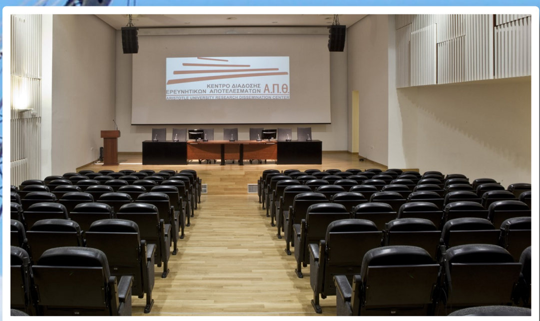
2nd Greek – Romanian Seminar

Lessons learned from Earthquakes and Geotechnical Failures



October 9th, 2025
Amphitheatre I, K.E.D.E.A.,
Aristotle University
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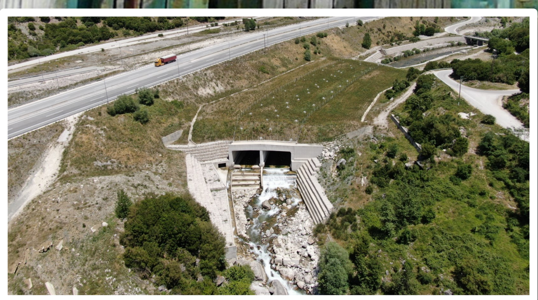

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2nd Greek-Romanian Seminar on Earthquake and Geotechnical Engineering "Lessons learned from Earthquake and Geotechnical Failures"

SOCIETATEA ROMÂNĂ DE GEOTEHNICĂ ȘI FUNDAȚII

The Hellenic Society for Soil Mechanics and Geotechnical Engineering (HSSMGE)

in cooperation with

the Romanian Society for Soil Mechanics and Foundation Engineering (SRGF)

proudly announce the

2nd Greek – Romanian Seminar

Lessons learned from Earthquakes and Geotechnical Failures

on October 9th 2025 at Amphitheatre I, K.E.D.E.A.,
Aristotle University of Thessaloniki

for acknowledging the cooperation between the respective professional societies and to create a platform sharing expertise and fostering collaboration between the earthquake and geotechnical communities of Romania and Greece.

With both Romania and Greece located in seismically active regions, the seminar will offer the opportunity to discuss the latest developments and challenges in earthquake and geotechnical engineering, especially through the challenging lense of failures.

This seminar follows a most successful 1st Romania – Greece Seminar on Earthquake and Geotechnical Engineering held in Bucharest in March 2025.

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Societatea Română de Geotehnică şi Fundaţii

This is the compiled pdf version of the presentations by speakers who presented at the 2nd Greek-Romanian Seminar on Earthquake and Geotechnical Engineering "Lessons learned from Earthquake and Geotechnical Failures", organized by the Hellenic Society for Soil Mechanics and Geotechnical Engineering (HSSMGE) and the Romanian Society for Soil Mechanics and Geotechnical Engineering (SRGF) in Thessaloniki, Greece, on the 9th of October 2025.

"Presenter/Presenters, 2025, Title of presentation, 2nd Greek-Romanian Seminar on Earthquake and Geotechnical Engineering "Lessons learned from Earthquake and Geotechnical Failures", Thessaloniki, Greece, 9th October 2025, pp. the pages from this compiled version."

Presentations edited in this compiled pdf version by M. Bardanis.

Cover and programme graphics by G. Belokas.

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2nd Greek-Romanian Seminar on Earthquake and Geotechnical Engineering "Lessons learned from Earthquake and Geotechnical Failures"

SOCIETATEA ROMÂNĂ DE GEOTEHNICĂ ȘI FUNDAȚII

2nd Greek – Romanian Seminar

Lessons learned from Earthquakes and Geotechnical Failures

October 9th 2025, *Amphitheatre I, K.E.D.E.A., Aristotle University of Thessaloniki*

09:15-09:30	Opening Addresses **Michalis Bardanis**, President, Hellenic Society for Soil Mechanics and Geotechnical Engineering **Loretta Batali**, President, Romanian Society for Soil Mechanics and Foundation Engineering **Yiannis Xenidis**, Head, School of Civil Engineering, Aristotle University of Thessaloniki **Ilias Pertzinidis**, President, Technical Chamber of Greece — Section of Central Macedonia
09:30-10:15	**Dimitrios Pitilakis** *Urban-scale risk assessment including SSI and site amplification*
10:15-11:00	**Loretta Batali** *Forensic geotechnical, hydrological and hydrogeological* analysis of instability phenomena occurred at a waste management centre
11:00-11:15	Coffee Break
11:15-12:00	**Dan Iancu** & Horatiu Popa *Influence of pile stiffness on behaviour of slender base isolated structures*
12:00-12:45	**Alexandra Ene**, Horațiu Popa, Loretta Batali & Dragoș Marcu *Treatment of uncertainties for a deep excavation project in complex ground conditions*
12:45-13:30	**Evangelia Garini** *Soil Effects and Geotechnical failures in the 2023 Kahramanmaraş Earthquakes in Turkey*
13:30-14:30	Lunch Break
14:30-15:15	**Evi Riga** *Verification of seismic risk models using observed damage from past earthquake*
15:15-16:00	**Anastasios Anastasiadis** *Subsurface Structure and Dynamic Soil Properties as Basis for Microzonation and Site Effects Studies: Lessons Learned from Studies in Greece and Cyprus*
16:00-16:45	**Michalis Bardanis** *Identification, investigation and remediation of slow-moving landslides*
16:45-17:30	Discussion and Closing Panel **Kyriazis Pitilakis**, **George Gazetas**, **Loretta Batali**, **Michalis Bardanis** Moderator: **Giorgos Belokas**

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Societatea Română de Geotehnică şi Fundaţii

The 2nd Greek-Romanian Seminar in Thessaloniki on the 9th of October 2025 – Opening Addresses

2nd Greek – Romanian Seminar: Lessons learned from Earthquakes and Geotechnical Failures

The 2nd Greek-Romanian Seminar in Thessaloniki, 9th of October 2025 - All speakers & panelists

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ
INVITED SPEAKERS
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Dimitris Pitilakis

Professor, Department of Civil Engineering, Faculty of Engineering, Aristotle University of Thessaloniki, Greece

Urban-scale risk assessment including SSI and site amplification

Dimitris Pitilakis is a Full Professor in the Department of Civil Engineering of the Aristotle University of Thessaloniki, Greece (M.Sc. in Engineering, University of California, Berkeley, Ph.D. in Earthquake Engineering from Ecole Centrale Paris, France). His research departs from soil-structure interaction and geotechnical earthquake engineering and aims toward the vulnerability assessment and resilient-based design of soil-foundation-structure systems at a local or urban scale. He has also been focusing on Geotechnical Seismic Isolation using soil mixtures with recycled materials, such as recycled tires. In addition, he has also been working on an earthquake early warning and early damage assessment of critical infrastructures, such as schools or industrial structures. He is a member of the TG207 committee of ISSMGE on soil-structure interaction and retaining walls. He is the author of more than 200 papers in peer-reviewed scientific journals and international conference proceedings. He is a member of national and international scientific societies on Earthquake Engineering and a reviewer of international scientific journals. He has developed software to simulate the soil-foundation-structure interaction, emphasizing nonlinear soil and structure behavior, and software for foundation design and analysis. He has extensive experience in experimental soil-foundation-structure interaction in small-scale (shaking table and centrifuge) and full-scale (EuroProteas in Euroseistest http://euroseisdb.civil.auth.gr/sfsis) facilities. He is currently in charge of the shaking table and the full-scale EuroProteas facility of the Laboratory of Soil Dynamics and Geotechnical Earthquake Engineering of the Aristotle University of Thessaloniki.

Urban-scale seismic risk assessment

Dimitris Pitilakis
Aristotle University of Thessaloniki

2nd Greek-Romanian Seminar
Lessons learned from Earthquakes

Who am I?

- Dimitris Pitilakis, MSc (UC Berkeley 2003), PhD (ECP, 2006)
- Civil Engineer
- Professor
- Department of Civil Engineering
- Aristotle University of Thessaloniki
- Director of the MSc program on Sustainable Design of Structures against Earthquakes and other Natural Hazards

Dimitris Pitilakis

Motivation

- There is/was no “direct” way to integrate the important effects of soil structure interaction (SSI) and site amplification (SAmp) in seismic risk analyses at a building / block / city scale

1. To propose a holistic and modular method to estimate the seismic risk of an existing building, including SA and SSI effects (Detailed structural modeling approach)

2. To propose an efficient and modular method to estimate the seismic risk of building stock, including SA and SSI effects (Simplified structural modeling approach)

Dimitris Pitilakis

3

Motivation

- There is/was no “direct” way to integrate the important effects of soil structure interaction (SSI) and site amplification (SAmp) in seismic risk analyses at a building / block / city scale

1. To propose a holistic and modular method to estimate the seismic risk of **one existing building**, including SAmp and SSI effects (*Detailed* structural modeling approach)

2. To propose an efficient and modular method to estimate the seismic risk of a building stock, at an **urban scale**, including SAmp and SSI effects (*Simplified* structural modeling approach)

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Framework

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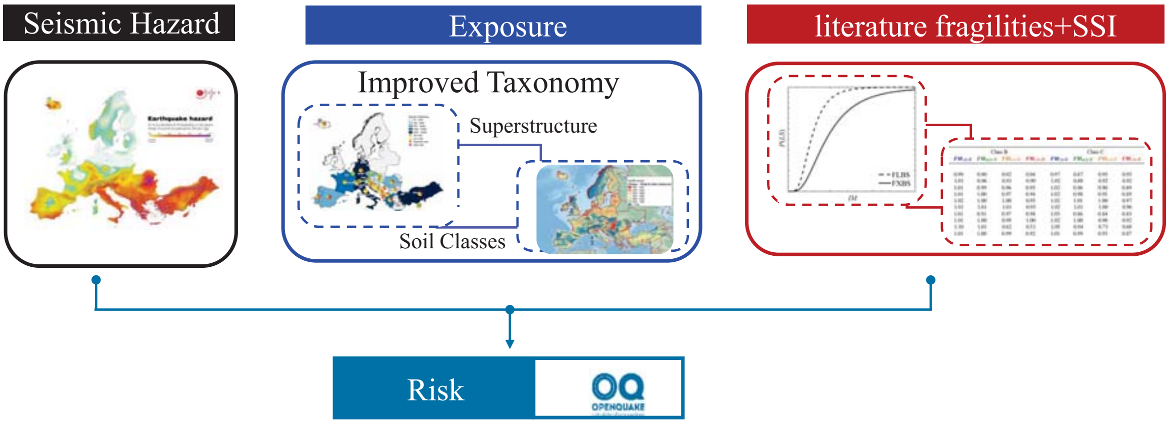
Risk=hazard·exposure·vulnerability



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Keywords

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- Risk assessment
- Fragility
- Vulnerability
- Soil-structure interaction
- Site amplification
- Nonlinear soil behavior
- Intensity measure
- Engineering demand parameter

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Detailed structural modeling "DSM"

Single existing structure

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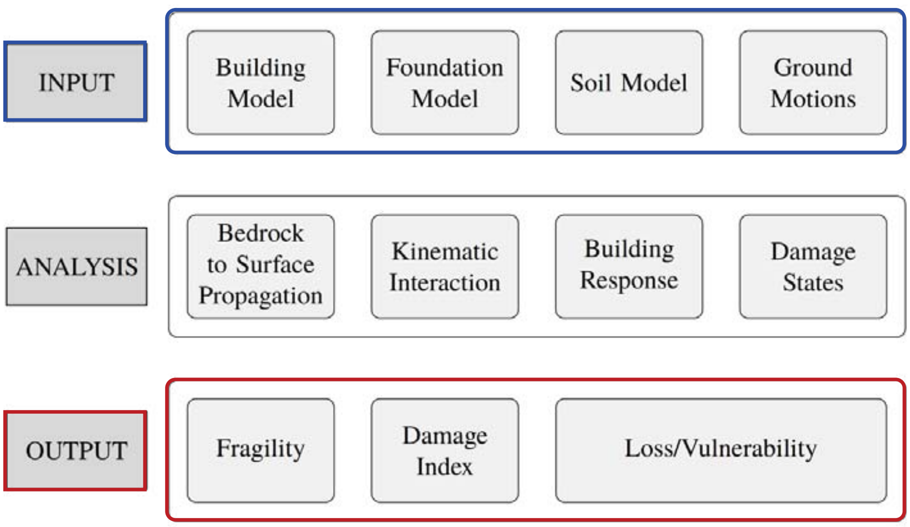
Basic methodology



[Petridis & Pitilakis, EQ Spectra, 2020]

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Key elements

- Run **IDA**, starting **from the soil** !
- Choose ground motions **recorded on** stiff soil / **rock** / EC8 type A
- Include (**nonlinear**) **site amplification** (SAmp) effects -> proper soil constitutive modeling
- Use **kinematic interaction** equations to get foundation input motion (FIM) from the free-field ground motion
- Model elaborately the soil-foundation system (for example, with the BNWF model)
- Run **IDA for the building** using the foundation motion
- Model essentially any kind of building (steel, masonry etc)
- Produce fragility curves for the soil-foundation-structure system
- Witness the effects of SAmp and/or SSI on the seismic fragility of the building

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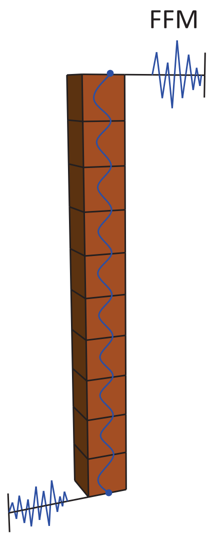
Soil profile and wave propagation



- Rock , $V_s \geq 800$m/s
- Clay or Sand, 30m deep
- *OpenSees: "PressureIndependMultiYield" / "PressureDependMultiYield"*
- IDA analyses
- Bedrock to free-field ground motion

[Petridis & Pitilakis, EQ Spectra, 2020]

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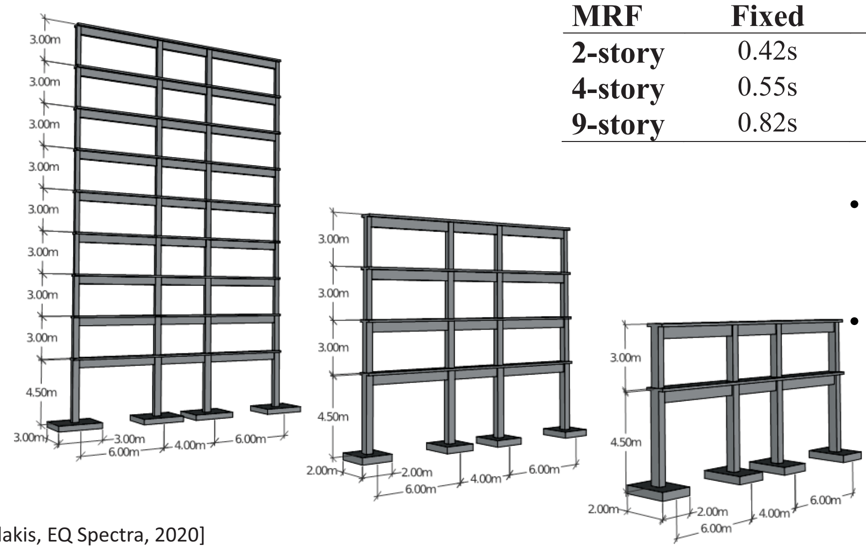
Existing RC MRF / Dual buildings



RC MRF	Foundation Fixed	BNWF
2-story	0.42s	0.47s
4-story	0.55s	0.63s
9-story	0.82s	0.92s

- Designed according to the 1959 Greek seismic code (non-ductile)
- Footings without connecting beams

[Petridis & Pitilakis, EQ Spectra, 2020]

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Fragility assessment

- Outcrop PGA as the intensity measure (IM)
- Maximum interstory drift as the engineering demand parameter (EDP)
- Lognormal cumulative distribution of PGA-Drift pairs (IM-EDP)
- 4 limit/damage states
  - Slight Damage (SD)
  - Moderate Damage (MD)
  - Extensive Damage (ED)
  - Complete Damage (CD)
- Uncertainties related to the limit state value, the capacity and the seismic demand

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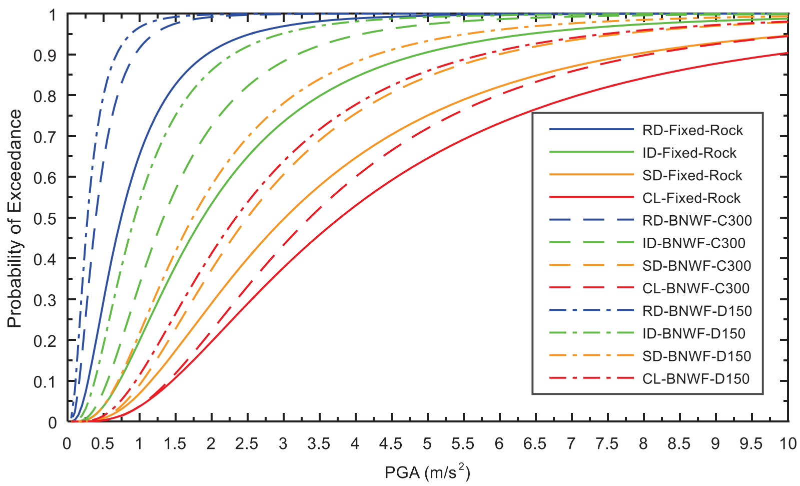
Fragility curves: flexible MRF on different soil



Here the fragility increases with SAmp and SSI

* The PGA is @bedrock!

[Petridis & Pitilakis, EQ Spectra 2020]

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What about dual systems

Dual systems: shear walls + moment resisting frames

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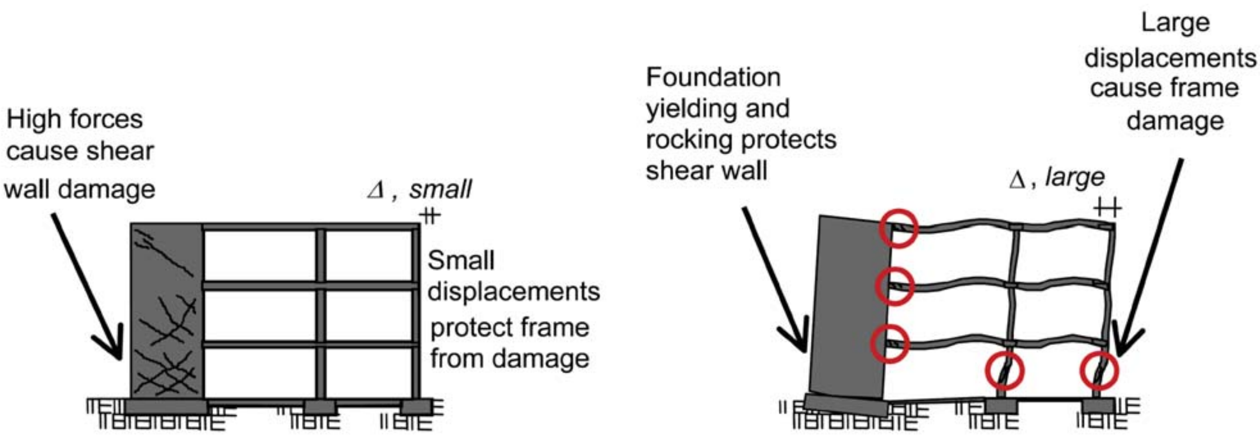
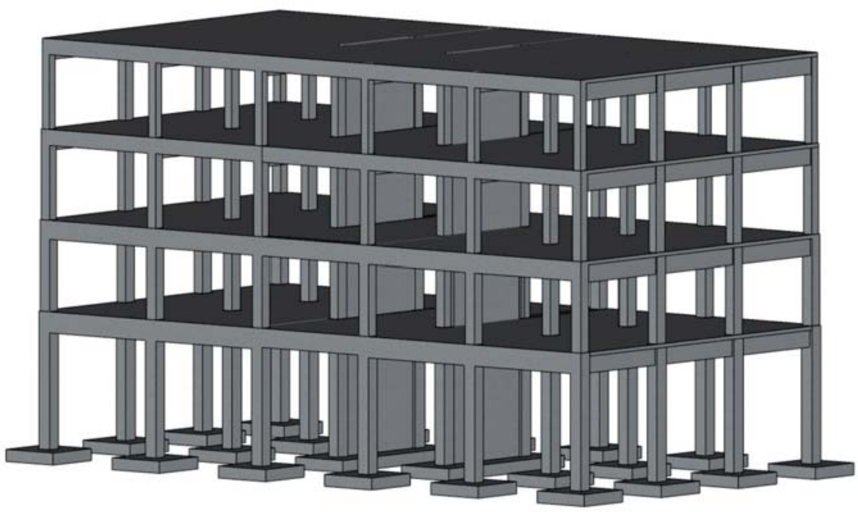
Effect of soil-foundation compliance

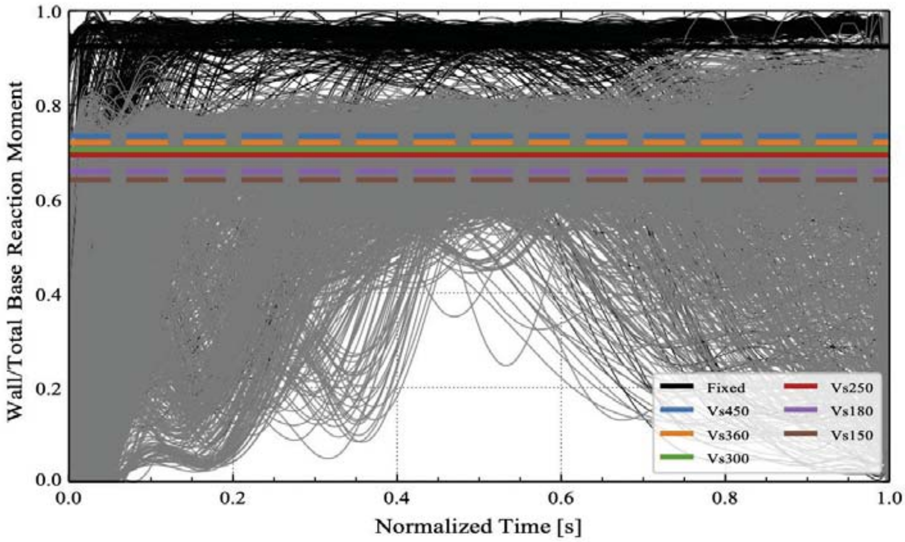


[ATC 1196, Raychowdhury 2011]

Typical 4-story RC dual bare building



Shear wall – to – total base moment (4-story)



The frame attracts larger portion of the lateral load because of SAmp and SSI

[Petridis & Pitilakis, EQ Spectra, 2020]

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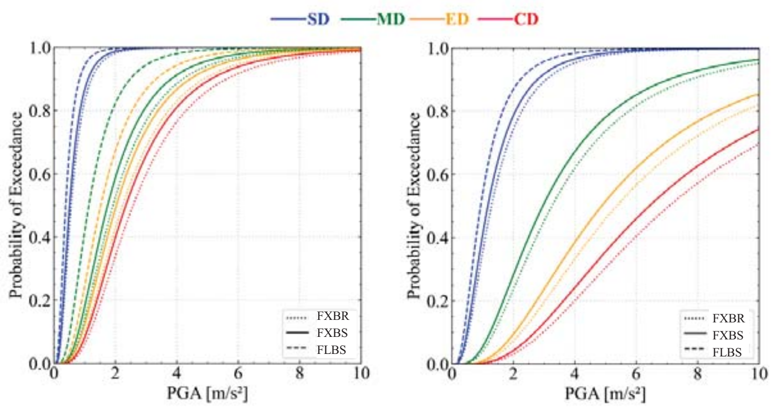
4-story MRF (left) and Dual (right) on clay

- Regularly infilled building on soil with V_s =250m/s



FXBR=fixed-base-rock
FXBS=fixed-base-soil
FLBS=flexible-base-soil

[Pitilakis & Petridis, Eng Str, 2022]

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Ok, but how do we **simplify** things?

For engineers or stakeholders

Fragility modifiers "FM"

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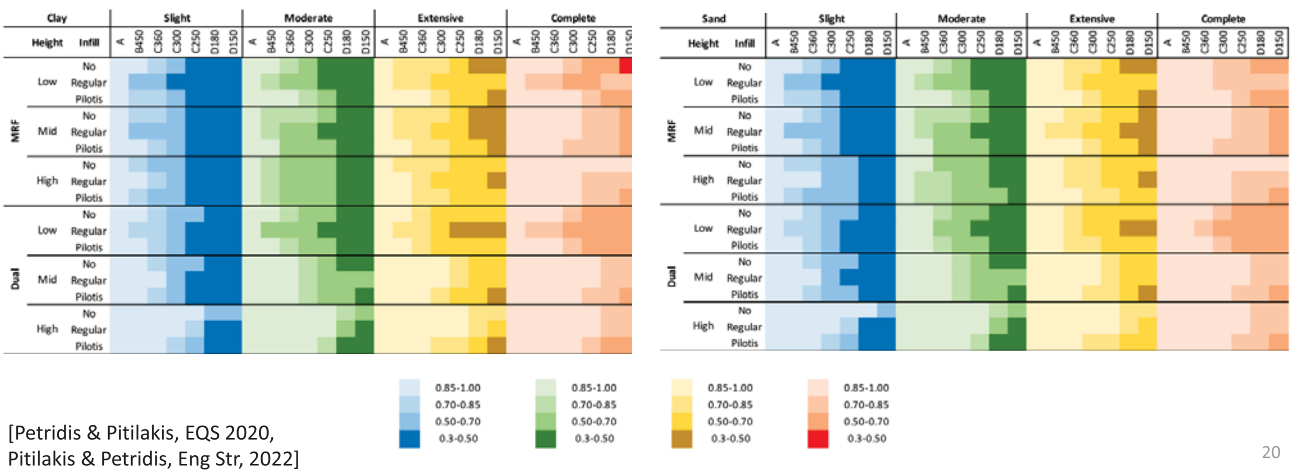
Clay

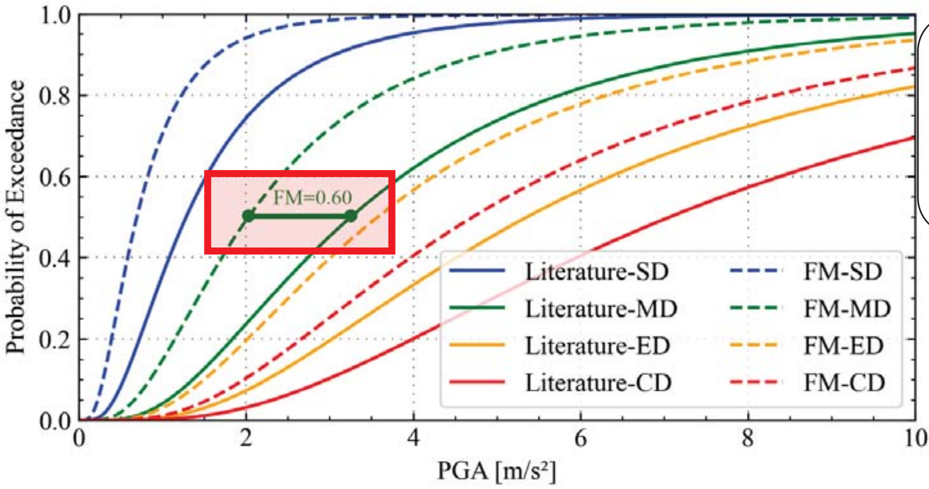
$$FM_i = \frac{PGA_{50\%,flexible-base,i}}{PGA_{50\%,fixed-base,i}}$$

Sand



[Petridis & Pitilakis, EQS 2020, Pitilakis & Petridis, Eng Str, 2022]

4-story regularly infilled dual system on clay Vs180



Read the FM value on the heatmap and multiply the literature curve

[Petridis & Pitilakis, EQ Spectra, 2020]

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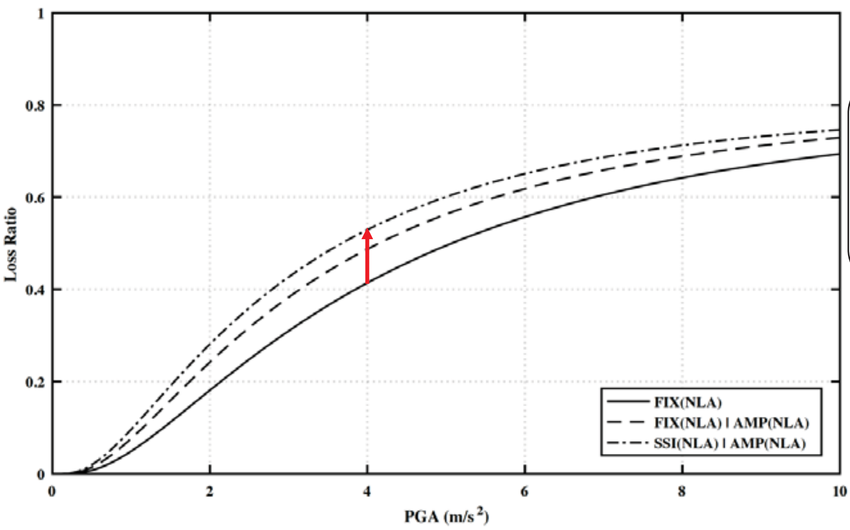
Loss ratio w/ soil amplification and SSI effects



Loss increases, well, it's good to know

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Simplified structural modeling "SSM"

Best-suited to large scale risk assessment

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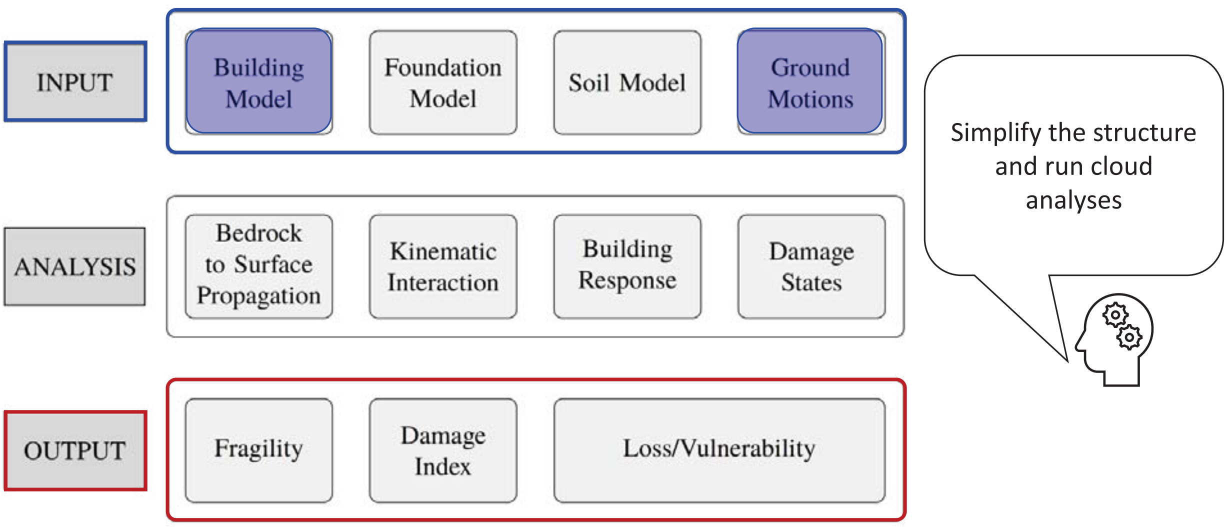
Basic methodology



[Amendola & Pitilakis, BEE, 2023]

Key elements

- Method designed explicitly for **urban-scale** risk assessment
- **Using globally available data** for the soil, foundation, and the building taxonomy
- Extensive set of ground motions
- The structure is modeled as an equivalent single-degree-of-freedom (**ESDoF**)
- An **enhanced taxonomy** is compiled to make the approach implementable in the **OpenQuake** platform, introducing
  - $\mathbf{V_{S,30}}$ and **H/B** as **proxies** for the **SAmp** and **SSI** effects

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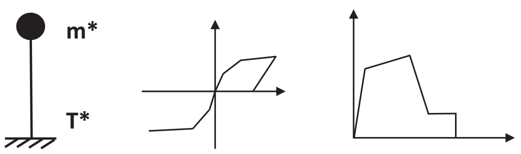
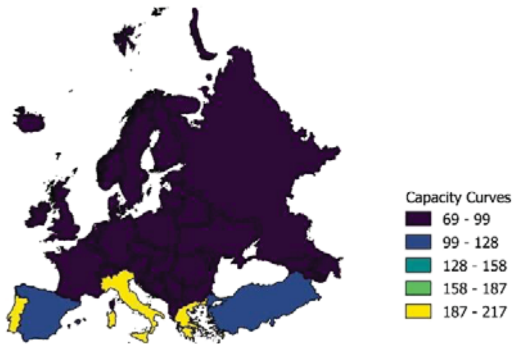
Simplified structural modeling

- ESDoF: equivalent mass m* and vibration period T*
- GEM database of capacity curves
- Different hysteretic lows (Opensees library)





[D' Ayala et al, 2014 , Martins & Silva, 2020]

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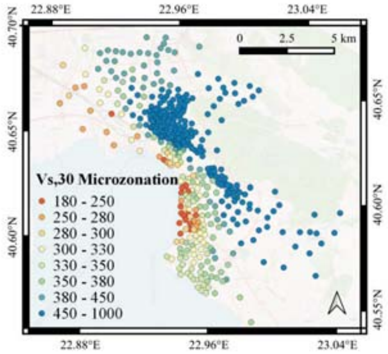
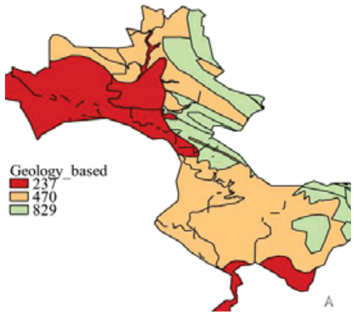
Soil: how to get $V_{S,30}$ from available data

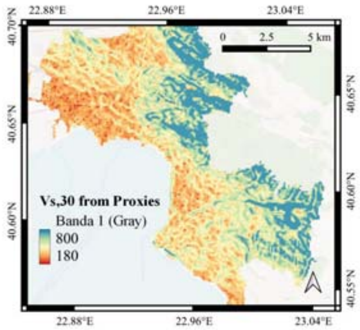
- $V_{S,30}$ is selected as a proxy for soil conditions. Different approaches are available to compute $V_{S,30}$ maps from globally available data



Microzonation studies



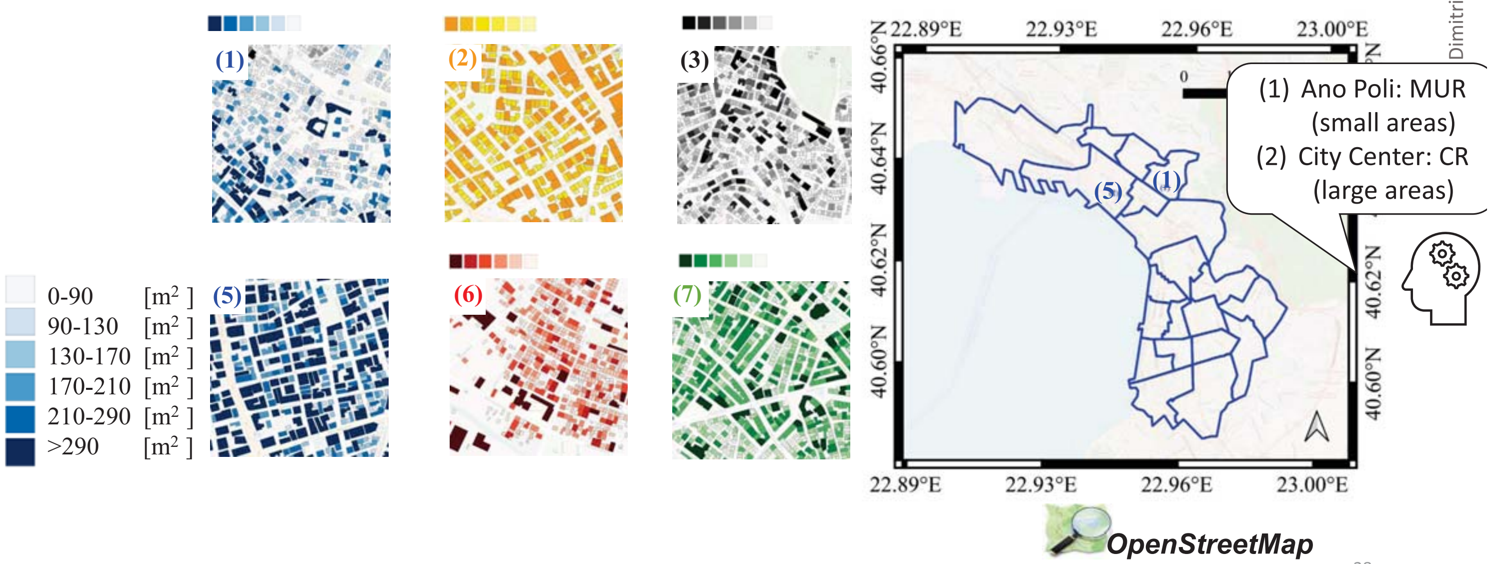
Local geology



Slope-based methods

[Anastasiadis et al, 2002, Wald & Allen, 2007]

Foundation footprint area from OpenStreetMap



Input: Records

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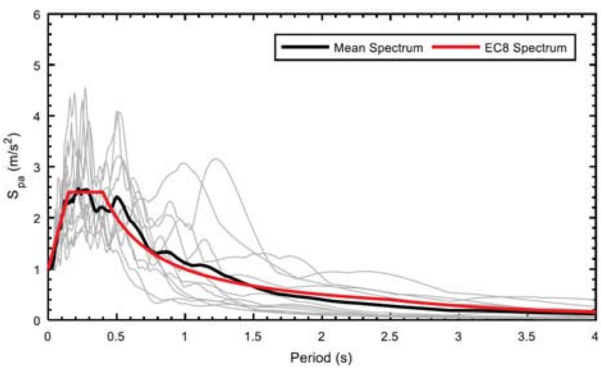
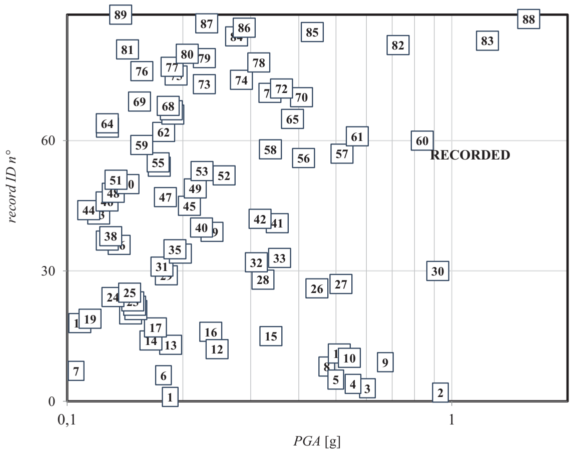
Spectrum-compatible: IDA, MSA



Not scaled: Cloud analysis



Results: fragility curves including SSI and SAmp

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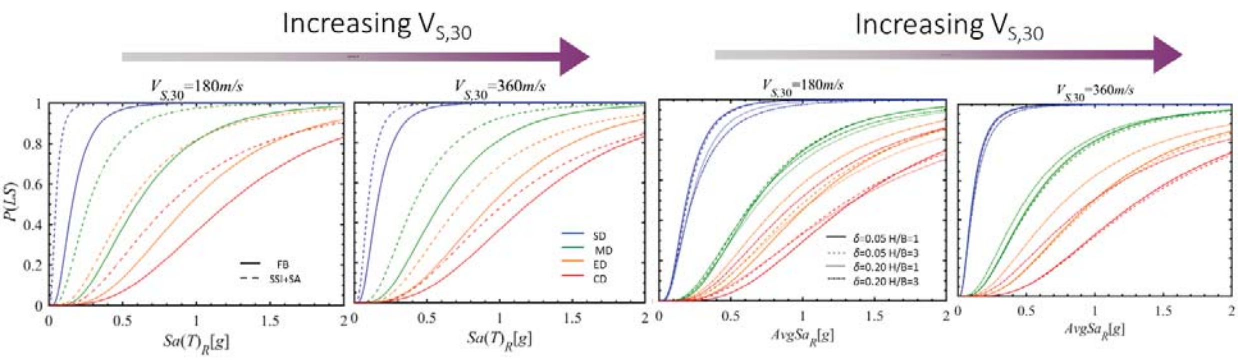
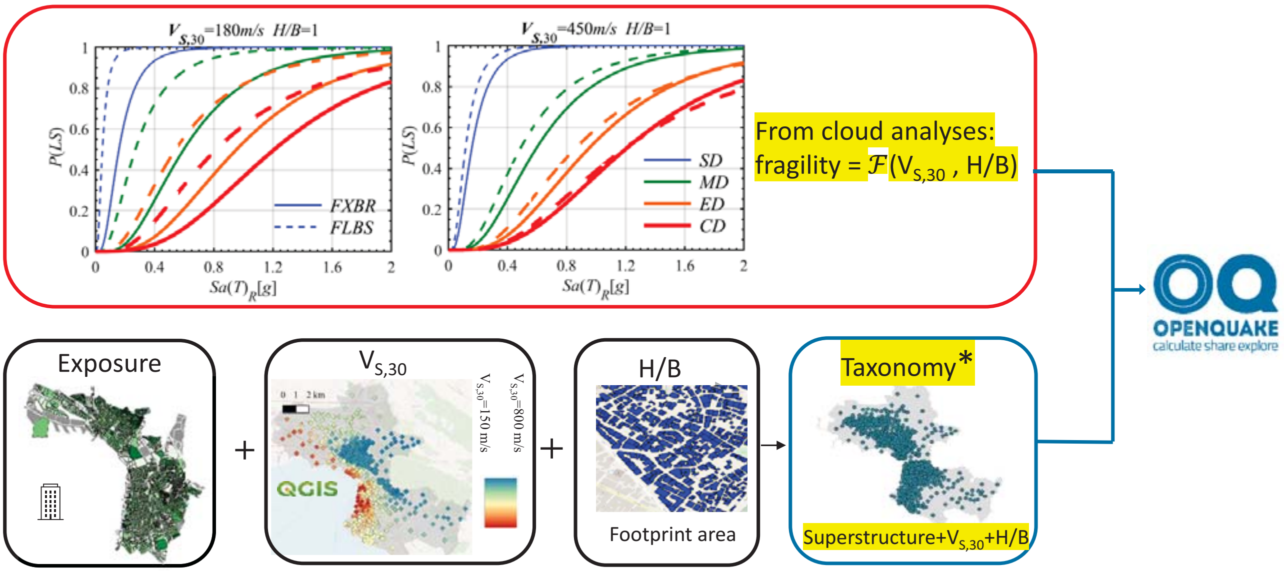


For very soft deposit:

- Considering SSI and SAmp → increase of the structural fragility
- Large H/B + δ ratios + nonlinear SSI→ increase of the structural fragility in high damage states (collapse cases occur)

Implementation in OpenQuake



[Amendola & Pitilakis, BEE 2023]

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Fragility assessment of a city block in Thessaloniki

Application of the DSM and SSM approaches for comparison

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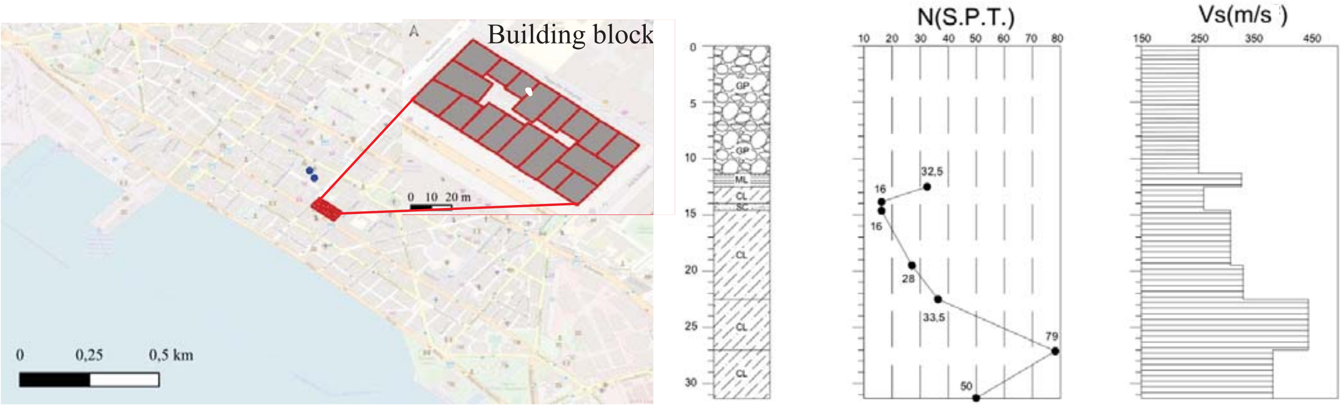
Building block at the historical city center



[Petridis & Pitilakis, BEE, 2021, Pitilakis et al, 3ECEES 2022]

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of stories / typology / seismic code / footprint

ID	N° stories	Str. Typology	Seismic Code	Area (m²)
k1	5	MRF	RD’59	1200
k2	9	MRF	RD’59	1710.78
k3	5	DUAL	RD’59	1278.45
k4	8	DUAL	RD’59	2160
k5	9	MRF	RD’59	1300
k6	5	DUAL	RD’59	962
k7	9	MRF	RD’59	2160
k8	3	MRF	RD’59	864
k9	9	DUAL	RD’59	1710.78
k10	7	MRF	RD’59	1212.75
k11	9	DUAL	RD’59	1519.56
k12	6	MRF	RD’59	1596.48
k13	7	MRF	RD’59	1919.4
k14	8	DUAL	AP’84	1814.4
k15	6	MRF	RD’59	1666.65
k16	9	DUAL	RD’59	1944
k17	7	MRF	RD’59	2016
k18	10	DUAL	RD’59	3880.46

[Petridis & Pitilakis, BEE, 2021, Pitilakis et al, 3ECEES 2022]

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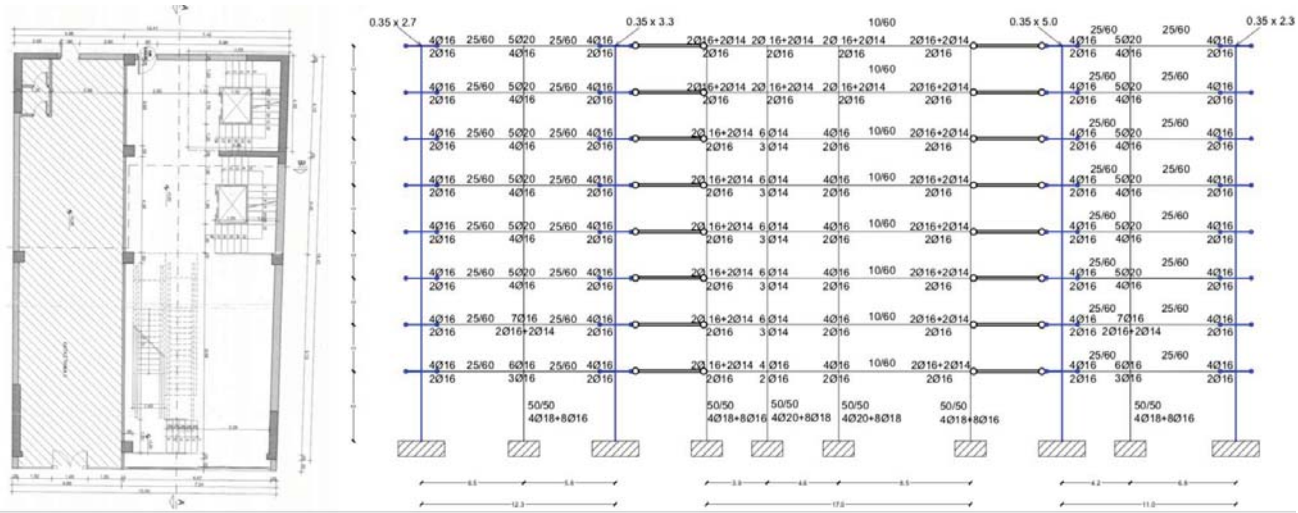
1.Detailed structural modeling (DSM)



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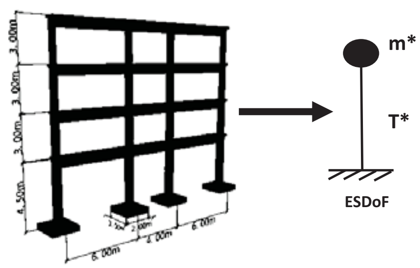
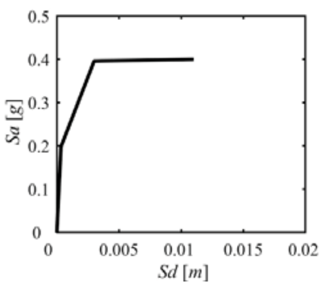
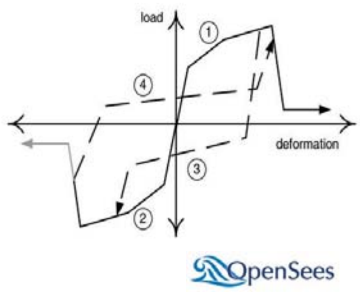
2.Simplified structural modeling (SSM)

- “two-node link" element
- Literature capacity curves from pushover analyses of 2D models
- Proper hysteresis model







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Comparison of DSM and SSM

Fragility curves for buildings K1 and K3 of the city block

DSM: fixed-base-on-soil vs flexible-base-on-soil

FXBS vs FLBS:
Effect of **SSI**

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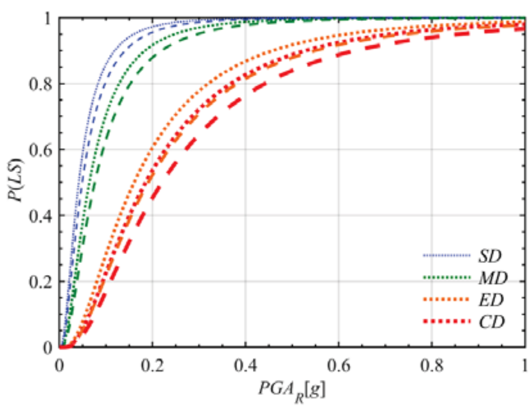
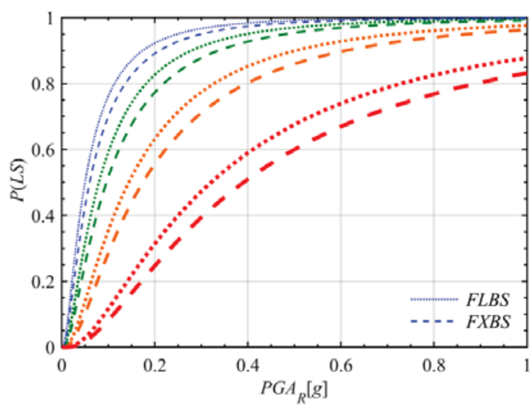
K1 - MRF

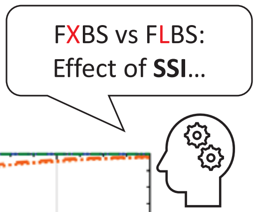


K3 - DUAL

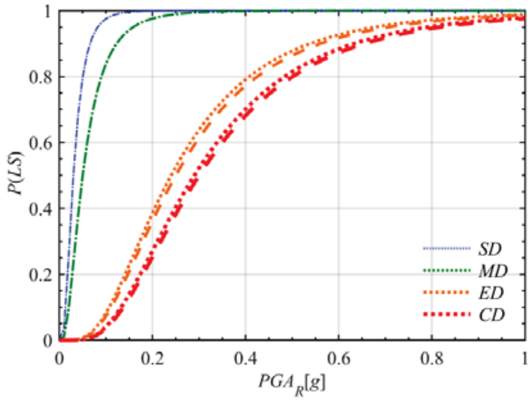


[Pitilakis et al, 3ECEES, 2022]

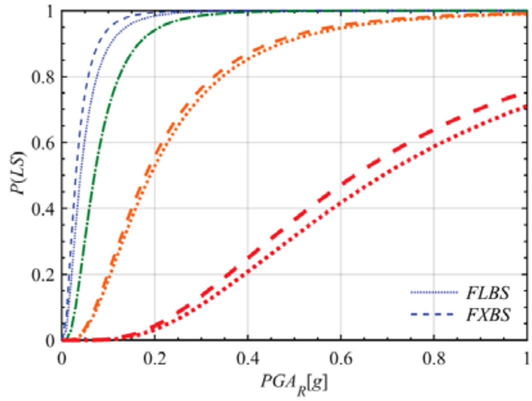
SSM: fixed-base-on-soil vs flexible-base-on-soil



K1 - MRF



K3 - DUAL



[Pitilakis et al, 3ECEES, 2022]

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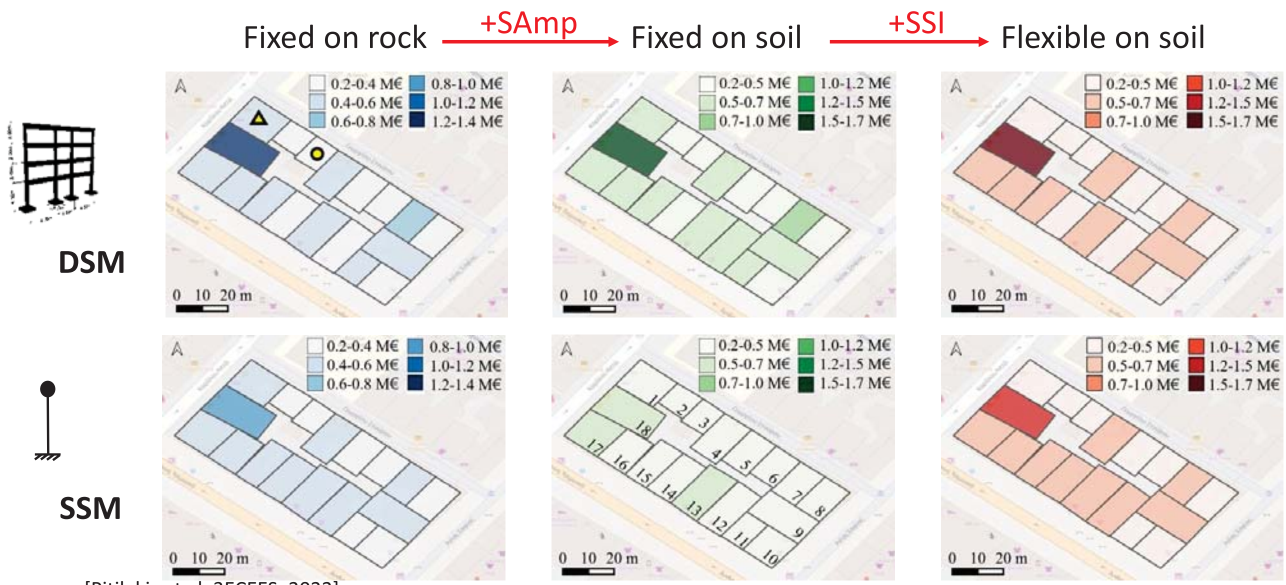
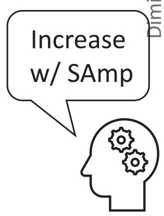
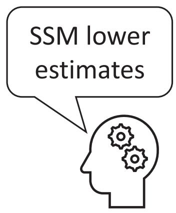
Distribution of loss (for the 1978 Thessaloniki EQ)







[Pitilakis et al, 3ECEES, 2022]

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Application to the whole Thessaloniki

SSM approach only, obviously

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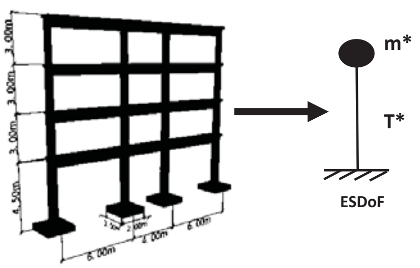
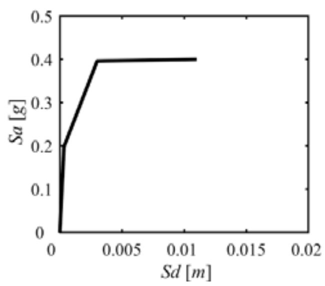
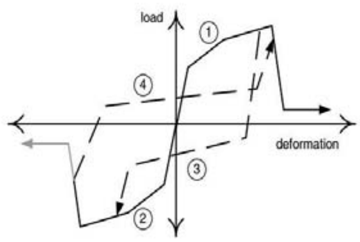
Simplified structural modeling: ESDOF

- “two-node link" element
- Literature capacity curves from pushover analyses of 2D models
- Proper hysteresis model

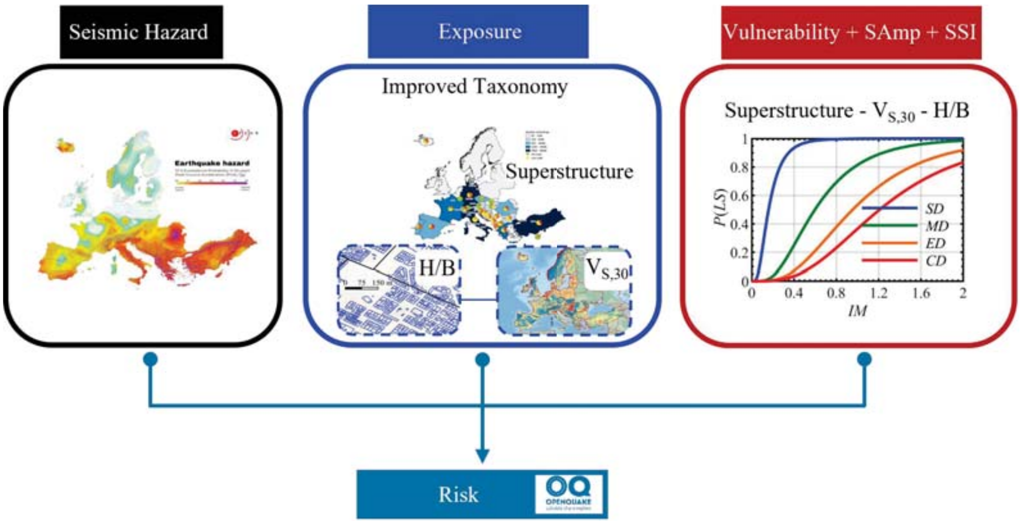






OpenSees

Risk assessment



[Amendola & Pitilakis, BEE, 2023]

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We applied two approaches for comparison

Common approach

- Ground motion at free field
- SAmp from amplification factors in GMPEs from measured Vs profiles
- *Fragility curves by GEM for fixed-base structures*

This study

- Ground motion at outcropping bedrock
- IMs from input EQ records (rock sites)
- *Fragility curves, including SSI and SAmp from site response analyses*

[Amendola & Pitilakis, BEE, 2023]

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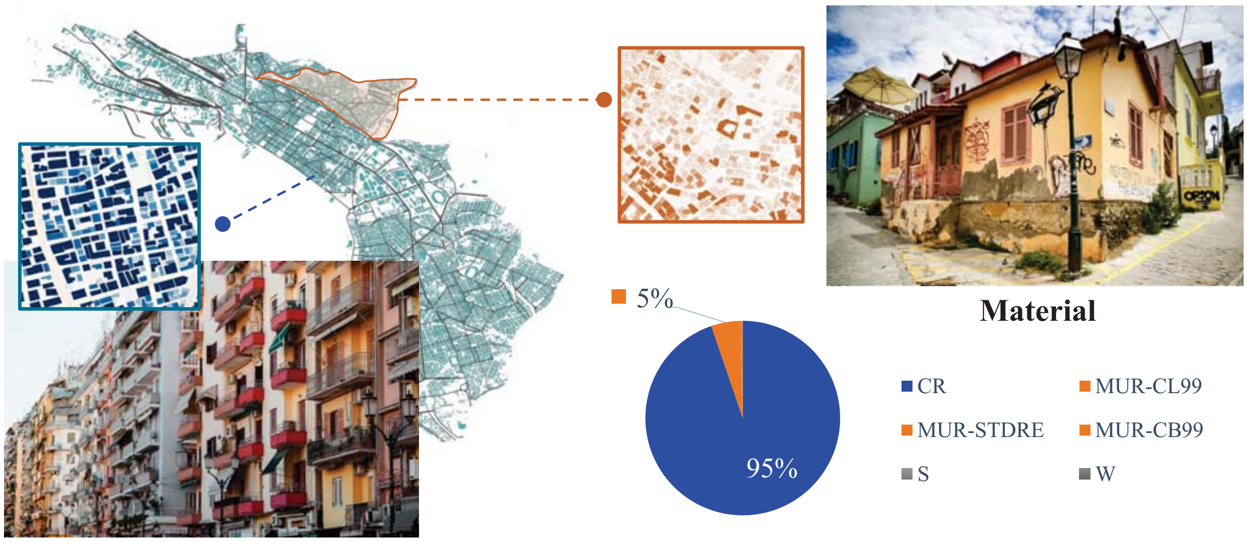
Application of SSM at the whole city



[Amendola & Pitilakis, BEE, 2023]

Case study: Mw6.5, Thessaloniki 1978 event

Scenario-based damage risk assessment

- Distribution of damages inferred by the Thessaloniki building stock for the Mw 6.5 1978 Thessaloniki Earthquake
- Considering:
  1. **Fixed-base-on-rock (FB):** Fixed-base fragility functions and no SAmp
  2. **Fixed-base-on-soil (FB+Samp):** Fixed-base fragility functions and SAmp using amplification factors in GMPEs
  3. **Flexible-base-on-soil (SAmp+SSI):** Fragility functions accounting for SSI and SAmp

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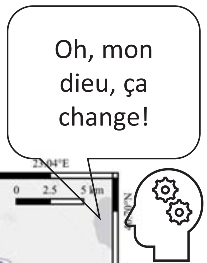
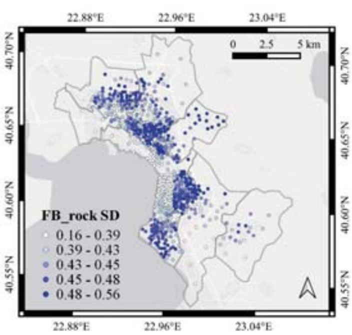
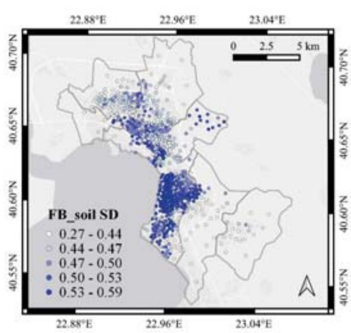
Results: Damage distribution

- % of n° of buildings in SLIGHT damage state

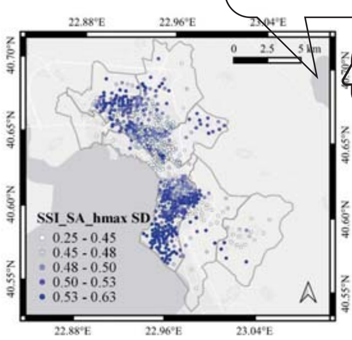




Fixed-base-on-**rock**



Fixed-base-on-**soil**



Flexible-base-on-**soil**

[Amendola & Pitilakis, EQS, 2023]

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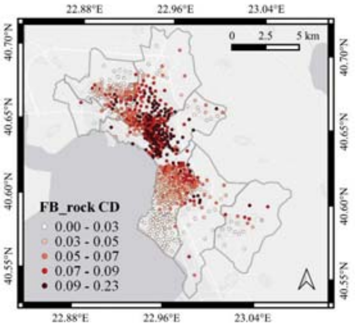
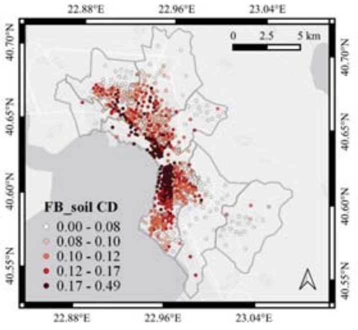
Results: Damage distribution

- % of n° of buildings in COMPLETE damage state

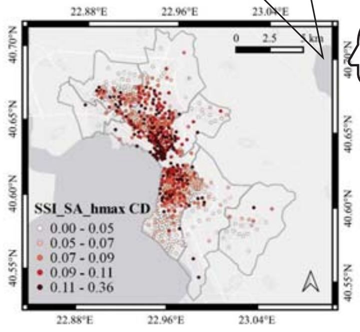
Oh, mon dieu, ça change!



Fixed-base-on-**rock**



Fixed-base-on-**soil**

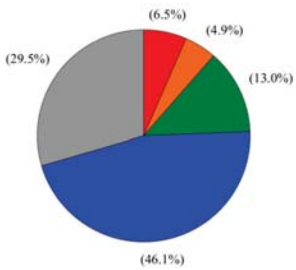


Flexible-base-on-**soil**

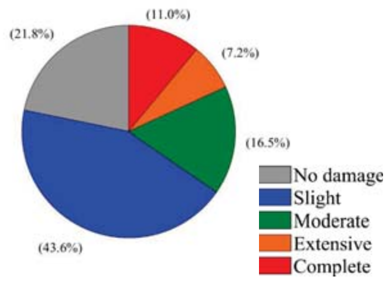
[Amendola & Pitilakis, EQS, 2023]

Results: Aggregated damages (@ city level)

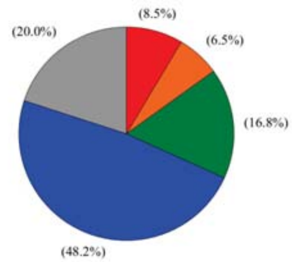
Fixed-base-on-**rock**



Fixed-base-on-**soil**



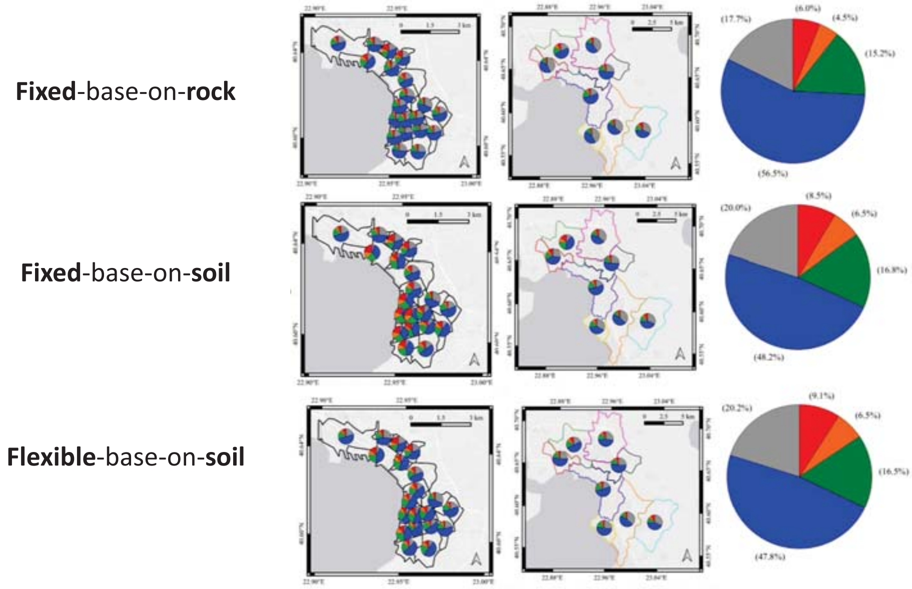
Flexible-base-on-**soil**



Aggregated results do not show much difference when comparing the **Fixed**-base-on-**soil** and the **Flexible**-base-on-**soil** approaches → <u>site amplification is the most important</u>

[Amendola & Pitilakis, EQS, 2023]

Results: Aggregated damages (@ different level)



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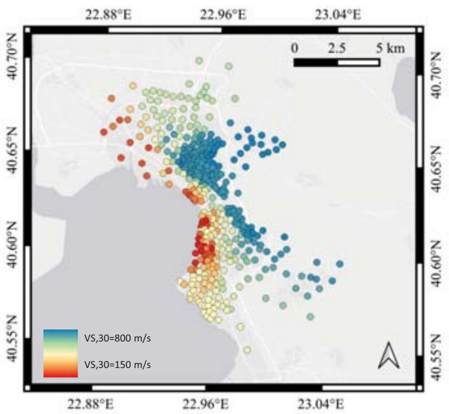
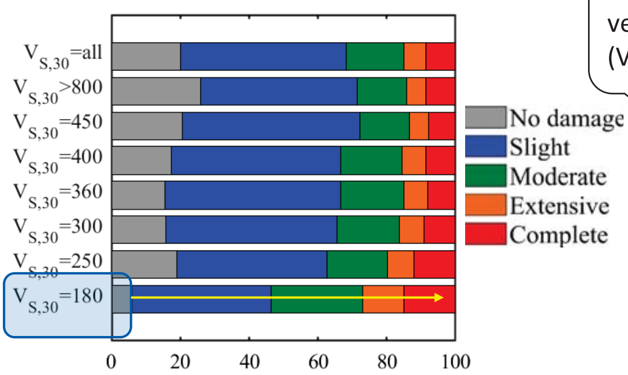
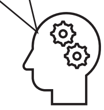
Damages aggregated by $V_{S,30}$

Damages aggregated in terms of $V_{S,30}$ show that for soft soil profiles there is an increase of the probability of having higher damages





especially for very soft soil ($V_{S,30}$=180 m/s)



[Amendola & Pitilakis, EQS, 2023]

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Comparison of damages

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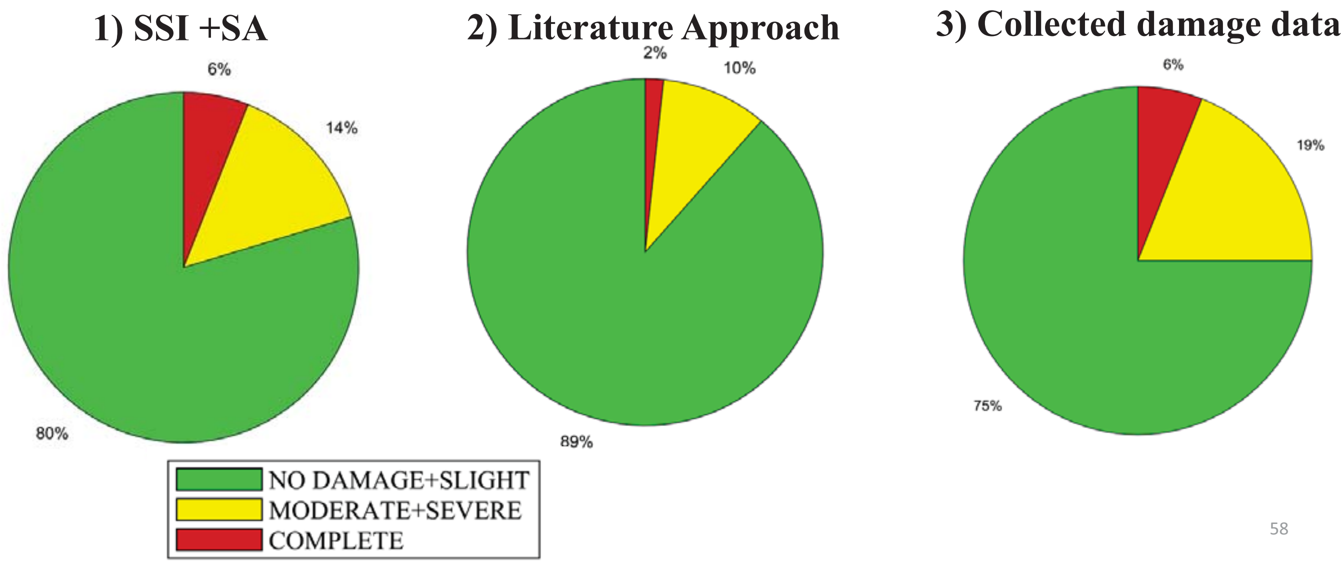

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Scenario damage analyses for Thessaloniki

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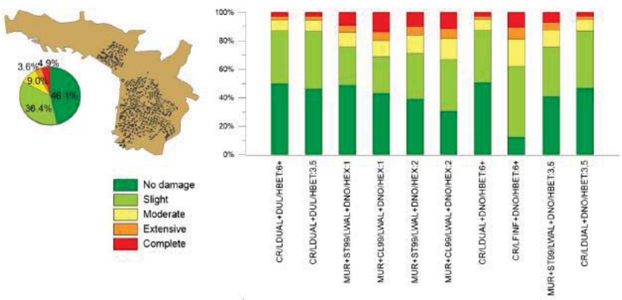
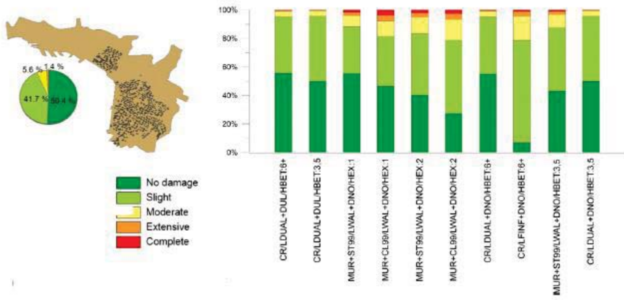
Thessaloniki 1978 EQ, 8,000 buildings

Fault rupture model



USGS ShakeMap



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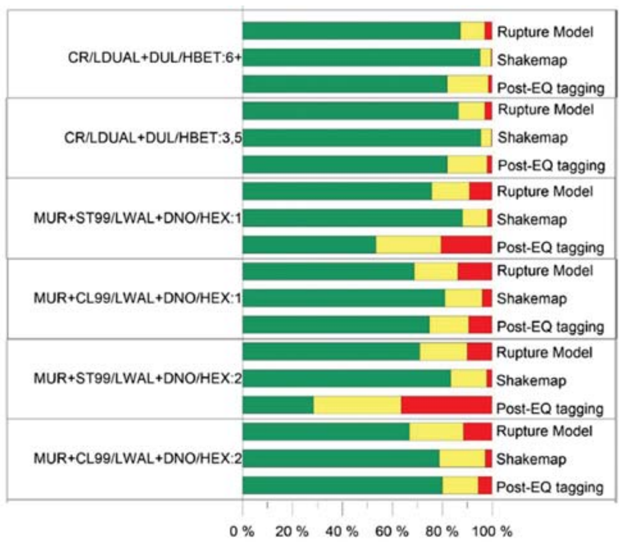
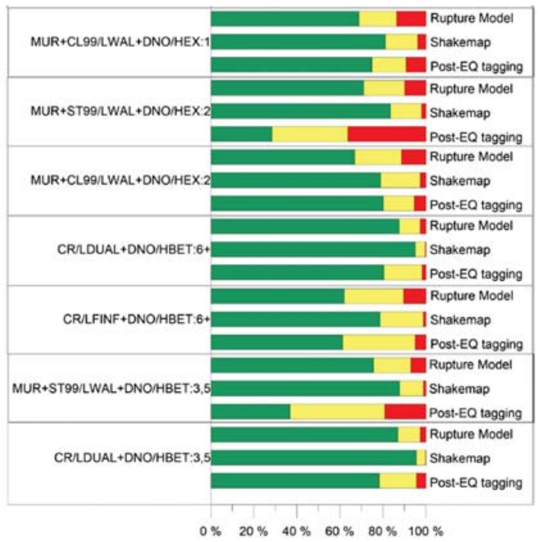
Estimated vs actual structural damages





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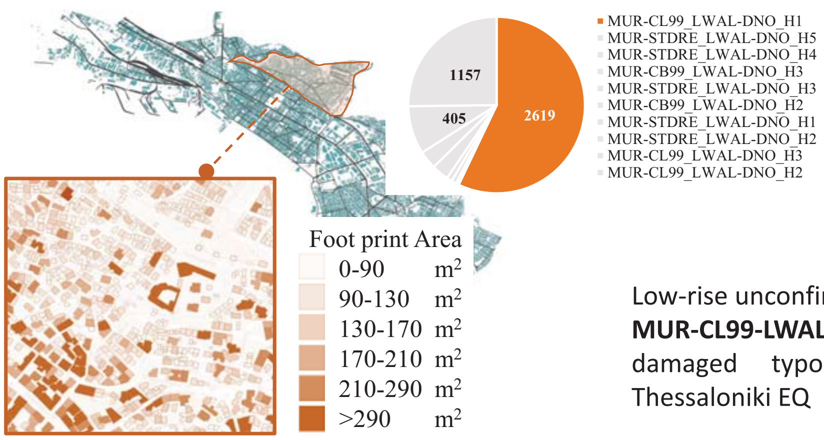
Validation: old (upper) city of Thessaloniki

- Application of the proposed methodology for seismic risk assessment of the historical center of Ano-Poli, northwest area of Thessaloniki.



Low-rise unconfined masonry structures **MUR-CL99-LWAL-DN0-H1**, the most heavily damaged typology during Mw 6.5, 1978 Thessaloniki EQ

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Validation: old (upper) city of Thessaloniki

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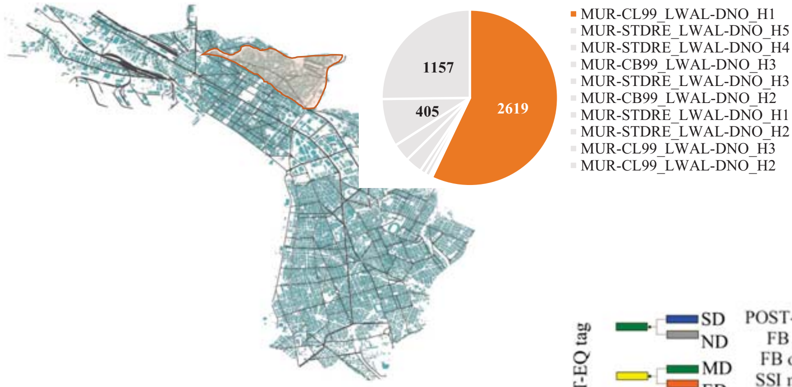
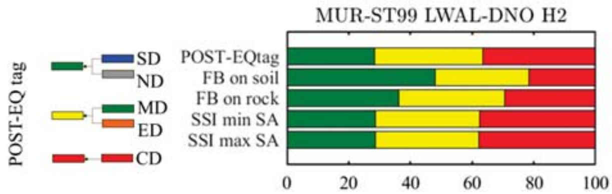
- The comparison with the observed damages is good when accounting for site amplification and SSI effects





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Application to Santorini

SSM approach only, obviously

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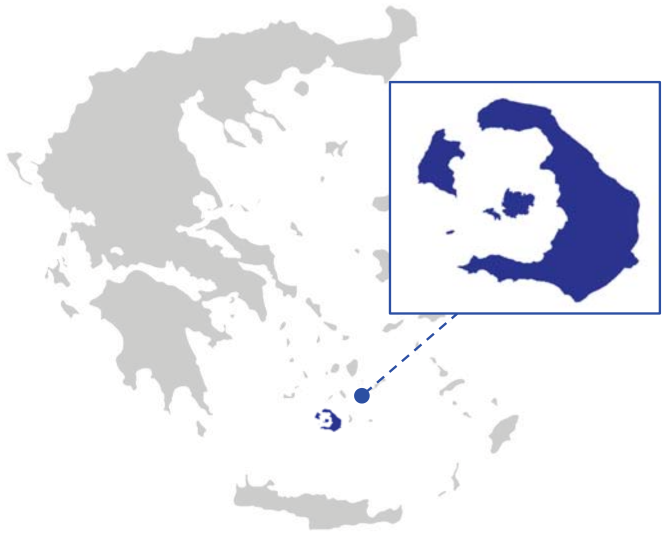
Motivations



the Santorini-Amorgos zone, located within the central Hellenic volcanic arc, is highly seismically active.

- The Mw 7.1 earthquake of 1956 caused important damages and the collapse of more than 500 structures and the generation of a tsunami of 25m estimated local run-up.

- Seismic activity of January-February 2025 with more than 2,000 recorded events in about two months period, including 14 events with M≥5.0

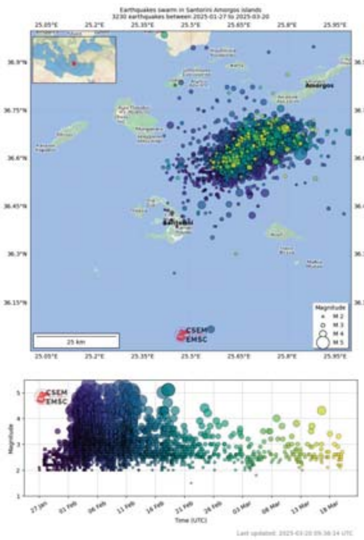
The 1956 Mw 7.5 historical EQ

Ruins of houses collapsed because of the 1956 earthquake

- 53 people were killed
- >100 people were injured
- 35 % of the houses collapsed
  45 % suffered major or minor damage.

[Gerassimos, EQS, 2023]

The seismic activity of January-February 2025



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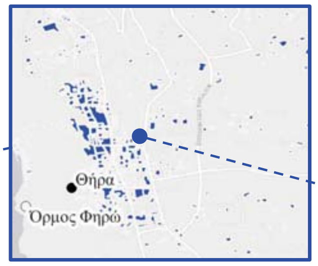
Santorini study area: typical architecture



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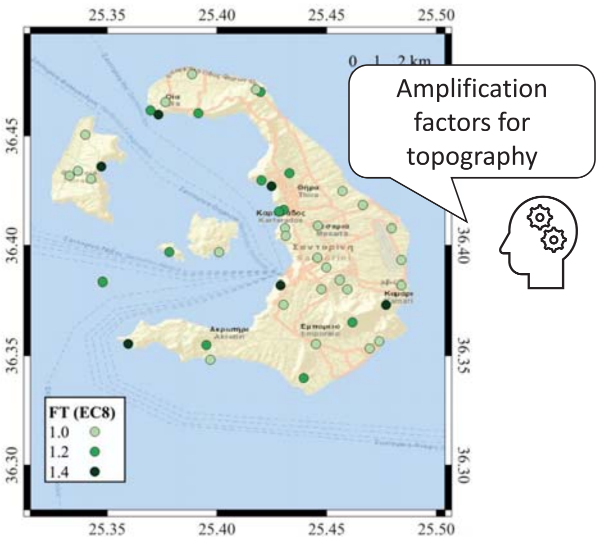
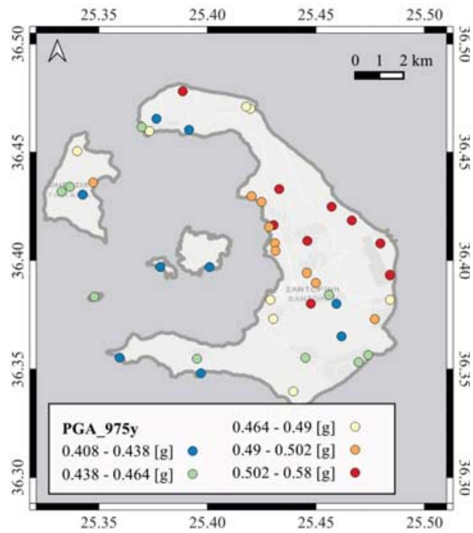
Simulation of seismic scenario: 975y Tr





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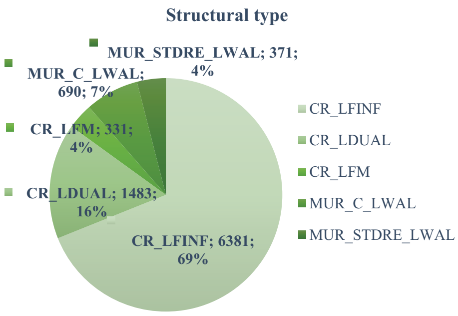
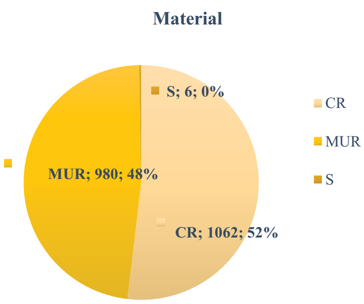
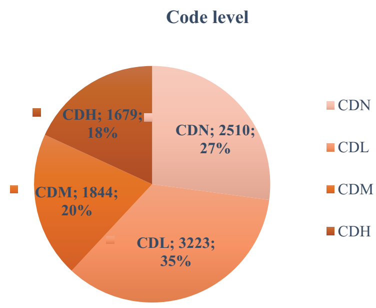
Santorini Exposure model

- After the disastrous earthquake of 1956, reinforced concrete has become the prevailing construction material used for reparation works and new constructions.
- (CDN=before the 7.7 earthquake)







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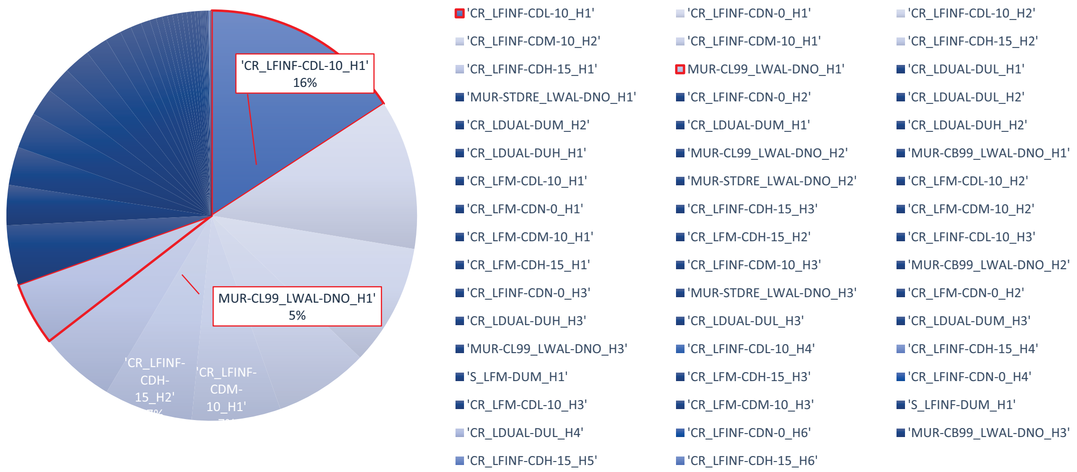
The most popular building classes



The most popular building classes

CR

Reinforced concrete (CR) buildings with infilled frame lateral resisting system (LFINF)

MUR

Unreinforced masonry structures survived from the 1956 earthquake (CDN)

[*Kazantzidou-Firtinidou et al. 2018*]

Fragilities of the most popular building classes

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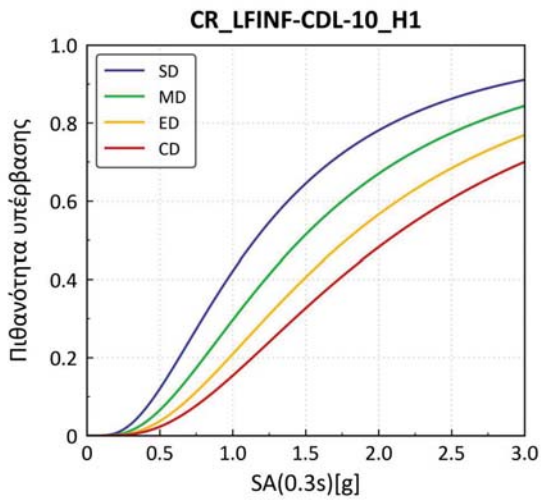
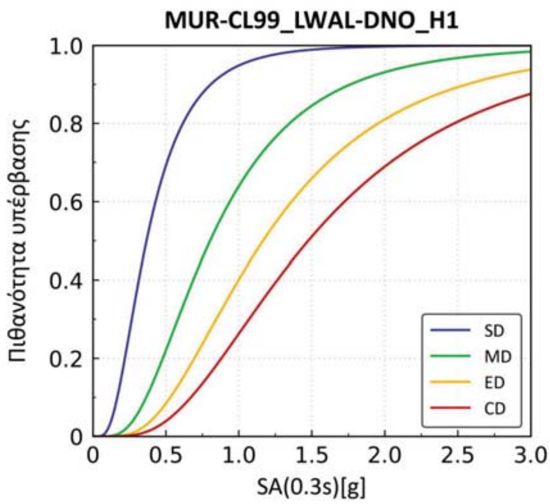
The significant vulnerability of masonry old structures dominates the overall vulnerability of Santorini





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Damage scenario for the 975y Tr

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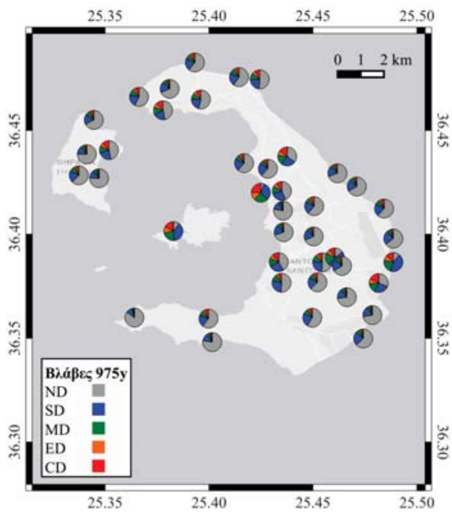

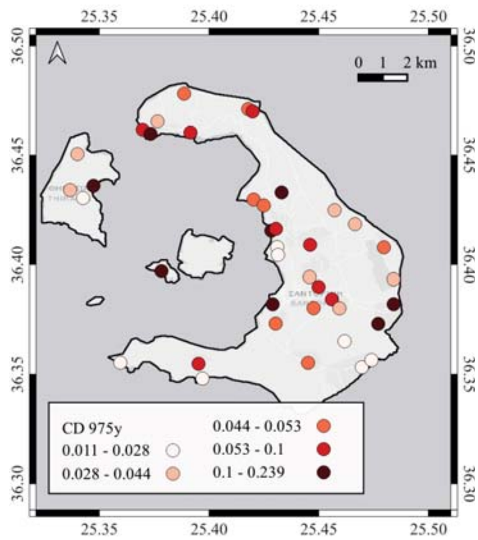

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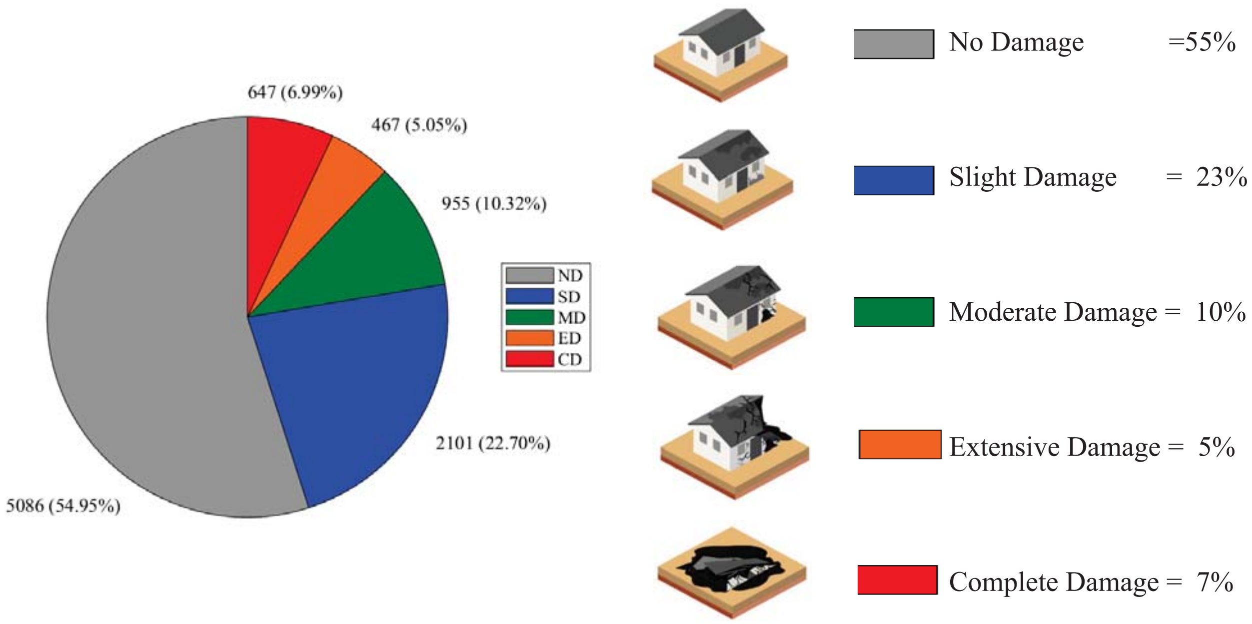
Aggregated for the whole Island



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How do we improve this approach?

SSM approach, can we do better?

Improvement to structural modeling: MDOF

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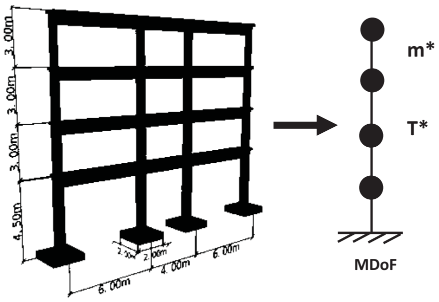
Multi-story concentrated-mass shear model for a building

- More demanding but still computational efficient
- Displacements and accelerations at floor level are directly estimated
- Contribution of higher modes is implicitly modelled



[Lu et al., 2014]

Structural modeling: MDOF

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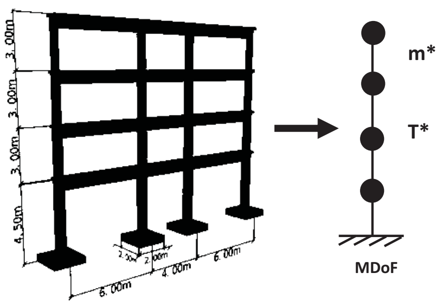
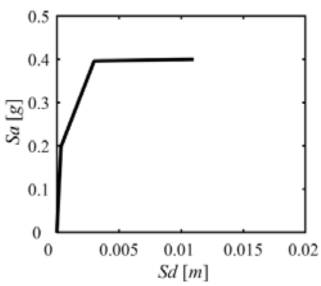
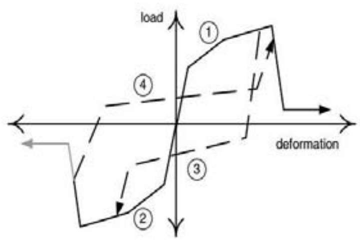
Multi-story concentrated-mass shear model for a building

- “two-node link" element
- Literature capacity curves from pushover analyses of 2D models
- Proper hysteresis model



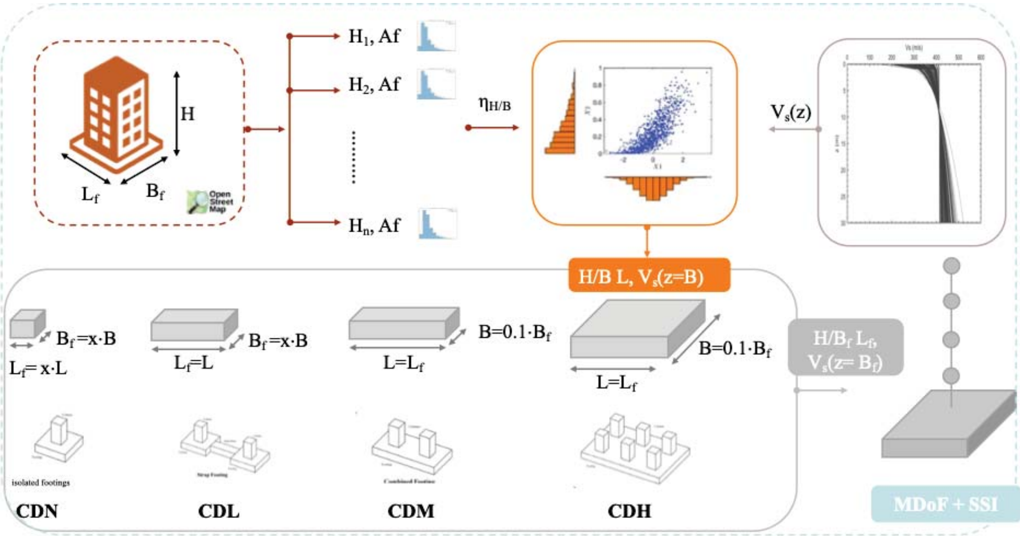




OpenSees

[Lu et al., 2014]

MDoF + SSI system: the framework



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Case study Attica: motivations

The **1999 Athens earthquake** occurred on September 7 caused:

- 143 dead, 800–1,600 injured
- $3–4.2 billion losses

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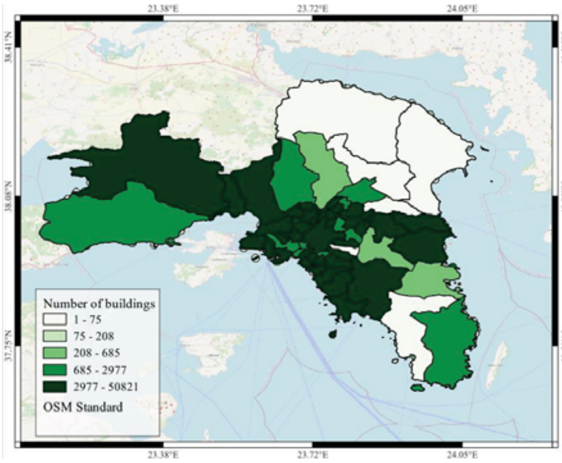
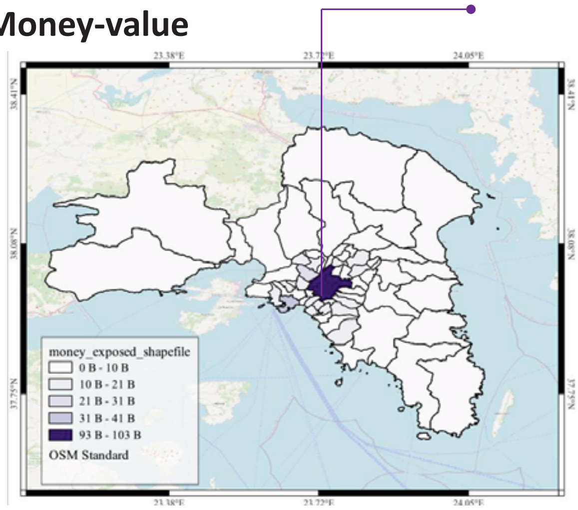
Exposed assets

Buildings

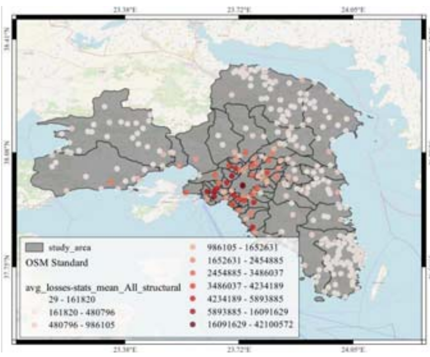


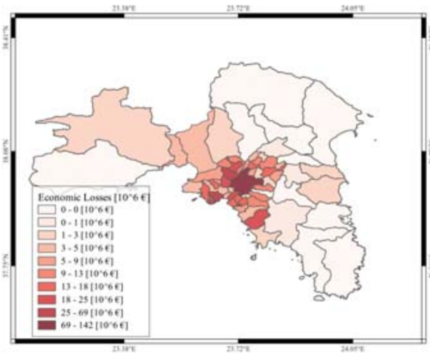
Money-value



Annual economic losses

- Aggregated at different levels: settlements->region
- Highest economy losses concentrated in Athens





Conclusions

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Dimitris Pitilakis

What to take home from this presentation?

- We should include, whenever possible, site amplification and SSI effects in the risk assessment of existing buildings

- Site amplification increases the fragility (in general)
- SSI increases the fragility for stiff structures

- Detailed structural modeling is more accurate (of course) but more expensive (of course)
- Simplified structural modeling is less accurate (of course) but less expensive (of course)

- (Even) more precise/accurate urban-scale seismic risk assessment is possible/promising

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Thanks to

- Chiara Amendola, PhD, Post-Doc
- Christos Petridis, PhD
- Evi Riga, PhD++, Faculty
- Stefania Apostolaki, PhD!
- SDGEE

Thank you

www.dpitilakis.work

Loretta Batali

Professor of Geotechnical Engineering, President of the Romanian Society for Soil Mechanics and Foundation Engineering, Vice Chair of CEN TC 250/SC7

Forensic geotechnical, hydrological and hydrogeological analysis of instability phenomena occurred at a waste management centre

Loretta Batali is full professor and habilitated for PhD research at the Technical University of Civil Engineering Bucharest (UTC), Department of Geotechnics and Foundations and Director of the Council for Doctoral Studies. She graduated the Hydraulic Works Faculty of UTCB in 1990, then she obtained a Master degree in 1993 and her PhD degree in 1997, both from INSA Lyon France (with a PhD thesis on the Use of geosynthetic clay liners for landfills). Topics of interest: Soil mechanics, Foundation engineering landfills, Geosynthetics, Retaining structures, Unsaturated soils, Slope stability. Loretta Batali led 4 research projects as director (2 international and 2 national) and was member of another 7 international and 14 national research projects. She published several speciality books and numerous scientific and technical papers in journals and conference proceedings. Loretta Batali also has a rich technical activity for geotechnical investigations, geotechnical design and consultancy, verification and expertise, as well as author of technical norms and standards and member of various state commissions. She was involved in the revision of the Eurocode 7 at CEN as member of PT1 and then leading TG B on design examples and from 01.01.2025 vice chair of SC7. Since 2021 Loretta Batali is the President of the Romanian Society for Soil mechanics and Foundation Engineering (SRGF), after being vice president of it for 9 years. She is also member of the International Society for Soil Mechanics and Foundation Engineering (ISSMGE) and chair of the Awards Board Level Committee (AWAC).

2nd Greek – Romanian Seminar

Lessons learned from Earthquakes and Geotechnical Failures

Forensic Geotechnical, Hydrological and Hydrogeological Analysis of Instability Phenomena Occurred at a Waste Management Centre

Prof. Loretta Batali

UTCB

Assoc. Prof. Dragos GAITANARU, Dr. Traian GHIBUS

UTCB

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

UTCB Technical University of Civil Engineering Bucharest

Societatea Română de Geotehnică şi Fundaţii

Thessaloniki, 9th October 2025

INTRODUCTION

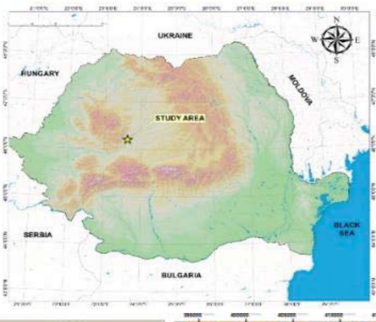
- *Waste Management Centre (WMC) - municipal landfill, a sorting and a mechanical – biological treatment stations and administrative area*
- *Located in a hilly region*
- ***Local instability phenomena** occurred in the man-made slopes, which have been locally remediated first by geometrical methods (berms, then slopes inclination has been reduced).*
- *The cause has been identified at that time as being the **"unpredictable" groundwater presence**, as the geotechnical investigation and the subsequent geotechnical design didn't take into consideration the groundwater.*
- *Several such instability phenomena occurred during the time, with **several remediation methods**, from the simplest ones (geometrical) to more complex ones including retaining walls supported by micro-piles and ground anchors, drainage works, monitoring etc.*

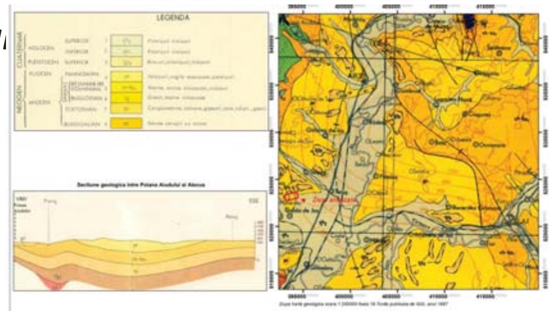
INTRODUCTION

- *The case ended up in court of law, which asked for a forensic judicial analysis and established several objectives of this analysis:*
- *origin of water infiltrations in the area,*
- *analysis of the stability of the ground, and*
- *to establish if the phenomena were unpredictable.*
- *The **existing investigations**, especially regarding groundwater, were insufficient and also the **geotechnical design** was poor.*
- *It has been required to conduct a **complex geotechnical, hydrological, and hydrogeological investigation and numerical modelling, including also unsaturated measurements. This has been completed with slope stability analyses.***

GEOLOGICAL, HYDROGEOLOGICAL, AND HYDROLOGICAL SETTINGS

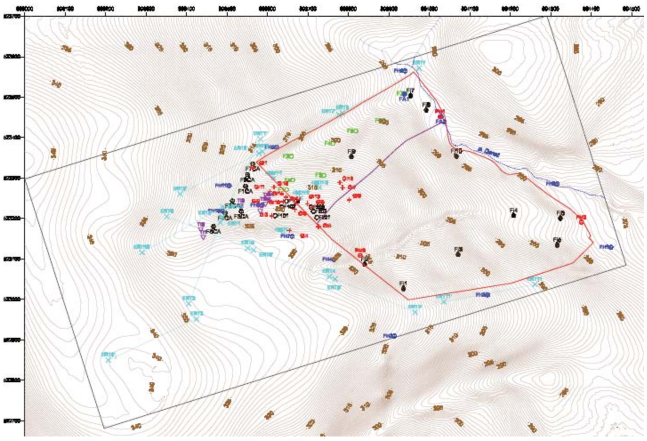
- *SE area of the Transylvanian Depression (central part of Romania).*
- *Deposits belonging to the Neozoic era (Paleogene, Neogene, and Quaternary) which rest on a foundation of crystalline schists.*
- *Representative: formations belonging to the Upper Neogene (Volhinian-Bessarabian and Pannonian) and the Quaternary (Pleistocene and Holocene)*
- *Bedrock: Pannonian marls, with a general inclination from SW to N weakly stratified, with or without sub-millimeter sand films.*
- *Quaternary deposits: deluvium, proluvium and alluvium deposits*





GEOLOGICAL, HYDROGEOLOGICAL, AND HYDROLOGICAL SETTINGS

- *Hydrographic basin of the Mureș River / Dăneț stream - 4.61 km2.*
- *Altitude: 270 - 430 meters above sea level. The maximum slope value within the basin is 190*
- *A proper aquifer consisting of porous permeable non-cohesive material has not been identified in the study area.*
- *The only non-cohesive lithologic type identified is represented by the gravelly sand (outside the limit of the waste deposit), which has a lenticular shape, with a narrow width estimated at 150-170 m, surrounded by sand and gravel in a clayey mass and limited in roof and bedding by clayey rocks - reservoir rock, but without a significant contribution to the groundwater dynamics reported for the entire studied perimeter.*



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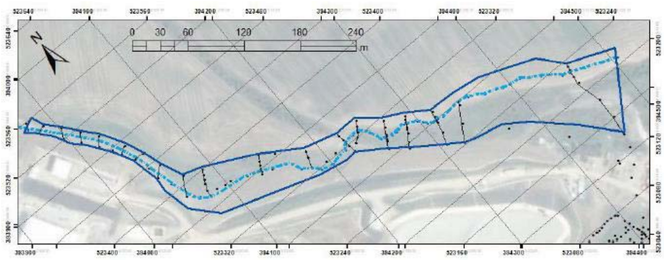
IN SITU INVESTIGATIONS

- *geological and hydrogeological drilling and piezometric wells,*
- *geological and morphological mapping,*
- *infiltration and hydraulic testing,*
- *topographical measurements,*
- *geophysical investigations,*
- *flow and river measurements.*
- *4 terrain investigation campaigns.*
- *hydrological mapping*
- *geomorphological mapping*
- *identification geomorphological processes (1) cracks and drying fissures , (2) detachment surfaces, (3) gully erosion and ravine processes.*



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IN SITU INVESTIGATIONS

- *geological and hydrogeological drilling and piezometric wells,*
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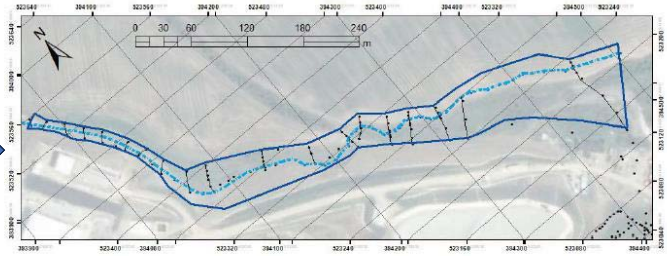

IN SITU INVESTIGATIONS

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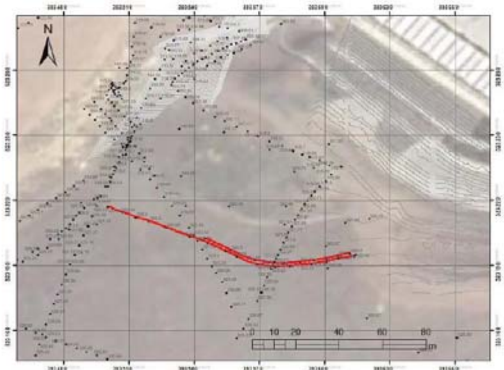
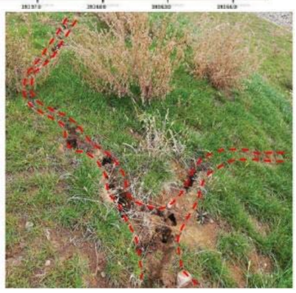
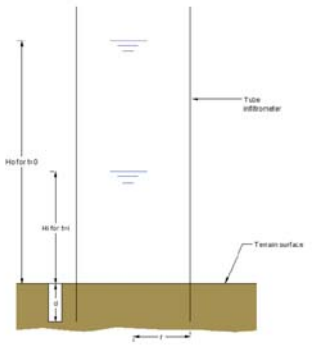
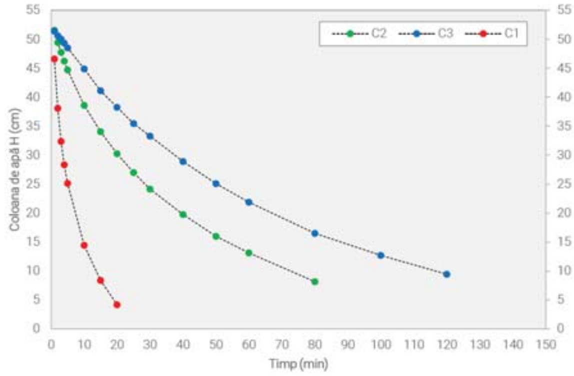




IN SITU INVESTIGATIONS

- *Tests and measurements were carried out in boreholes and in the field.*
- ***Hydrogeological tests** were conducted using the recovery method (bail test).*
- ***Hydraulic tests** consisted of evaluating infiltration coefficients in the unsaturated zone at various locations upstream of the WMC area. The tests were performed using the Single Ring Infiltrometer Method. During the field campaigns, a total of 13 tests were carried out.*





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IN SITU INVESTIGATIONS

- *Within the site, a total of **11 electrical resistivity tomography** (ERT) investigation profiles were conducted. The total profile length was of approximately 2500 m*

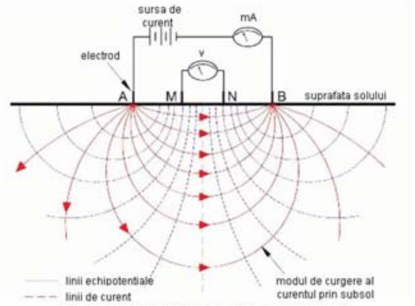
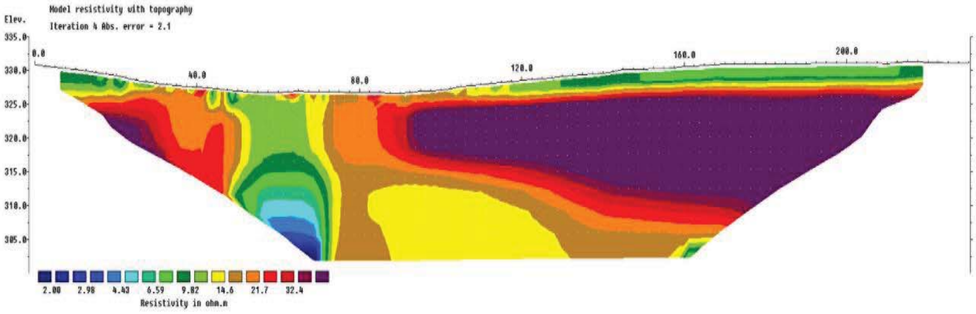


Fig. 3-1 Dispozitiv ABMN şi distribuţia curentului în sol



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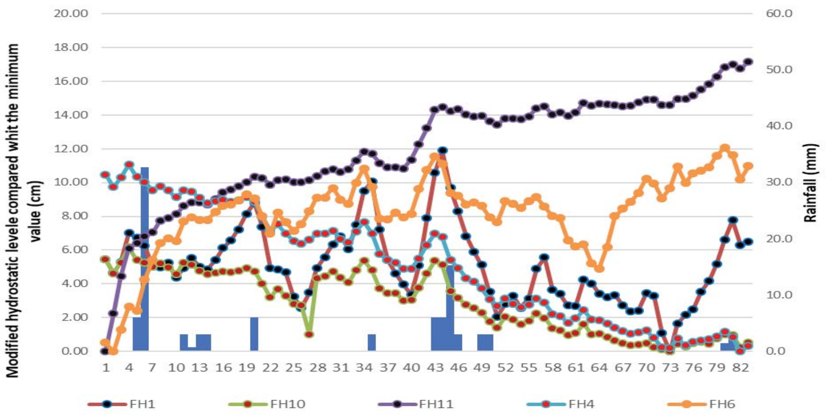
HYDROGEOLOGICAL MONITORING

- *5 hydrogeological wells were monitored during a period of 90 days (autumn – winter).*
- *The groundwater level and temperature were monitored using automated pressure and temperature data loggers*



HYDROGEOLOGICAL ANALYSIS

***Models**:*

- ***Morphological and topographical model** (digital terrain model - DTM) based on direct differential GPS measurements and morphological mapping (visible landslides, cracks, etc.).*
- ***Geological 3D model** - based on DTM, lithological data and geophysical data.*
- ***Hydrological model** (rainfall – runoff) - based on the DTM, the hydrological flow measurements, land cover classes and climatological data - using the HEC-HMS software package.*
- ***Groundwater flow model** – based on the hydraulic in-situ data (pumping hydraulic tests, groundwater head measurements, infiltration hydraulic tests).*
- *The **hydrogeological model** was developed in 2 stages:*

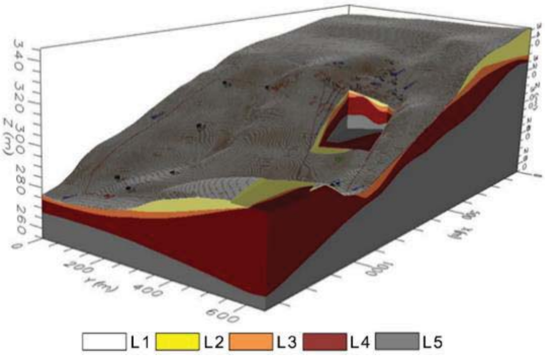
(1) a first general model developed on a certain time stamp (the period of in-situ investigation) in steady conditions and

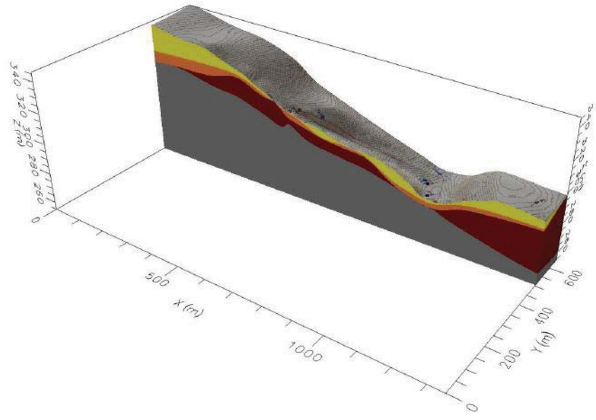
(2) an unsteady hydrogeological model to simulate the groundwater flow under different scenarios.

HYDROGEOLOGICAL ANALYSIS

Models:

- ***Geological 3D model***





L1-topsoil,
L2 - silty clay,
L3 - gravel and sand, with or without boulders in clay matrix,
L4 - silty marly clay
L5 - grey marl

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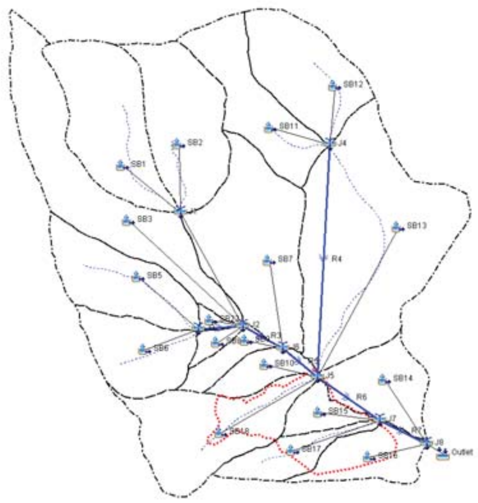
HYDROGEOLOGICAL ANALYSIS

Models:

- ***Hydrological model***



The model considered:
(a) The average slope of each sub-watershed;
(b) Land use and soil type;
(c) Surface runoff parameters (Soil Conservation Service method, Curve Number);
(d) Base flow;
(e) Precipitation.

The watershed was divided into 18 sub-watersheds of different sizes based on surface runoff accumulation, with areas ranging from 0.8 km² to 0.01 km².

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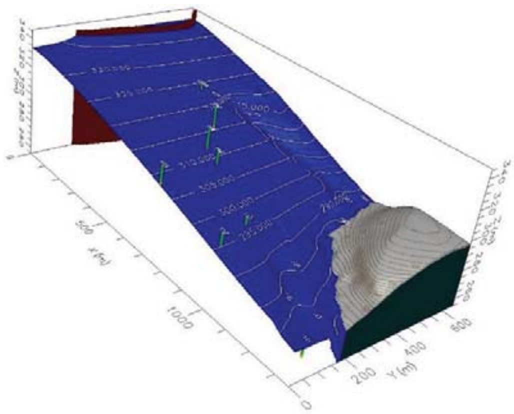
HYDROGEOLOGICAL ANALYSIS

Models:

- **Hydrogeological model**



3D representation of the groundwater table

- $k_x=k_y=k_z=k=0.02245$ m/day
- $n=0.55$
- $n_e=S_y=0.0605$
- due to the presence of contraction fissures on the ground surface, many with centimetre-sized openings and development in depth, the aquitard hydro-structure was considered with a free water level

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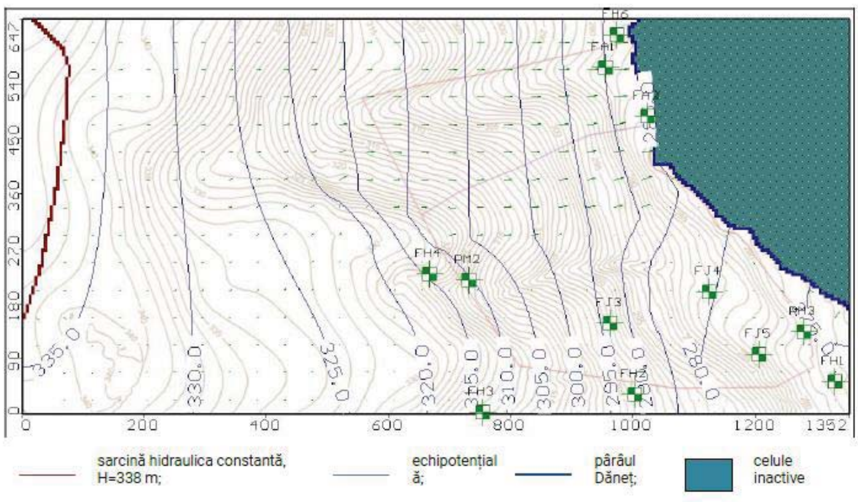
HYDROGEOLOGICAL ANALYSIS

Models:

- **Hydrogeological model**



Isolines of equal potential - Steady state

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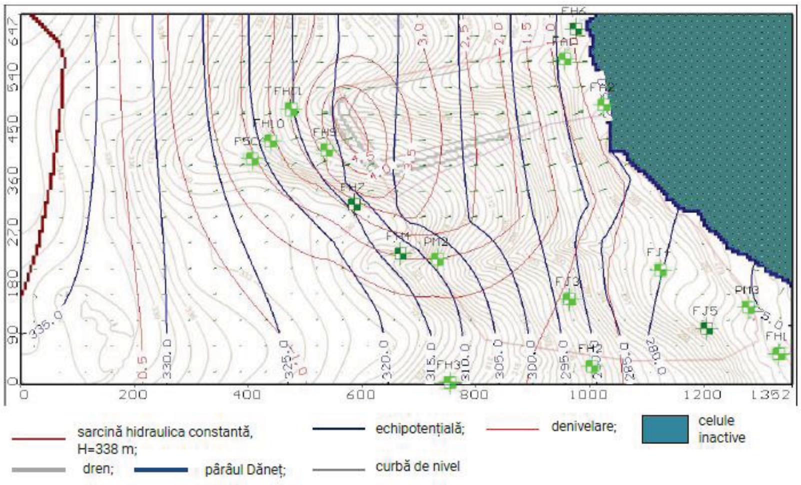
HYDROGEOLOGICAL ANALYSIS

Models:

- ***Hydrogeological model***

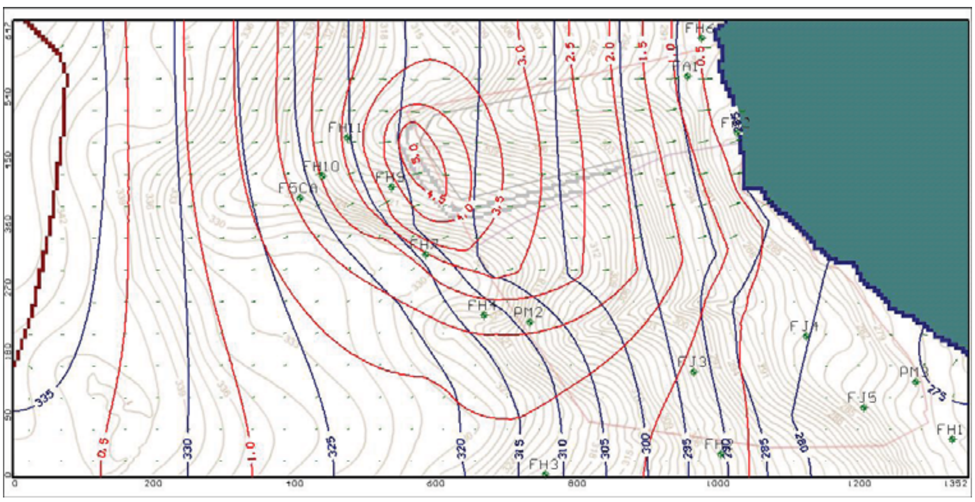


Isolines of equal potential - Steady state – with drainage

HYDROGEOLOGICAL ANALYSIS

Models:

- ***Hydrogeological model***



Isolines of equal potential - Unsteady state – after 720 days

HYDROGEOLOGICAL ANALYSIS

Conclusions:

- ✓ *The hydrological model confirmed the field measurements.*
- ✓ *The average annual flow rate of the receiving WMC basin ranges between 0.5-1 l/s.*
- ✓ *During dry periods, the flow rate drops below 0.5 l/s.*
- ✓ ***Variability of flow rates** is observed, ranging from values below 0.1 l/s during dry periods to 1.8 m^3/s during rainy periods.*
- ✓ *Except for the upper horizon of yellow clays, **groundwater was identified in all lithological types of clay or marl nature at the bottom**, which is why they were included in a single hydro structure when defining the hydrogeological model.*

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HYDROGEOLOGICAL ANALYSIS

Conclusions:

- ✓ *The **hydro structure in the analysed area is an aquitard type with slow dynamic**, leading to the wrong conclusion regarding the existence of groundwater.*
- ✓ *From the hydrogeological model in steady state, before the landfill was built (initial situation), it results that the **recharge of the hydro structure in the area is mainly through groundwater flow from upstream to downstream** (88.34 m^3/day).*
- ✓ *The **vertical recharge from rainwater is low** (3.99 m^3/day) due to the high slope of the ground. However, it is possible that in the case of high intensity rainfall events occurring after a long dry period, this recharge rate may increase through **water infiltration into the contraction cracks** until these are close.*

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GEOTECHNICAL ANALYSIS

The forensic analysis showed that the occurred landslides have been produced due to a ***cumulative effect of natural and anthropogenic aspects****:*

- ✓ ***Site lithology****: swelling clays, laying over the marly bedrock;*
- ✓ ***Groundwater presence****: this is influencing both by increasing the water pressure in the fissures and by reducing shear strength of the clays. The water infiltrated in the soil is drained very slowly due to low hydraulic conductivity and to slow-dynamics hydro-structure.*
- ✓ ***Excavations****: the earthworks performed on site created a drainage path for the water accumulated in the ground, which outflowed and ran off and through the slope;*
- ✓ ***Swelling clays****: these led to contraction fissures in the ground, through which rainwater could infiltrate;*
- ✓ ***Slope inclination****: designed slopes were too steep with regard to above mentioned soil and water conditions.*

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GEOTECHNICAL ANALYSIS

Numerous site investigations have been conducted previously:

- ✓ ***6 geotechnical studies*** *(3 before and 3 after the landslides occurred),*
- ✓ ***1 hydrogeological investigation*** *(but for water supply purpose) before the one during the forensic analysis.*
- ✓ *None of the 3 geotechnical studies emphasized the stability problems of the site, the groundwater infiltrations, the presence of swelling clays and the gullies on the site*
- ✓ ***No stability analyses*** *have been conducted prior to instability phenomena.*
- ✓ *The* ***slopes inclination*** *didn't take into consideration the specific behaviour of swelling clays which imposes very low inclination of the slopes.*
- ✓ *The national regulation in force with this respect recommends slope inclinations of 1: 3 – 1:4, while the slopes on site were inclined 1:1.5 for 26 m maximum height, with no berms;*

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GEOTECHNICAL ANALYSIS

Numerous site investigations have been conducted previously:

- ✓ *An additional geotechnical study, conducted after the first local instabilities occurred, included **stability analyses, but using a horizontal lithology and no groundwater**.*
- ✓ *Given the clayey nature of the ground, the good practice and the national design guidance according to Eurocode 7 impose to consider a **saturation hypothesis**. Also, **the swelling clays presence requires to check the stability with reduced shear strength parameters due to possible contact with water.***
- ✓ *The national technical norm on geotechnical investigation in force at that date had some provisions with regard to hydrogeological data, being requested to provide **groundwater level and type of aquifer, the possible excess pore water pressure and hydrogeological cross sections**. But, it also refers to EN 1997-2, where detailed provisions are included in 2.1.4 and 3.6. that cover all possible general situations.*

GEOTECHNICAL ANALYSIS

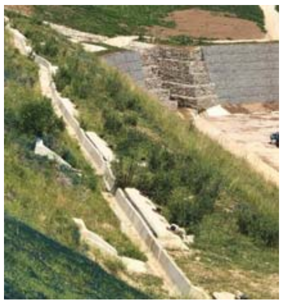
***Consolidation measures**, based on reports of several experts and all these seems to be a kind of "trial-and-error" approach.*

***Stage 1:** less steep slope (1:2.5) including also berms (2 berms on the slope of max. 26 m height), drainage works in one corner of the site where water sources were identified, collection of runoff water on each berm, surface protection against erosion;*

***Stage 2:** sub-horizontal drainages and drainage trenches; discontinue, reinforced concrete, retaining walls founded on micropiles and anchored, with a "saw teeth" disposal (in one section of the wall on one of the berm, the following section on the other berm at higher or lower level); monitoring;*

***Stage 3:** new interventions on the retaining walls as new instability phenomena occurred, new drainage works.*



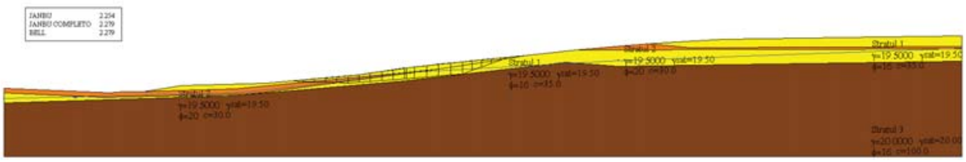

GEOTECHNICAL ANALYSIS

New Stability analyses

- *Initial slope (reconstituted) - LEM*

Case no.	Description	Overall safety coefficient using characteristic values		ODF (over-design factor = F_s/ $F_{s\ allow}$)	
		static	seismic	Static $F_{s\ allow}$ = 1.5	Seismic $F_{s\ allow}$ = 1.1
Natural slope					
1	Circular slip, GWL as measured	3.77	2.05	2.50	1.86
2	Circular slip, GWL at GL	2.07	1.66	1.38	1.51
3	Polygonal slip at contact with the marl, GWL as measured	4.38	2.25	2.92	2.04
4	Polygonal slip at contact with the marl, GWL at GL	3.49	1.79	2.33	1.63
5	Circular slip, no GWL, shear strength parameters reduced at 30%	1.28	0.70	0.96	0.63

- ✓ The initial natural slope was stable in almost all situations, except the one with reduced shear parameters.
- ✓ In case of a natural slope this hypothesis is not so relevant, as its surface is not exposed as in the case of a cut slope, being protected by vegetation.
- ✓ Causes are related to the works and the errors committed at the design stage.



GEOTECHNICAL ANALYSIS

Stability analyses

- *Man-made slopes (initial inclination 1:1.5) - LEM*

Case no.	Description	Overall safety coefficient using characteristic values		Analysis according to EN 1997 (over-design factor) Approach 3 (ODF = F_s/ $F_{s\ prescribed}$)	
		static	seismic	Static $F_{s\ prescribed}$ = 1.35	Seismic $F_{s\ prescribed}$ = 1
Slopes 1:1.5 without berms (initial design – Figure 10)					
1	Circular slip, no GWL (as designed)	2.33	1.89	1.33	1.43
2	Circular slip, GWL at GL	1.86	1.48	1.07	1.13
3	Circular slip, no GWL, shear strength parameters reduced at 30%	0.69	0.56	0.40	0.43
Slopes 1:1.5, without berms – another profile not considered in the design					
4	Circular slip, GWL as measured	1.52	1.23	0.87	0.95
5	Circular slip, GWL at GL	1.17	0.95	0.68	0.75
6	Circular slip, no GWL, with reduced shear strength parameters at 30%	0.54	0.44	No further reduction of parameters	No further reduction of parameters
Remodelled slope – (Figure 11)					
7	Circular slip, GWL as measured	2.41	1.77	1.38	1.37
8	Circular slip, GWL at GL	2.10	1.54	1.21	1.20
9	Circular slip, no GWL, reduced shear strength parameters at 30%	0.97	0.70	0.55	0.54

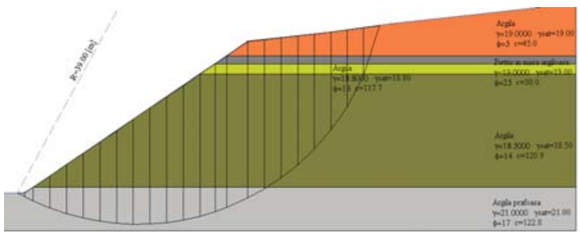


Fig. 10 – Initial design

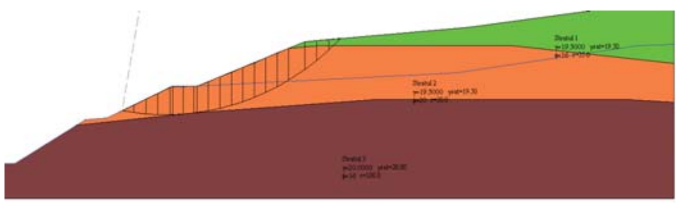


Fig. 11 – Remodelled slope

GEOTECHNICAL ANALYSIS

Stability analyses - conclusions

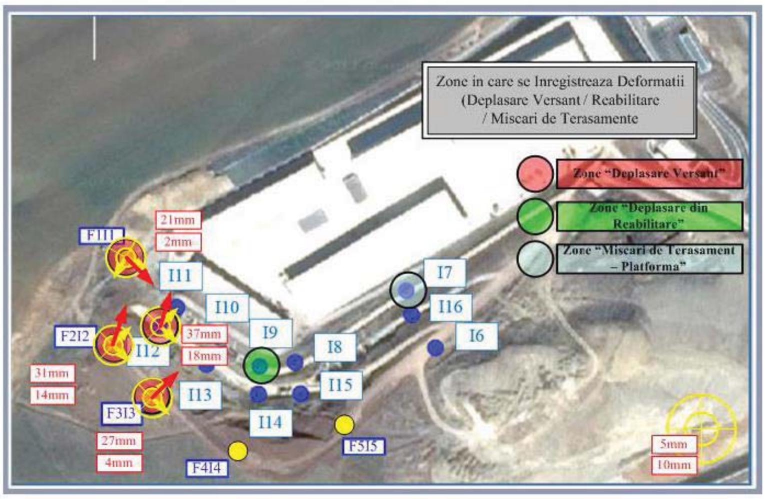
- ***groundwater*** *was the primary cause of the instability phenomena and the design should have consider it*
- ***basic design errors****, as slopes were designed too steep and didn't consider the presence of swelling clays and that water can infiltrate through the contraction fissures, reducing drastically the shear strength parameters.*
- ***remodelling interventions*** *of the slope (berms and partially less steep slopes) -* ***not sufficient***
- ***More interventions****: 39 sub-horizontal drains for collecting groundwater infiltrations, 46 segments of reinforced concrete retaining walls each supported by 5 – 6 micro-piles and 3 – 4 pre-tensioned ground anchors, disposed in saw teeths on the berms (at the lower and middle section of the slope).*
- *Despite these extensive consolidation measures the* ***swelling characteristi****c of the soil has still* ***not been considered*** *and* ***no detailed stability analyses*** *performed.*
- *In some sections the consolidation measures are not effective, as not being enough deep or not located properly (not intercepting the slip surfaces)*

GEOTECHNICAL ANALYSIS

Stability analyses - conclusions

- ***Monitoring***

- *At the date of the analysis (2019)- 2 yrs after the interventions - displacements still occurring*
- *At the date of the site visit - point F313:*



UNPREDICTABILITY ANALYSIS

Lithology: *upper clayey layer over a marl bedrock, prone to slippage when water penetrates.*

Groundwater: *excess of porewater pressure in the fissures, reduction of shear strength parameters of the clayey soil, slow drainage due to geotechnical and hydrogeological features of the site. Slow hydro-structure (aquitard).*

Excavation works *on a natural slope: leading to a drainage path for the water accumulated in the ground, which ran off through and on the cut slope.*

Swelling clays *presence: leading to drying fissures through which rainwater infiltrated easily.*

Investigation: *Unproper hydrogeological investigation, no long-term monitoring. The aquitard is difficult to be identified.*

Design: *too steep slopes for the site conditions, unproper stability analyses, non-respect of national legislation and of good practice, no measures regarding groundwater*

Legislation: *with regard to the hydrogeological investigation, the national legislation is not very clear, sometimes separating the geotechnical investigation from the hydrogeological one.*

Conclusion: combination of natural and man-made causes, but this could be avoided by proper investigation and design, therefore it cannot be considered as unpredictable.

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CONCLUSIONS

- ✓ ***complex forensic analysis*** *of instability phenomena*
- ✓ *despite the first impression of simplicity (cut slopes in clay soil), showed **a hydrogeological specificity** not so easily to be correctly identified (aquitard with a slow dynamics), a **geotechnical specificity** (swelling clays), **improper investigation and design**, non-respect of good practice and legislation,*
- ✓ *but also, a **lack of clarity in the technical norms** which has to be corrected (the most recent revision of the national technical norm for geotechnical investigation relied on these findings).*
- ✓ ***complex investigation, analysis and numerical modelling**.*
- ✓ *importance of a **good cooperation between geotechnical engineers, geologists, hydrogeologists etc**., the need for **long term monitoring**, rarely carried out due to time and financial constraints.*
- ✓ *large discrepancy between the price of a proper hydrogeological investigation and monitoring and the one for extensive consolidation measures that had to be taken.*
- ✓ *the analysis showed that these are not sufficient for stopping the phenomenon, therefore more measures have to be foreseen.*

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ
Forensic Geotechnical, Hydrological and Hydrogeological Analysis of Instability Phenomena Occurred at a Waste Management Centre, L. Batali, D. Gaitanaru, T. Ghibus
Societatea Română de Geotehnică și Fundații

Dan Iancu

Structural engineer, Managing Partner, DI&A Design, Consulting Bucharest, Romania

Influence of pile stiffness on behaviour of slender base isolated structures

Dan Iancu is a structural engineer with over three decades of professional experience, having practiced since 1993. He is appointed as Verifier and Expert by the Romanian Ministry of Public Works, a designation that confirms his qualifications in structural engineering oversight and technical evaluation. Since 2012, Dan Iancu has served as managing partner at DI&A Design, Consulting, a consulting firm based in the Bucharest area. The company provides structural engineering design and consulting services for projects throughout Romania. His professional focus centers on seismic design methodologies, with particular emphasis on seismic base isolation systems. This field involves implementing technologies designed to reduce earthquake forces transmitted to building structures by decoupling them from ground motion. Soil-structure interaction has played a key role in the seismic design approaches he applies. Throughout his career, Dan Iancu has worked on earthquake-resistant structural designs in Romania, where seismic activity is significant, particularly in the Vrancea region. His work focuses on pioneering the introduction of base isolation systems and added damping in Romanian structural designs, applying international standards and engineering solutions to enhance building performance during seismic events. He is active in several professional organizations: AICPS (Romanian Association of Structural Design Engineers), where he currently serves as Administrative Director; SRGF (Romanian Geotechnical Society), where he is an Executive Committee member; ASSISI (Anti-Seismic Systems International Society, Inc.); and C.T.S. A (National Committee for Mechanical Resistance and Stability). As a practicing engineer and recognized expert, Dan Iancu continues to contribute to structural engineering standards development and seismic protection technology implementation in Romania's construction industry.

Influence of pile stiffness on behavior of slender base isolated structures

Dan Iancu

DI&A Design, Consulting srl

Professor dr. Horațiu Popa

U.T.C.B.

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

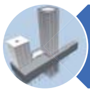
UT CB Universitatea Tehnică de Construcții București

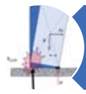
Societatea Română de Geotehnică şi Fundaţii

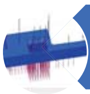
Thessaloniki, 9th October 2025

Summary:


Short introduction 04.03.1977 Vrancea earthquake


Structure description


Uplift and pounding - The challengers


Geotechnical findings


Complex condition –computing model


Conclusions

Short introduction to the consequences of 04.03.1977 Vrancea earthquake

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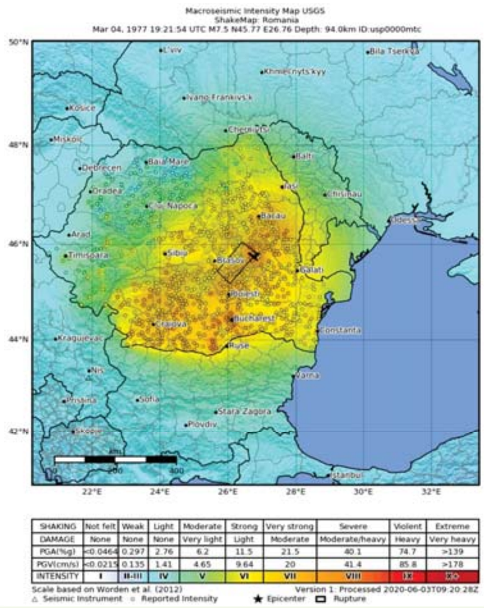
Short introduction to the consequences of 04.03.1977 Vrancea earthquake

- **1 578** fatalities
- **11 221** injured
- Moldavia & Bulgaria affected



https://en.wikipedia.org/wiki/1977_Vrancea_earthquake

Short introduction to the consequences of 04.03.1977 Vrancea earthquake

- Direct losses over $2 billion USD, with Bucharest alone accounting for approximately 70% of the total damage.
- The earthquake destroyed more than 32,000 buildings among them 33 were high rise,
- Left roughly 35,000 families homeless,
- Damaged hundreds of industrial units,
- Affected social-cultural infrastructure, hospitals, and public buildings.
- Total reported damages reached over 7.25 billion Romanian Lei, with indirect losses pushing the overall economic impact as high as $4.5 billion USD—about 13% to 21% of Romania's GDP at the time.

THE ROMANIAN EARTHQUAKE OF MARCH 4, 1977 REVISITED: NEW INSIGHTS INTO ITS TERRITORIAL, ECONOMIC AND SOCIAL IMPACTS AND THEIR BEARING ON THE PREPAREDNESS FOR THE FUTURE, Emil-Sever GEORGESCU , Antonios POMONIS

2nd Greek – Romanian Seminar: Lessons learned from Earthquakes and Geotechnical Failures

Short introduction to the consequences of 04.03.1977 Vrancea earthquake

- Industries suffered direct damage and prolonged production interruptions,
- Agriculture, transport, health, and education also faced substantial indirect losses.
- The destruction led to an economic crisis starting in 1979 that persisted for more than a decade, contributing to infrastructure vulnerability and slowing national development.
- Recovery imposed significant strain on state resources, limiting funds available for repairs and upgrades, which subsequently affected future resilience against major earthquakes.

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ
Influence of pile stiffness on behavior of slender base isolated structures. Dan Iancu & prof. dr. Horațiu Popa
Societatea Română de Geotehnică și Fundații

Short introduction to the consequences of 04.03.1977 Vrancea earthquake

- Geotechnical failure?
- OD16 block of flats

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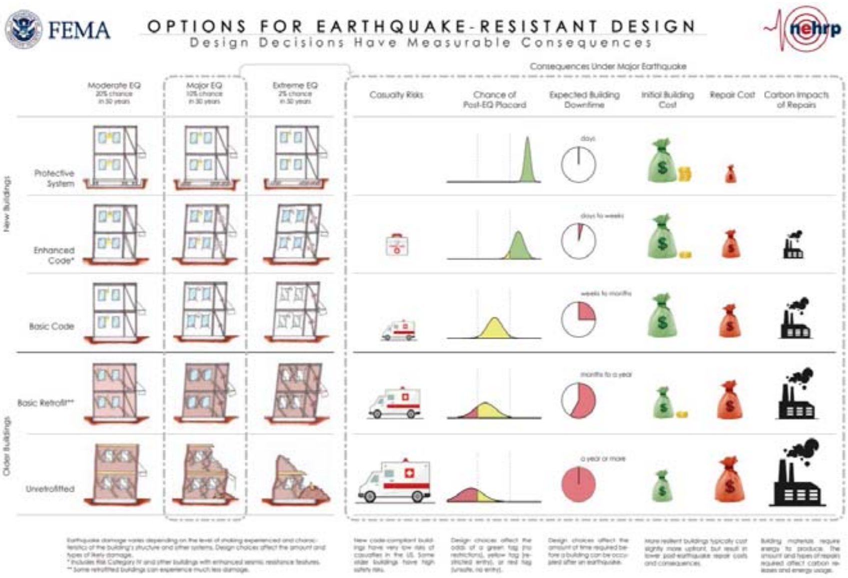
What would be the solution for immediate occupancy?

- Immediate occupancy can be reached:
  - Designing the structure and components elastically to the actual acceleration resulting from earthquake (designing with huge accelerations)
  - Seismic isolation and/or added damping



ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ
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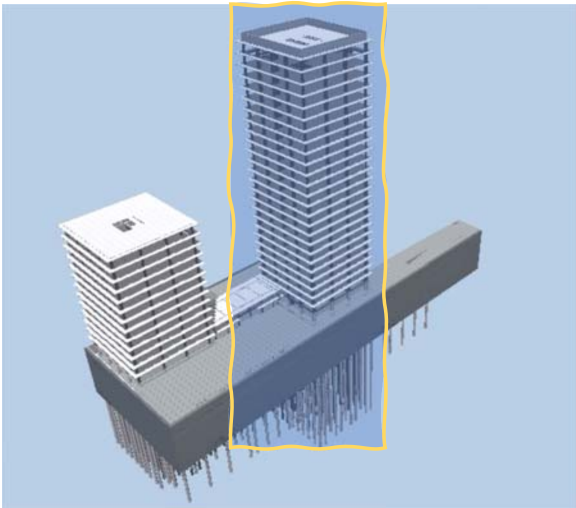
Structure description of the situation



3D rendering of the structure

The project consists of two high-rise multi-storey buildings and a low-rise building, with a common infrastructure with two basement levels.
Tower 1 is a reinforced concrete structure of 26 floors above ground, while tower 2 is a reinforced concrete structure of 12 floors above ground.
The GF+1E building is located between the two towers, being separated by the movement joints from the two towers.
The seismic isolation for both structures is in the infrastructure, above the basement, having joints between the fixed structure and the isolated structure at all levels where the movements must be accommodated.

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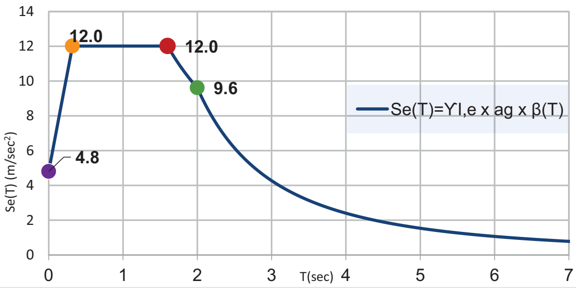
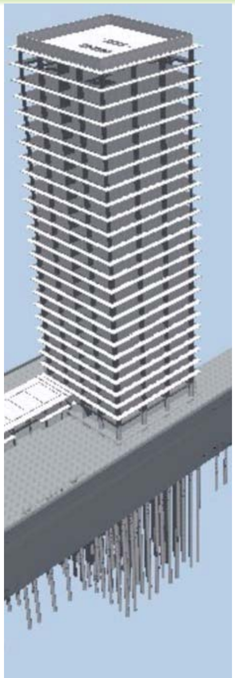
Structure - description of the situation

Main spectral parameters controlling the design for a recurrence period of 475 years, for Tower 1:

Control period (corner)	Tc	=**1.6 sec**
Maximum amplification factor of the ground horizontal acceleration by a SDOF system	β_0	=**2.5**
Peak ground acceleration value for design	ag	=**0.35g**
Importance-exposure factor	ϒe	=**1.4**
Behavior factor	q	=**1.5**
Ductility class		**L**



Elastic response spectrum of absolute accelerations for 5% damping, -Bucharest



ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ
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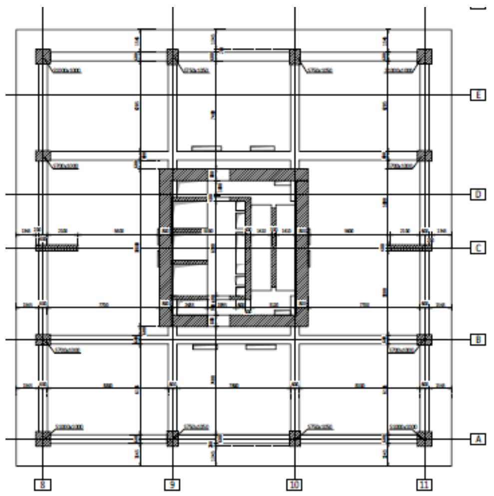


Current floor plan

Tower 1

The wall thicknesses are 25, 30, 40, and 80 cm at the base, and decrease progressively according to the effort level and deformation limitations.

The column sections were designed as follows:

•**All columns (from basement 2 to floor 1):** circular section with a diameter of 130 cm.

•**Corner columns:**

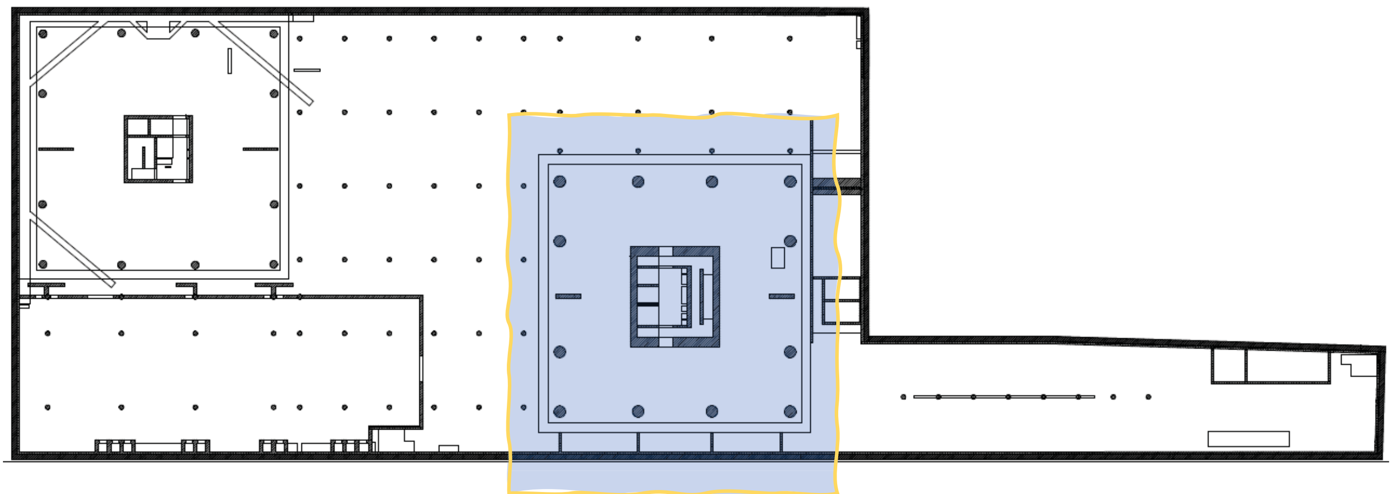
- from the 2nd to the 12th floor: rectangular section 100x100 cm;
- from the 13th to the 23rd floor: circular section with a diameter of 90 cm;
- on floors 24 and 25: circular section with a diameter of 85 cm.

•**Columns on the North and South facades:** from the 2nd floor to the top floor: rectangular section 75x105 cm.

•**Columns on the East and West facades:** from the 2nd floor to the top floor: rectangular section 100x70 cm.

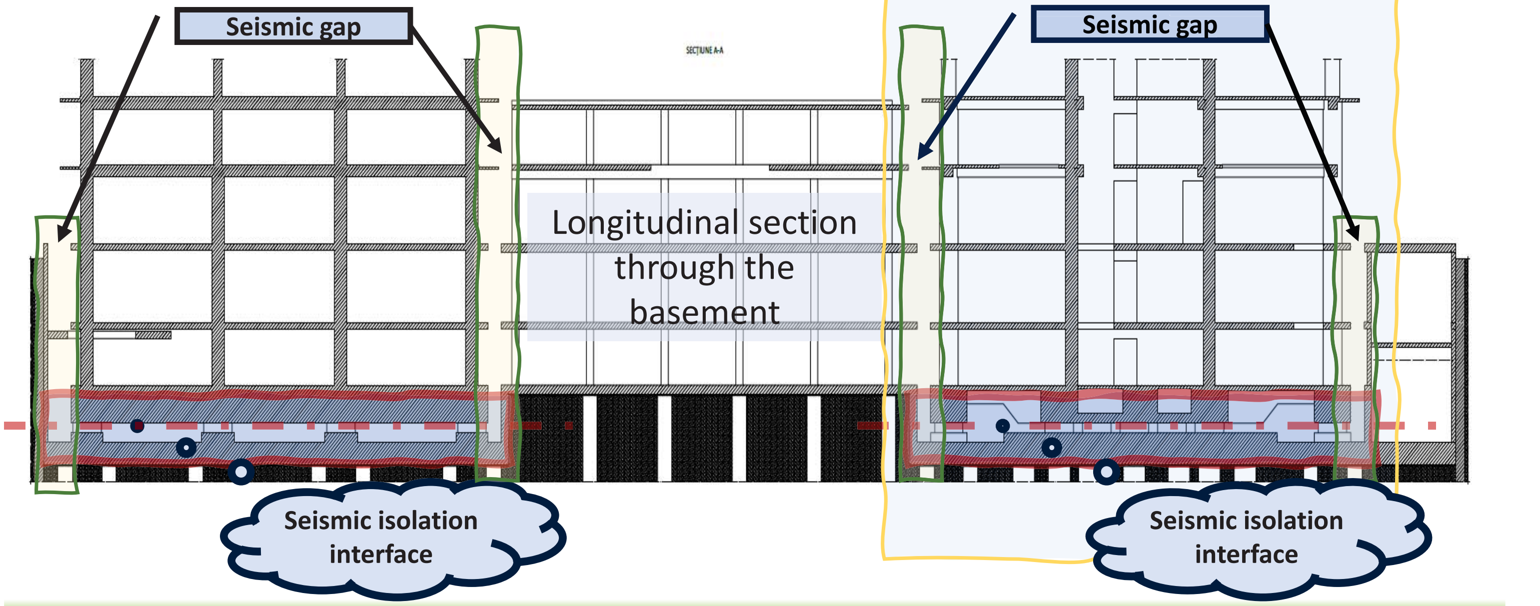
All beams are 60 x 65 cm in section, except for coupling beams between the core walls, which are 85 cm high.

Structure - description of the situation

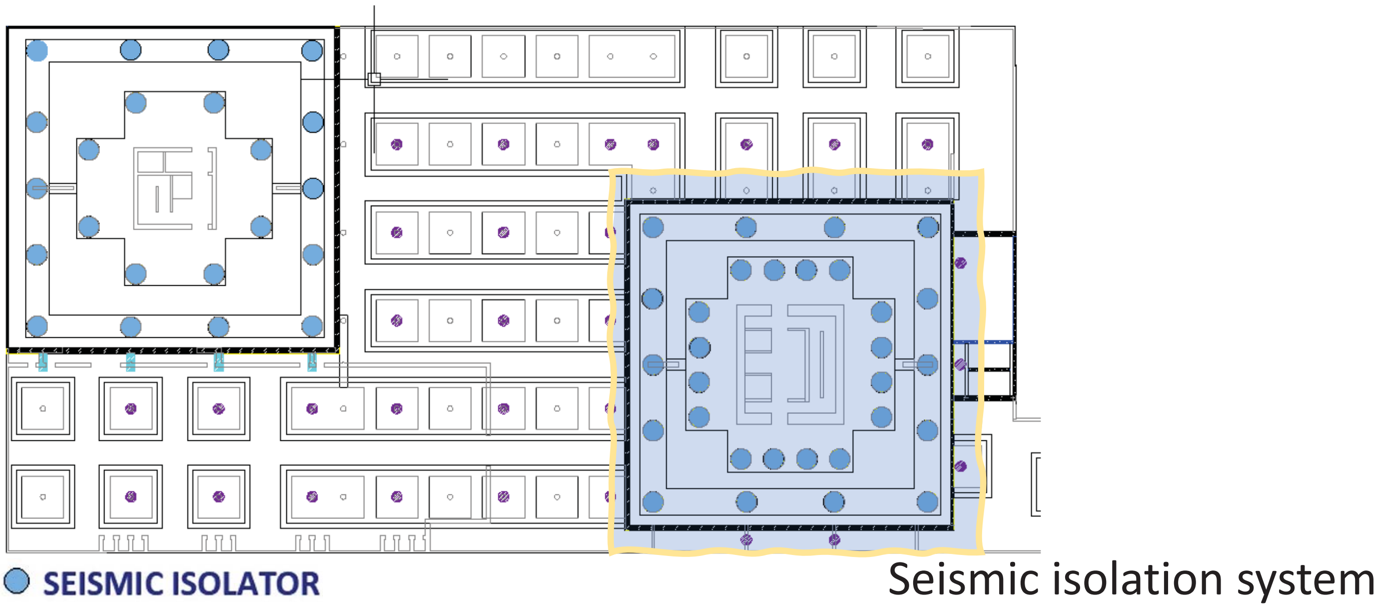


Slab above second basement layout

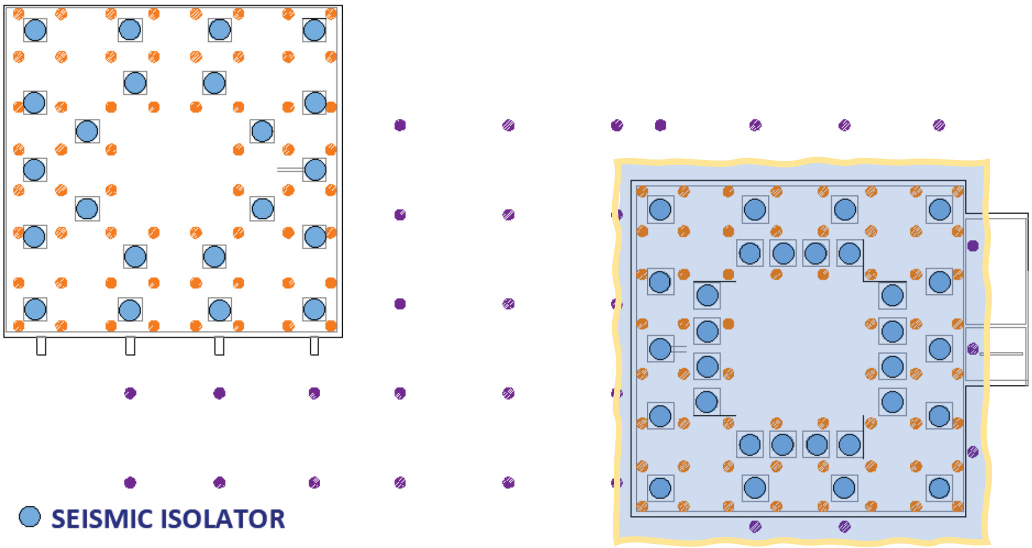
Structure description of the situation



Structure - description of the situation

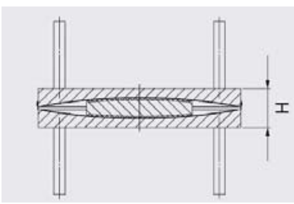


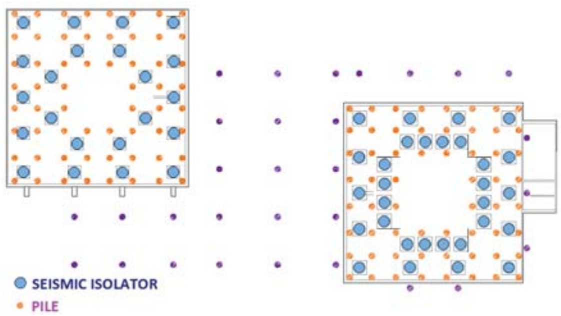
Structure description of the situation



Seismic Isolation System Overlay on Pile Layout

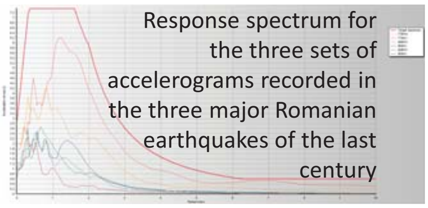
Structure description of the situation



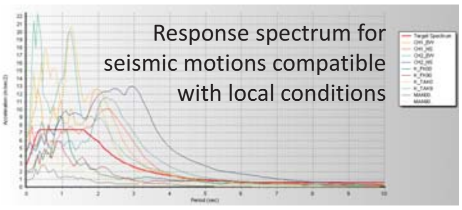


- The total number of seismic isolators is 30, all of the double friction pendulum type, to minimize their overall dimensions.
- The superstructure transfers loads to the isolators through a reinforced concrete structure that includes a ring of external beams (250x200 cm section) and an internal plate 200 cm thick, topped by a thin plate.
- There are 14 isolators positioned beneath the columns and outer walls.
- Under the central transfer plate, 16 isolators are arranged evenly around its perimeter.
- All isolators are installed above the foundation slab, with piles grouped around each to ensure proper load transfer into the foundation ground.

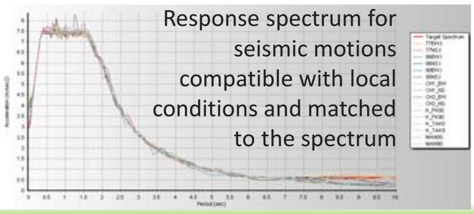
Brief summarizing of the results



Response spectrum for the three sets of accelerograms recorded in the three major Romanian earthquakes of the last century



Response spectrum for seismic motions compatible with local conditions



Response spectrum for seismic motions compatible with local conditions and matched to the spectrum

The response spectrum for the three sets of accelerograms recorded from the three major Romanian earthquakes of the last century is presented. These Romanian earthquakes, with their dates and magnitudes, are:

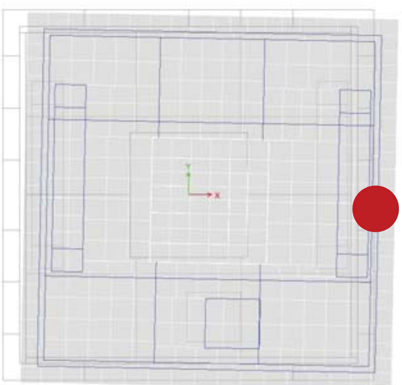
- •March 4, 1977, Vrancea, magnitude 7.4 Mw
- •August 30, 1986, Vrancea, magnitude 7.1 Mw
- •May 30, 1990, Vrancea, magnitude 6.9 Mw

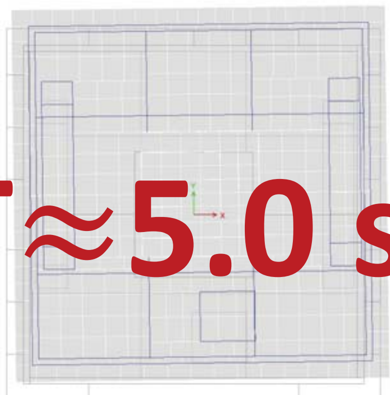
These earthquakes were significant in terms of impact and contributed valuable accelerograms for seismic behavior analysis compatible with local ground conditions.

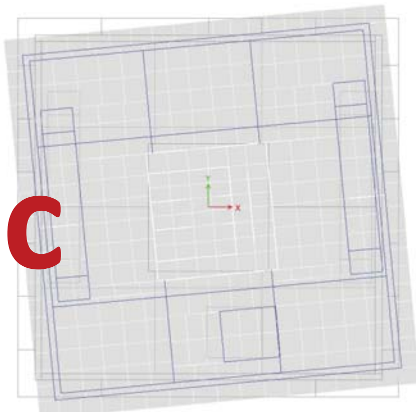
Additionally, from the PEER database, five sets of records were selected that are compatible with the fault mechanism and local terrain conditions (in terms of the corner period Tc):

- •Chuetsu-oki, Kashiwazaki City Center, 16.07.2007 (CH1)
- •Chuetsu-oki, Kariwa, 16.07.2007 (CH2)
- •Kobe Japan, Fukushima, 16.01.1995 (K_FK)
- •Kobe Japan, Takatori, 16.01.1995 (K_TAK)
- •Manjil Iran, Tonekabun, 20.06.1990 (MAN)

Tower1







•T≈5.0 sec

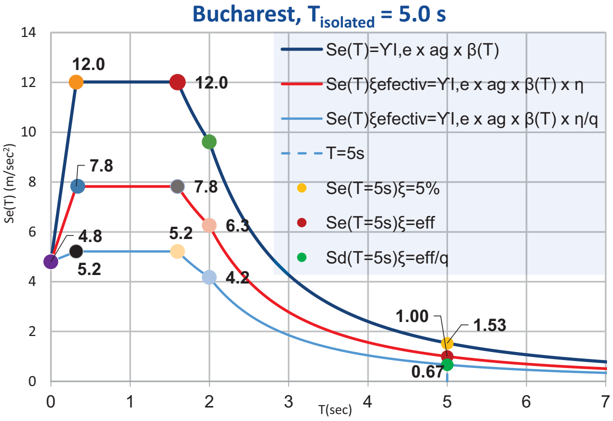
1st Mode of vibration: translation along the X axis: **T1=5.117sec**

•2nd Mode of vibration: translation along the X axis: **T2=5.071 sec**

•3nd Mode of vibration torsion: **•T3=4.477 sec**

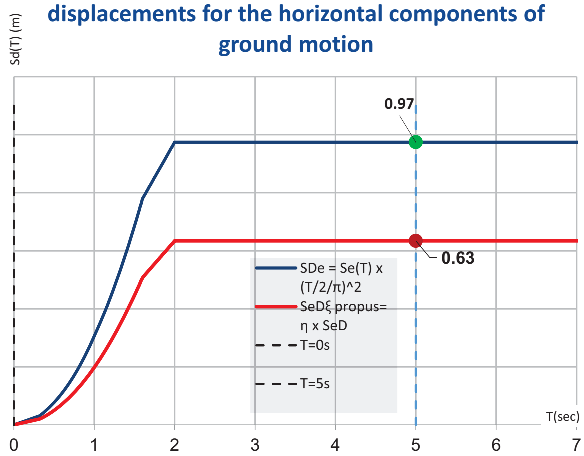
Structure - description of the situation

Elastic response spectrum of absolute accelerations for 5%, 18.6% damping, 18.6% damping divided by behaviour factor (q=1.5) - Bucharest, $T_{isolated}$ = 5.0 s

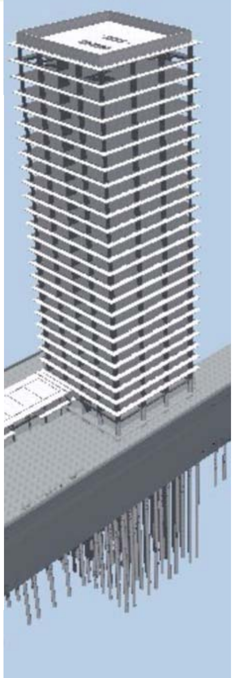


Elastic design spectra- accelerations (recurrence period 475years)

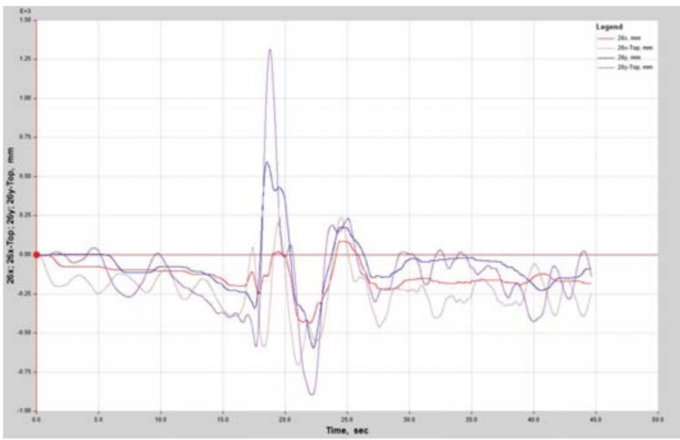
5%/ξeff response spectrum of relative displacements for the horizontal components of ground motion



Design spectra- displacements (5% damping) (recurence period 475years)

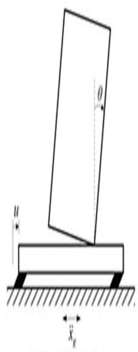


Brief summarizing of the results



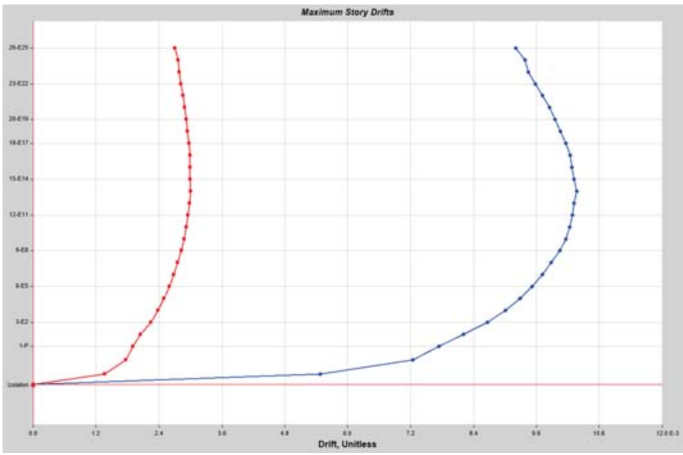
Tower 1

- maximum directional displacement in the isolator: **590 mm**;
- design displacement of the isolator / maximum combined directional displacement (SRSS) in the isolator: **720 mm**;
- maximum displacement in the structure: **723 mm**;
- Total maximum displacement at the top of the building, combining seismic isolator displacement and structural deformation itself: **1313 mm**.



(c) Rocking regime

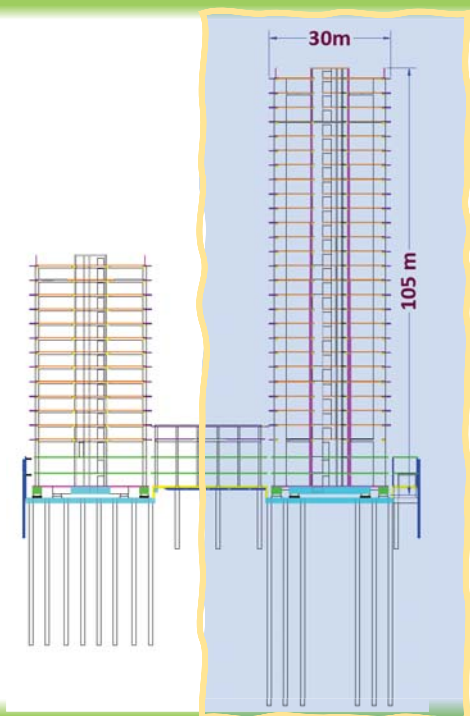
Brief summarizing of the results



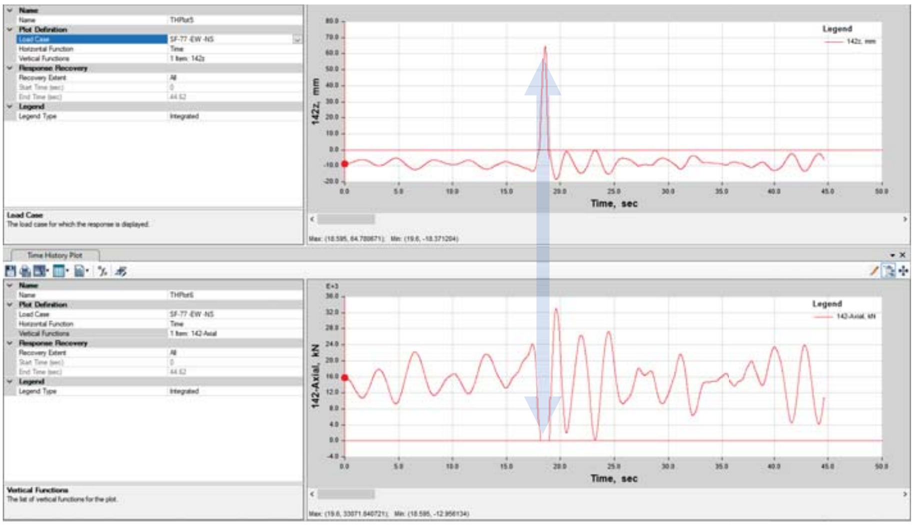
- Drift associated with the ultimate limit state: max 1.04%
- Based on the relationships in P100-1, the SLS drift = 0.52%

Uplift and pounding - The challengers

Ratio width/height >3
Overturning most probable



Brief summarizing of the results



The plots present the variation of the vertical displacement(top) and vertical reaction (bottom) in one outer isolator.

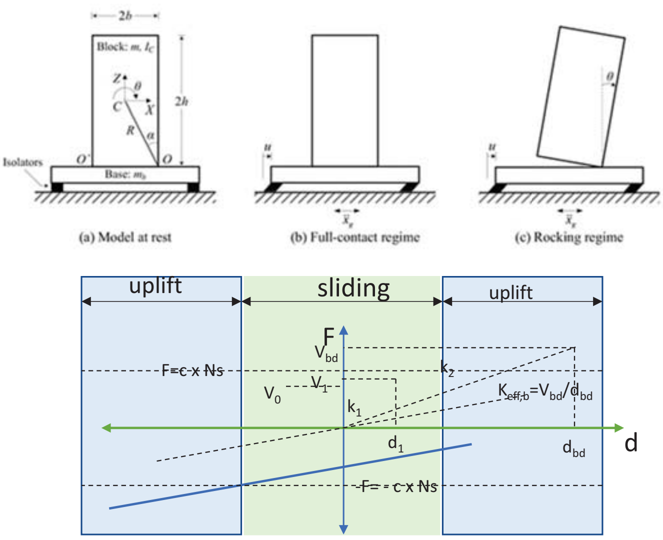
1.Uplift of the isolators

Throughout the time steps of the time history analyses there are some time steps when the energy input in the structure lead to large overturning moments that causes uplift in some isolators.

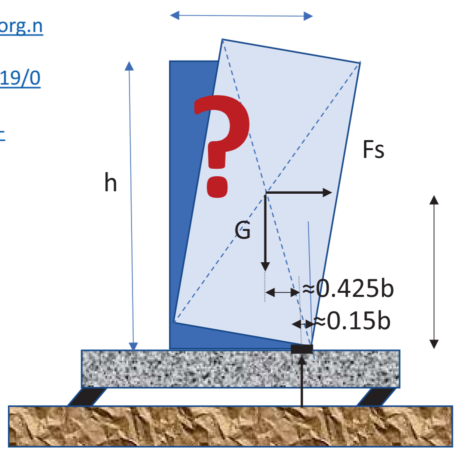
This uplift appears at the time step of the peak acceleration of the recording, after 18s.

Uplift and pounding - The challengers

Fig. (1) Model at rest and oscillation patterns considered.

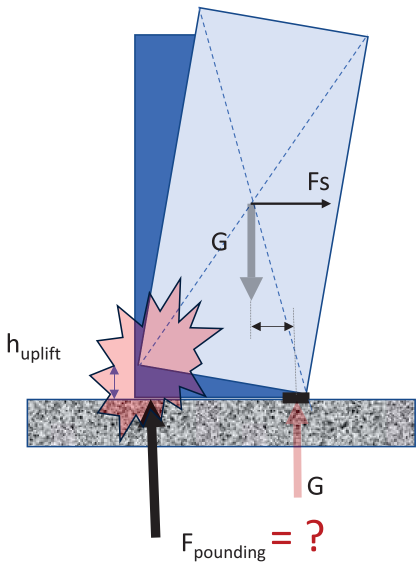


https://www.nzsee.org.nz/wp-content/uploads/2019/06/2825-Seismic-Isolation-Guidelines-Digital.pdf



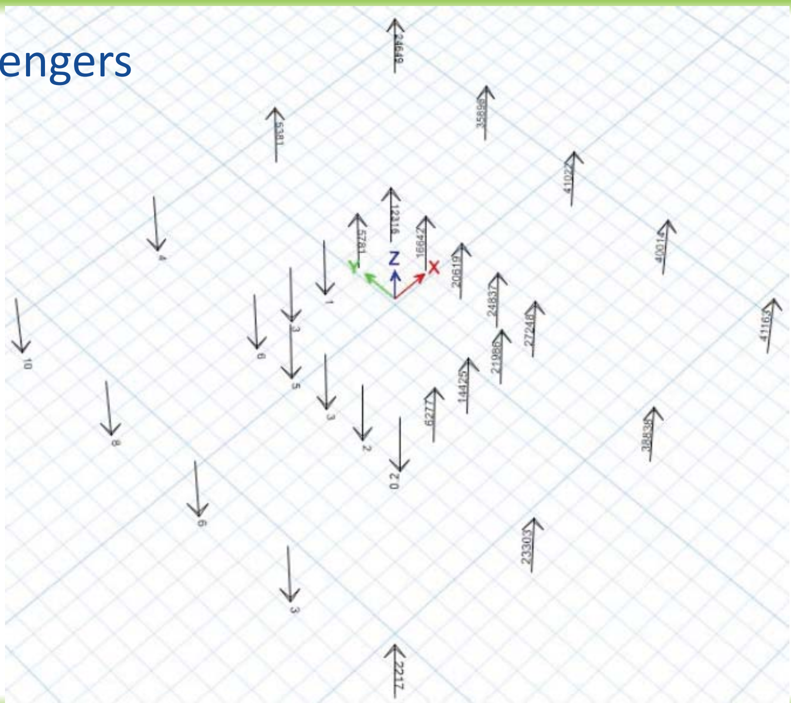
Uplift and pounding - The challengers

Magnitude of pounding force is dependent on support stiffness – therefore mainly on the raft/soil/piles stiffness



Uplift and pounding - The challengers

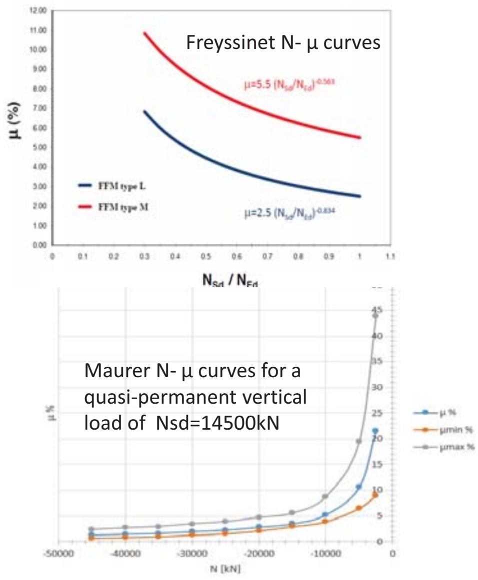
The system is stable because the overturning moment is balanced with larger vertical reaction in the compressed isolators and the mobilized bending moments in the supporting structure and superstructure beams that work as a lever.



Variable conditions for seismic isolation

The behavior of the sliding seismic isolator depends on:

- Vertical force actioning on seismic isolator – main variables:
  - Stiffness of the foundation
    - piles 3 values (LB+UP NIST GCR / NP123/Plaxis model)
    - Soil – neglected in the end
    - Foundation Raft (RC)
    - Slurry walls on contour
  - Stiffness of the superstructure.
    - RC slab and beams
- Friction on the surface
  - The friction coefficient µ depends on:
    - **Pressure on the sliding surface**
    - Temperature
    - Sliding velocity
    - There is un upper and lower bound due also to the fabrication process



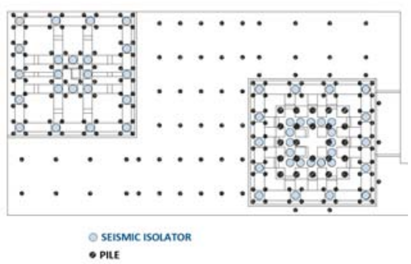
Influence of pressure magnitude on the friction coefficient of a sliding surface

Geotechnical findings

Estimation of the proportion to be carried by piles and raft (Poulos Davis Randolph).

On simple models it was assessed the contribution to the total stiffness of the piles and soil.

A group of 4 piles and surrounding (tributary) raft has been taken into account for analysis



Caracteristici geometrice și caracteristici de teren:

L_r	Lungimea radierului pilotat	17.00	m
B_r	Lățimea radierului pilotat	17.00	m
A_{rp}	Aria radierului pilotat	289	m^2
Q_{pilot}*	Capacitatea la compresiune a piloților	9362	kN
$Nr._{piloti}$	numărul de piloți	16	buc.
P_{piloti}	capaciatatea de preluare a forțelor de compresiune a piloților	149792	kN
r_0	raza pilotului	0.5	m
$s_{ultim\ pilot}$	tasarea la curgere a grupului de piloți	0.1	m
Rigiditate grup			
k_{rp}	rigiditatea radierului pe teren (met însumării pe straturi)	718	
k_r	rigiditatea radierului pe teren propunere	718	kN/m^3
k_p	rigdiditatea unui pilot	93620	kN/m
K_p	kp x nr pil (rigid. Gr. Pil.)	1497920	kN/m
K_r	kr x (Bradier x Lradier-nr (rig radier)	198479	kN/m
K_{rp}	krp x (Bradier x Lradier-nrpil*Apil)	198479	kN/m
K_{pr}	rigiditatea radierului pilotat		
r_c	raza echivalentă = (LradierxBradie/nr.pil/π)^0.5	2.40	
H	lungime pilot	50	m
D	adâncime de fundare	11.85	m
E_{sl}	Modul de elasticitae în stratul unde e vf pilot	20500	kPa
E_{sb}	Modul de elasticitae în stratul sub vf pilot	20500	kPa
E_{sav}	Modul de elasticitate mediu de-a lungul pilotului	19416	kPa
ν	Coeficientul lui Poisson	0.47266	
ξ	Esl/Esb	1	
ρ	Esav/Esl	0.947121951	
		.43	m
		.83	

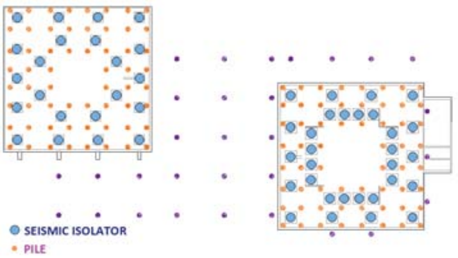
Proportion of load carried by the raft and piles

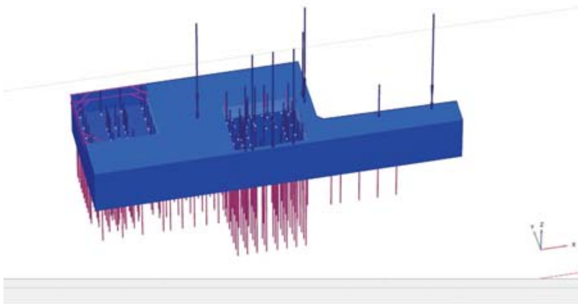
Xp	Proportion carried by piles:	0.85%	0.85 %
Xr	Proportion carried by raft:	0.15 %	0.15 %

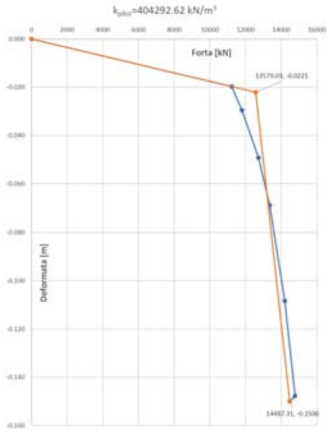
α_{cp}	$\alpha_{cp} = \frac{1 - \ln(r_c / r_0)}{\zeta}$	-0.0614	
	$K_{pr} = \frac{K_p + K_r \cdot (1 - \alpha_{cp})}{1 - \alpha_{cp}^2 \cdot K_r / K_p}$	439297	kN/ml
P_1	Forța totală la cedarea piloților	43903	kN
s_1	Tasarea la care piloții își epuizează capacitatea portantă	0.085	m
s_u	Tasarea la care radierul își epuizează capacitatea portantă	1.055	m

Geotechnical findings

NP 112/Plaxis model/Etabs implementation







3D model of the infrastructure, the slurry walls and piles

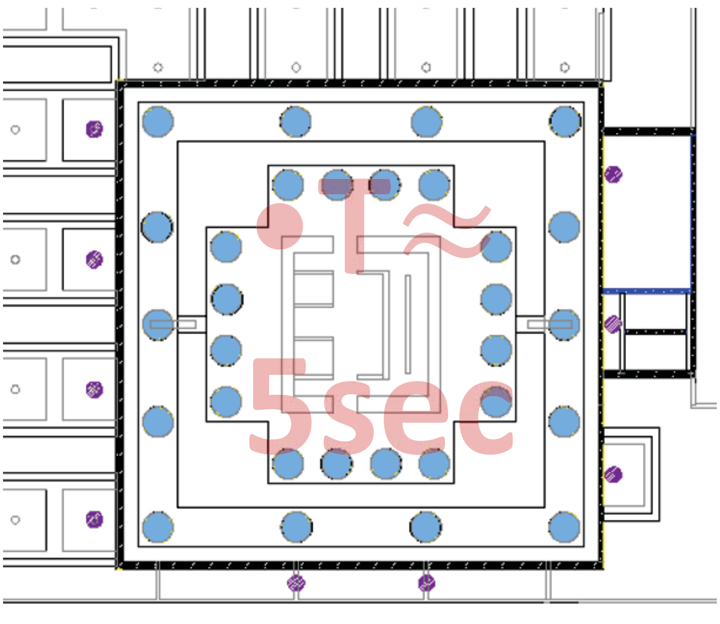
Geotechnical findings

Foundation system stiffness according to NIST GCR 12-917-21

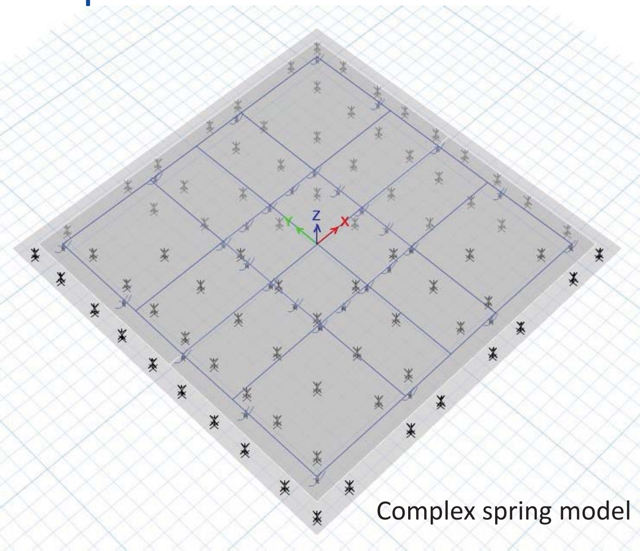
- **Vertical stiffness of the raft foundation**- 11.25MN/m^3 with an increase of 10.3% for the dynamic value.
- **-1.5m diameter pile** with 50m length:

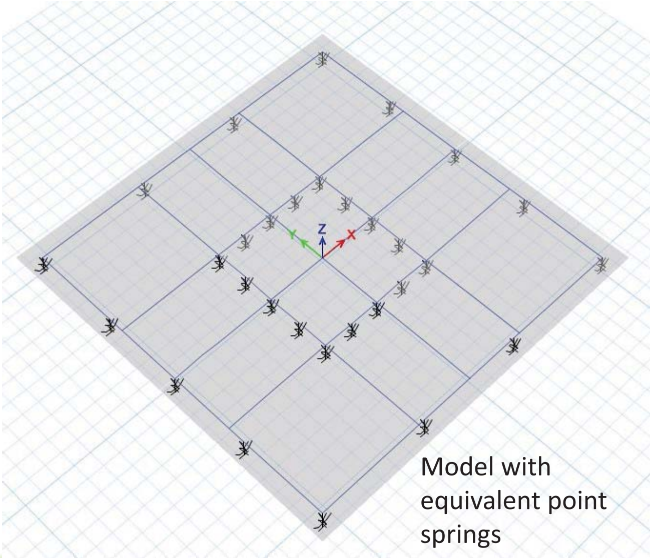
A dynamic vertical stiffness of 2243MN/m (dynamic stiffness modifier being 1.0178%).

- **-1.0m diameter pile** with 50m dynamic vertical stiffness of 1491MN/m (dynamic stiffness modifier being 1.0146%).



Complex condition – big computing model – we needed simplifications

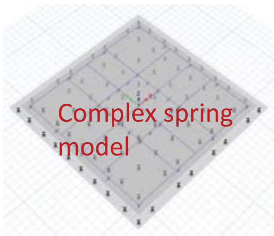


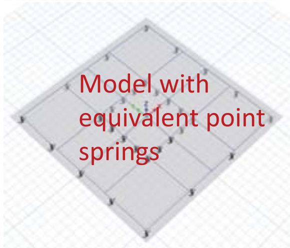


Complex condition – big computing model – we needed simplifications

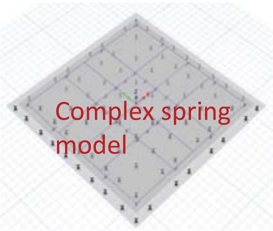
During the process it was seen that upper bound of the stiffness of the piles lead to really big vertical forces in the seismic isolators.

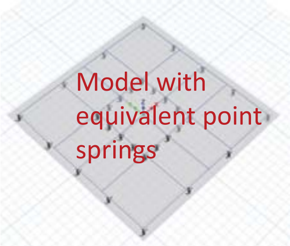
- Actual distribution of isolators under central core is highly dependent on pile stiffness.
- We therefore propose to move isolators in order to have a well-controlled system (concrete beams) to limit the amount of compression to be taken by seismic isolators.
- With the distribution of the isolator directly above the piles the upper bound for vertical stiffness according to NIST GCR 12-917-21 lead to vertical reactions in the isolators above 70MN that exceeded the maximum testing capacity of any facility (no damping).
- Several iterations targeted the change in distribution and position of isolators to obtain a minimum stable value for the vertical reaction (not so highly dependent on highly variable pile stiffness).

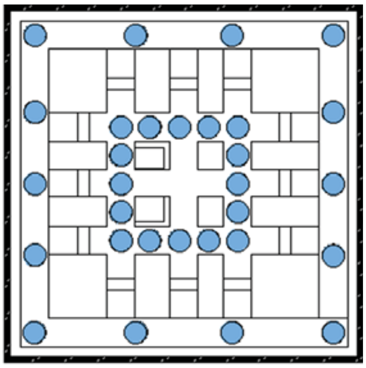




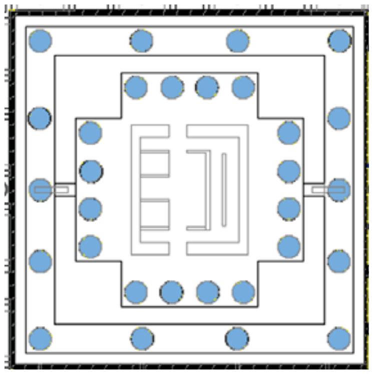
Complex condition – big computing model – we needed simplifications







First layout of the isolators



Revised layout of the isolators

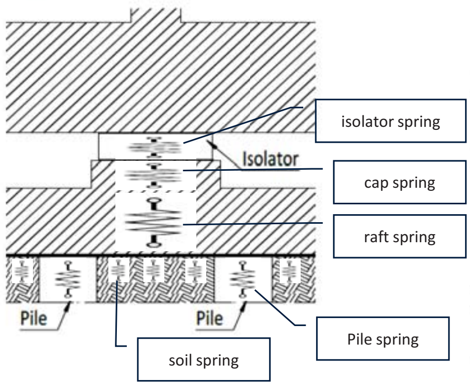
Complex condition – big computing model – simplifications

Piles were combined as parallel springs along with the soi and then group of piles, concrete raft and isolator were assembled as serial springs.

The serial spring distribution is as follows:

1. Isolator cap
2. Raft foundation
3. Piles (parallel to soil stiffness)

Stiffness evaluation



Isolator	K	15.5	E6	kN/m
	VSF	1.3		
	K1	20.15	E6	kN/m

Isolator cap	B	2.5		m
	L	2.5		m
	H	0.5		m
	E	37	E6	kN/m
	K2	462.5		kN/m

Equivalent spring stiffness 1	$k_{serial1}$	19.30876411	E6	kN/m

Raft foundation	B	4.9		m
	L	4.9		m
	H	1.2		m
	K	740.31	E6	kN/m

Equivalent spring stiffness 2	$k_{serial2}$	18.82	E6	kN/m

		Nominal		
Piles	k1	1.8	E6	kN/m
	n	4		pcs.
	K	7.2		kN/m

Complex condition – big computing model – we needed simplifications

The serial spring distribution was studied for 3 values according to the American design methodology:
Nominal, with pile stiffness value as previously mentioned.
Lower bound with half the pile stiffness.
Upper bound with double the pile stiffness.

We found that the most conservative approach, when studying spring distributions, is to consider only the variation in pile stiffness resulting from uncertainties in soil characteristics, while keeping the other components at their nominal values—this helps avoid an overly cautious and costly design. The likelihood that all components will simultaneously reach their upper bound capacity is low, and, in any case, the safety factors used for each component combined with potential load redistributions are sufficient to prevent failure.
For the raft foundation, the design distribution area was taken as the isolator cap area, offset by the thickness of the raft foundation.

	Nominal			
Piles	k1	1.8	E6	kN/m
	n	4		pcs.
	K	7.2		kN/m

Equivalent spring stiffness 3	$k_{serial3}$	5.21	E6	kN/m

Lower Bound			
k1	0.9	E6	kN/m
n	4		pcs.
K	3.6	E6	kN/m

$k_{serial3}^{LB}$	3.02	E6	kN/m

Upper Bound			
k1	3.6	E6	kN/m
n	4		pcs.
K	14.4	E6	kN/m

$k_{serial3}^{UB}$	8.16	E6	kN/m

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Complex condition – big computing model – we needed simplifications- Results

The serial spring distribution was studied for 3 values according to the American design methodology for assessing the results variability in terms of vertical forces:
Nominal, with pile stiffness value as previously mentioned.
Lower bound with half the pile stiffness.
Upper bound with double the pile stiffness.
It was found that variability of results is small and structural layout is not dependent on large variability of the pile stiffness.

Crt. No.	Isolator point label	μ 3.0% & K=3.02E6 KN/m	μ 3.0% & K=5.21E6 KN/m	μ 3.0% & K=8.16E6 KN/m		μ 5.6% & K=3.02E6 KN/m	μ 5.6% & K=5.21E6 KN/m	μ 5.6% & K=8.16E6 KN/m
1	178	36384	37277	37559		41904	43043	44254
2	335	33227	34385	35339		35721	37420	38769
3	339	34064	35402	36482		36270	38112	39608
4	188	37168	38215	38540		42923	44295	44796
5	171	33661	33958	33811		39019	39519	39408
6	337	33146	34421	35529		35611	37466	39003
7	341	34148	35607	36797		36321	38230	39773
8	173	37743	38898	39288		43496	44961	45559
9	246	37565	39206	40084		42875	45367	46862
10	259	36228	39182	41460		37389	40354	42795
11	256	36359	39407	41873		38025	41446	44261
12	241	35345	36497	37023		40464	42421	43539
13	268	37678	39440	40394		43239	45919	47561
14	333	35042	37609	39566		36290	39045	41317
15	282	34813	37343	39293		36228	38977	41218
16	278	38185	39972	40929		43738	46417	48076
17	142	30392	30569	30684		33315	33592	33790
18	143	29322	29158	29042		31061	30817	30664
19	144	29588	29415	29307		31381	31156	30991
20	145	30805	31012	31155		33753	34065	34283
21	23	30847	31107	31282		33806	34180	34420
22	24	29534	29420	29342		31303	31133	31011
23	25	29408	29227	29106		31202	30961	30780
24	26	30846	31058	31200		33802	34120	34335
25	123	22968	22767	22759		24182	24304	24476
26	84	20664	18577	16956		21368	19446	17967
27	44	22316	22080	22045		23565	23667	23815
28	126	22510	22272	22243		23683	23770	23920
29	91	20643	18688	17182		21367	19580	18207
30	49	22583	22328	22275		23783	23874	24030

Vertical reaction variation for supporting structure stiffness and isolator friction coefficient

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ
Influence of pile stiffness on behavior of slender base isolated structures. Dan Iancu & prof. dr. Horațiu Popa
Societatea Română de Geotehnică și Fundații

Complex condition – big computing model – we needed simplifications- Results

- Static stiffness (NP112) of 150MN/m for the interior piles and 180MN/m for the exterior piles;
- Dynamic stiffness of 2 times the value resulted from NIST GCR 12-917-21: 4800MN/m;
- Dynamic stiffness of half the value resulted from NIST GCR 12-917-21: 1200MN/m;

Case	+NS +EW	+EW+NS
Spring model	Max. Vertical Reaction (KN)	
Static stiffness	43616	45129
0.5*Dynamic stiffness NIST	38561	38139
2.0*Dynamic stiffness NIST	40939	44495

Case	+NS +EW	+EW+NS
Spring model	Uplift (mm)	
Static stiffness	2.81	5.27
0.5*Dynamic stiffness NIST	8.54	11.07
2.0*Dynamic stiffness NIST	6.47	7.9

Vertical reaction variation for supporting structure stiffness and isolator friction coefficient

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Conclusion

- When addressing complex problems, the stiffness of the foundation system becomes critically important.
- Adding further uncertainties makes the problem even more challenging to solve.
- Step-by-step calibration models are necessary to bridge interdisciplinary knowledge—combining theoretical insight with computational techniques.
- For a successful design, safety must not be overly sensitive to wide variations in input parameter values.

Influence of pile stiffness on behavior of slender base isolated structures. Dan Iancu & prof. dr. Horațiu Popa

Ευχαριστώ πάρα πολύ!

I sincerely appreciate the opportunity to have your attention!

Alexandra Ene

Civil engineer, UTCB, Secretary General of the Romanian Society for Soil Mechanics and Foundation Engineering

Treatment of uncertainties for a deep excavation project in complex ground conditions

Alexandra Ene is a civil engineer mastered in geotechnical engineering, working in geotechnical and structural design since 2010. Most of the projects she took part in involved geotechnical structures such as retaining walls for deep excavations and foundations for buildings. Also, in her activity, there are site investigations and geotechnical and structural monitoring works, coordinating a team of about 10 people working in geotechnics. She is affiliated member of several professional associations and governmental experts group in Romania, including the Romanian Society of Geotechnics and Foundations where she has been General Secretary between 2021 and 2025. Alexandra has been involved in the revision of Eurocode 7 both as a working group expert and national delegate since 2006. She is also a Phd student at the Technical University of Civil Engineering in Bucharest, in geotechnics and foundations domain, where she is doing research on reliability-based design methods for geotechnical structures with the aim to apply these in practice.

Alexandra Ene's presentation

is not publicly available

on the basis of confidentiality agreements

with stakeholders involved in the case presented

Evangelia Garini

Assistant Professor, Civil Engineer (Ph.D., M.S., Diploma), Technical University of Crete

Soil Effects and Geotechnical failures in the 2023 Kahramanmaraş Earthquakes in Turkey

Since 2023 serves as an Assistant Professor at the Department of Mineral Resources Engineering of the Technical University of Crete in the subject of "Static and Dynamic Analysis of Geotechnical Structures". Holds a Master of Science in Geotechnical engineering from the State University of New York at Buffalo and a Diploma in Civil Engineering from the National Technical University of Athens. Obtained a Ph.D. degree in Geotechnical Engineering supervised by Professor G. Gazetas, on soil dynamics and soil-structure-interaction. During 2015–2022 she worked as a Researcher at NTUA in the Soil Dynamics Laboratory, where she was awarded twice with the IKY-Excellence-Siemens fellowship program (2013-14 and 2014-15). Her work has resulted in 22 articles in journals and 55 in conference proceedings. As a member of GEER (Geotechnical Extreme Events Reconnaissance, USA), HSSMGE (Hellenic Society of Soil Mechanics and Geotechnical Engineering), ETAM (Hellenic Society of Earthquake Engineering), and GeoWB (GeoEngineers Without Borders, ISSMGE) she participated in several Post-earthquake expeditions, and has published Reconnaissance Reports on Earthquake and Natural Disasters on: the Noto Peninsula (Japan) Earthquake of 1.1.24; the 5.9.23 Daniel Storm Flooding in Thessaly, Greece; the Mw 7.8 and Mw 7.5 earthquakes of 6.2.23 in Turkey ; the Puebla (Mexico) 19-9-17 Mw 7.2 Earthquake; the 2016 Kaikoura Mw7.8 New Zealand Earthquake; and the 2014 January 26th and February 2nd Cephalonia, Greece, Earthquakes.

Soil effects and Geotechnical observations from the February 6th, 2023 Turkey earthquakes

Evangelia GARINI

Assistant Professor, Technical University of Crete

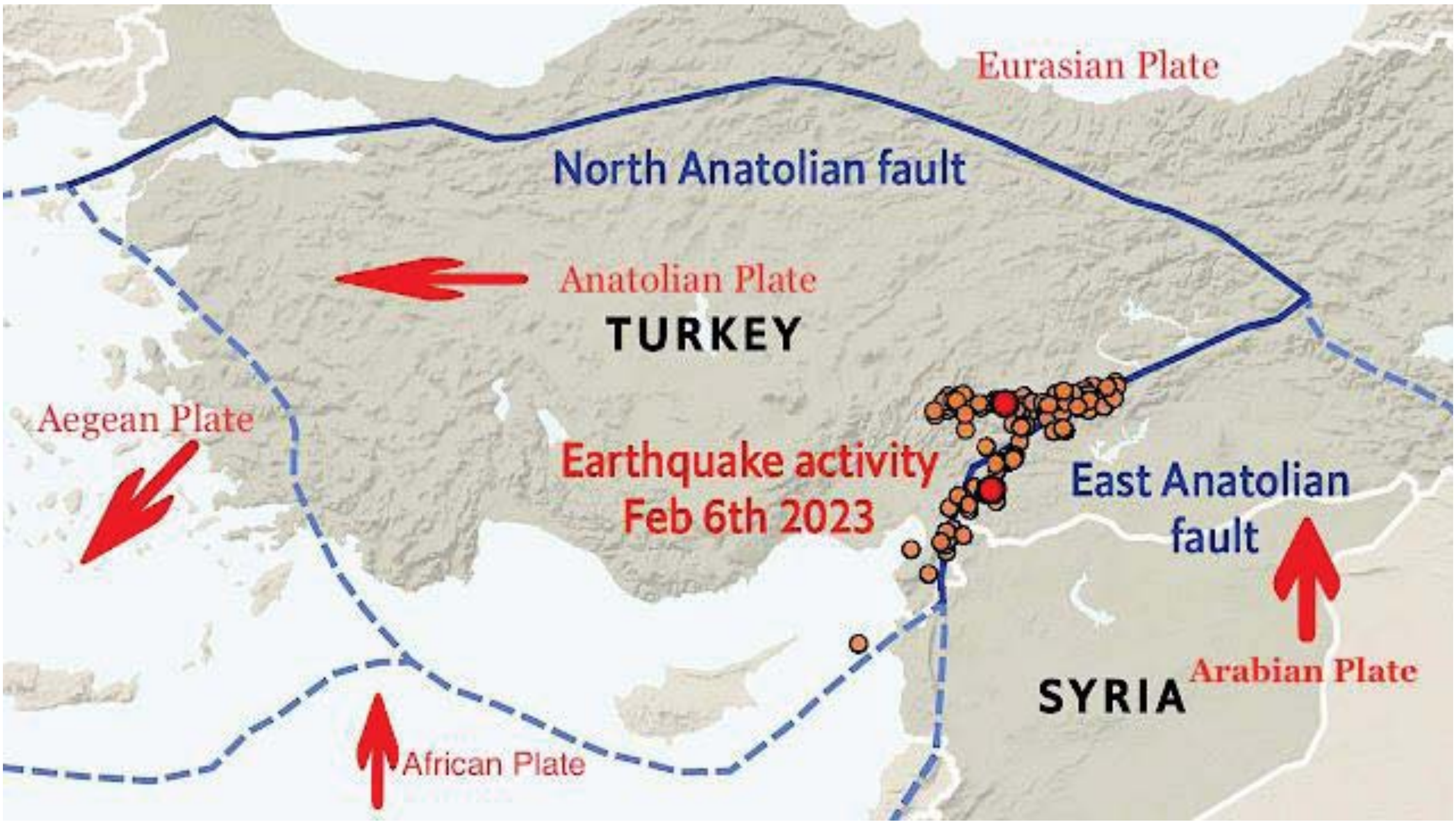
George GAZETAS

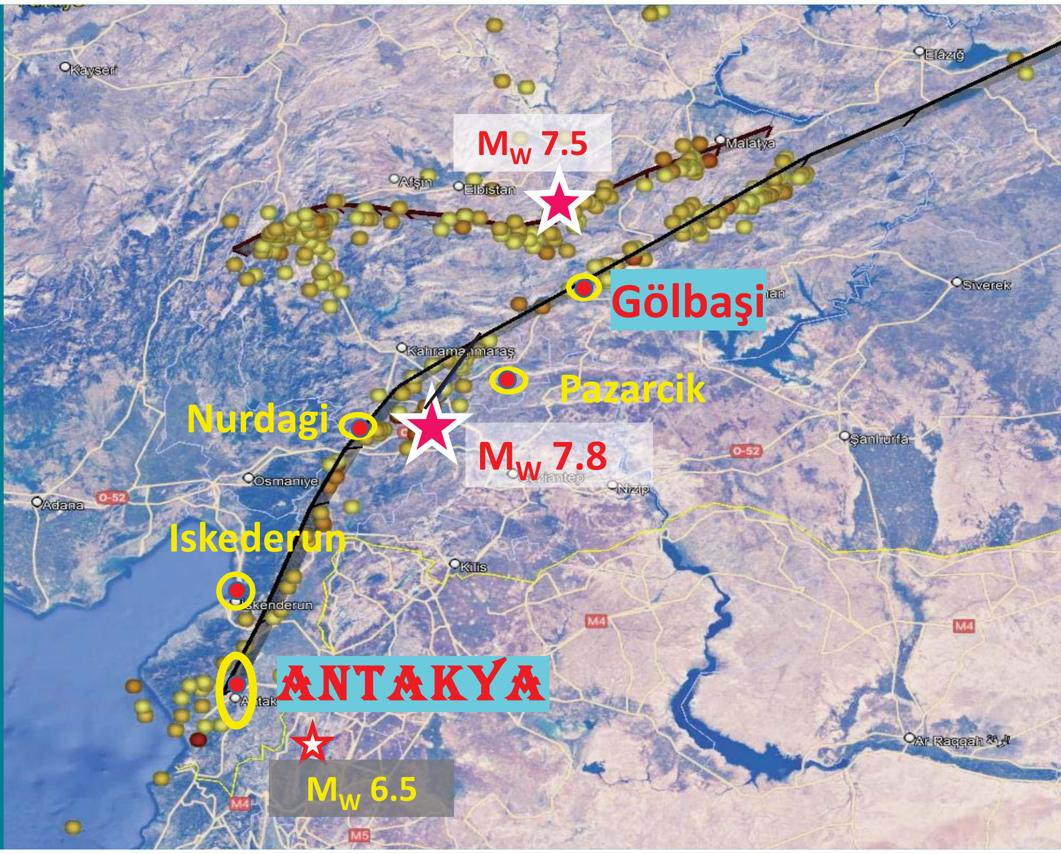
Emeritus Professor, National Technical University of Athens

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

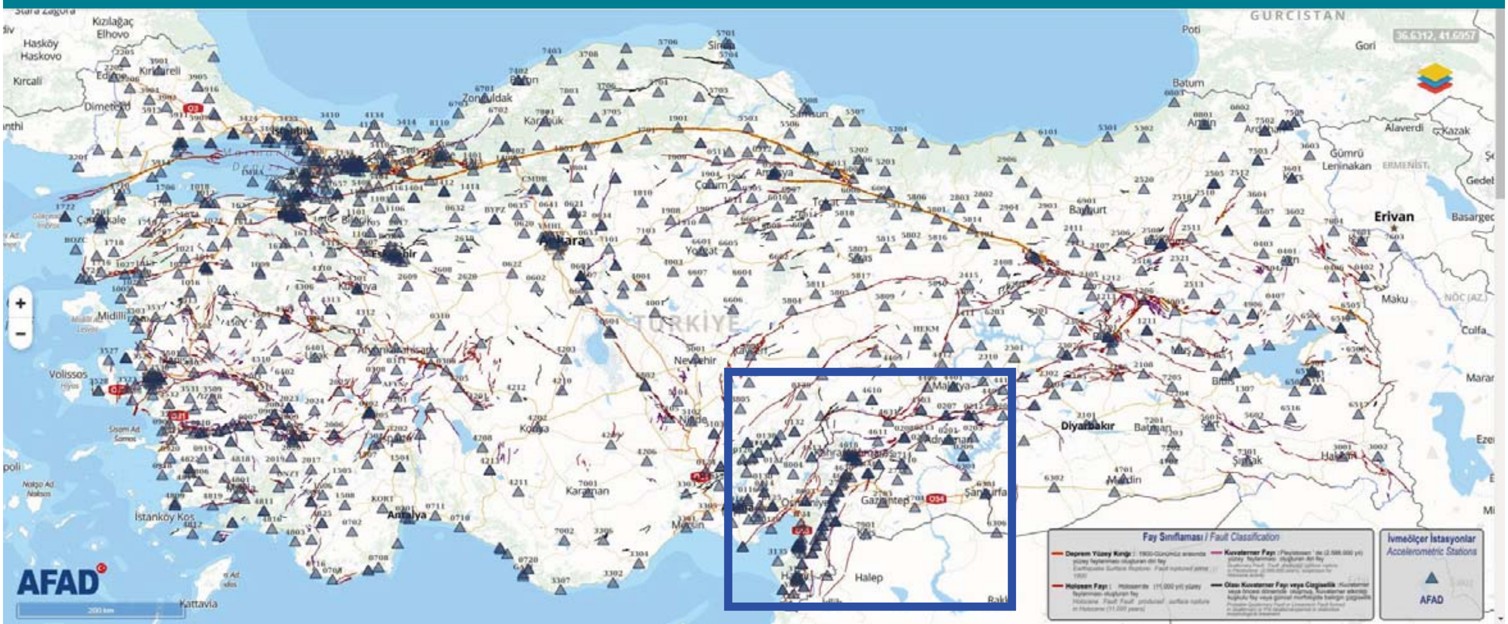
Thessaloniki, 9th October 2025

Societatea Română de Geotehnică şi Fundaţii

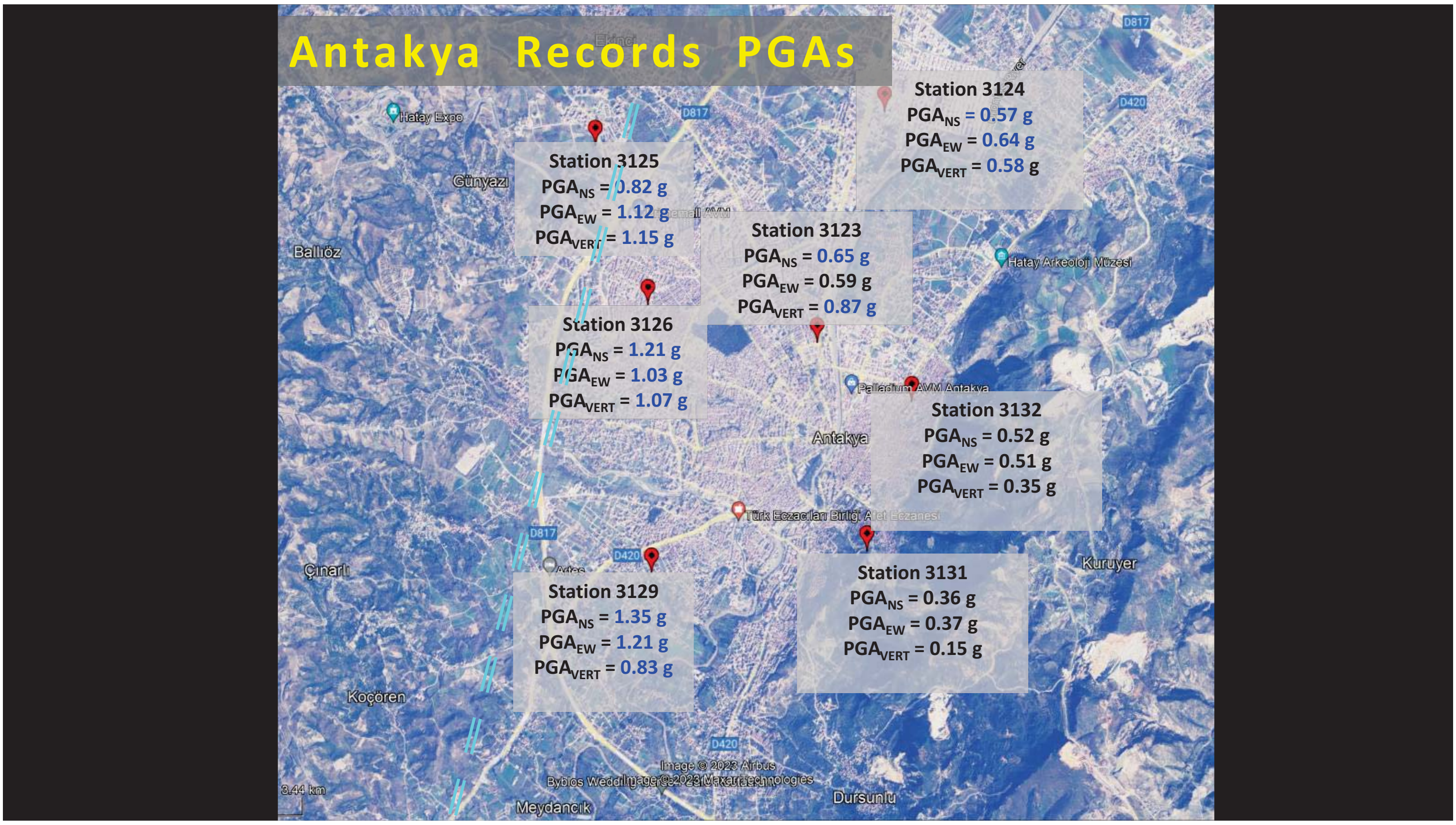




The outstanding Strong Motion Network of Turkey



Total > 1000 recording stations !!!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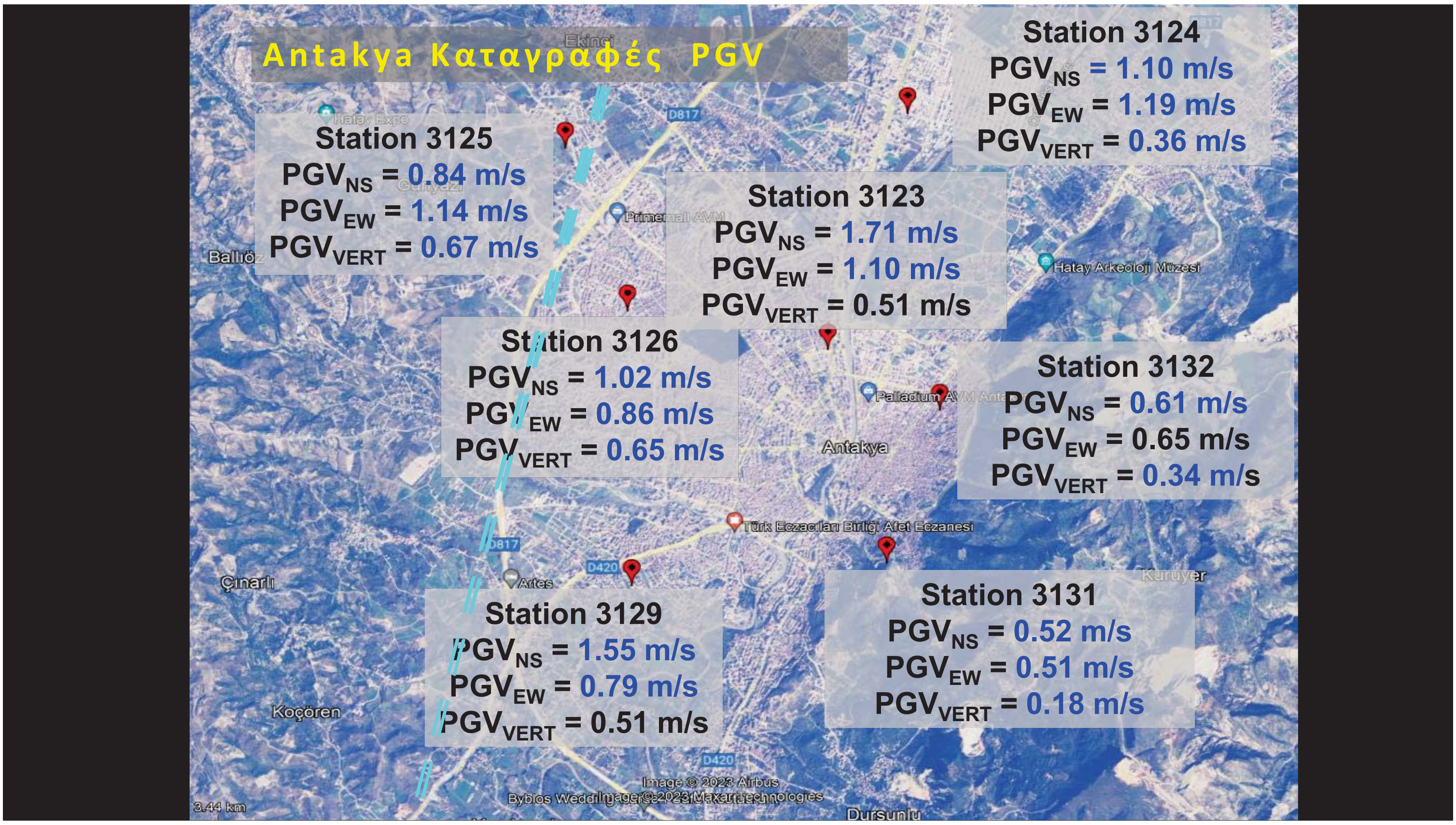
PGA_VERT = 0.58 g
Station 3125
PGA_NS = 0.82 g
PGA_EW = 1.12 g
PGA_VERT = 1.15 g
Station 3123
PGA_NS = 0.65 g
PGA_EW = 0.59 g
PGA_VERT = 0.87 g
Station 3126
PGA_NS = 1.21 g
PGA_EW = 1.03 g
PGA_VERT = 1.07 g
Station 3132
PGA_NS = 0.52 g
PGA_EW = 0.51 g
PGA_VERT = 0.35 g
Station 3129
PGA_NS = 1.35 g
PGA_EW = 1.21 g
PGA_VERT = 0.83 g
Station 3131
PGA_NS = 0.36 g
PGA_EW = 0.37 g
PGA_VERT = 0.15 g
Antakya
Güneyazı
Ballıöz
Çınarlı
Koçören
Meydancık
Dursunlu
Kuruyer
Hatay Expo
Hatay Arkeoloji Müzesi
Palladium AVM Antakya
Türk Eczacıları Birliği Afet Eczanesi


Antakya Καταγραφές PGV
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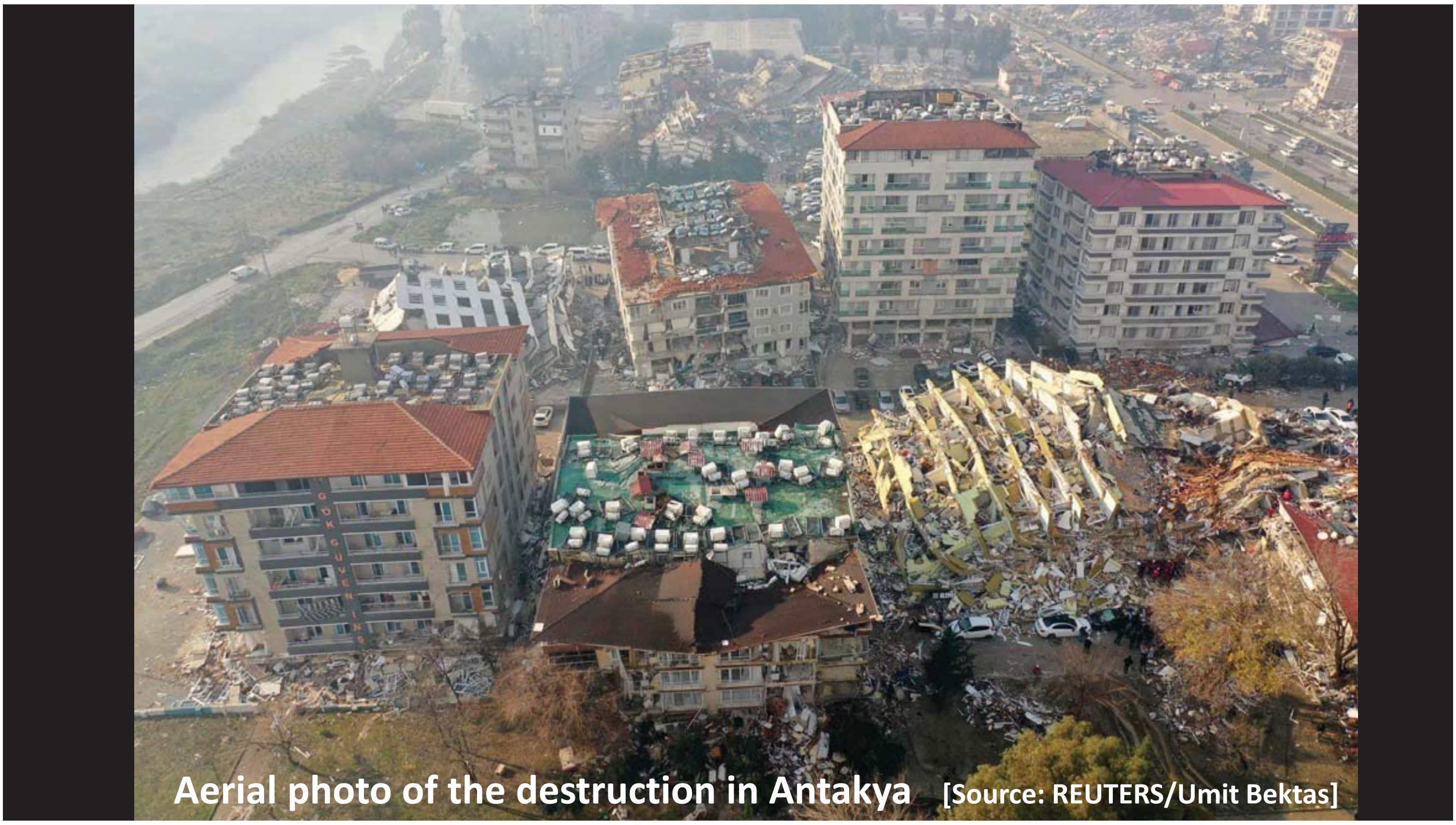
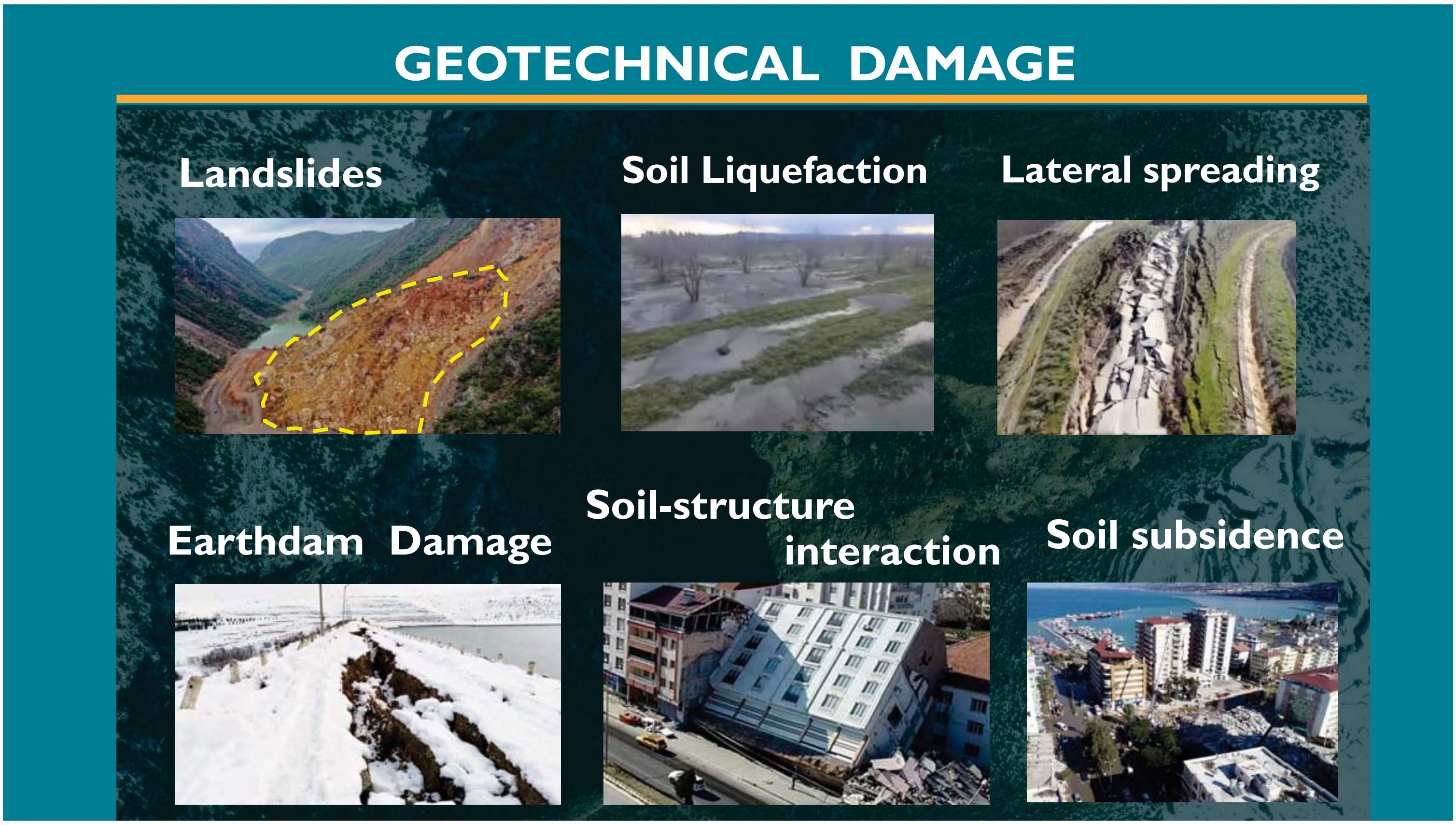
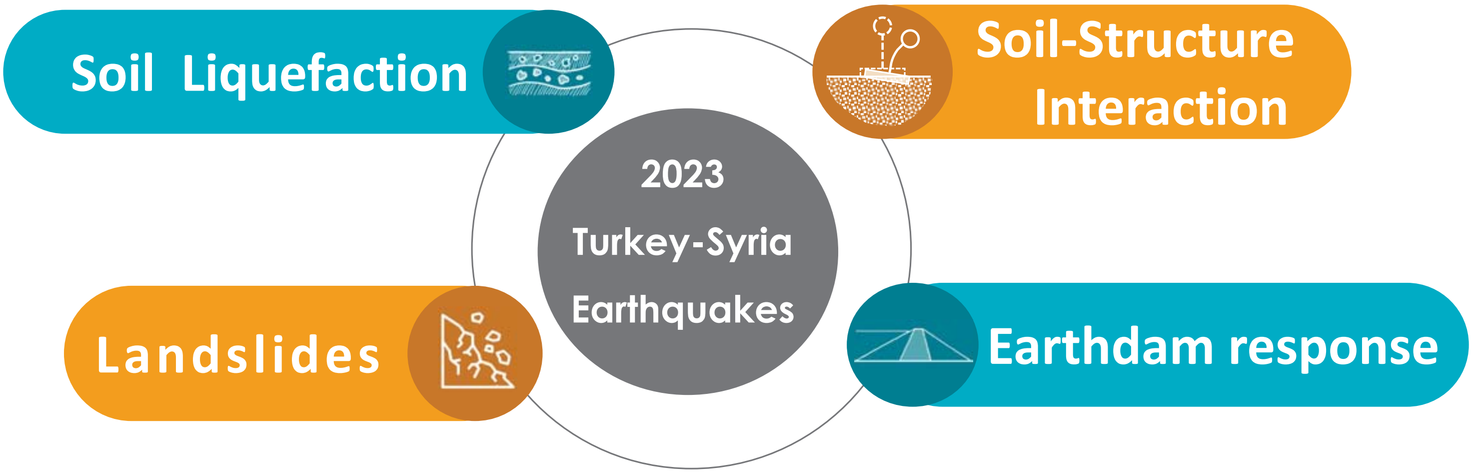
PGV_NS = 1.10 m/s
PGV_EW = 1.19 m/s
PGV_VERT = 0.36 m/s
Station 3125
PGV_NS = 0.84 m/s
PGV_EW = 1.14 m/s
PGV_VERT = 0.67 m/s
Station 3123
PGV_NS = 1.71 m/s
PGV_EW = 1.10 m/s
PGV_VERT = 0.51 m/s
Station 3126
PGV_NS = 1.02 m/s
PGV_EW = 0.86 m/s
PGV_VERT = 0.65 m/s
Station 3132
PGV_NS = 0.61 m/s
PGV_EW = 0.65 m/s
PGV_VERT = 0.34 m/s
Station 3129
PGV_NS = 1.55 m/s
PGV_EW = 0.79 m/s
PGV_VERT = 0.51 m/s
Station 3131
PGV_NS = 0.52 m/s
PGV_EW = 0.51 m/s
PGV_VERT = 0.18 m/s
Antakya
Ballıöz
Çınarlı
Koçören
Dursunlu
Hatay Arkeoloji Müzesi
Türk Eczacıları Birliği Afet Eczanesi

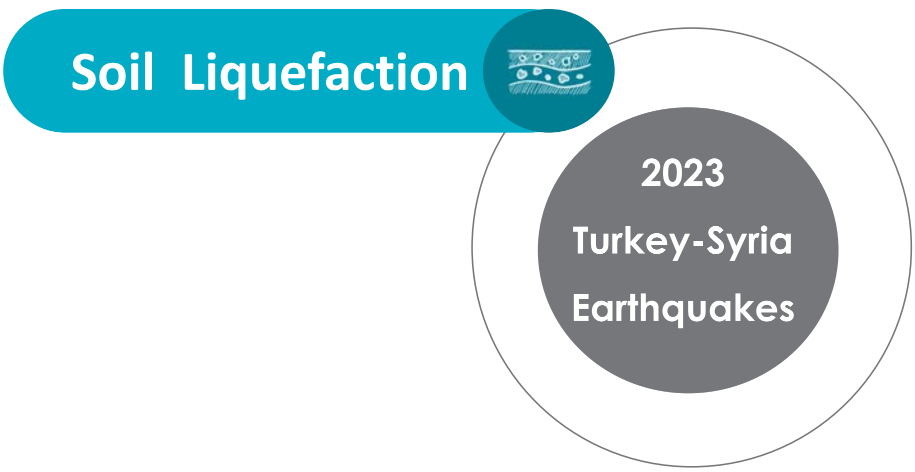

Aerial photo of the destruction in Antakya [Source: REUTERS/Umit Bektas]


GEOTECHNICAL DAMAGE
Landslides
Soil Liquefaction
Lateral spreading
Earthdam Damage
Soil-structure interaction
Soil subsidence

Glance over



Glance over



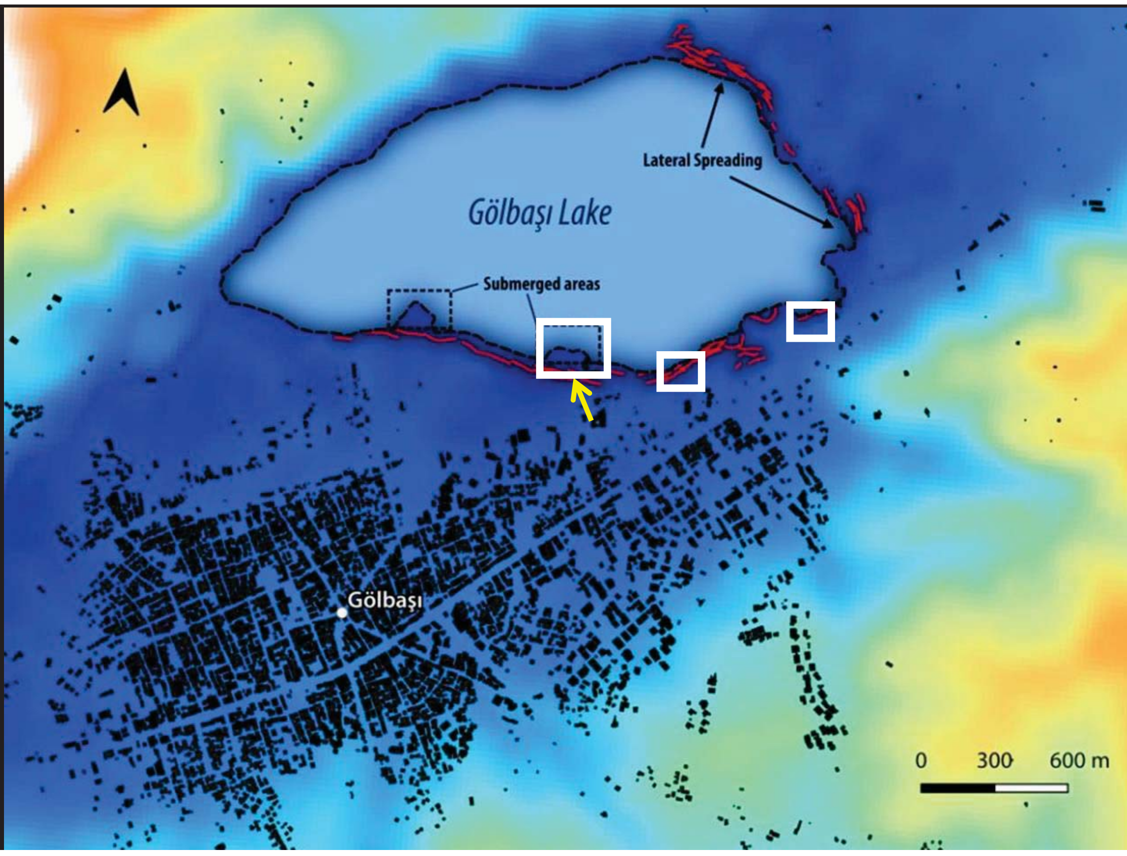

Location: Emiroğlu
Photo: Kemal Önder Çetin
Location: Sekeroba
Photo: Ozdemir Alpay

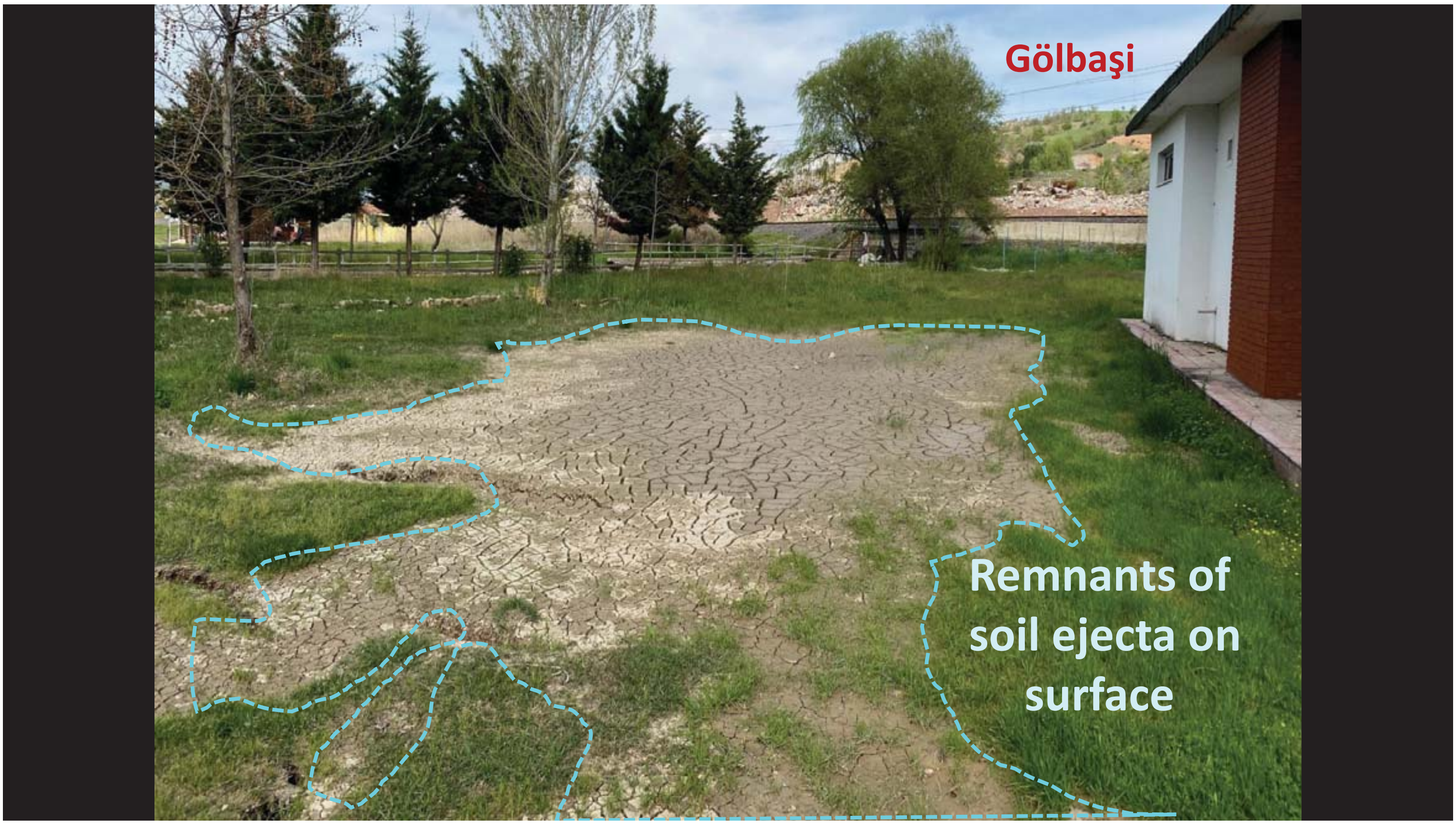

Soil Liquefaction in Syria
Source: http://yourweather.co.uk.

Extensive Soil Liquefaction Cases

- Gölbaşi
- Iskenderun

SOIL LIQUEFACTION AND LATERAL SPREADING IN GÖLBAŞI

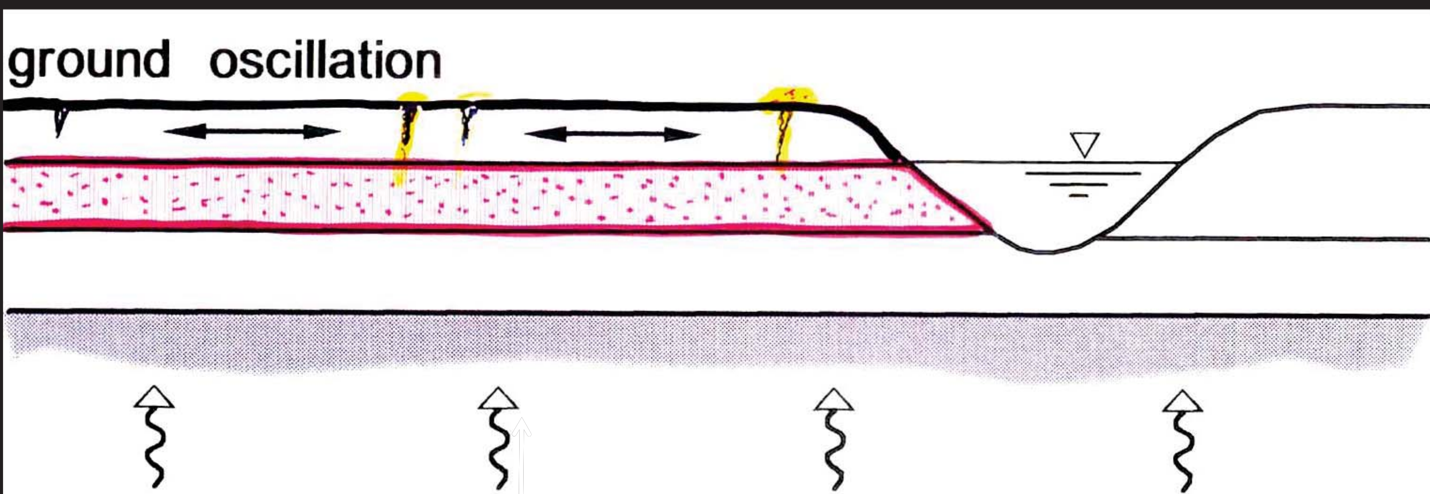



Gölbaşi
Remnants of soil ejecta on surface

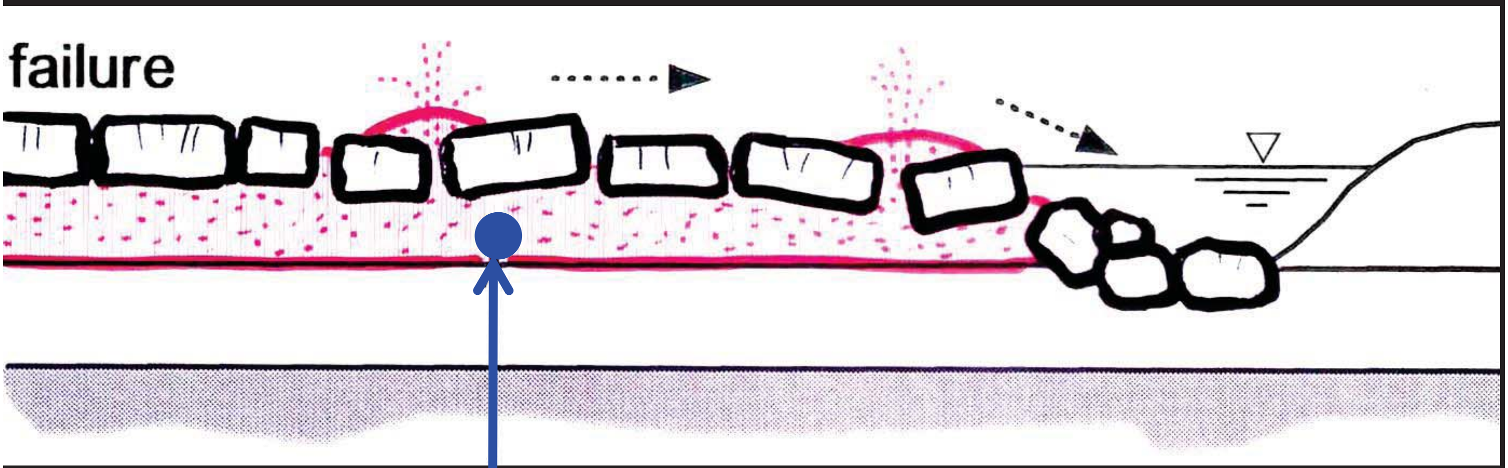

Gölbaşi
Details of the soil ejecta

Lateral spreading near Gölbaşi Lake

Mechanism of Lateral spreading

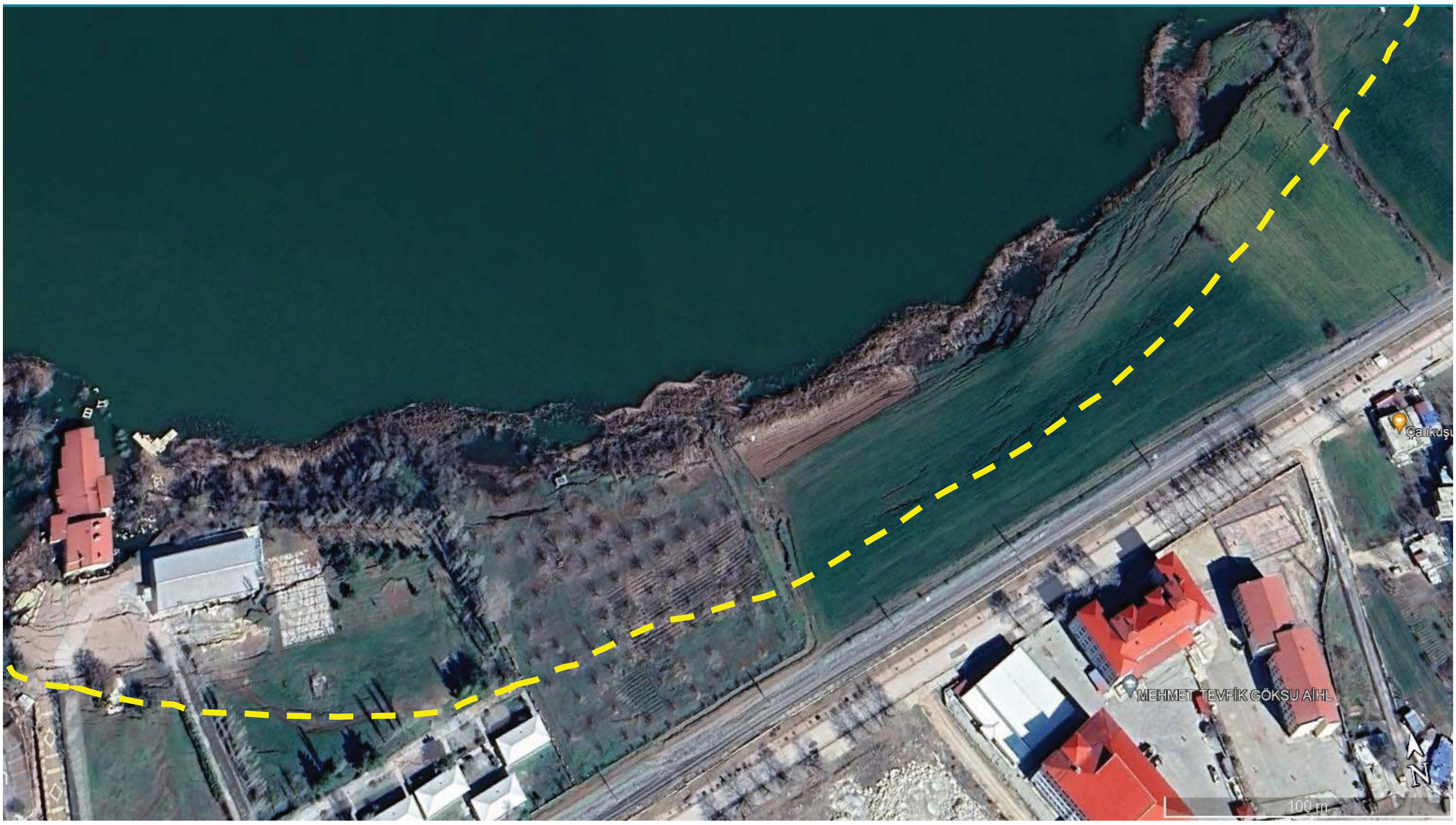


Mechanism of Lateral spreading

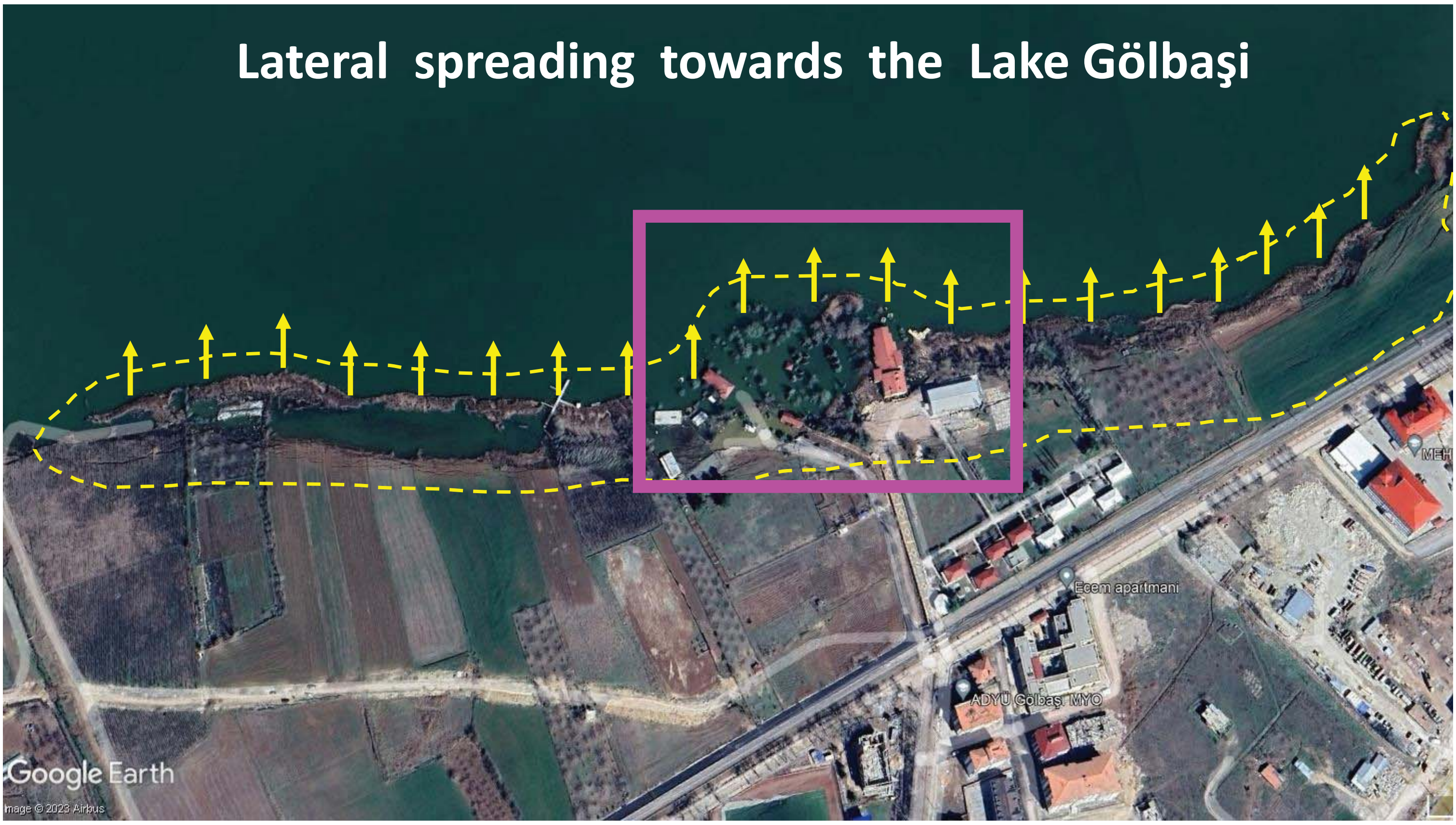


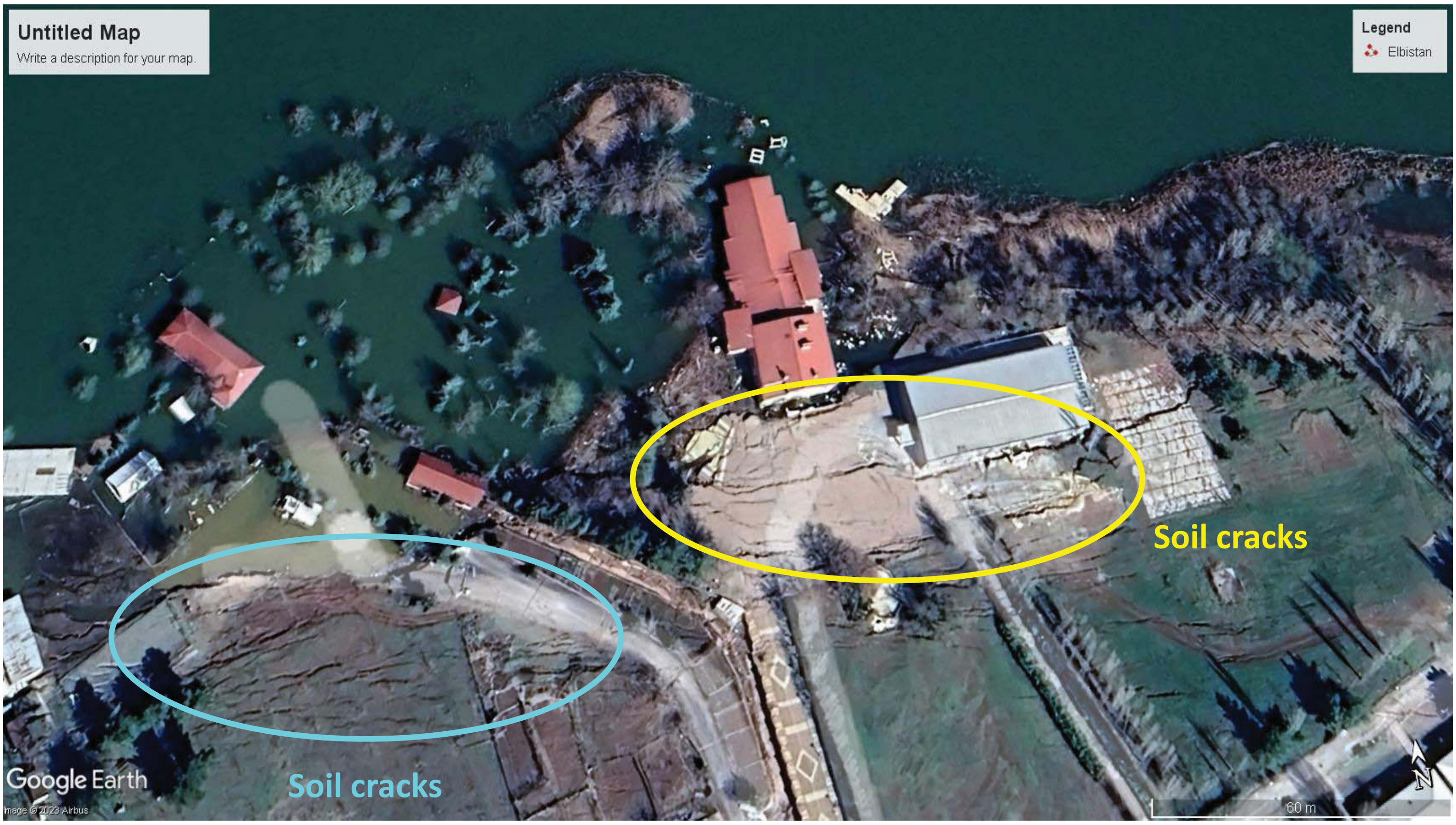
Soil liquefaction $\Delta u \rightarrow \bar{\sigma}_{vo}$

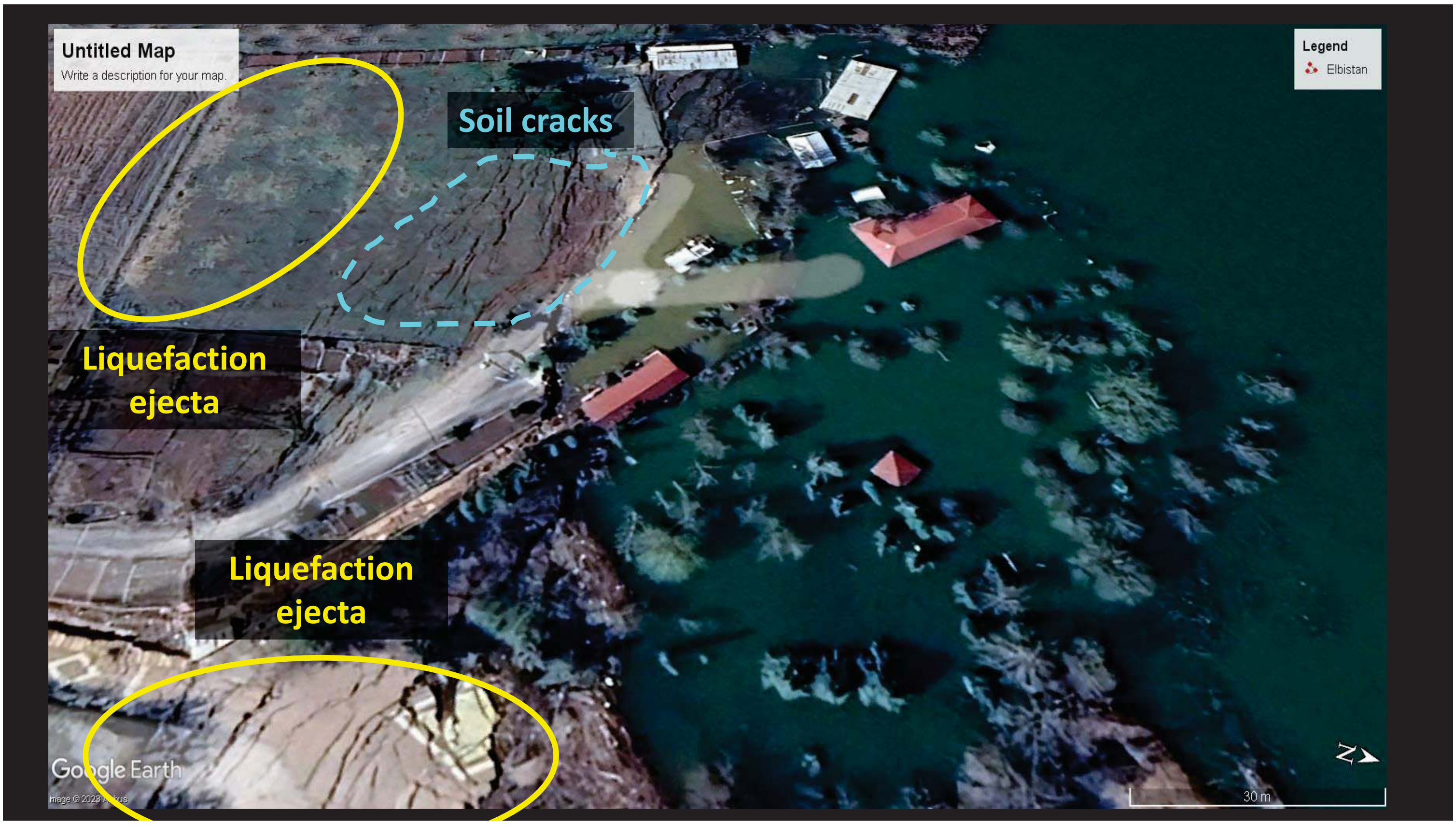
Lateral spreading towards the Lake Gölbaşi


MEHMET TEVFİK GÖKSU AİHL
100 m

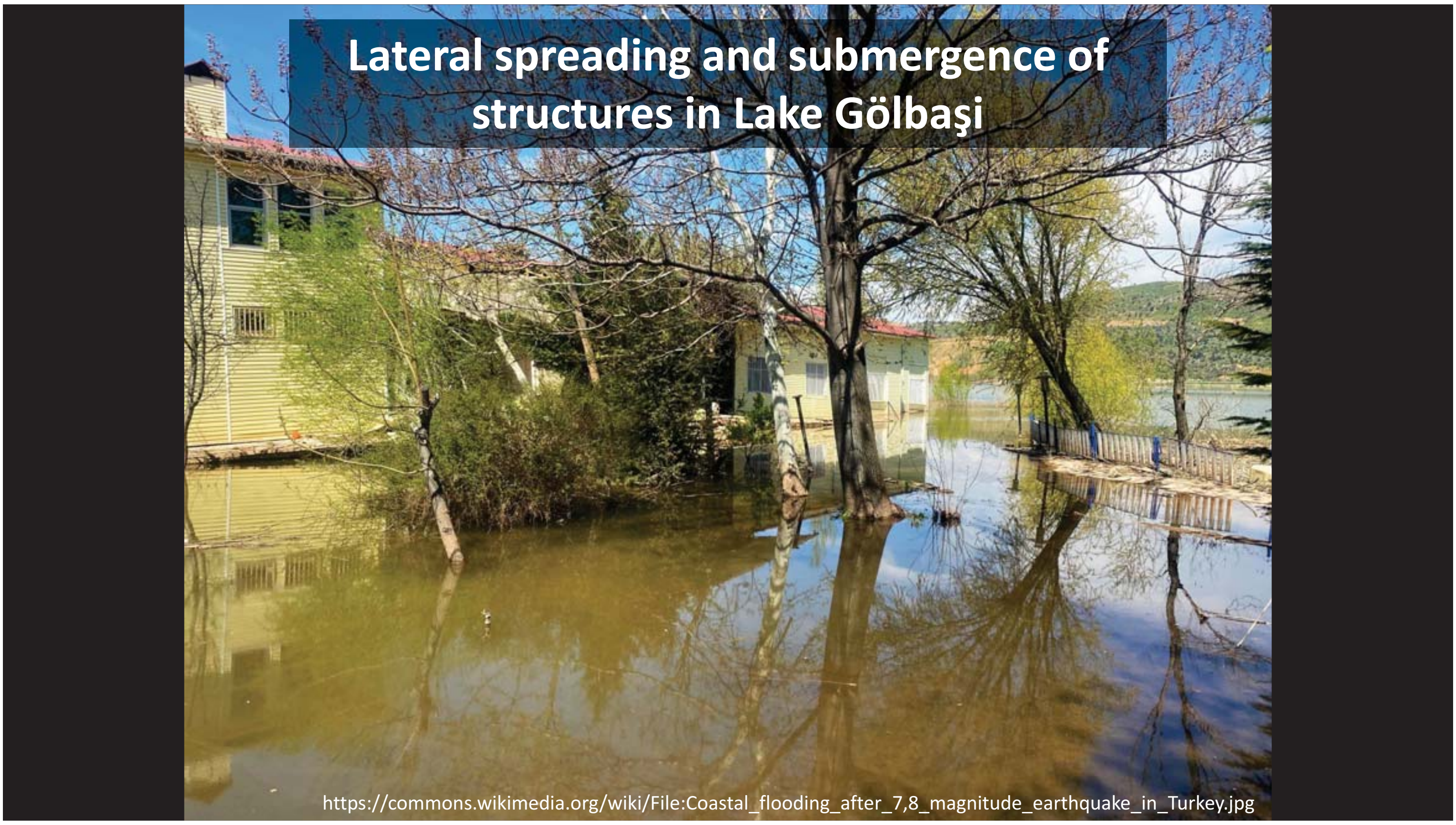

Surface cracks
Google Earth
Image © 2023 Airbus


Lateral spreading towards the Lake Gölbaşi
Ecem apartmanı
ADYU Gölbaşı MYO
Google Earth
Image © 2023 Airbus


Untitled Map
Write a description for your map.
Legend
Elbistan
Soil cracks
Soil cracks
Google Earth
Image © 2023 Airbus
60 m


Untitled Map
Write a description for your map.
Soil cracks
Liquefaction ejecta
Liquefaction ejecta
Legend
Elbistan
Google Earth
30 m


Total submergence of the building due to lateral spreading
Photo courtesy of Prof. Anastasopoulos


Lateral spreading and submergence of structures in Lake Gölbaşi
https://commons.wikimedia.org/wiki/File:Coastal_flooding_after_7,8_magnitude_earthquake_in_Turkey.jpg

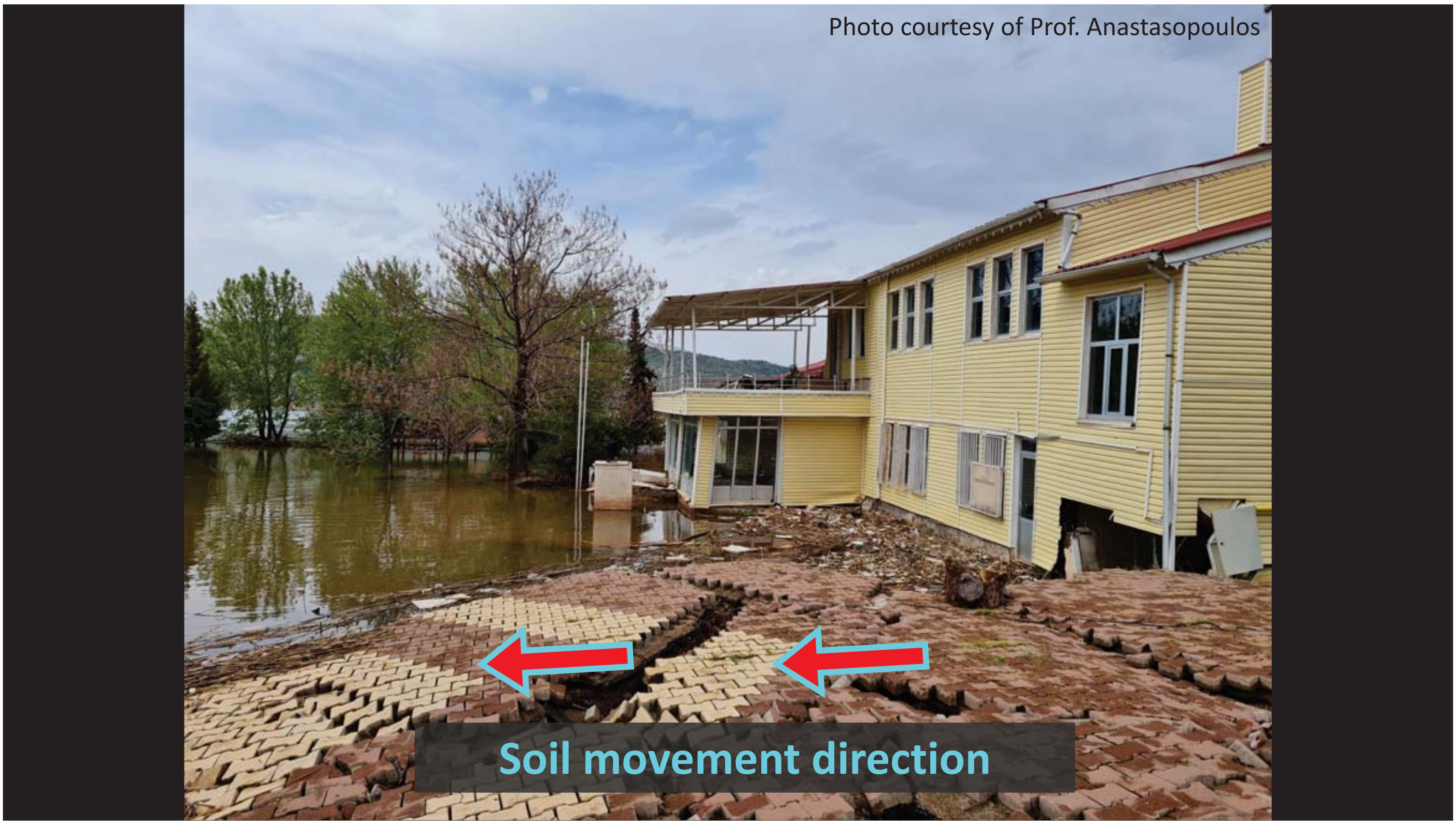


Photo courtesy of Prof. Anastasopoulos
Soil movement direction

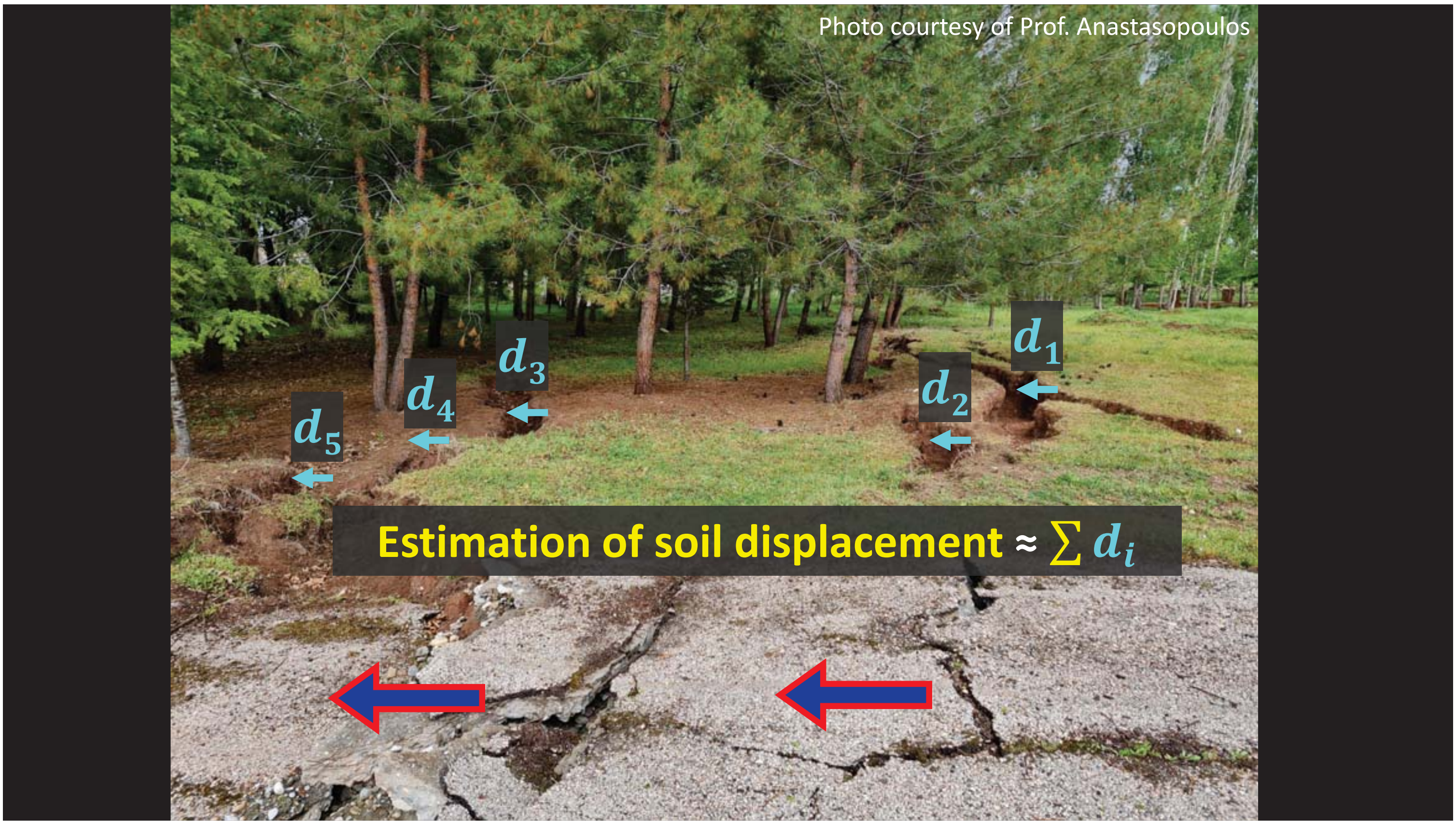


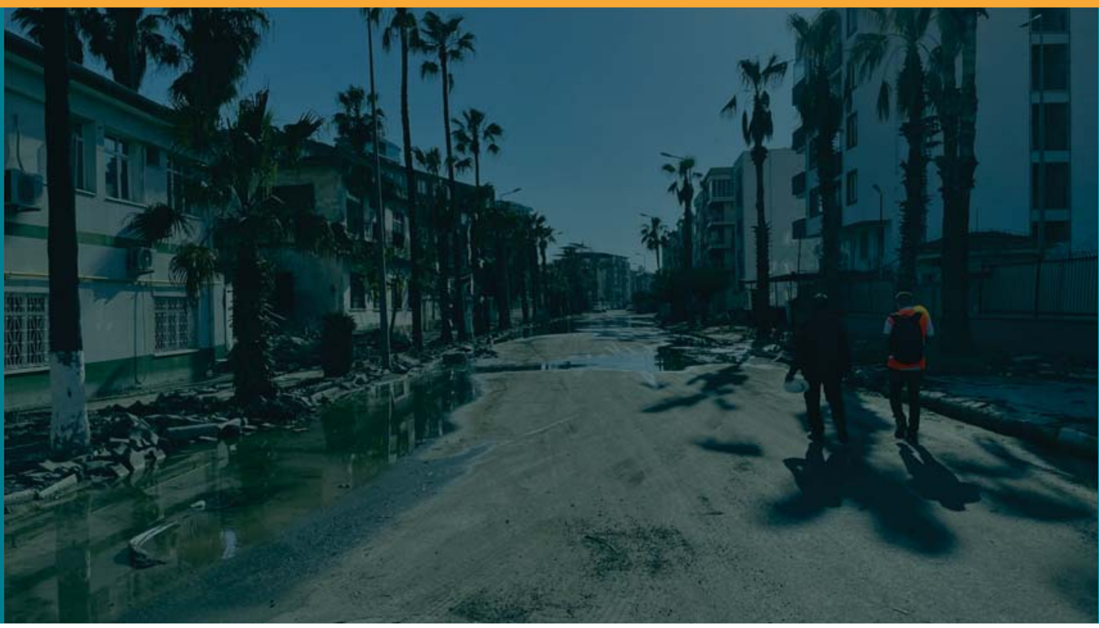
Photo courtesy of Prof. Anastasopoulos
d_1
d_2
d_3
d_4
d_5
Estimation of soil displacement ≈ $\sum d_i$


1.2 m
Photo courtesy of Prof. Anastasopoulos

Extensive Soil Liquefaction Cases

- Gölbaşi
- Iskenderun

SOIL LIQUEFACTION AND SOIL SUBSIDENCE IN ISKENDERUN

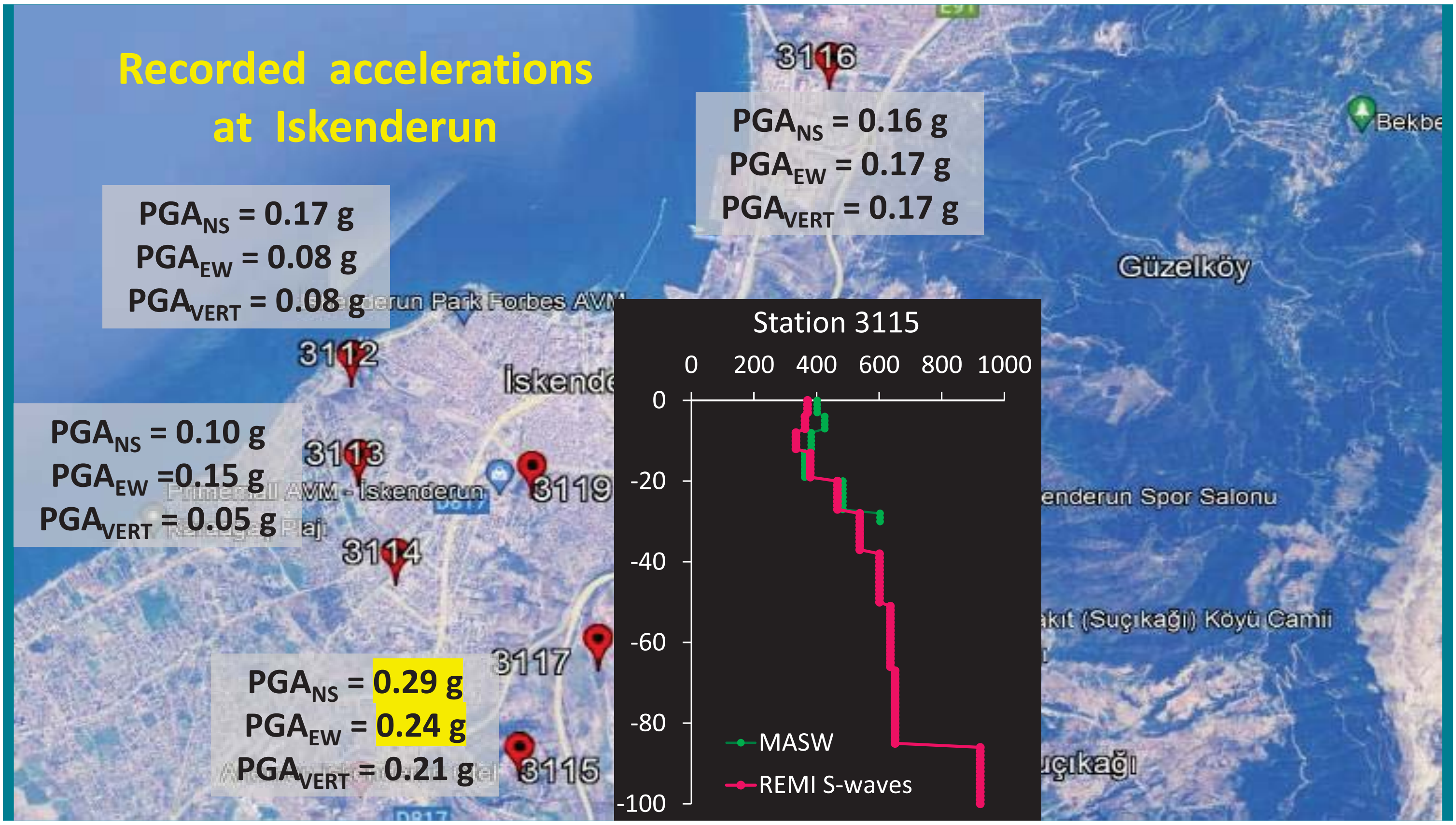


Extensive soil liquefaction along the whole Iskenderun Port

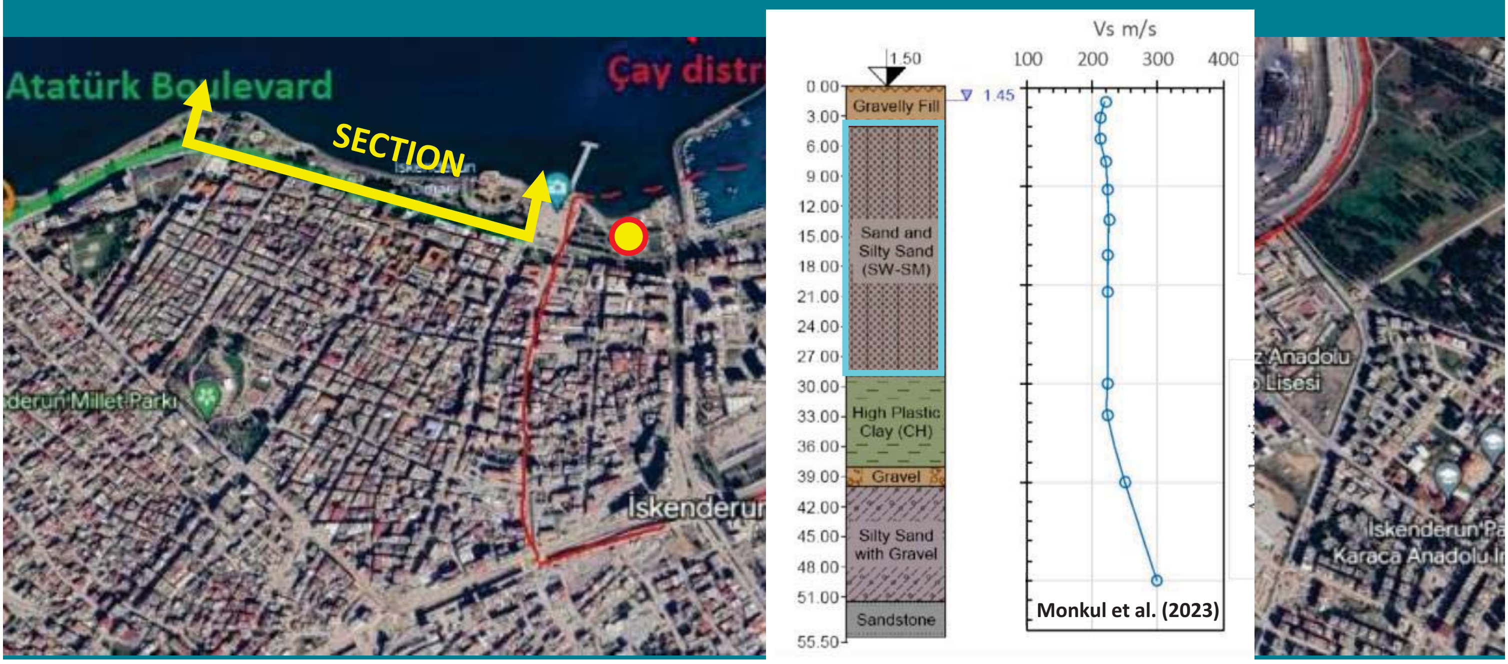
Source: Anatolu Agency


Immediately after the quake: sinking of roads


Immediately after the quake: sinking of roads



Extensive soil liquefaction along the whole Iskenderun Port



SPT soil-profile along the section

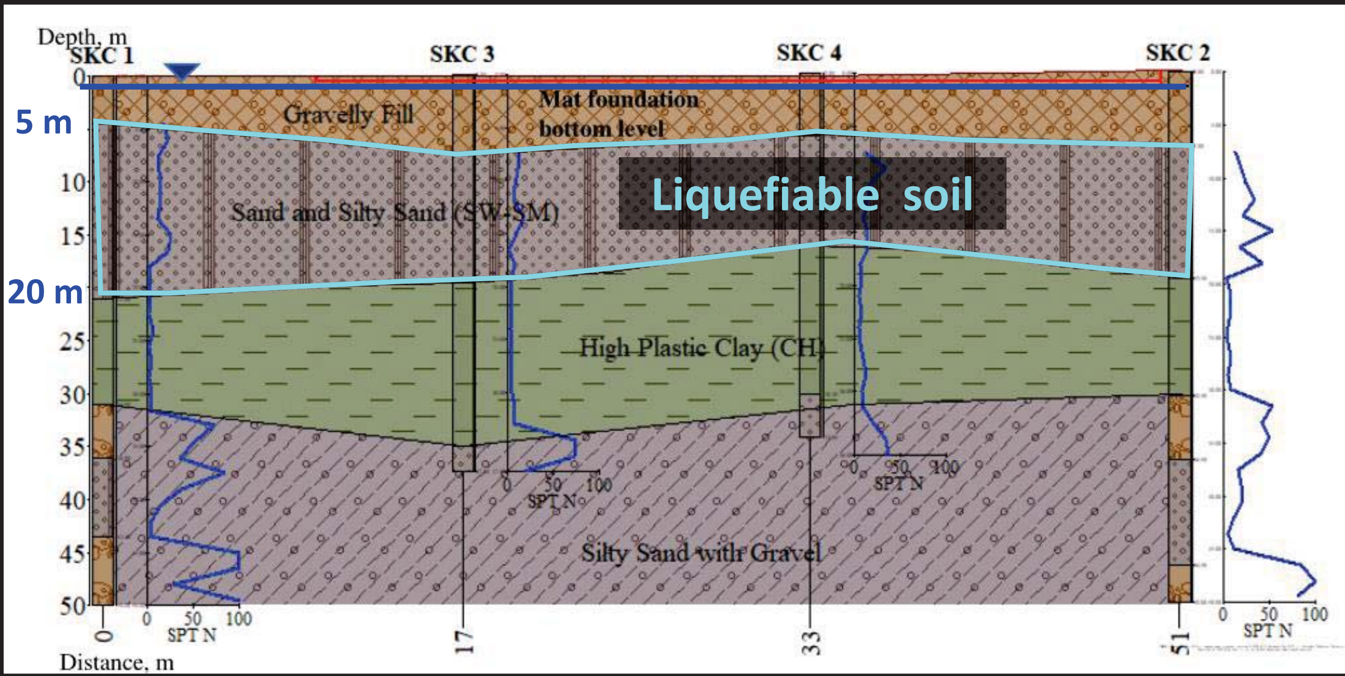
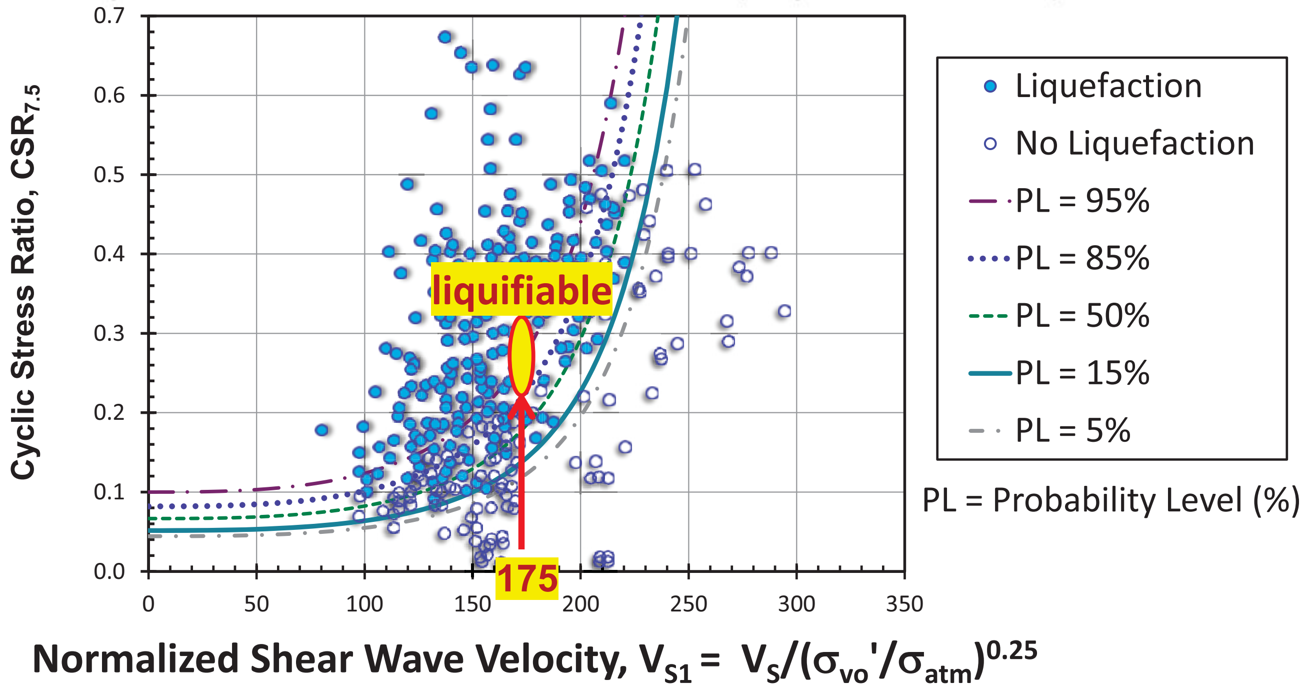
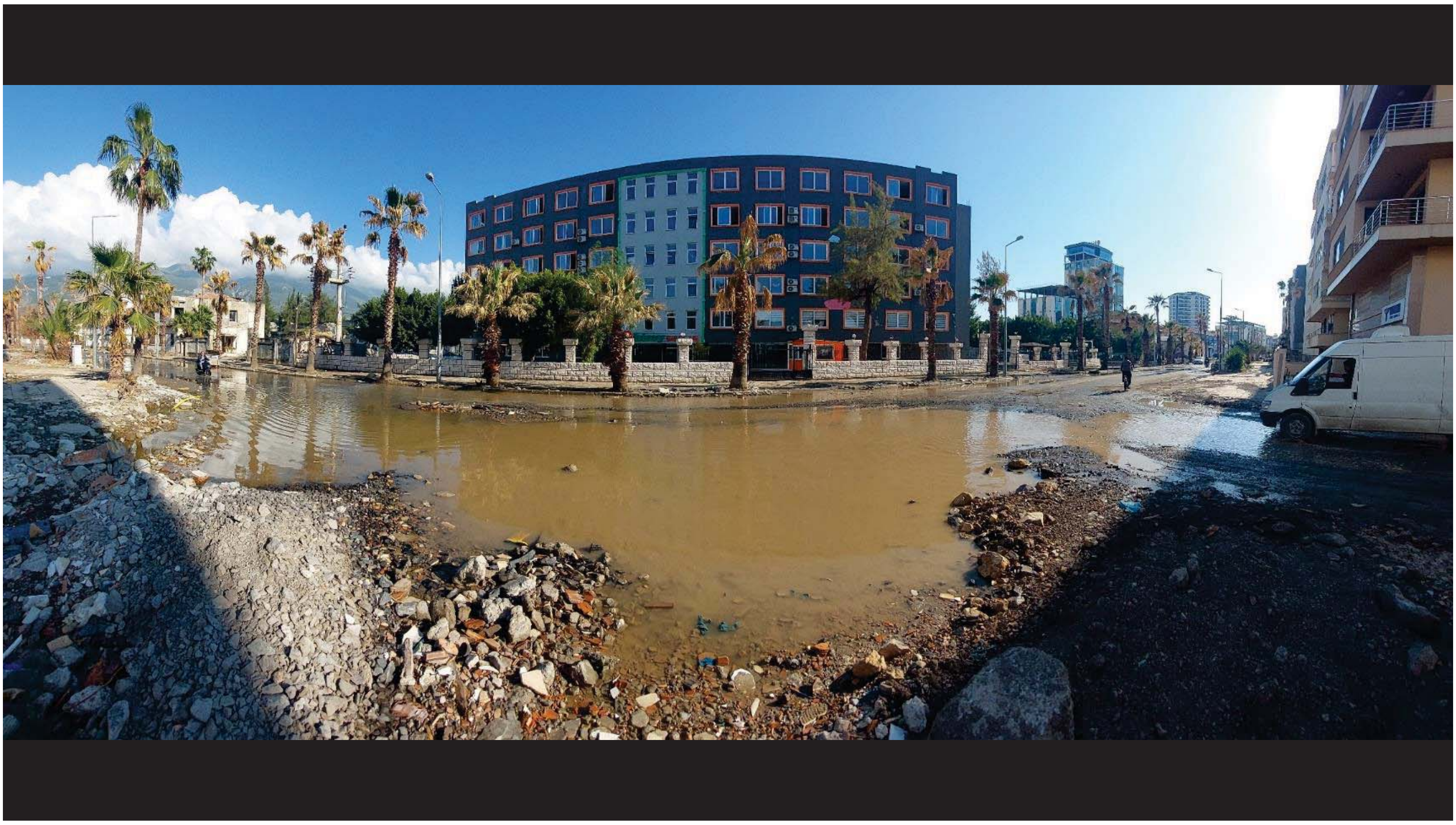


Figure: Monkul et al. (2023)

Evaluate Liquefaction Potential by Shear Wave Velocity

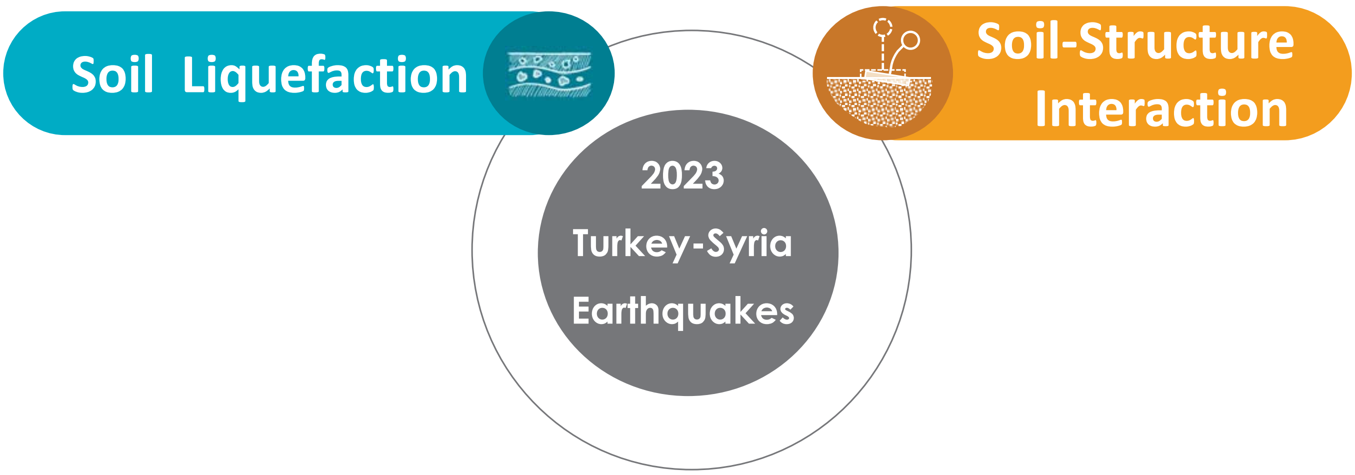
Updated with 301 New Case Studies (Kayen et al. 2013)






HEDEF
Yeminli Mali Müşavirlik
Limited Şirketi
İbrahim YARAN
ASLAN
APT. NO: 22
22
20 cm

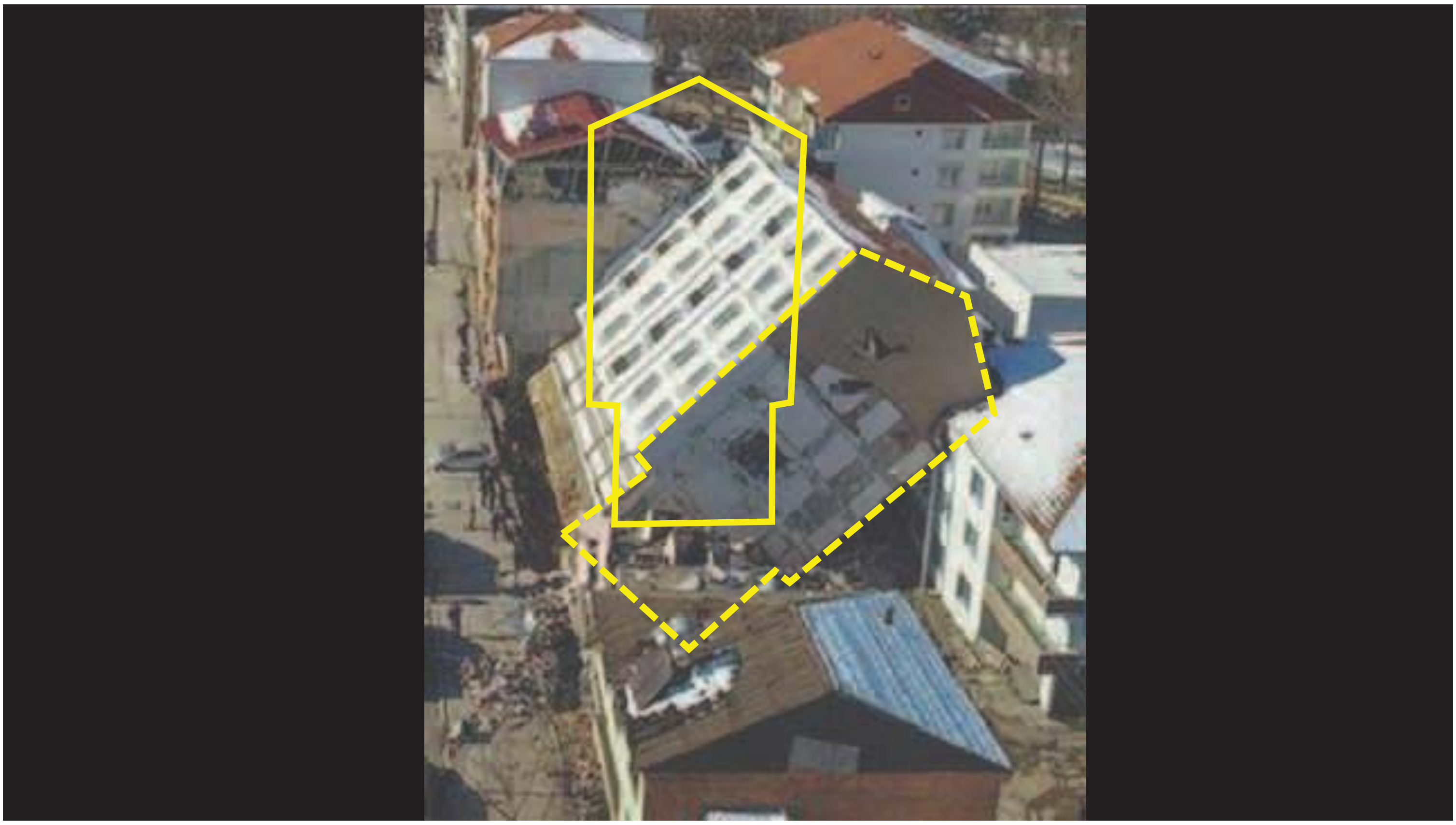
Glance over

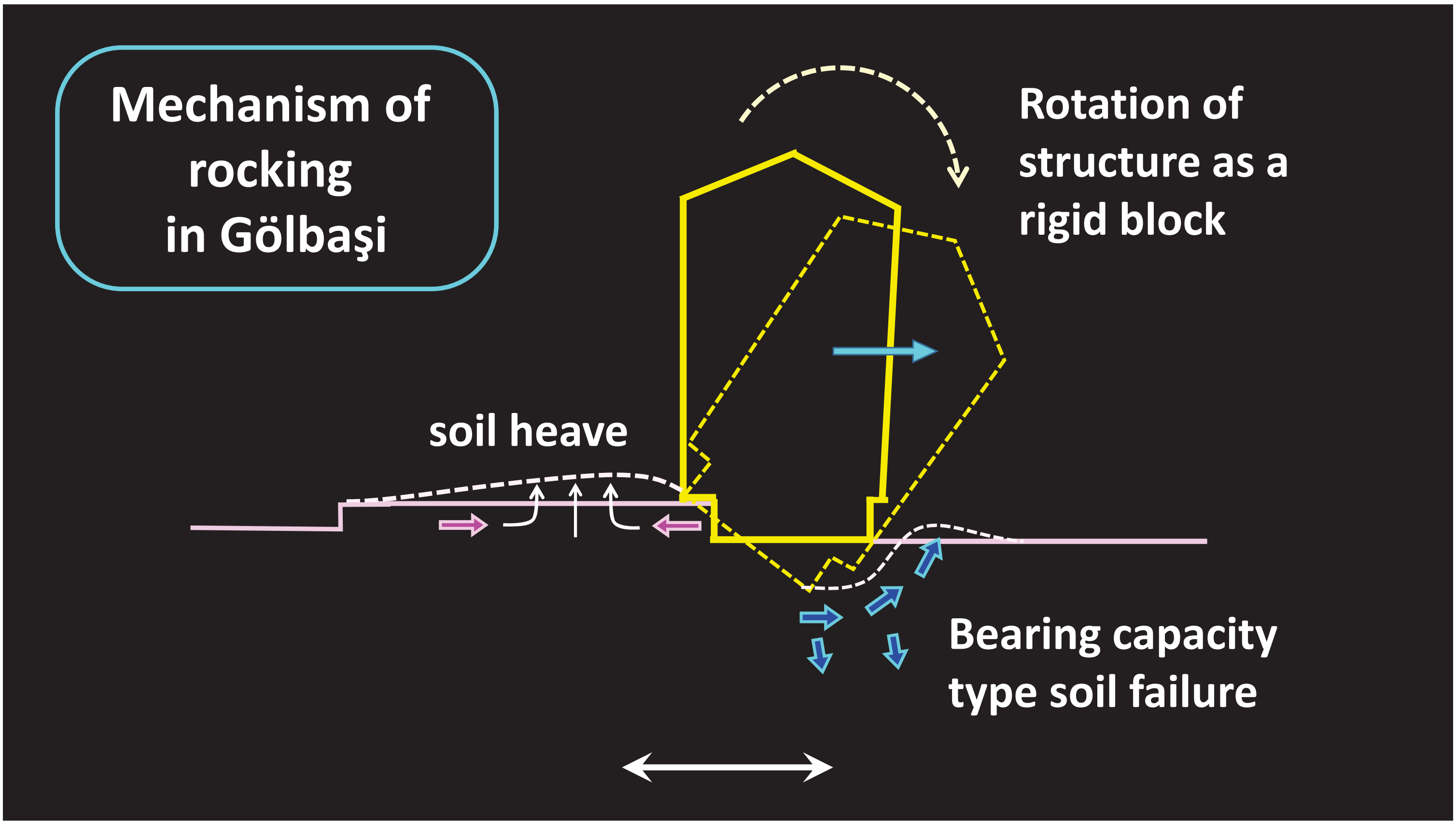


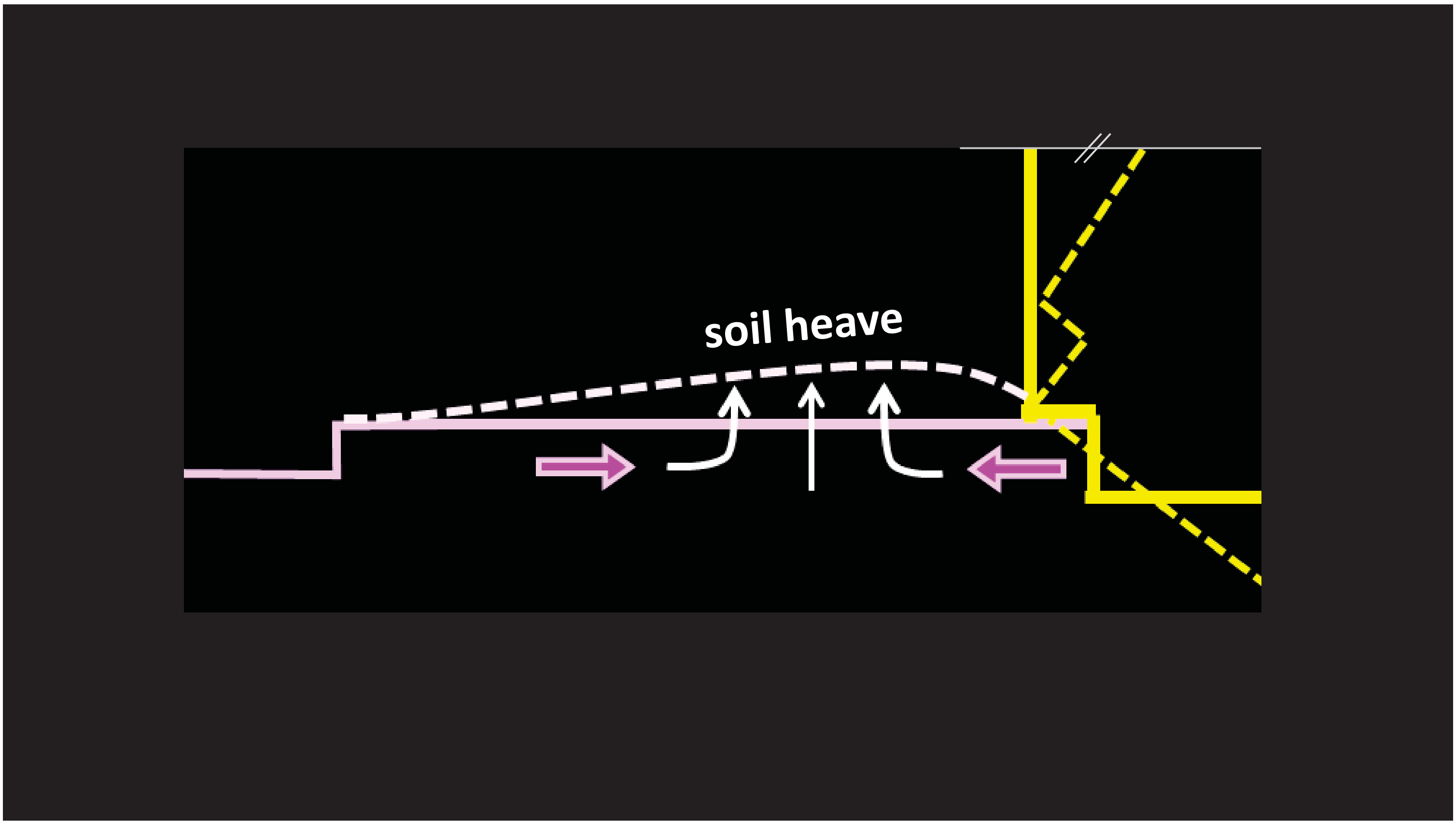
ROCKING OF STRUCTURES IN GÖLBAŞI

A toppled building in Gölbaşi


No structural damage
Flat slab foundation: without a crack!

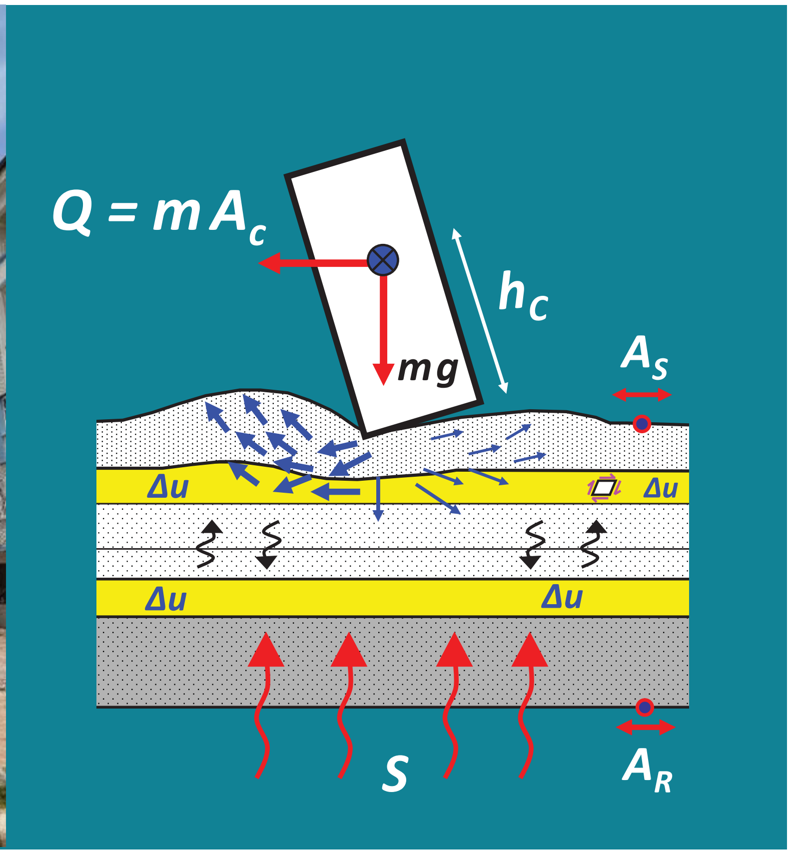



Mechanism of rocking in Gölbaşi
Rotation of structure as a rigid block
soil heave
Bearing capacity type soil failure


soil heave


Rocking of whole buildings


DOGAN APT.


Q = m A_C
h_C
mg
A_S
Δu
Δu
Δu
Δu
S
A_R


Gölbaşi


22o
Gölbaşi


Gölbaşi
What happened?? broken story ??


Gölbaşi

This **rocking type** of structural response is **not** unusual

It happened before:

In **1999 Izmit M7.4** earthquake in **Adapazari**

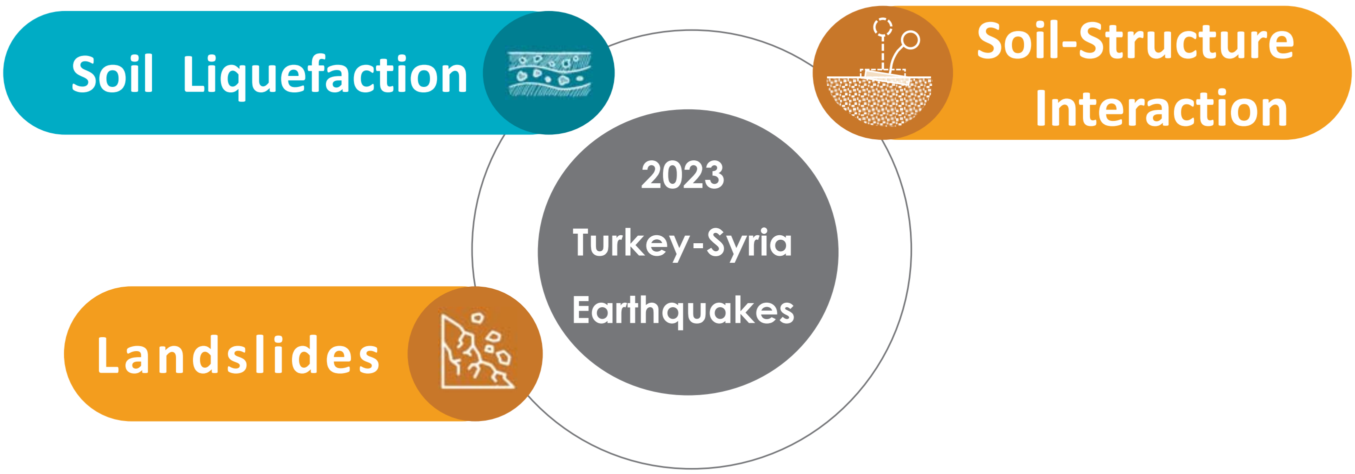
SOIL-STRUCTURE INTERACTION

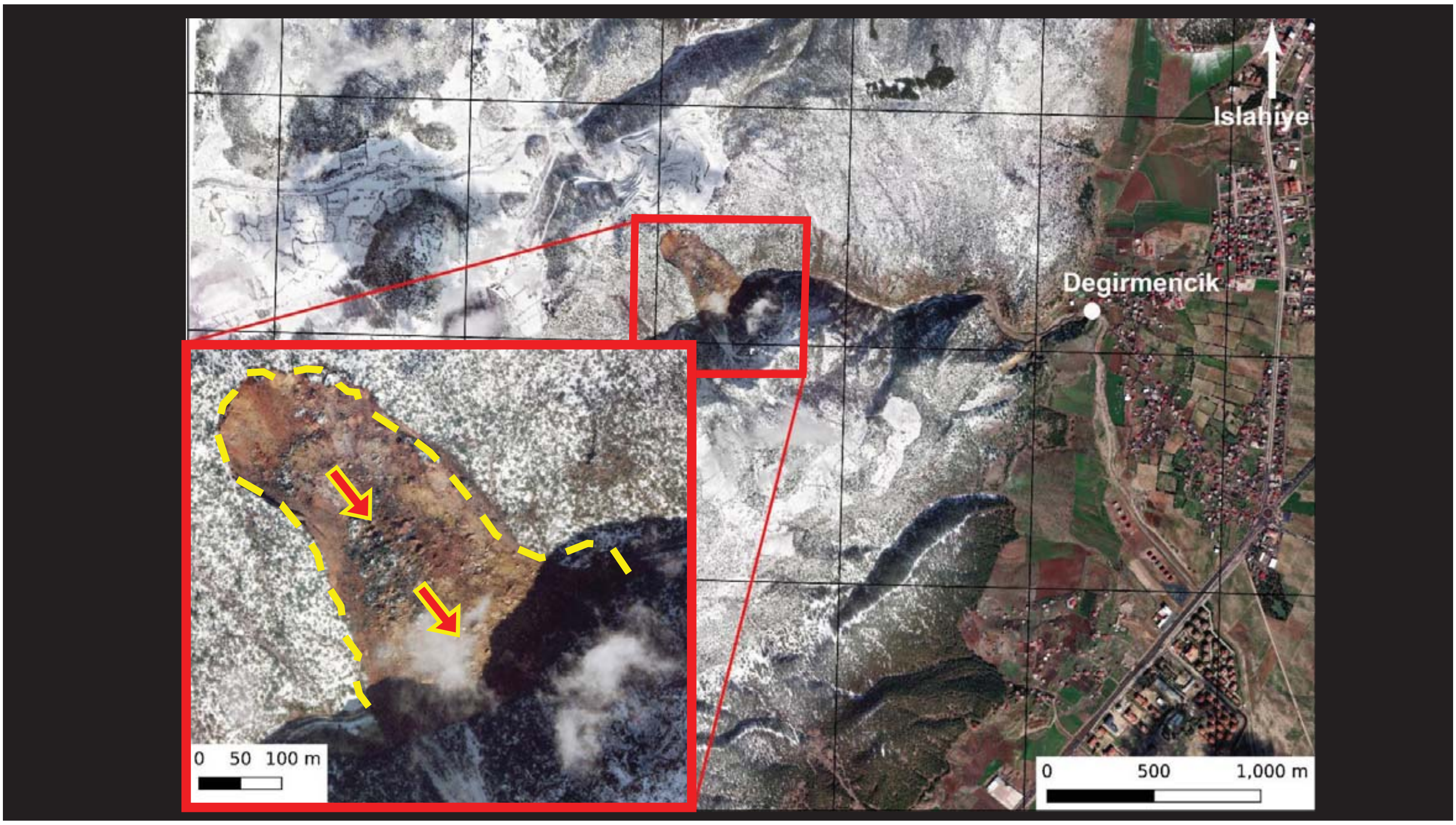
Bridge abutment rotation due to seismic induced foundation failure

Photo: **METU Report**

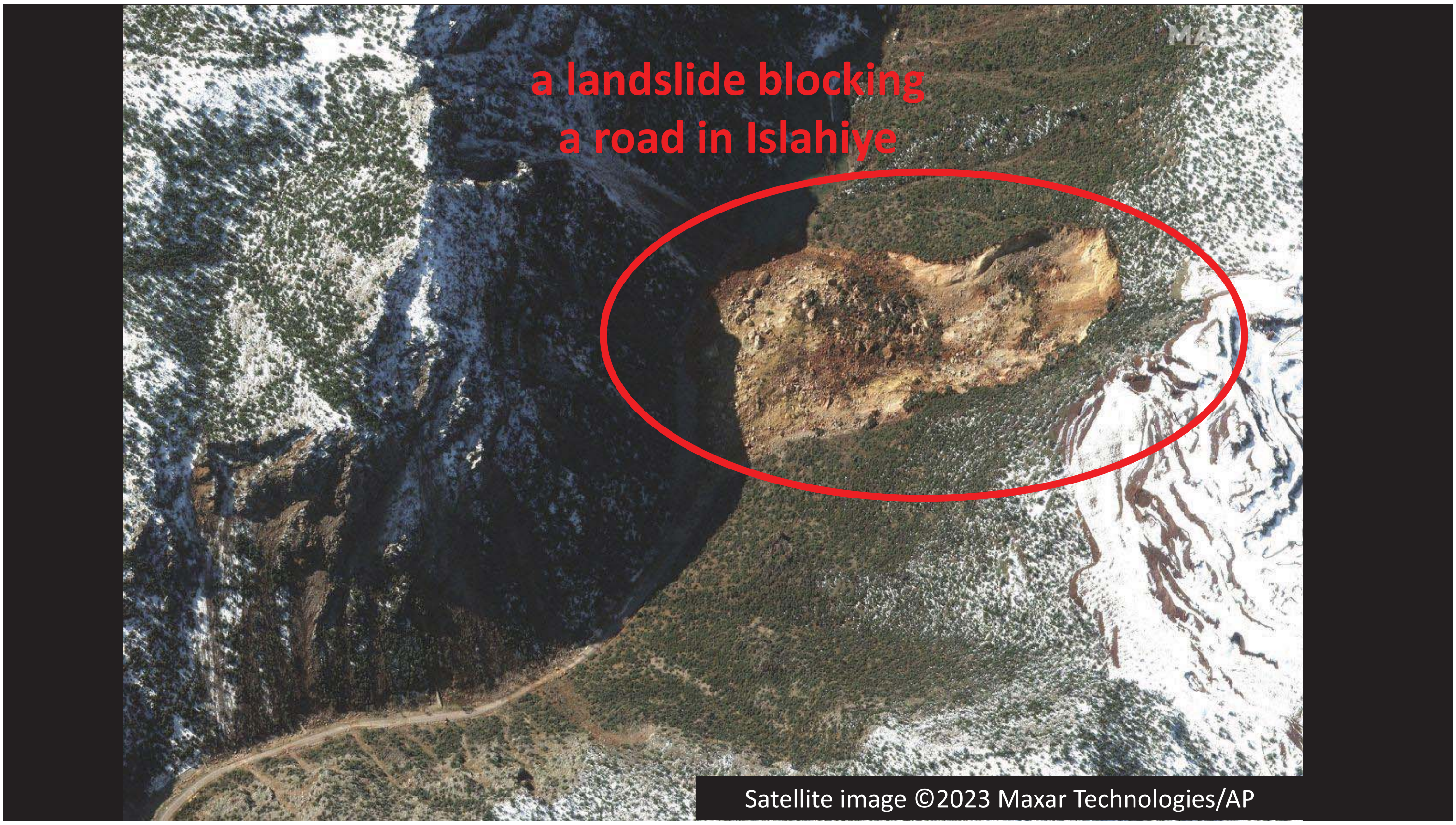
Photo: **METU Report**

Glance over

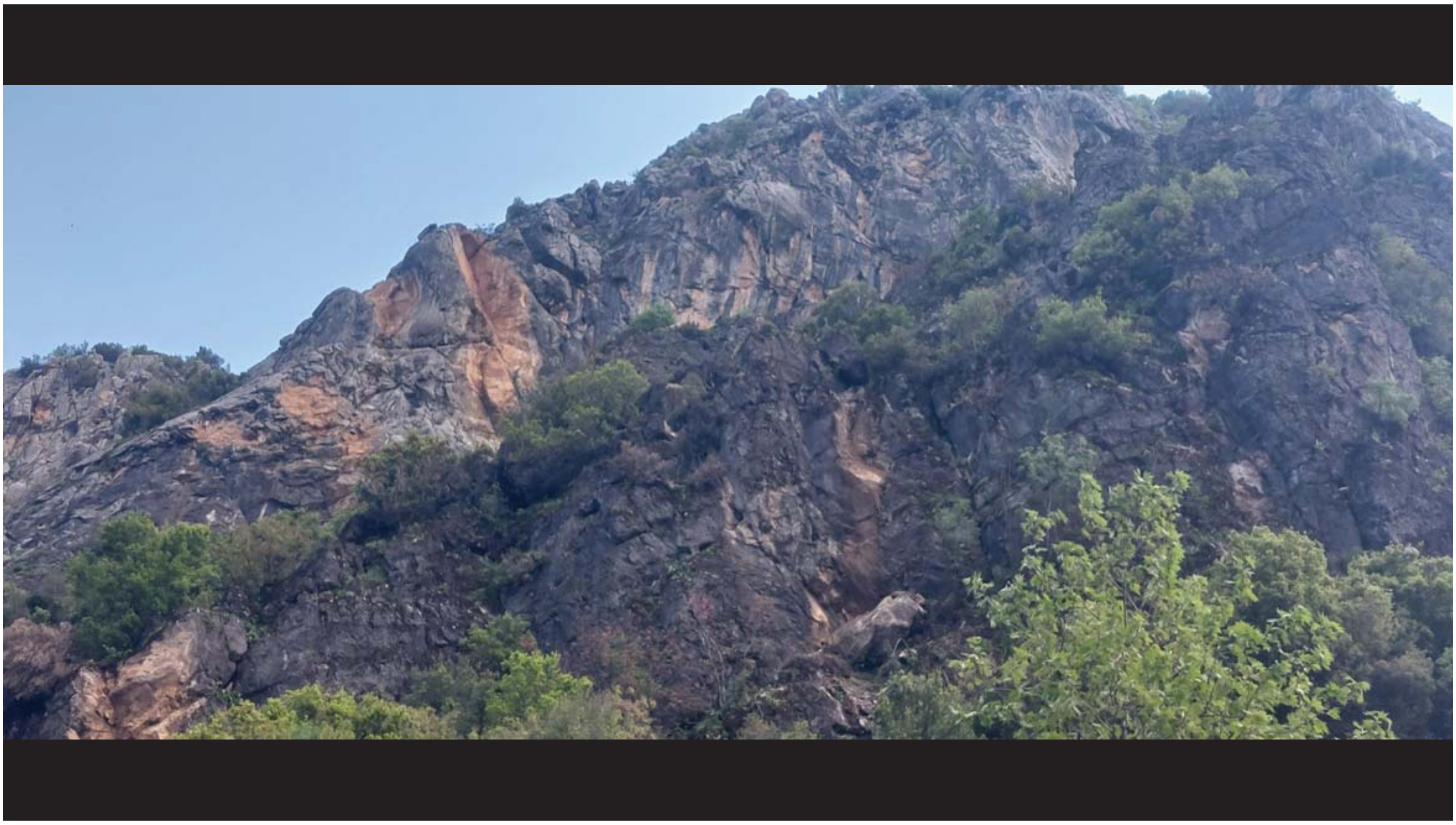


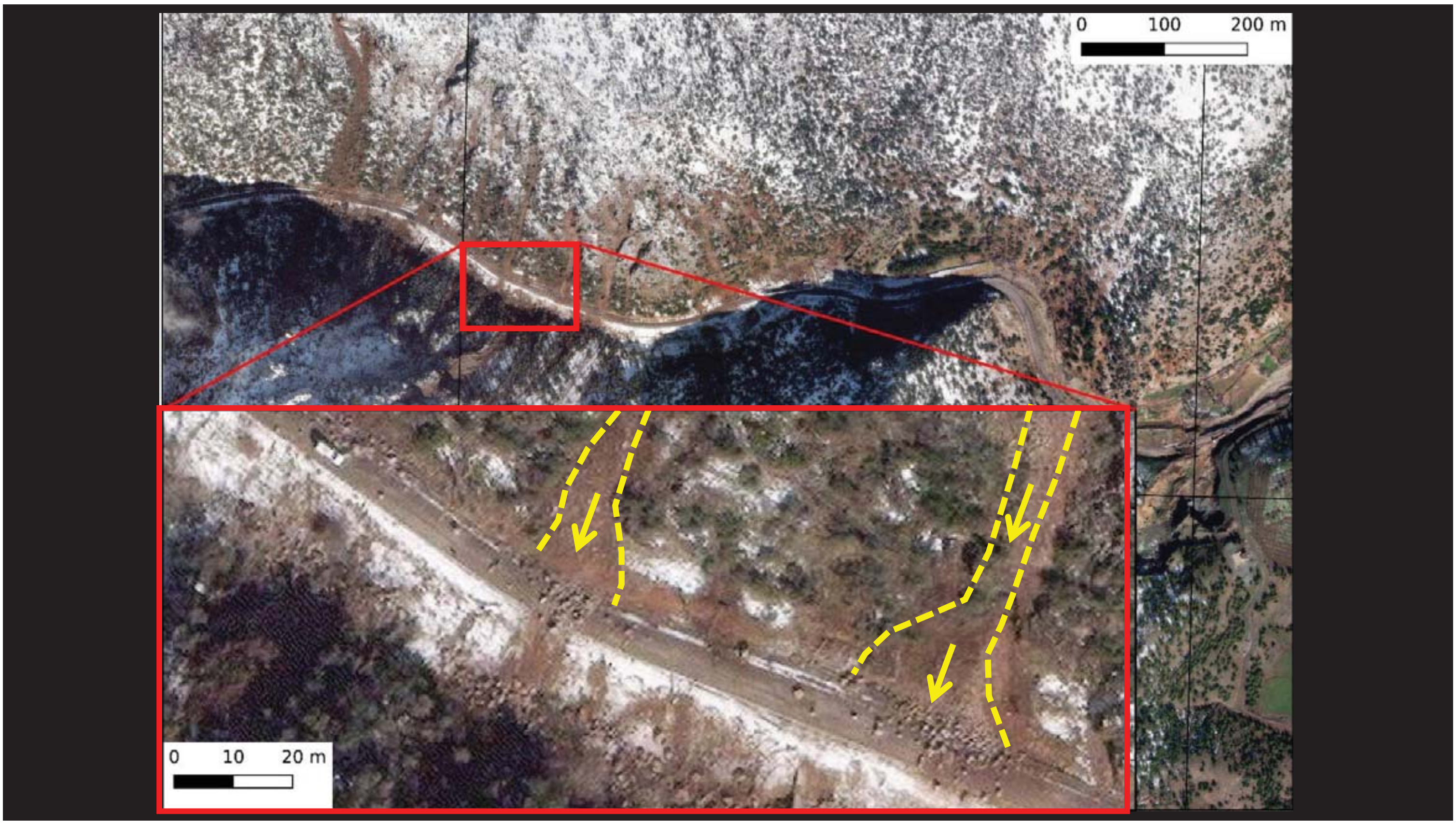


Islahiye
Degirmencik
0 50 100 m
0 500 1,000 m

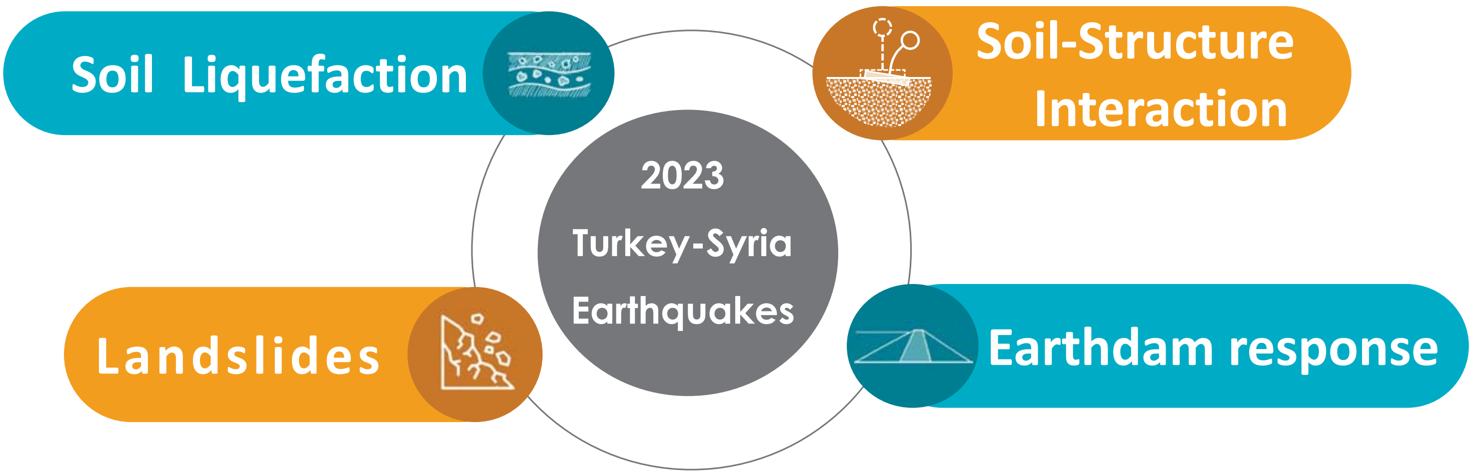


a landslide blocking
a road in Islahiye
Satellite image ©2023 Maxar Technologies/AP




0 100 200 m
0 10 20 m

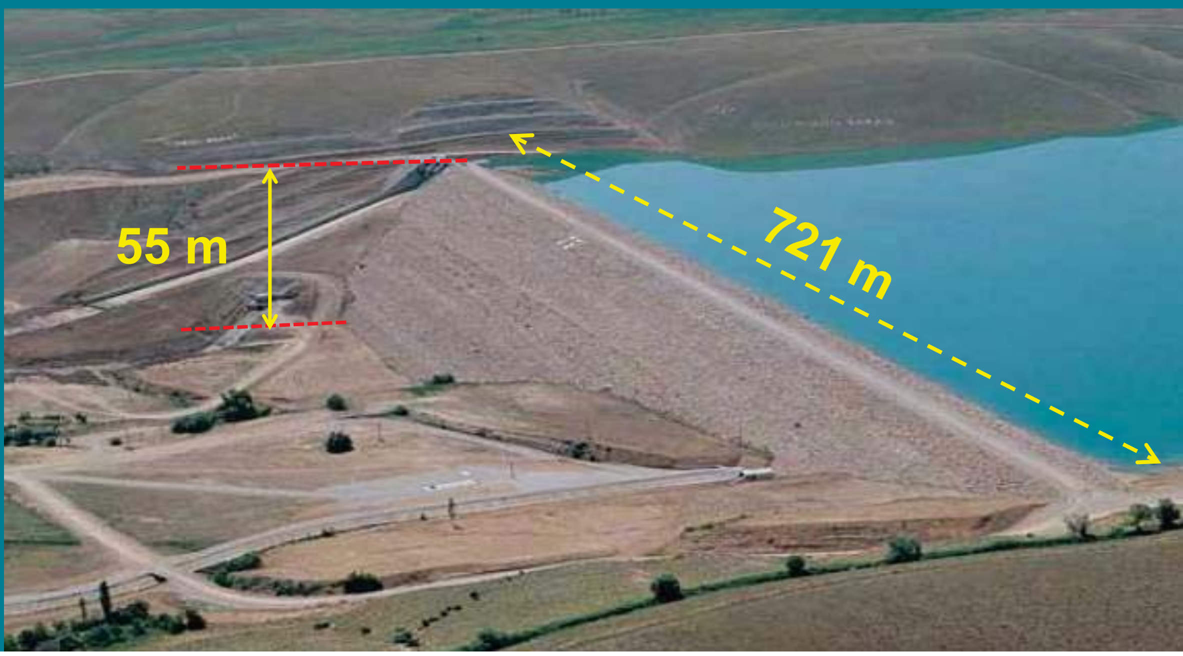
Glance over



Βλάβες σε Χωμάτινα Φράγματα

Sultansuyu στην Malatya

ΠΡΙΝ τους σεισμούς



Βλάβες σε Χωμάτινα Φράγματα

Sultansuyu στην Malatya

ΜΕΤΑ τους σεισμούς


Φράγμα Ariklikas 12 km από το Ρήγμα (κοντά στο Nurdagi)
Ύψος 10 m
Μήκος στην Στέψη 200 m


≈ 4 m πάχος
≈ 20 m μήκος

Βλάβες σε Χωμάτινα Φράγματα

Maydanki

ΠΡΙΝ τους σεισμούς

Βλάβες σε Χωμάτινα Φράγματα

Maydanki

Άνοιξαν τον Υπερχειλιστή προκειμένου να καταβιβάσουν την στάθμη του ταμιευτήρα

Αποτέλεσμα : Πλημμύρισαν τα κοντινά χωριά στην Τουρκία και την Συρία

Thank you

Evi Riga

Researcher and Teaching faculty at the Department of Civil Engineering, Aristotle University of Thessaloniki, Greece

Verification of seismic risk models using observed damage from past earthquake events

Dr. Evi Riga is a researcher and teaching faculty at the Department of Civil Engineering, Aristotle University of Thessaloniki (AUTH), specializing in Geotechnical Earthquake Engineering and Engineering Seismology. She earned her Civil Engineering diploma (2005) and MSc in Earthquake Engineering (2006) from AUTH, followed by a PhD (2015, with distinction) on "New elastic spectra, site amplification and aggravation factors for complex subsurface conditions, towards the improvement of Eurocode 8". Her research expertise covers local site effects and their incorporation into seismic hazard analyses and codes, soil and site characterization in geotechnical and earthquake engineering, seismic hazard and risk assessment at different scales (local, urban, national, European), seismic vulnerability of buildings, infrastructures and lifelines, as well as GIS-based applications. She has participated in 27 European and national research projects (e.g., SHARE, SERA, SERIES, SYNER-G, EPOS) contributing among others to seismic risk modeling, site effect characterization, and code-related developments. She is the author or co-author of more than 90 scientific publications in peer-reviewed journals, book chapters, and conference proceedings, with more than 900 citations and an h-index of 13 (Scopus). Alongside her research, Dr, Riga has extensive teaching experience in undergraduate and postgraduate courses at AUTH. She also serves as a reviewer for international journals and an evaluator of research proposals and participates in numerous international working groups and committees related to earthquake engineering, including EFEHR and the ORFEUS strong motion committee.

Verification of seismic risk models using observed damage from past earthquake events

Evi Riga

Aristotle University of Thessaloniki, Greece

S. Apostolaki[1], K. Pitilakis[1], D. Pitilakis[1], S. Karahan[2], C. Gokceoglu[3], G. Tsinidis[4]

1. Aristotle University of Thessaloniki, Greece 2. Directorate General of Turkish Railway System,
3. Haceteppe University, Turkey 4. University of Thessaly, Greece

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

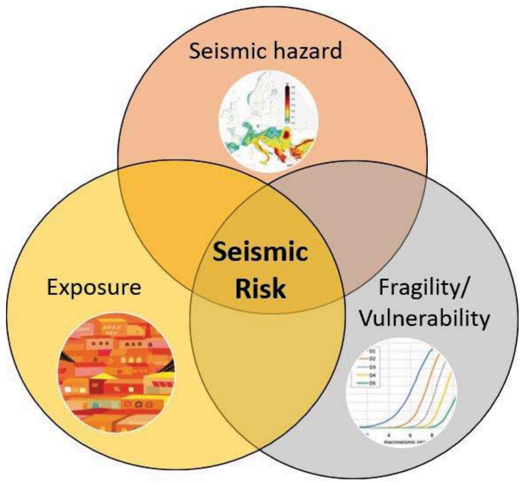
SDGEE

Thessaloniki, 9th October 2025

Societatea Română de Geotehnică şi Fundaţii

Introduction

- ❑ Seismic risk = Exposure * Hazard * Vulnerability
- ❑ Seismic risk models (ESRM20, GEM)
- ❑ Application in rapid damage assessment
- ❑ Verification (and calibration) through comparison of risk model estimates with observed (recorded) damage



Outline

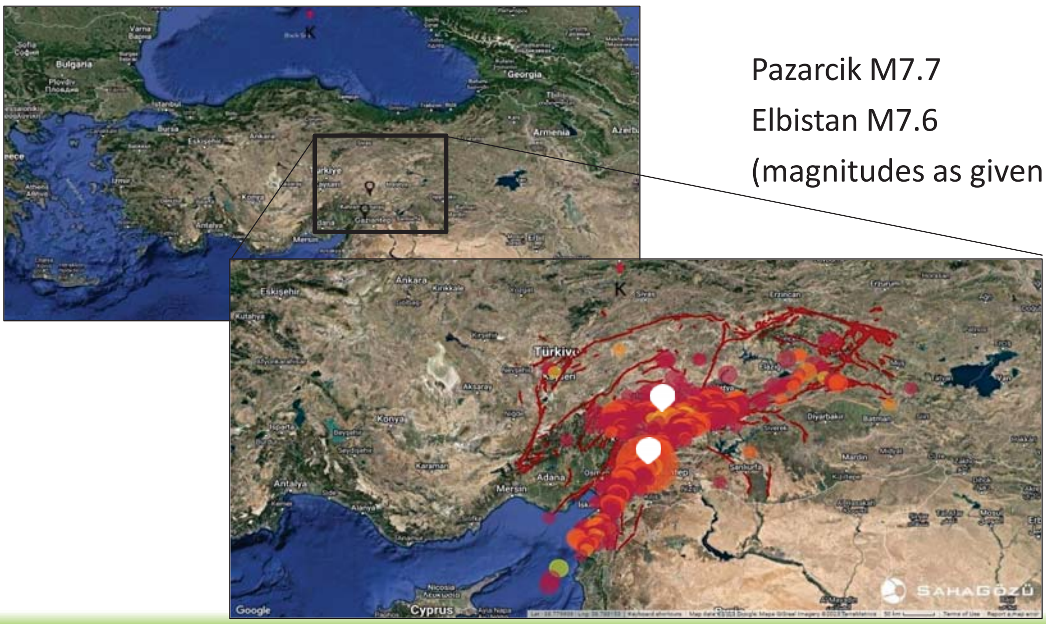
- The **Kahramanmaraş** earthquake sequence of February 6, 2023
- **Seismic performance of tunnels** and **verification** of available **seismic risk models for tunnels** using observed damage
- **Verification** of available **seismic risk models** for **residential buildings** in the framework of rapid damage assessment

Apostolaki S, Riga E, Pitilakis D (2024). Rapid Damage Assessment Effectiveness for the 2023 Kahramanmaraş Türkiye earthquake sequence, International Journal of Disaster Risk Reduction, Vol. 111, https://doi.org/10.1016/j.ijdrr.2024.104691.

Apostolaki S, Karahan S, Riga E, Tsinidis G, Gokceoglu C, Pitilakis K (2025). Seismic performance of tunnels and verification of available seismic risk models for the 2023 Kahramanmaraş earthquakes, Tunnelling and Underground Space Technology, Vol. 156, https://doi.org/10.1016/j.tust.2024.106185.

2nd Greek – Romanian Seminar: Lessons learned from Earthquakes and Geotechnical Failures 4

The Kahramanmaraş February 2023 earthquake sequence



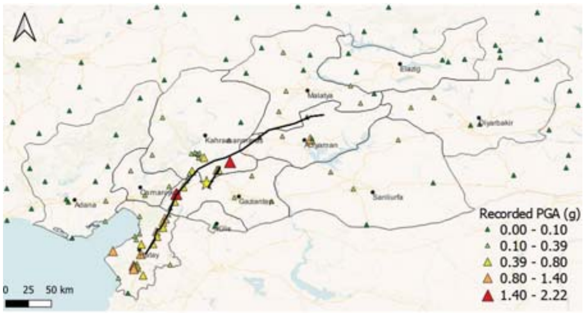
Pazarcik M7.7
Elbistan M7.6
(magnitudes as given by AFAD)

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ
Verification of seismic risk models using observed damage from past earthquake events. Riga et al.
Societatea Română de Geotehnică şi Fundaţii

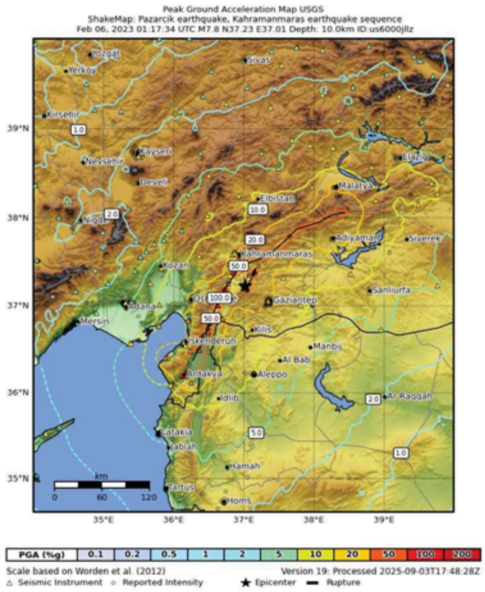
M7.7 Pazarcik earthquake

- M7.7 - depth 10km
- 06/02/2023, local time 4:17 am
- rupture of a 280 km long fault in the East Anatolia fault zone
- recorded by 379 accelerometric stations
- PGA values as high as 2.07g in Pazarcik, 1.38g in Hatay, 0.90g in Adiyaman and 0.37g in Kahramanmaraş (AFAD)

PGA recorded by AFAD



USGS ShakeMap

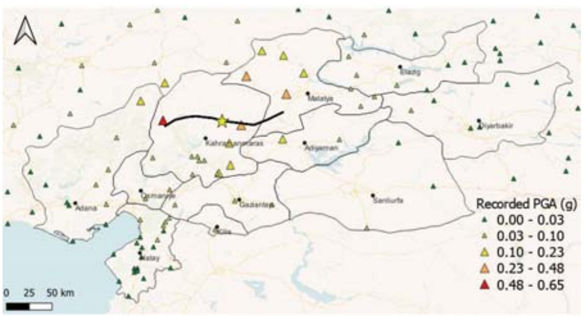


https://earthquake.usgs.gov/earthquakes/eventpage/us6000jllz/

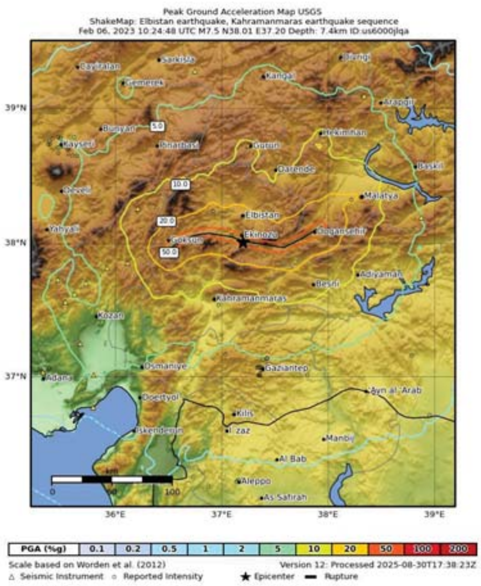
M7.6 Elbistan earthquake

- M7.6 - depth 10km
- 06/02/2023, local time 1:24 pm.
- rupture of the 140 km long Çardak fault
- recorded by 370 accelerometric stations
- largest recordings reported at Göksun (0.65g) and Nurhak stations (0.62g) (AFAD)

PGA recorded by AFAD



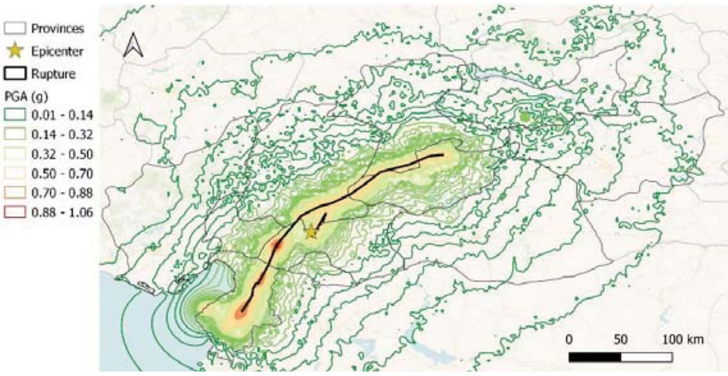
USGS ShakeMap



https://earthquake.usgs.gov/earthquakes/eventpage/us6000jlqa

USGS ShakeMaps

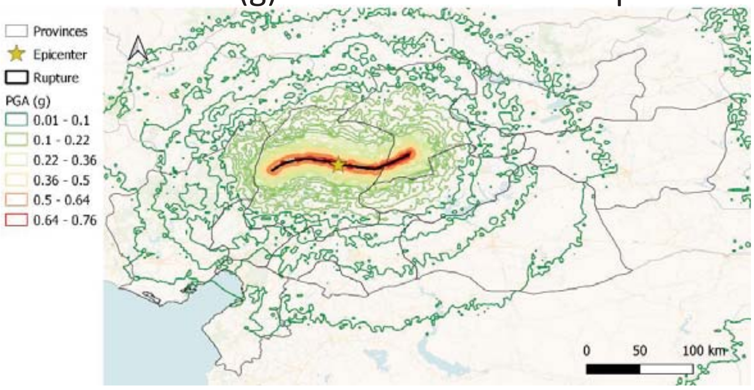
PGA (g) - M7.7 Pazarcik earthquake



v.017 – 14/04/23, 18:03

- First version of the ShakeMap: 10 minutes after the event
- 19 versions

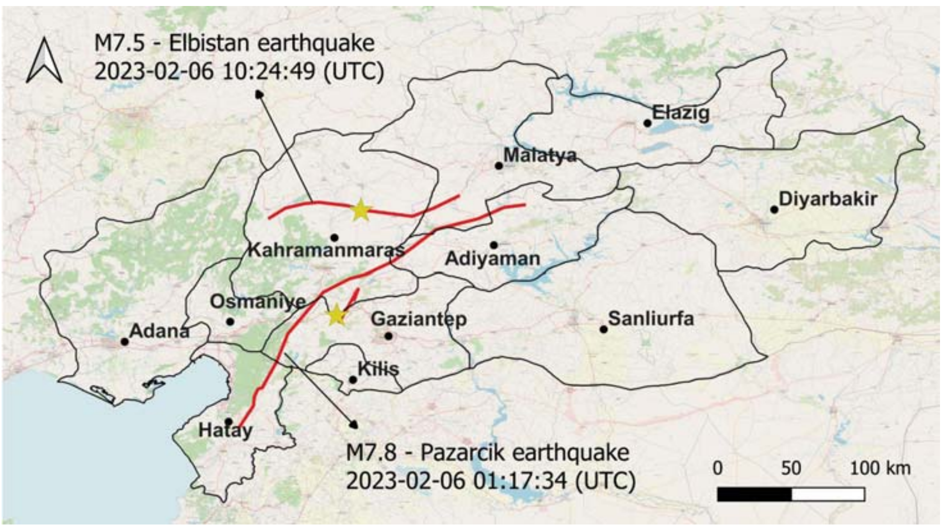
PGA (g) - M7.6 Elbistan earthquake



v.012 – 14/04/23, 12:32

- First version of the ShakeMap: 20 minutes after the event
- 12 versions

Affected area



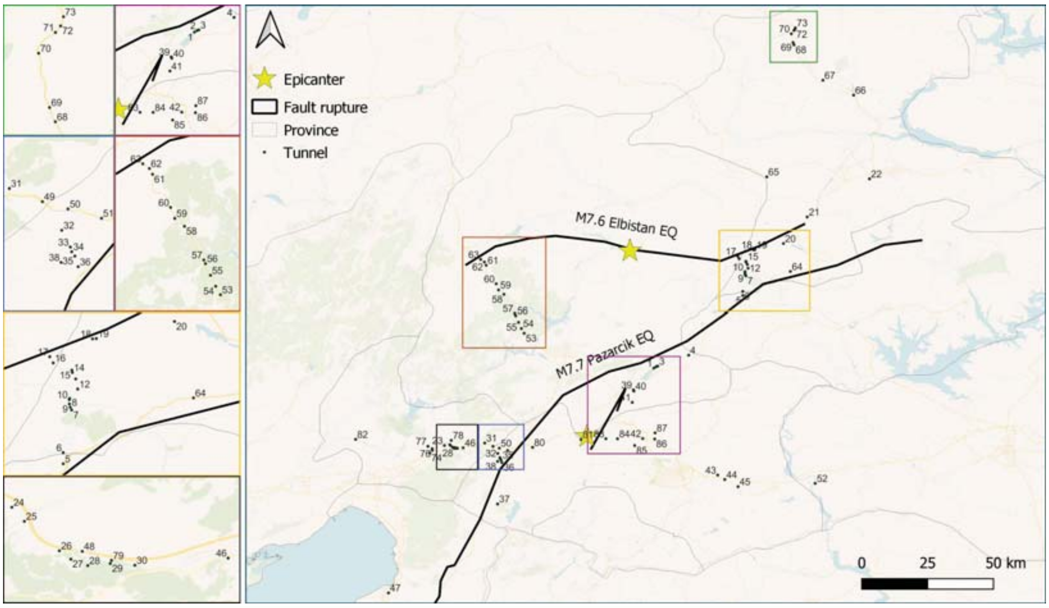
Affected area: 110 km^2 across 11 provinces

Tunnels

Aim

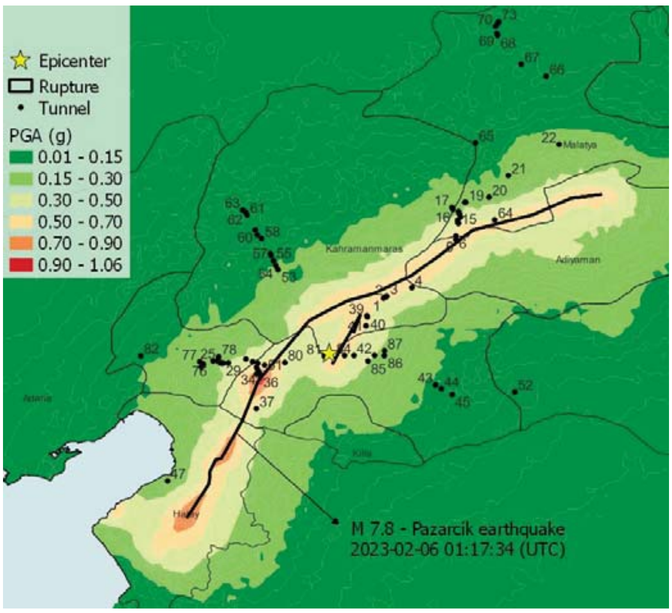
- To report and **describe the observed damages** to numerous tunnels located in the affected area during the 2023 Kahramanmaraş earthquake sequence
- To investigate the **main causes of the observed damages** in terms of:
  - (a) tunnels typology
  - (b) ground conditions and depth
  - (c) distance to the faults
  - (d) ground motion characteristics, i.e. PGA values at the location of the tunnels
- To **verify existing fragility models** for tunnels

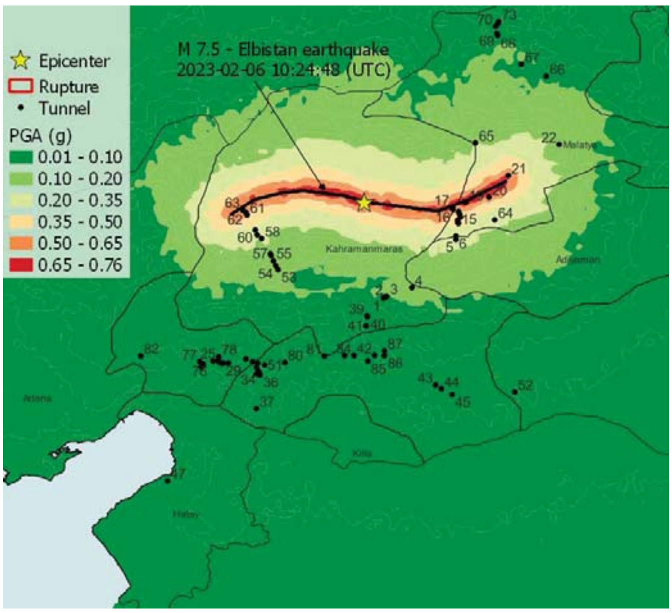
Tunnels in the affected area



- 87 tunnels in the affected area
- black dots indicate midpoints of the tunnels

Tunnels in the affected area



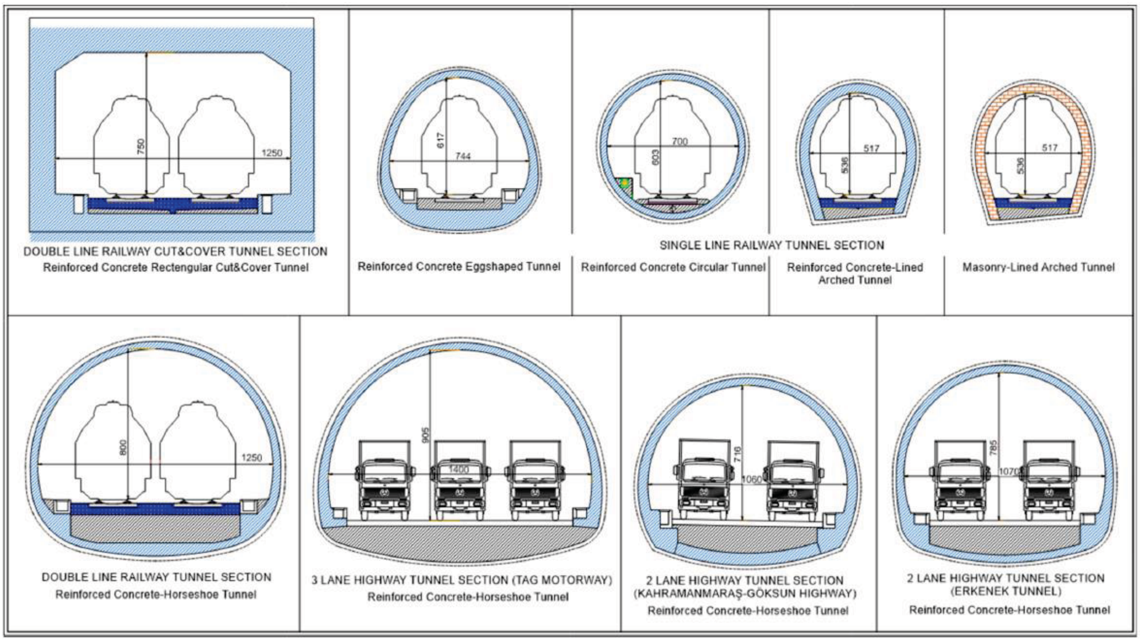


Tunnels in the affected area

- Tunnels in the region have been constructed during an approximate 120 years time-span (1912 – present), either before 1975 (first widely adopted Turkish seismic code CSCDA) or after 1995 (introduction of TEC-1998 regulation).
- 1912: railway tunnels, single-lane, **masonry-lined** arched structures
- 1927 – 1937: railway tunnels of various types (**concrete, RC or hybrid**) were constructed in the framework of the Fevzipaşa – Diyarbakır Railway Project
- Last three decades: RC state highway tunnels (two-lane), highway (three-lane) and railway tunnels (one/two-lane)




Typical tunnel sections



Tunnel inventory

For the 87 tunnels in the affected area, the following data was collected:

- ❑ construction year
- ❑ lining material (masonry, concrete, RC, hybrid)
- ❑ section type (arched, rectangular, horseshoe, circular)

} typology

- ❑ length
- ❑ width
- ❑ height
- ❑ maximum depth

- ❑ Geological Strength Index (GSI)
- ❑ proxy-based Vs,30

- ❑ distance from the two faults
- ❑ PGA from the two events

- ❑ **damage level** (no damage, slight, moderate, collapse)

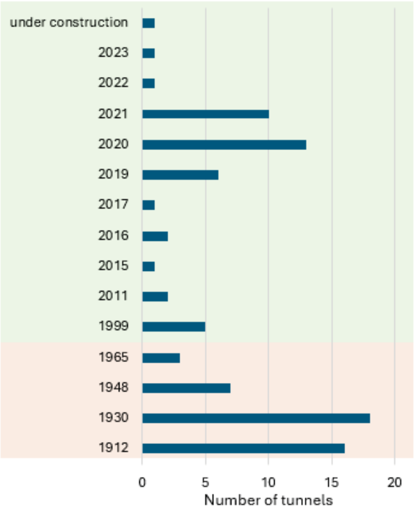
Gokceoglu C, Karahan S. (2023). Seismic performance of transportation tunnels in the region affected by the 6 February 2023 Türkiye earthquake sequence: A general assessment. ARMA 57th US Rock Mechanics Geomechanics Symposium. https://doi.org/10.13140/RG.2.2.19031.85928

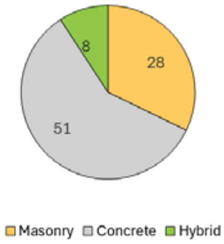
Tunnel inventory

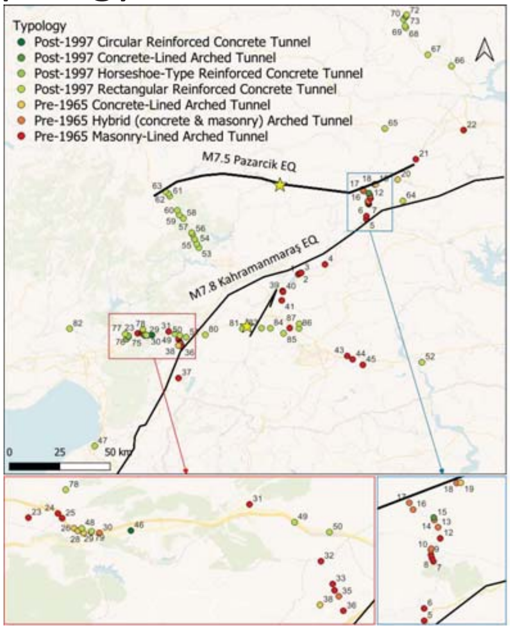
id	Typology (Construction date – lining and shape)	Length (m)	Width (m)	Height (m)	max depth (m)	GSI	Distance from the rupture of the M7.7 event (km)	PGA (g) – M7.7 event	Distance from the rupture of the M7.6 event (km)	PGA (g) – M7.6 event	Damage
1	1965 – Masonry-lined arched tunnel	726	4.9	5.38	63.6	45	4.79	0.58	53.89	0.1	No damage
2	1965 – Masonry-lined arched tunnel	316	4.9	5.55	31.0	40	4.66	0.58	53.11	0.1	No damage
3	1965 – Masonry-lined arched tunnel	172	4.9	5.8	11.0	20	4.74	0.58	52.74	0.1	No damage
4	1930 – Masonry-lined arched tunnel	215	4.9	5.26	10.0	20	6.33	0.54	45.76	0.12	No damage
5	1930 – Masonry-lined arched tunnel	657	4.9	5.15	51.1	15	1.58	0.66	18.98	0.28	Collapsed due to fault crossing
• • •											
85	2019 – RC rectangular tunnel	1280	8	8	52.9	40	17.07	0.32	91.23	0.06	No damage
86	2015 – RC rectangular tunnel	3240	12.5	8	70.2	50	22.14	0.26	87.02	0.08	No damage
87	2016 – RC rectangular tunnel	1760	12.5	8	55.8	50	20.7	0.28	84.11	0.08	No damage

All tunnel attributes, seismic-performance data, and the results of the analyses are available in an open Mendeley Data repository: https://data.mendeley.com/datasets/jj33zc587r/1

Tunnel inventory – tunnel typology

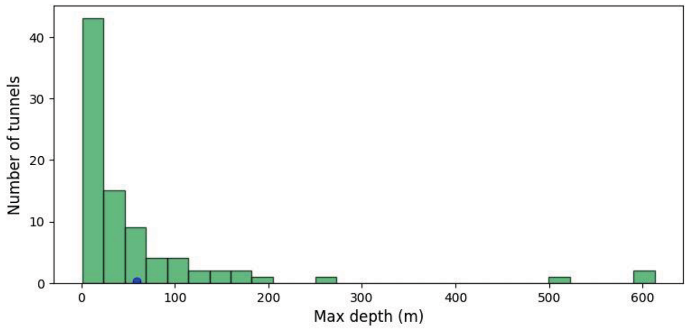


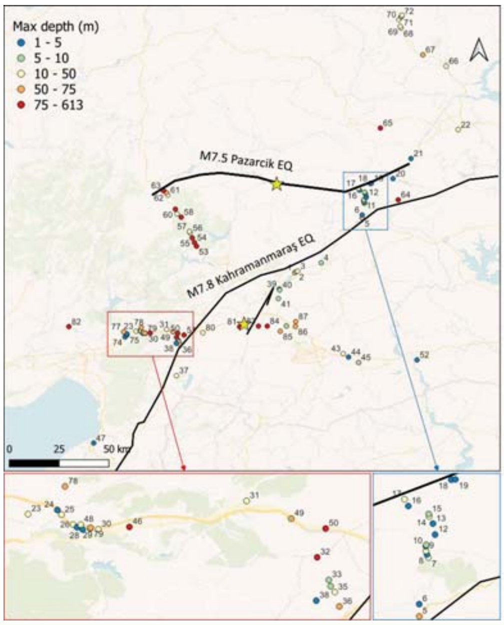




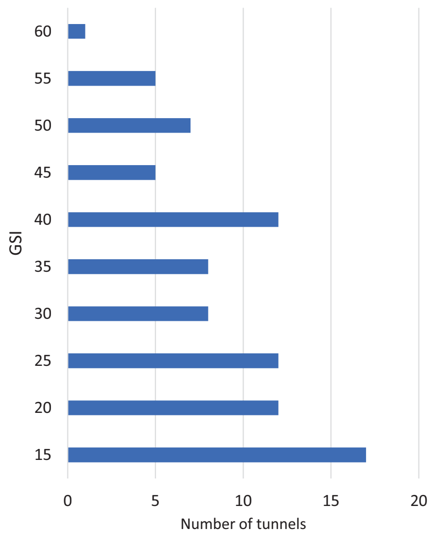
Tunnel inventory – maximum depth

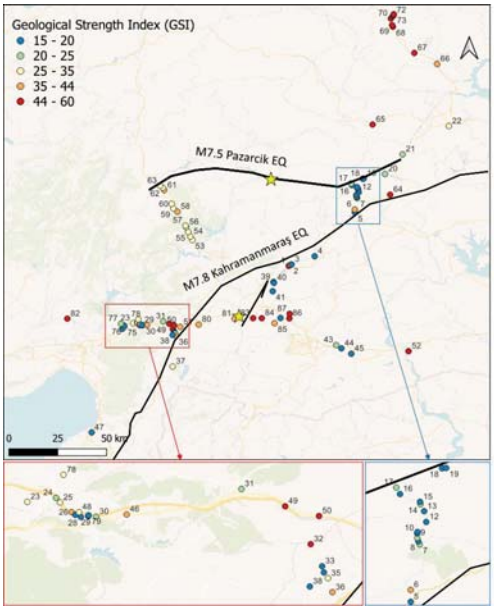
Max depth category	Number of tunnels	Percentage
<=10m	30	34.5%
10m - 40m	26	29.9%
40m - 100m	19	21.8%
100m - 200m	8	9.2%
>=200m	4	4.6%





Tunnel inventory - Geological Strength Index (GSI)

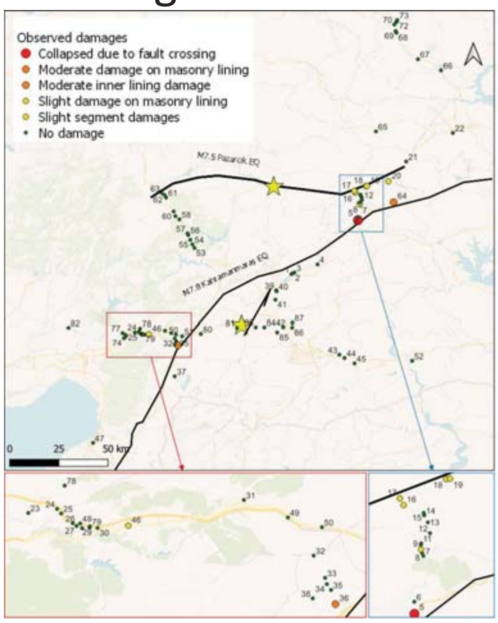




Tunnel inventory - observed damage

Observed damages	Number of tunnels	Percentage
No damage	77	88.5%
Slight	7	8.0%
Moderate	2	2.3%
Collapse	1	1.1%

id	Observed Damage
5	Collapsed due to fault crossing
36	Moderate damage on masonry lining
64	Moderate inner lining damage
9	Slight damage on masonry lining
16	Slight damage on masonry lining
17	Slight damage on masonry lining
18	Slight damage on masonry lining
19	Slight damage on masonry lining
20	Slight damage on masonry lining
46	Slight segment damages

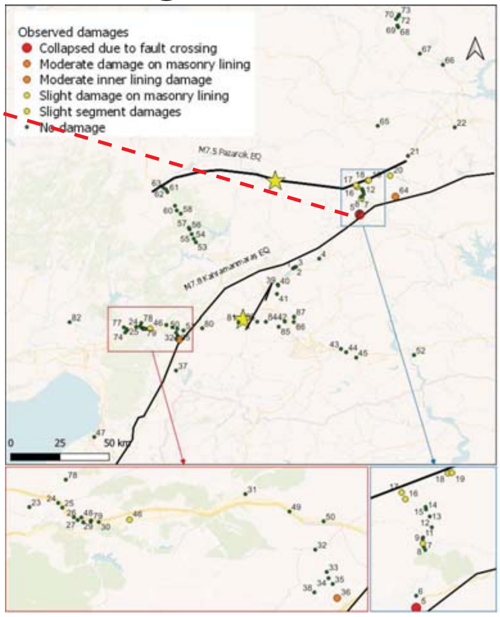


Tunnel inventory - observed damage

id 5

Malatya-Narli T3 railway tunnel in Adiyaman province
150 m of the tunnel collapsed

lining	Masonry
construction date	1930
Length (m)	657
Width (m)	4.9
Height (m)	5.15
max depth (m)	51.1
GSI	15
Distance from the rupture of the M7.7 event (km)	1.58
PGA (g) – M7.7 event	0.66
Distance from the rupture of the M7.6 event (km)	18.98
PGA (g) – M7.6 event	0.28



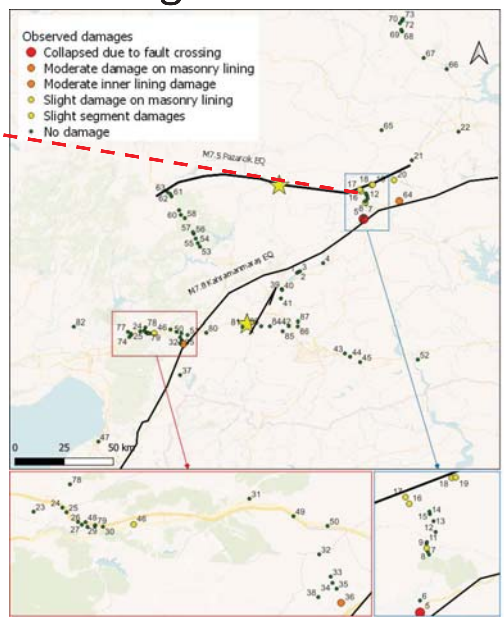
Gokceoglu C, Karahan S (2023). Seismic performance of transportation tunnels in the region affected by the 6 February 2023 Türkiye earthquake sequence: A general assessment. ARMA 57th US Rock Mechanics Geomechanics Symposium. https://doi.org/10.13140/RG.2.2.19031.85928

Tunnel inventory - observed damage

id 16

Toprakkkale-Meydanekbez T14 railway tunnel
slight damage on masonry lining

lining	Hybrid
construction date	1930
Length (m)	163
Width (m)	4.9
Height (m)	5.25
max depth (m)	4.6
GSI	15
Distance from the rupture of the M7.7 event (km)	16.4
PGA (g) – M7.7 event	0.26
Distance from the rupture of the M7.6 event (km)	2.12
PGA (g) – M7.6 event	0.66



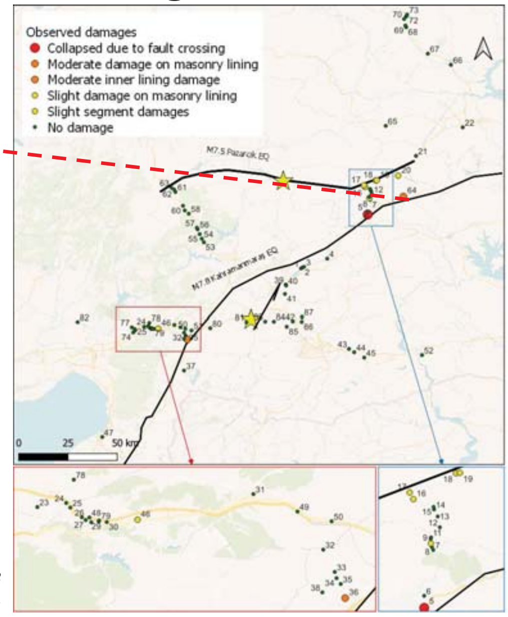
Gokceoglu C, Karahan S (2023). Seismic performance of transportation tunnels in the region affected by the 6 February 2023 Türkiye earthquake sequence: A general assessment. ARMA 57th US Rock Mechanics Geomechanics Symposium. https://doi.org/10.13140/RG.2.2.19031.85928

Tunnel inventory - observed damage

id 64

Malatya-Gaziantep Erkenek highway tunnel
moderate inner lining damage

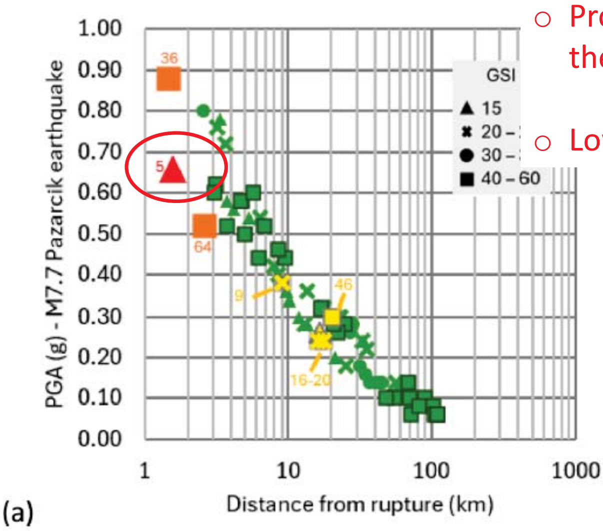
lining	RC
construction date	2017
Length (m)	1816
Width (m)	8
Height (m)	5
max depth (m)	107.4
GSI	45
Distance from the rupture of the M7.7 event (km)	2.59
PGA (g) – M7.7 event	0.52
Distance from the rupture of the M7.6 event (km)	17.18
PGA (g) – M7.6 event	0.26



Gokceoglu C, Karahan S (2023). Seismic performance of transportation tunnels in the region affected by the 6 February 2023 Türkiye earthquake sequence: A general assessment. ARMA 57th US Rock Mechanics Geomechanics Symposium. https://doi.org/10.13140/RG.2.2.19031.85928

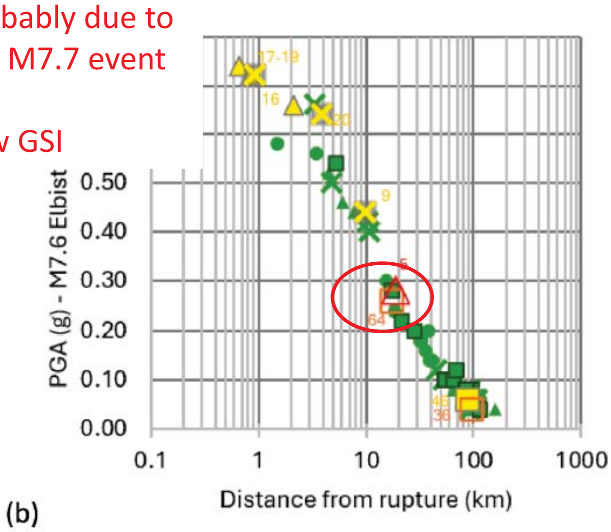
Damages in terms of Distance – PGA plots

M7.7 Pazarcik Event



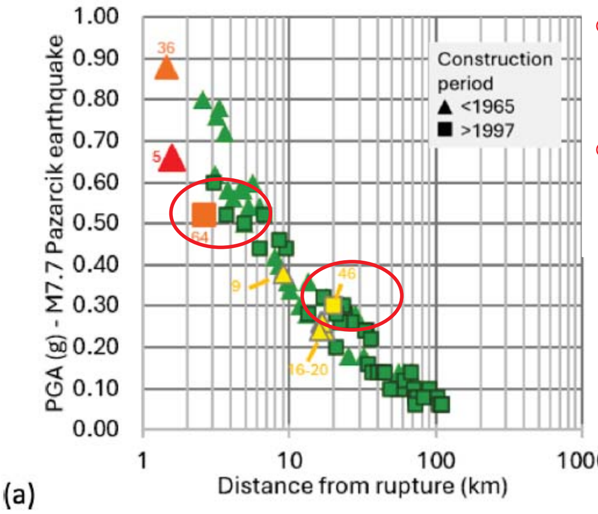
- Probably due to the M7.7 event
- Low GSI

M7.6 Elbistan Event

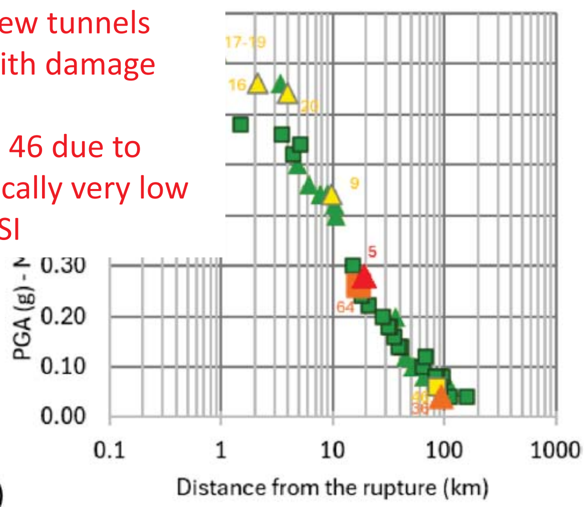


- Tunnels considered as points
- PGA from USGS ShakeMaps
- Shortest distance between point and rupture
- **Colors indicate damage level**
- **Symbols represent GSI**

Damages in terms of Distance – PGA plots



- New tunnels with damage
- id 46 due to locally very low GSI



- Symbols represent **construction period**
- Several old tunnels performed well
- Modern RC tunnels performed better

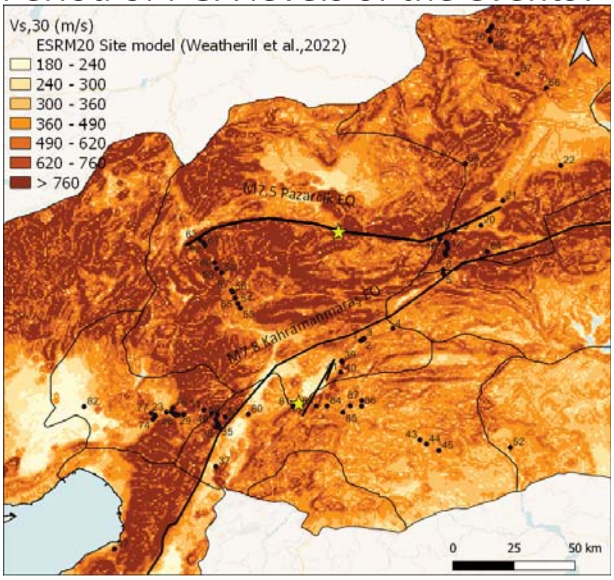
Some preliminary observations:

- ❑Out of the 87 examined tunnels in the broader affected area only **10 experienced varying degrees of damage**, and from these 10 tunnels only **one collapsed**.
- ❑The **proximity to the faults** seems to be the controlling parameter for the level of damages.
- ❑The role of the PGA at the ground surface, the ground conditions or even the age of construction or the typology of the tunnel (masonry or reinforced concrete), seem to play a complementary role.

Can we make an estimate for the mean Return Period of PGA levels of the events?

- A first-order attempt to quantify **how rare the PGA levels** experienced by the tunnels might have been, supporting in this way the critical evaluation of the subsequent risk assessments.

- **Probabilistic Seismic Hazard Analysis** (PSHA) for the affected area to obtain seismic hazard curves at the locations of the tunnels
  - OpenQuake-Engine (Pagani et al., 2014)
  - seismogenic source model and GMM from ESHM20 (Danciu et al., 2021, Kotha et al., 2020)
  - local site conditions (Vs,30) from ESRM20 (Weatherill et al., 2022)
  - tunnels approximately represented by their midpoints

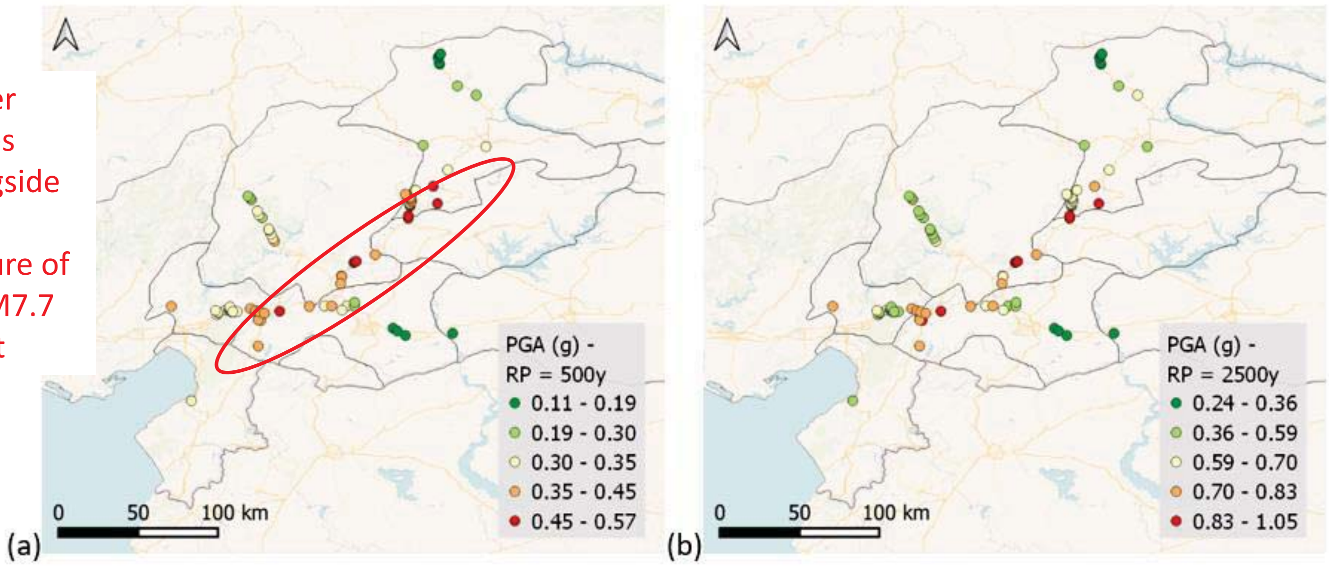


Weatherill G.A, Crowley H., Roullé A., Tourlière B, Lemoine A., Gracianne Hidalgo C., Kotha S.R., Cotton, F., Dabbeek J. (2021). European Site Response Model Datasets Viewer (v1.0). DOI: 10.7414/EUC-EUROPEAN-SITE-MODEL-DATA-VIEWER

Can we make an estimate for the mean Return Period of PGA levels of the events?

PGA values at the locations of the tunnels obtained from PSHA

higher values alongside the rupture of the M7.7 event



Can we make an estimate for the mean Return Period of PGA levels of the events?

- To **compare the PSHA results and the USGS ShakeMaps** at the tunnel locations for the two events we selected a subset of 65 out of 87 tunnels with $V_{s,30} \geq 500$ m/s
- For the locations of these tunnels, we extracted the mean PGA values for each of the two main events from the USGS ShakeMaps
- Then, we calculated from the hazard curves obtained from PSHA the mean return periods that correspond to the PGA levels of the ShakeMaps
- These return periods refer to the **exceedance of specific levels of PGA at different sites in the study region** and not to the occurrence of an event of specific magnitude occurring at a specific location.

Can we make an estimate for the mean Return Period of PGA levels of the events?

For example, for the tunnel with id 5 (collapse)

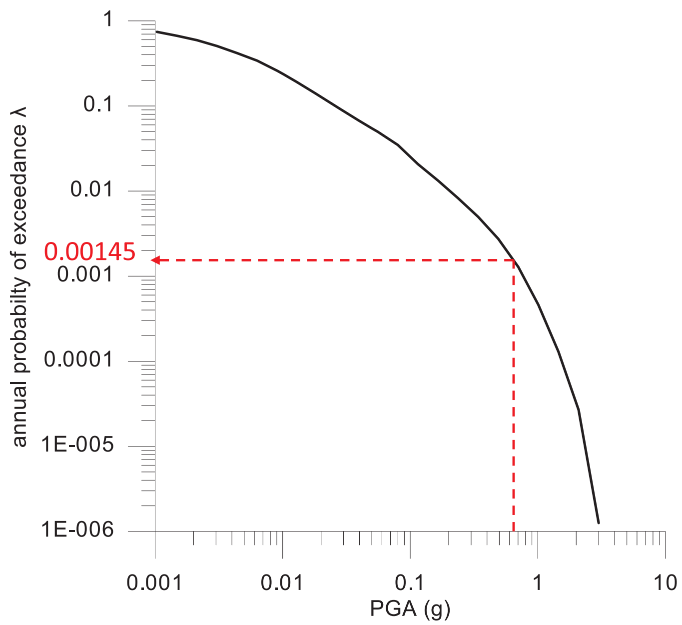
PGA from USGS ShakeMap for the **M7.7 Pazarcik** earthquake = 0.66g

Annual probability of exceedance λ=0.00145

Return period = 688 years

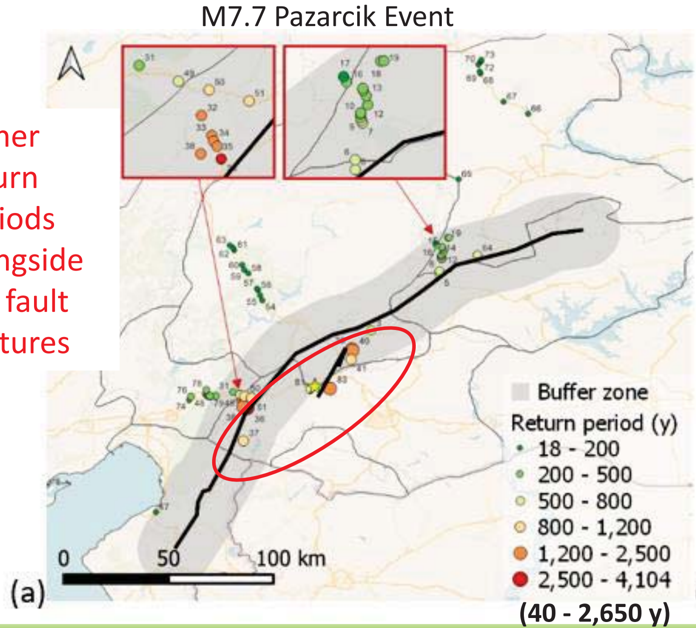
PGA from USGS ShakeMap for the **M7.6 Elbistan** earthquake: 0.28g

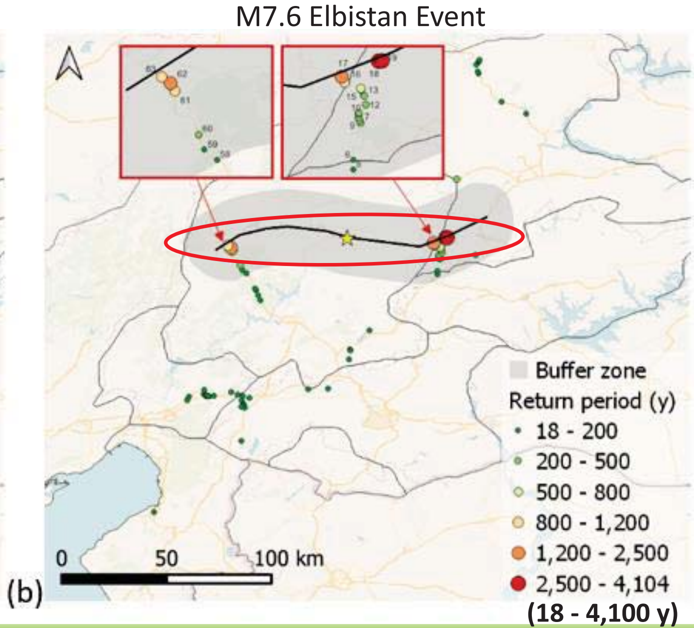
Return period = 156 years



Can we make an estimate for the mean Return Period of PGA levels of the events?

higher return periods alongside the fault ruptures



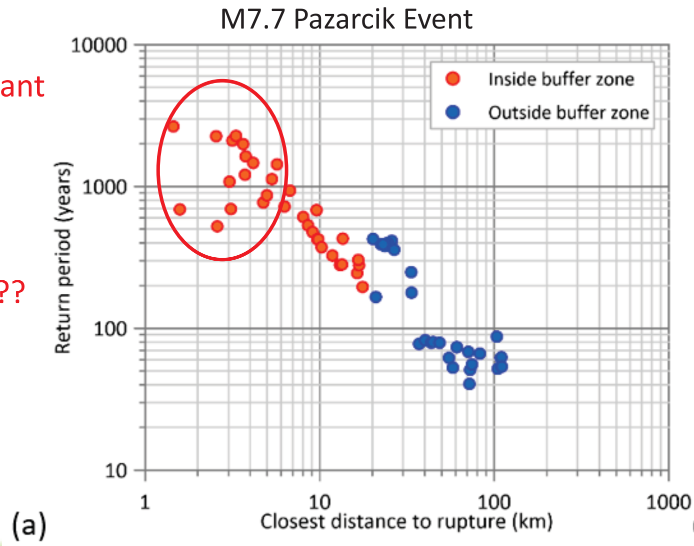


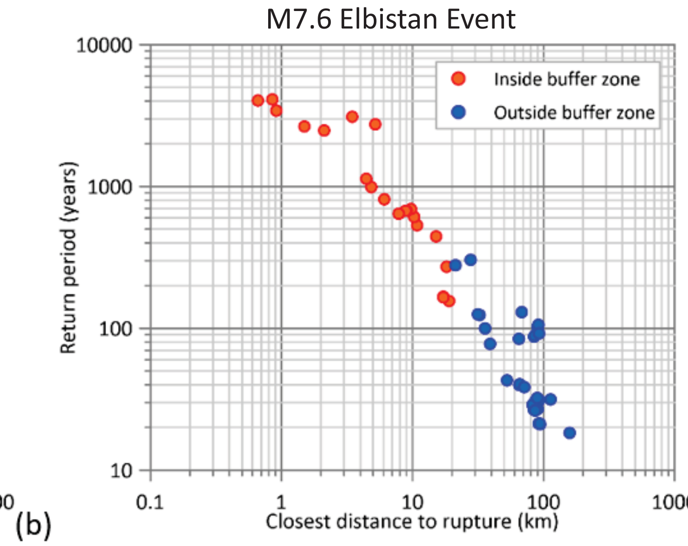
Can we make an estimate for the mean Return Period of PGA levels of the events?

Distribution of the calculated return periods with shortest distance to rupture

Significant scatter

Near-source effects??





Verification of risk models for tunnels

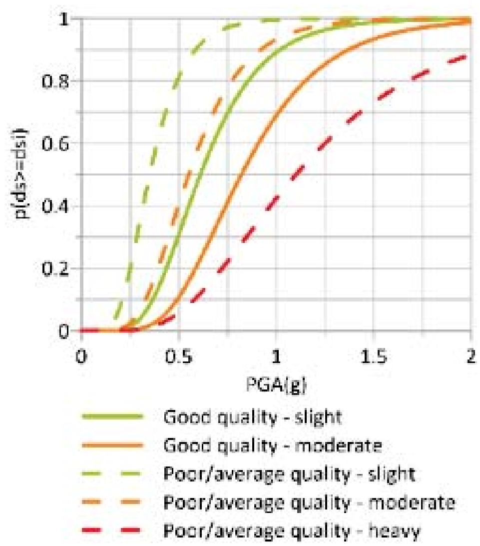
- The **vast amount of data on the seismic performance of tunnels** under strong ground shaking obtained after the Kahramanmaraş earthquake sequence provide a great opportunity to check the efficiency of available seismic risk models for tunnels.
- Scenario-type seismic risk analyses with the OpenQuake-engine to estimate the **expected damages** to the herein studied tunnels due to the two events
  - Seismic demand: **latest USGS ShakeMaps** for the two main events
  - Fragility modelling: **three generic fragility models** from the literature, two empirical, the HAZUS (2004) and the ALA (2001), and one analytically derived developed by Andreotti and Lai (2019)
- **Comparison of the estimated damages with the observed ones**.

Fragility models for tunnels

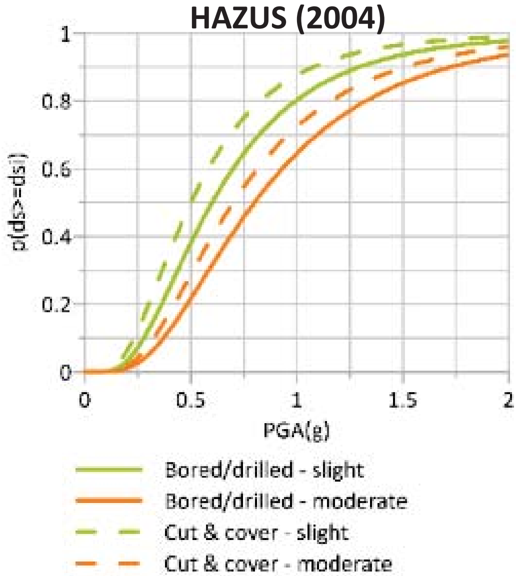
ALA (2001) & HAZUS (2004):

- **empirical fragility functions** based **on expert judgment** or **statistical process of reported damage data**
- quality of construction (ALA, 2001) and type of construction (HAZUS 2004)
- PGA-based fragility functions
- Only ALA (2001) provides heavy damage state

ALA (2001)



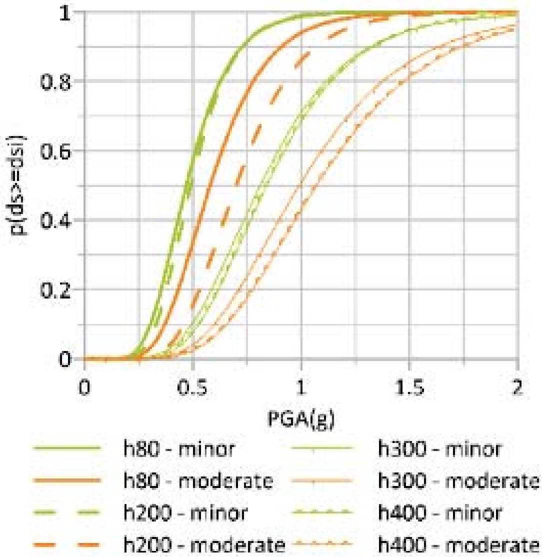
HAZUS (2004)



Fragility models for tunnels

Andreotti and Lai (2019)

- 2D nonlinear dynamic analyses of various tunnel–ground configurations
- PGA- and PGD-based fragility functions
- Tunnel depth is considered
- Only minor and moderate damage states



Comparison of the results of the analyses and the observed damages

	HAZUS (2004)		ALA (2001)		Andreotti and Lai (2019)		**Observed Damages**
Damage State	M7.8	M7.5	M7.8	M7.5	M7.8	M7.5	
No damage	68	77	63	75	64	75	**77**
Slight	6	3	9	4	6	3	**7**
Moderate	13	7	11	6	17	10	**2**
Heavy/ Collapse	-	-	3	1	-	-	**1**

- all fragility models tend to overestimate the number of tunnels with moderate or heavy damage
- comparisons for slight damage are fairly good

Comparison of the results of the analyses and the observed damages

However…

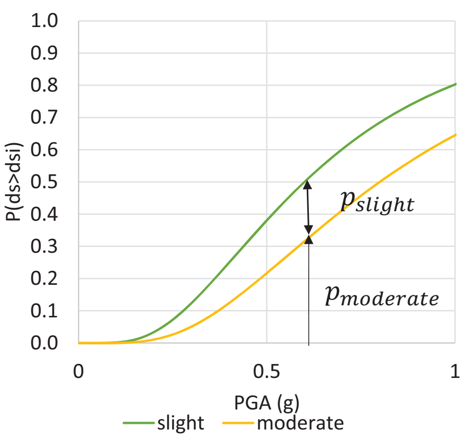
direct comparison between the estimated damages using empirical or analytical vulnerability models and the actual damages is challenging as:

- ❑ differences in the number and definitions of damage states between the selected fragility models
- ❑ characterization of the actual damages may involve inherent uncertainties due to the subjective nature of expert judgment
- ❑ field surveys for the damages on tunnels began after the second seismic event, making it difficult to attribute the observed/actual damages directly to either of the two major seismic events
- ❑ the representation of each tunnel in the exposure model as a single point increases the uncertainties for longer tunnels

➡ **probability of damage of any level** is introduced as damage metric

Comparison of the results of the analyses and the observed damages

e.g. using the fragility curves of HAZUS (2004) for the bored/ drilled tunnels

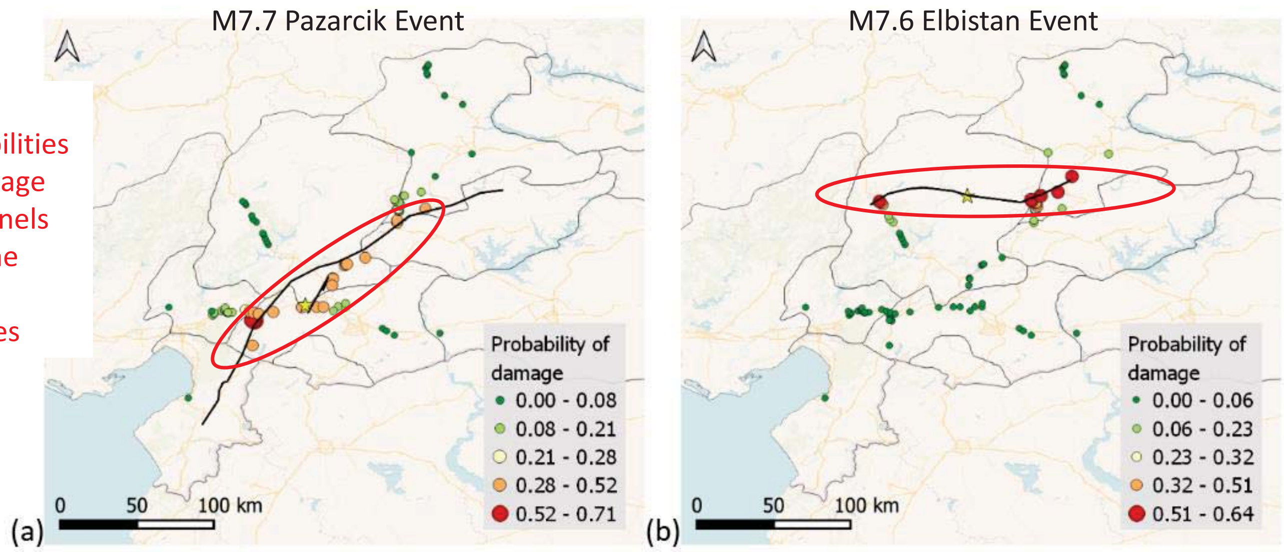


$$Probability\ of\ damage = p_{slight} + p_{moderate}$$

Comparison of the results of the analyses and the observed damages

Probability of damage with the fragility model of **HAZUS (2004)** and USGS ShakeMaps

higher probabilities of damage for tunnels near the fault ruptures



Effect of selected fragility model

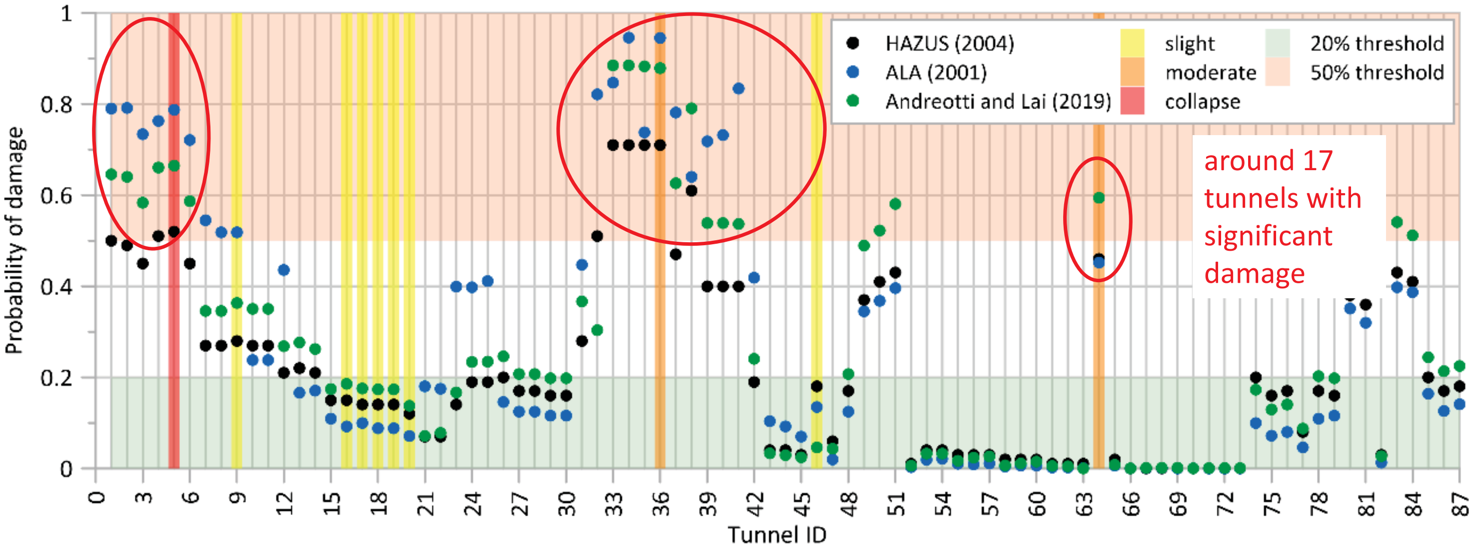
M7.7 Pazarcik Event

To facilitate comparisons with observed damage data, we assume two thresholds for the probability of damage:

- ❑ 20% below which minimal damage is expected
- ❑ 50% above which a higher probability of severe damage is expected

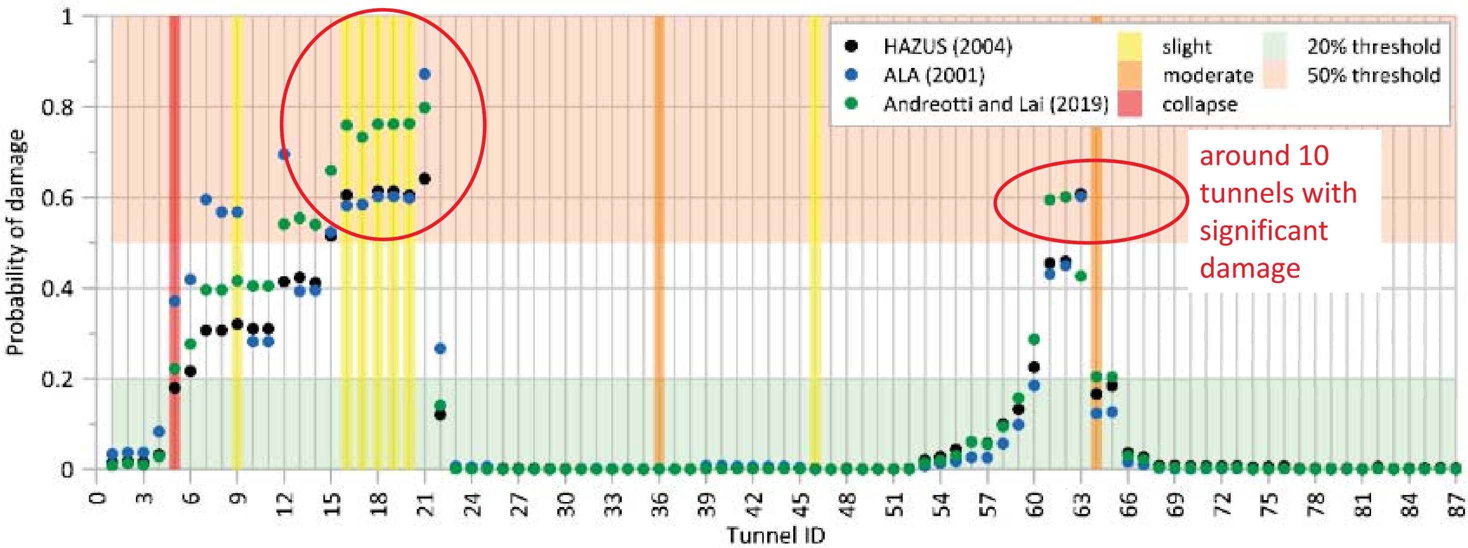
Effect of selected fragility model

M7.7 Pazarcik Event



Effect of selected fragility model

M7.6 Elbistan Event



Conclusions (on tunnels)

- Despite the very strong ground motions recorded in the affected area, the **observed reported damages for tunnels were limited**.
- The general opinion that **the underground structures perform better than above-ground structures** is valid at least in this case where the problem is exclusively related to ground shaking and not to permanent ground deformation phenomena (fault crossing, landslide, subsidence, liquefaction).
- In the case of ground shaking the main parameters affecting the level of damages is the **distance to the fault** and the **intensity of ground motion**. The typology of the tunnels, the year of construction, the burial depth and the ground properties may also affect the level of the damages, but at a lower degree.
- If we would like to propose a **threshold for the appearance of at least slight damage**, we could propose the following:

  R<20km and PGA_{sur}>0.2g
- Available fragility models for the damage assessment of tunnels are generally conservative, but **achieve to provide a useful first order estimate of the damage**

Residential buildings




Methodology

- We apply **open data and software**, freely available before or shortly after the Kahramanmaraş earthquake sequence:
  - Different versions of the **USGS ShakeMaps**
  - **ESRM20 exposure model** (administration level 1) for Turkey (Crowley et al., 2020)
  - **ESRM20 fragility models** (Romão et al., 2021)
  - OpenQuake-Engine ('Scenario from ShakeMap Calculator')

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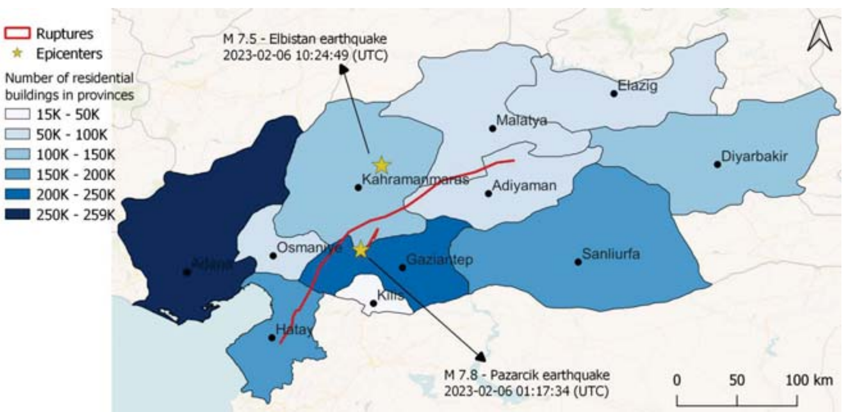
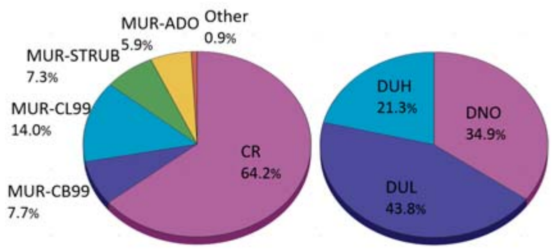
Building exposure model

- **11 provinces** with **1,321,186 residential buildings**
- Location of each asset is described by the density-weighted-average centroid
- Building classification using the GED4ALL Building Taxonomy (Silva et al., 2022)



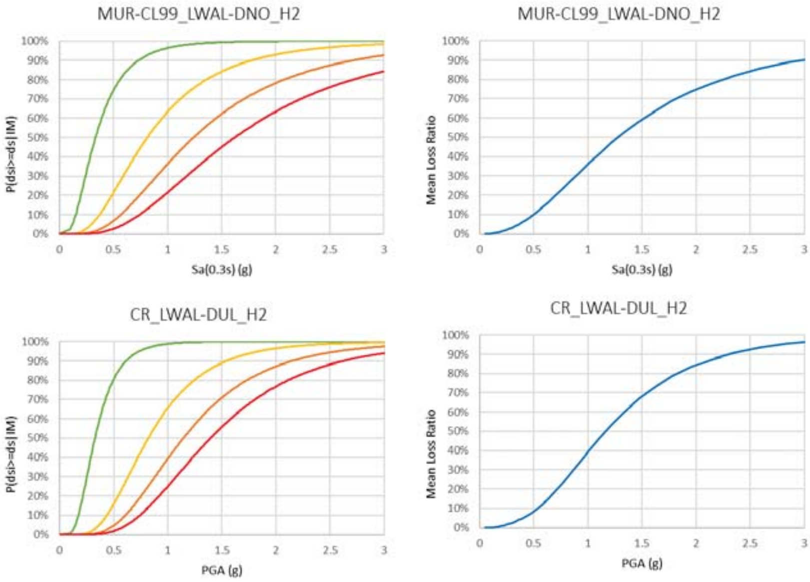


https://gitlab.seismo.ethz.ch/efehr/esrm20/-/tree/main/Exposure

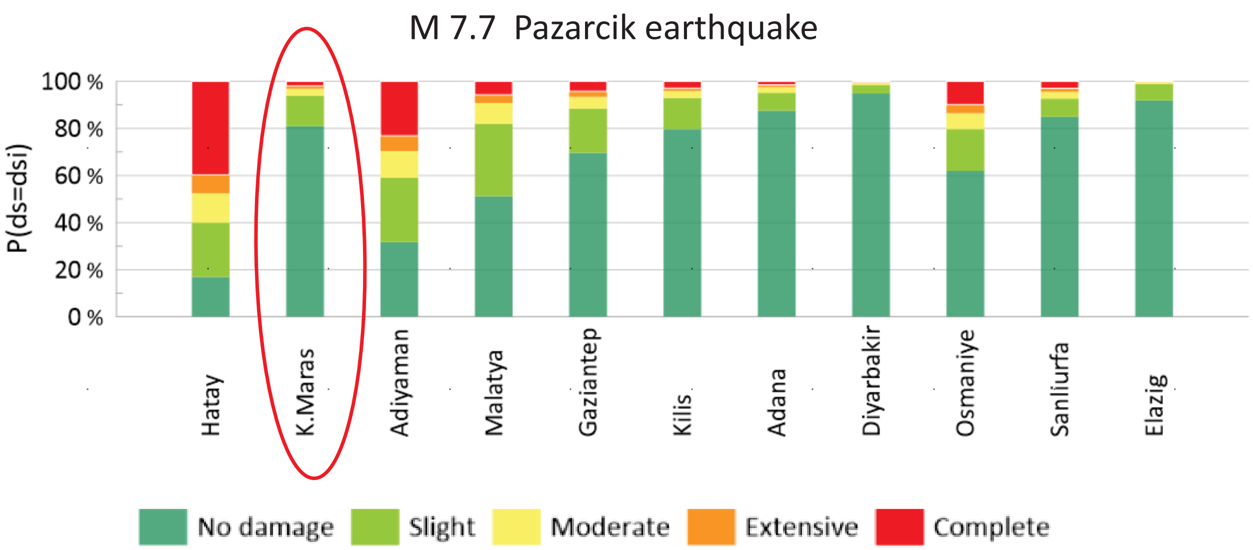
Verification of seismic risk models using observed damage from past earthquake events. Riga et al.

ESRM20 fragility models for ShakeMap analyses

ESRM20 fragility and vulnerability curves (Romão et al., 2021) for the most common building typologies in the examined areas

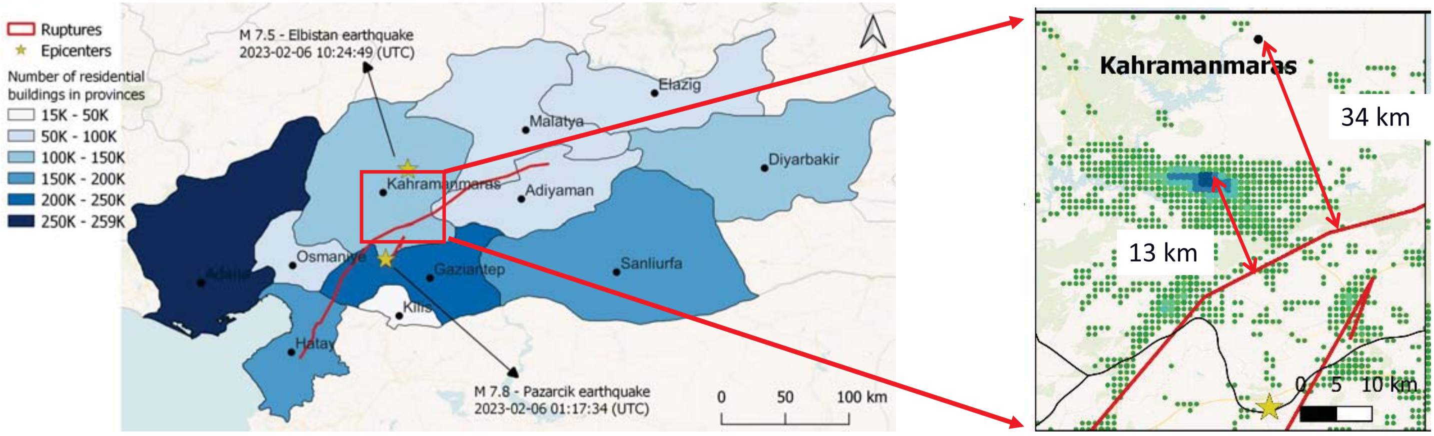


Estimated percentages of buildings being in each damage state



Kahramanmaras province

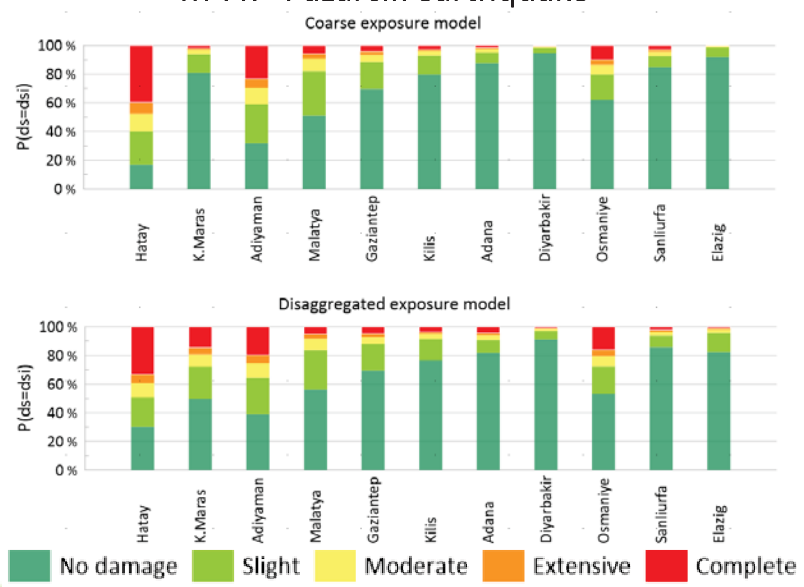
- ❑ Is the resolution of this exposure model enough?



significant deviation between the centroid of the polygon (coarse exposure model) and the region with the highest population (disaggregated exposure model)

Effect of exposure model resolution

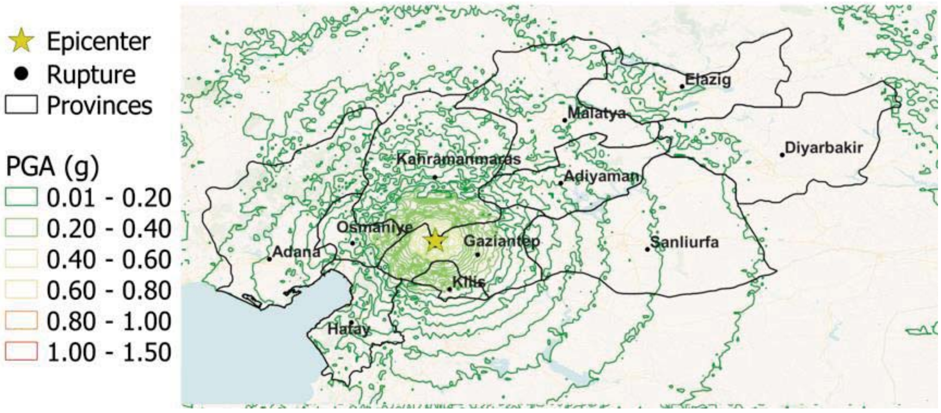
M 7.7 Pazarcik earthquake



Effect of ShakeMap version

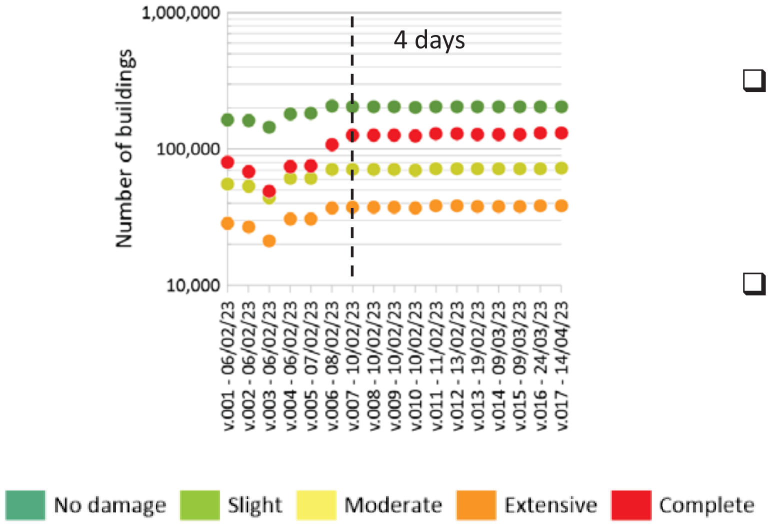
M 7.7 Pazarcik earthquake

Version - date: v.001 - 06/02/23
Process timestamp: 06/02/2023 01:29, Mw : 7.8, Depth : 24.1 km



Effect of ShakeMap version

M 7.7 Pazarcik earthquake



- Estimated extensive and complete damages increase significantly from the first to the latest ShakeMap version; however, **v001 (~10 min post-event) already indicated severe impacts**
- Estimated damages **converge by v007 (~4 days post-event)** when the full rupture model was incorporated.

Comparison with officially announced data

Officially announced data for the whole study area (ÇŞİDB, 2023)

Damage state	Number of buildings
No observable damage	860,006
Light damage	434,421
Moderate damage	40,228
Heavy damage	179,786
Partial collapse/ Collapse/ Urgent need for demolition	52,846
Could not detect	147,895
Total	1,715,182

Comparison with officially announced data

M 7.7 Pazarcik earthquake

Heavily damaged or collapsed buildings

	Officially announced data (ÇŞİDB, 2023)	**Scenario analysis** (coarse exposure)	**Scenario analysis** (disaggregated exposure)
Percentage	13.6%	11.6%	12.5%

Conclusions (on buildings)

- Open-access datasets and models may serve as valuable tools for the risk assessment as well as rapid damage assessment at large scale following significant earthquakes
- The effect of the ShakeMap version is minimized few days after the event
- Analyses performed with a more refined (disaggregated) exposure model are in better agreement with the observed damages
- A more thorough comparison with officially announced data at province level may contribute to the further validation of the models

Some more relevant results

Thessaloniki 1978

Damage state	Rupture model (%)	ShakeMap (%)	Color tag	Rupture model (%)	ShakeMap (%)	Post-earthquake tagging (Kappos et al. 2008) (%)
No damage	46.1	50.4	Green	82.5	92.2	74.50
Slight	36.4	41.7				
Moderate	9.0	5.6	Yellow	12.6	7.0	19.10
Extensive	3.6	1.4				
Complete	4.9	0.8	Red	4.9	0.8	6.40

Athens 1999 EQ

Damage state	Rupture model (%)	ShakeMap (%)	Color tag	Rupture model (%)	ShakeMap (%)	Post-earthquake tagging (ESYE 1999) (%)
No damage	55.8	41.7	Green	86.3	83.8	62.5
Slight	30.5	42.1				
Moderate	7.1	9.7	Yellow	9.9	13.0	32.8
Extensive	2.8	3.3				
Complete	3.8	3.2	Red	3.8	3.2	4.8

Post-earthquake tagging:

Green: No reduction of seismic capacity. Immediately usable.
Yellow: Reduced seismic capacity. Usage not permitted before repair- strengthening.
Red: Unsafe. Usage or entry is prohibited.

Riga E, Karatzetzou A, Apostolaki S, Crowley H and Pitilakis K (2021). Verification of seismic risk models using observed damages from past earthquake events. Bulletin of Earthquake Engineering, Vol. 19, pp. 713–744, https://doi.org/10.1007/s10518-020-01017-5

Conclusions

- There are plenty open-access datasets, models and tools that can be used for the (rapid) risk assessment of different types of structures
- Comparisons with observed damage from past earthquake events have shown that such models can be useful for **a first order estimate** of the expected damages of a future event at **large scale**
- But… still a lot of uncertainties that should be addressed in future validation efforts (e.g. site modelling)

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Mulțumesc foarte mult!

Ευχαριστώ πολύ!

Thank you!

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ
Verification of seismic risk models using observed damage from past earthquake events. Riga et al.
Societatea Română de Geotehnică și Fundații

Anastasios Anastasiadis

Professor, Department of Civil Engineering, Faculty of Engineering, Aristotle University of Thessaloniki, Greece

Subsurface Structure and Dynamic Soil Properties as Basis for Microzonation and Site Effects Studies: Lessons Learned from Studies in Greece and Cyprus

Anastasios Anastasiadis is a professor of geotechnical engineering in the Department of Civil Engineering at the Aristotle University of Thessaloniki. Prior to joining the university, he worked as a senior researcher at the Institute of Engineering Seismology and Earthquake Engineering (ITSAK-EPPO). His specialties include geotechnical earthquake engineering, with an emphasis on laboratory and in-situ testing; soil characterization in geotechnical and earthquake engineering; experimental and theoretical studies on seismic soil response; studies on the influence of local soil conditions; micro-zone studies; soil liquefaction; and improvement studies. He served as secretary of the ISSMGE Seismic Geotechnical Engineering Technical Committee (2008-2015), and he is a member of the ERTC-12 European Technical Committee, which evaluates and comments on Eurocode 8. He is also a member of several professional associations, including the ISSMGE, the ISRM, and the GCOLD. Since 2023 he is an elected board member of the Hellenic chapter of ISSMGE. He has worked as a scientific supervisor and as a researcher on more than 60 projects in Greece and the EU, primarily in the fields of geomechanics and geotechnical earthquake engineering. He has also worked as a consultant on over 20 major dynamic geotechnical and earthquake engineering projects. Throughout his career, he has published more than 150 scientific papers in peer-reviewed journals and international conference proceedings.

Subsurface Structure and Dynamic Soil Properties as Basis for Microzonation and Site Effects Studies: Lessons Learned from Studies in Greece and Cyprus

Anastasios Anastasiadis

Professor AUTH

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

Thessaloniki, 9th October 2025

Societatea Română de Geotehnică şi Fundaţii

Contributions & Acknowledgements

Professor Dimitris Raptakis
Professor Maria Manakou
Dr. Konstantia Makra
Dr. Evi Riga
Dr. Paschalis Apostolidis
Dr. Angelos Tsinaris
….and many others……individuals, governmental and public authorities….

Outline

- SDGEE – Lab & In-Situ Facilities
- Determination of ground/soil structure and properties

  Urban areas - Detailed Level - Ground Shaking

  Site Characterization - Dynamic Properties
- Validation
- Modeling Site Effects
- Microzonation studies
- Concluding Remarks

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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Outline

- The lecture will be based on various results from microzonation studies but only regarding their geotechnical and site response part

- Geotechnical investigation: The scale effect (local, **district, city level,** regional, national)

- Microzonation studies cannot and will never replace the geotechnical surveys and cannot substitute the seismic codes where they should be used as a complementary information for specific applications

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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Research Projects (selected)

RISK-UE: An advanced approach to earthquake risk scenarios with applications to different European cities (EU) - Barcelona, Bitola, Bucharest, Catania, Nice, Sofia and Thessaloniki. 2001-2004, www.risk-ue.net

LESSLOSS: Earthquake disaster scenario predictions and loss modeling for infrastructures (EU) - www.lessloss.com

SRM-LIFE: Development of a global methodology for the vulnerability assessment and risk management of lifelines, infrastructure and critical facilities. Application in the Metropolitan area of Thessaloniki. coord.K.Pitilakis, GSRT-GR, 2003-2007

SYNER-G: Systemic seismic vulnerability and risk analysis for buildings, lifeline networks and infrastructures safety gain www.syner-g.eu

STREST: Harmonized approach to stress tests for critical infrastructures against natural hazards www.strest-eu.org

SIBYL: Seismic monitoring and vulnerability framework for civil protection www.sibyl-project.eu

SHARE: Seismic hazard harmonization in Europe www.share-eu.org

Microzonation studies: Thessaloniki, Volos, Larissa, Kozani, Kalamata, Aegion, Grevena, Lemessos, Paphos, Ammochostos, Duzce (TR)

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Research Unit of Soil Dynamics and Geotechnical Earthquake Engineering (SDGEE) –

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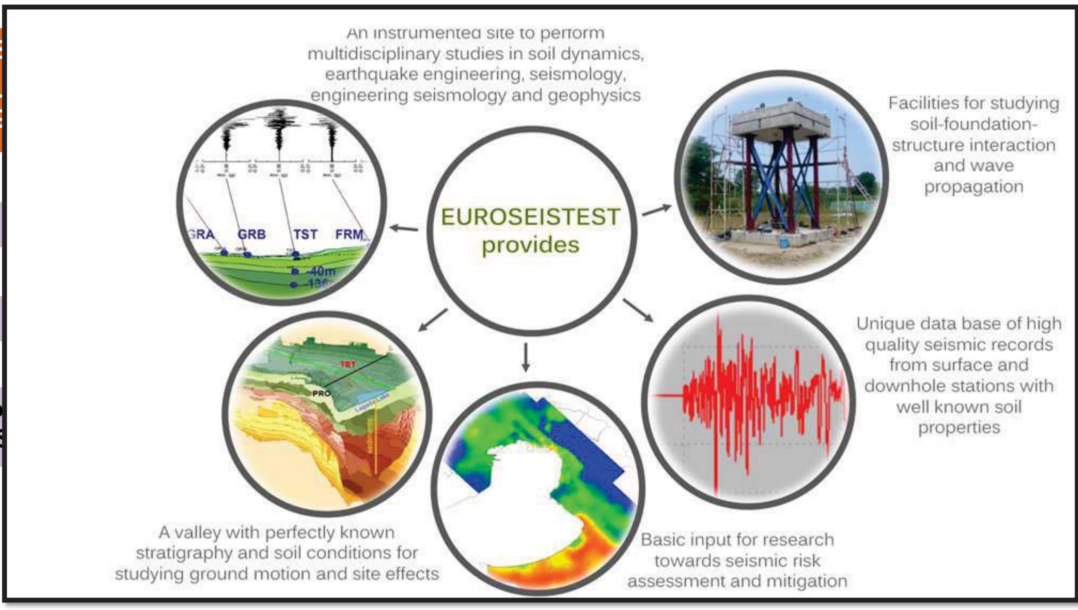

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ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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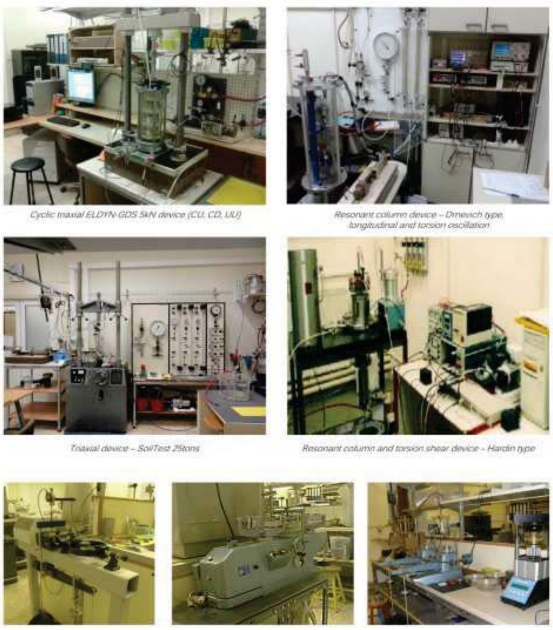
AUTH - SDGEE :Laboratory equipment and testing

Research Unit of Soil Dynamics and Geotechnical Earthquake Engineering



- Current value of infrastructure estimated to 3000 KEuro
- Area of 450 m^2
- Soil mechanics and Rock mechanics Testing:
- 2 Cyclic Triaxial Devices
- Resonant Column - Torsion Shear Device
- Resonant Column – Fixed free Device
- Bender Element testing
- Large Shear Box Device
- 1D – Shake Table Equip. (Laminar Boxes - Full instr. devices)

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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Strong ground motion monitoring (Recording of seismic activity, earthquake early warning, etc.)

SDGEE- FIELD MONITORING AND SURVEYS

- Seismometers (30sec-100HZ) CMG-40T (Guralp Ltd.)
- Recording systems (24bits) DAS-130 (Reftek Inc.)
- 3D Surface accelerographs CMG-5T (Guralp Ltd.)
- Etna accelerometers (Kinemetrics Inc.)
- K2 accelerometers (Kinemetrics Inc.)
- 3D down-hole accelerometers CMG-5TB, CMG-5T & High-quality digitizer CMG-24 (Guralp Ltd.)
- 3D down-hole accelerometers ES-DH EPI (Kinemetrics, Inc.)
- 1 18ch Mt. Whitney recording systems (Kinemetrics Inc.)

- 24ch seismograph Strataview (Geometrics, Inc./OYO)
- P & S surface geophones of 4.5Hz (Geometrics, Inc./OYO)
- 3D geophones for Down-hole & Cross-hole tests (Geostuff)
- 1 Cross-hole hammer (Bison)
- 2 -12m mems Borehole Accel. – ShapeArray SAAR (Measurand)
- 2 -1.2m mems Borehole Accel. – ShapeArray SAAR (Measurand)
- Long-Stroke vibration exciter 1kN – (Spectra)

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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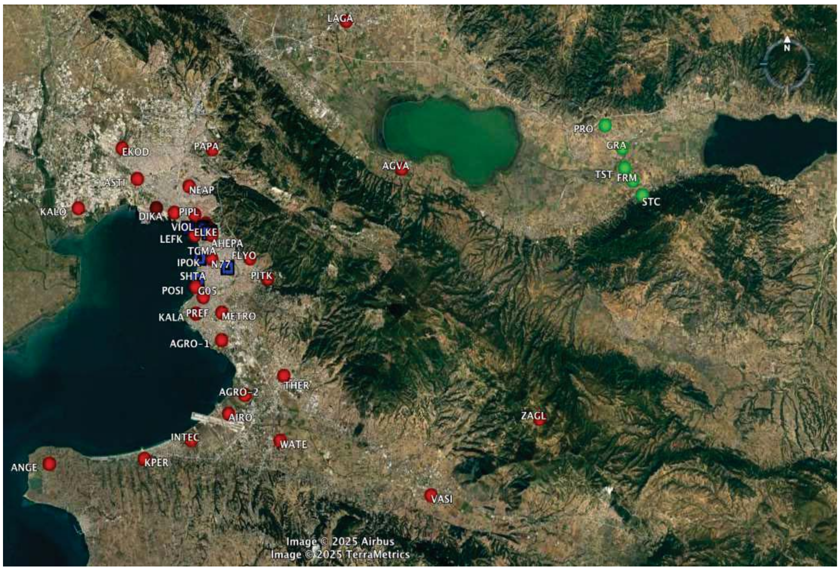
THESSNET



Microzonation studies: Background



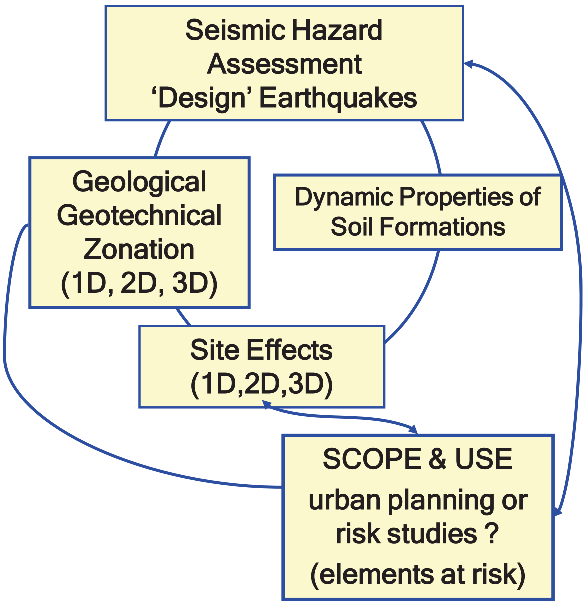

- Manual for Zonation on Seismic Geotechnical Hazards, TC4, ISSMFE, 1993, 1995
- Guidelines for Seismic Microzonation Studies, AFPS, 1995
- MERP, Seismic Microzonation for Municipalities, World Institute for Disaster Risk Management, Rep.of Turkey, MPWS, 2004
- ITALY, Presidency of the Council of Ministers of the Italian Republic Civil Protection Department, 2015

And

- Many micfrozonation studies performed by AUTH LSMGEE
- Participation in numerous relevant research projects
- EUROSEISTEST experimental facility (**http://euroseisdb.civil.auth.gr)**

Microzonation Studies: Main Stages & Typical Results

The key issue affecting the applicability and the feasibility of any microzonation study is the usability and reliability of the parameters selected for microzonation (Ansal & Biro, 2004)



- Microzonation is an efficient tool to mitigate the earthquake risk by hazard related land use management
- Microzonation does not replace the existing Building and Constructions Codes
- Values proposed by Codes has always to be respected as a minimum requirement

Ground response is controlled by the dynamic soil properties

- Soil acts like a kind of filter/amplifier in wave propagation
- Strength parameters
- Dynamic physical and dynamic properties
- Description of local and spatial soil conditions
- Scale effects
- Non-linear behavior

Ground and soil conditions

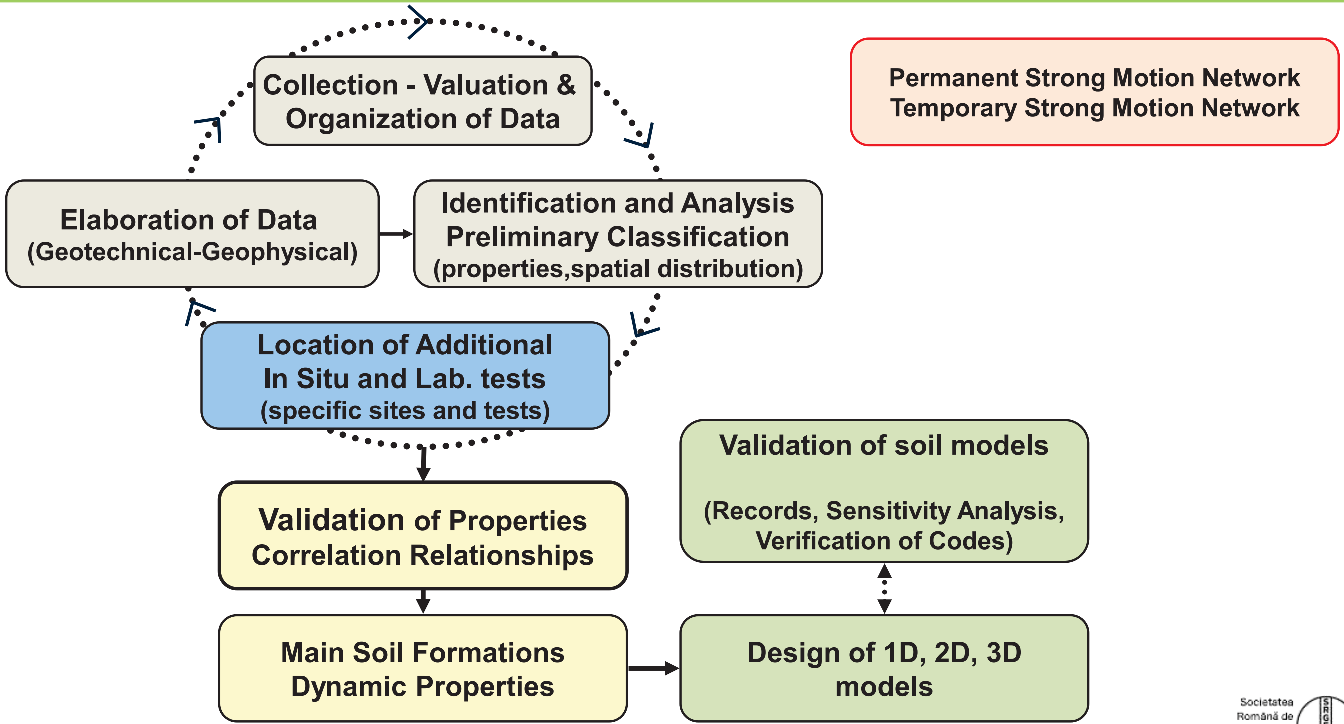
- Geology - Topography
- Groundwater Level
- Physical, mechanical and dynamic properties
- Vertical and lateral variation of soil properties
- Geometry of different soil layers
- Boundaries and volume of deposits
- Depth and dynamic properties of bedrock

✓ oriented to site effects studies

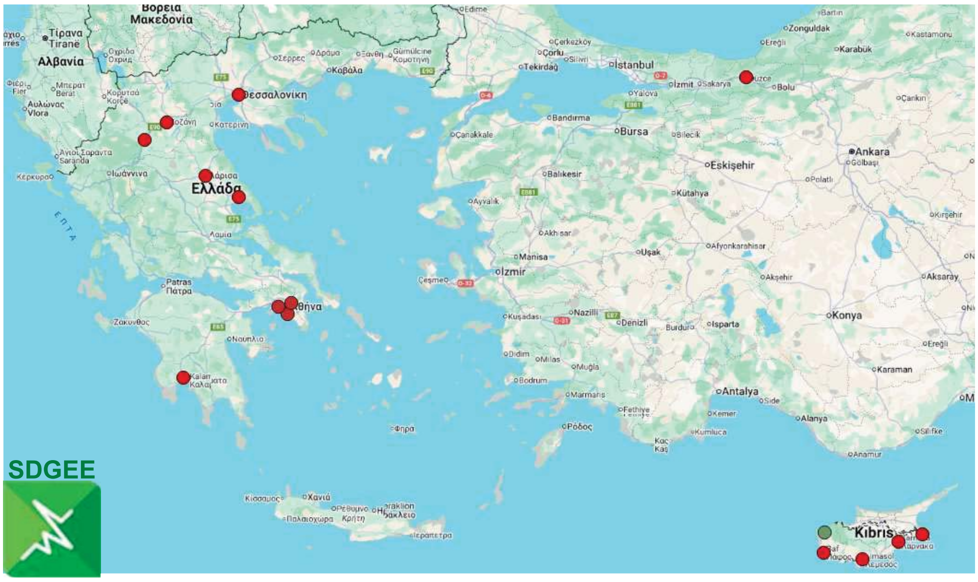
- study of geotechnical problems, land use planning
- risk studies, interpretation of damage distribution
- modeling and understanding of site effects physics

Steps for Determining soil structure and conditions

SDGEE



Microzonation Studies performed by LSDGEE-AUTH



Greece:
- Thessaloniki
- Kalamata
- Kozani
- Larisa
- Volos
- Grevena

- Duzce – Turkey

Cyprus
- Lemesos
- Pafos
- Ammochostos
- Chrysochous

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Case Study - Thessaloniki, Greece

Population>1,200,000 people).

Area (Urban): 275000 km^2

Area (Metropolitan): 1285000 km^2

History: 2400years

Latest major earthquake (1978)

M=6.5, R=25Km (NE)

PGA= 0.15g (?)

Social and economical results:

50 deaths, 220 injured

800,000 temporarily homeless

One collapse and few partial collapses and extended damage to buildings and Monuments

Calculated cost: >250 million $

The first destructive earthquake in a large urban area

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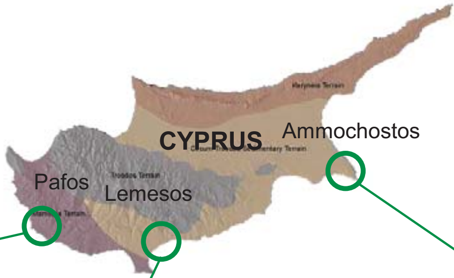
Case Studies Cyprus



- Lemesos: 160 km^2
- Pafos: 165 km^2
- Ammochostos: 135 km^2

1953 M=6.5
1996 M=6.8
1999 M=5.6
2022 M=6.6

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Geotehnică și Fundații

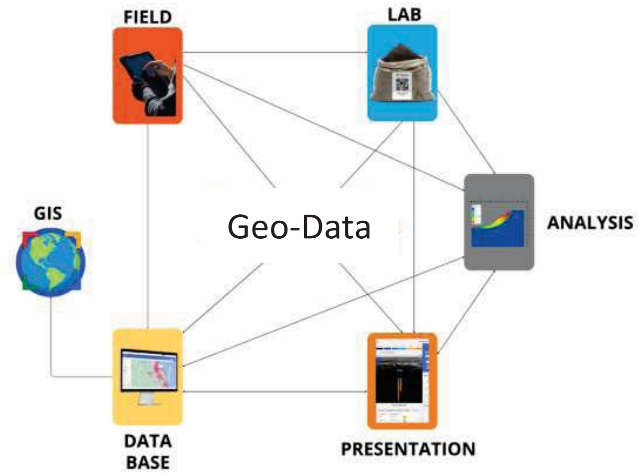
2nd Greek – Romanian Seminar: Lessons learned from Earthquakes and Geotechnical Failures

Collection - Evaluation & Organization of Data

Processing of the Data : Geotechnical-Geophysical Preliminary Classification

Geotechnical Data = Geospatial Data - Interchange Formats

how works



- Borehole locations
- In-situ tests
- Laboratory samples
- Monitoring data

Common Geotechnical formats :

AGS: Association of Geotechnical and Geoenvironmental Specialists.

DIGGS: uses XML as its data format, which is a flexible allows for the creation of custom tags, enabling detailed and hierarchical data representation.

GEF: (Geotechnical Exchange Format) is a family of Dutch text-based file formats

Open source:

GEF-BORE
GEF-CPT

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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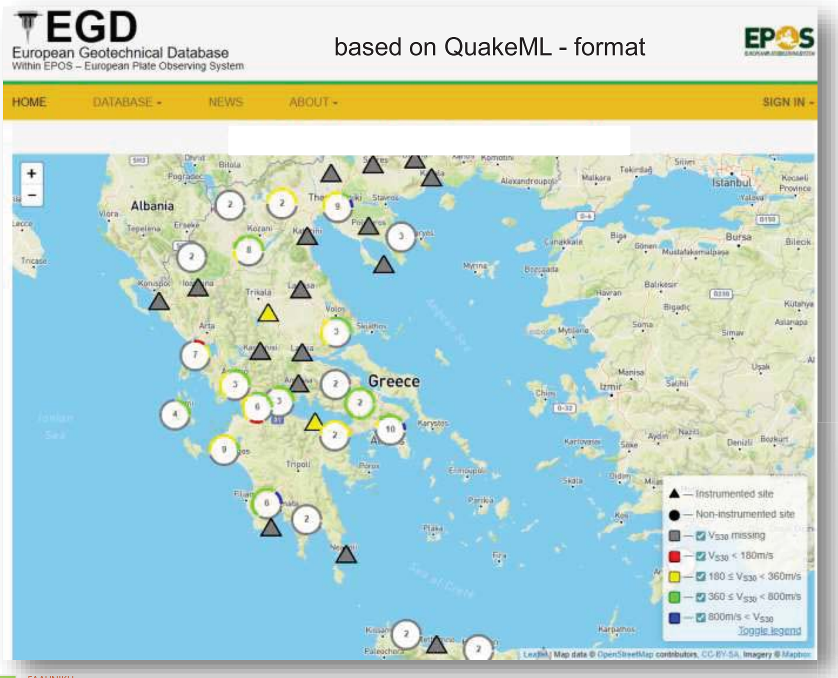
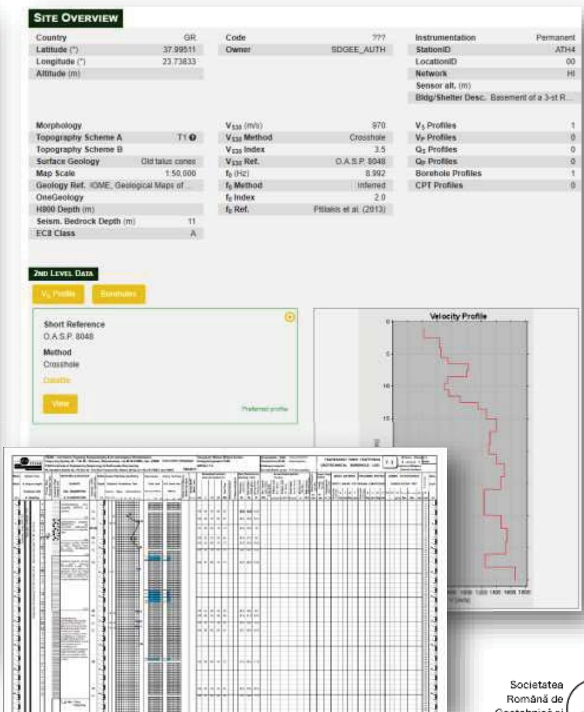
European Geotechnical Database (EGD) http://egd-epos.civil.auth.gr/





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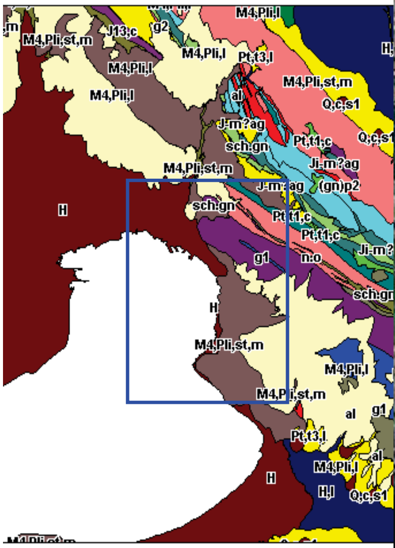
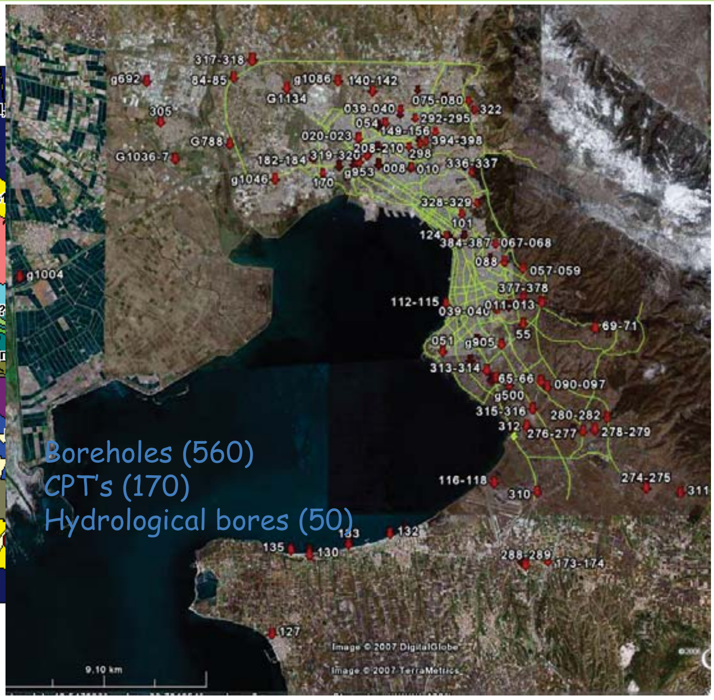
Collection - Valuation & Organization of Data

Geology





>>700bores (15-50m)
>>20000m

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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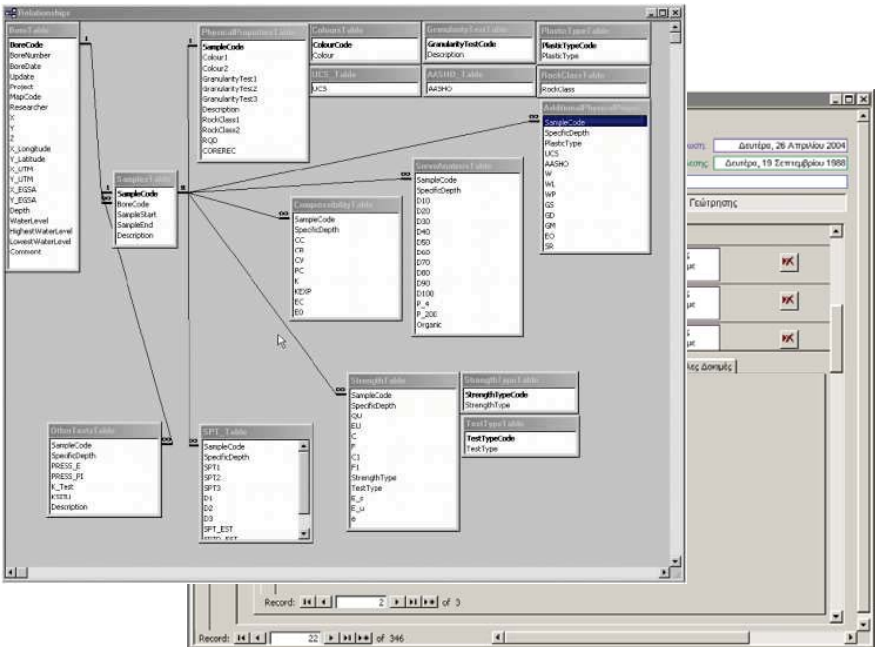
Collection - Valuation & Organization of Data

Database of Geotechnical Data

Data structure :

- Borehole-Site (21 fields)
- Samples (76 fields)
- simple - expandable
- 'Total layer description'
- Valuation-Validation
- Data at multiple depths (lab. & InSitu tests)
- Joined with GIS
- Spatial & user defined search



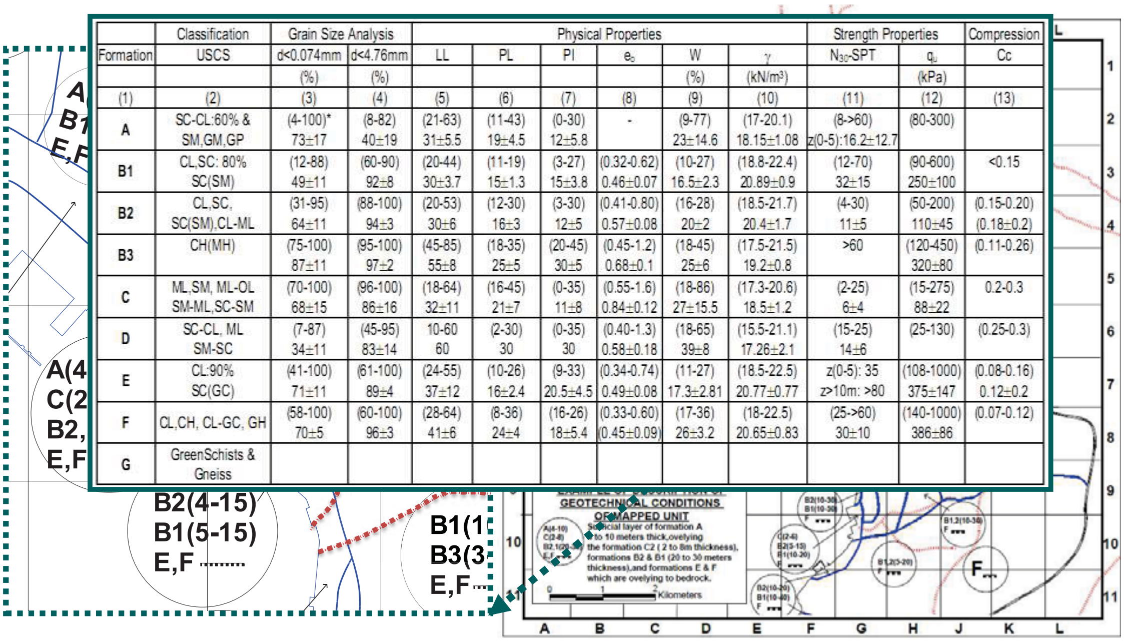
ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

Română de Geotehnică şi Fundaţii

Thessaloniki: Geotechnical Zonation

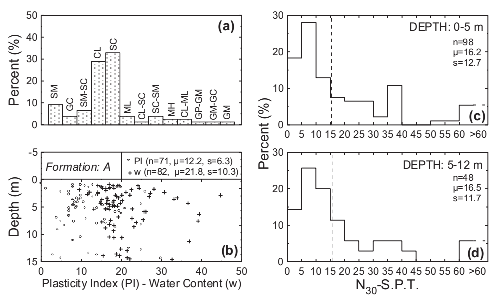
Mean physical & Mechanical Properties

	Classification	Grain Size Analysis		Physical Properties						Strength Properties		Compression
Formation	USCS	d<0.074mm	d<4.76mm	LL	PL	PI	e_o	W	γ	N_{30}-SPT	q_u	Cc
		(%)	(%)					(%)	(kN/m³)		(kPa)	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
A	SC-CL:60% & SM,GM,GP	(4-100)* 73±17	(8-82) 40±19	(21-63) 31±5.5	(11-43) 19±4.5	(0-30) 12±5.8	-	(9-77) 23±14.6	(17-20.1) 18.15±1.08	(8->60) z(0-5):16.2±12.7	(80-300)	
B1	CL,SC: 80% SC(SM)	(12-88) 49±11	(60-90) 92±8	(20-44) 30±3.7	(11-19) 15±1.3	(3-27) 15±3.8	(0.32-0.62) 0.46±0.07	(10-27) 16.5±2.3	(18.8-22.4) 20.89±0.9	(12-70) 32±15	(90-600) 250±100	<0.15
B2	CL,SC, SC(SM),CL-ML	(31-95) 64±11	(88-100) 94±3	(20-53) 30±6	(12-30) 16±3	(3-30) 12±5	(0.41-0.80) 0.57±0.08	(16-28) 20±2	(18.5-21.7) 20.4±1.7	(4-30) 11±5	(50-200) 110±45	(0.15-0.20) (0.18±0.2)
B3	CH(MH)	(75-100) 87±11	(95-100) 97±2	(45-85) 55±8	(18-35) 25±5	(20-45) 30±5	(0.45-1.2) 0.68±0.1	(18-45) 25±6	(17.5-21.5) 19.2±0.8	>60	(120-450) 320±80	(0.11-0.26)
C	ML,SM, ML-OL SM-ML,SC-SM	(70-100) 68±15	(96-100) 86±16	(18-64) 32±11	(16-45) 21±7	(0-35) 11±8	(0.55-1.6) 0.84±0.12	(18-86) 27±15.5	(17.3-20.6) 18.5±1.2	(2-25) 6±4	(15-275) 88±22	0.2-0.3
D	SC-CL, ML SM-SC	(7-87) 34±11	(45-95) 83±14	10-60 60	(2-30) 30	(0-35) 30	(0.40-1.3) 0.58±0.18	(18-65) 39±8	(15.5-21.1) 17.26±2.1	(15-25) 14±6	(25-130)	(0.25-0.3)
E	CL:90% SC(GC)	(41-100) 71±11	(61-100) 89±4	(24-55) 37±12	(10-26) 16±2.4	(9-33) 20.5±4.5	(0.34-0.74) 0.49±0.08	(11-27) 17.3±2.81	(18.5-22.5) 20.77±0.77	z(0-5): 35 z>10m: >80	(108-1000) 375±147	(0.08-0.16) 0.12±0.2
F	CL,CH, CL-GC, GH	(58-100) 70±5	(60-100) 96±3	(28-64) 41±6	(8-36) 24±4	(16-26) 18±5.4	(0.33-0.60) (0.45±0.09)	(17-36) 26±3.2	(18-22.5) 20.65±0.83	(25->60) 30±10	(140-1000) 386±86	(0.07-0.12)
G	GreenSchists & Gneiss											

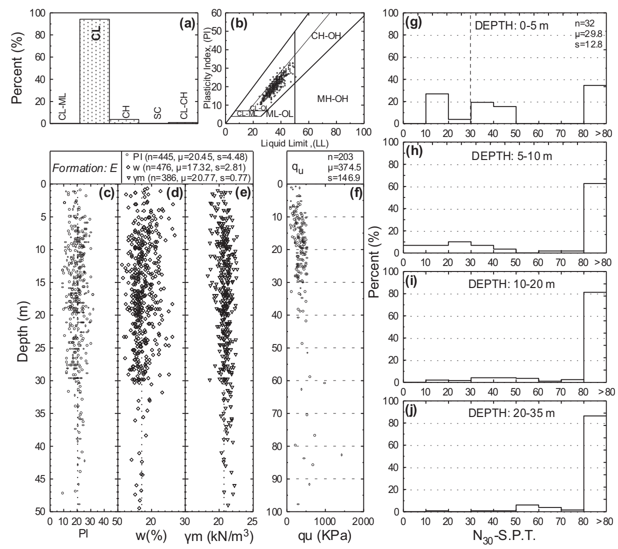


Thessaloniki: Elaboration of Physical & Mechanical Characteristics

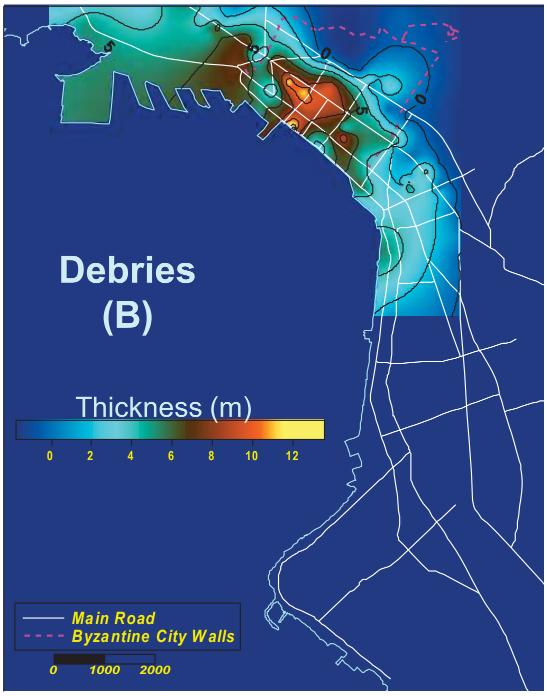
Soil Formation A- Artificial Fills-Debris

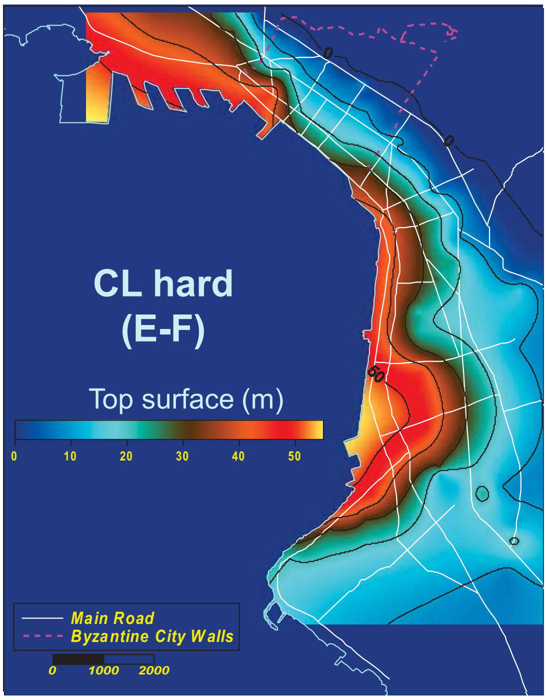


Soil Formation E - Very Stiff to hard Clays



Thessaloniki: Soil and Site Characterization





ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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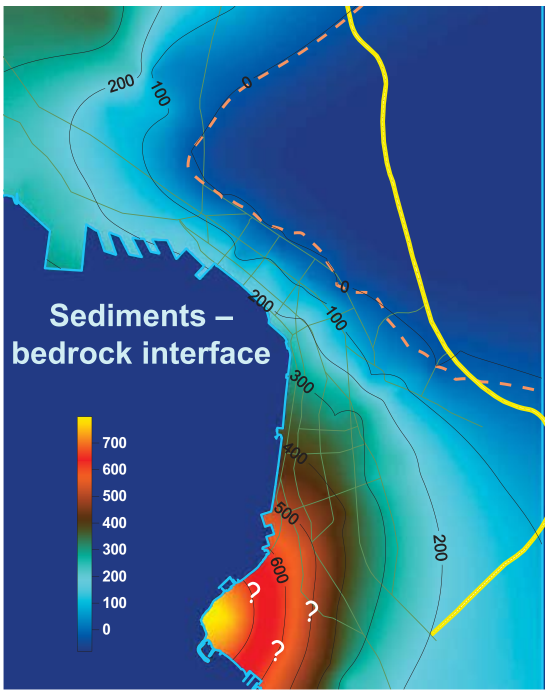
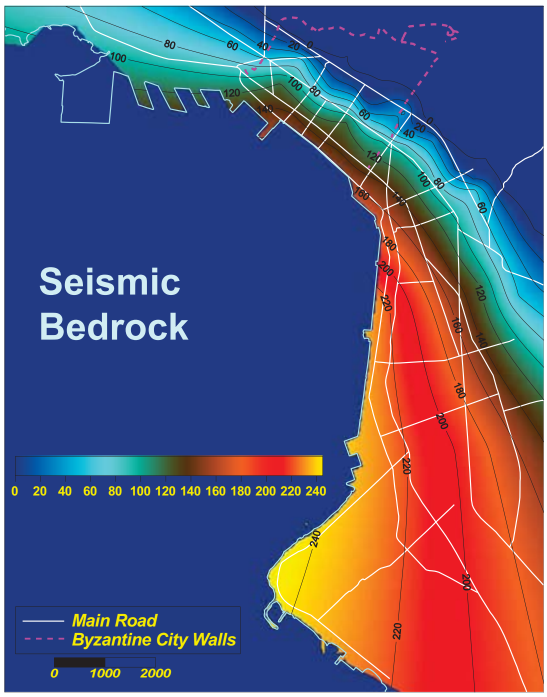
Thessaloniki: Soil and Site Characterization





ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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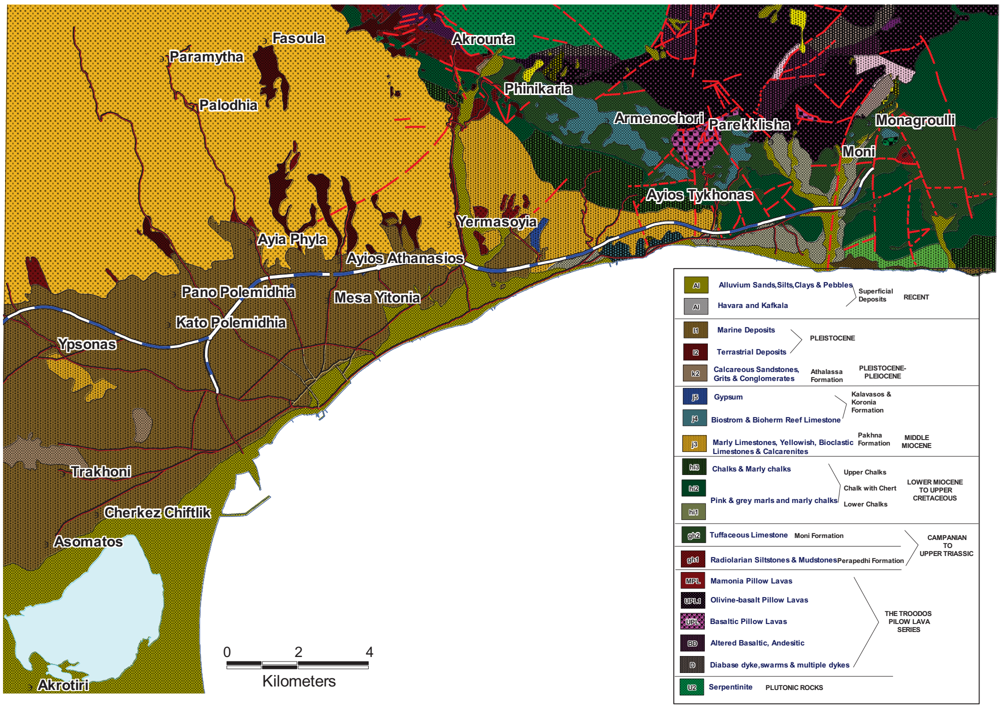
Lemesos – Geological Map



ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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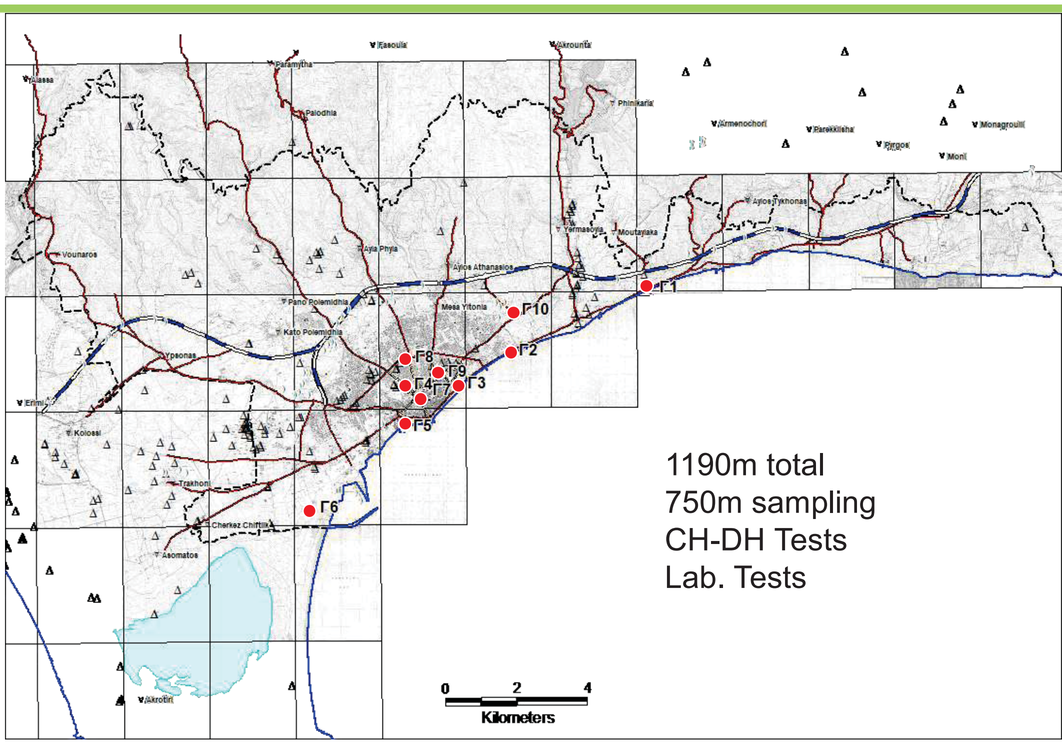
Lemesos: In Situ and Laboratory tests & Seismic prospecting



1190m total
750m sampling
CH-DH Tests
Lab. Tests

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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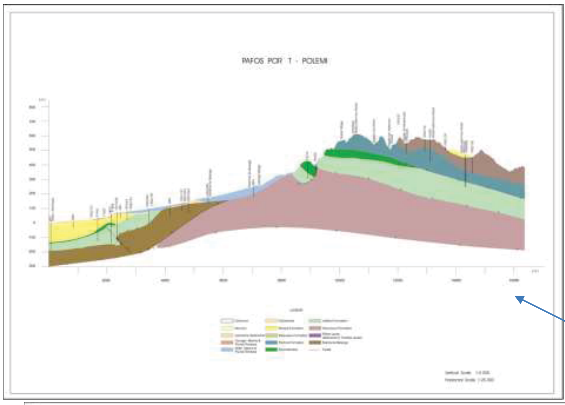
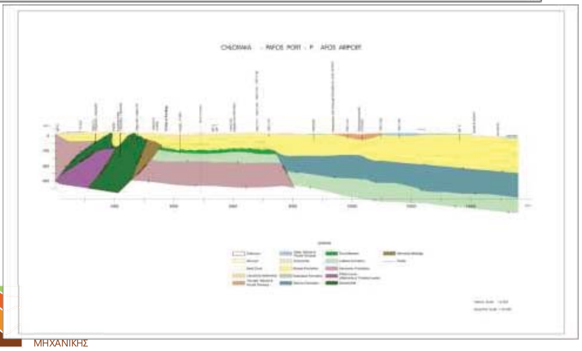
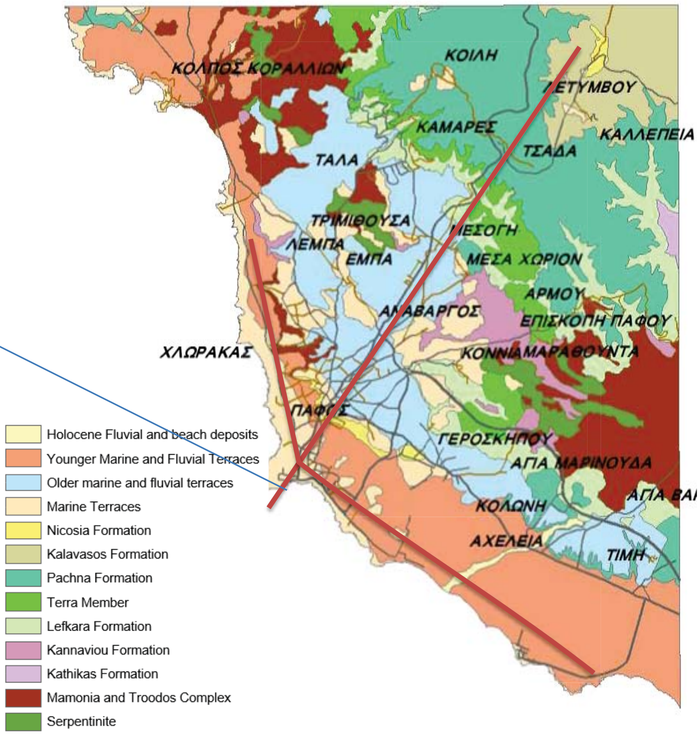
Pafos – Geological Map







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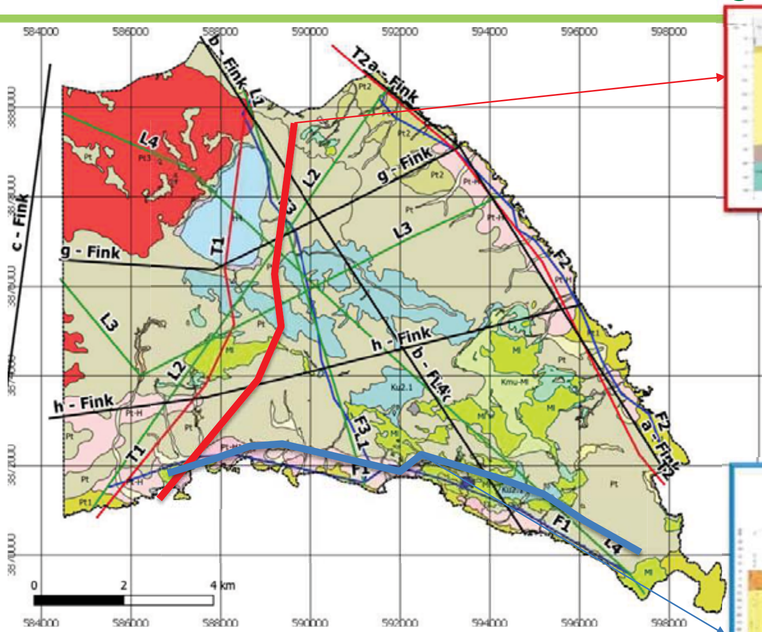
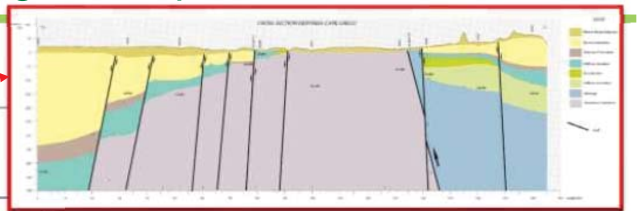
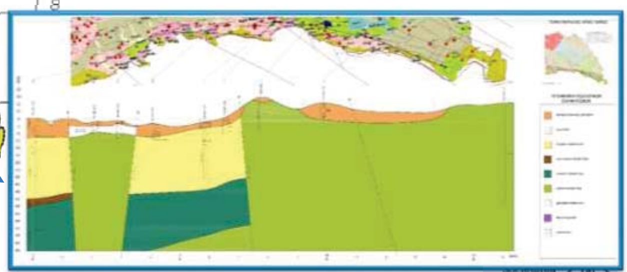
Ammochostos – Geological Map







ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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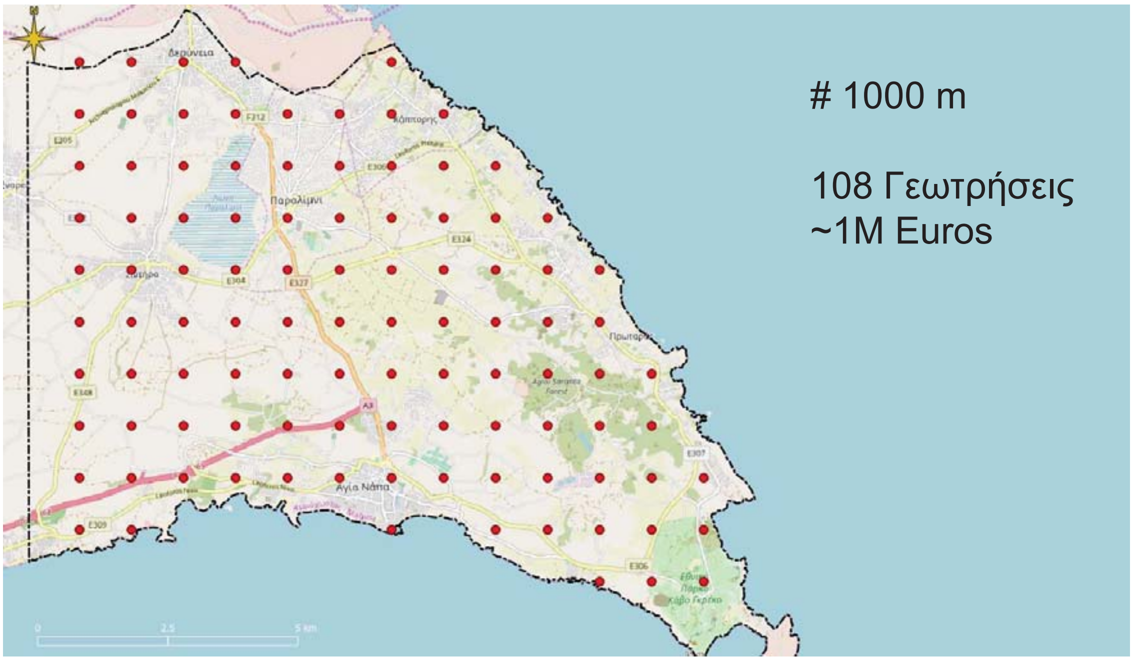
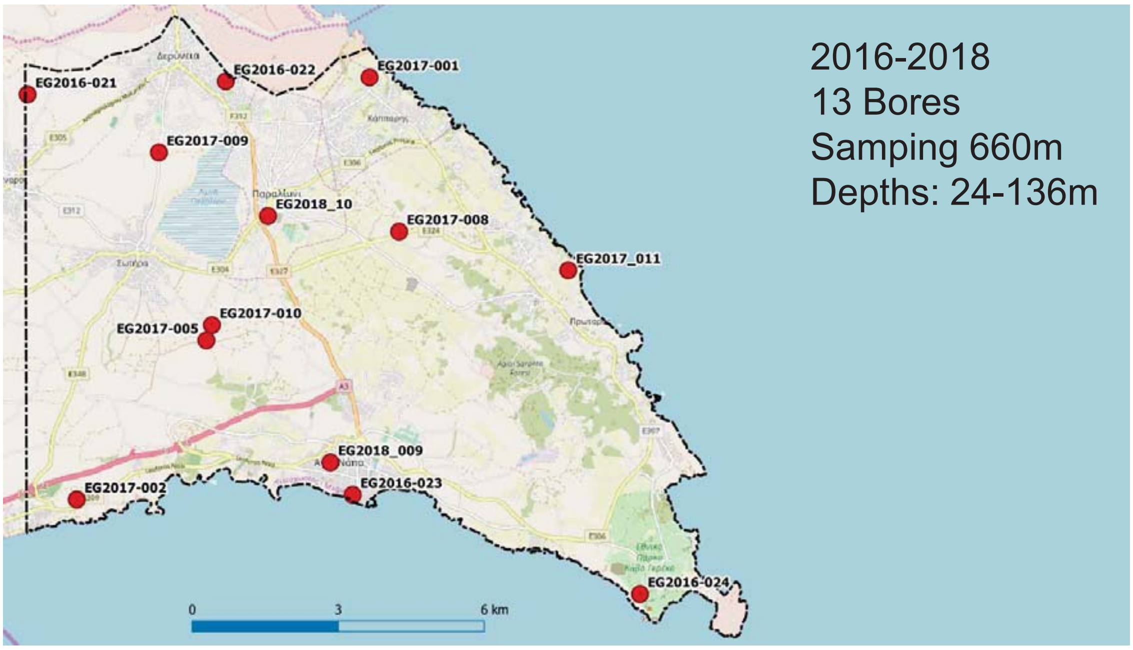
Ammochostos: Geotechnical Data



1000 m

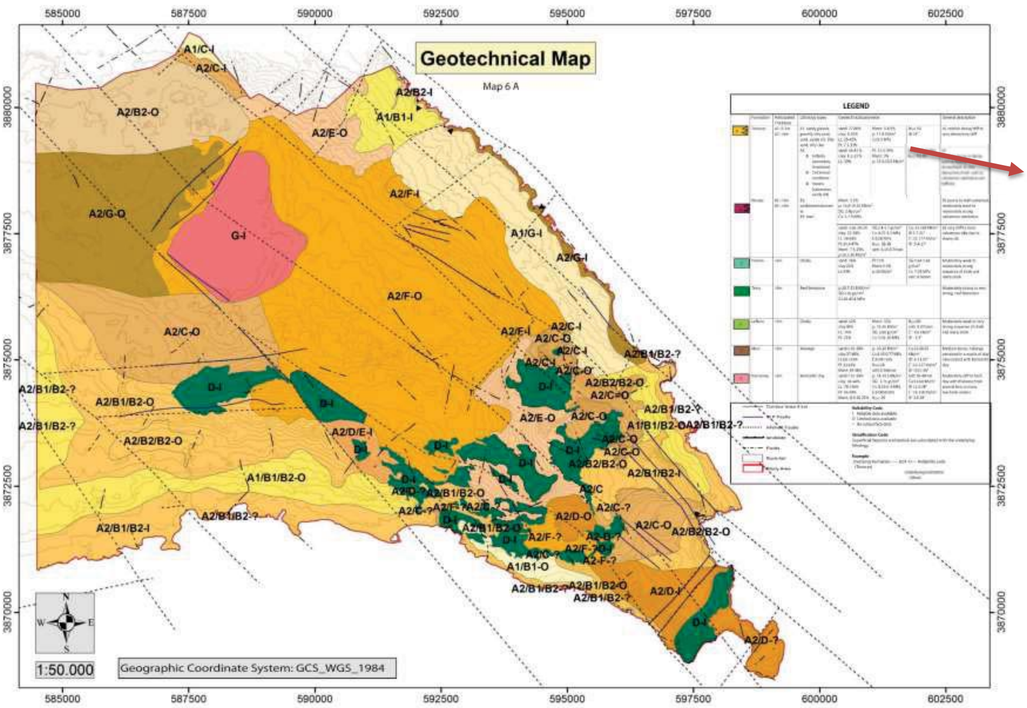
108 Γεωτρήσεις
~1M Euros

Additional In Situ and laboratory tests & Seismic prospecting



2016-2018
13 Bores
Samping 660m
Depths: 24-136m

Ammochostos: Final Geotechnical Map



thickness
Strength Parameters
Density
Atterberg Parameters
Consolidation Parameters

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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Ammochostos: Main soil formations

Formation: Kannaviou

n.Boreholes: 3 | n.Samples: 12 | Depth: 3.0 - 23.5

	Gravels	Sand	Silt	Clay	w	Fines	LL	PL	IP	γd	γ	Cu	φ	Qu	z_SPT	N30-SPT
	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)		(kN/m^3)	(kN/m^3)	(kN/m^2)	(°)	(Mpa)	(m)	
n	5	5	5	5	12	5	5	5	5	12	8	3	3	3	10	10
min	0	0.0	25.0	33.0	18.0	90.0	50.0	26.0	24.0	11.7	18.4	141.7	13.0	0.31	3.0	15.0
max	0	10.0	65.0	75.0	47.0	100.0	100.0	63.0	46.0	17.4	21.5	295.8	20.0	2.40	20.0	40.0
Average	0	2.6	42.4	55.0	29.7	97.4	80.8	45.2	35.6	14.9	19.8	197.5	15.6	1.05		26.6
StDev	0	4.2	18.8	18.7	8.0	4.2	21.4	15.6	8.4	1.83	0.86	85.41	3.81	1.17		8.8
Xk												154.8	13.7	0.5		22.2

Notes: R_values: 0

Formation: Nicosia

n.Boreholes: 5 | n.Samples: 52 | Depth: 4.5-39.0

	Gravels	Sand	Silt	Clay	w	Fines	LL	PL	IP	γd	γ	Cu	φ	Qu	z_SPT	N30-SPT
	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)		(kN/m^3)	(kN/m^3)	(kN/m^2)	(°)	(Mpa)	(m)	
n	18	18	18	18	33	18	18	18	18	44	9	11	11	13	34	34
min	0	1.0	18.0	18.0	28.7	59.0	50.0	23.0	21.0	9.5	16.7	13.5	0.7	0.02	4.5	14.0
max	0	41.0	65.0	80.0	64.4	99.0	160.0	123.0	47.0	15.5	19.0	177.0	37.9	1.62	21.5	55.0
Average	0	12.5	38.4	49.2	38.6	87.6	74.1	43.8	30.2	13.4	18.4	76.2	14.9	0.46		31.9
StDev	0	12.5	11.0	19.0	8.4	12.6	27.7	23.4	7.2	1.38	0.74	57.65	11.45	0.53		11.9
Xk												47.4	9.2	0.2		26.0

Notes: R_values: 0

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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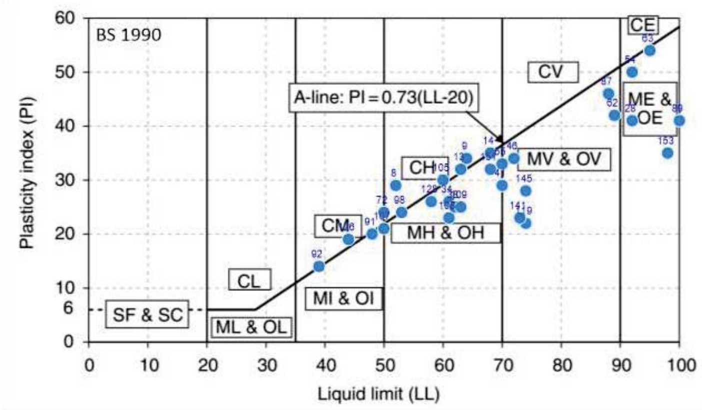
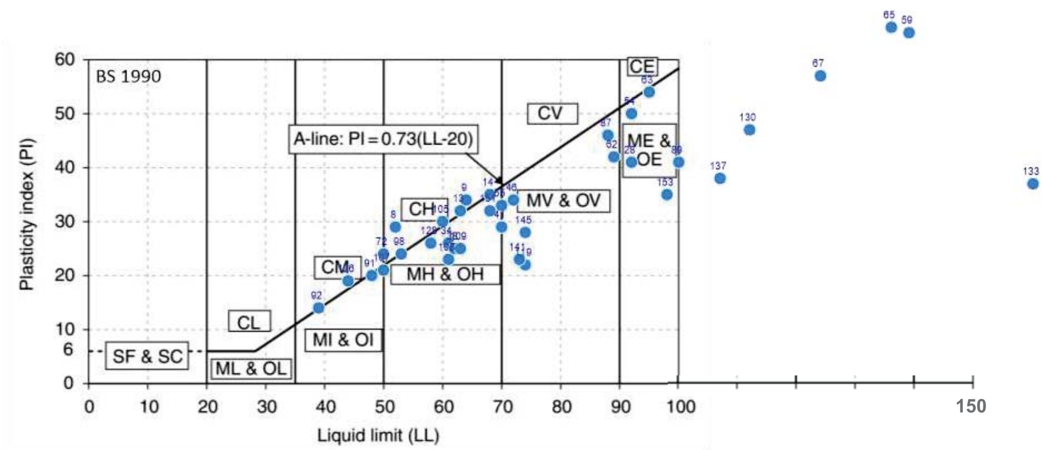
Ammochostos and Pafos: High Plasticity Clays



High, Very High, to Extremely High Plasticity Clays and Silts

Ammochostos and Pafos: High Plasticity Clays



High, Very High, to Extremely High Plasticity Clays and Silts

Lessons Learned

Collection - Valuation & Organization of Data

- Geotechnical data is naturally Geospatial data : Digital format needed (AGS. DIGGS, GEF)
- Structure of DataBase/GIS has to be simple - expandable form - allowing total description of layer and original Borehole log
- Input of data: preliminary elaboration of type of soil, so..
  geotechnical engineers with experience and a good knowledge of the site's geology
- Geotechnical reports often not include metadata = data related to in situ tests (SPT, CPT, etc.)

2nd Greek – Romanian Seminar: Lessons learned from Earthquakes and Geotechnical Failures

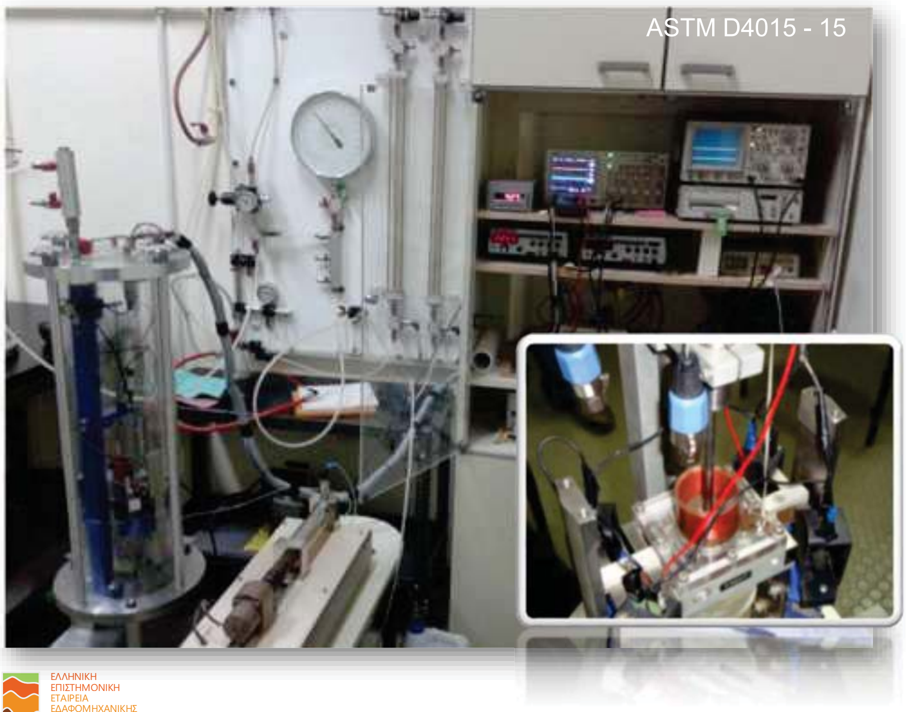
In situ and laboratory dynamic tests

Validation of soil properties - Correlations

Dynamic Properties of Main Soil Formations

Subsurface Structure and Dynamic Soil Properties as Basis for Microzonation and Site Effects Studies: Lessons Learned from Studies in Greece and Cyprus
Anastasiadis and Pitilakis
Societatea Română de Geotehnică și Fundații

Laboratory Equipment: RC Device



- Resonant Column Apparatus – free-fixed type
- Type: true Fixed – Free)
- Loading/Vibration: Longitudinal and Torsional vibration
- Low Strain amplitude Tests with time may be performed at clayey specimens.
- Optional: Measure of P and S-wave using BE testing
- Strain amplitudes: depend on specimen size and material stiffness from 10^{-6} to 2.10^{-3}
- Sample Sizes: Solid Cylindrical Diameter 36mm or 71mm (height 80mm or 142mm)
- Frequency Range: 0 to 400Hz
- Max σ_3: 700kPa
- Accuracy of Specimen's height : 0.001mm
- Accuracy Resonant frequency : 0.01Hz
- Accuracy of Cell - Pore water Pressure: 1kPa
- Accuracy of specimen's Volume : 1mm3

>> 1500 tests

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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Laboratory Equipment: CTX Device

- Fully PC digitally controlled and acquisition
- Max Frequency 5Hz
- Max Load Range: 5kN – for 50mm diameter corresponds @2500kPa q-stress (for 70mm diameter corresponds to 1250kPa)
- max σ_3 : 1000kPa for dynamic, 3400kPa for static loading
- max Axial Displacement: +/-50mm
- max Double Cyclic Axial Amplitude: 8mm @2Hz, 12mm @1Hz, 100mm @0.1Hz.
- Sampling: up to 1000 points per cycle.
- Measurement & control accuracy :
  - Axial Displacement: 0.2μm
  - Axial Loading: < 0.1% of max loading (5kN), (i.e. 8N for 8kN; 16bit resolution ≈ 0.3N)
  - Pore Pressure/Volume: 1kPa and $1mm^3$.
- Sample Sizes: D50mm, D70mm and D100mm (H/D – 2:1)
- Optional: P and S-wave using BE testing

>> 1000 tests

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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Vs Wave Determination Tests: BE Testing – Representative Results

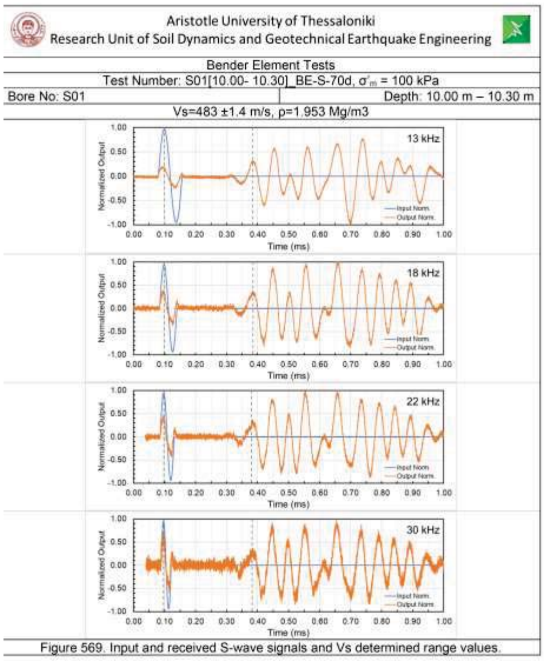

Figure 569. Input and received S-wave signals and Vs determined range values.

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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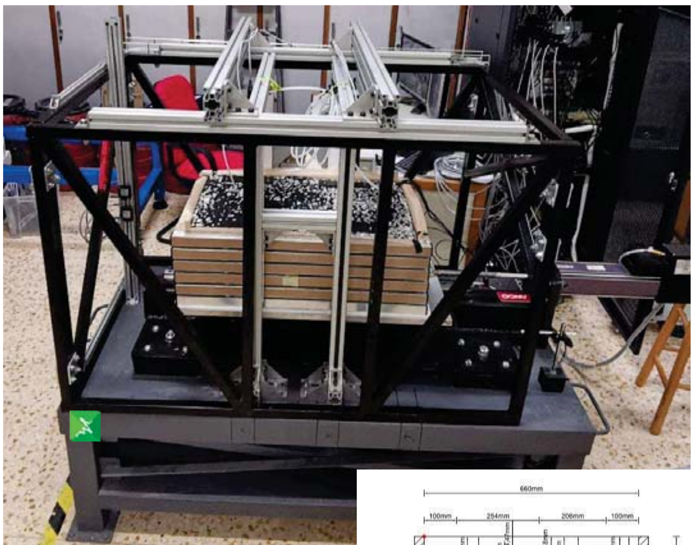
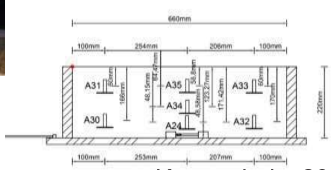
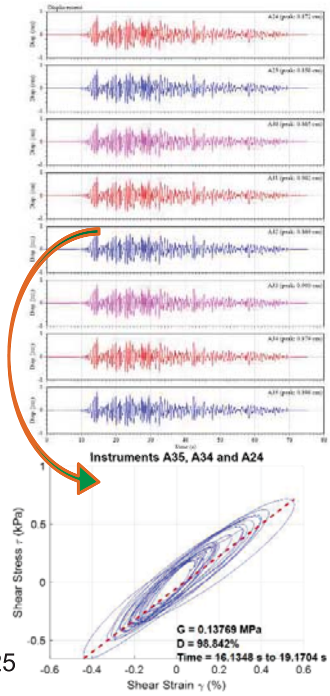
Shake Table Device







Kapouniaris, 2025

- Fully PC digitally controlled and acquisition
- Small Scale models
- 1.5 g @ 80kg payload
- Frequency: 0- 20 Hz
- Peak Displacement ±12cm
- Peak Velocity 50cm/s
- DAQ1: 16 Ch. 16 bit NI
- DAQ2: 16 Ch. 24 bit Kyowa 430A
- 2 Laminar Boxes height 50cm
- Sand Pluviation System
- Accelerometers
- Non-Contact Laser Distance
- Shear Hammer Device
- Bender Element Measurements

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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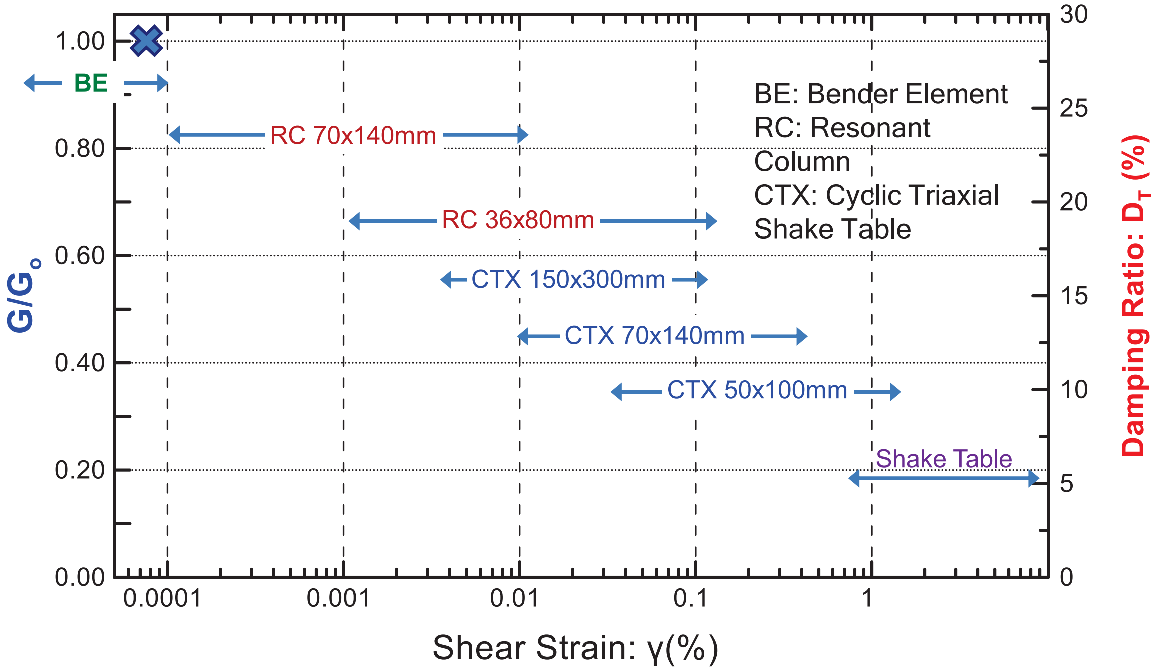
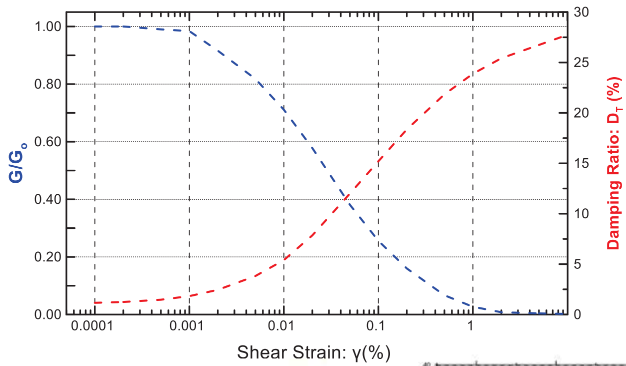
Combining BE, RC, CTX & Shake Table Tests

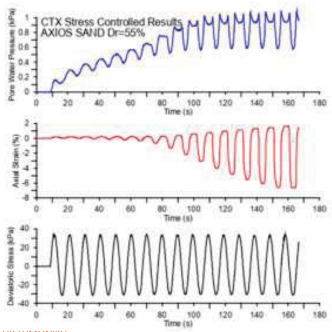


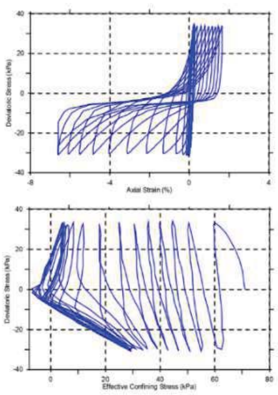
Normalized Shear Modulus (G/Go) vs shear strain (γ) 10^{-4} - 2 %

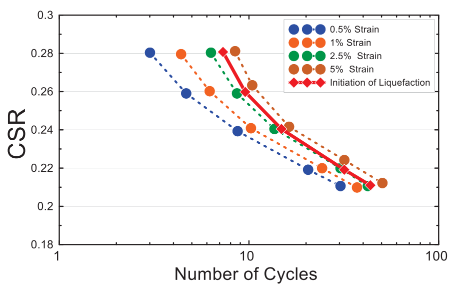
Combined BE, RC, CTX & Shake Table Tests – Representative Results



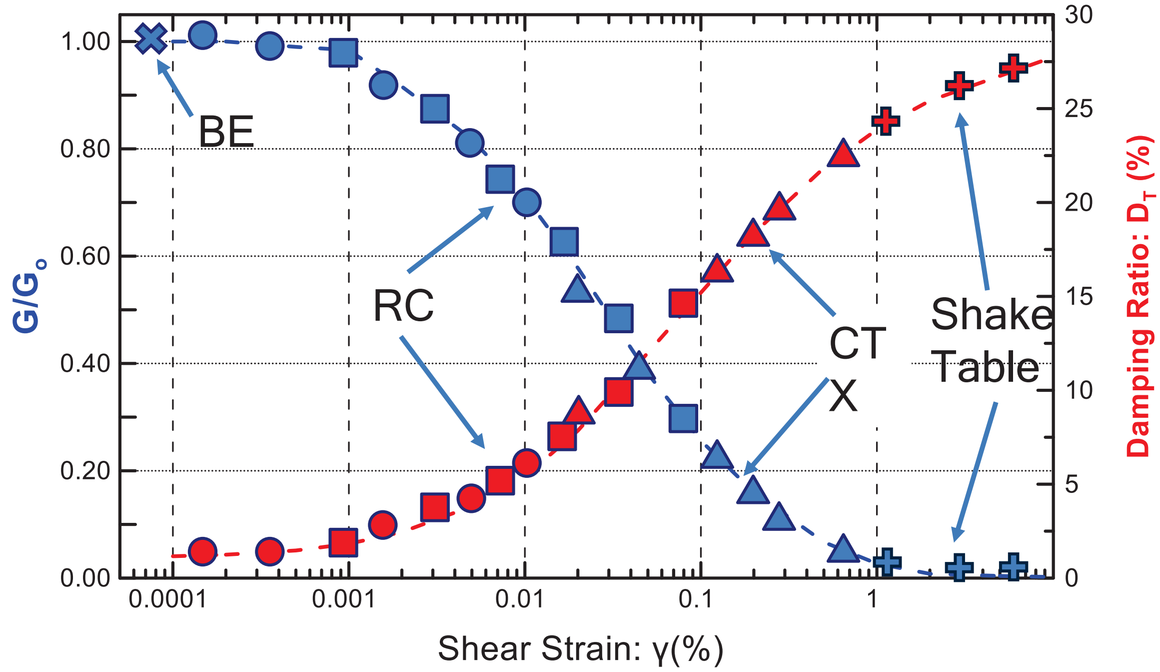
Aim: to determine
- Shear Modulus and Damping vs strain
- Strength Parameters
- Residual Deformations
  Excess Pore Water Behavior
- Liquefaction Potential







Combined BE, RC, CTX & Shake Table Tests – Representative Results



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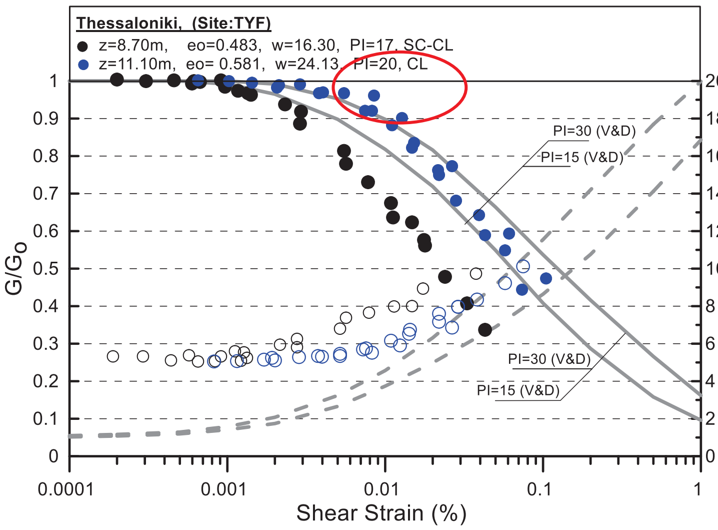
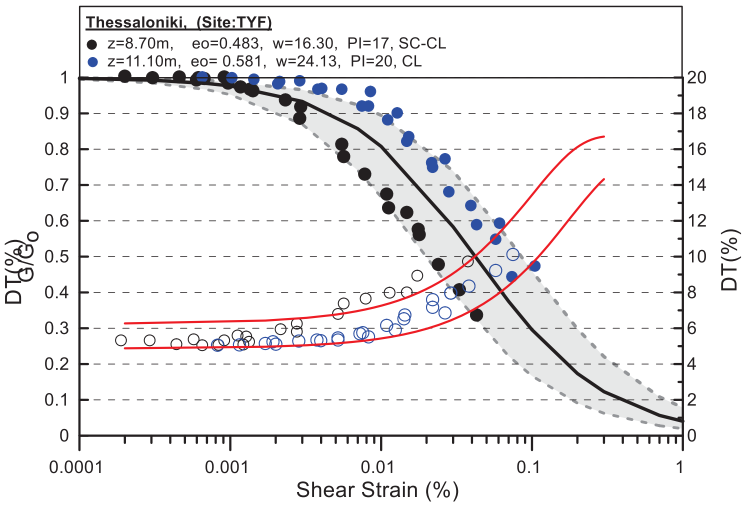
Non-Linear curves





Nonlinear properties are diverse. We usually consider a normal distribution.
The standard deviation is typically calculated using the Darendeli and Stokoe model, but this is not always the case.

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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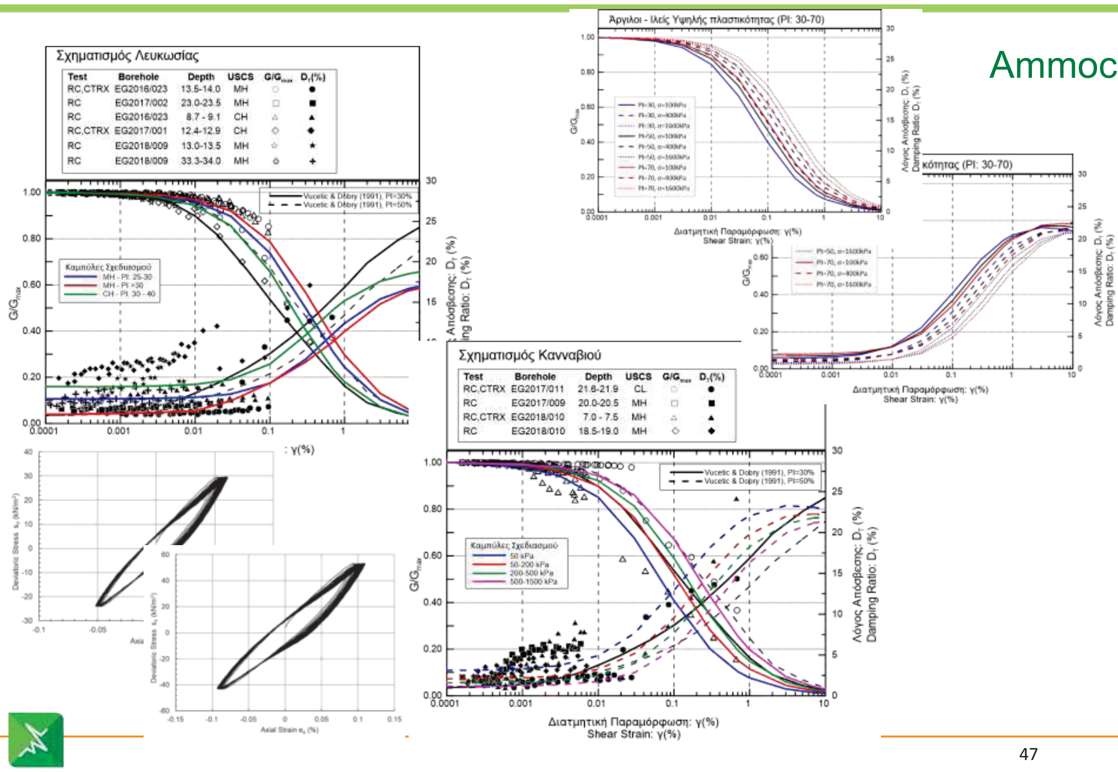
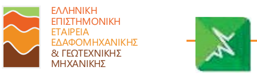
Construction of non-linear curves

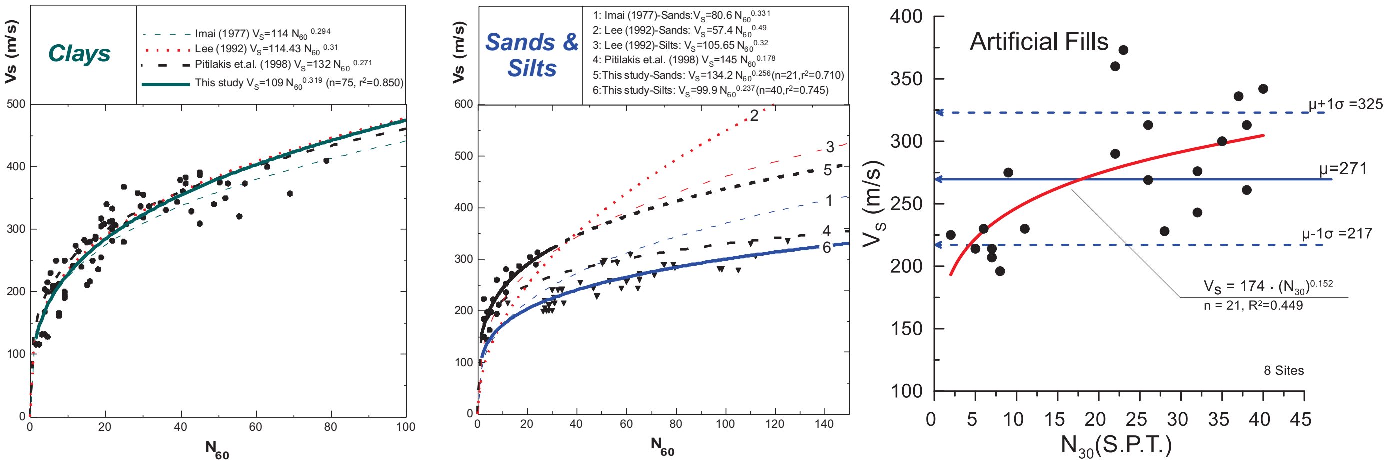
Ammochostos





Correlation Relationships

Vs - N(S.P.T. - Thessaloniki)



Design of Soil models

Site Response Analysis

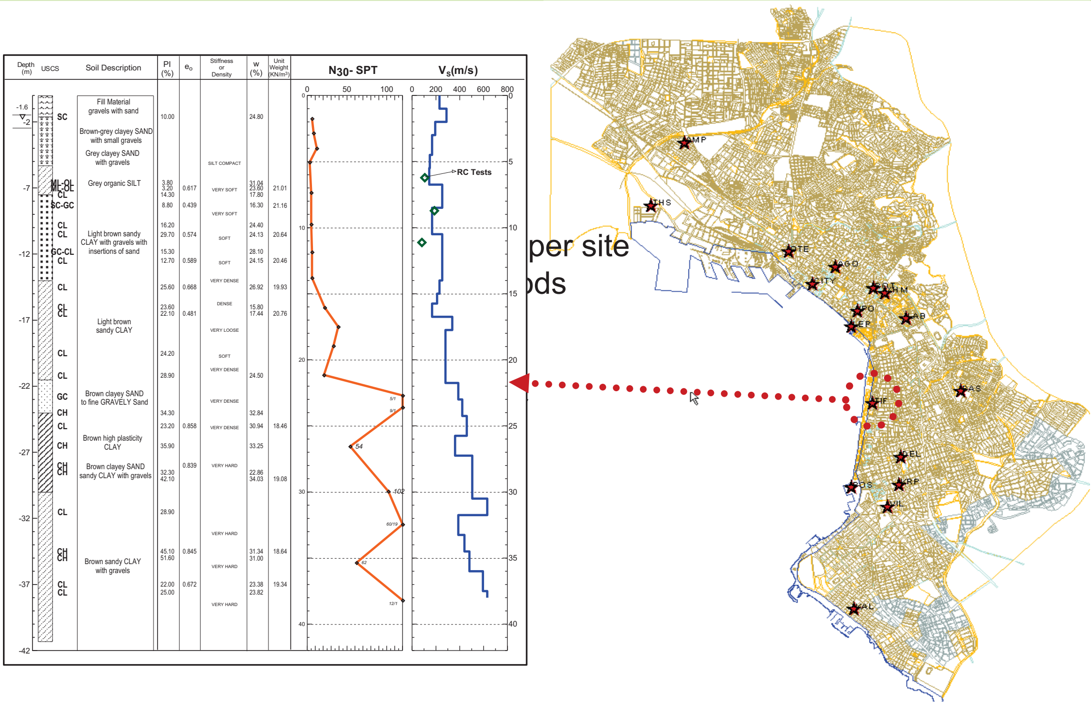
1D analysis is always the basic reference

Seismic Response

- Wave propagation is a Complex problem
- Depends on source (e) – path (p) – site effects (s)
- Local geology (1 / 2 / 3 dimensions)
- Approaches
  - Instrumental (SSR, HVSR, GIS)
  - theoretical (1D, 2D, 3D)
- Quantitative and qualitative evaluation of ground response parameters
- why, where and how is amplified the seismic motion ?

$$r(t) = e(t) * p(t) * s(t)$$

Thessaloniki: In situ and laboratory tests & Seismic prospecting



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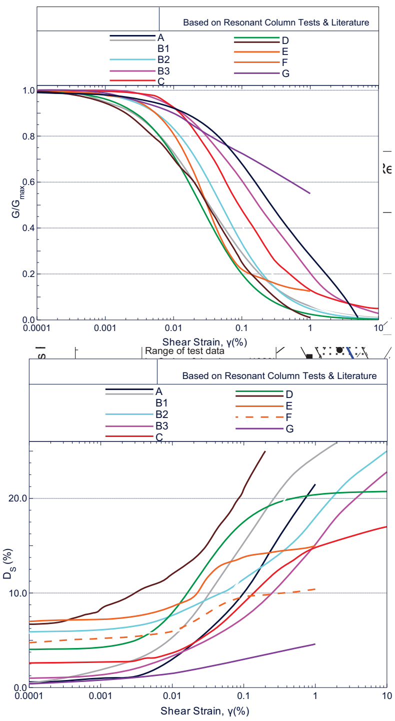
Evaluating the dynamic properties of main soil formations in Thessaloniki



G-γ%-Ds% curves for 9 different soil types in Thessaloniki

Formation (1)	Description (2)	V_S (m/s) (5)	V_P (m/s) (6)	Q_S (7)
	Surficial			
A	Artificial Fills, demolition materials & debris parts	200–350 (250)	400–1700	8–20 (15)
B1	Very Stiff sandy-silty clays to clayey sands, low plasticity	300–400 (350)	1900	15–20 (20)
B2	Soft sandy-silty clays to clayey sands, low to medium plasticity	200–300 (250)	1800	20–25 (20)
B3	Stiff to hard high plasticity clays	300–400 (350)	1800	20–40 (30)
C	Very soft buy mud and silty sands	120–220 (180)	1800	20–25 (25)
D	Alluvium deposits, sandy-silty clays to clayey sands-silts, low strength and high compressibility	150–250 (200)	1800	15–25 (20)
	Subbase			
E	Stiff to hard sandy-silty clays to clayey sands	350–700 (600)	2000	6–30 (30)
F	Very stiff to hard low to medium plasticity clays to sandy clays Overconsolidated with rubble and thin layers of gravels	700–850 (750)	3200	50–60 (60)
G	GreenSchists & Gneiss	1750–2200 (2000)	4500	180–200 (200)

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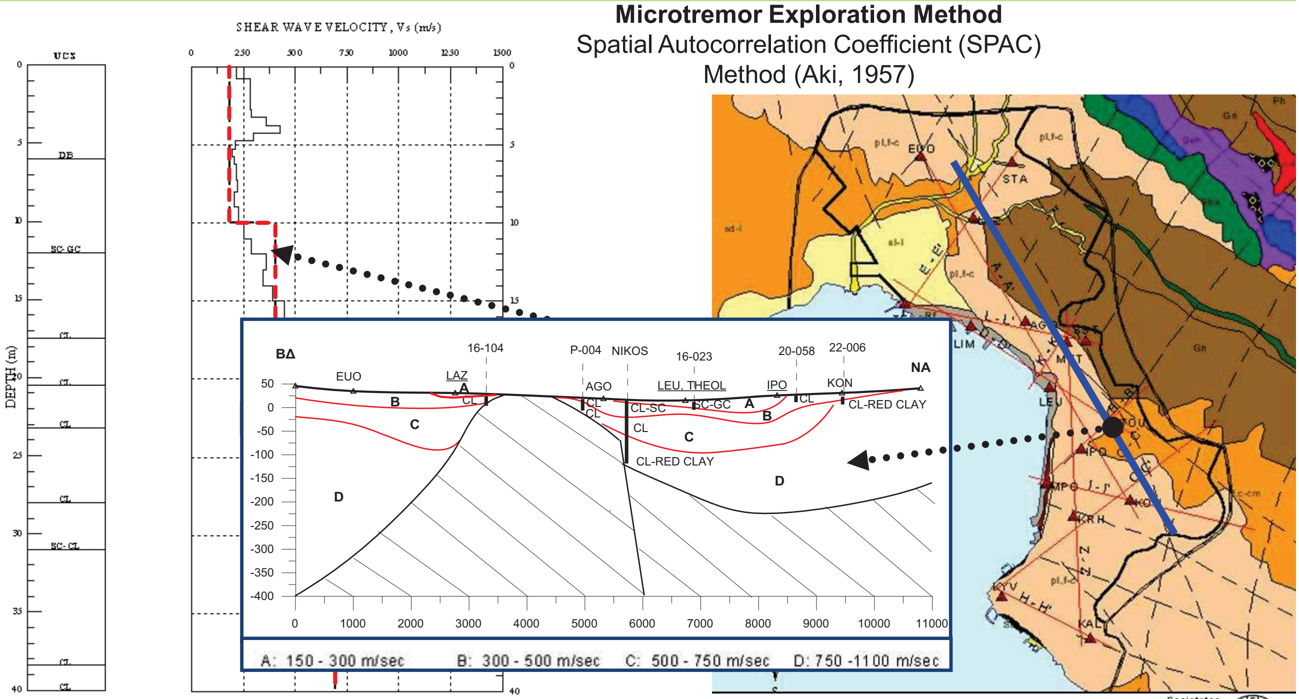
In Situ and laboratory tests & Seismic prospecting

Microtremor Exploration Method
Spatial Autocorrelation Coefficient (SPAC)
Method (Aki, 1957)



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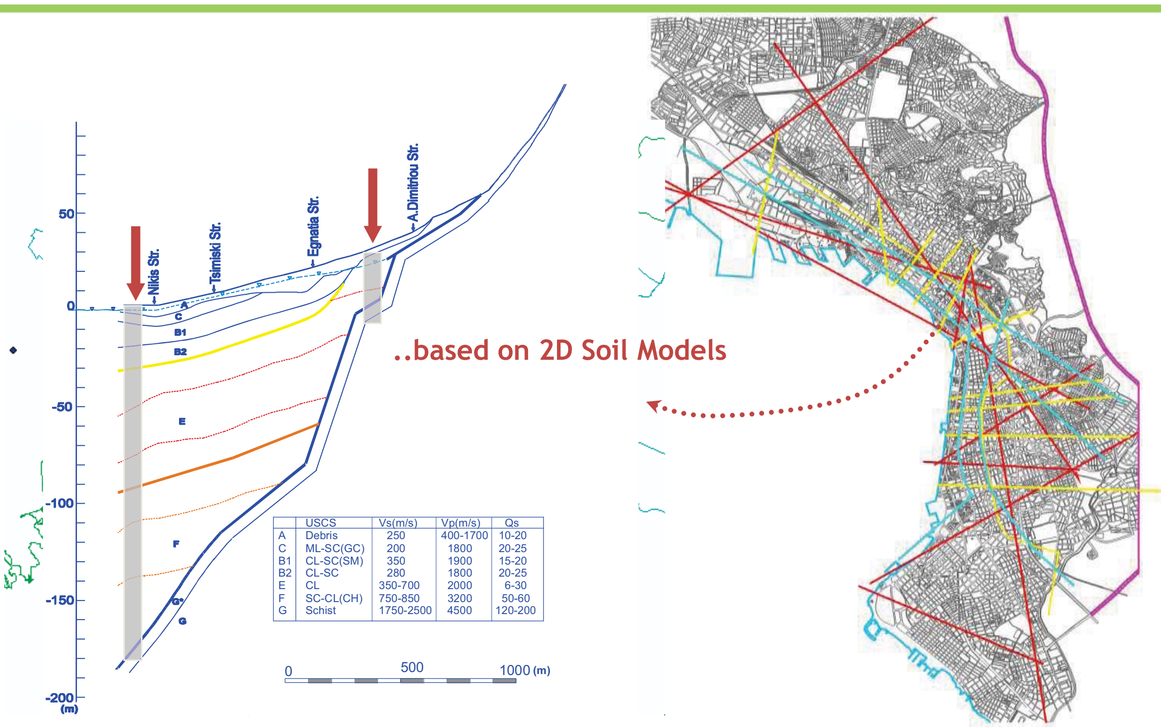
Thessaloniki: Construction of 1D soil models



	USCS	Vs(m/s)	Vp(m/s)	Qs
A	Debris	250	400-1700	10-20
C	ML-SC(GC)	200	1800	20-25
B1	CL-SC(SM)	350	1900	15-20
B2	CL-SC	280	1800	20-25
E	CL	350-700	2000	6-30
F	SC-CL(CH)	750-850	3200	50-60
G	Schist	1750-2500	4500	120-200

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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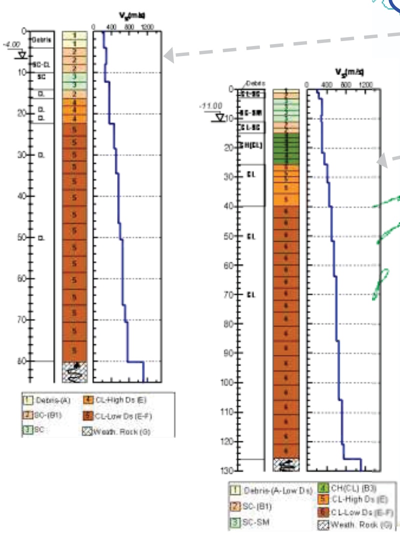
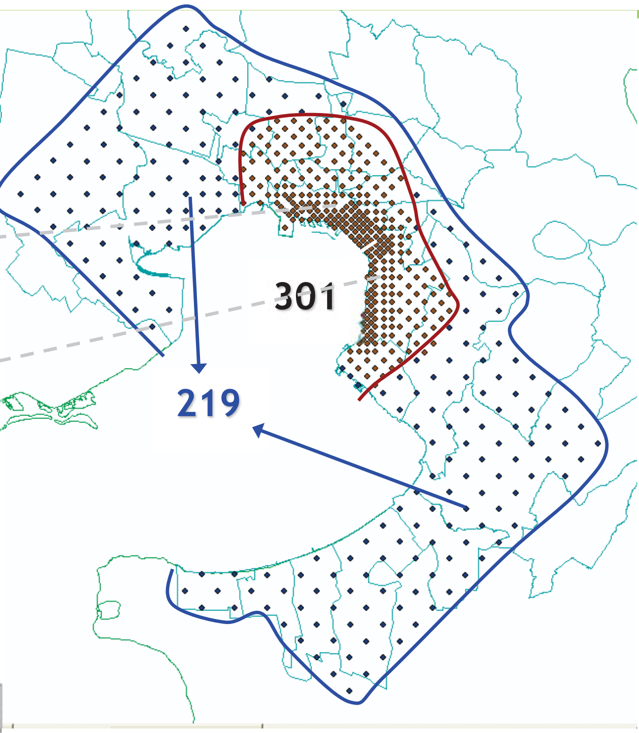
Thessaloniki: Construction of soil models

520 - 1D soil models

Grid size: 250x250, 500x500
1000x1000 m





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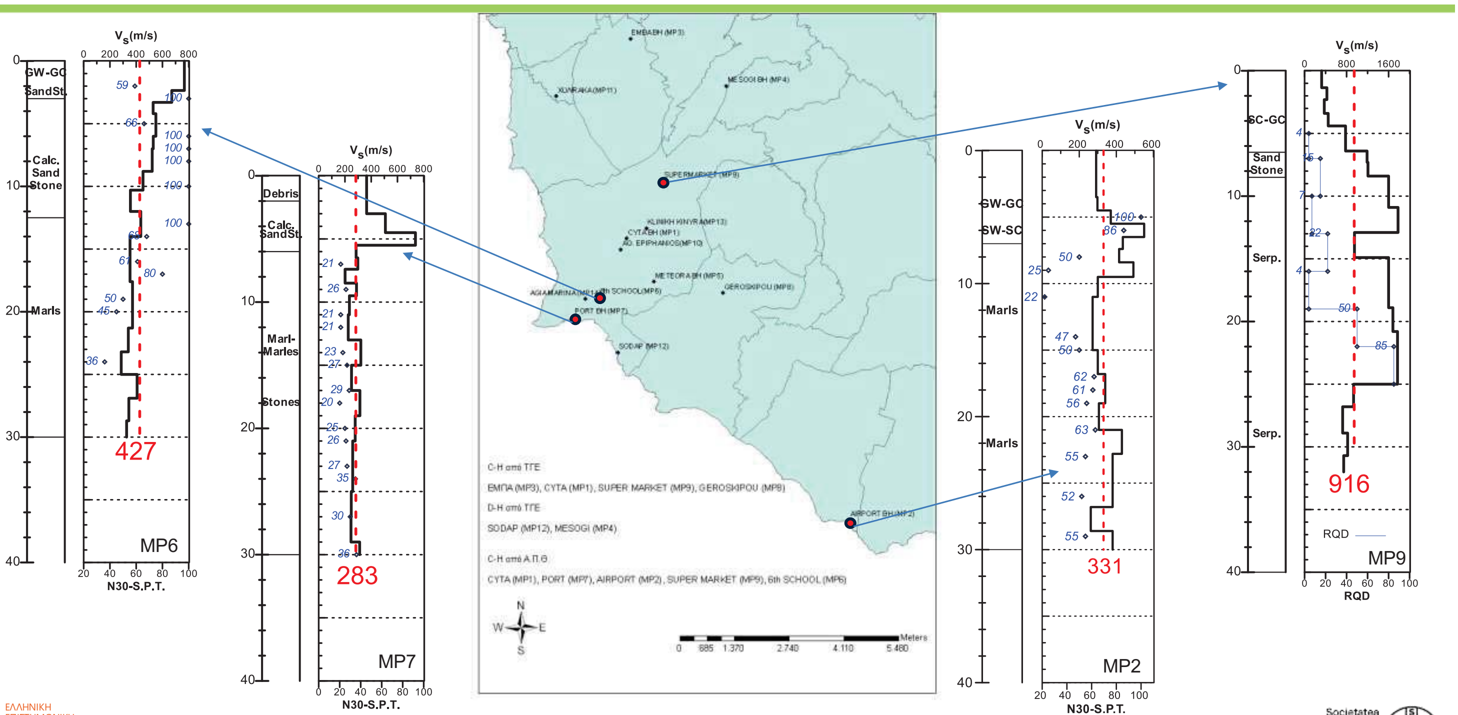
Pafos: In Situ and laboratory tests & Seismic Prospecting



Elevated Marine Terraces: Gradual Decrease of Vs with depth – Large Variability !

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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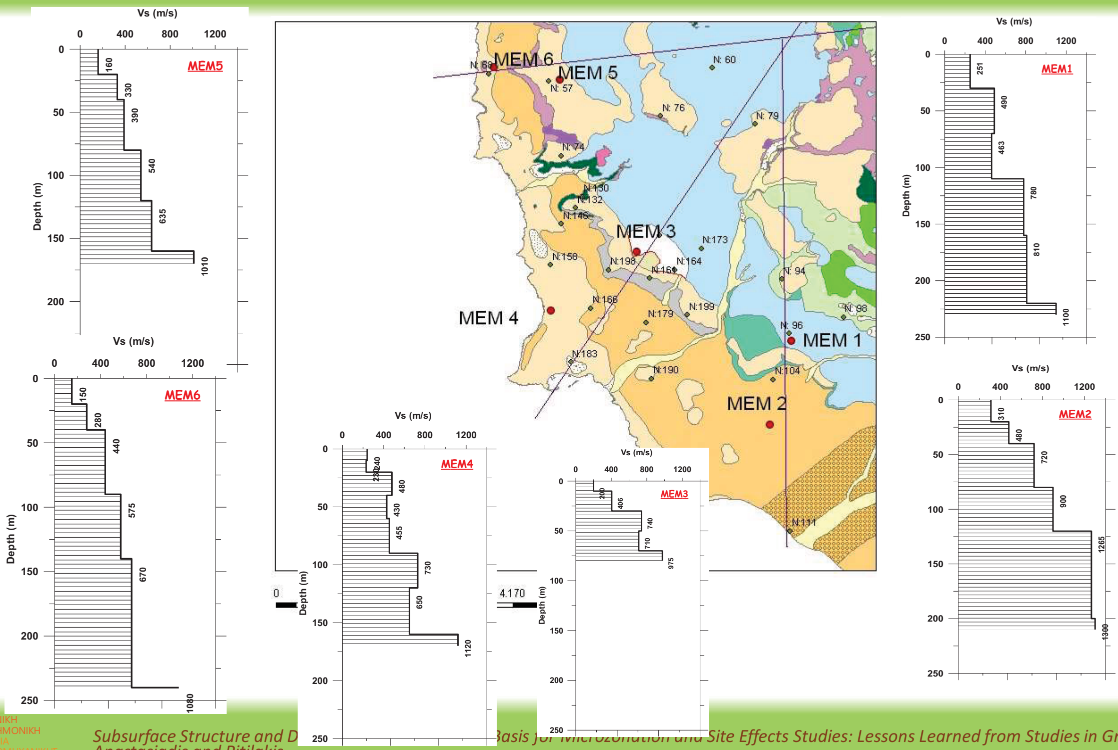
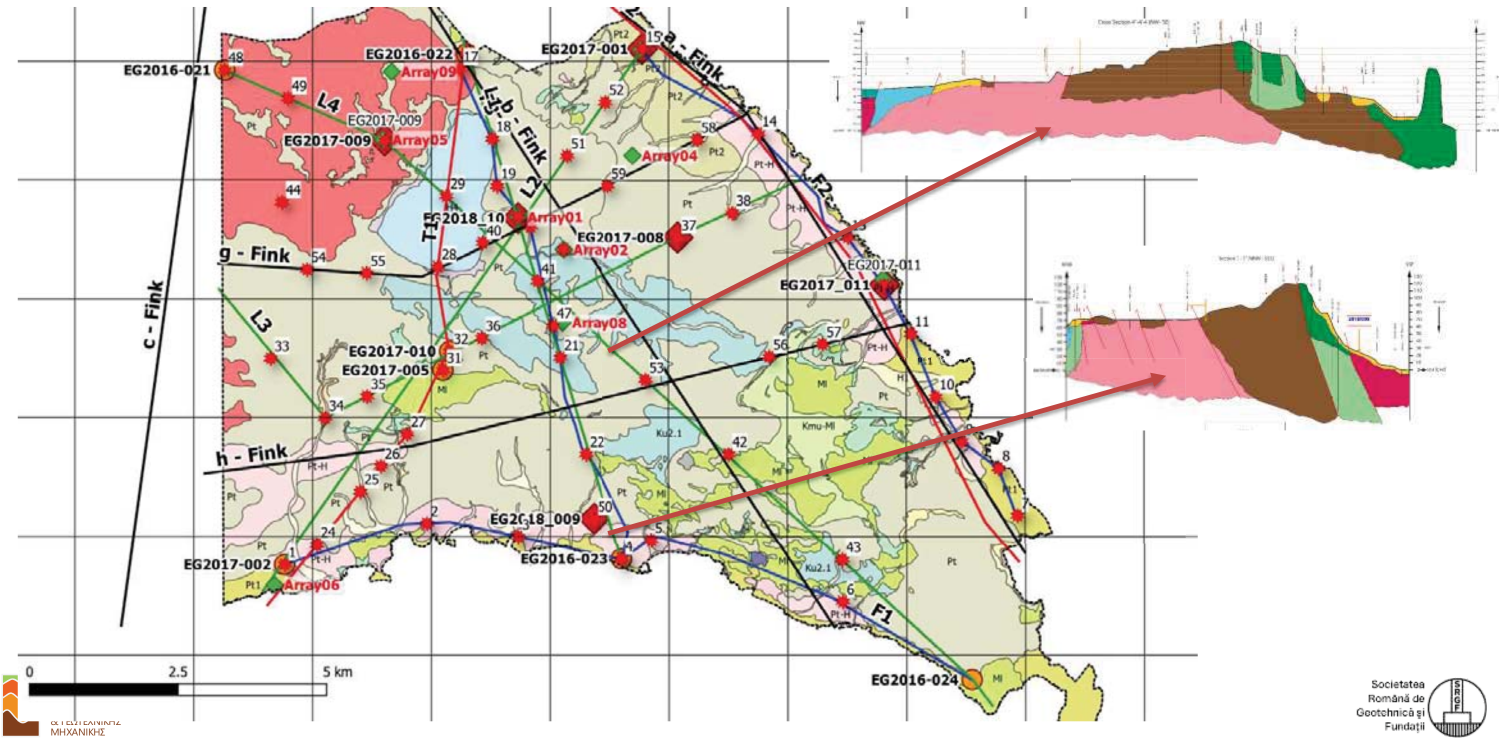


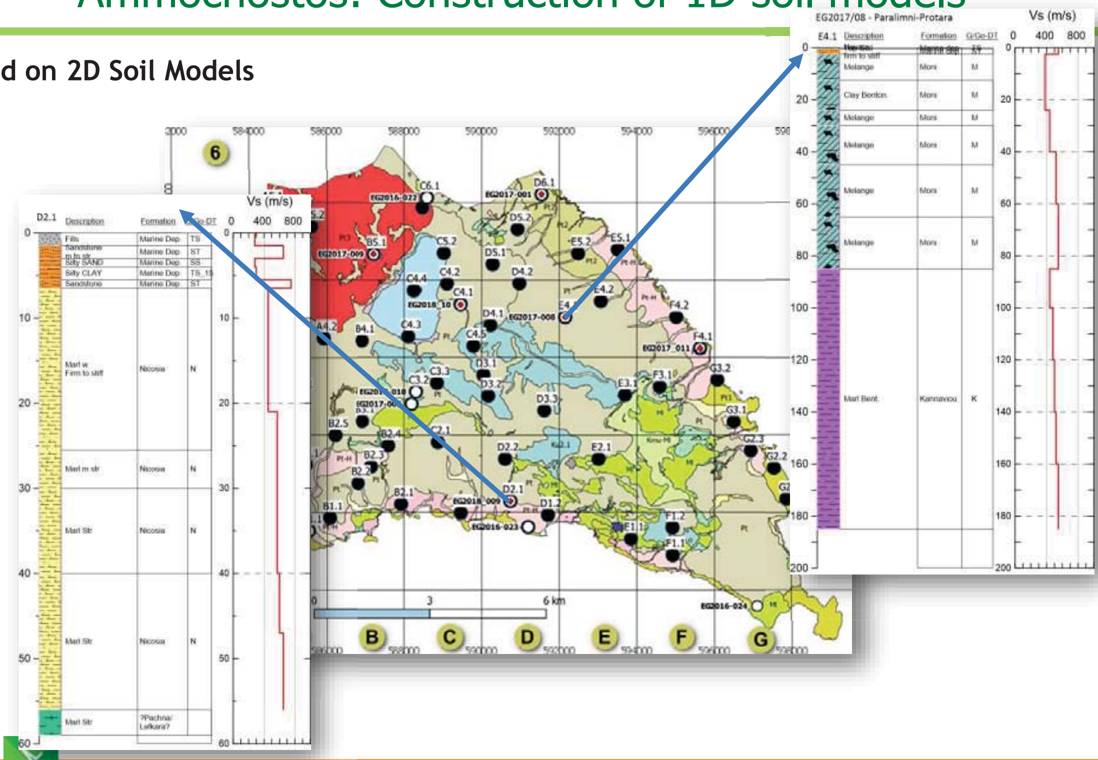
Array Measurements of Microtremors (MEM)

Ammochostos – Construction of 1D Soil Models



Ammochostos: Construction of 1D soil models

..based on 2D Soil Models



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Lessons Learned

In situ and laboratory dynamic tests
Design of Soil models

- Properties and Models - Required Level of information – Level of Site response analysis/ M.S.

- Combined CH, DH and microtremor tests – Detailed data from surface to high depths
- Combined Laboratory Tests – Variation of properties over wide strain levels
- 'Physical' Soils – often significant variations from literature results
- Statistical analysis of results – Uncertainties on seismic response
- Validation between results stemming from different methods - essential
- Correlation e.g. Vs-SPT offers a link among common in geotechnical practice

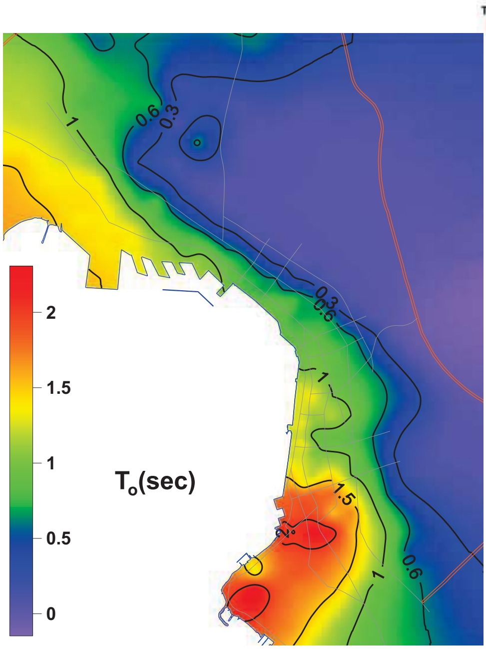
- 1D Soil model is always the basic reference
- Defining models in a grid simplifies the presentation of results/maps with the results of seismic response analysis – require many 2D cross sections, if its not possible
- Defining models on 2D sections - spatial distribution and coverage of the study area

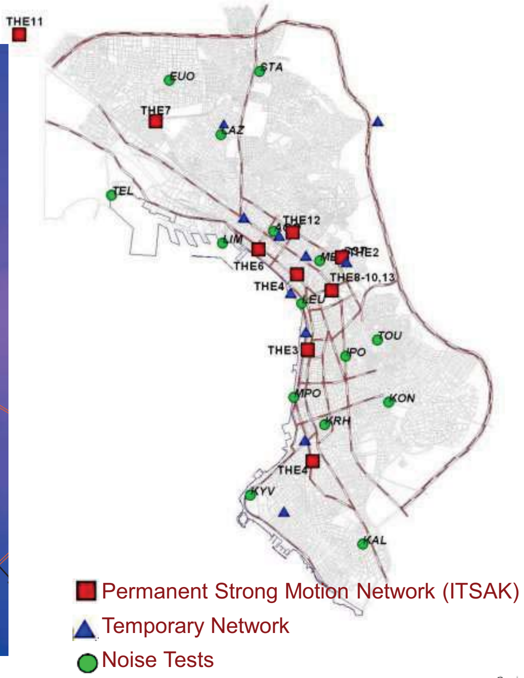
Validation of the 1D soil models

The importance of real seismic recordings and Noise Tests

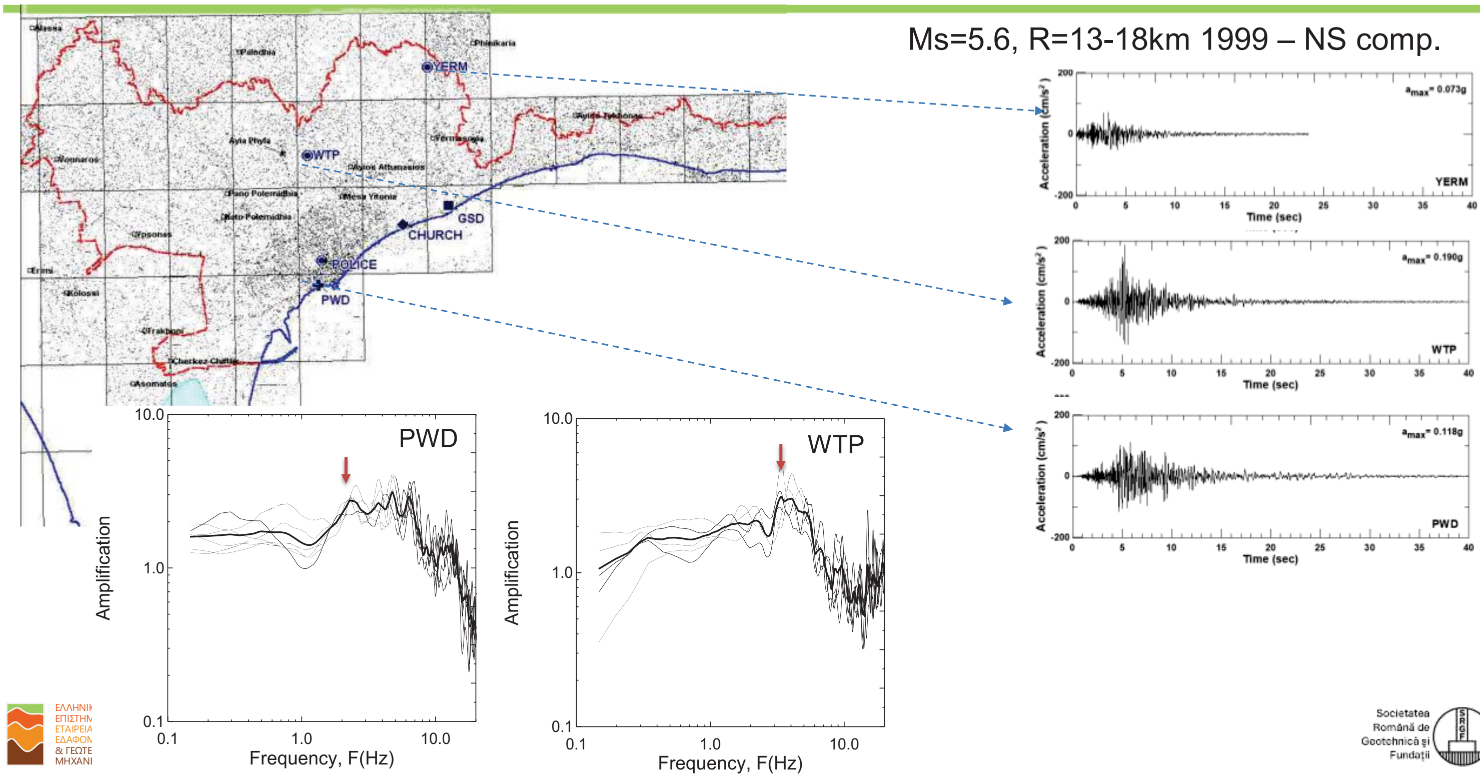
Validation of 1D soil models

To (sec)	SITES
2.2-3	KYV- KAL -MPO
1.7-2	TIF- TEL -KRH
0.9-1.25	KON- LEP/LEP- LIM- IPO
0.4-0.7	OTE- TOU- ROT
0.25-0.3	MET-AGO- LAB





Lemesos: Validation of 1D soil models

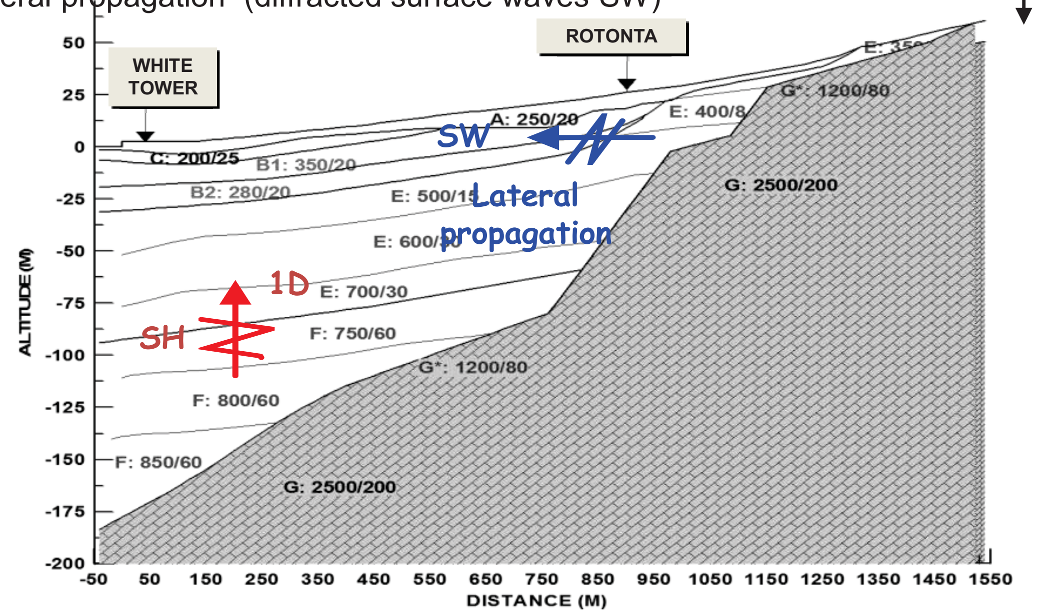


2D site effects: Thessaloniki

COMMON PATTERNS (shallow grabens → Thessaloniki area)

1D (SH waves)

1D & lateral propagation (diffracted surface waves SW)



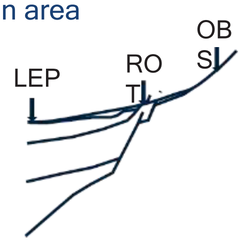
1D - 2D site effects: Thessaloniki

Mw=4.8, R>100km, 16/12/1993

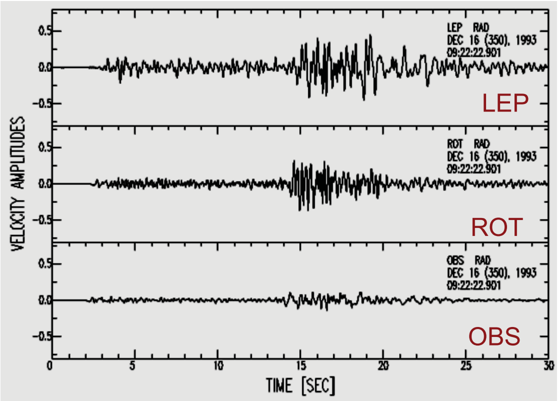
Recorded at several different locations within the Thessaloniki metropolitan area

3 recordings along a typical cross-section of the city:

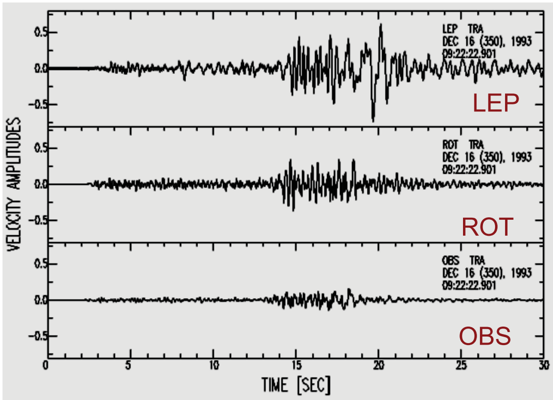
- White Tower (LEP)
- Rotonta (ROT)
- Seismological station (OBS) -> OUTCROP



radial component



transversal component



Raptakis et.al., 2004

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1D - 2D site effects: Thessaloniki

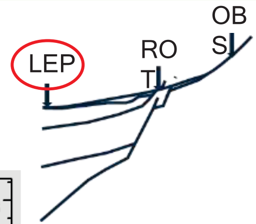
Transfer functions

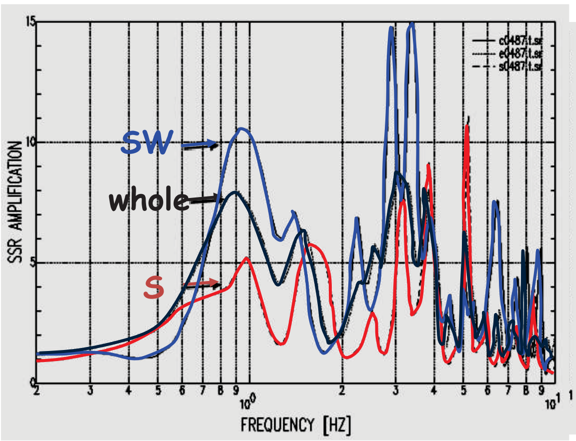
location under study / "rock"

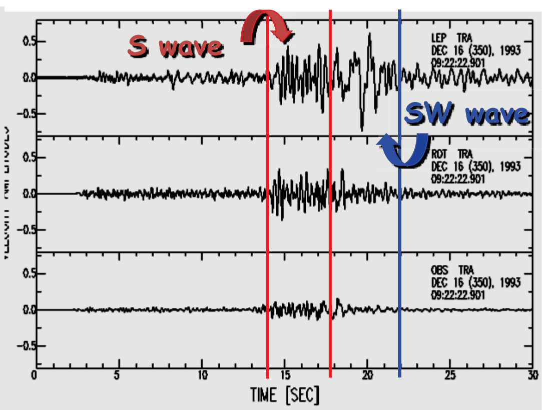
- LEP / OBS

"windows" of the recording at LEP

- Whole signal (S+SW)
- S waves (1D)
- SW waves (lateral propagation)







Raptakis et.al., 2004

Lateral propagation:

- amplifies the 1D resonance for the fundamental frequency
- induces qualitative changes (interaction of S & SW)

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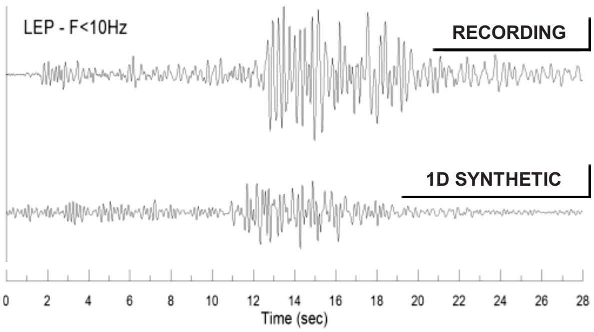
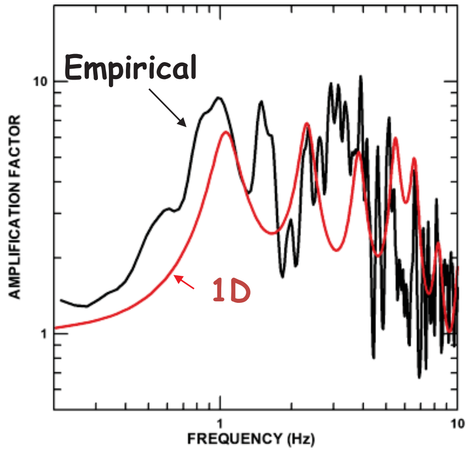
1D - 2D site effects: Thessaloniki

1D Simulation
- 1D soil model
- 1D theoretical transfer function (T.F.)
- convolution [of OBS recording]*[1D T.F.]
- 1D time-history

Comparison with recording at LEP location
- amplitude
- frequencies
- duration





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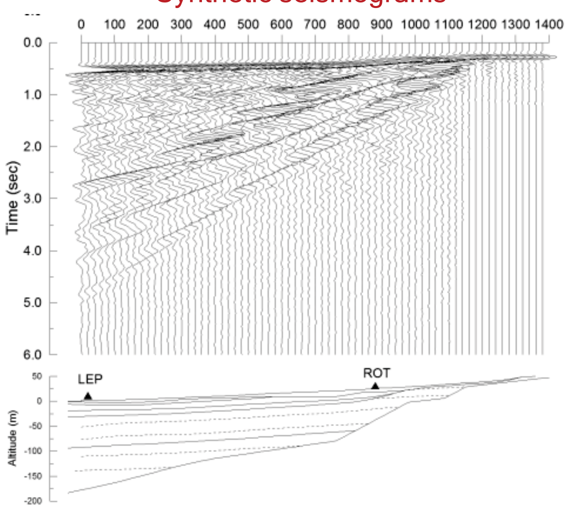
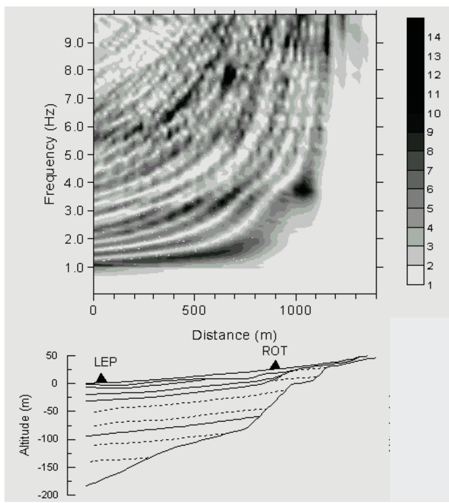
1D - 2D site effects: Thessaloniki

Synthetic seismograms



Transfer functions



Raptakis et.al., 2004

The results of the analyses reveal complex phenomena:
- lateral propagation of locally generated surface waves
- cumulative impact on 1D resonance
- mainly for the fundamental resonant frequency
- prolongation of the duration of seismic excitation

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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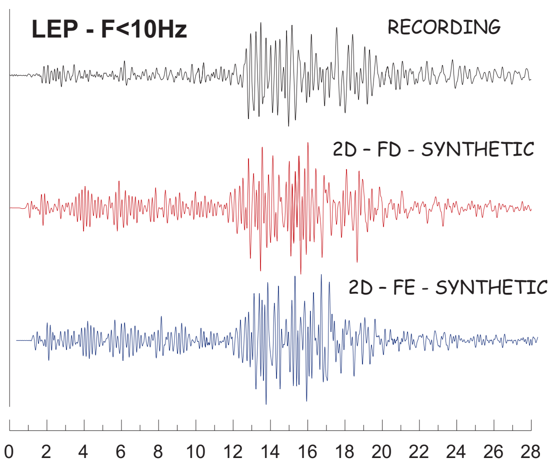
1D - 2D site effects: Thessaloniki

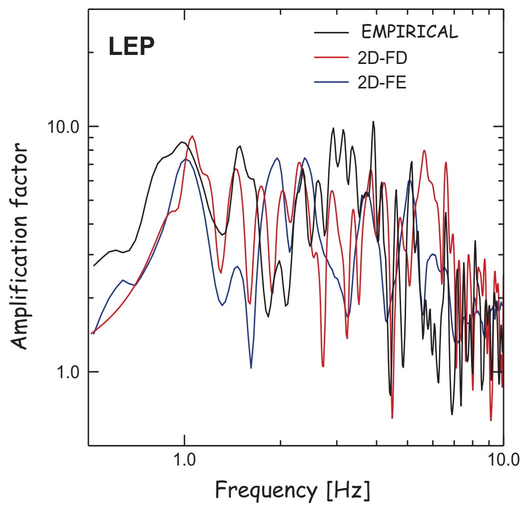
Comparison of recordings & 2D simulations - WHITE TOWER

Weak shock (Mw=4.8, R>100km)

Time-histories



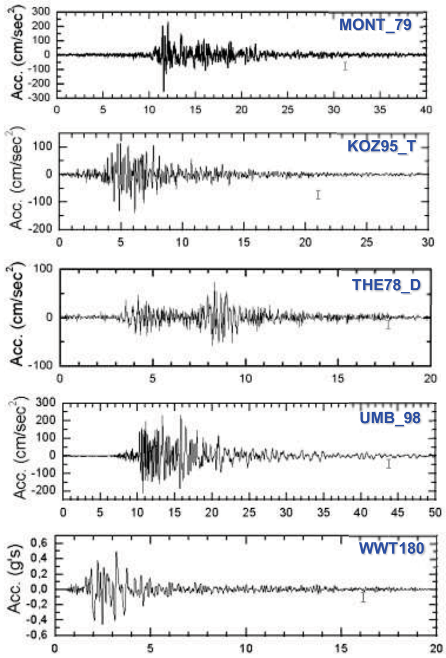
Transfer functions

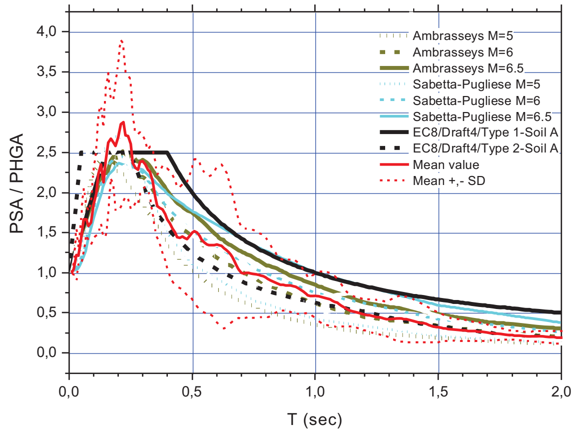


Microzonation Studies

Typical Results and Zonation

Selection of time histories

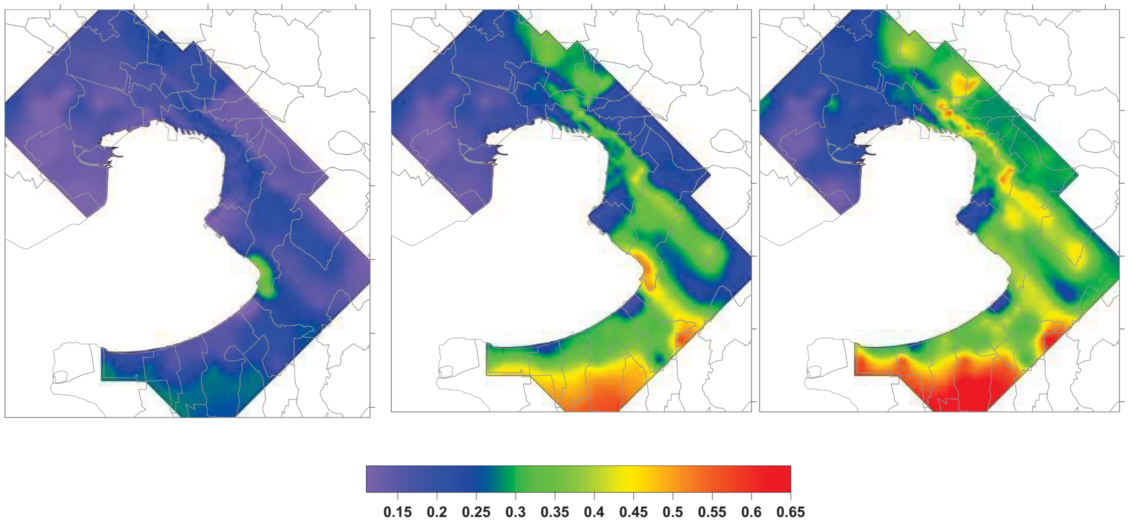




Thessaloniki: 1D Seismic Response Analysis: Typical Results

Distribution of mean peak Acceleration PHGA(g) values

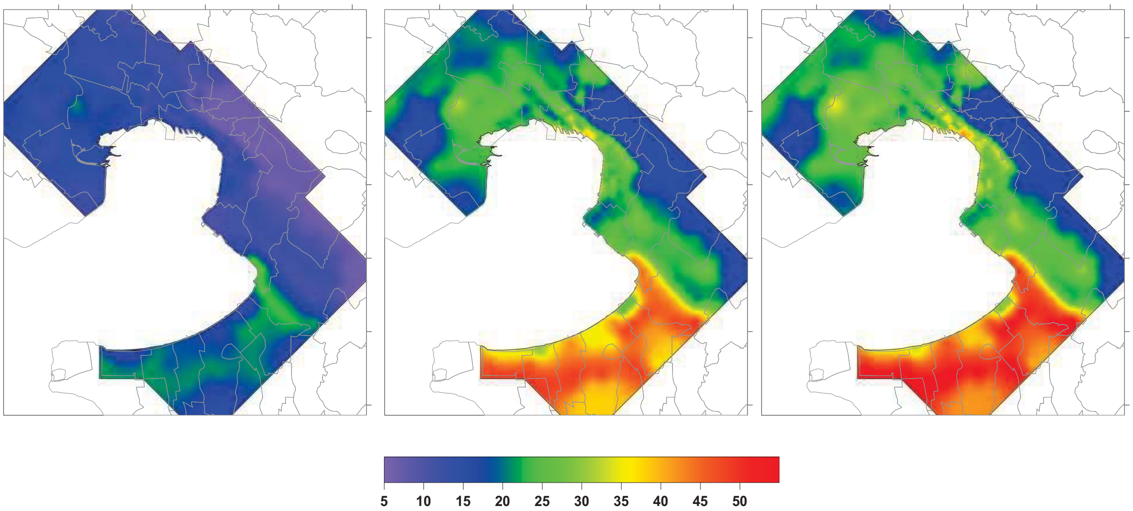
100 years | 500 years | 1000 years



1D Seismic Response Analysis: Typical Results

Distribution of mean peak velocity PHGV (cm/s) values

100 years | 500 years | 1000 years



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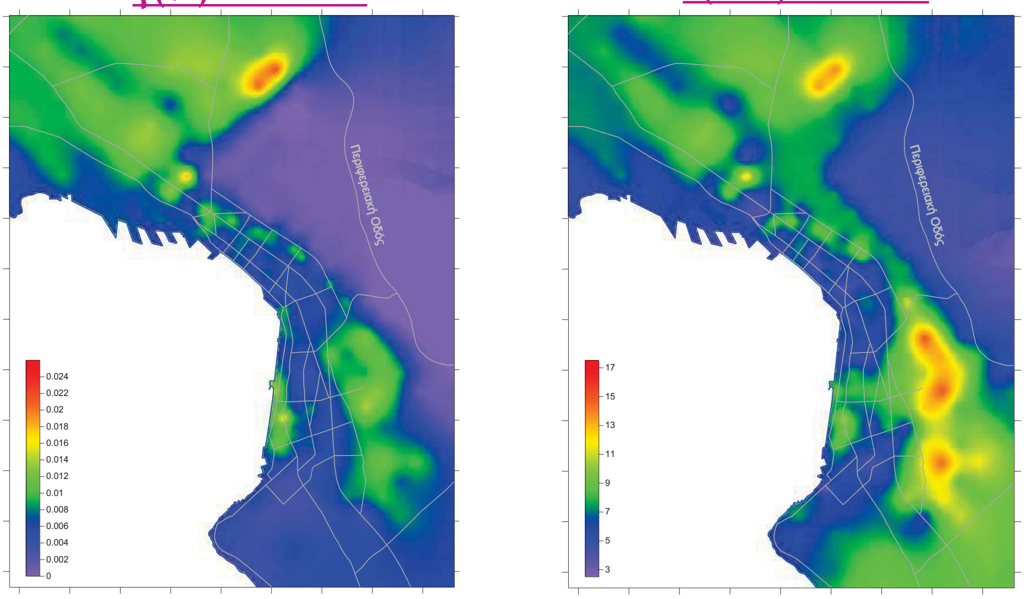
1D Seismic Response Analysis: Typical Results

Distribution of mean peak shear strain and stress at depth -3.0m

γ(%) @ -3.0mc | τ(kPa) @ -3.0m



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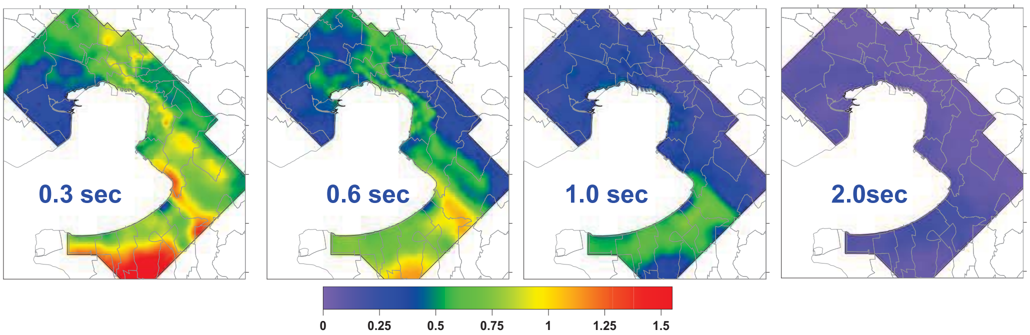
Thessaloniki: 1D Seismic Response Analysis: Typical Results

Distribution of mean peak spectral values, PSA(g's)



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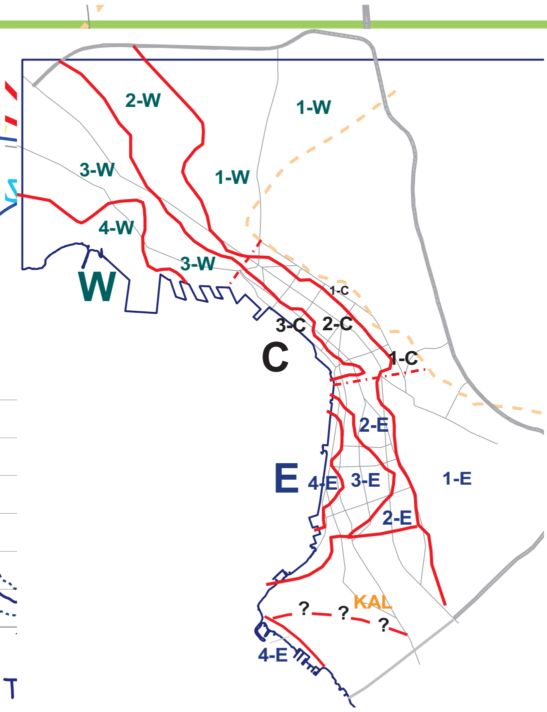
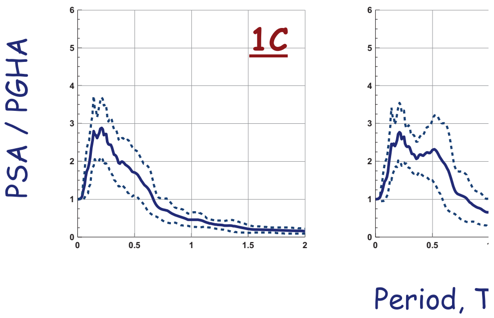
Thessaloniki: Zonation

- Geotechnical Characterization
- Vs, To

'central region'

- Spectral Amplifications (T=0.3,1.0)
- Amplification Ratios





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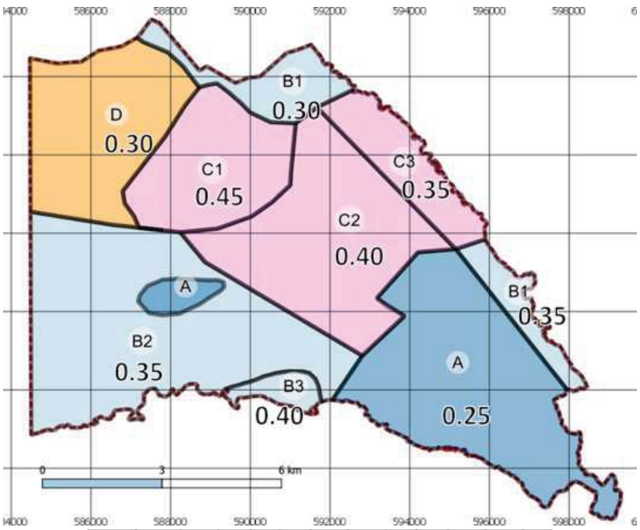
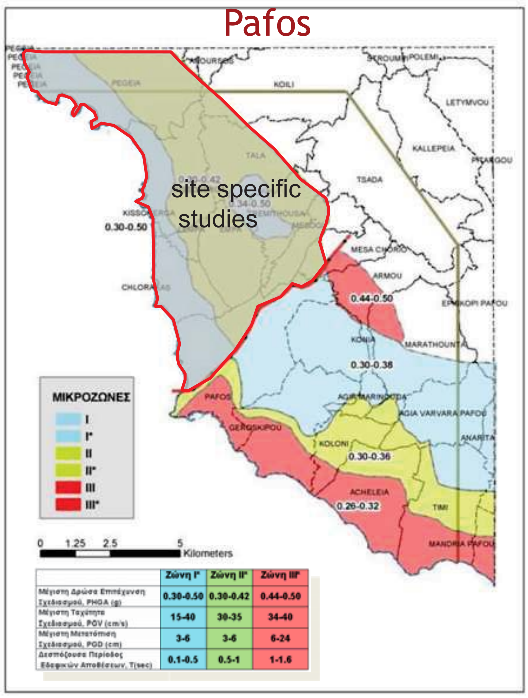
Final Zonation in Ammochostos and Pafos



Ζώνη	Class	SsRP	S1RP	Fs	F1	Ss	S1	Fo	TA	TB	TC	TD
A	A	0.625	0.25	1.0	1.0	0.63	0.250	2.5	0.03	0.08	0.40	3.5
B1	B2	0.625	0.25	1.4	1.5	0.875	0.375	2.5	0.03	0.1	0.43	3.5
B2	B2	0.625	0.25	1.4	1.5	0.875	0.375	2.5	0.03	0.1	0.43	3.5
B3	B2	0.625	0.25	1.6	1.5	1.000	0.375	2.5	0.03	0.08	0.38	3.5
C1	C3	0.625	0.25	1.8	2.6	1.125	0.650	2.5	0.03	0.1	0.58	3.5
C2	C3	0.625	0.25	1.6	2.6	1.000	0.650	2.5	0.03	0.1	0.65	3.5
C3	C3	0.625	0.25	1.4	2.6	0.875	0.650	2.5	0.03	0.1	0.74	3.5
D	C3	0.625	0.25	1.2	2.6	0.75	0.650	2.5	0.03	0.1	0.87	3.5



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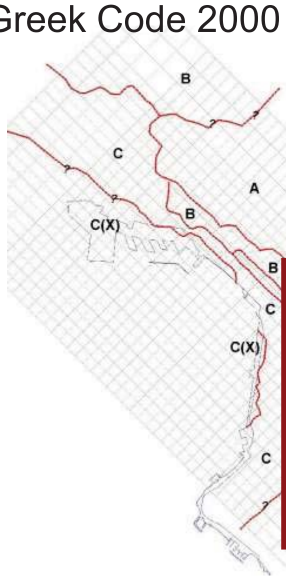
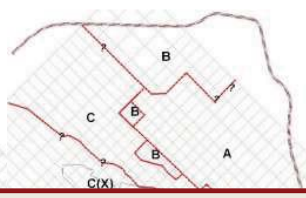
Soil Classification - Zonation

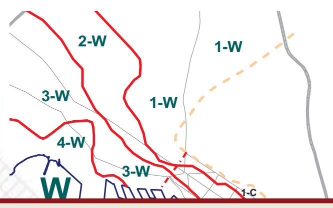
Greek Code 2000



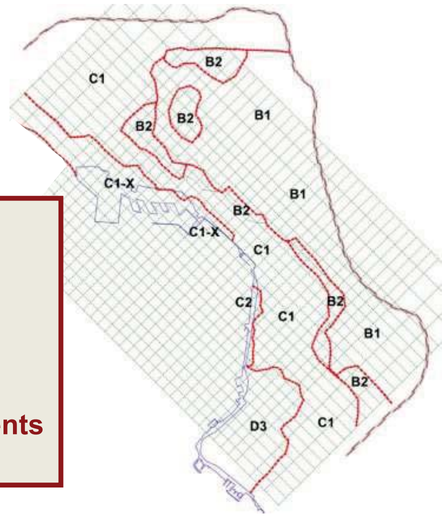
EC8



M.S.



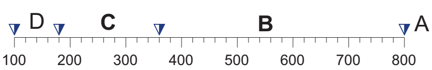
Pitilakis et.al, 2013



Vs,30:

- **advantages**

- **Major disadvantage:**
  - **assumes that properties below 30m are the same**
- **Amplification = f(impedance contrast)**
  - **Taking into account only the properties of sediments the actual amplification will be different**



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Soil conditions and Site Effects

- Methods & tools for interpretation of soil structure
- Validation of results stemming from different methods
- Determination of soil conditions oriented to site effects studies
- Zonation: should not disregard soil type, classical geotechnical parameters and impedance contrast of sediments/rock
- Validation of 1D modelling with weak and strong motion records
- Need for shared Databases and Recordings at well documented sites

Future of Geotechnical Exploration for Microzonation studies

Most likely will be shaped on:

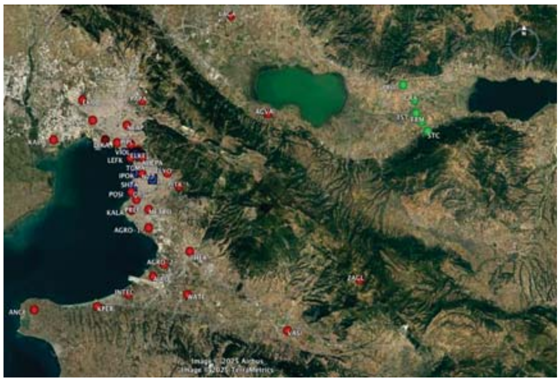
- Geophysical and Geotechnical exploration using **UAVs (drones)**
- **AI and Machine Learning**– Predictive models improve risk assessments by analyzing vast datasets, identifying hidden patterns, and providing real-time hazard forecasts.
- **Remote Sensing and Satellite Data**– Geological and soil mapping will be enhanced by advanced satellite imagery, LiDAR, and InSAR (interferometric synthetic aperture radar).
- The development of user-friendly, high-resolution, **low-cost MEMS** accelerometers enables the development of real-time networks in cities and infrastructure. These data is essential for the validity of soil models and seismic response.
- **Smart Cities and IoT Integration -** Sensors embedded in urban infrastructure will provide continuous updates allowing for adaptive zoning and emergency response.
- **Climate Change Considerations -** Because environmental changes affect soil stability and water tables, microzonation must account for evolving seismic risks.
- **Global Standardization -** The adoption of standardized microzonation techniques by more countries will improve cross-border risk assessment and disaster preparedness.

Soil Structure and Site Effects

Site specific seismic ground motion parameters can be accurately evaluated in order to use them to assess vulnerability, risk and losses for specific seismic scenarios

Physical and epistemic uncertainties

Need for more high-quality records in well documented sites



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As seismic microzonation and geotechnics continue to evolve, cities will become more resilient, and infrastructure will be built with better earthquake resistance and people communities will be more safe

Ευχαριστώ

Multumesc

Thank you

Michalis Bardanis

Partner & Director of the Geotechnical Laboratory, EDAFOS Engineering Consultants S.A., Athens, Greece, President, Hellenic Society for Soil Mechanics and Geotechnical Engineering

Identification, investigation and remediation of slow-moving landslides

Michalis Bardanis is Partner and Director of the Geotechnical Laboratory of EDAFOS Engineering Consultants S.A., a geotechnical consultancy based in Athens, Greece. He holds a Diploma in Civil Engineering from the National Technical University of Athens (NTUA), an MSc in Soil Mechanics from Imperial College, London, and a PhD degree in Unsaturated Soil Mechanics from NTUA. He has worked as a geotechnical engineer since 1998 on projects in Greece, Cyprus and Bulgaria, including large landslide remediation projects, highways, dams, airports, investigation and restoration of historical monuments etc. His research interests include unsaturated soil mechanics, laboratory and field testing, mechanical behaviour of reconstituted and structured soils, slope stability and landslide remediation, geotechnics of historical monuments and sites etc. Between 2018 and 2021 he was a Visiting Lecturer at Neapolis University Paphos, Cyprus, teaching in the Undergraduate Course of Civil Engineering, and between 2022 and 2023 a Lecturer under temporary contract at the MSc Course on Integrated Design of Hydraulic and Geotechnical Structures of the University of West Attica. He has been elected several times on the Executive Committee of the Hellenic Society for Soil Mechanics and Geotechnical Engineering (HSSMGE), serving as its Secretary General between 2015 and 2019, and as its President since 2019 (first elected in 2019, re-elected in 2023). He was the Chairman of the 8th International Conference on Unsaturated Soils, Milos, Greece, 2-5 May 2023, and the Chairman of the 9th National Hellenic Conference on Geotechnical Engineering, Athens, Greece, 4-6 October 2023. He has authored and co-authored 82 papers in journals and conferences and has delivered 16 lectures after invitation.

Identification, investigation and remediation of slow-moving landslides

Michalis Bardanis

EDAFOS Engineering Consultants S.A.

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ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

Thessaloniki, 9th October 2025

Societatea Română de Geotehnică şi Fundaţii

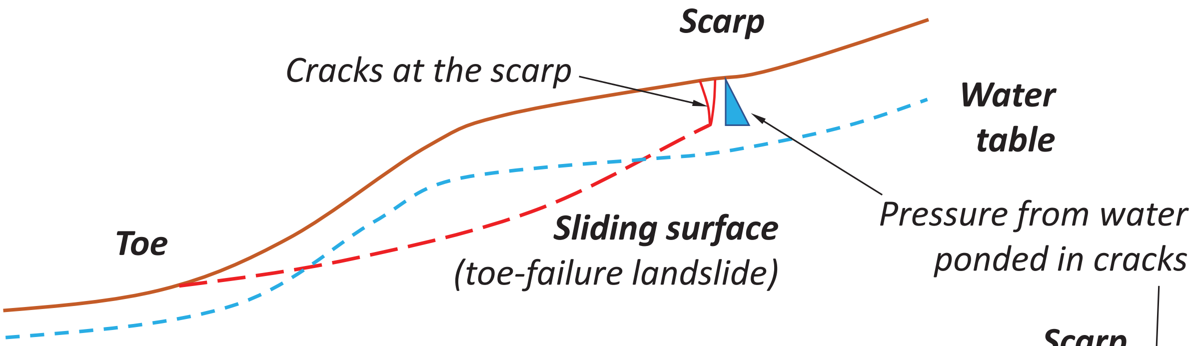
In our first seminar we addressed issues through the lense of codes, namely EC7 and EC8

In our second seminar we have tried to address issues of common interest through the challenging lense of failures, some of the most common among them:

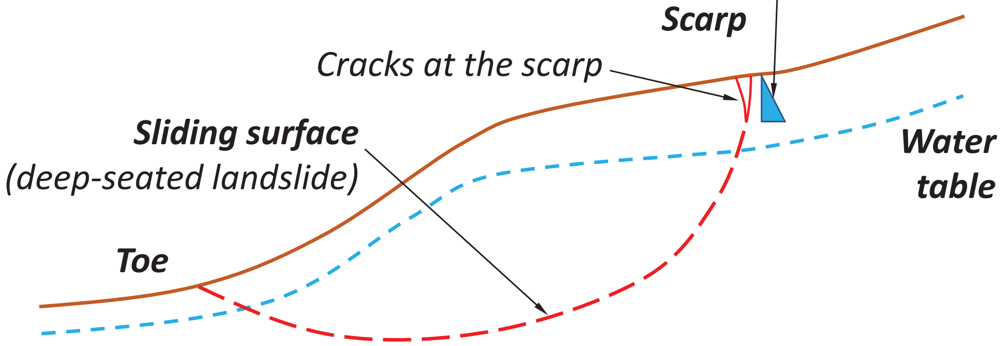
LANDSLIDES

What is a landslide?

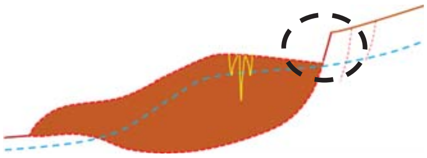
A ***landslide*** in general is the ***movement*** of a mass of rock, debris, or soil down a slope, in the form of a solid block, a totally disaggregated mass or a combination of both, with a ***rate of movement*** ranging from very slow to extremely fast because of a ***trigger*** causing a disturbance of a previously established equilibrium, usually along a specific ***sliding surface***. The landform resulting from this movement, may be called by extension a landslide as well.



Typical parts of a landslide



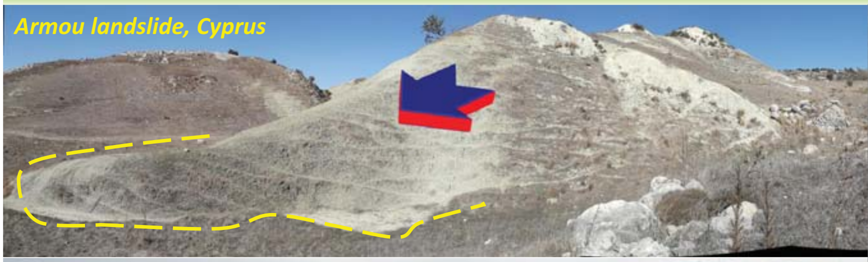
Sesklo Clay Mine southwestern landslide



And what do scarps look like?

Sesklo Clay Mine western landslide

St Dimitrios Bastion north landslide
in the old city of Chania

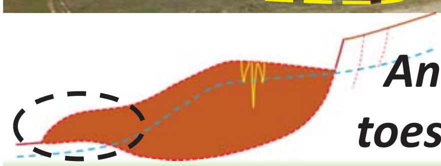


Armou landslide, Cyprus

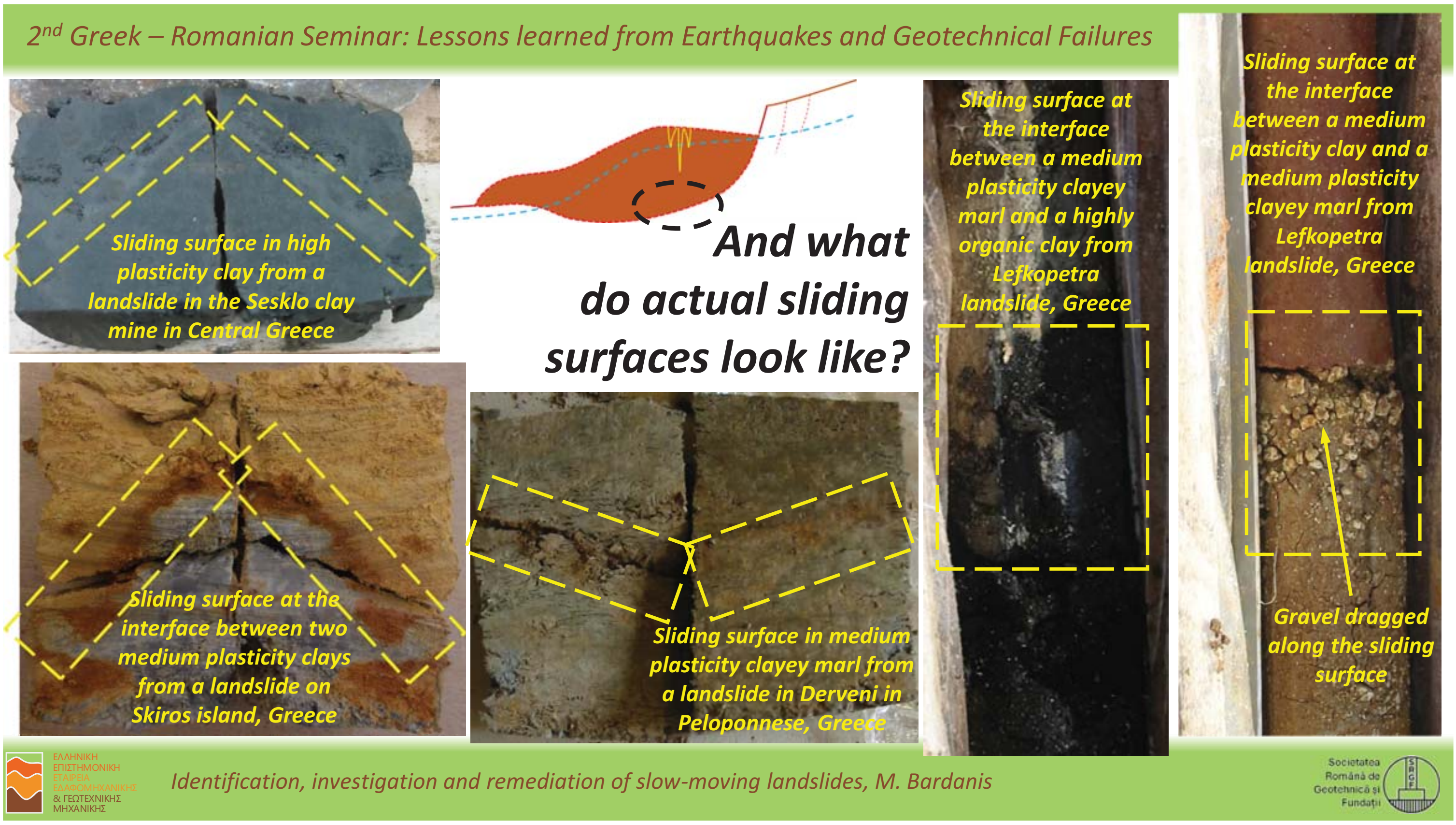
Sesklo Clay Mine western landslide

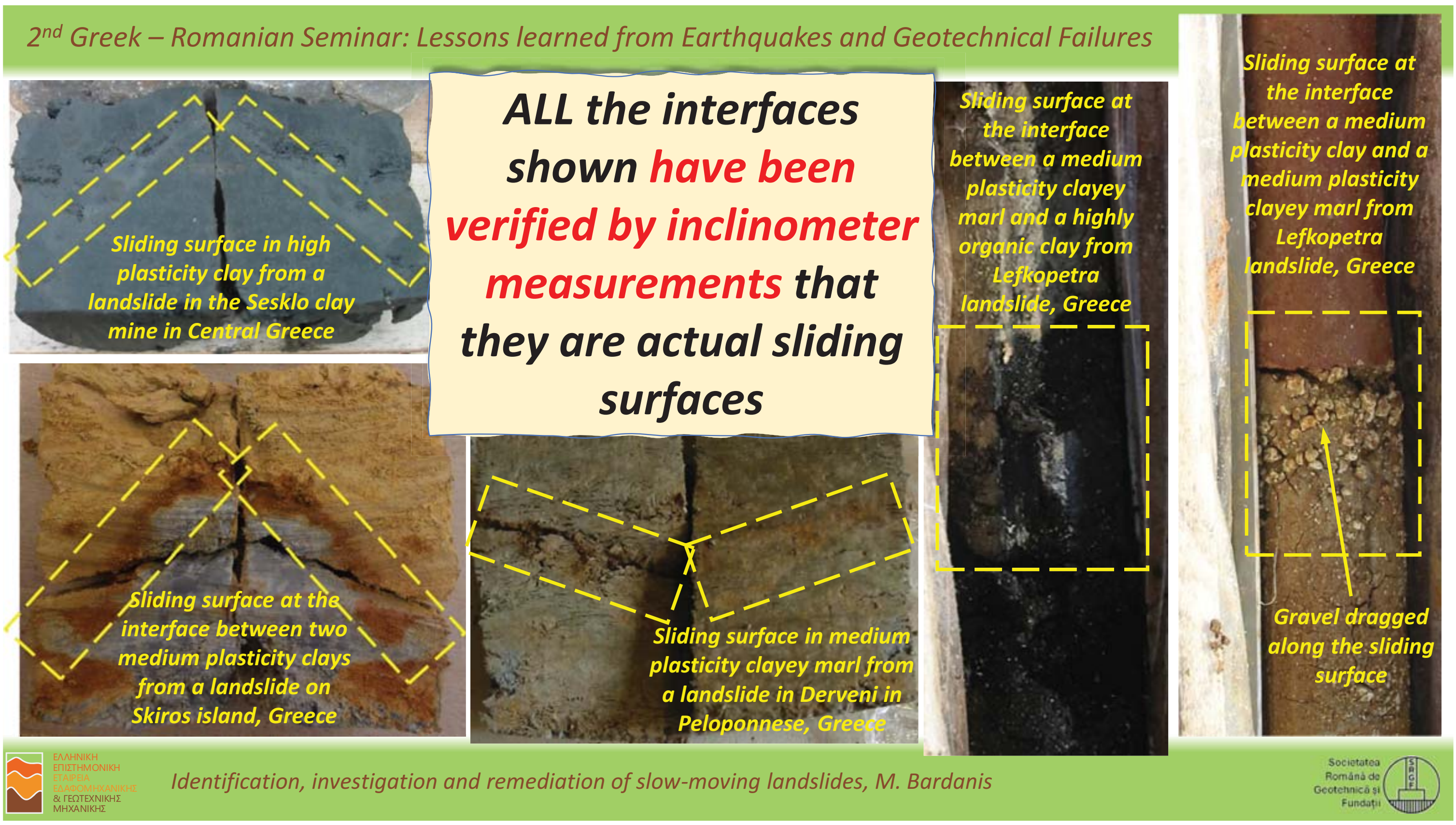
St Dimitrios Bastion north landslide
in the old city of Chania ↑

Sesklo Clay Mine southwestern landslide ↓



And what do toes look like?


2nd Greek – Romanian Seminar: Lessons learned from Earthquakes and Geotechnical Failures
Sliding surface in high plasticity clay from a landslide in the Sesklo clay mine in Central Greece
And what do actual sliding surfaces look like?
Sliding surface at the interface between a medium plasticity clayey marl and a highly organic clay from Lefkopetra landslide, Greece
Sliding surface at the interface between a medium plasticity clay and a medium plasticity clayey marl from Lefkopetra landslide, Greece
Sliding surface at the interface between two medium plasticity clays from a landslide on Skiros island, Greece
Sliding surface in medium plasticity clayey marl from a landslide in Derveni in Peloponnese, Greece
Gravel dragged along the sliding surface
Identification, investigation and remediation of slow-moving landslides, M. Bardanis


2nd Greek – Romanian Seminar: Lessons learned from Earthquakes and Geotechnical Failures
ALL the interfaces shown have been verified by inclinometer measurements that they are actual sliding surfaces
Sliding surface in high plasticity clay from a landslide in the Sesklo clay mine in Central Greece
Sliding surface at the interface between a medium plasticity clayey marl and a highly organic clay from Lefkopetra landslide, Greece
Sliding surface at the interface between a medium plasticity clay and a medium plasticity clayey marl from Lefkopetra landslide, Greece
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Sliding surface in medium plasticity clayey marl from a landslide in Derveni in Peloponnese, Greece
Gravel dragged along the sliding surface
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How do we classify landslides?

Lithology, geometry, morphology

(various editions of the original by Varnes, 1978)

Rock Debris Soil

Falls

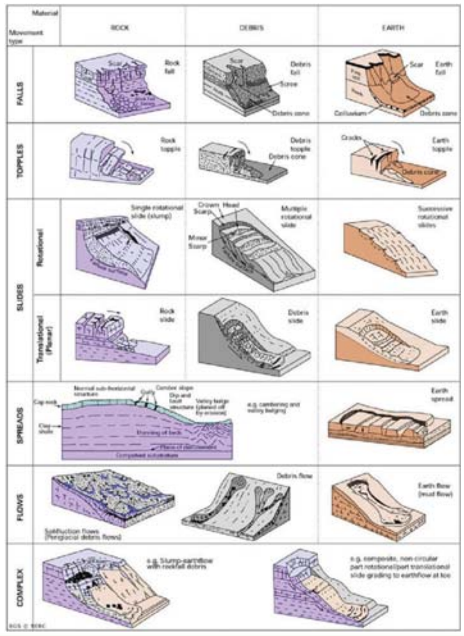
Topples

Slides – Rotational / Translational

Spreads

Flows

Complex



Very useful for the lithological and geometrical/morphological description. Not necessary so useful for design & managements decision.

How do we classify landslides? Velocity!

(Table after Hungr, et. al. 2014)

Velocity class	Description	Velocity (mm/s)	Typical velocity	Possible response
7	Extremely rapid	5×10^3	5 m/s	No response possible
6	Very rapid	5×10^1	3 m/min	No response possible
5	Rapid	5×10^{-1}	1.5-2.0m/hr	Evacuation
4	Moderate	5×10^{-3}	10-15 m/month	Evacuation
3	Slow	5×10^{-5}	1-2 m/year	Maintenance / Stabilisation
2	Very slow	5×10^{-7}	1-5 cm/year	Maintenance / Stabilisation
1	Extremely slow		Practically not measurable	No response necessary*

Only early identification & proactive stabilisation

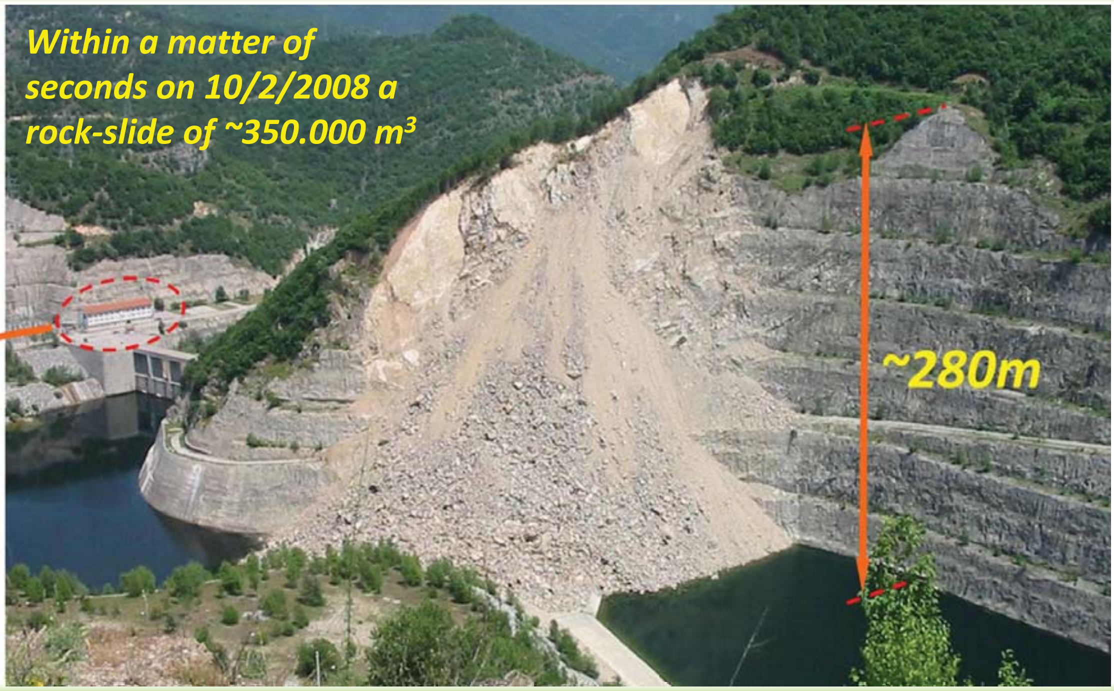
Plus remediation post-mortem

Only early identification & design of evacuation plans

Design of maintenance/ stabilization measures

Identification & monitoring (*beware of *structures*)

2nd Greek – Romanian Seminar: Lessons learned from Earthquakes and Geotechnical Failures



Three-storey building (for scale)

Rock-slide downstream of the Thisavros Dam on Nestos River in Northeastern Greece

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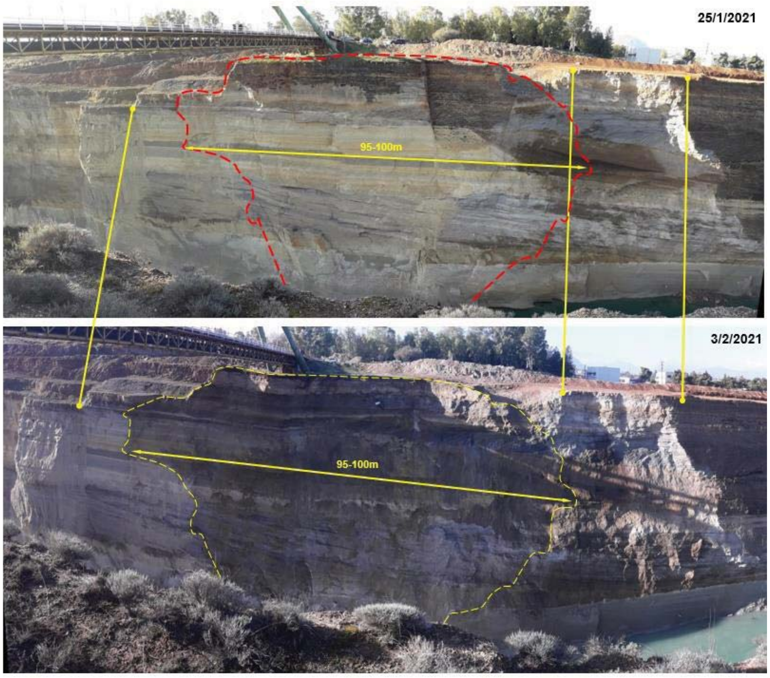
Within a matter of seconds on February the 3rd 2021 approximately 20-25.000 m^3 fell to the sea



The major landslide on the Peloponnese slope of the Corinth Canal on 3/2/2021 in Central Greece

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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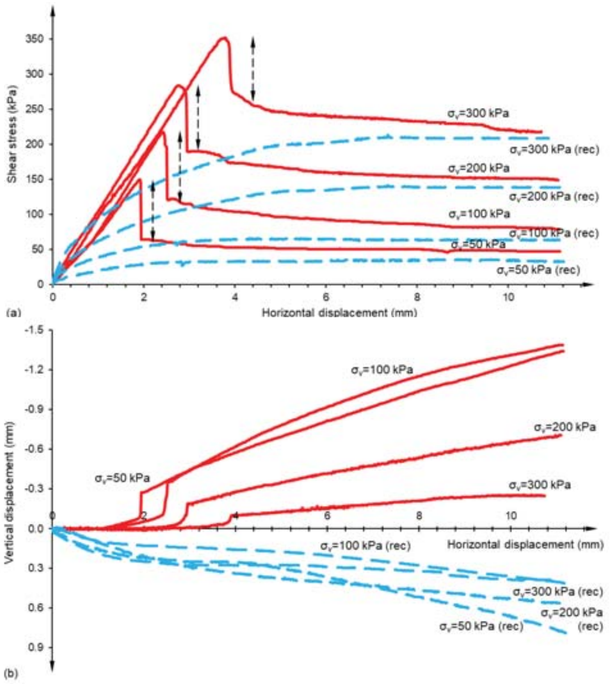
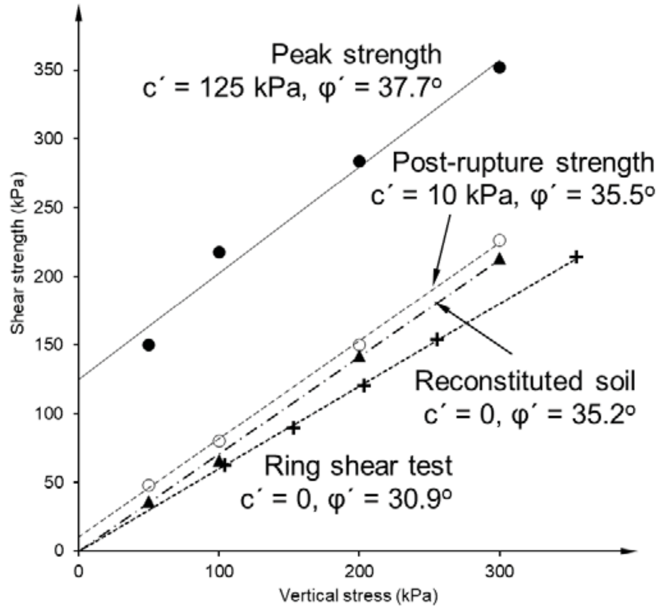
The “notorious” Corinth Canal marls



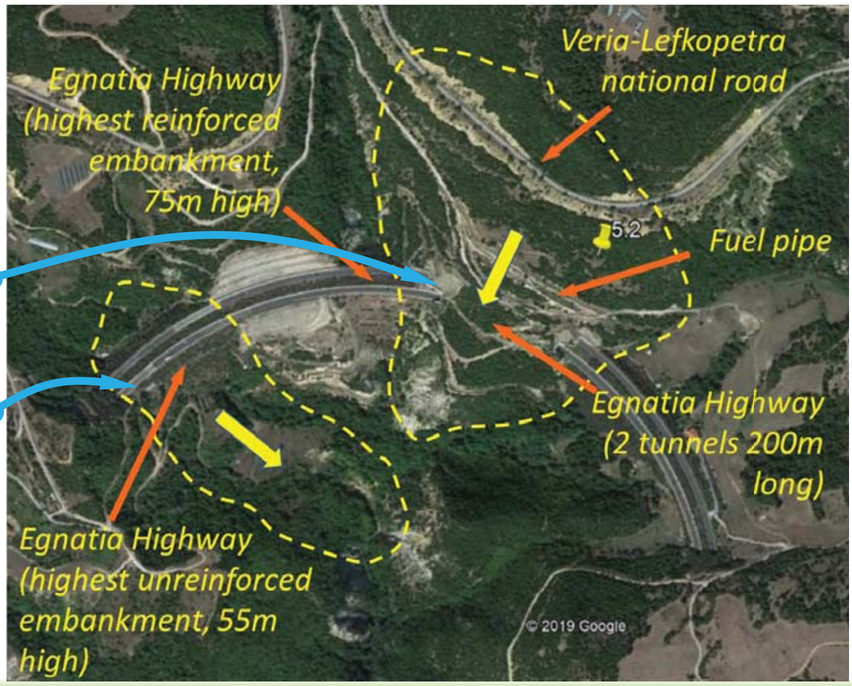


The slow moving landslides along Egnatia Highway (Veria-Lefkopetra section) in Northern Greece

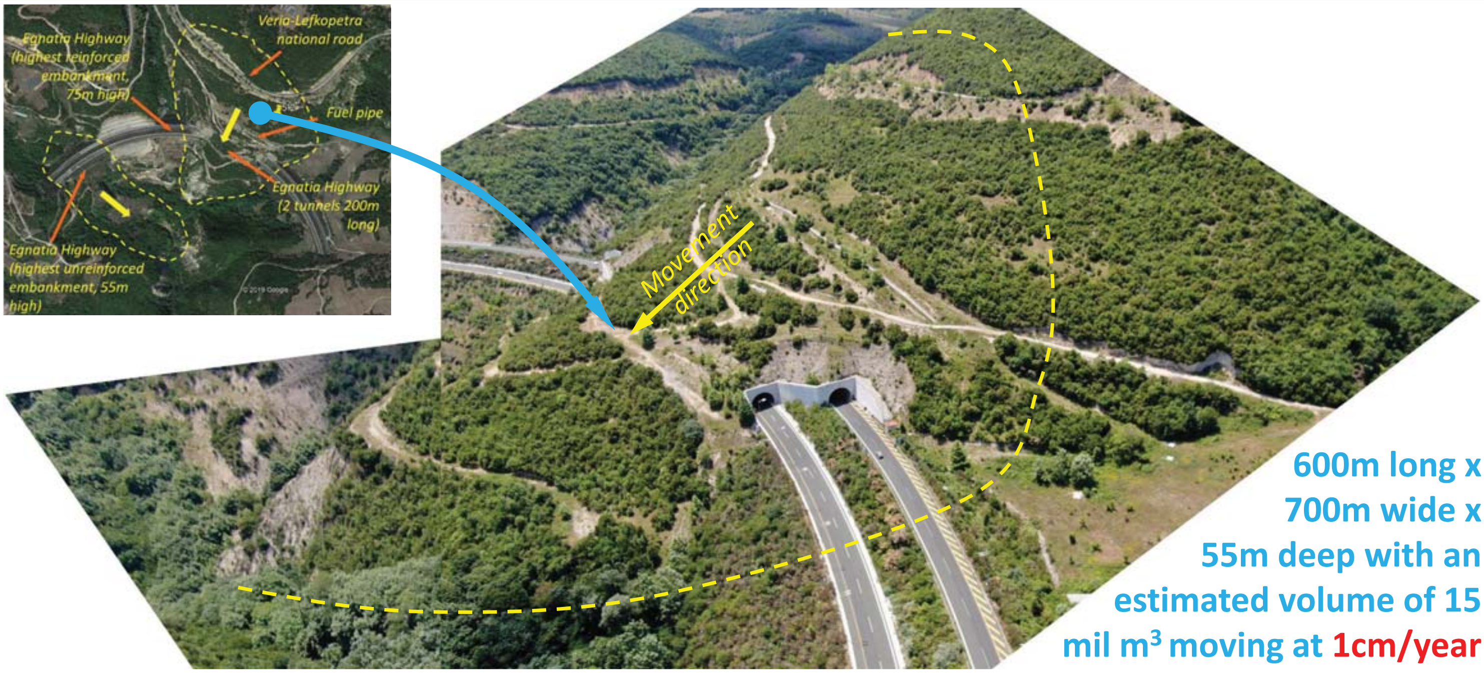
600m long x 700m wide x 55m deep with an estimated volume of 15 million m³ moving at a rate of 1cm/year

600m long x 200m wide x 60m deep with an estimated volume of 5 million m³ moving at a rate of 1-2cm/year

The solution designed included a *very large stabilising berm of 1 million m³* combined with drainage and scarp excavations



600m long x 200m wide x 60m deep with an estimated volume of 5 million m³ moving at a rate of 1-2cm/year



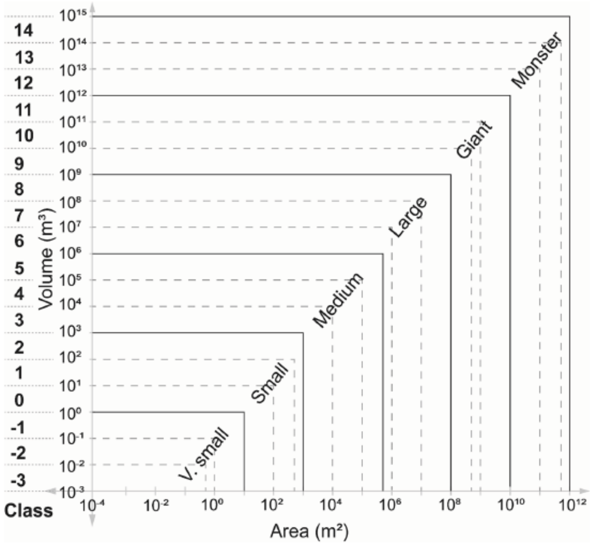
600m long x 700m wide x 55m deep with an estimated volume of 15 mil m³ moving at 1cm/year

How do we classify landslides? Size! (from McColl & Cook, 2024)

Proposed universal classification

CLASS	Descriptor	Minimum volume (m^3)	Minimum area (m^2)
14	Monster (trillions)	≥ 100,000,000,000,000	≥ 500,000,000,000
13		≥ 10,000,000,000,000	≥ 100,000,000,000
12		≥ 1,000,000,000,000	≥ 10,000,000,000
11	Giant (billions)	≥ 100,000,000,000	≥ 1,000,000,000
10		≥ 10,000,000,000	≥ 500,000,000
9		≥ 1,000,000,000	≥ 100,000,000
8	Large (millions)	≥ 100,000,000	≥ 10,000,000
7		≥ 10,000,000	≥ 1,000,000
6		≥ 1,000,000	≥ 500,000
5	Medium (thousands)	≥ 100,000	≥ 100,000
4		≥ 10,000	≥ 10,000
3		≥ 1000	≥ 1000
2	Small (ones)	≥ 100	≥ 500
1		≥ 10	≥ 100
0		≥ 1	≥ 10
−1	Very small (thousandths)	≥ 0.1	≥ 1
−2		≥ 0.01	≥ 0.5
−3		≥ 0.001	≥ 0.1

Size descriptor terms to be avoided or alternatives



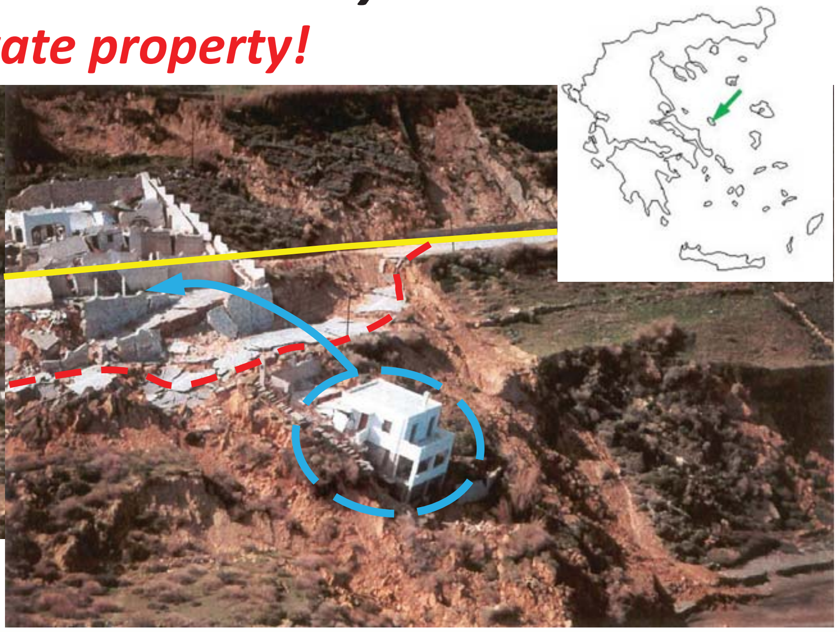
Examples of landslides & associated myths! Landslides are only relevant to infrastructure/mining & the countryside!

Small landslips in "everyday" projects

The highest unreinforced highway embankment in Greece (55m high) constructed at the scarp of a palaeolandslide

A major landslide along Egnatia Highway in Northern Greece

Examples of landslides & associated myths! Landslides are INDEED relevant to private property!



A major palaeolandslide at Bassale, Skiros island, Aegean Sea, Greece

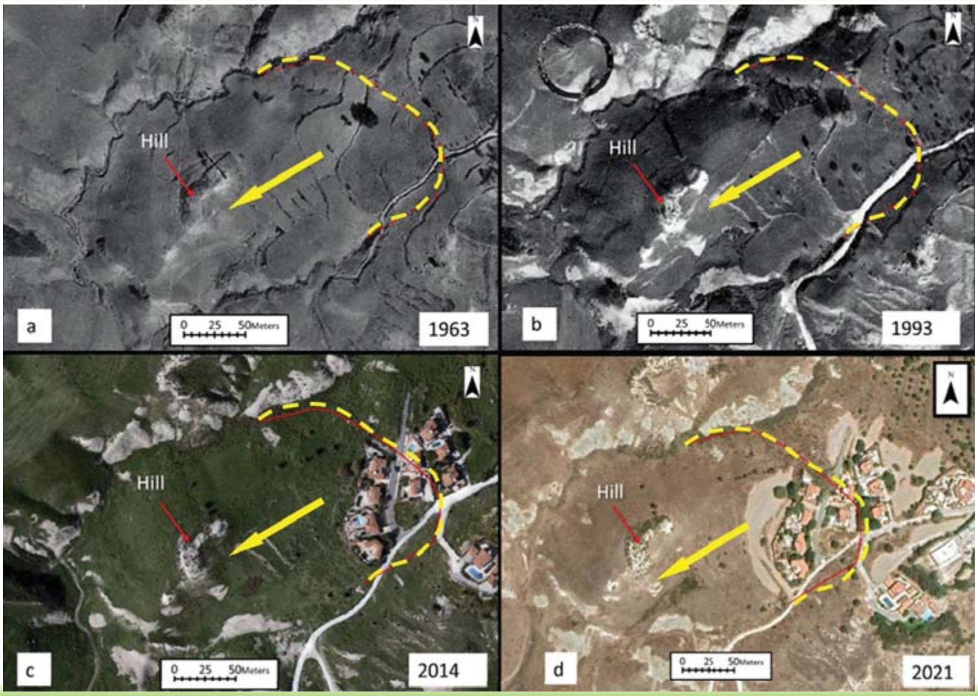
A major ***palaeolandslide*** *at Bassale, Skiros island, Aegean Sea, Greece*

A "tongue" shaped toe

Armou landslide, Cyprus, in bentonitic clays

Armou landslide, Cyprus, in bentonitic clays

A landslide at least 250m long and 150m wide with a depth larger than 25m having caused damaged to at least 6 houses (founded on piles) to a level beyond repair. (Koulermou, et al., 2025, under review)



Ok. You made your point!

Landslides can be relevant both to civil infrastructure, open-pit mining AND private property.

BUT, only in the countryside!
They are totally irrelevant to urban environments!

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Identification, investigation and remediation of slow-moving landslides, M. Bardanis

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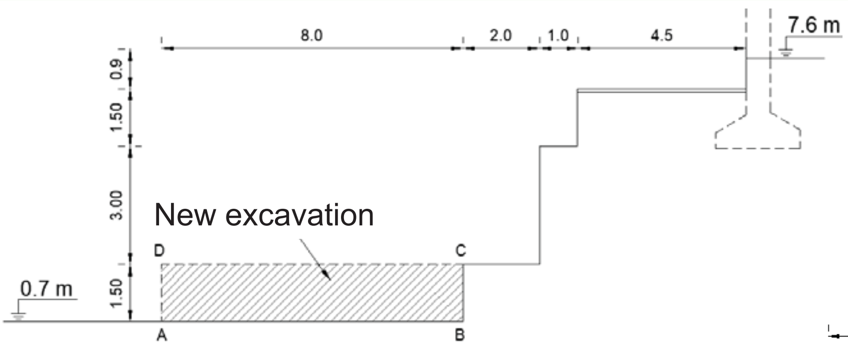
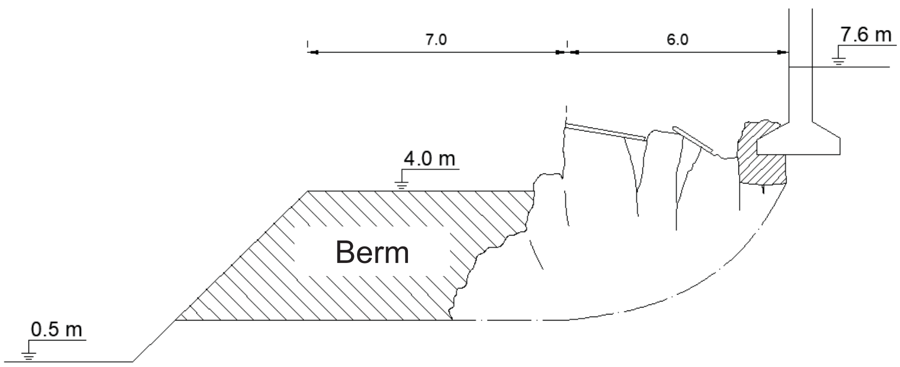
A landslide in Kipseli, the most densely populated area in Athens, Greece, on the side of an excavation to construct a new block of flats adjacent to an existing one

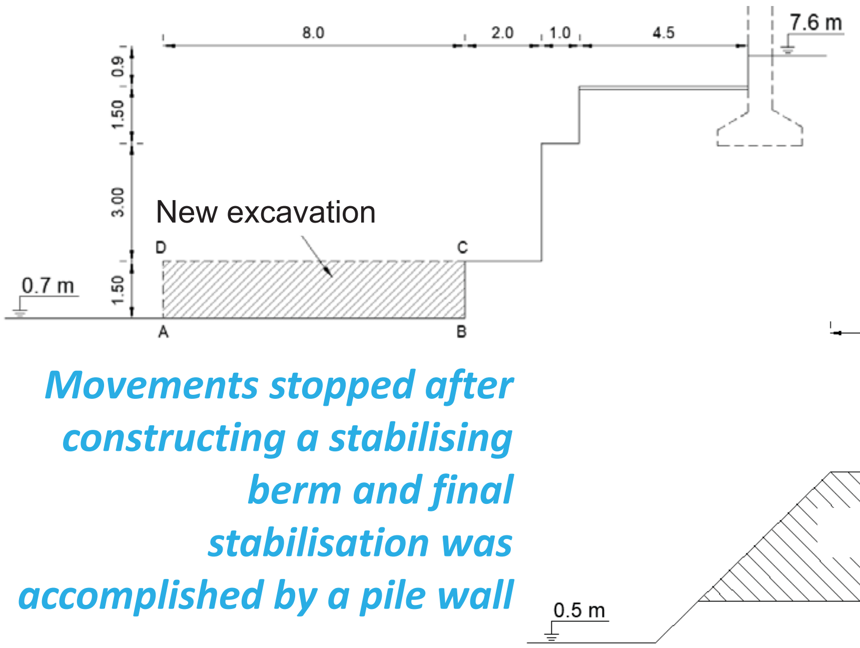
Cavounidis, et al., 1992
Cavounidis, 2016

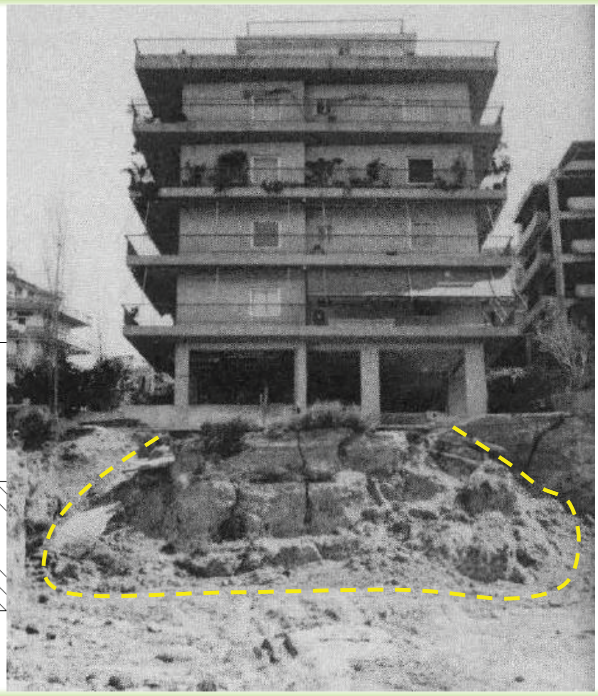


The landslide occurred in a slope, stable for many years, after an additional excavation at the foot of just 1.5m

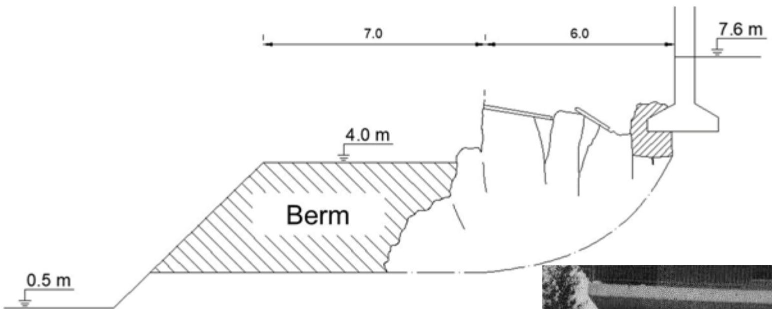
Movements stopped after constructing a stabilising berm and final stabilisation was accomplished by a pile wall







Movements stopped after constructing a stabilising berm and final stabilisation was accomplished by a pile wall



A back-analysis led to values of c΄=5 kPa & φ΄=21.5° for the Athenian Schist and a calculated F.O.S. for the footing part of which left in the air of only 1.24!

A view of the constructed pile wall stabilising the block of flats up-slope and securing the excavation for the construction of the block of flats down-slope.

So…

Landslides are relevant to civil infrastructure, open-pit mining, monuments AND private property,

BOTH in the countryside AND in urban environments.

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Identification & Investigation

- Rely (*heavily*) on engineering geology reports and colleagues for mapping, identification of mechanisms and triggers
- Use instrumentation!
- Inclinometers
- Instrumentation to measure pore pressures (*better: identify the pore pressure regime*)

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ
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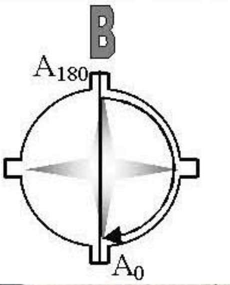
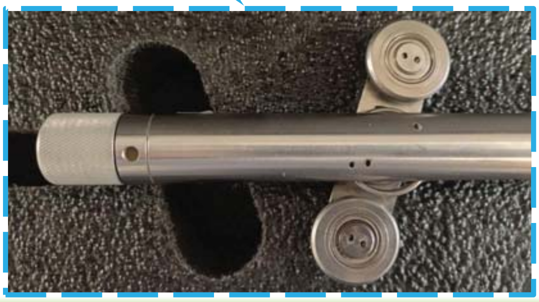
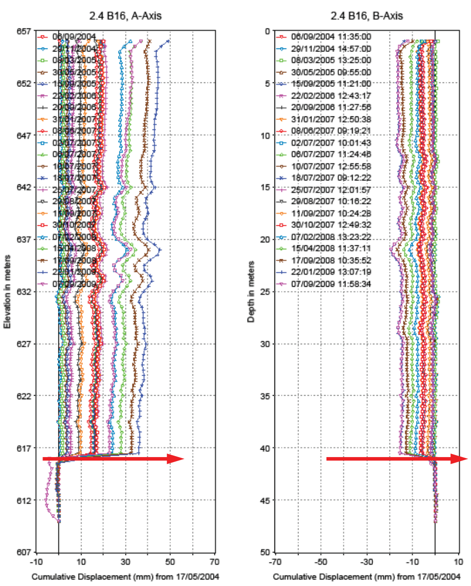
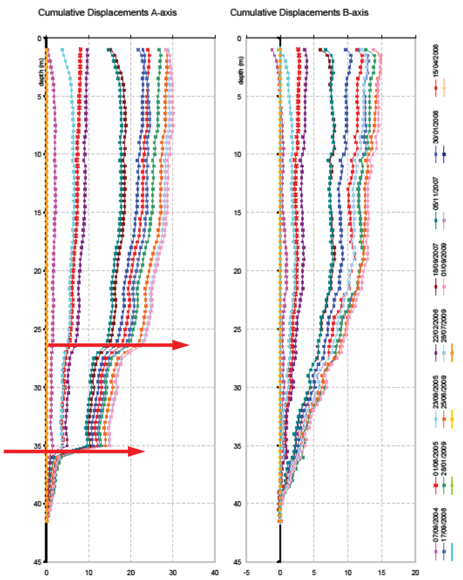
Inclinometers



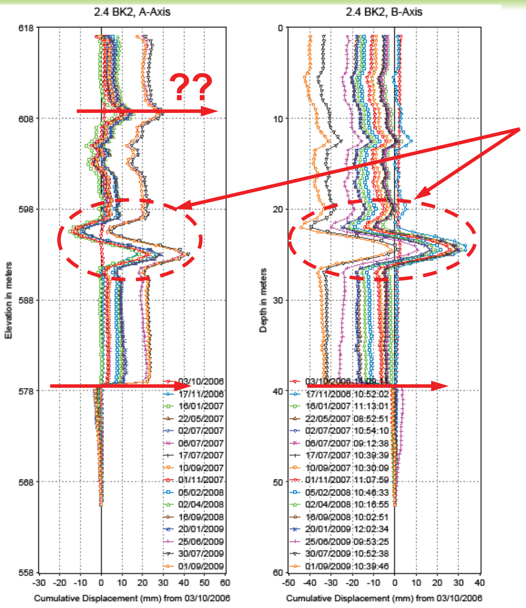




One clearly identified point on both axes

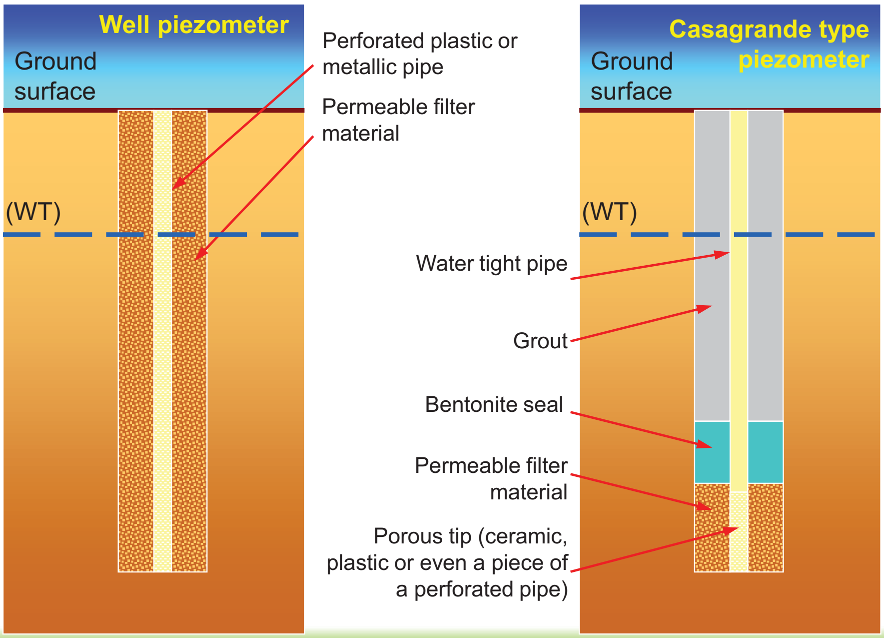


Two clearly identified points on one axis



One clearly identified point on both axes, another one (maybe?) plus noise, probably from poorly constructed joints

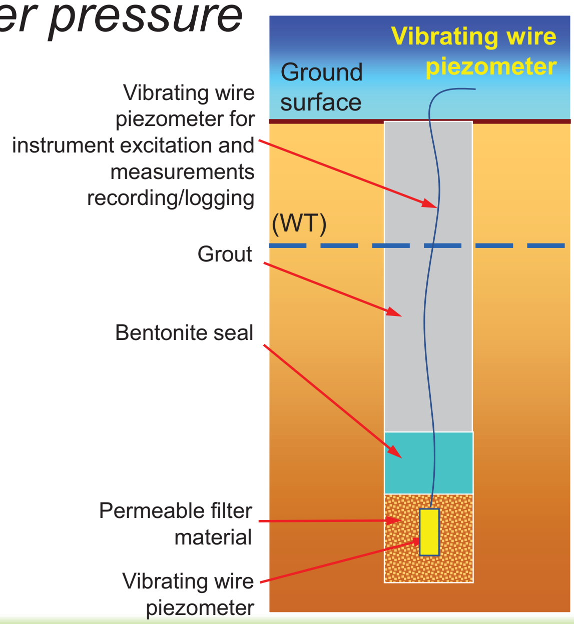
Measurement of the –positive– pore water pressure



Examples of plastic porous tips

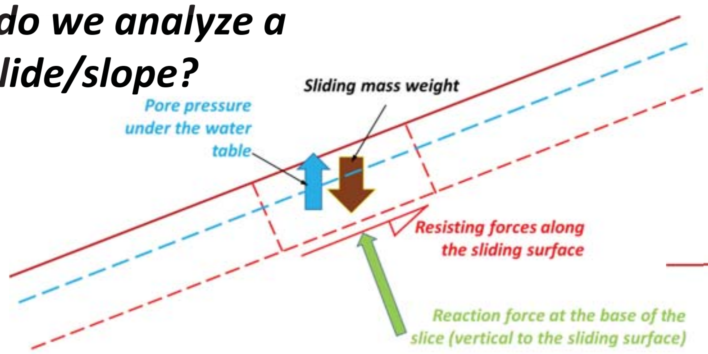
A water level meter with acoustic and light signal

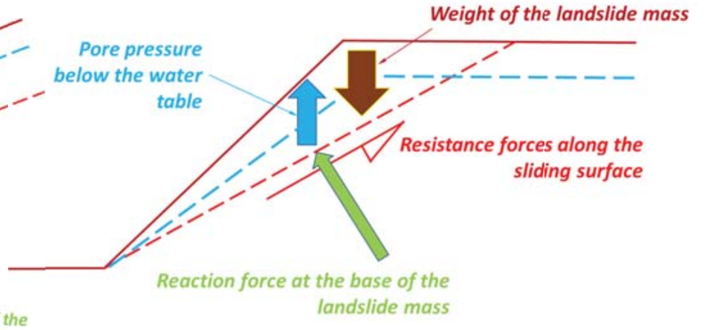
Measurement of the pore water pressure

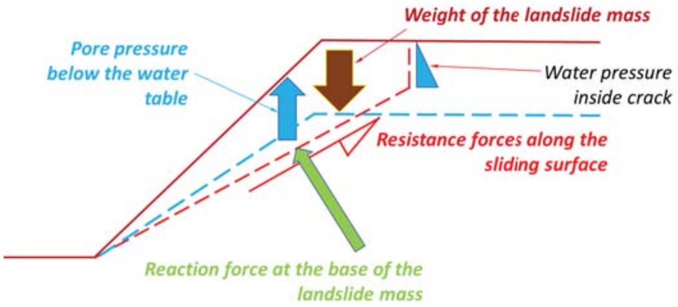


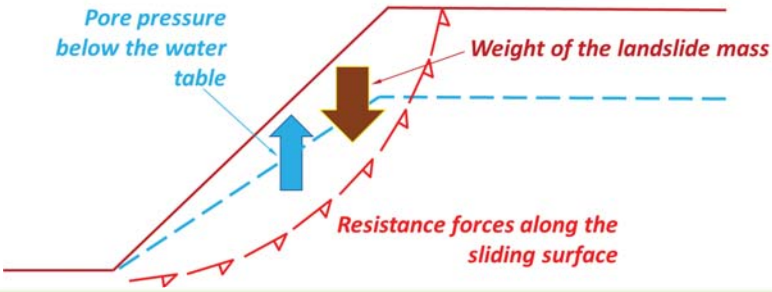
Collection of basic tools/instruments for installing a vibrating wire piezometer and take measurements with it.

How do we analyze a landslide/slope?

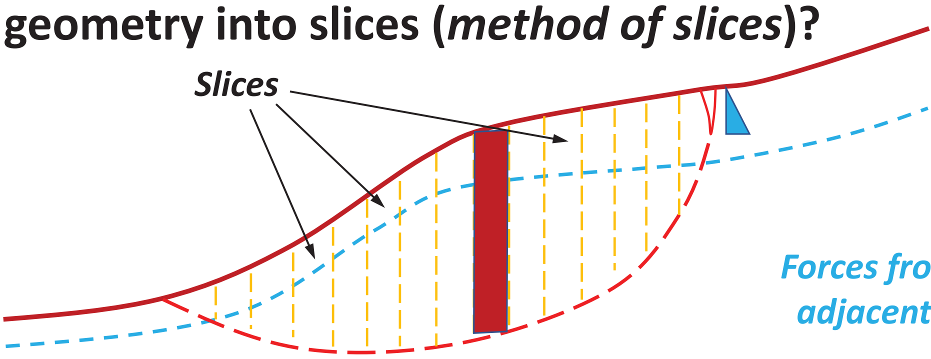


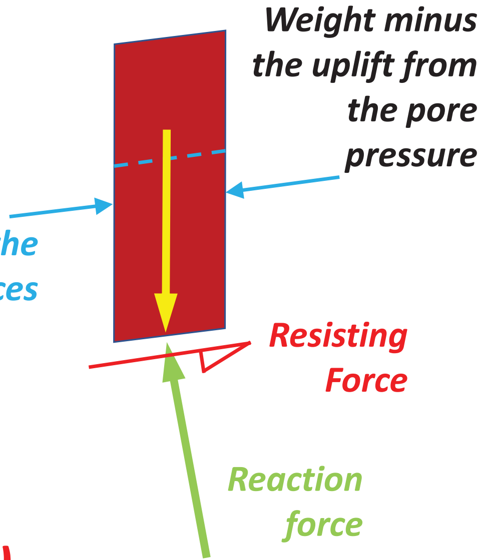






If we discretized a landslide of more complex geometry into slices (*method of slices*)?





*In search of an equilibrium of forces or moments acting on the slices (with assumptions for the location of the sliding surface and the interslice actions) we can define a **global Factor of Safety** on the basis of forces or moments:*

F.O.S. = Σ(F or M $_{resisting}$ ***) / Σ(F or M*** $_{disturbing}$ ***)***

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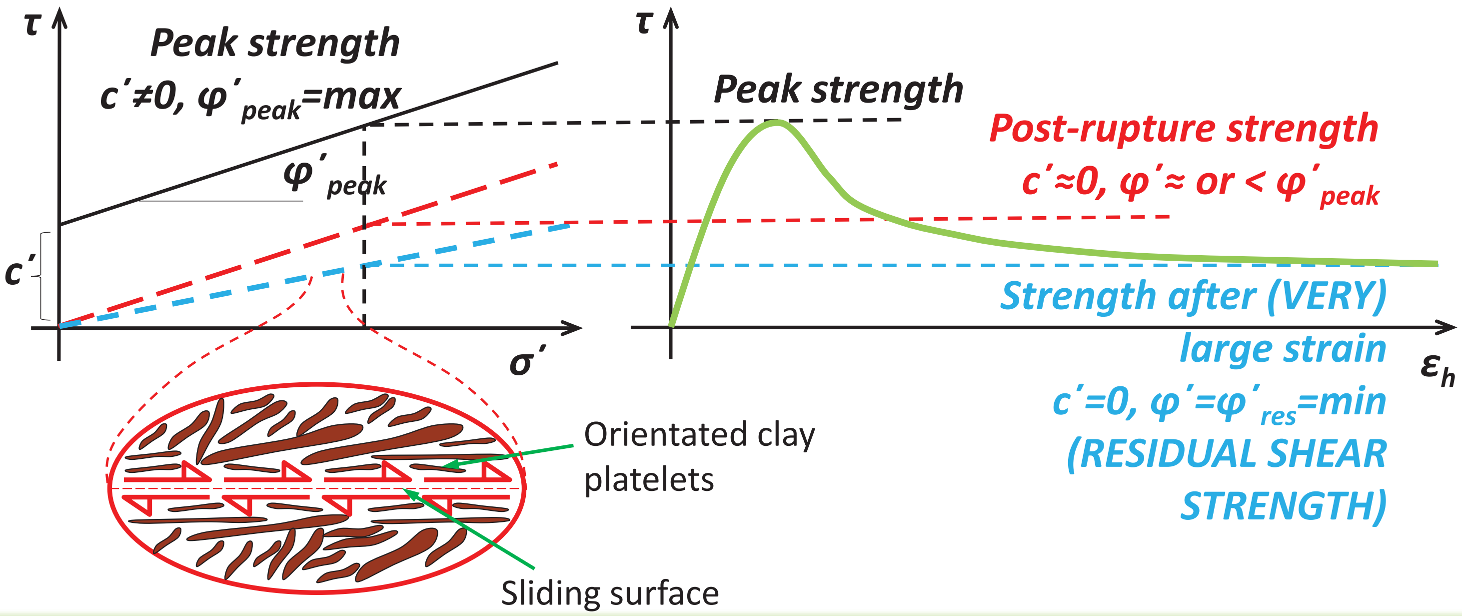
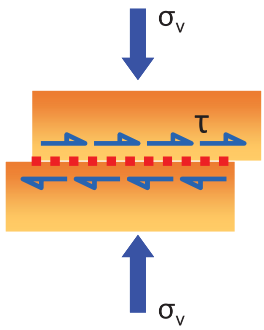
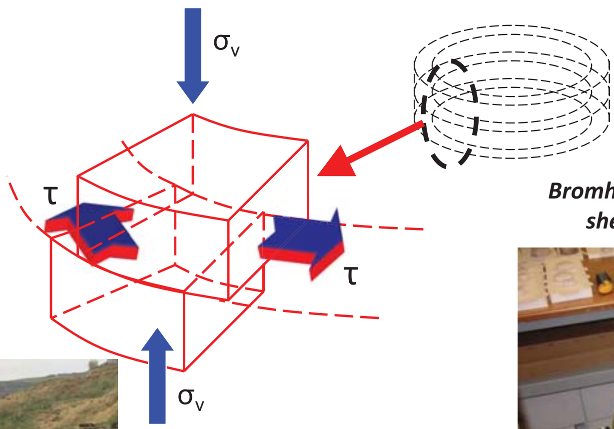
Peak, post-rupture & residual shear strength

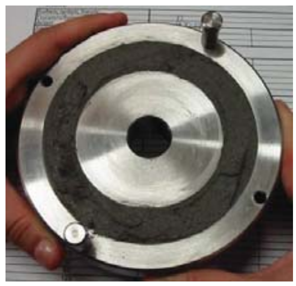


How do we apply so large deformation in the laboratory?



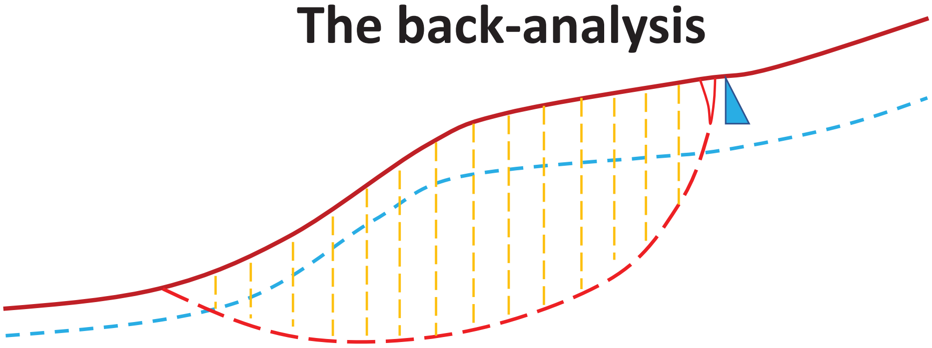
How large can strain become in the field?





Bromhead type ring shear apparatus

The back-analysis

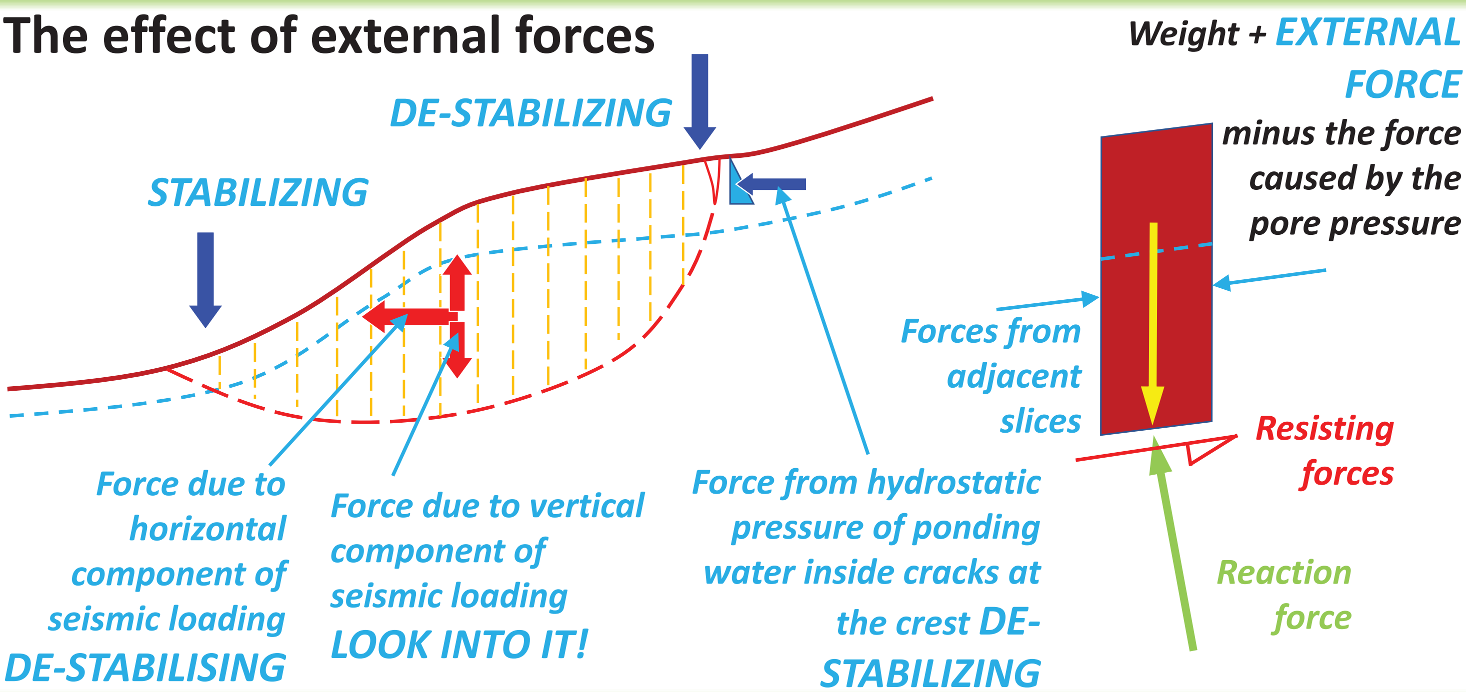


If we know the location and shape pf the sliding surface and the location of the water table, we can look for the shear strength parameters, so as:

F.O.S. = 1

Especially if we have a landslide with 'verifiably' so large deformation along the sliding surface that c΄ = 0, then we shall obtain the angle of residual shear strength that is representative of the specific sliding surface

The effect of external forces

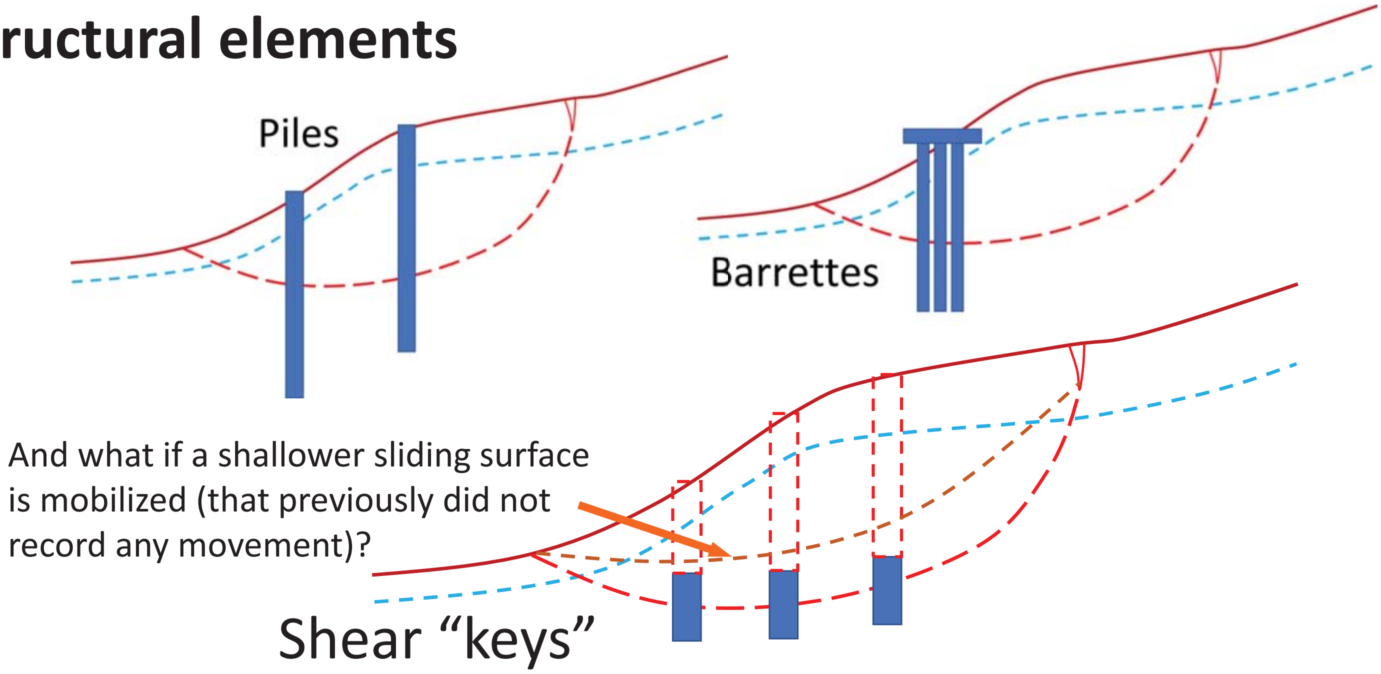


Remediation measures:

The Factor of Safety increases:

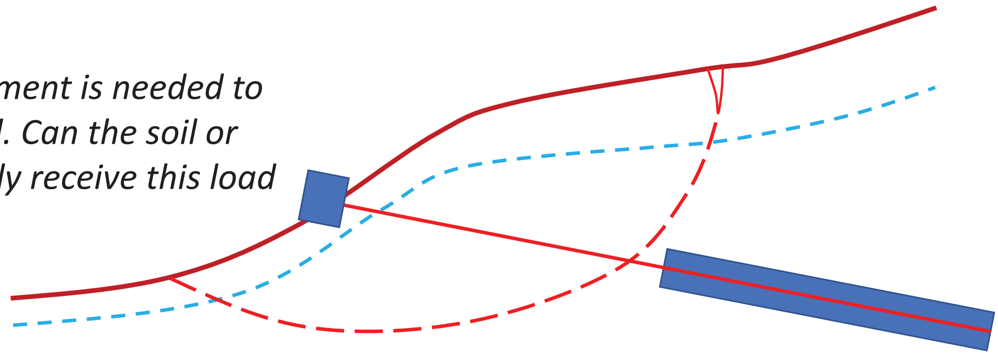
- by decreasing ***driving forces***,
- by increasing ***resisting forces***,
- by both decreasing ***driving forces*** and increasing ***resisting forces***

Structural elements



Bolts and **soil nailing** = There is not initial prestress load = movement of the sliding mass is expected so that these elements can contribute additional stabilizing force
Prestressed anchors = we introduce by prestressing an additional stabilizing force

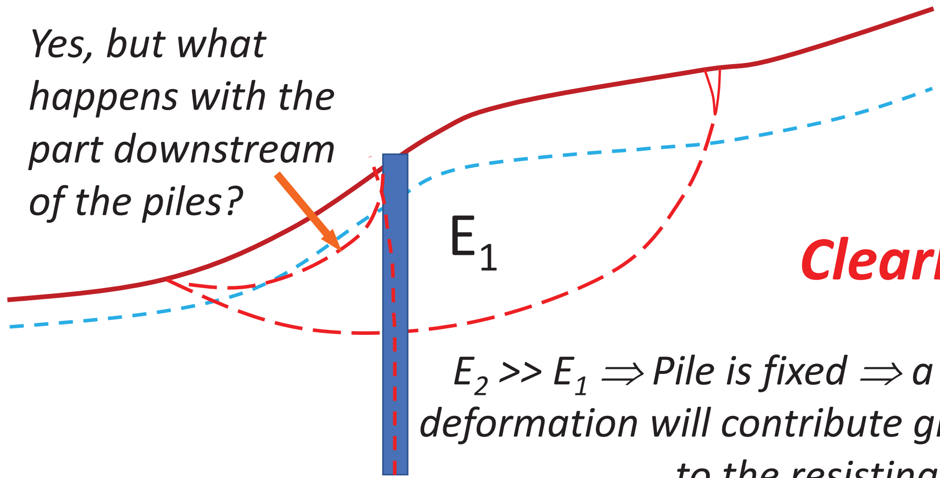
A structural element is needed to receive the load. Can the soil or rock there locally receive this load safely?



A sufficient bond length is needed for the design load to be received (does the soil or rock have the properties to receive this safely? For what length and borehole diameter can it be received safely? Can a large diameter hole be easily drilled?

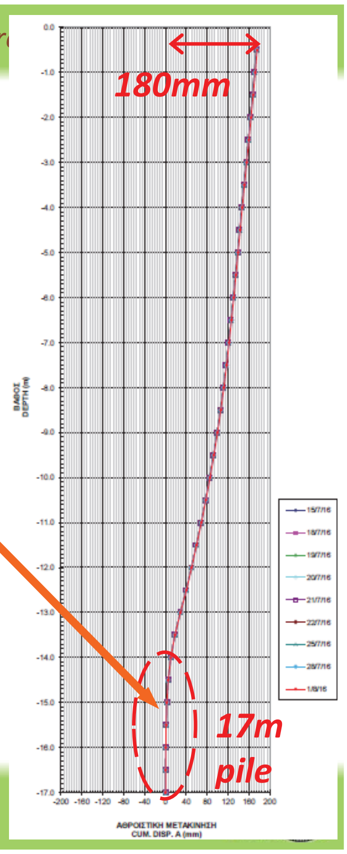
Do we have the difference in the modulus of elasticity in the sliding mass and the stable mass below so that structural elements can have an effect? **If YES**

Yes, but what happens with the part downstream of the piles?

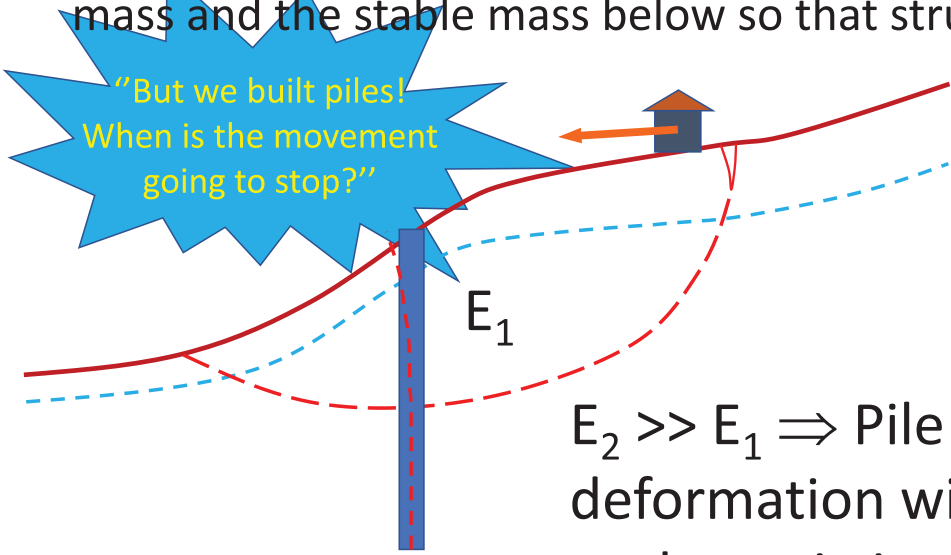


Clearly fixed!

$E_2 >> E_1 \Rightarrow$ Pile is fixed $\Rightarrow$ a small deformation will contribute greatly to the resisting force



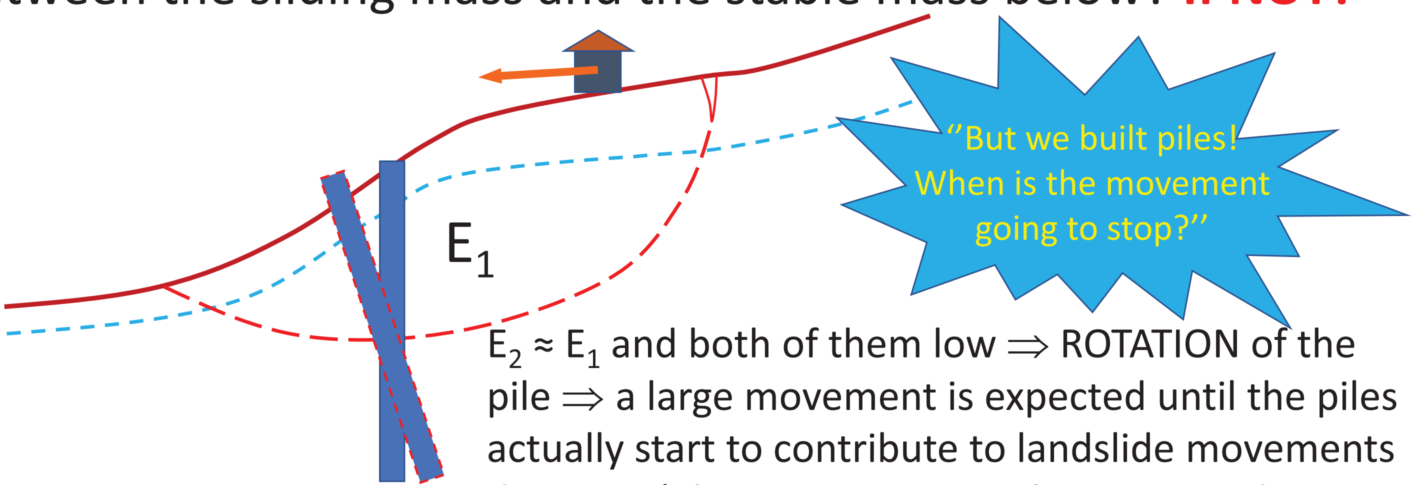
Do we have the difference in the modulus of elasticity in the sliding mass and the stable mass below so that structural elements can have an effect? **If YES**



Yes but if E_1 is very small, when is the deformation far away from the piles going to stop?

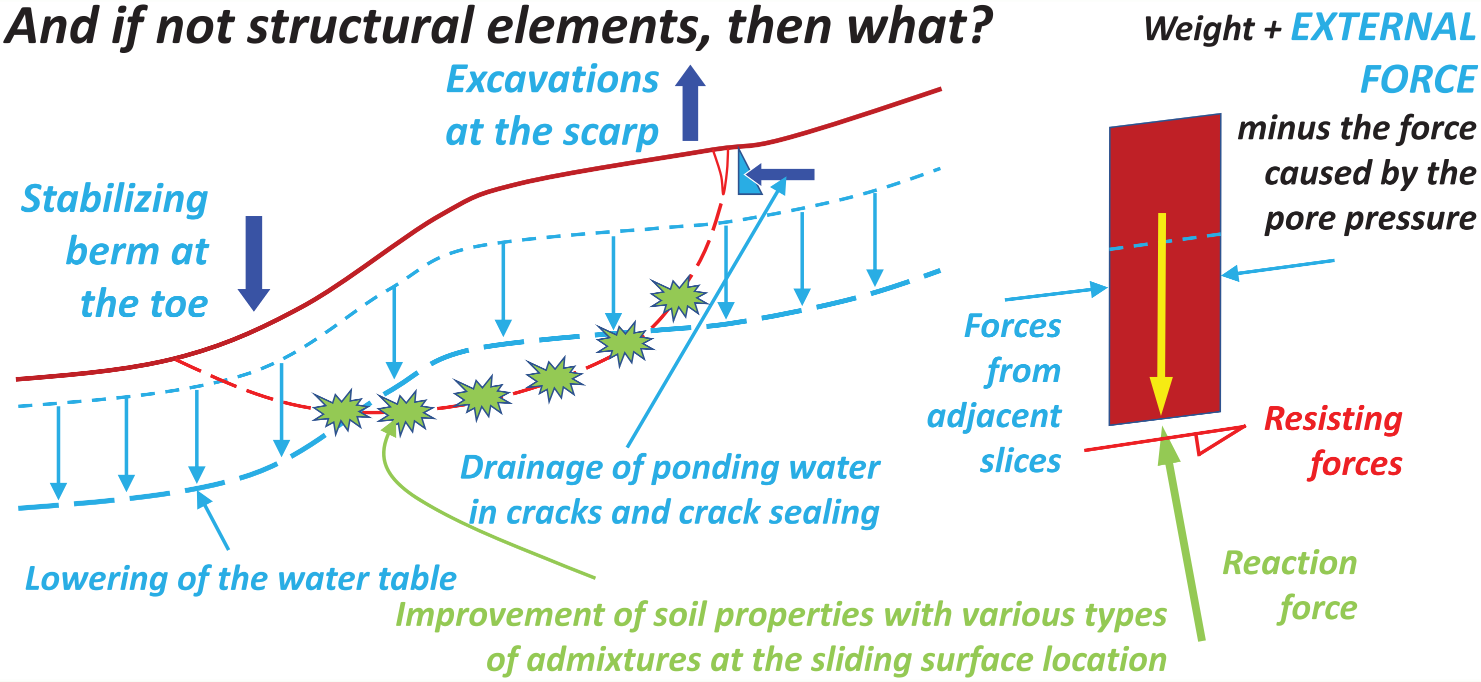
$E_2 >> E_1 \Rightarrow$ Pile is fixed $\Rightarrow$ a small deformation will contribute greatly to the resisting force

Is there a large enough difference in the moduli of elasticity between the sliding mass and the stable mass below? **If NOT?**

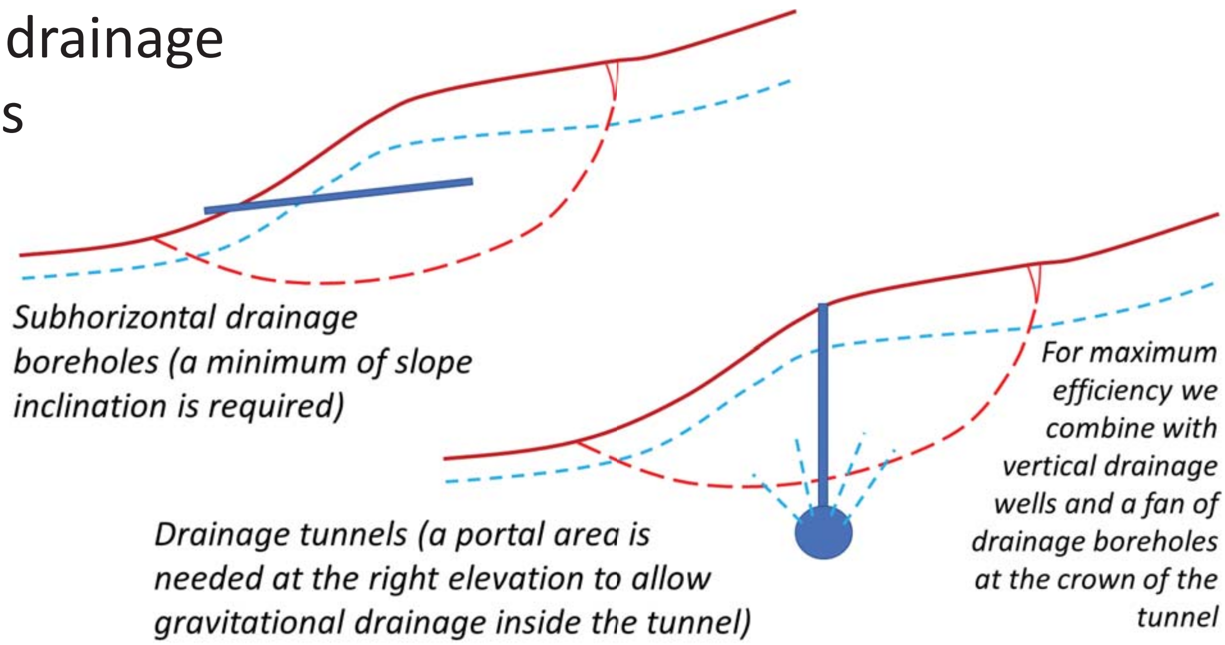


$E_2 \approx E_1$ and both of them low $\Rightarrow$ ROTATION of the pile $\Rightarrow$ a large movement is expected until the piles actually start to contribute to landslide movements decrease (plus are we certain there are no deeper sliding surfaces that may be mobilized?)

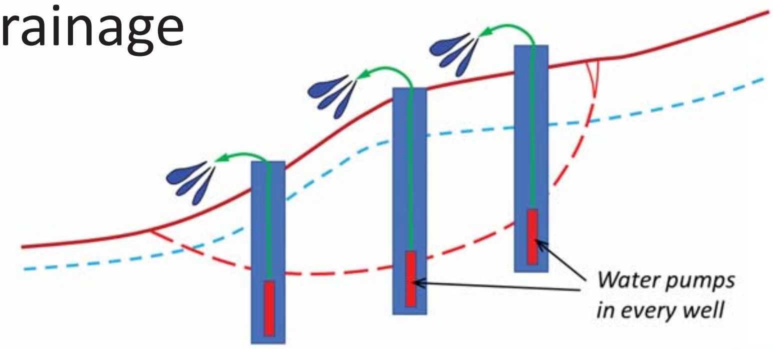
And if not structural elements, then what?



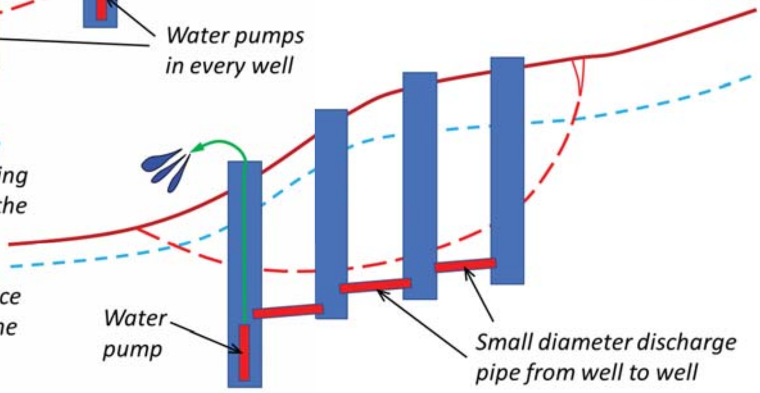
Types of drainage measures



Types of drainage measures

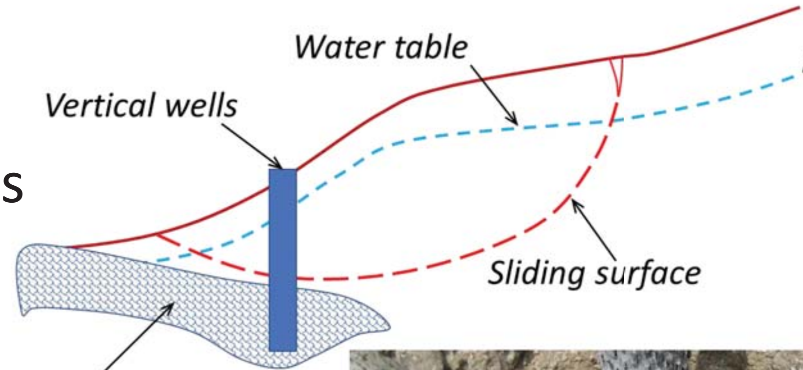


Randomly located or along lines ***vertical to the direction of movement*** *pumped wells (costly in terms of energy construction, requires maintenance and very careful electrical design at the beginning)*



Drainage wells along lines ***parallel to the direction of movement*** *with water draining from upstream to downstream and only the lowest well being pumped (less costly in terms of energy construction, requires larger well diameters but less maintenance and less meticulous electrical design at the beginning) –RODREN method, Bruce & Bianco, 1991.*

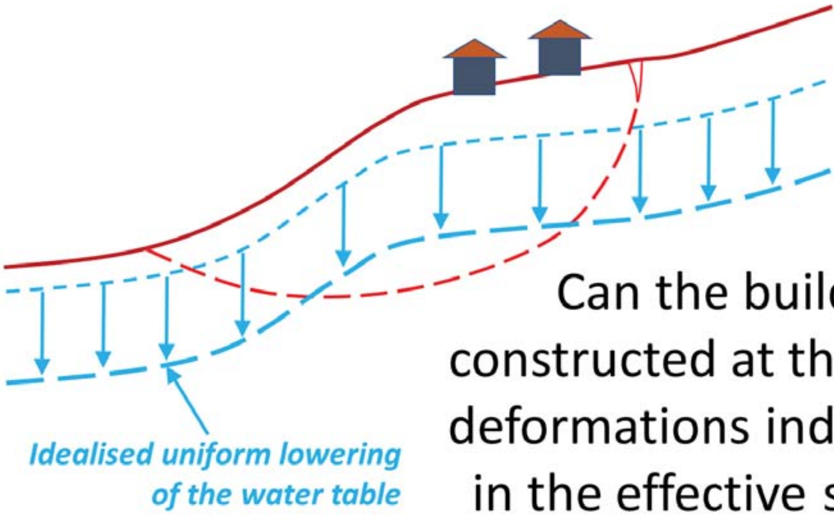
Types of drainage measures



We investigate for any permeable underlying layers of lower pore pressure, so that when vertical wells penetrate them the sliding mass drains to theis lower layer (they need only maintenance, they do NOT consume energy)

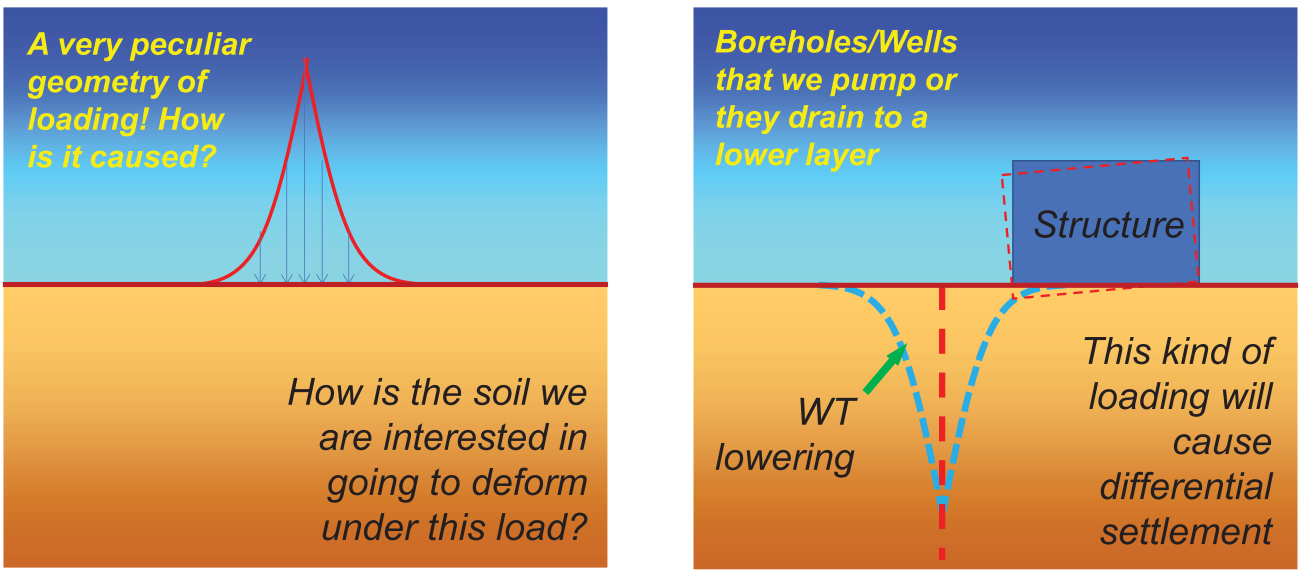
Bardanis, et al., 2022

Watch out for the deformation of buildings/structures!



Can the buildings/structures constructed at the surface sustain the deformations induced by the increase in the effective stress caused by the pore-pressure decrease?

Watch out for this strange kind of load drainage wells/boreholes cause!



Can we try to summarise all of this?

I would begin with one first question:
Do we have structures that are severely affected by movements?
*If **no**, let's start with the following approach…*

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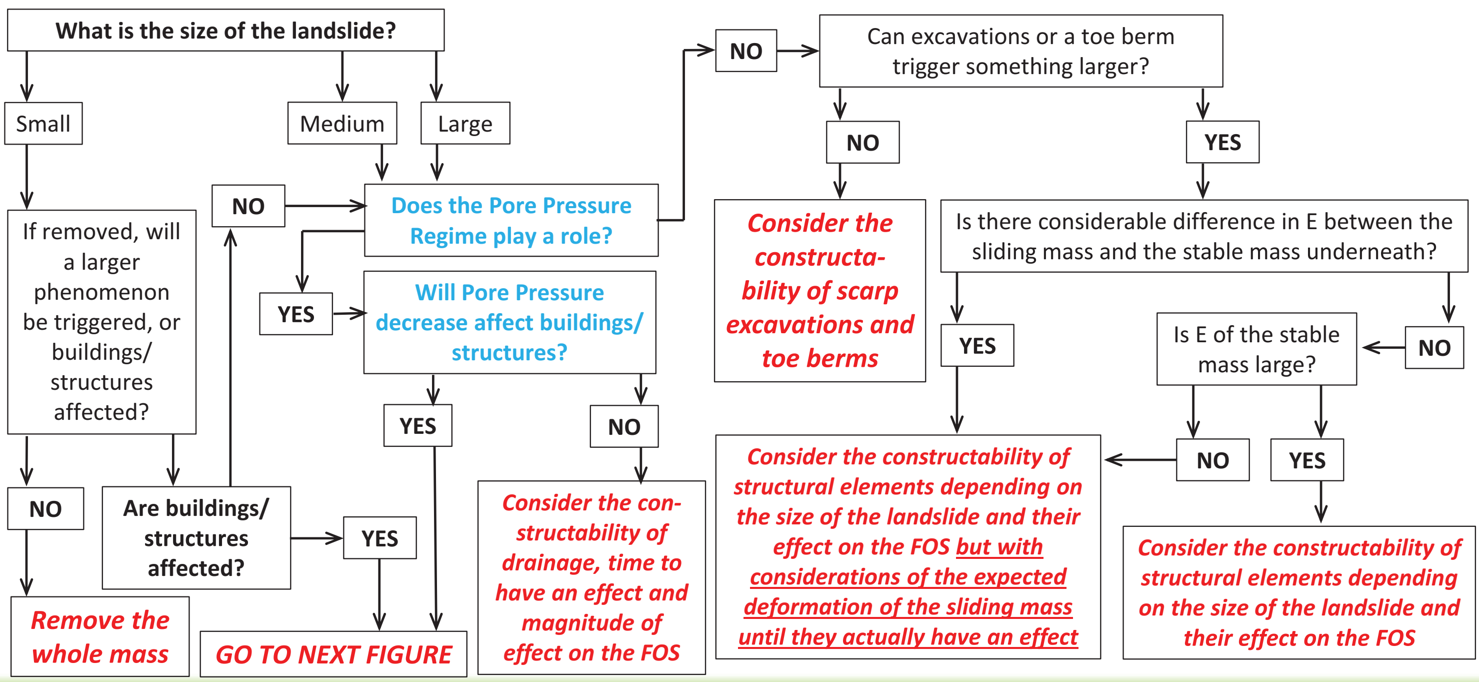

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What is the size of the landslide?

↓

Small

↓

If removed, will a larger phenomenon be triggered, or buildings/ structures affected?

↓

NO

↓

Remove the whole mass

60m wide, 6m deep in an 18m high slope

D1 landslide in fully weathered Pindos Flysch on Egnatia Highway in Northwestern Greece

What is the size of the landslide?

↓

Small

↓

If removed, will a larger phenomenon be triggered, or buildings/ structures affected?

↓

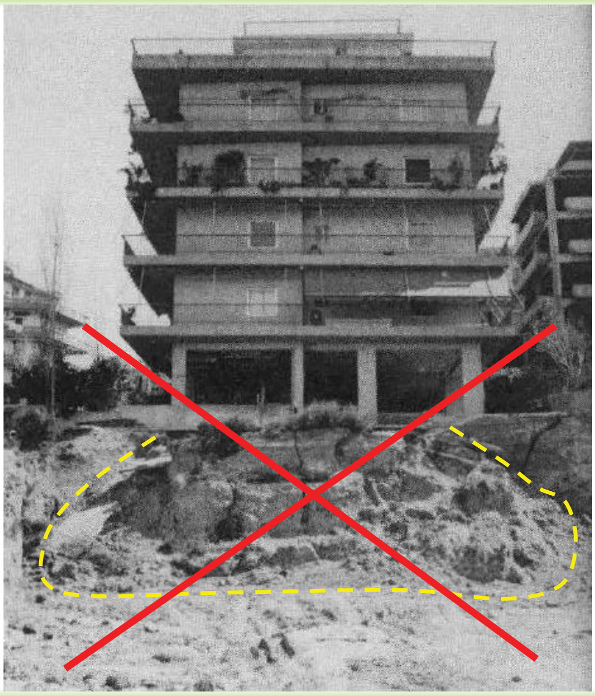
NO

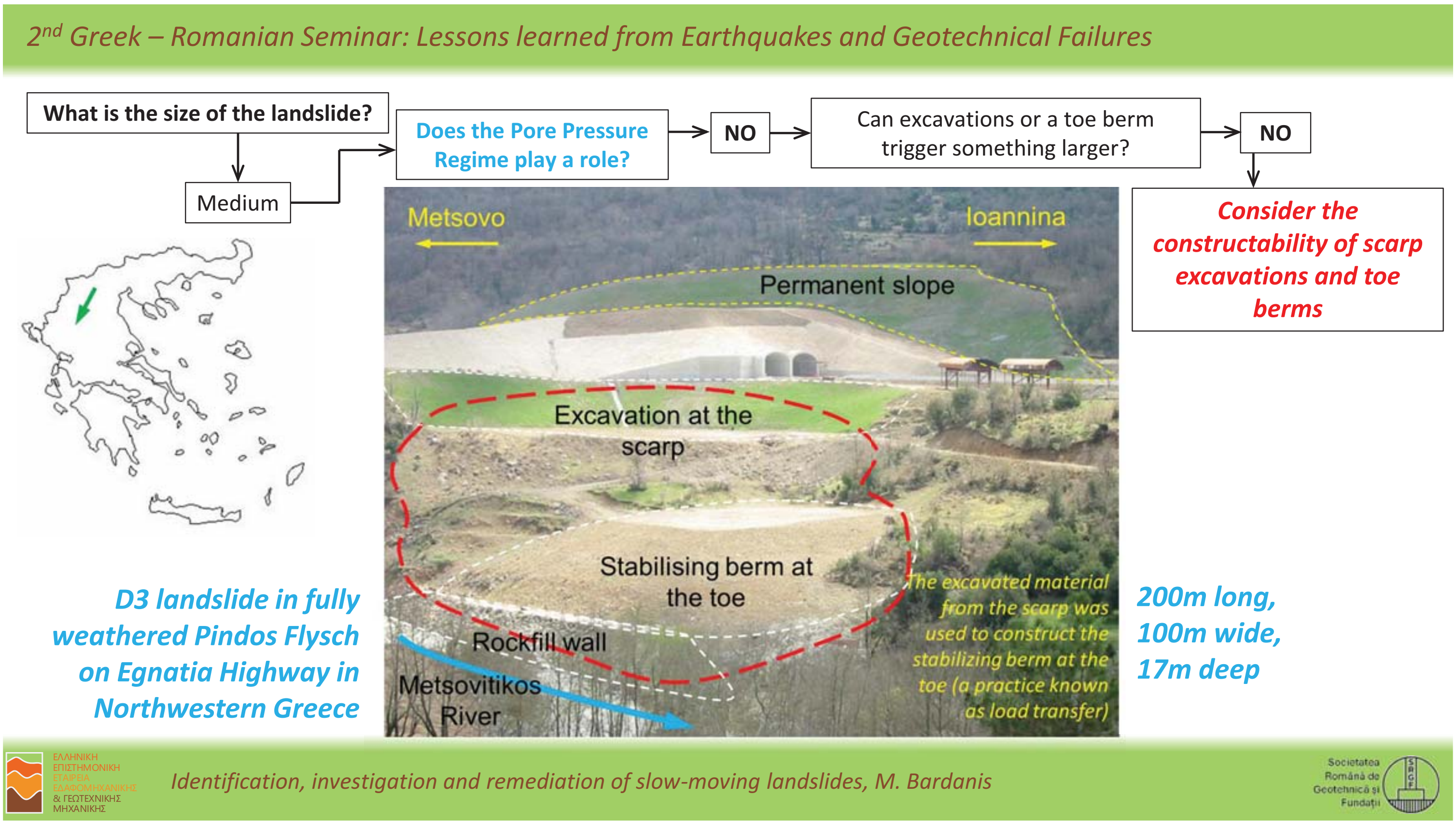
↓

Remove the whole mass

Could we do it in either of these?

NO!





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What is the size of the landslide?
Medium
Does the Pore Pressure Regime play a role?
NO
Can excavations or a toe berm trigger something larger?
NO
Consider the constructability of scarp excavations and toe berms
Metsovo
Ioannina
Permanent slope
Excavation at the scarp
Stabilising berm at the toe
Rockfill wall
Metsovitikos River
The excavated material from the scarp was used to construct the stabilizing berm at the toe (a practice known as load transfer)
D3 landslide in fully weathered Pindos Flysch on Egnatia Highway in Northwestern Greece
200m long, 100m wide, 17m deep
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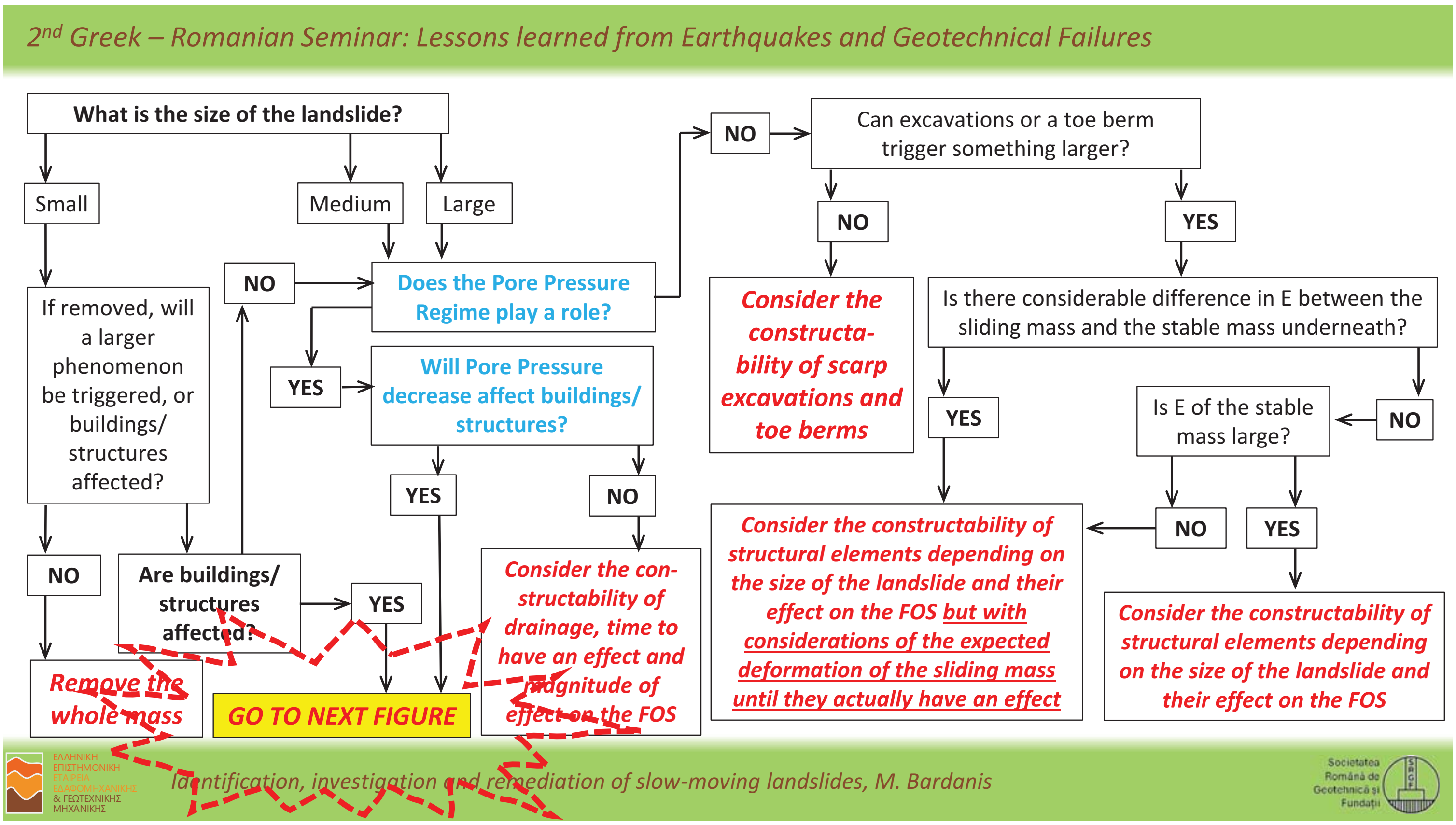

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What is the size of the landslide?
Small
Medium
Large
If removed, will a larger phenomenon be triggered, or buildings/ structures affected?
NO
Are buildings/ structures affected?
YES
Remove the whole mass
GO TO NEXT FIGURE
NO
Does the Pore Pressure Regime play a role?
YES
Will Pore Pressure decrease affect buildings/ structures?
YES
NO
Consider the constructability of drainage, time to have an effect and magnitude of effect on the FOS
NO
Can excavations or a toe berm trigger something larger?
NO
YES
Consider the constructability of scarp excavations and toe berms
Is there considerable difference in E between the sliding mass and the stable mass underneath?
YES
NO
Is E of the stable mass large?
NO
YES
Consider the constructability of structural elements depending on the size of the landslide and their effect on the FOS but with considerations of the expected deformation of the sliding mass until they actually have an effect
Consider the constructability of structural elements depending on the size of the landslide and their effect on the FOS
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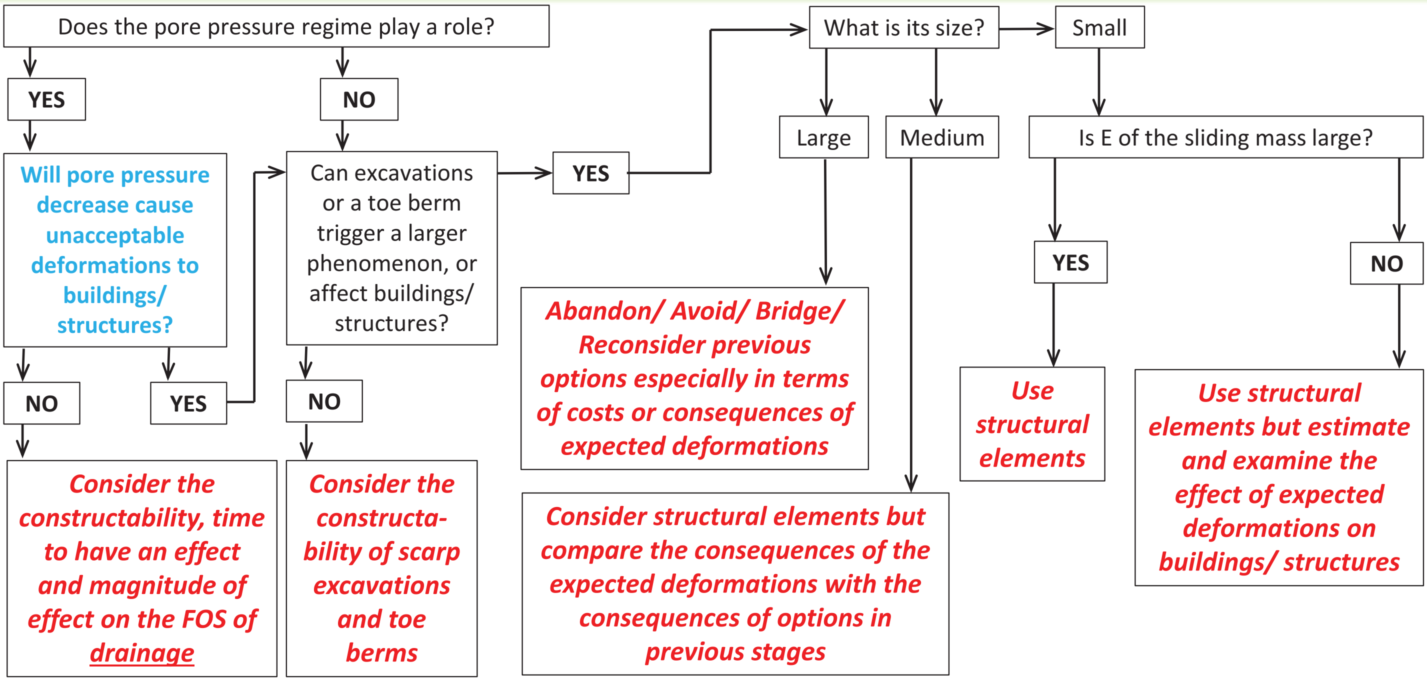

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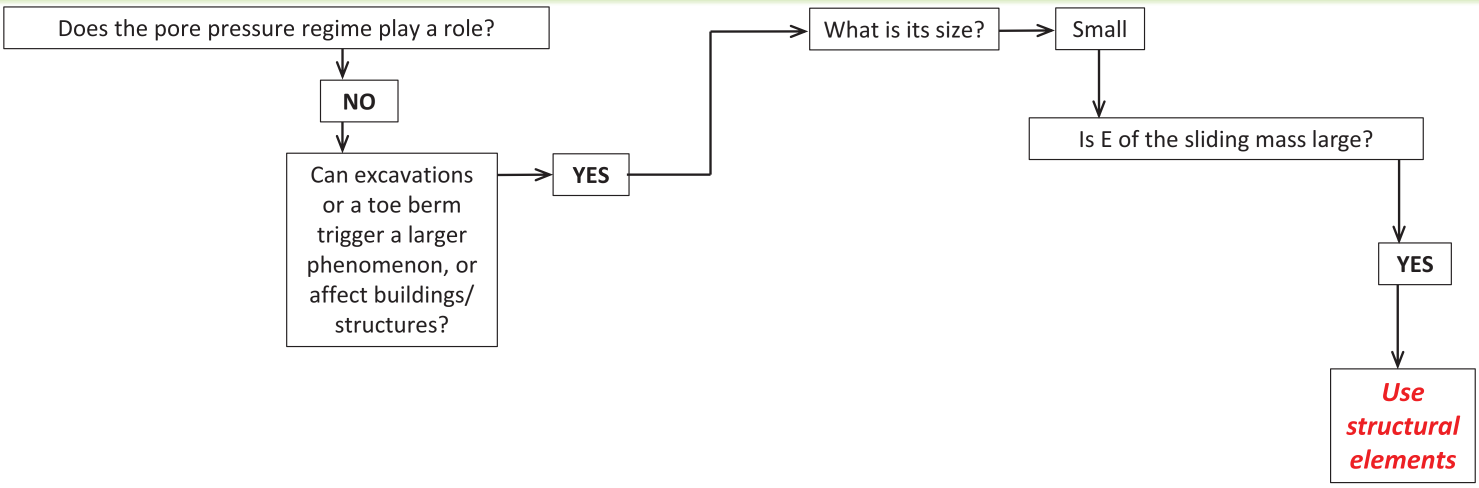

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ
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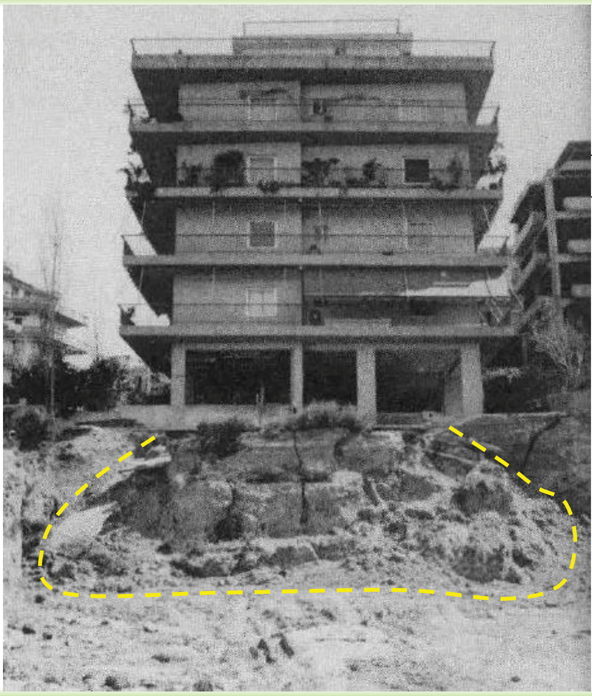
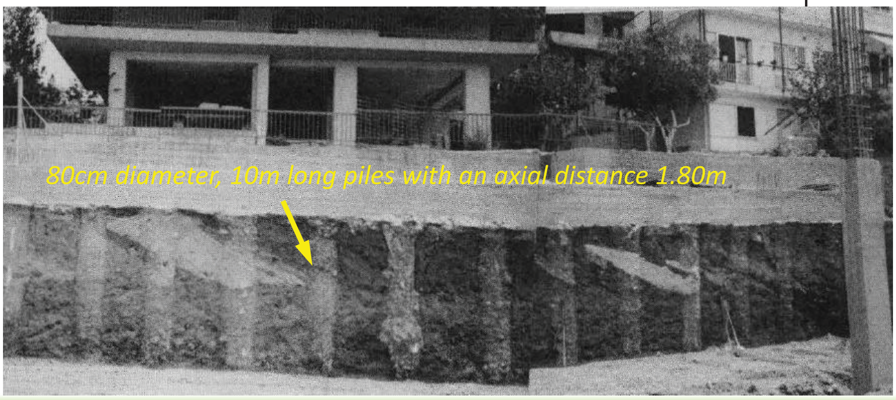


ES → What is its size? → Small

Is E of the sliding mass large?

YES



Does the pore pressure regime play a role?

YES → **Will pore pressure decrease cause unacceptable deformations to buildings/ structures?** → YES → Can excavations or a toe berm trigger a larger phenomenon, or affect buildings/ structures? → YES → What is its size?

Large → ***Abandon/ Avoid/ Bridge/ Reconsider previous options especially in terms of costs or consequences of expected deformations***

Instead of conclusions:

1. Trust your geological mapping, your geotechnical investigation and your monitoring data.

a) **Believe big**!

b) **Find the clay**!

c) If you 've not found the clay, **look for it again**!

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Instead of conclusions:

2. Then: Rate of movements →
Effect of movements on structures →
Size →
Effect of pore pressures →
Structural Elements

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ
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Colleagues and co-workers (current & former) at EDAFOS Engineering Consultants S.A.

S. Cavounidis
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M. Lotidis
P. Valvi

L. Sotiropoulos
E. Liberis
V. Karintzis
A. Ntouroupi
A. Anagnostopoulos

D. Kokoviadis
D. Tsoutsas
V. Gikas
S. Sakellariou
S. Kompogiorgas
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Closing Discussion Panel

The seminar will end with the Closing Discussion Panel moderated by HSSMGE Secretary General Giorgos Belokas and panelists Loretta Batali, Kyriazis Pitilakis, George Gazetas and Michalis Bardanis, discussing the scientific footprint and potential of these seminars on the engineering communities of the two countries, Greece and Romania, and the importance of similar bilateral activities among member societies of the ISSMGE.





G. Belokas *L. Batali* *K. Pitilakis* *G. Gazetas* *M. Bardanis*

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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SOCIETATEA ROMÂNĂ DE GEOTEHNICĂ ȘI FUNDAȚII