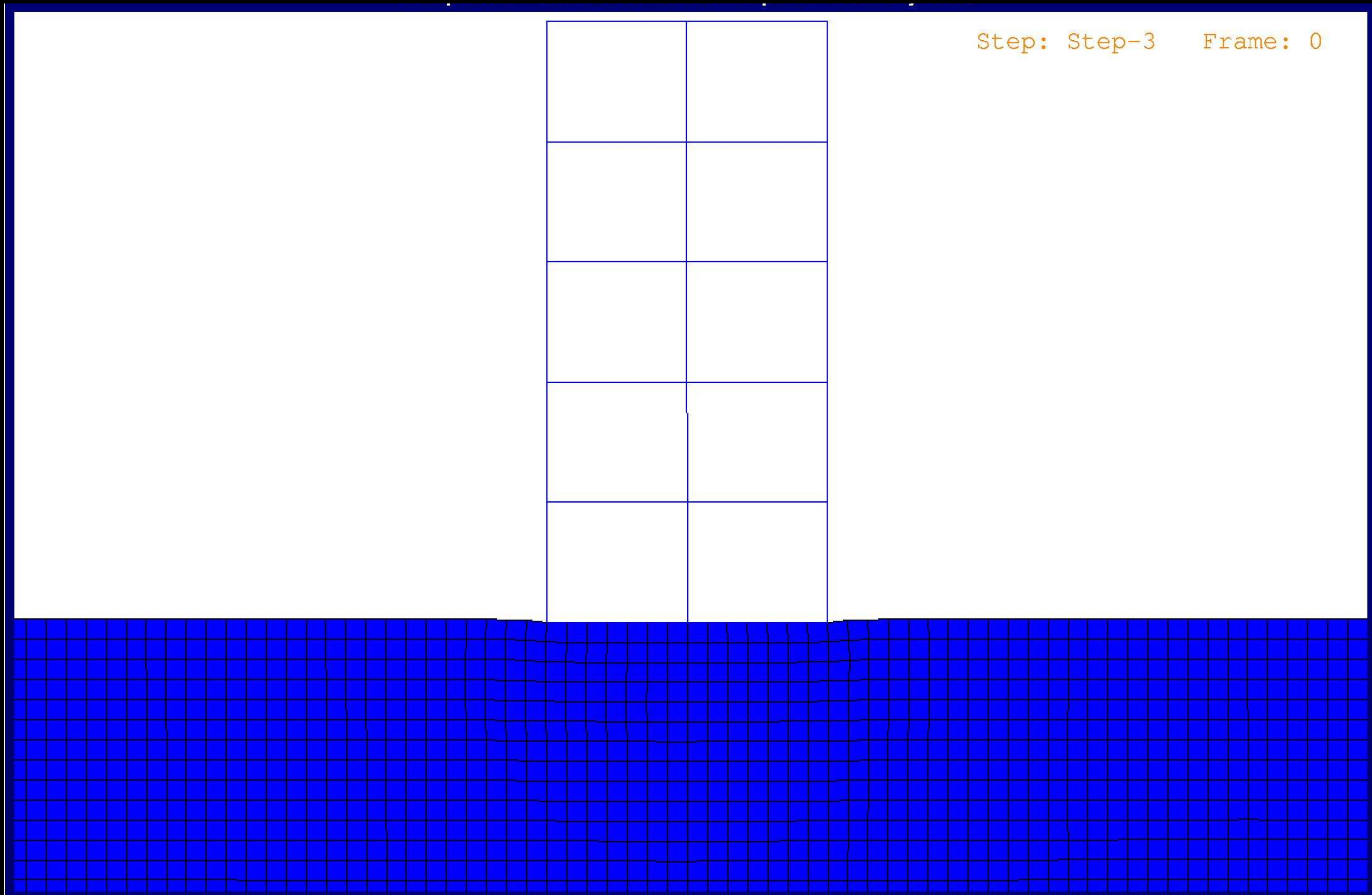
Excitation : SKR

Nonlinear Dynamic SFSI Analysis

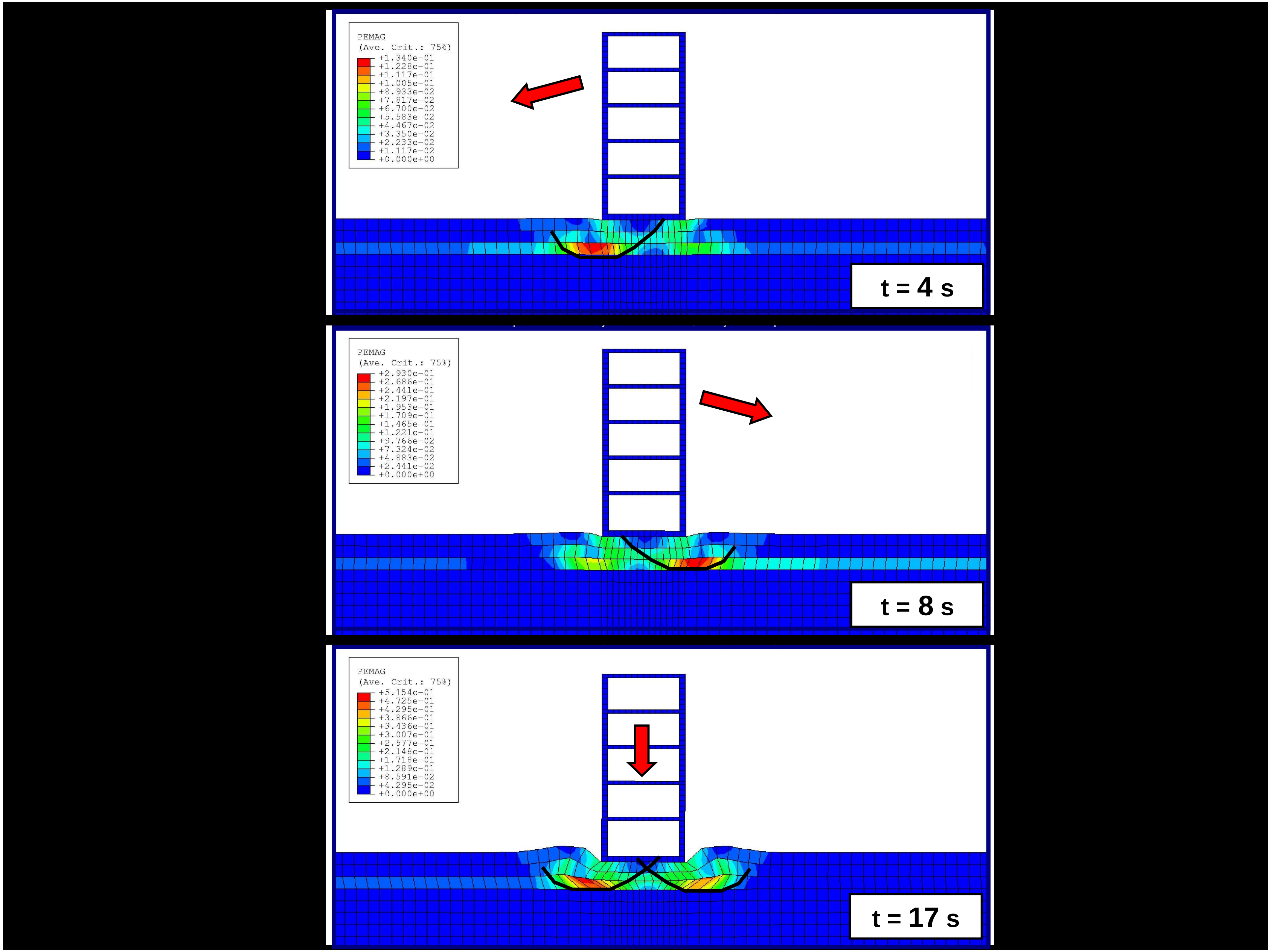




P = 275 kN
M
Q
600
400
200
0
0
50
100
150
M : kNm
Q : kN
h_e = (2/3) H
M = Q h_e
A_c = 0.13 g



***Deformation Scaling* = x 3**


PEMAG
(Ave. Crit.: 75%)
+1.340e-01
+1.228e-01
+1.117e-01
+1.005e-01
+8.933e-02
+7.817e-02
+6.700e-02
+5.583e-02
+4.467e-02
+3.350e-02
+2.233e-02
+1.117e-02
+0.000e+00
t = 4 s
PEMAG
(Ave. Crit.: 75%)
+2.930e-01
+2.686e-01
+2.441e-01
+2.197e-01
+1.953e-01
+1.709e-01
+1.465e-01
+1.221e-01
+9.766e-02
+7.324e-02
+4.883e-02
+2.441e-02
+0.000e+00
t = 8 s
PEMAG
(Ave. Crit.: 75%)
+5.154e-01
+4.725e-01
+4.295e-01
+3.866e-01
+3.436e-01
+3.007e-01
+2.577e-01
+2.148e-01
+1.718e-01
+1.289e-01
+8.591e-02
+4.295e-02
+0.000e+00
t = 17 s

This is a Critical Moment

for our Approach:

it predicts Non-Overturning

when in REALITY the building
Overturned !

Many Plausible Causes

(1) Actual Base Motion Stronger

missing NS comp. : "Fault Normal" (FN),

used EW comp. : "Fault Parallel" (FP) :

FN > FP

(2) Soil at D > 20 m : Detrimental Role

due to larger soil amplification

(3) 2-D and 3-D wave focusing

due to irregular bedrock geometry

(4)

But what about

Out–of–Phase Response of

Adjacent Buildings,

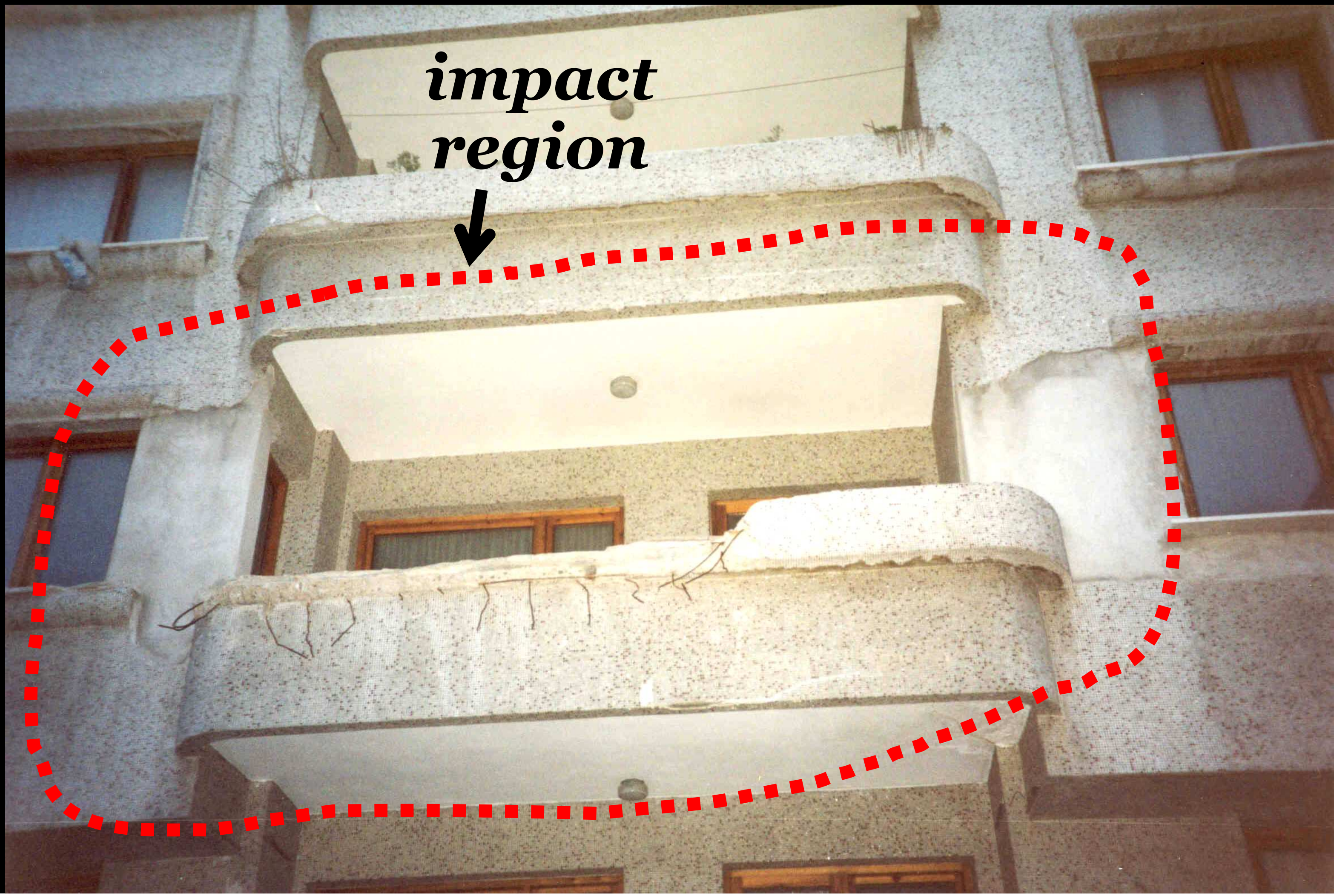
and hence IMPACT Forces ??

Did this Play any ROLE in Adapazari ??

NO Sign of IMPACT between the 2 buildings

Insignificant Damage to the "Host" Buildings

⇨ **Impact Velocity very small**



(5)

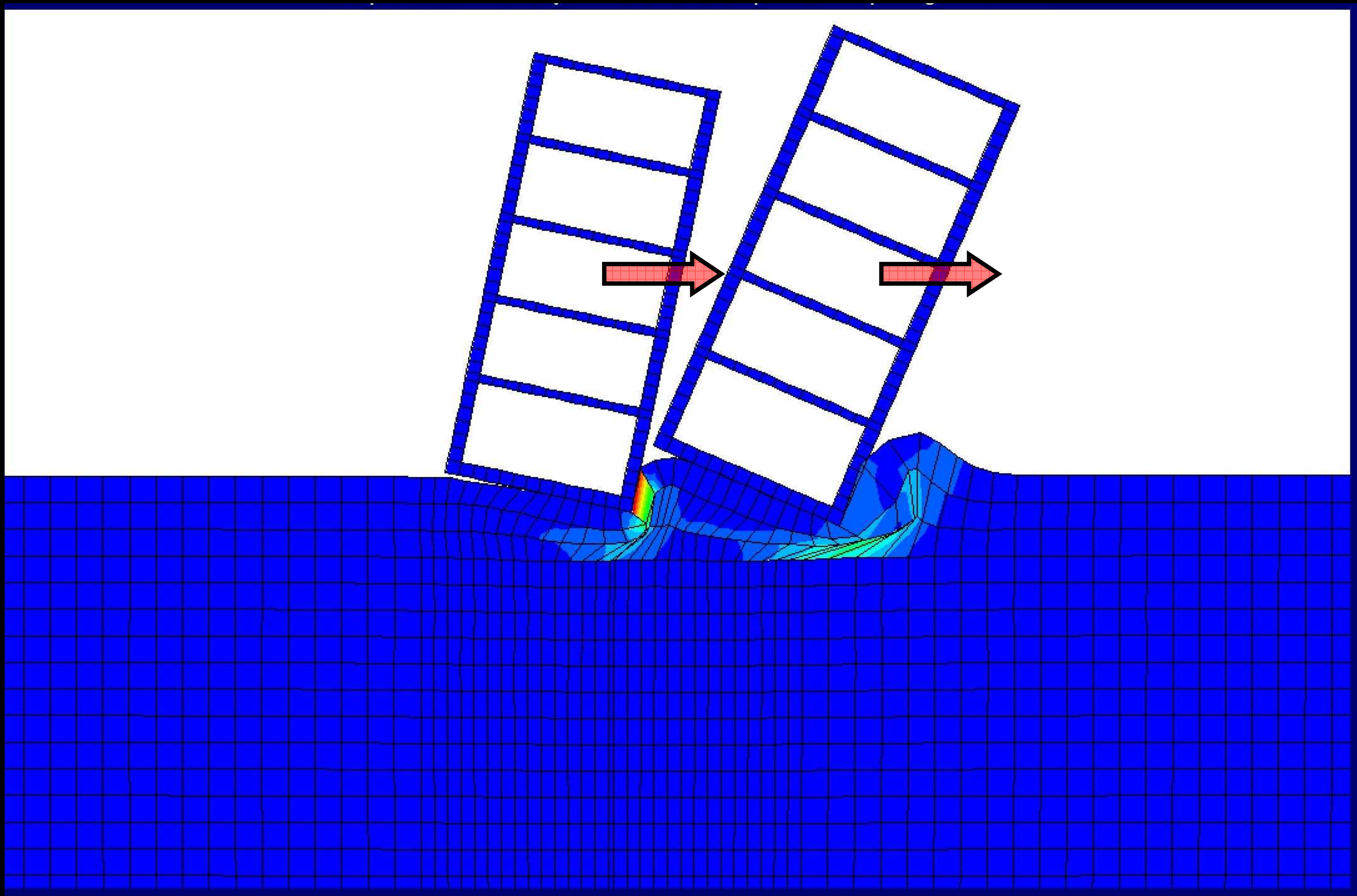
Now what about

the very Presence of Adjacent Buildings ?

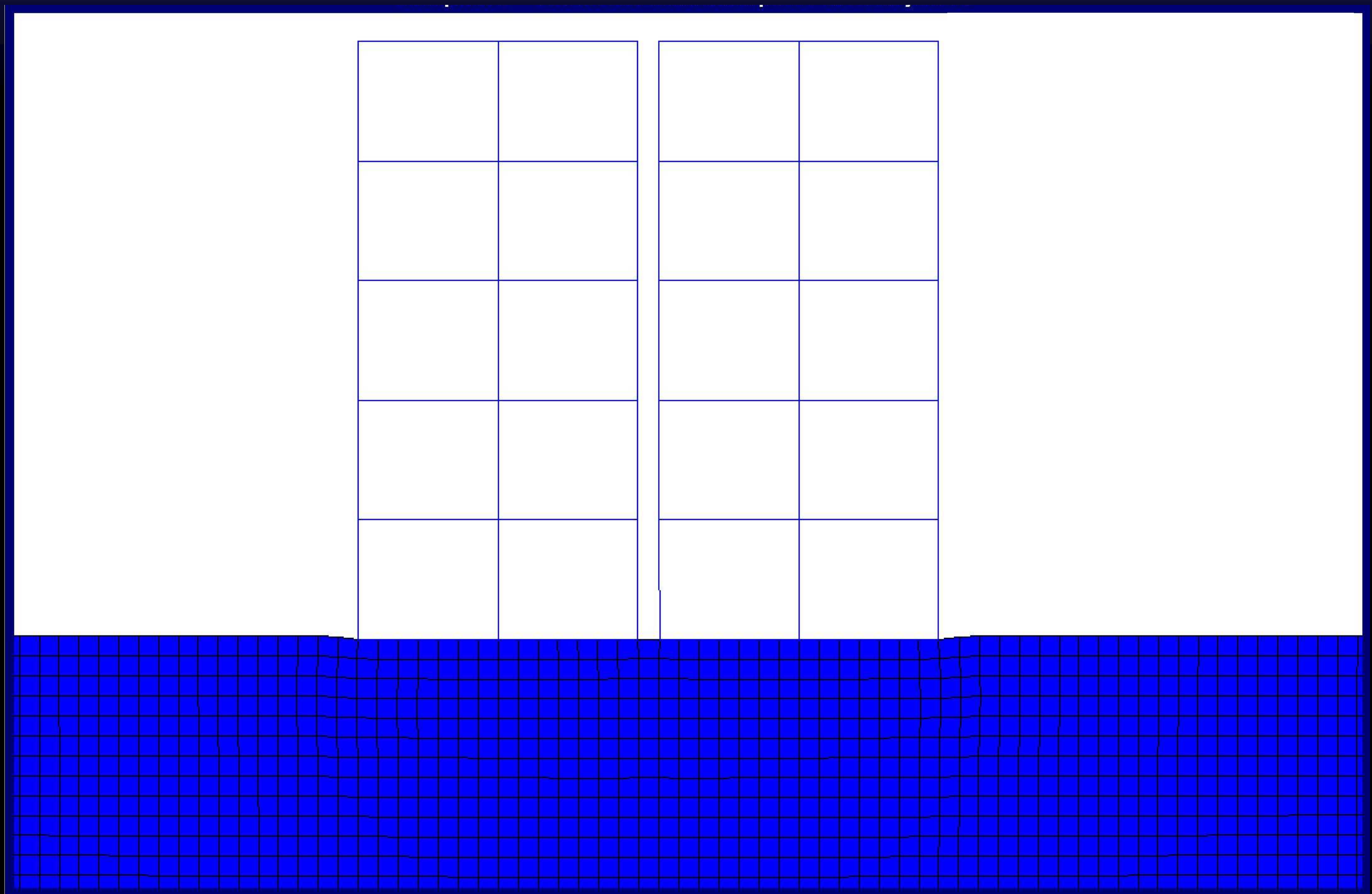
$M_{ult}^{+} >> M_{ult}^{-}$: due to greater confinement of the soil

Reversal of Plastic Rotation : inhibited

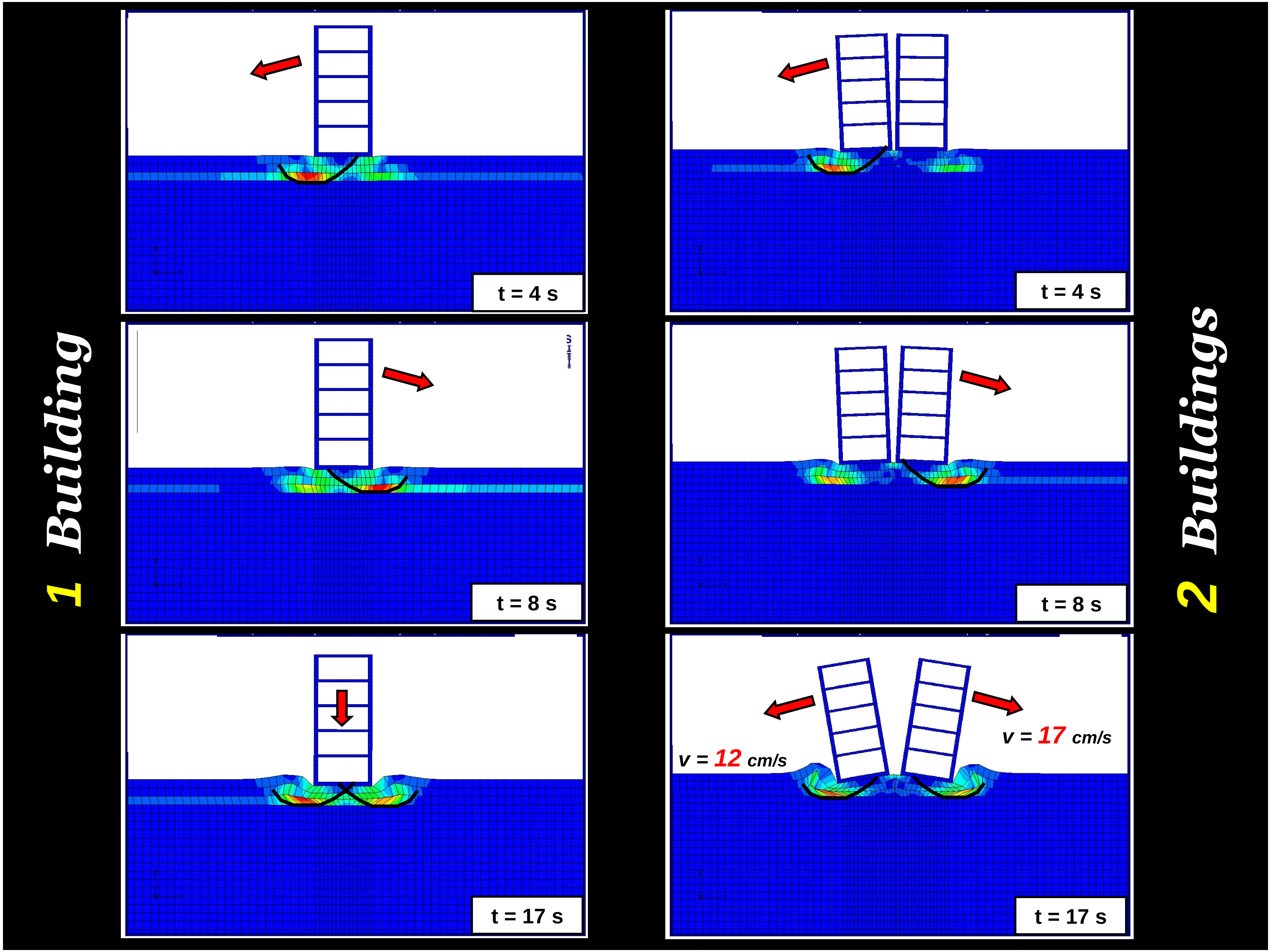
Analysis of 2 Buildings



Buildings 2 , 3



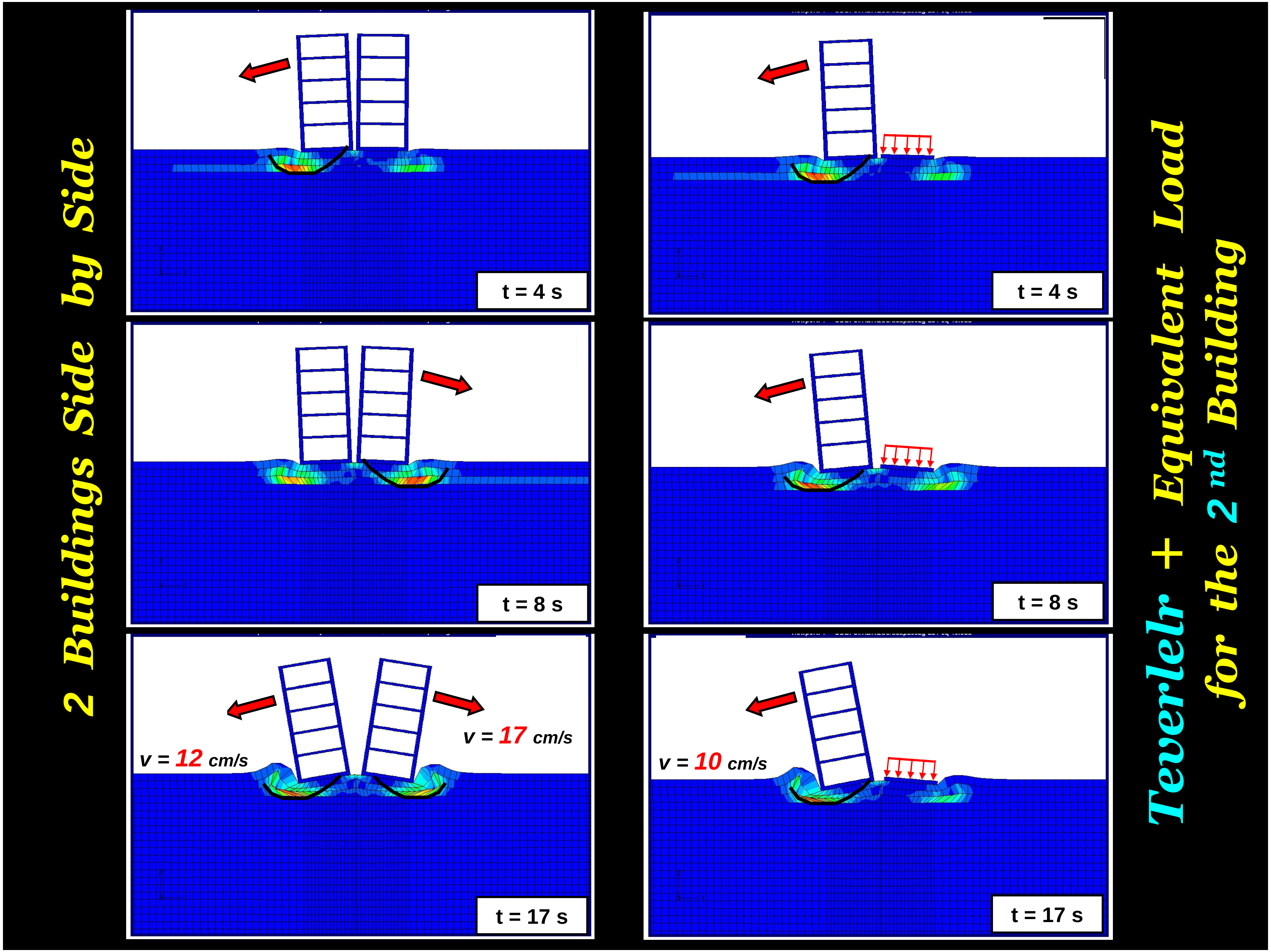
Deformation Scaling : X **3**


1 Building
2 Buildings
t = 4 s
t = 4 s
t = 8 s
t = 8 s
t = 17 s
t = 17 s
v = 12 cm/s
v = 17 cm/s

Let us further explore
this possible role

by replacing the adjacent building

by its Vertical Pressure


2 Buildings Side by Side
Teverlelr + Equivalent Load for the 2nd Building
t = 4 s
t = 4 s
t = 8 s
t = 8 s
v = 12 cm/s
v = 17 cm/s
t = 17 s
v = 10 cm/s
t = 17 s

The Role of the Adjacent Building :

One–Directional Accumulation of Tilt

NO Reversal of PLASTIC Deformation,

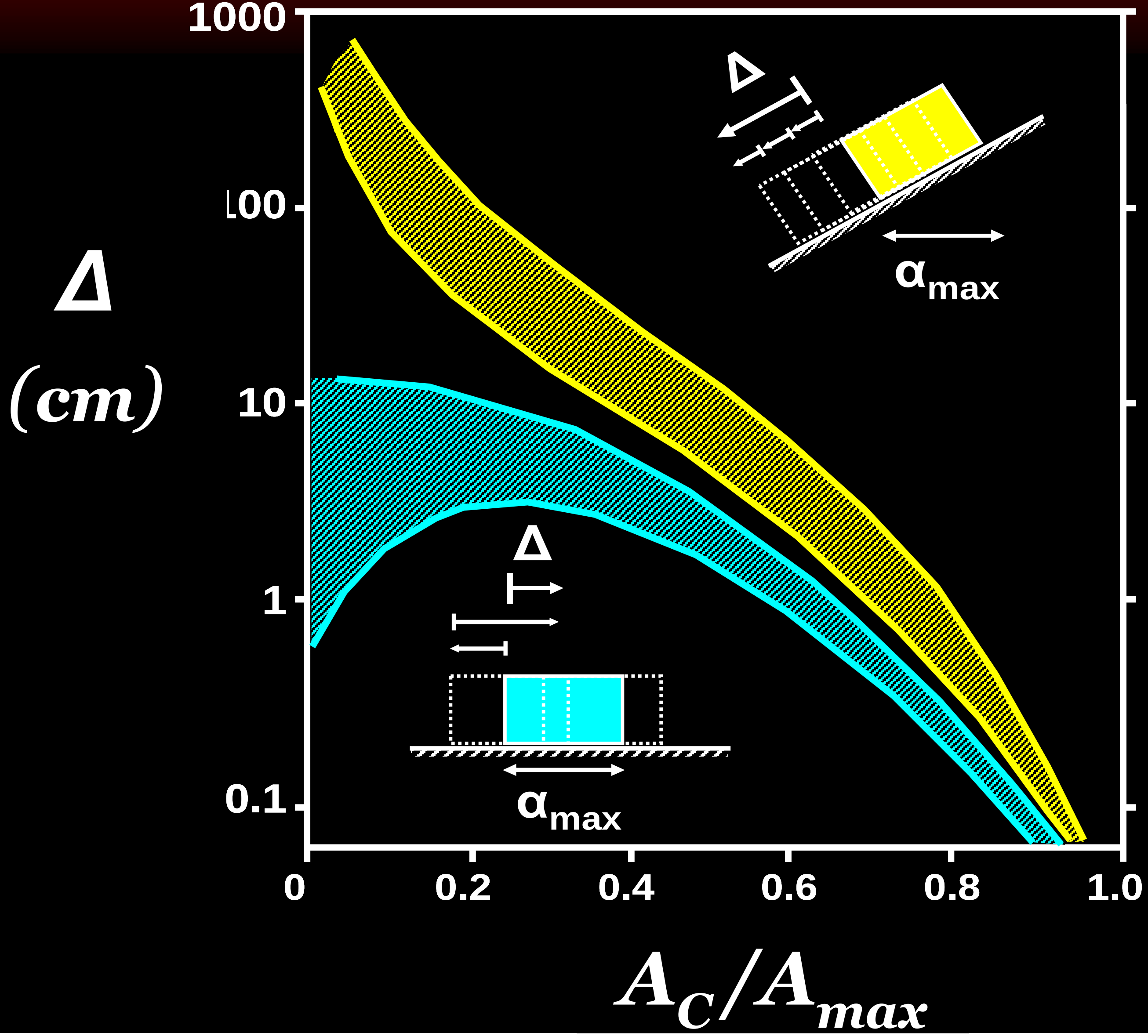
Asymmetric Yielding

Here is a

Mechanical

Analogue

One-Directional vs. Two-Directional SLIPPAGE



“Lonely” Buildings

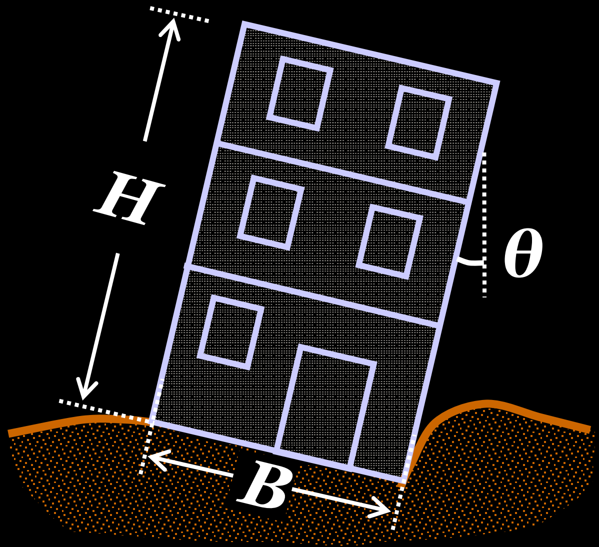
did not fail

Buildings surrounded by others

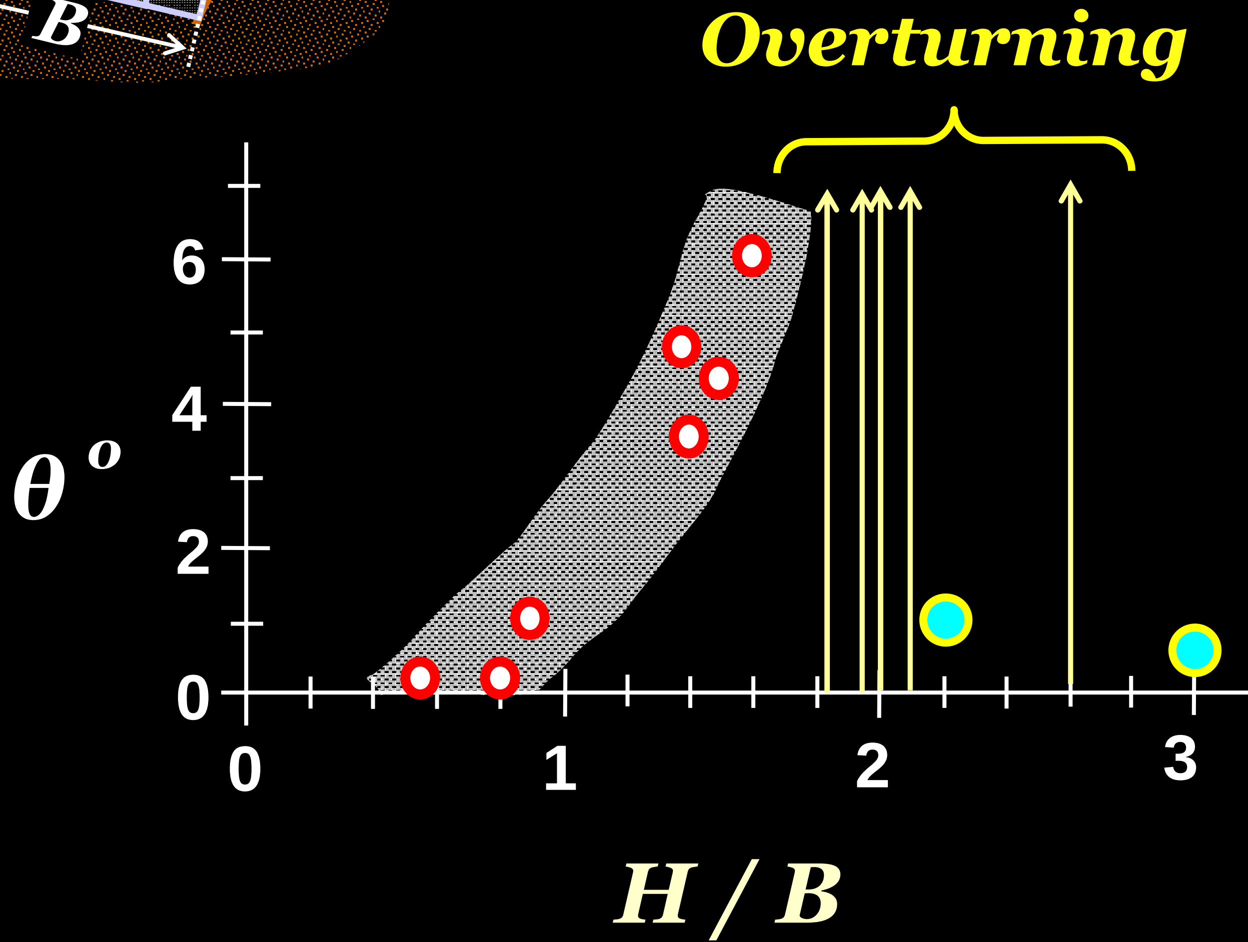
did not fail

… even if they were very slender !

Here is some (further) evidence :

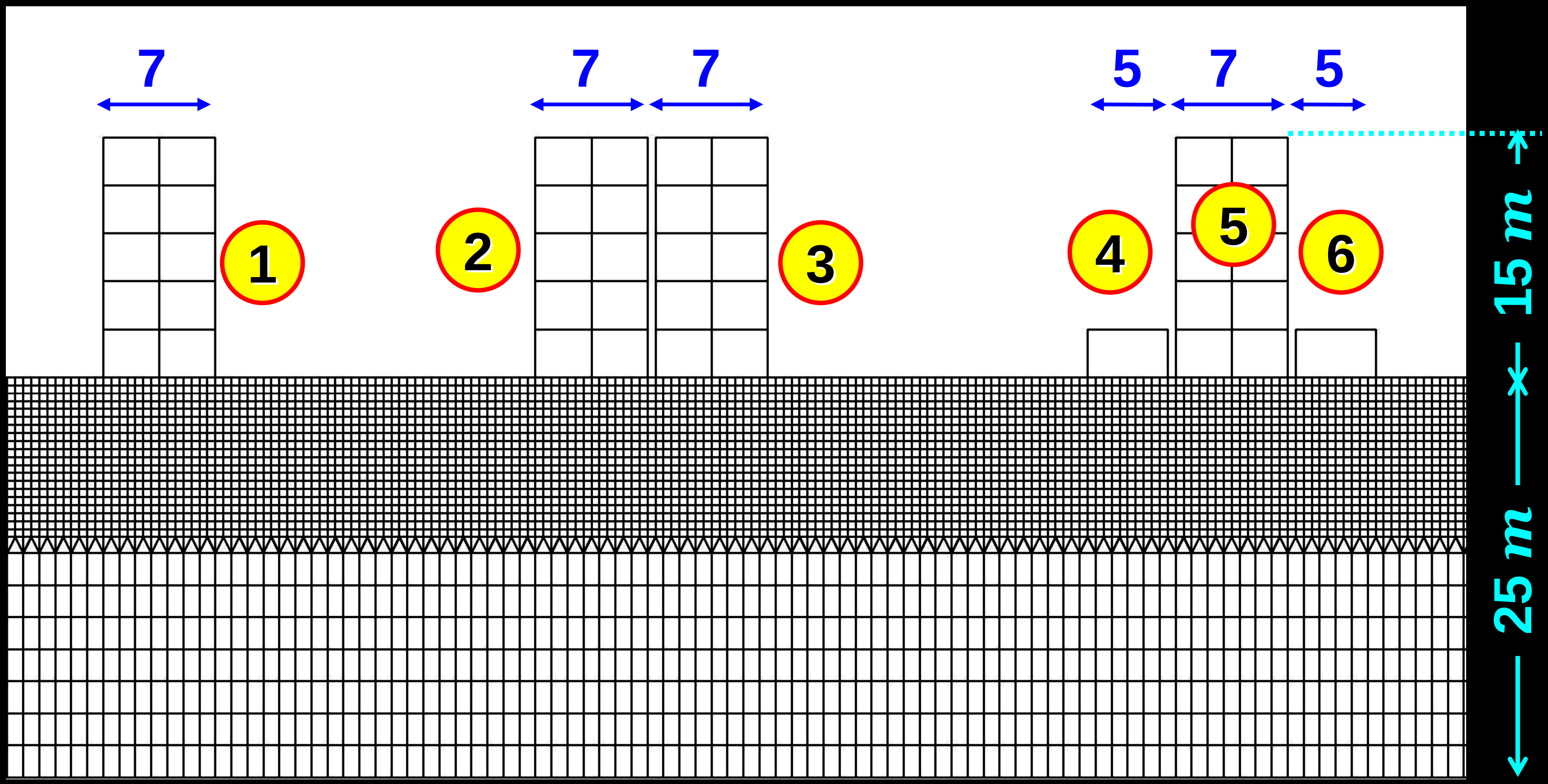

H
θ
B


AM

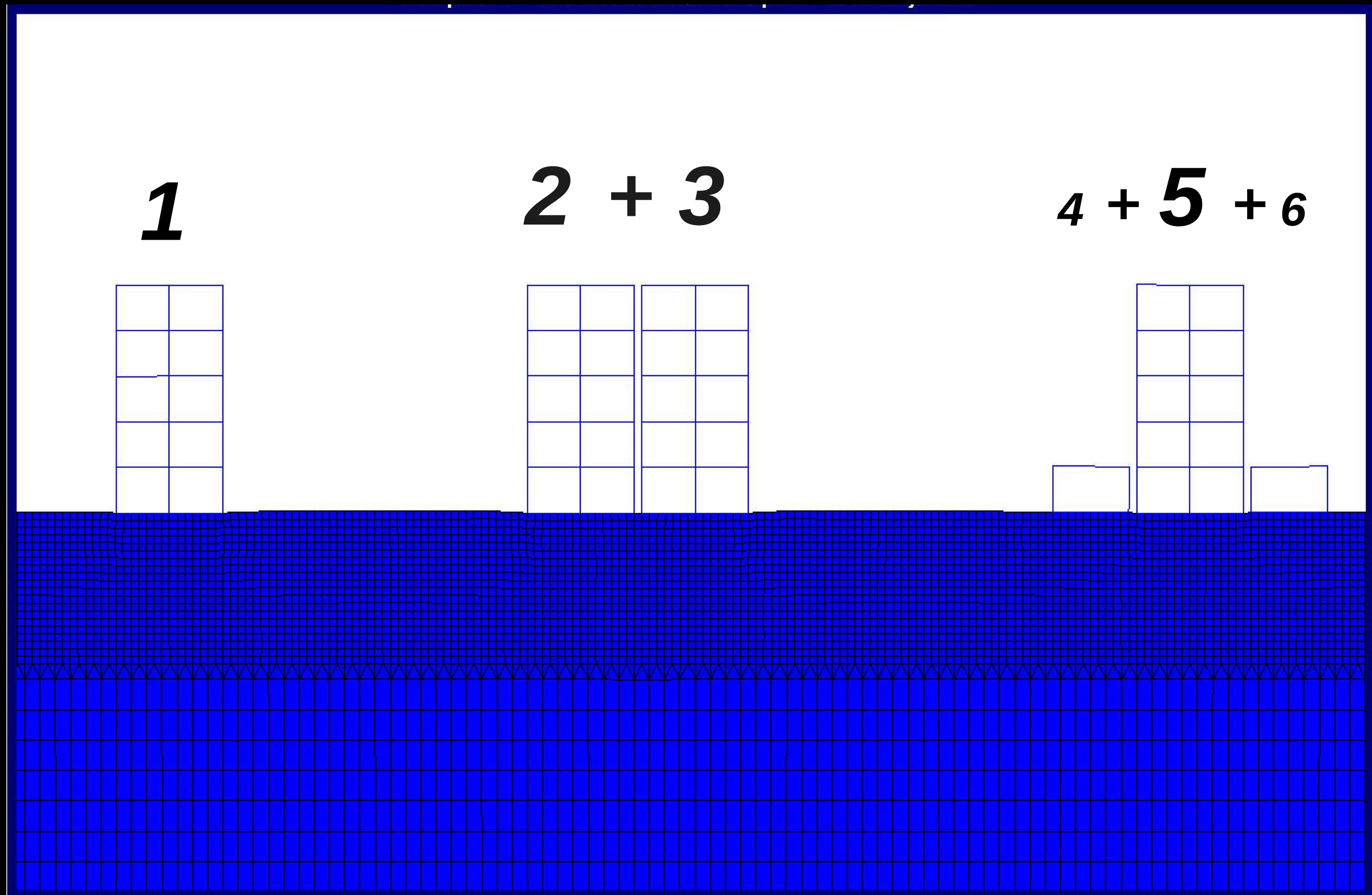

Overturning
θ°
0
2
4
6
0
1
2
3
H / B


GÜMÜŞSUYU
DİLEK HALICILIK ve MEFRUŞAT
HALI-ÇEYİZ
MEFRUŞAT
Aspect
Ratio
$H/B \approx 3$

2–D Seismic Response of Adapazari

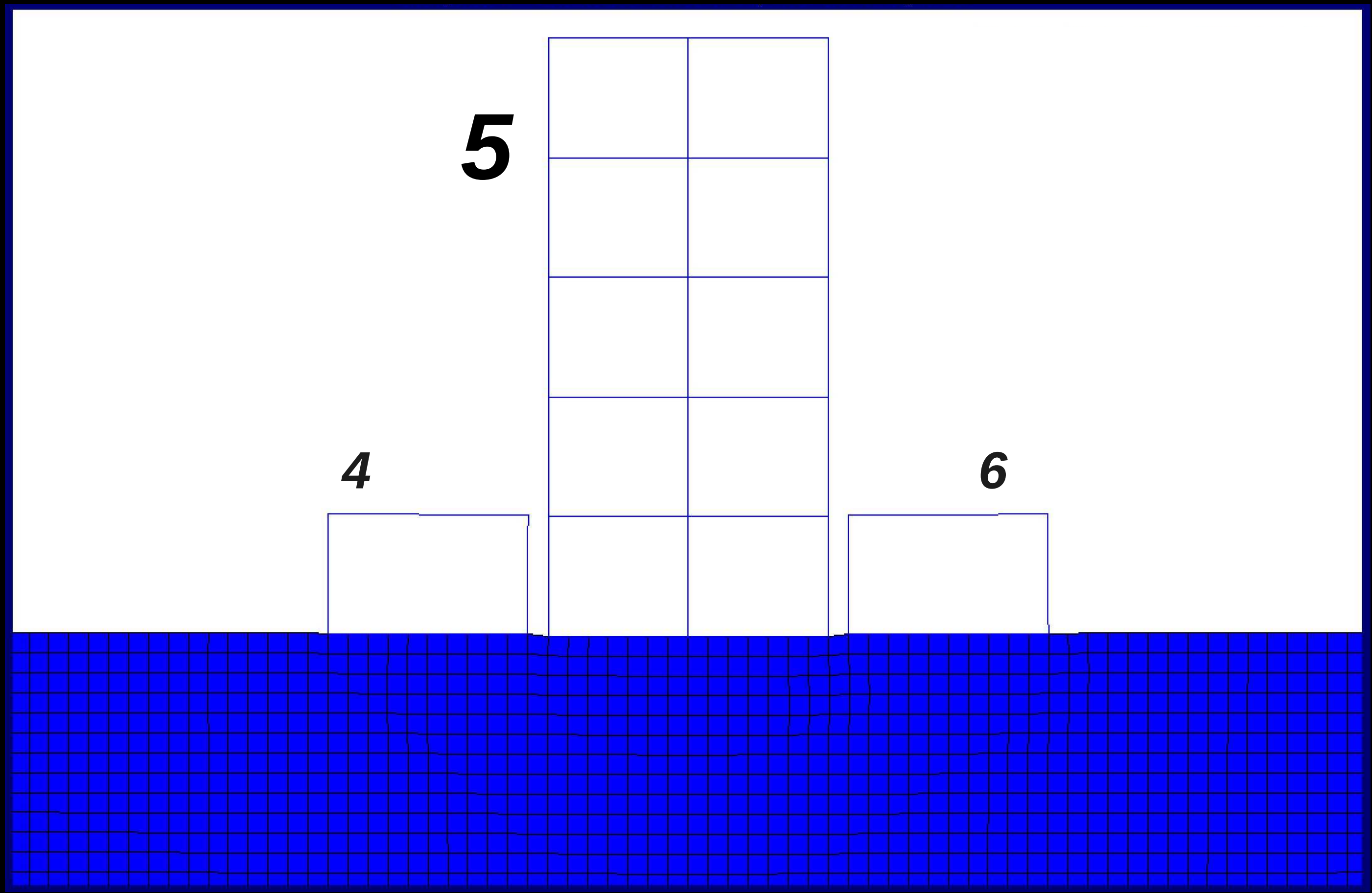


Buildings : 1, 2 + 3, 4 + 5 + 6



Deformation Scale : X 3

Buildings : 4 + 5 + 6



CONCLUSION:

The MAIN CAUSES of FAILURES

1. Large Overturning Moment
+ Very Soft Soils :

Bearing Capacity Failure

Lateral Soil Displacement
(squeezing out)

Volumetric Compression

2. *Large Periods* (*T* $\geq$ *2 sec*)

of ground oscillation with

A* ≈ *0.20 g — 0.30 g

But causes (1) and (2) are

(at least in some of the cases)

not sufficient

to explain the overturning even

of very slender buildings

3. ***A key culprit appears to be***

the PRESENCE of ADJACENT

Buildings ! ⇩

One–Directional Accumulation of Tilt

- ***as with downward sliding***
  on INCLINED plane,

- ***in contrast to the symmetric sliding***
  on HORIZONTAL plane.

This presentation was possible only thanks to my co-workers at NTUA:

Ioannis Anastasopoulos

Nikos Gerolymos

Marios Apostolou

Marianna Loli

Evangelia Garini

F I N

Merci Beaucoup

Pour votre attention