SOCIETATEA ROMÂNĂ DE GEOTEHNICĂ ȘI FUNDAȚII
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Universitatea Tehnică de Construcții București

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

Romania - Greece Seminar

Earthquake & Geotechnical Engineering

Conference Centre UTCB

March 27th, 2025

The Romanian Society for Soil Mechanics and Foundation Engineering (SRGF), together with
the Technical University of Civil Engineering Bucharest (UTCB), in cooperation with
the Hellenic Society for Soil Mechanics and Foundation Engineering (HSSMGE) are announcing

the 1st Romania – Greece Seminar on Earthquake and Geotechnical Engineering

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.

Special guest speakers from Greece:

George Gazetas

(Emeritus Professor of Geotechnical Engineering at the National Technical University of Athens)

Foundation Design and Soil–Structure Interaction in the new Ec8

George Gazetas is Emeritus Professor of Geotechnical Engineering at the National Technical , University of Athens (NTUA, "Metsovion"), in which he served as Professor for 30 years, following an academic career in the US, where he taught at SUNY-Buffalo, Rensselaer (RPI), and Case Western Reserve University. He had studied as undergraduate at NTUA (Diploma in Civil Engineering) and as graduate at MIT (MS and PhD in Geotechnical Earthquake Engineering). His main research interests have focused on the dynamic response of footings, piles and caissons; the seismic response of earth dams and quay-walls; soil amplification of seismic waves; and soil–structure interaction under static and seismic excitation. Much of his research has been inspired by observations after destructive earthquakes. An active lecturer, he was the keynote speaker in many international conferences. He is the author of significant journal publications (achieving an h-index of 84, the highest among all Greek civil engineers, and the highest worldwide in Soil Dynamics). He is a member of the Technical Committee for the revision of Eurocode 8, and has served as President of both, the Greek Committee of Soil Mechanics & Geotechnical Engineering and the Hellenic Association of Earthquake Engineering. He has also been a consultant or referee in some major dynamic geotechnical projects in several countries, including the: Rion-Antirrion Bridge (Greece), Diablo Canyon Nuclear Power Plant (USA), Brunsbüttel Reactor Building (Germany), Olympic Stadium Opening Ceremony Shaft (Greece), Queensboro and Williamsburg NY Bridges (USA), Tagus River Bridge (Portugal), Messochora CFR Dam (Greece), Ohba-Ohashi Bridge (Japan). Recipient of several awards for his research, he has delivered some prestigious lectures including the "Coulomb", "Ishihara", "Keneth Lee", and "Michele Maugeri" Lectures. In 2015 he was awarded the Excellence in University Teaching Prize, the ultimate teaching award in Greece. Recipient of the prestigious European Research Council's [ERC] Ideas Advanced Grant award, he was honored as the 59th Rankine Lecturer, 2019, in London, and as a GeoLegend by ASCE's Geotechnical Institute in the GeoStrata magazine, 2022.

Kyriazis Pitilakis

(Emeritus Professor, Aristotle University Thessaloniki, Vice President of the European Association of Earthquake Engineering)

Definition of seismic actions in the revised EC8 and implication in the seismic risk assessment

Professor Kyriazis Pitilakis has more than 45 years of intensive academic, research and professional experience in civil, earthquake and geotechnical engineering. President and since 2022 and since then Vice President of the European Association of Earthquake Engineering (EAEE) (www.eaee.org), ex-Chairman of the Technical Committee "Geotechnical Earthquake Engineering and Associated Problems" (TC203) of the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE), and past President of the Greek Society of Earthquake Engineering, he is presently Professor Emeritus in Aristotle University, Thessaloniki, Greece and *since 2019 visiting Professor in Tongji University, ILEE, Shanghai, China. With almost 700 publications in scientific journals and conference proceedings (h-index 66) he is according to the recent Stanford classification among the top 10 leading researchers in Civil Engineering in Greece, and among the top 10 leading researchers worldwide in Soil Dynamics, Geotechnical Earthquake Engineering and Engineering Seismology. His scientific and research interests cover a wide range from soil dynamics, site effects and microzonation studies, seismic hazard, strong ground motion, and risk assessment to various topics in geotechnical earthquake engineering from the seismic design, vulnerability and risk assessment of buildings, to tunnels, infrastructures and geotechnical structures. He has been invited keynote lecturer in many international and world conferences including (recently) the European Conference in Earthquake Engineering and the World Conference of Tunnels (2023). He is member of the Technical Committee and Working Groups for the revision of Eurocode 8 (Part 1-Seismic Actions and Part 5-Seismic design of foundations, retaining structures, soil-structure interaction, liquefaction, slope stability and underground structures). Many of his PhD and post-doc students hold academic positions in Greece and abroad, namely in China, Italy, UK, France, USA, Egypt, Hong-Kong. Honors: Chevalier dans l'Ordre des Palmes Academiques, French Republic.*

Giorgos Belokas

(Assistant Professor in Geotechnical Engineering, University of West Attica, General Secretary of the Hellenic Society on Soil Mechanics and Geotechnical Engineering)

Ultimate Limit State Design Analysis of Foundations and Representative Strength of Soils in the new Ec7

Dr Giorgos Belokas is an Assistant Professor in Geotechnical Engineering and the Head of the "Hydraulic and Geotechnical Engineering Division" at the University of West Attica. He holds a Diploma in Civil Engineering from National Technical University of Athens, an MSc in Soil Mechanics from Imperial College and a PhD from NTUA. His lecturing experience covers a wide range of geotechnical engineering courses, while currently he teaches Soil Mechanics, Geotechnical Works, Slope Stability – Embankments and Retaining Walls and Deep Excavations. His research work and interest includes constitutive modelling of anisotropic structured clays and of unsaturated soils, coupled deformation consolidation theory, field measurements and laboratory testing of unsaturated soils, numerical and probabilistic analyses of geotechnical works, soil – atmosphere effect on soil slope stability and foundation problems and, finally, resilience quantification methods for infrastructures. His professional experience is in the geotechnical engineering analysis and design of geotechnical major infrastructure works, with emphasis on the investigation and remediation of landslides, and in laboratory testing, with emphasis on the implementation and application of laboratory ISO 17025, while he has been a consultant for EIB. He is the Convenor of WG7 of ELOT/TC67 (mirroring CEN/TC 250/SC 7) and also member of TG A2 "NSB contact group" and TG B2 "Design Examples" for the ongoing development of the 2nd generation of CEN/TC 250/SC 7 "Eurocode 7 - Geotechnical design". He has been a member of various ISSMGE TCs including currently the ERTC10 "Evaluation of Eurocode 7" and the TC202 "Field Monitoring in Geomechanics. Since 2010 he is an elected board member and since 2019 the General Secretary of the Hellenic chapter of ISSMGE.

Special invited speakers from Romania:

Radu Văcăreanu

(Professor of Structural Reliability and Risk Analysis at the Technical University of Civil Engineering Bucharest, President of the European Association of Earthquake Engineering)

Probabilistic seismic hazard assessment and calibration of elastic design spectra in Romania

Radu Văcăreanu graduated Civil Engineering in 1991 from the Technical University of Iasi, Romania. He got his Ph.D. from the Technical University of Civil Engineering of Bucharest (UTCB) in 1999 in the field of seismic risk assessment. Currently, he is Professor of Structural Reliability and Seismic Risk Analysis at UTCB. Radu Văcăreanu is President of the European Association for Earthquake Engineering (EAEE), National Delegate of Romania at the International Association for Earthquake Engineering (IAEE) and Executive President of the National Commission for Earthquake Engineering of Ministry of Development, Public Works and Administration of Romania. In between 2002 and 2008 he served as the director of the National Centre for Seismic Risk Reduction, implementing the JICA (Japan Technical Cooperation Agency) Project on Seismic Risk Reduction for Buildings and Structures in Romania. Radu Văcăreanu participated in international projects and coordinated national projects aiming at seismic risk reduction. He published numerous papers in peer-reviewed journals and international conferences proceedings. His research interest covers mainly the seismic hazard, fragility and risk analyses. Since March 2016 he serves as Rector of the Technical University of Civil Engineering of Bucharest.

Loretta Batali

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

Evolution and perspectives in the geotechnical design according to the 2nd generation of Eurocode 7

Loretta Batali is full professor and habilitated for PhD research at the Technical University of Civil Engineering Bucharest (UTCB), 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 (TC 250/SC7), first as member of PT1 and then leading TG B on design examples and from 1.01.2025 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).

Cristian Arion

(Professor of Structural Reliability and Risk Analysis at the Technical University of Civil Engineering Bucharest)

Direct and proxy seismic site characterisation in Romania

Cristian Arion is associate professor of Structural Reliability and Risk Analysis at UTCB. He was an author of the national building codes for wind action, snow action, and earthquake resistance. As part of his research he worked at CEDEX Madrid, Building Research Institute, Tsukuba, Japan, Tokyo University (Towhata lab), Waseda University (Hamada lab) and at Tokyo Soil Research Co.Ltd. He holds a postgraduate diploma in Earthquake Engineering of the International Institute of Seismology and Earthquake Engineering Tsukuba, Japan. He has expertise in site seismic analysis for industrial facilities, hotels, banks; seismic design of reinforced concrete buildings; acquiring ground and underground vibration measurements for buildings; assessment of soil amplification factors and the dynamic characteristics of soils; running the dynamic triaxial equipment; seismic prospecting; seismic monitoring; and the development of GIS microzonations. He has conducted studies of the probabilistic and deterministic seismic hazard of Romania, seismic sources affecting Romania, seismic response, and geophysical exploration (downhole, microtremor measurements).

Contributions are welcomed and proposals can be sent before **January, 30th 2025** by email (loretta.batali@utcb.ro).

The final agenda and the details for the registration to the event will be shared in due time, beginning of 2025.

We invite all engineers, researchers, students, and professionals interested in earthquake and geotechnical engineering to join us for this unique opportunity to exchange knowledge and expertise with the Greek specialists.

We look forward to your participation in this exciting and informative event!

Prof. Loretta Batali
President SRGF

This is the compiled pdf version of the presentations by speakers who presented at the 1st Romania-Greece Seminar on Earthquake and Geotechnical Engineering, organized by the Romanian Society for Soil Mechanics and Geotechnical Engineering (SRGF) and the Hellenic Society for Soil Mechanics and Geotechnical Engineering (HSSMGE) in Bucharest, Romania, on the 27th of March 2025.

"Presenter/Presenters, 2025, Title of presentation, 1st Romania-Greece Seminar on Earthquake and Geotechnical Engineering, Bucharest, Romania, 27th of March 2025, pp. the pages from this compiled version."

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Earthquake & Geotechnical Engineering

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AGENDA March 27th, 2025

9 :30 – 10 :00 – *Welcome coffee*

10 :00 – 10 :15 – *Opening*
Prof. Loretta Batali – President SRGF
Prof. Radu Văcăreanu – Rector UTCB, President European Association of Earthquake Engineering
Dr. Michalis Bardanis – President Hellenic Society of Soil Mechanics and Foundation Engineering

10:15 – 11 :00 – Prof. Loretta Batali - *Evolution and perspectives in the geotechnical design according to the 2nd generation of Eurocode 7*

11:00 – 11:45 – Prof. Kyriazis Pitilakis - *Definition of seismic actions in the revised EC8 and implication in the seismic risk assessment*

11:45 – 12:30 – Prof. George Gazetas - *Foundation Design and Soil–Structure Interaction in the new EC8*

12:30 – 13:15 – Assist. Prof. Giorgos Belokas - *Ultimate Limit State Design Analysis of Foundations and Representative Strength of Soils in the new EC7*

13:15 – 14:00 – *Lunch break*

14:00 – 14:45 – Prof. Radu Văcăreanu - *Probabilistic seismic hazard assessment and calibration of elastic design spectra in Romania*

14:45 – 15:30 – Assoc. Prof. Cristian Arion - *Direct and proxy seismic site characterisation in Romania*

15:30 – 15:45 – Prof. Alexandru Aldea (UTCB), Florin Pavel (UTCB), Etienne Bertrand (Université Gustave Eiffel) - *UTCB site response based on 20 years of observation*

15:45 – 16:00 – Eng. Alexandra Ene – *Assessment of soils stiffness for small strains by in situ tests and correlations for sites in Bucharest*

16:00 – 16:15 – Assist. Prof. Anabella Coţovanu - *Local site conditions in hybrid strong ground motion simulation*

16:15 – 16:45 – *Discussions and closing*

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Honouring Prof Vacareanu (top) and Prof Batali (bottom) on behalf of HSSMGE for their efforts in organizing the 1st Romania-Greece Seminar.

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Evolution and perspectives in the geotechnical design according to the 2nd generation of Eurocode 7

Prof. Loretta Batali
UTCB

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Summary

1. Objectives of the revision of EC7
2. Key changes in the 2nd generation of EC7
3. Guidance documents
4. Further actions

1. Objectives of the revision of EC7

- Standard suitable for all common design cases without demanding disproportionate effort
- Ease of use: Clear language, same structure in all Eurocodes,
  Avoid alternative rules
  No rules of little practical use, no "textbook"
- Harmonization: Common ULS-verifications
- Developments: Numerical Methods
  Probabilistic design
  New geotechnical structures
  Sustainability (thermo-active geostructures – piles, diaphragm walls)

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1. Objectives of the revision of EC7

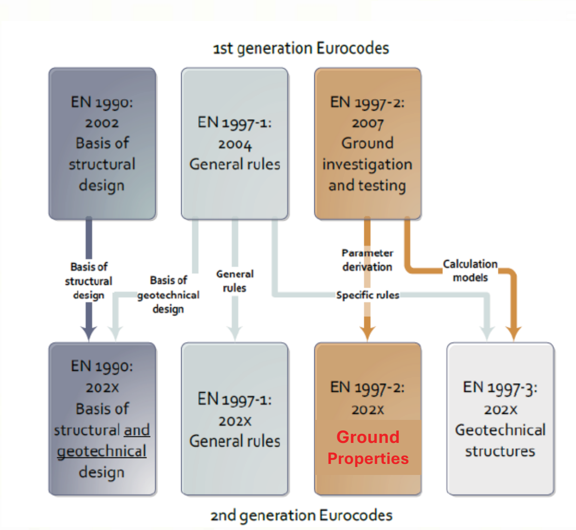
- **Reorganisation**

1st generation Eurocode:

- EN1990 – Basis of structural design
- EN1997-1 – General rules (for geotechnical design)
- EN1997-2 – Ground investigation and testing

2nd generation Eurocode:

- EN1990 – Basis of structural and geotechnical design
- EC7 Part 1 – General rules
- EC7 Part 2 – Ground Properties
- EC7 Part 3 – Geotechnical structures



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1. Objectives of the revision of EC7

At present:

EN 1997-1:2024 – Geotechnical design. Part 1: General rules

EN 1997-2:2024 – Geotechnical design. Part 2: Ground properties

FprEN 1997-3:2024 – Geotechnical design. Part 3: Geotechnical structures

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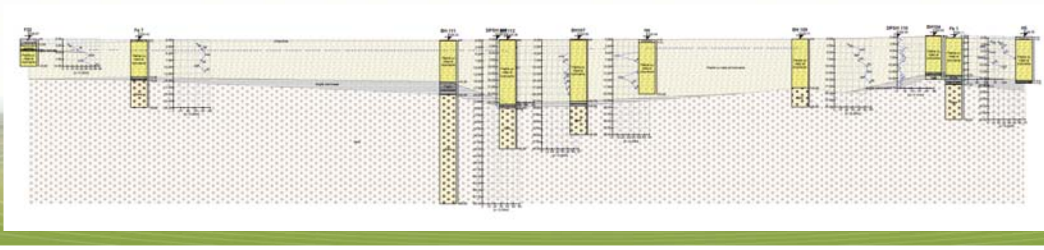
1. Objectives of the revision of EC7

REVISION

- New developments

New concepts:

- **Rock engineering included everywhere (Soil + Rock = Ground!)**
- **Geotechnical Category is now combination of Complexity of structure (CC) and Ground (GCC)**
- **Representative value - "cautious estimate" and/or "statistical approach**
- **New Focus for Part 2 – "How do I derive a ground property?" Focus from "Output from Testing" to "Input for Design"**
- **Ground Model and Geotechnical Design Model**



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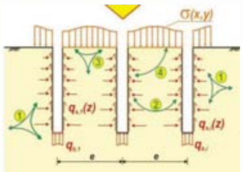
1. Objectives of the revision of EC7

REVISION

- **New developments**

New technical developments in Part 3:

- **Pile groups, piled rafts**
- **Reinforced fill**
- **Rock bolts, soil nails**
- **Ground Improvement techniques: stone columns, grouting, rigid inclusions**
- **Measures for groundwater control**



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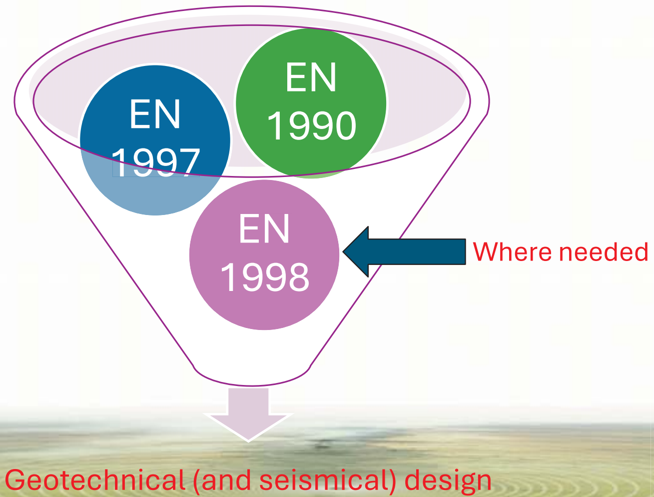
2. Key changes in the 2nd generation of EC7

Toolkit



2. Key changes in the 2nd generation of EC7

Toolkit - What info in what document?

EN 1990	EN 1997-1 – General rules	EN 1997-2 – Ground properties	EN 1997-3 – Geotechnical structures
• General design principles • Consequence classes and consequence factors K_F • Partial coefficients for actions and effects of actions • Verification cases for ULS	• Geotechnical categories (GC) • Representative values of geotechnical parameters (characteristic, nominal) X_{rep} • Geotechnical design model (GDM) • Partial factors on ground properties (γ_M) • Consequence factors K_M, k_R • ULS and SLS for geotechnical design	• Ground model (GM) and derived ground properties • Laboratory and ground investigation	**For each geotechnical structure** • Conditions for ground investigation • Groundwater • Geotechnical analysis • ULS and SLS verifications • Partial coefficients applied to resistances γ_R • Model factors γ_{Rd} • Implementation of design • Testing • Reporting

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2. Key changes in the 2nd generation of EC7

EN1990 – Basis of structural and geotechnical design

ULS - Partial coefficients for actions and effect-of-actions – verification cases VC1 – VC4

A.1.9 (NDP) — Consequence factors for buildings and geotechnical structures

Consequence class (CC)[a]	Description of consequences	Consequence factor k_F
CC3	High	1,1
CC2	Normal	1,0
CC1	Low	0,9

[a] The provisions in Eurocodes cover design rules for structures classified as CC1 to CC3, see 4.3.

Action or effect				Partial factors γ_F and γ_E for Verification Cases 1 - 4				
Type	**Group**	**Symbol**	**Resulting effect**	**Structural**	**Static equilibrium and Uplift**		**Geotechnical Design**	
				Foundations Raft/piled	Uplift - water		Slopes	Retaining walls
				VC1	VC2(a)	VC2(b)	VC3	VC4
Permanent Action (G_k)	All	γ_G	unfavourable/ destabilising	1,35 K_F	1,35 K_F	1,0	1,0	G_K is not factored
	Water	$\gamma_{G,w}$		1,2 K_F	1,2 K_F			
	All	$\gamma_{G,stb}$	stabilising	Not used	1,15		Not used	
	Water	$\gamma_{G,w,stb}$			1,0			
	(All)	$\gamma_{G,fav}$	favourable	1,0	1,0		1,0	
Variable action (Q_k)	All	γ_Q	unfavourable	1,5 K_F	1,5 K_F		1,3	1,1 (≈1.5/1.35)
	Water	$\gamma_{Q,w}$		1,35 K_F	1,35 K_F		1,15	1,0
	(All)	$\gamma_{Q,fav}$	favourable	0				
Effects-of-actions (E)		γ_E	unfavourable	Effects are not factored				1,35 K_F
		$\gamma_{E,fav}$	favourable					1,0

2. Key changes in the 2nd generation of EC7

EN1990 – Basis of structural and geotechnical design

- **VC1** is used both for structural and geotechnical design.
- **VC2** is used for the combined verification of strength and static equilibrium, when the structure is sensitive to variations in permanent action arising from a single-source. VC2(a) or VC2(b), whichever gives the less favourable outcome.
- **VC3** is typically used for the design of slopes and embankments, spread foundations, and gravity retaining structures.
- **VC4** is typically used for the design of transversally loaded piles and embedded retaining walls and (in some countries) gravity retaining structures.

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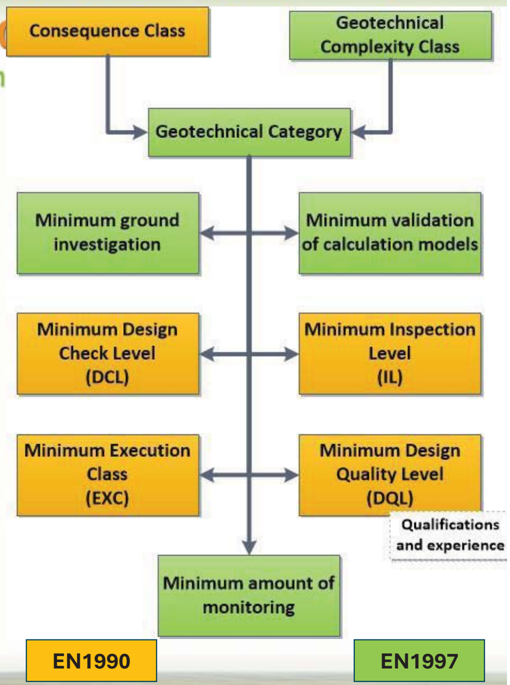
2. Key changes in the 2nd generation of EC7

Key changes in **EN 1997-1**

Risk mitigation by using the **new Geotechnical Category (GC):**

Geotechnical category (GC) = Consequence class CC x Geotechnical complexity class (GCC)

Consequence Class (CC)	Geotechnical Complexity Class (GCC)		
	Lower (GCC1)	Normal (GCC2)	Higher (GCC3)
High (CC3)			GC3
Medium (CC2)		GC2	
Low (CC1)	GC1		



(van Seters, 2023)

2. Key changes in the 2nd generation of EC7

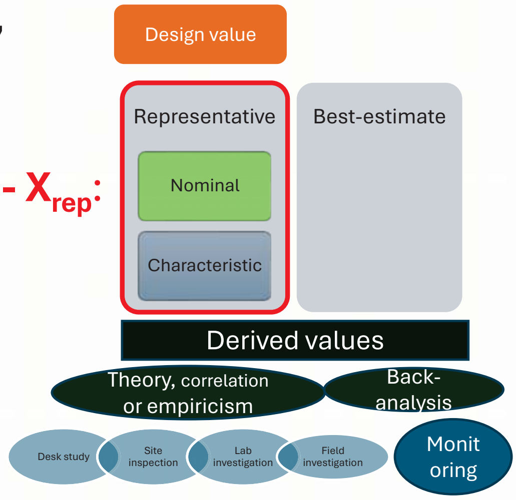
Key changes in **EN 1997-1**

Representative value of geotechnical parameters - X_{rep}:

Characteristic value – statistically determined

Nominal value – based on engineering judgement – cautious estimate

+ Best-estimate – for prognosis



(van Seters, 2023)

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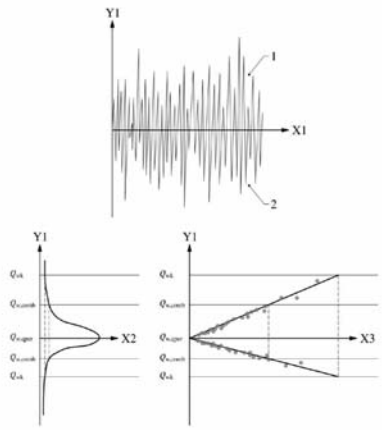
2. Key changes in the 2nd generation of EC7

Key changes in EN 1997-1

Representative value of groundwater pressure:

- long-time records of the groundwater level - the representative value of the groundwater pressure is based on:
  - one single permanent value of the hydrostatic level – upper or lower value (the most unfavourable one)
  - a combination between a permanent value (average) and a variable component

If there are not sufficient data – **nominal value based on cautious estimate**

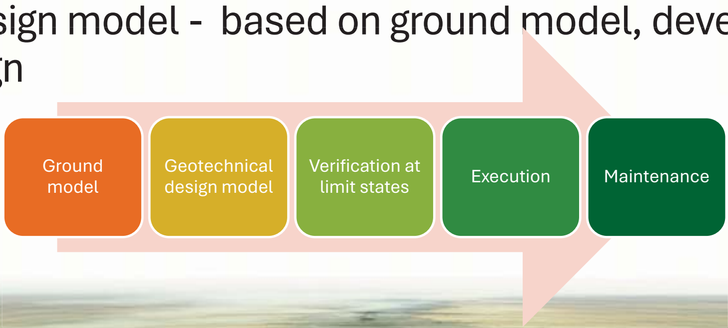


2. Key changes in the 2nd generation of EC7

Key changes in **EN 1997-1**

Ground model and Geotechnical design model:

- Ground model – output of the geotechnical investigation (part 2)
- Geotechnical design model - based on ground model, developed during the geotechnical design



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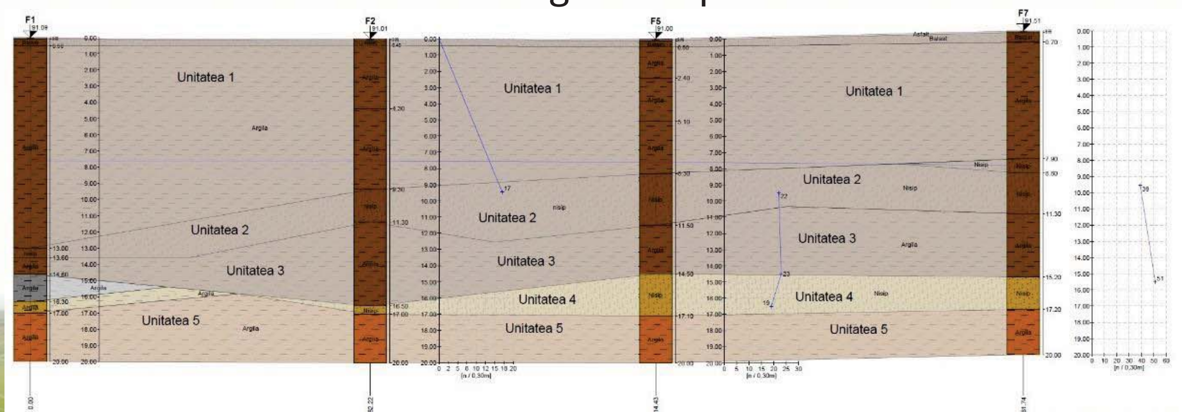
2. Key changes in the 2nd generation of EC7

Key changes in **EN 1997-1**

Ground model and Geotechnical design model

- Ground model – Geotechnical investigation report



2. Key changes in the 2nd generation of EC7

Key changes in **EN 1997-1**

Ground model and Geotechnical design model

- Geotechnical design model – Geotechnical design

Ground model + Geotechnical structure + Relevant limit states + Relevant design situation + Repres. values of geotech. param. + Design values of geotech. param. = GEOTECHNICAL DESIGN MODEL

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2. Key changes in the 2nd generation of EC7

Key changes in **EN 1997-1**

Ground model and Geotechnical design model – must be validated

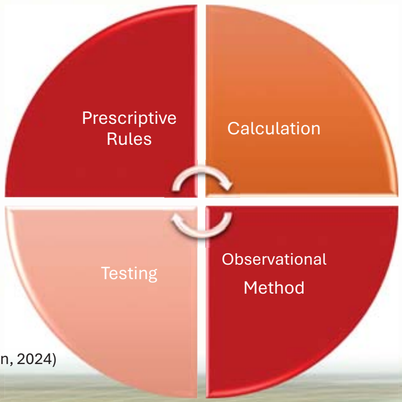
- to demonstrate that using the GDM the required reliability level was reached
- if not, additional data / investigation are needed
- validation offers info regarding the remaining uncertainties in GDM that have to be considered

Table 4.5(NDP) — Measures to validate the Geotechnical Design Model

Geotechnical Category	Measures
GC3	All items given for GC2 and, in addition: — sensitivity analyses of key ground properties for the design to identify need of additional information to cover all anticipated design situations; — sensitivity analyses of key geometrical properties for the design to identify need of additional measures; — check that the information available is sufficient to determine the variability of the ground properties and groundwater conditions.
GC2	All items given for GC1 and, in addition: — comparison of derived values from different sources within each geotechnical unit to determine representative values of ground properties with appropriate level of confidence; — check that GDM includes all ground properties and groundwater conditions affecting the design situation; — check that GDM is appropriate and compatible with the considered ultimate limit states (failure modes) and serviceability limit states; — check that the ground properties are determined for a time frame compatible with the considered limit states and design situation.
GC1	All items given below: — check the consistency of assumed geotechnical units and geotechnical properties with available information from the desk study and comparable experience; — confirmation of the Geotechnical Design Model with information from site inspection.

2. Key changes in the 2nd generation of EC7

Key changes in **EN 1997-1** Verification limit states



(Franzen, 2024)

Verification by prescriptive rules

- pre-determined, experienced-based, and suitably conservative rules for design

Verification by observational method

- continuous, managed, integrated process of design, construction control, monitoring and review that enables previously defined modifications to be incorporated during or after construction as appropriate

Verification by calculation

- using partial factors
- **$E_d < R_d$**
- **MFA – Material Factor Approach – partial factors applied to materials**
- **RFA – Resistance Factor Approach – partial factors applied to resistances**

Verification by testing

- testing performed to verify that the performance of the geotechnical structure (or part of the structure) is within the limiting values

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2. Key changes in the 2nd generatic

Key changes in **EN 1997-1**

Partial factors on ground properties (γ_M)

Table 4.9 (NDP) — Consequence factors k_M

Consequence class (CC)	Description of consequences	Consequence factor k_M
CC3	Higher	1,1
CC2	Normal	1,0
CC1	Lower	0,9

$$X_d = X_{rep}/(\gamma_M \times k_M)$$

Table 4.8 (NDP) — Partial factors on ground properties for persistent and transient design situations

Ground property	Symbol	M1[a]	M2[a]
Soil and fill			
Shear strength in effective stress analysis[b] (τ_f)	$\gamma_{\tau f}$	1,0	1,25 k_M
Coefficient of peak friction ($\tan \varphi'_p$)[d]	$\gamma_{\tan\varphi,p}$	1,0	1,25 k_M
Peak effective cohesion (c'_p)	$\gamma_{c,p}$	1,0	1,25 k_M
Coefficient of friction at critical state ($\tan \varphi'_{cs}$)[d]	$\gamma_{\tan\varphi,cs}$	1,0	1,1 k_M
Coefficient of residual friction ($\tan \varphi'_r$)[d]	$\gamma_{\tan\varphi,r}$	1,0	1,1 k_M
Residual effective cohesion (c'_r)	$\gamma_{c,r}$	1,0	1,1 k_M
Shear strength in total stress analysis[b] (c_u)	γ_{cu}	1,0	1,4 k_M
Unconfined compressive strength (q_u)	γ_{qu}	Same as γ_{cu}	
Rock material and rock mass[f]			
Shear strength (τ_r)	$\gamma_{\tau r}$	1,0	1,25 k_M
Unconfined compressive strength[c] (q_u)	γ_{qu}	1,0	1,4 k_M
Rock discontinuities			
Shear strength[e]	$\gamma_{\tau dis}$	1,0	1,25 k_M
Coefficient of residual friction[f] ($\tan \varphi'_{dis}$)	$\gamma_{\tan\varphi dis,r}$	1,0	1,1 k_M
Interface			
Coefficient of ground/structure interface friction ($\tan \delta$)	$\gamma_{\tan\delta}$	1,0	1,25 k_M

[a] M1 and M2 are alternative sets of material factors. prEN 1997-3:2022 specifies which set to use for specific geotechnical structures.
[b] Intended to be used for numerical models.
[c] Used for foundation purposes only.
[d] Partial factor is applied to $\tan\varphi$.
[e] Used when roughness component is neglected.
[f] Values of partial factors shown for soil and fill can be used for weak, highly fractures rock masses, in cases when soil mechanics concepts are found to apply.

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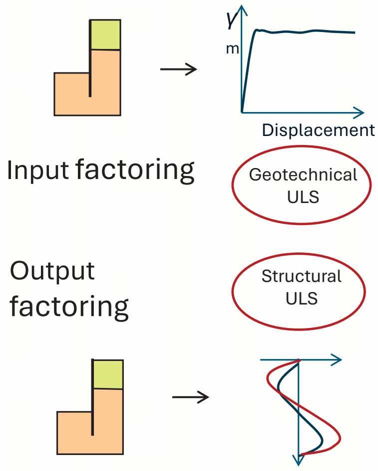
2. Key changes in the 2nd generation of EC7

Key changes in **EN 1997-1**

Verification ULS by using numerical methods:

- A **Input factoring**
  Partial coefficients applied to actions, γ_F for VC3
  Partial coefficients applied to materials, γ_M (M2)
  AND Output factoring:
  Partial coefficients applied to effects – of – actions, γ_E for VC4
  Partial coefficients applied to materials, γ_M (M1 = 1,0)
  No partial coefficients applied to resistances

- B **ONLY Output factoring**:
  Partial coefficients applied to effects – of – actions, γ_E for VC4
  Partial coefficients applied to resistances, γ_R (EN 1997-3 for each geotechnical structure)
  Partial coefficients applied to materials, γ_M (M1 = 1,0)



(van Seters, 2023)

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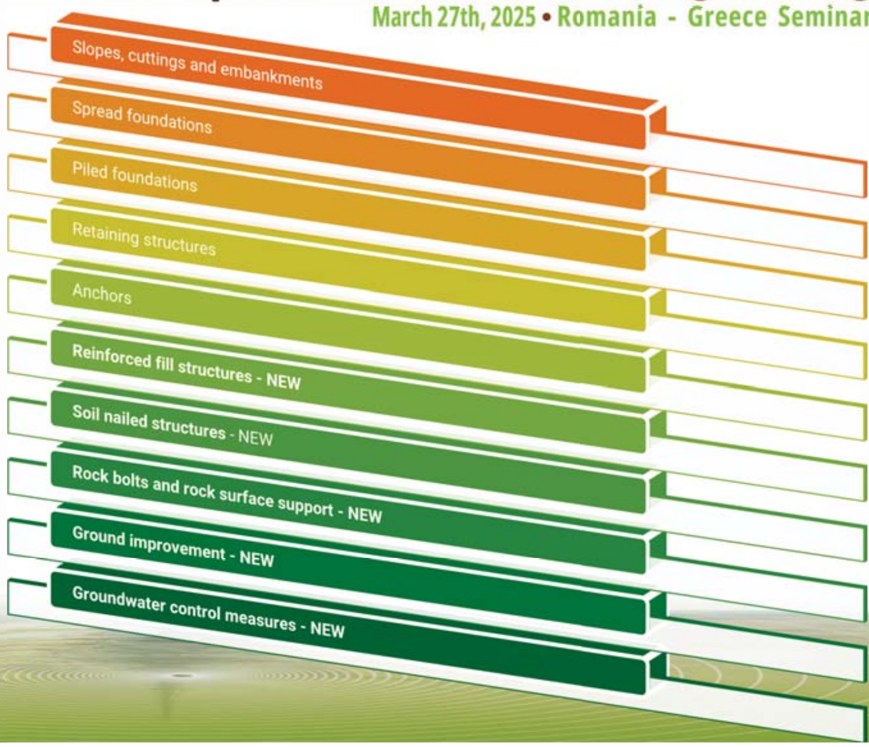
2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures



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2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures

For each geotechnical structure

- Conditions for ground investigation
- Groundwater
- Geotechnical analysis
- ULS and SLS verifications
- Partial coefficients applied to resistances γ_R
- Model factors γ_{Rd}
- Implementation of design
- Testing
- Reporting

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2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures – Slopes

- Calculation models for soils/fills and rocks

Table A.1 — Calculation models for analysing the stability of soil and fill

	Calculation model[c]	Type of method and assumed failure[a,b]	Special design conditions/limitations	Comments and assumptions
1	Bishop (simplified and rigorous)	Slices, circular arc	Not recommended with external horizontal loads	Simplified ignores interslice shear forces when interslice forces are horizontal
2	Generalized limit equilibrium	Slices, any shape of surface	Applicable with all slope geometries and soil profiles	---
3	Janbu generalized (modified)	Slices, circular arc, non-circular, polyline		Location of interslice normal force is assumed by a line of thrust
4	Morgenstern-Price			Direction of interslice forces by variable user function
5	Spencer			Constant interslice forces function
6	Sarma	Slices, polyline	Seismic loading, critical acceleration. Static conditions: horizontal load set to zero	Can include non-vertical slices and multi-wedge failure mechanisms
7	Kinematical approach of limit analysis	Multiple body, blocks, circular, planar or logarithmic spiral	---	Based on the compatibility of velocity fields
8	Block/wedge method	Multiple body, polyline	Pre-defined planar failure surface. Divided into three segments	Earth-pressure can be used as driving and resisting force. Rotational failure (assessed by moment equilibrium) not considered
9	Multiple wedge method	Multiple body, blocks, wedges, plane surfaces	---	Rotational failure (assessed by moment equilibrium) not considered
10	Infinite slope	Single body, plane surface	Long shallow slopes	
11	Culmann, finite slope		Steep slopes, drained analysis	
12	Logarithmic spiral	Single body; logarithmic spiral	Homogeneous soil, drained analysis	Rotational failure (assessed by moment equilibrium) only considered

[a] Where ground or embankment material is relatively homogeneous and isotropic, circular failure surfaces can normally be assumed, except when high external loads are present.
[b] Polyline includes interconnected plane surfaces.

2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures – Slopes
- Calculation models for soils/fills and rocks

Table A.2 — Calculation models and considerations for analysing the stability of rock masses

No.	Type of method and assumed failure	Special design conditions/limitations	Comments and assumptions
1	Circular failure Large slope deformations[g]	Blocky or weathered rock mass[b] Tension crack with or without water	Method of slices, circular (see Table A4.1)
2	Plane failure	Tension crack with or without water	Plane surface, blocks
3	Wedge failure	Tension crack with or without water	Wedge
4	Block toppling	---	Blocks
5	Flexure toppling	---	Columns
6	Block-flexure toppling	---	Blocks and columns
7	Secondary toppling	---	---
8	Rock fall[c]	Block trajectories, bounce heights, velocities, energies, run out distances	Blocks

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2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures – Partial coefficients for resistances

Slopes
- only MFA

Table 4.2 (NDP) — Partial factors for the verification of ground resistance of slopes, cuttings, and embankments for fundamental (persistent and transient) design situations

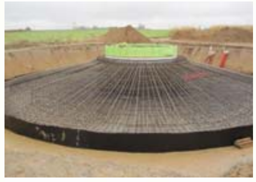
Verification of	Partial factor on	Symbol	Material Factor Approach
Overall stability	Actions	γ_F	VC3[a]
	Ground properties[c]	γ_M	M2[b]
Bearing resistance	see Clause 5		

[a] Values of the partial factors for Verification Case 3 (VC3) are given in EN 1990:2023, Annex A.
[b] Values of the partial factors for Set M2 are given in FprEN 1997-1:2024, 4.4.1.3.
[c] Also includes ground properties of Class AI ground improvement (see Clause 12).

2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures





Spread foundations

-Application to:

Pad, strip and raft foundations

Unreinforced working platforms

Load transfer platforms (LTP) over rigid inclusions (partly)

Deep foundations as caissons (behaving as spread foundations)

Gravity retaining walls (partly)



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2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures – Spread foundations

- general bearing capacity formula in the main text (5.5), with values for the N_c, N_q and N_γ-factors in B.4.
- new depth factor for the bearing capacity (d)
- Specific bearing calculation models are given in B.5 (strong over weak layer), B.6 (Menard pressuremeter tests) and B.16 (rock).

2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures – Partial coefficients for resistances

Spread foundations

ULS-analysis, both MFA and RFA are allowed

Table 5.2 (NDP) — Partial factors for the verification of ground resistance of spread foundatio for fundamental (persistent and transient) design situations

Verification of	Partial factor on	Symbol	Material Factor Approach, either both combinations (a) and (b) or the single combination (c)			Resistance Factor Approach, either combination (d) or (e)[c]	
			(a)	(b)	(c)[d]	(d)	(e)
Overall stability	See Clause 4						
Bearing and sliding resistance	Actions, Effects of actions	γ_F, γ_E	VC1[a]	VC3[a]	VC1[a]	VC1[a]	VC4
	Ground properties	γ_M	M1[b]	M2[b]	M2[b]	Not factored	
	Bearing resistance	γ_{RN}	Not factored			1,4	
	Sliding resistance	γ_{RT}	Not factored			1,1	
	Passive resistance	$\gamma_{RT,face}$	Not factored			1,4	

[a] Values of the partial factors for Verification Cases (VCs) 1, 3, and 4 are given in EN 1990:2023, Annex A[b] Values of the partial factors for Sets M1 and M2 are given in FprEN 1997-1:2024, 4.4.1.3

[c] Use combination (d) except where specified otherwise in 5.6.6(2)

[d] In this combination, the consequence factor on material properties is omitted

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2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures

Spread foundations

- SLS-analysis – Guidance on bearing pressures for structural analysis
  - Rigid and flexible foundations
  - Relative stiffness of a spread foundation and subgrade modulus
  - Linear elastic spring stiffness

- limits to the load eccentricity

(1) To limit a potential physical gap forming beneath the foundation, the eccentricity of load at the serviceability limit state shall not exceed specified limits.

NOTE The specified limits are given in Table 5.3 (NDP) unless the National Annex gives different limits.

Table 5.3 (NDP) — Limits to load eccentricity at the serviceability limit state

Loading effects	Strip foundation	Circular foundation	Rectangular foundation
Permanent action effects only (No tension gap)	$\frac{e_B}{B} \leq \frac{1}{6}$	$\frac{e}{R} \leq \frac{1}{4}$	$\frac{e_L}{L} + \frac{e_B}{B} \leq \frac{1}{6}$
Permanent and variable action effects	$\frac{e_B}{B} \leq \frac{1}{3}$	$\frac{e}{R} \leq 0{,}59$	$\left[\frac{e_L}{L}\right]^2 + \left[\frac{e_B}{B}\right]^2 \leq \frac{1}{9}$

e is the eccentricity of the load
e_B is the eccentricity of the load in the direction of the smaller foundation width B
e_L is the eccentricity of the load in the direction of the greater foundation width L
R is the radius of a circular foundation

2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures – Piled foundations
- extensively revised
- classification (also for partial factors for ULS)
- single piles, pile groups and piled rafts

Table 6.1 (NDP) —Classification of piles

Pile type	Description	Class
Displacement pile	Pile installed in the ground without excavation of material	Full displacement
		Partial displacement
Replacement pile	Pile installed in the ground after the excavation of material	Replacement
Pile not listed above	---	Unclassified

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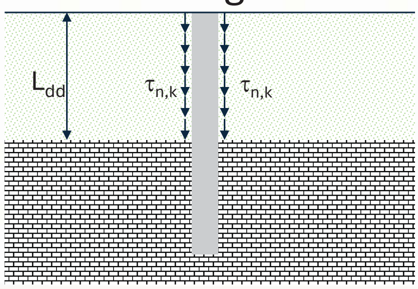
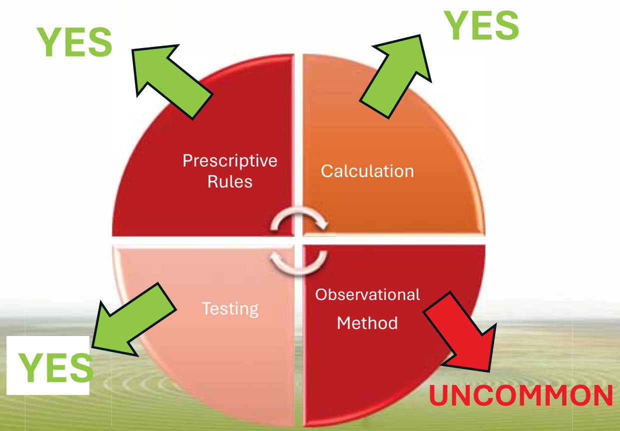
2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures – Piled foundations
- cyclic and dynamic actions
- effect of ground displacement – downdrag, heave, transverse loading



2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures – Piled foundations
- Resistance of a single pile (compression, traction, transversal)



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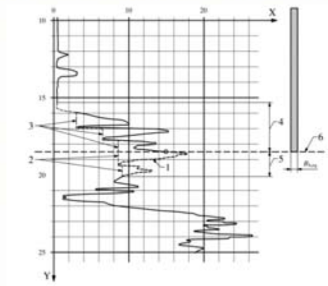
2. Key changes in the 2nd generation of EC7



EN1997-3 – Geotechnical structures – Piled foundations
- Axial resistance of single piles

Design by calculation: 2 methods

- **Ground Model Method** – ground properties determined by laboratory and in situ tests (details in C.4 and C.5)
- **Model Pile Method** - individual pile resistance profiles determined from correlations with field test results – (details in C.6 (CPT) and C.7 (pressuremeter))

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2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures – Piled foundations

- Axial resistance of single piles

Design by calculation: Model factor

Table 6.4 (NDP) —Model factor $\gamma_{Rd,pile}$ for verification of axial pile resistance by calculation

Verification by	Based on	Model factor $\gamma_{Rd,pile}$	
Ground Model Method	Ultimate pile load tests	1,15	
	Extensive comparable[a,b] experience without site-specific pile load tests	1,3	
	Serviceability pile load tests	1,35	
	No pile load tests and limited comparable experience[a,c]	1,55	
		Compressive resistance	**Tensile resistance**
Model Pile Method	Ménard pressuremeter test[d,f]	1,15	1,4
	Cone penetration test[d]	1,1	1,1
	Profiles of ground properties based on field or laboratory tests[d,e]	1,2	1,2

[a] Comparable experience assumes documented records (or database) of static pile load test results conducted on similar piles, in similar ground conditions, under similar loading conditions from a certain number of sites n

[b] Extensive comparable experience assumes $n \geq 10$

[c] Limited comparable experience assumes $0 < n < 10$

[d] Value can be multiplied by 0,9 when accompanied by ultimate pile load tests

[e] Ground strength properties determined at least every 1,5 m along the vertical profile

[f] The value of the model factor corresponds to the calculation method given in C.7

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2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures – Piled foundations
- Axial resistance of single piles

Design by testing:
- Static pile load tests – ULS and SLS for piles in compression and tension
- Dynamic load tests and rapid load test – ULS for piles in compression

2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures – Piled foundations

- Axial resistance of single piles

Design by testing: Model factor

Table 6.5 (NDP) — Model factor $\gamma_{Rd,pile}$ for verification of axial pile resistance by testing or assisted by testing

Verification by		Model factor $\gamma_{Rd,pile}$		
		Fine soils	Coarse soils	Rock[c]
Static load tests		1,0	1,0	1,0
Rapid load tests (multiple load cycles)[a]		1,4	1,1	1,2
Rapid load tests (single load cycle)[a]		1,4	1,1	1,2
Dynamic impact tests (signal matching)[b]	Shaft resistance	1,5	1,1	1,2
	End resistance	1,4	1,25	1,25
Dynamic impact tests (multiple blow)[b]	Shaft resistance	1,5	1,1	1,2
	End resistance	1,4	1,2	1,2
Dynamic impact tests (closed form solutions)[b]	Shaft resistance	Not permitted	Not permitted	Not permitted
	End resistance	Not permitted	1,3	1,3
Wave equation analysis		Not permitted	1,6	1,5
Pile driving formulae		Not permitted	1,8	1,7

[a] When dynamic impact tests or rapid load tests are not calibrated by site-specific static load testing, but by comparable experience only (see Table 6.4 (NDP)), the values for $\gamma_{Rd,pile}$ are increased by:
0,1 when calibration is based on extensive comparable experience; or
0,25 when calibration is based on limited comparable experience.

[b] When dynamic impact tests or rapid load tests are carried out on cast-in-place piles, the values for $\gamma_{Rd,pile}$ are increased by 0,2

[c] If the test results demonstrate an elastic behaviour without any significant permanent movement, the values for $\gamma_{Rd,pile}$ can be decreased by 0,1 as long as the model factor remains greater than or equal to 1,0

2. Key changes in the 2nd generation of EC7

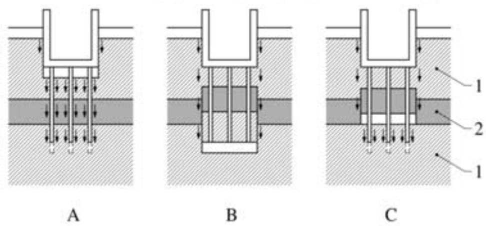
EN1997-3 – Geotechnical structures – Piled foundations
- Resistance of pile groups – numerical, analytical, empirical, observed performance

Design of pile groups should consider the effect of:
- Pile – soil interaction
- Pile – pile interaction

Group of piles in compression and tension

C.10 Pile groups subject to axial tension

NOTE Possible mechanisms for groups of tension piles in layered ground are illustrated in Figure C.6.



Key
A pull-out from ground
B lift-off a block of ground
C combined pull-out and lift-off
1 sand
2 clay

Figure C.6 — Possible mechanisms for groups of tension piles in layered soils

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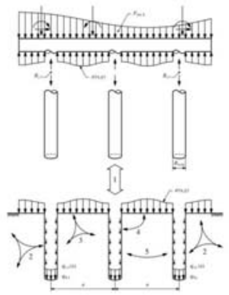
2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures – Piled foundations
- Piled rafts

To consider effects of:
- pile – soil interaction
- pile – pile interaction
- raft – soil interaction
- pile – raft interaction



Key
1 interaction between piled raft and ground
2 piled-ground-interaction
3 raft-ground-interaction
4 piled-raft-interaction
5 pile-pile-interaction
e distance between piles
$B_{s,eq}$ pile diameter

Figure 6.1 — Interaction effects of a piled raft

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2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures –

Piled foundations - Partial coefficients for resistances
- axial resistance – only RFA allowed
- transverse resistance – both MFA and RFA

Calculation - Ground Model Method

Table 6.9 (NDP) — Partial factors for the verification of ultimate resistance of single piles for fundamental (persistent and transient) design situations – Ground Model Method

Verification of	Partial factor on	Symbol	Material Factor Approach – both combinations		Resistance Factor Approach		
			(a)	(b)	Pile class	Ground Model	
Axial compressive resistance	Actions, Effects of actions[a]	γ_F, γ_E	Not Used		All	VC1	
	Drag force	$\gamma_{F,drag}$				1,35	
	Ground properties[b]	γ_M				Not factored	
	Base and shaft resistance in compression	γ_{Rb}, γ_{Rs}				Base	Shaft
					Full displacement	1,2	1,05
					Partial displacement	1,3	1,1
					Replacement	1,4	1,15
					Unclassified	1,5	1,25
	Total resistance in compression	γ_{Rc}			Full displacement	1,1	
					Partial displacement	1,2	
					Replacement	1,3[d]	
					Unclassified	1,4	
Axial tensile resistance	Actions, Effects of actions[a]	γ_F, γ_E	Not Used		All	VC1	
	Ground properties[b]	γ_M				Not factored	
	Shaft resistance in tension	γ_{Rst}			Full displacement	1,2	
					Partial	1,25	
					Replacement	1,3	
					Unclassified	1,5	
Transverse resistance	Actions, Effects of actions[a]	γ_F, γ_E	VC4 or VC1	VC3	All	VC4 or VC1	
	Transverse ground load	$\gamma_{F,tr}$			All	1,35	
	Ground properties[b]	γ_M	M1	M2	All	Not factored	
	Transverse resistance	γ_{Rtr}	Not factored		All	1,4	

[a] Values of the partial factors for Verification Cases (VCs) 1, 3, and 4 are given in EN 1990:2023, Annex A

[b] Values of the partial factors for Sets M1 and M2 are given in FprEN 1997-1:2024, 4.4.1.3

2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures –

Piled foundations
Partial coefficients for resistances

Calculation Model Pile Method

Table 6.10 (NDP) — Partial factors for the verification of ultimate resistance of single piles for fundamental (persistent and transient) design situations – Model Pile Method

Verification of	Partial factor on	Symbol	Material Factor Approach – both combinations (a)	(b)	Resistance Factor Approach: Pile class	Model Pile Method	
Axial compressive resistance	Actions, Effects of actions[a]	γ_F, γ_E	Not Used		All	VC1	
	Drag force	$\gamma_{F,drag}$				1,35	
	Ground properties[b]	γ_M				Not factored	
	Base and shaft resistance in compression	γ_{Rb} | γ_{Rs}				Base	Shaft
					Full displacement	1,2	1,0
					Partial displacement	1,2	1,0
					Replacement	1,2	1,0
					Unclassified	1,35	1,25
	Total resistance in compression	γ_{Rc}			Full displacement	1,1	
					Partial displacement		
					Replacement		
					Unclassified	1,3	
Axial tensile resistance	Actions, Effects of actions[a]	γ_F, γ_E	Not Used		All	VC1	
	Ground properties[b]	γ_M				Not factored	
	Shaft resistance in tension	$\gamma_{R,st}$			Full displacement	1,15	
					Partial displacement		
					Replacement		
					Unclassified	1,4	

[a] Values of the partial factors for Verification Cases (VCs) 1, 3, and 4 are given in EN 1990:2023 Annex A. For transverse resistance, VC1 can be used as alternative to VC4

[b] Values of the partial factors for Sets M1 and M2 are given in FprEN 1997-1:2024, 4.4.1.3

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2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures –

Piled foundations
Partial coefficients for resistances

Design by testing

Table 6.11 (NDP) — Partial factors for the verification of ultimate resistance of single piles for fundamental (persistent and transient) design situations – Design by testing

Verification of	Partial factor on	Symbol	Material Factor Approach – both combinations (a)	(b)	Resistance Factor Approach: Pile class	
Axial compressive resistance	Actions, Effects of actions[a]	γ_F, γ_E	Not Used		All	VC1
	Drag force	$\gamma_{F,drag}$				1,35
	Ground properties[b]	γ_M				Not factored
	Total resistance in compression	γ_{Rc}			Full displacement	1,1
					Partial displacement	1,1
					Replacement	1,1
					Unclassified	1,1
Axial tensile resistance	Actions, Effects of actions[a]	γ_F, γ_E	Not Used		All	VC1
	Ground properties[b]	γ_M				Not factored
	Shaft resistance in tension	$\gamma_{R,st}$			Full displacement	1,25
					Partial displacemen	1,25
					Replacement	1,25
					Unclassified	1,25

[a] Values of the partial factors for Verification Cases (VCs) 1, 3, and 4 are given in EN 1990:2023, Annex A

[b] Values of the partial factors for Sets M1 and M2 are given in FprEN 1997-1:2024, 4.4.1.3

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2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures –

Piled foundations
Partial coefficients for resistances

Pile groups and piled rafts
- vertical resistance – both MFA and RFA
- combined axial and transverse resistance – only MFA

Table 6.12 (NDP)— Partial factors for the verification of ultimate resistance of pile groups and piled rafts for fundamental (persistent and transient) design situations

Verification of	Partial factor on	Symbol	Material Factor Approach – both combinations (a)	Material Factor Approach – both combinations (b)	Resistance Factor Approach
Vertical resistance	Actions, Effects of actions[a]	γ_F, γ_E	VC4	VC3	VC1
	Ground properties[b]	γ_M	M1	M2	Not factored
	Vertical resistance	$\gamma_{R,group}$			1,4
		$\gamma_{R,piled\text{-}raft}$			1,4
Combined axial and transverse resistance (see FprEN 1997-1:2024, 8.2)	Actions, Effects of actions[a]	γ_F, γ_E	VC4 or VC1	VC3	Not used
	Ground properties[b]	γ_M	M1	M2	
	Compressive and transverse resistance	$\gamma_{R,group}$	Not factored		

[a] Values of the partial factors for Verification Cases (VCs) 3 and 4 are given in EN 1990:2023, Annex A
[b] Values of the partial factors for Sets M1 and M2 are given in FprEN 1997-1:2024, Table 4.8

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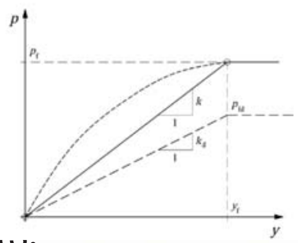

EN1997-3 – Geotechnical structures – Piled foundations
SLS
Displacement of piled foundations
- Single piles – settlement and transverse displacement – load tests, analytical, numerical, empirical, prescriptive rules
- Pile groups and piled rafts – empirical, analytical, numerical
- Calculation models for single lateral and axial displacement using load transfer functions (C.11 and C.12)
- Calculation model on buckling and second order effects (C.13).
- Annex K, thermoactive geostructure design

2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures – Retaining structures
- Formulae for the determination of earth pressures (active, passive, at-rest) are given
- Calculation models for Basal Heave, Vertical wall stability and Anchor length
- Rules for application of Limit equilibrium models, Beam-on-spring models, Numerical continuum models
- Annex K, thermoactive geostructure design

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2. Key changes in the 2nd gener

EN1997-3 – Geotechnical structures -

Retaining structures
Partial coefficients for resistances

- both MFA and RFA allowed

Table 7.2 (NDP) —Partial factors for the verification of ground resistance against retaining structures for fundamental (persistent and transient) design situations

Verification of	Partial factor on	Symbol	Material Factor Approach - both combinations (a) and (b) or the single combination (c)			Resistance Factor Approach	
			(a)	(b)	(c)	(d)	(e)
Overall stability	See Clause 4						
Bearing/sliding resistance of gravity walls	Actions, Effects of actions	γ_F, γ_E	VC4[a]	VC3[a]	VC1[a]	VC1[a,c]	VC4[a,c]
	Ground properties	γ_M	M1[b]	M2[b]	M2[b]	Not factored	
	Bearing resistance	γ_{RN}	Not factored			1,4	
	Sliding resistance	γ_{RT}				1,1	
Bearing/rotational resistance of embedded walls	Actions, Effects of actions	γ_F, γ_E	VC4[a]	VC3[a]	Not used	VC1[a]	VC4[a]
Basal heave[d]	Ground properties	γ_M	M1[b]	M2[b]	Not used	Not factored	
	Vertical resistance, basal heave	γ_R	Not factored			1,4	
	Passive earth resistance	γ_{Re}				1,4	

[a] Values of the partial factors for Verification Cases (VCs) 3 and 4 are given in EN 1990:2023, Annex A
[b] Values of the partial factors for Sets M1 and M2 are given in FprEN 1997-1:2024, 4.4.1.3
[c] For basal heave, see Annex D

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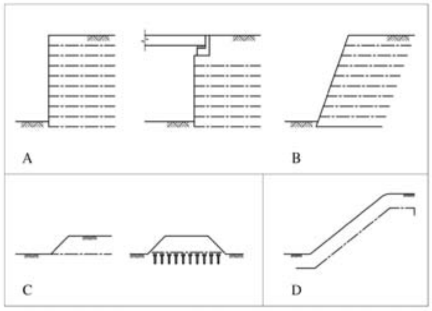
2. Key changes in the 2nd generation of EC7

NEW

EN1997-3 – Geotechnical structures – Reinforced fill structures
- ULS-design
- tensile strength of the reinforcing element
- pull-out resistance
- sliding resistance along the interface between the reinforcement and the ground (direct shear)
- resistance of the connections between facing and reinforcing element



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Earth

2. Key changes in the 2nd generati

EN1997-3 – Geotechnical structures –

NEW

Reinforced fill structures
- partial coefficients for resistances
- both MFA and RFA allowed

Table 9.4 (NDP) — Partial factors for the verification of resistance of reinforced fill structures for fundamental (persistent and transient) design situations

Verification of	Partial factor on		Symbol	Material Factor Approach	Resistance Factor Approach
Overall and compound failure mechanisms	See Clause 4				
Bearing resistance and sliding	See Clause 5				
Overturning	See Clause 7				
Internal failure mechanisms	Actions, Effect of actions		γ_F, γ_E	VC3[a]	VC1[a]
	Ground properties		γ_M	M2[b]	Not factored
Pull-out and direct shear	Pull-out resistance of reinforcing elements		$\gamma_{R,po}$	Not factored	1,25
	Resistance to direct shear along interface		$\gamma_{R,ds}$	Not factored	1,25
Rupture of reinforcing elements	Tensile resistance of	geosynthetic reinforcing elements	$\gamma_{M,gs}$	1,25	
		structural steel per EN 10025-2 or EN 10025-4	γ_{M0}	specified in EN 1993-1-1	
			γ_{M2}	specified in EN 1993-1-1	
		reinforcing steel per EN 10080	γ_S	specified in EN 1992-1-1	
			γ_t	1,25	
		polymeric coated steel wire mesh reinforcing elements	$\gamma_{M,pwm}$	1,25	
Rupture of connections to facing	Tensile resistance at connection	Reinforcing elements	$\gamma_{R,con,el}$	As specified above for rupture of reinforcing elements	
		Connectors	$\gamma_{R,con,c}$	As specified in the relevant material Eurocode	
		Facing elements	$\gamma_{R,con,f}$	As specified in the relevant material Eurocode	

[a] Values of the partial factors for Verification Cases (VCs) 3 and 4 are given in EN 1990:2023, Annex A.

[b] Values of the partial factors for sets M1 and M2 are given in FprEN 1997-1:2024, 4.4.1.3

Earth

2. Key changes in the 2nd generatio

EN1997-3 – Geotechnical structures –

NEW

Soil nailed structures
- partial coefficients for resistances
- both MFA and RFA allowed

Table 10.3 (NDP) — Partial factors for the verification of resistance of soil nailed structures for fundamental (persistent and transient) design situations

Verification of	Partial factor on	Symbol	Material Factor Approach	Resistance Factor Approach
Overall and compound failure mechanisms	See Clause 4			
Bearing resistance and sliding	See Clause 5			
Overturning	See Clause 7			
Internal failure mechanisms and facings	Actions, Effects of Actions	γ_F, γ_E	VC3[a]	VC1[a]
	Ground and fill properties	γ_M	M2[b]	Not factored
Geotechnical resistance, mobilised at the interface between soil nail and ground	Pull-out resistance	$\gamma_{R,po}$	Not factored	1,25
Structural resistance soil nail and any connections	Structural steel per EN 10025, EN 10210, EN 10219	γ_{M0}, γ_{M2}	As specified in EN 1993-1-1	
	Reinforcing steel per EN 10080, pre-stressing steel per EN 10138	γ_s	As specified in EN 1992-1-1	
Wire mesh	Tensile resistance of steel wires or ropes		As specified in EN 1993-1-1	
	Tensile and puncture resistance of wire mesh	$\gamma_{M,wm}$	1,25	
	Connection of adjacent wire mesh panels	$\gamma_{R,con}$	1,25	
	Connection to soil nails[c]		As specified in EN 1993-1-1	
Sprayed concrete	Structural resistance of sprayed and any connections		As specified in EN 1992-1-1	
Other facing elements	Structural resistance of other facing elements and any connections		As specified in relevant standard	

[a] Values of the partial factors for Verification Cases (VCs) 1 and 3 are given in EN 1990:2023, Annex A
[b] Values of the partial factors for Set M2 are given in FprEN 1997-1:2024, 4.4.1.3
[c] See EN 1993-1-1 for the verification of shearing and punching resistance at the interface of the head plate and the wire mesh

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2. Key changes in the 2nd generation of EC7

NEW

EN1997-3 – Geotechnical structures – Ground improvement

- AI and AII – use the improved ground parameters
- BI and BII - interaction between the inclusions and the surrounding ground and a possible supporting structure (e.g. differences in stiffness, possibility of buckling of the inclusion) must be taken into account.
- AII and BII – the representative unconfined compressive strength is evaluated on the basis of samples retrieved from the field

Table 12.1 — Classification of ground improvement

Class	A - Diffused	B - Discrete
I	**AI - Diffused with no measurable unconfined compressive strength** The improved ground has an increased shear strength or stiffness higher than that of the original ground. The improved ground can be modelled as a ground with improved properties.	**BI - Discrete with non-rigid inclusions** Inclusions, installed in the ground, with higher shear capacity and stiffness compared to the surrounding ground. The unconfined compressive strength of the inclusion is not measurable.
II	**AII - Ground improvement zone with measurable unconfined compressive strength** The improved ground is modified from its original natural state, has a measurable unconfined compressive strength and is significantly stiffer than the surrounding ground. Usually, it comprises a composite of a binder and ground.	**BII - Discrete with rigid inclusions** Rigid inclusions, installed in the ground, with unconfined compressive strength and significantly higher stiffness than the surrounding ground. The inclusions can be an engineered material such as timber, concrete/grout or steel or a composite of a binder and ground.

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2. Key changes in the 2nd generation of EC7

EN1997-3 – Geotechnical structures –

NEW

Ground improvement

Table 12.3 (NDP) — Methods used to verify ultimate limit states of ground improvement

Class	A - Diffused	B - Discrete
I	1. Determine improved ground properties according to 12.3.2 and FprEN 1997-1:2024, 4.3.2 2. Verify ULS according to 12.2.5.1, 12.5.2 and the appropriate clauses in this document	1. Determine properties of non-rigid (Class BI) inclusions according to 12.3.2 and FprEN 1997-1:2024, 4.3.2 2. Verify ULS of the system using separate ground and inclusion properties; 3. Verify ULS according to 12.2.5.1, 12.5.3 and the appropriate clauses in this document 4. Verify compression and shear resistance in inclusions and the ground 5. For geotextile encased inclusions, determine the strength of the reinforcing element according to 9.6
II	1. Determine properties of Class AII improved ground according to12.3.2 2. Verify ULS according to12.2.5.1, 12.5.2 and 12.5.4 and the appropriate clauses in this document. 3. Verify structural resistance	1. Determine properties of Class BII inclusions according to 12.3.2.2. 2. Verify ULS according to 12.2.5.1, 12.5.3 and 12.5.4 and the appropriate clauses in this document. 3. Verify structural resistance of the rigid inclusions

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2. Key changes in the 2nd generation of EC7

NEW

EN1997-3 – Geotechnical structures –

Ground improvement - partial coefficients for resistances

- both MFA and RFA allowed
- BI and BII – MFA or RFA
- BII for the improved ground - a separate partial factor γ_M is applied
- axial design resistance of rigid inclusions (Class BII) is similar to pile groups

Table 12.4 (NDP) — Partial factors for the verification of ultimate resistance of ground improvement for fundamental (persistent and transient) design situations

Verification of	Partial factor on	Symbol	Material Factor Approach, both combinations (a) and (b): (a)	(b)	Resistance Factor Approach
Overall stability	See Clause 4				
Compressive and transverse resistance of diffused (Class A) ground improvement	Actions, Effects of actions[a]	γ_F, γ_E	Refer to other clauses as appropriate		
	Ground properties[b]	γ_M			
	UCS of Class II materials	γ_M	1,5		
	UCS of concrete, steel, and timber	γ_M	EN 1992-1-1, EN 1993-1-1 and EN 1995-1-1		
Compressive and transverse resistance of discrete (Class B) ground improvement	Actions, Effects of actions[a]	γ_F, γ_E	VC1 or VC4	VC3	VC1 or VC4[c]
	Ground properties[b]	γ_M	M1	M2	Not factored
	UCS of Class II materials	γ_M	1,5		
	UCS of concrete, steel, and timber	γ_M	EN 1992-1-1, EN 1993-1-1 and EN 1995-1-1		
	Overall system resistance (Class BI)	$\gamma_{R,sys}$	Not factored		1,4
	Resistance of rigid inclusion system (Class BII)	$\gamma_{R,ri}$	Not factored		1,4
	Resistance of treated ground (Class BII)	$\gamma_{R,g}$	Not factored		1,4
	Transverse resistance	γ_{Rt}	Not factored		Refer to other clauses as appropriate

[a] Values of the partial factors for Verification Cases (VCs) 1, 3, and 4 are given in EN 1990:2023, Annex A

[b] Values of the partial factors for Sets M1 and M2 are given in FprEN 1997-1:2024, 4.4.1.3

[c] Always use VC1 except for the computation of the effects on actions due to an embankment

3. Guidance documents

- JRC documents – already published – free access

https://eurocodes.jrc.ec.europa.eu/learning-corner/publications

2025

Determination of representative values from derived values for verification with limit states with EN 1997

This guideline is an essential tool for designers to understand how to determine representative values for the design and verification of geotechnical structures, in accordance with the procedures outlined in the second generation of Eurocode 7.

2024

Assembling the ground model and the derived values

This guideline addresses the process of assembling the Ground Model, highlighting the importance of progressively upgrading it with an increase in knowledge of the ground within the zone of influence of the specific structure.

2024

Implementation of Design during Execution & Service Life

This guideline provides advice on the content to consider into plans for design implementation, with examples for different geotechnical structures; it indicates methods to align the design code requirements with the practical considerations of the execution phase of the project.

2024

Reliability-based verification of limit states for geotechnical structures

The present document serves as a guideline for reliability-based verification of limit states in design and assessment of geotechnical structures within the safety and reliability concepts of EN 1990-1 and EN 1997-1.

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3. Guidance documents

- JRC documents – under processing

TaskGroup B1 - Evaluation of Eurocode EN 1997
Testing by next-generation engineers using case studies

TC 250 SC7

During drafting of EN 1997, testing of code:

- **Apply the code on real projects**
- 1st test - PT-draft (2021)
- 2nd test - Formal Vote draft (2023)
- Set up a Geotechnical Design Report (GDR)
- Comment on improvement, ease-of-use
- Differences between 1st and 2nd Generation
- Change requests for improvement

Figure 5. Illustration of the common view on design situation, actions, Limit state and GDM

	Design situation		Actions	
	Ease-of-use 1 to 5	Applicability 1 to 5	Ease-of-use 1 to 5	Applicability 1 to 5
B1.1 Slope, cuttings and embankments	4	4	4	4
B1.2 Spread foundations	3	4	3	3
B1.3 Piled foundations	3	4	4	4
B1.4 Retaining structures including anchors	3,25	3	2,5	3
B1.5 Reinforced fill	3	4	4	3
B1.6 SN, RB, RSS				
B1.7 Ground improvement				
B1.8 Groundwater control	2	2	4	4

	Limit state		Geotechnical Design Model	
	Ease-of-use 1 to 5	Applicability 1 to 5	Ease-of-use 1 to 5	Applicability 1 to 5
B1.1 Slope, cuttings and embankments	3	3	4	4
B1.2 Spread foundations	3	4	3	3
B1.3 Piled foundations	4	4	3,5	3,5
B1.4 Retaining structures including anchors	3,5	3,5	4	3,75
B1.5 Reinforced fill	3	4	3	3
B1.6 SN, RB, RSS				
B1.7 Ground improvement				
B1.8 Groundwater control	[illegible]	[illegible]	4	4

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3. Guidance documents

- JRC documents – under processing

Taskgroup B2 – Design examples

TC 250 SC7

- Over 25 design examples for main geotechnical structures
- Guidance on application of EN 1997
- Comparison between 2nd and 1st Generation EN 1997
- Many examples from check of 1st Generation (2011-2015)
- Nice Flow charts for design with EN1997!

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4. Further actions

2nd Generation Eurocode	Title	
EN1997-1	General rules	Published by CEN - Oct 2024
EN1997-2	Ground properties	Published by CEN - Oct 2024
EN1997-3	Geotechnical structures	Formal Vote Oct – Nov 2024
EN1990-1	Basis of design – new structures	Formal Vote Oct – Nov 2025
EN1990-2	Basis of design – existing structures	Formal Vote Oct – Nov 2025

National Annexes

Date of Publication (DoP): Latest date **Eurocode implemented nationally** (incl National Annex): September 2027
Date of Withdrawal (DoW): Latest date **1st Generation** National Standards must be withdrawn: March 2028

4. Further actions

- Translation EN 1997-1, EN 1997-2
- National Annexes for EN 1997-1, EN 1997-2
- Benchmarking exercise organised by TG B2 – results presented in Paris (October 2024) – only few countries participated, and the results showed that there is a lack of understanding and a need of future actions both at European and national level
- NSB contact group within SC7 – discussing different aspects for the National Annexes

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4. Further actions

- Joint actions with ISSMGE
- Session at 21st ICSMGE Vienna, 2026 - 12 papers, possible a Workshop

495	*Submission Type / Conference Track:* **Sweden: Swedish Geotechnical Society** **2nde Generation of Eurocode 7 – Verification of limit states - Use of partial factor and prescriptive rules** Burlon, Sebastian *Organization(s):* Terrasol, Setec
769	*Submission Type / Conference Track:* **Sweden: Swedish Geotechnical Society** **2nd Generation of Eurocode 7 – Overview** Franzén, Gunilla (1); Batali, Loretta (2); Burlon, Sebastian (3) *Organization(s):* 1: SC7 chair, GeoVerkstan, Sweden; 2: SC7 vice-chair, UTC, Bucharest, Romania; 3: SC7 vice-chair, Terrasol, Setec, France
770	*Submission Type / Conference Track:* **Sweden: Swedish Geotechnical Society** **2nd Generation of Eurocode 7 – Implementing rock** Lamas, Luis (1); Virely, Didier (2); Pereira, Renato (1); Estaire, José (3) *Organization(s):* 1: LNEC, Lisbon, Portugal; 2: Cerema, Toulouse, France; 3: CEDEX, Madrid, Spain
771	*Submission Type / Conference Track:* **Sweden: Swedish Geotechnical Society** **2nd Generation of Eurocode 7 – Verification of limit states Reliability-based methods and the Observational Method** Schweckendiek, Timo *Organization(s):* Deltares, Netherlands
774	*Submission Type / Conference Track:* **Sweden: Swedish Geotechnical Society** **2nd Generation of Eurocode 7 - Verification of Serviceability Limit States** Estaire, José (1); Bogusz, Witold (2); Bond, Andrew (3) *Organization(s):* 1: Laboratorio de Geotecnia – CEDEX, Madrid (Spain); 2: Jacobs, Warsaw (Poland); 3: Geocentrix, Banstead (United Kingdom)
775	*Submission Type / Conference Track:* **Sweden: Swedish Geotechnical Society** **2nd Generation of Eurocode 7 – Geotechnical Reliability and Representative value** Franzén, Gunilla (1); Batali, Loretta (2) *Organization(s):* 1: SC7 chair, GeoVerkstan, Sweden; 2: SC7 vice-chair, UTC, Bucharest, Romania
778	*Submission Type / Conference Track:* **Sweden: Swedish Geotechnical Society** **2nd Generation of Eurocode 7 – Slopes, Cuttings and embankments** Kamp, Stefan (1); Axelsson, Gary (2) *Organization(s):* 1: Fugro, Nederland; 2: ELU, Sweden
779	*Submission Type / Conference Track:* **Sweden: Swedish Geotechnical Society** **Title: 2nd Generation of Eurocode 7 – From ground investigation to design implementation in execution, with the Ground Model as one tool** Hard, David (1); Garin, Håkan (2) *Organization(s):* 1: Bachy Soletanche Ltd; 2: GeoVerkstan, Sweden

780	*Submission Type / Conference Track:* **Sweden: Swedish Geotechnical Society** **2nd generation of Eurocode 7 - Foundation** Moormann, Christian (1); Lesney, Kerstin (2) *Organization(s):* 1: University of Stuttgart, Germany; 2: University of Siegen, Germany
781	*Submission Type / Conference Track:* **Sweden: Swedish Geotechnical Society** **2nd Generation of Eurocode 7 - Retaining the ground - EN 1997-3, 7 Retaining structures and EN 1997-3, 8 Anchors** Bond, Andrew (1); Dietz, Klaus (2); Jeanmaire, Thierry (3) *Organization(s):* 1: Geocentrix Ltd, United Kingdom; 2: Dietz Geotechnik Consult GmbH, Germany; 3: Soletanche-Bachy, France
782	*Submission Type / Conference Track:* **Sweden: Swedish Geotechnical Society** **2nd Generation of Eurocode 7 - Reinforcing the ground or engineered fill** Maca, Natalia (1); Bräu, Gerhard (2) *Organization(s):* 1: Ischebeck Titan Polska sp. z o.o., Kraków, Poland; 2: Technische Universität München - Zentrum Geotechnik, München, Germany
783	*Submission Type / Conference Track:* **Sweden: Swedish Geotechnical Society** **2nd Generation of Eurocode 7 – Ground improvement** Denies, Nicolas (1); Bohn, Cecilia (2); Pandrea, Paul (2); Topolnicki, Michal (3); Plomteux, Cyril (4); Trybocka, Karolina (5) *Organization(s):* 1: Buildwise, Belgium; 2: Keller, Germany; 3: Keller, Poland; 4: Menard, France; 5: Menard, Polan

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4. Further actions

- JRC Online Workshop
"The Second Generation Eurocodes: key changes and benefits through design examples" – 3 – 5 June 2025

3 June 2025

Time	Topic
09.00 – 09.30	Welcome and introductions (CEN-CENELEC, DG GROW, JRC)
Session 1: Introduction, basis of structural design, actions	
09.30 – 10.00	Eurocodes Overview
10.00 – 11.15	EN 1990 "Eurocode: Basis of structural and geotechnical design"
11.15 – 11.30	Coffee
11.30 – 12.45	EN 1991 "Eurocode 1: Actions on structures"
12.45 – 13.45	Lunch
Session 2: Metal and Timber	
13.45 – 15.00	EN 1993 "Eurocode 3: Design of steel structures"
15.00 – 16.15	EN 1999 "Eurocode 9: Design of aluminium structures"
16.15 – 16.30	Coffee
16.30 – 17.45	EN 1995 "Eurocode 5: Design of timber structures"

4 June 2025

Time	Topic
Session 3: Concrete, steel and concrete, masonry	
09.00 – 10.15	EN 1992 "Eurocode 2: Design of concrete structures"
10.15 – 11.30	EN 1994 "Eurocode 4: Design of composite steel and concrete structures"
11.30 – 11.45	Coffee
11.45 – 13.00	EN 1996 "Eurocode 6: Design of masonry structures"
13.00 – 14.00	Lunch
Session 4: Geotechnical and seismic design	
14.00 – 15.15	EN 1997 "Eurocode 7: Geotechnical design"
15.15 – 15.30	Coffee
15.30 – 16.45	EN 1998 "Eurocode 8: Design of structures for earthquake resistance"

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4. Further actions

- JRC Online Workshop
"The Second Generation Eurocodes: key changes and benefits through design examples" – 3 – 5 June 2025

5 June 2025

Time	Topic
Session 5: New areas – Glass, FRP, Membrane structures	
09.00 – 10.15	EN 19100 "Eurocode 10: Structural Glass"
10.15 – 11.30	CEN/TS 19101 "Design of fibre-polymer composite structures"
11.30 – 11.45	Coffee
11.45 – 13.00	CEN/TS 19102 "Design of tensioned membrane structures"
13.00 – 13.10	Closure

Thank you!

Definition of seismic actions in the revised EC8 and implication in the seismic risk assessment

Kyriazis Pitilakis
Professor Emeritus AUTH

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- **Definition of seismic actions in the revised EC8**
- **Proposition of the New Seismic Hazard Zones and Seismic Actions in Greece**
- **Implication in the seismic risk assessment**

Prof. Anastasios Anastasiadis
Dr Evi Riga
Stefania Apostolaki PhD candidate

4

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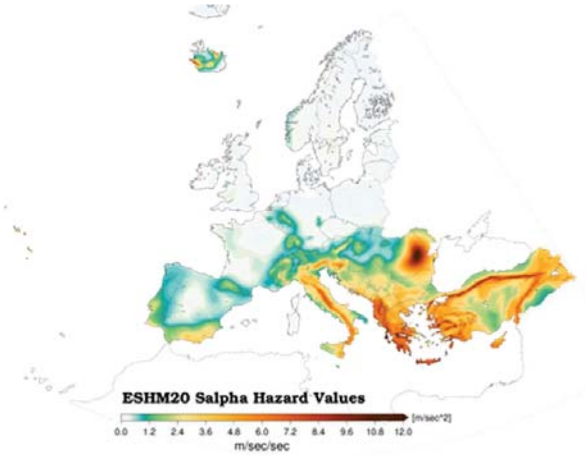
Definition of seismic actions in the revised EC8

General Concepts

- Eurocodes classify structures into Consequence Classes (CC), depending on the consequences of failure or malfunction in terms of loss of human life, economic, social or environmental consequences
- Parameters that concern safety are National Determined Parameters (NDPs). Default values are provided in Eurocode 8
- Eurocode 8 requires that structures are designed in such a way that specified Limit States are not exceeded under prescribed seismic actions
- For new structures, it is required that, at least, the non-exceedance of the Significant Damage (SD) limit state be verified



6

General Concepts

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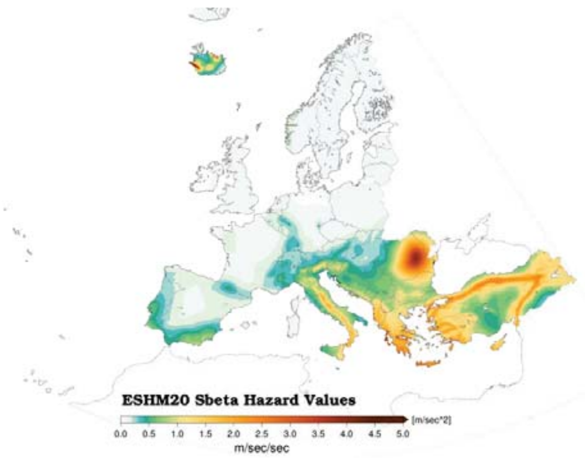


7

General Concepts

The seismic actions are no more defined in terms of the PGA,
Instead, it is proposed to use two spectral values $S_α$ **and** $S_β$ at $T_α$ = 0.3sec and $T_β$=1.0sec

Seismic hazard:
$S_{α,475}$ και $S_{β,475}$ are given for return period of 475 χρόνων at rock conditions (Vs>800m/s). For different return periods magnification factors are provided (equal for all spectral periods) according to the **Consequence Classes και Limit States**

The soil and site **amplification factors $F_α$ και $F_β$** are intensity and soil depended to account for the non-linear behavior of soil deposits.

Both $S_{α,475}$ και $S_{β,475}$ and $F_α$ και $F_β$ may be NDP

8

General Concepts

The seismic design cases should be categorised in seismic action classes according to table below, depending on the value of the seismic action index $S_δ$ defined by

$$S_δ = δF_α F_T S_{α,475}$$

- $δ$ coefficient that depends on the consequence class of the considered structure. The values are given in EN 1998 according to the consequence classes of each structure
- $F_α$ site amplification factor
- F_T topography amplification factor
- $S_{α,475}$ spectral acceleration for rock conditions at low periods for the return period 475 years

Seismic action class	Range of seismic action index $S_δ(\frac{m}{s^2})$
Very low	$S_δ < 1.30m/s^2$
Low	$1.30m/s^2 < S_δ < 3.25m/s^2$
Moderate	$3.25m/s^2 < S_δ < 6.50m/s^2$
High	$S_δ > 6.50m/s^2$

9

General Concepts

Consequences classes defined in EN 1990:2021, **3.1.2.32**, and **4.3(1)**

- Structures should be regarded as belonging to consequence class CC2 if not otherwise specified
- Consequence class CC3 may be divided into two subclasses, CC3-a and CC3-b depending on the importance of structures for public safety and civil protection in the immediate post-earthquake period

NOTE: For a given type of structure, guidance for classification in CC3-a or CC3-b is given in the relevant Part of EN 1998

$$S_\delta = \delta F_a F_T S_{a,475}$$

CC1	$\delta = 0.60$
CC2	$\delta = 1.00$
CC3-a	$\delta = 1.25$
CC3-b	$\delta = 1.60$

δ is NDP for each type of structure

General Concepts

In EN1998-1-1 the seismic actions are defined according to the consequence classes of the structure and the seismic action class

However, in case of important structures (CC3-a and CC3-b) and despite the NDP values which are given, **EC8 suggests the performance of site-specific seismic hazard analysis**

- Deterministic Seismic Hazard Analysis (DSHA): normally applied when the seismic catalogue is poor and there are well-identified active seismic faults affecting directly the structure
- Probabilistic Seismic Hazard Analysis (PSHA): quantifies the uncertainties in the analysis and develops a range of expected ground motions with their probabilities of occurrence

General Concepts

Design response spectra for various return periods

$$S_a = F_a F_T S_{a,RP}$$
$$S_\beta = F_\beta F_T S_{\beta,RP}$$

RP is the return period associated to the CC and Limit States under consideration

For instance, default value for a CC3-a tunnel verified at the Near Collapse (NC) limit state the return period *RP* is 2500 years

$$S_{a,RP} = \gamma_{SD,CC} S_{a,Ref}$$
$$S_{\beta,RP} = \gamma_{SD,CC} S_{\beta,Ref}$$

$S_{a,RP} = S_{a,475}$ in most countries

EN1998-2 Return period for various CC and LS

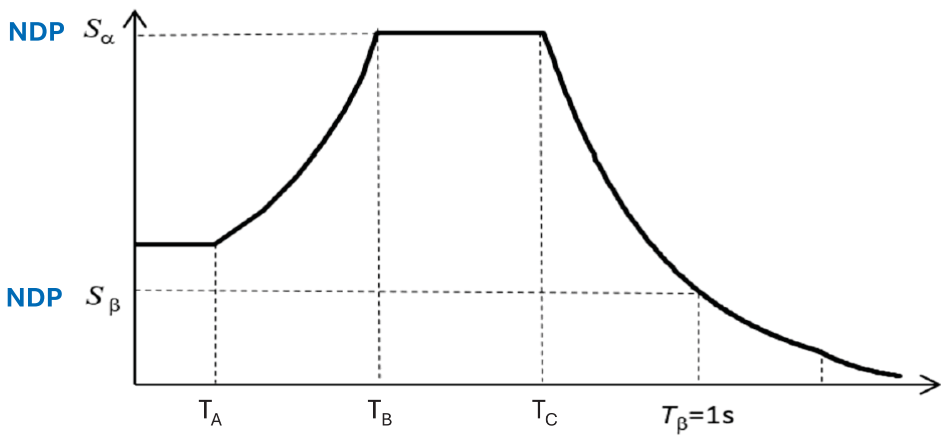
	CC1	CC2	CC3-a	CC3-b
NC	800	1600	2500	5000
SD	250	475	800	1600
DL	50	60	100	200

EN1998-2 $\gamma_{SD,CC}$ for various CC and LS

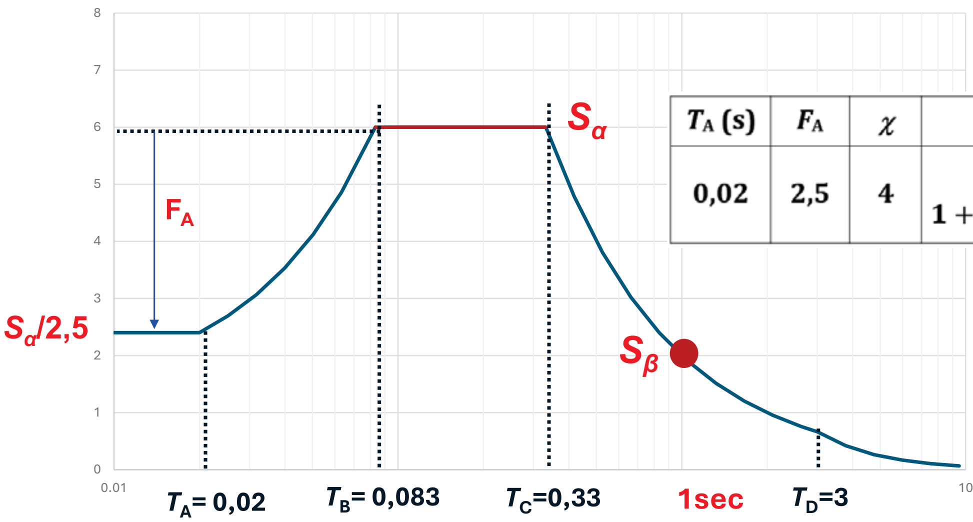
	CC1	CC2	CC3-a	CC3-b
NC	1.2	1.5	1.8	2.2
SD	0.8	1.0	1.25	1.5
DL	0.5	0.5	0.6	0.7

General Concepts

$S_{\alpha,ref}$ and $S_{\beta,ref}$ defined for site category A (rock) for return period T_{ref} = 475y in most EU countries



General Concepts



NDPs

T_A (s)	F_A	χ	T_D (s)
0,02	2,5	4	2 if $S_{\beta,RP} \leq 1\ \mathrm{m/s^2}$ $1 + S_{\beta,RP}$ if $S_{\beta,RP} > 1\ \mathrm{m/s^2}$

$T_c = S_\beta / S_a$

Improved damping correction factor for x ≠ 5%.

General Concepts

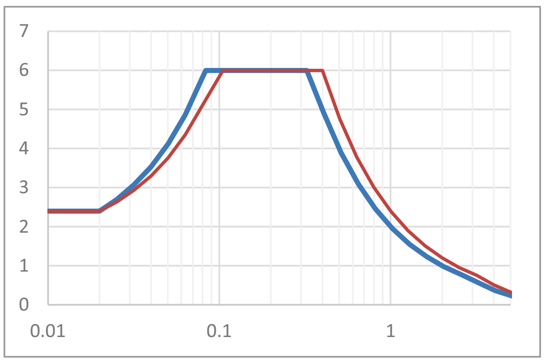
Table 5.2 — Range of $S_{\alpha,475}$ values to define seismicity levels

Seismicity level	$S_{\alpha,475}$ (m/s²)
Very low	< 1,0
Low	1,0 - 2,5
Moderate	2,5 - 5,0
High	≥ 5,0

Approximate $S_{\beta,\mathrm{ref}}$

$$S_{\beta,\mathrm{ref}} = f_h\, S_{\alpha,\mathrm{ref}}$$

f_h = 0,2 for low and very low seismicity levels
f_h = 0,3 for moderate seismicity levels
f_h = 0,4 for high seismicity levels



General Concepts: Soil and site classification

Main parameters for soil and site categories: H_{800} και $V_{s,H}$

Table 5.1 — Standard site categorisation

		Ground class	stiff	medium stiffness	soft
	Depth class	$v_{s,H}$ range / H_{800} range	400 - 800 m/s	250 -400 m/s	150 -250 m/s
$V_{s,H} = V_{s,H_{800}}$	very shallow	$H_{800} \leq 5$ m	A	A	E
	shallow	5 m < $H_{800} \leq 30$ m	B	E	E
$V_{s,H} = V_{s,30}$	intermediate	30 < $H_{800} \leq 100$ m	B	C	D
	deep	$H_{800} > 100$ m	B	F	F

down to 100 m/s for low seismic action class.

- H_{800} is the depth of the seismic bedrock formation identified by V_s at least equal to 800 m/s.
- **If the information on H_{800} and/or $V_{s,H}$ is not available or it is incomplete, Annex B may be used.**

Table B.1 — Simplified description of site categories

Site category	Description
A	Rock or other rock-like geological material, including **very shallow** layers of very dense, dense or medium-dense sand, gravels, very stiff or stiff clay.
B	Deposits consisting prevalently of very dense sand, gravel, or very stiff clay, with representative values of the geotechnical parameters in the range **defined in Table B.2 for stiff ground.**
C	Intermediate-depth deposits consisting prevalently of dense or medium-dense sand, gravel or stiff clay, with representative values of the geotechnical parameters in the range defined in Table B.2 for medium ground.

Table B.2 — Correspondence between ground class and geotechnical parameters

Test	Parameter	Ground class		
		stiff	medium	soft
SPT	N_{60} (SPT, ER = 60%) [blows/30 cm]	> 60	30-60	15-30
CPT	q_c – sands (MPa)	> 30	15-30	5-15
	q_c – clays (MPa)	> 6	3-6	1,5-3
FVT or lab tests	c_u (kPa)	> 300	150-300	50-150

In case of simplified description of site categories, default values of site amplification factors should be used

General Concepts: Soil and site classification

Site classification using f_0

Combination of f_0 (Hz) and $v_{s,H}$ (m/s)	Site cat.
$f_0 \geq 10$ and $v_{s,H} \geq 250$	A
$f_0 < 10$ and $400 \leq v_{s,H} < 800$	B
$v_{s,H}/250 \leq f_0 < v_{s,H}/120$ and $250 \leq v_{s,H} < 400$	C
$v_{s,H}/250 \leq f_0 < v_{s,H}/120$ and $150 \leq v_{s,H} < 250$	D
$v_{s,H}/120 \leq f_0 < 10$ and $150 \leq v_{s,H} < 400$, or $f_0 \geq 10$ and $150 \leq v_{s,H} < 250$	E
$f_0 < v_{s,H}/250$ and $150 \leq v_{s,H} < 400$	F

and $H = \dfrac{v_{s,H}}{4f_0}$

General Concepts: Site amplification factors

Table 5.4 — Site amplification factors F_α and F_β for the standard site categories

Site category	F_α		F_β	
	H_{800} and $v_{s,H}$ available	Default value	H_{800} and $v_{s,H}$ available	Default value
A	1,0	1,0	1,0	1,0
B	$\left(\frac{v_{s,H}}{800}\right)^{-0,40\,r_\alpha}$	$1,3\,(1-0,1\,S_{\alpha,RP}/g)$	$\left(\frac{v_{s,H}}{800}\right)^{-0,70\,r_\beta}$	$1,6\,(1-0,2\,S_{\beta,RP}/g)$
C		$1,6\,(1-0,2\,S_{\alpha,RP}/g)$		$2,3\,(1-0,3\,S_{\beta,RP}/g)$
D		$1,8\,(1-0,3\,S_{\alpha,RP}/g$		$3,2\,(1-S_{\beta,RP}/g)$
E	$\left(\frac{v_{s,H}}{800}\right)^{-0,40\,r_\alpha \frac{H}{30}\left(4-\frac{H}{10}\right)}$	$2,2\,(1-0,5\,S_{\alpha,RP}/g)$	$\left(\frac{v_{s,H}}{800}\right)^{-0,70\,r_\beta \frac{H}{30}}$	$3,2\,(1-S_{\beta,RP}/g)$
F	$0,90\,\left(\frac{v_{s,H}}{800}\right)^{-0,40\,r_\alpha}$	$1,7\,(1-0,3\,S_{\alpha,RP}/g)$	$1,25\cdot\left(\frac{v_{s,H}}{800}\right)^{-0,70\,r_\beta}$	$4,0\,(1-S_{\beta,RP}/g)$
	with $r_\alpha = 1 - \frac{S_{\alpha,RP}/g}{v_{s,H}/150}$ and $r_\beta = 1 - \frac{S_{\beta,RP}/g}{v_{s,H}/150}$			

$$S_\alpha = F_T\,F_\alpha\,S_{\alpha,RP}$$
$$S_\beta = F_T\,F_\beta\,S_{\beta,RP}$$

Paolucci, R., Aimar, M., Ciancimino, A. et al. (2021) Pitilakis K, Riga E, Anastasiadis A (2013).
.

General Concepts

Peak Ground Displacement

$$PGD_e = S_{De}(T_F) = 0{,}025\, T_\beta T_D \mathbf{F_L}\, F_T\, S_{\beta,RP}$$

$\mathbf{F_L}$ is the long period site amplification factor

$$\mathbf{F_L} = \left(\frac{\boldsymbol{v_{s,H}}}{\mathbf{800}}\right)^{\mathbf{-0,4}}$$

Peak Ground Velocity

$$PGV_e = 0{,}06(S_\alpha\, S_\beta)^{0{,}55}$$

General Concepts

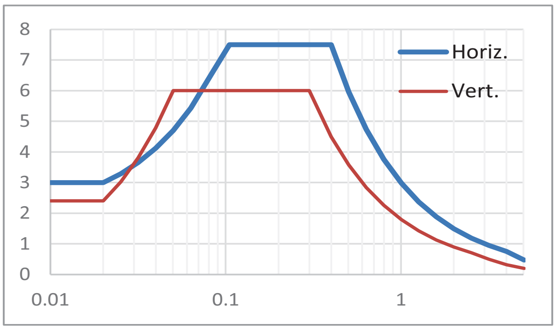
Vertical acceleration

$$S_{\alpha v} = f_{v\alpha}\, S_\alpha \qquad f_{v\alpha} = \begin{cases} 0{,}6 & \text{if } S_\alpha < 2{,}5\ \mathrm{m/s^2} \\ 0{,}04\, S_\alpha + 0{,}5 & \text{if } 2{,}5\ \mathrm{m/s^2} \le S_\alpha \le 7{,}5\ \mathrm{m/s^2} \\ 0{,}8 & \text{if } S_\alpha > 7{,}5\ \mathrm{m/s^2} \end{cases}$$

$$S_{\beta v} = f_{v\beta}\, S_\beta \qquad f_{v\beta} = 0{,}6$$

$T_{Bv} = 0{,}05$ s

$T_{CV} = S_{\beta v} / S_{\alpha v}$



General Concepts

Several other clauses dealing with different other aspects of the design ground motion

- Oblication for site specific ground response analysis
- Spatial variability of ground motion
- Fault proximity and crossing
- Selection of time histories
- Topographic amplification

- Reduced elastic spectrum for the force-based approach

$q = q_R \, q_S \, q_D$

q_R is the behaviour factor component accounting for **overstrength** due to the redistribution of seismic action effects in redundant structures;
q_S is the behaviour factor component accounting for **overstrength** due to all other sources;
q_D is the behaviour factor component accounting for the **deformation capacity and energy dissipation capacity.**

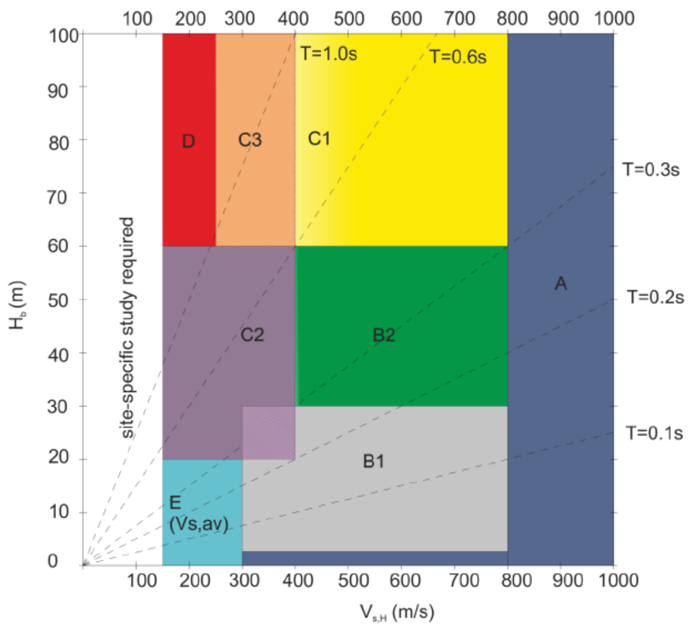
Alternative proposal for site classification and amplification

- Based on more than 3000 well documented soil and site data from a worldwide data base (Pitilakis K, Riga E, Anastasiadis A (2013))
- Correlations of soil classes with the average shear wave velocity of the entire soil deposit, $V_{s,av}$, and average values of standard penetration test blow count, N-SPT, and undrained shear strength, S_u
- **Easier to apply**
- 6 main soil classes (A, B, C, D, E and X) with sub-classes for classes B and C
- Main classification parameters:
  - approximate thickness of the soil deposit, H_B (depth of seismic bedrock)
  - equivalent shear wave velocity of the superficial soil deposit, $V_{s,H}$, which is equal to $V_{s,30}$ for soil deposits with depth >30 m.
- T_0 is used as a supplementary parameter and to distinguish between specific subclasses
- Ranges of H_B, $V_{s,H}$, T_0 and $Vs_{,av}$ for site classes were derived based on statistics from good quality experimental data from the SHARE-AUTH database and when needed from theoretical analyses of representative models of realistic soil conditions applying classical statistics.

Alternative proposal for site classification and amplification

- Main classification parameters:
  - approximate thickness of the soil deposit, H_B (depth of seismic bedrock)
  - equivalent shear wave velocity of the superficial soil deposit, $V_{s,H}$, which is equal to $V_{s,30}$ for soil deposits with depth >30 m.
  - T_0 fundamental period of the site
  - Geotechnical parameters: N-SPT, PI, undrained shear strength S_u

Pitilakis et al. 2013, 2018, 2020



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Alternative proposal for site classification and amplification

Site class	Description	H_B (m)	$V_{s,H}$ (m/s)	T_0 (s)	Remarks
A	- Rock - Slightly weathered/ segmented rock formations with weathered layer of thickness z <5.0 m		≥ 800	≤ 0.2	For weathered zone: z<5m: Vs,av ≥ 300 m/s
B1	- Weathered/soft rock - Shallow very stiff soil deposits, consisting either of very dense sand/gravel or very stiff to hard clay	≤ 30	300-800	0.2 ± 0.1	Vs,av : 400 - 800 m/s N-SPT > 50 Su> 150 kPa
B2	Intermediate depth stiff soil deposits, consisting either of sand or clay, whose mechanical properties increase with depth	30 - 60	400-800	0.4 ± 0.2	Vs,av : 400 - 800 m/s N-SPT > 50 Su> 150 kPa
C1	Deep stiff soil deposits, consisting either of sand/gravel or clay	> 60	400-800	0.6 ± 0.2	Vs,av : 400 - 800 m/s N -SPT> 50 Su > 150 kPa
C2	Intermediate depth soil deposits, consisting of medium dense sand and gravel and/or medium stiffness clay (PI > 15, fines > 30%)	20 - 60	150-400	0.5 ± 0.2	Vs,av: 200 - 500 m/s N -SPT> 20 150 kPa> Su>70 kPa

Alternative proposal for site classification and amplification

Site class	Description	H_B (m)	$V_{s,H}$ (m/s)	T_0 (s)	Remarks
C3	Deep soil deposits, consisting of medium dense sand and gravel and/or medium stiffness clay	> 60	250-400	1.2 ± 0.5	Vs,av: 300 - 500 m/s N -SPT> 20 150 kPa> Su >70 kPa
D	Deep soil deposits consisting of soft to medium stiffness clays and/or loose sandy to sandy-silt formations with substantial fines percentage (potentially non-liquefiable)	> 60	150-250	2.0 ± 0.8	Vs,av: 200 - 400 m/s N-SPT < 20 Su < 70 kPa The dominant soil formations may be interrupted by layers of very soft clays (Su<25 kPa, W>40%, PI>25) or sands and sandy clays of relatively small thickness (<10m)
E	Shallow soil deposits, generally classified as type C2 or D according to its geotechnical propertied, which overlie type A formations	< 20	150-300	≤ 0.5	
X	Loose fine sandy-silty soils with high water table, potentially liquefiable Loose granular or soft silty-clayey soils, provided they have been proven to be hazardous in terms of dynamic compaction or loss of strength. Soils near obvious tectonic faults Steep slopes covered with loose soil deposits Recent loose landfills Soils with a very high percentage in organic material Peat and/or highly organic clays (H>3m) and/or very high plasticity clays (H>8m) and /or very thick. soft/medium stiff clays (H>30m) Loess Special soils and site conditions requiring site-specific evaluations - not included in types A – E				

Site amplification factors

Short period site amplification factor F_s

Site class	S_{sRP} (maximum response spectral acceleration at short period on site class A in g)					
	S_{sRP}<0.25	S_{sRP}=0.25	S_{sRP}=0.5	S_{sRP}=0.75	S_{sRP} =1.0	S_{sRP} ≥1.25
A	1.00	1.00	1.00	1.00	1.00	1.00
B1	1.30	1.30	1.30	1.20	1.20	1.20
B2	1.30	1.30	1.20	1.20	1.20	1.10
C1	1.70	1.70	1.60	1.50	1.50	1.40
C2	1.60	1.50	1.30	1.20	1.10	1.00
C3	1.70	1.60	1.40	1.20	1.20	1.10
D	1.80	1.70	1.50	1.40	1.30	1.20
E	1.70	1.60	1.60	1.50	1.50	1.40
X	-	-	-	-	-	-

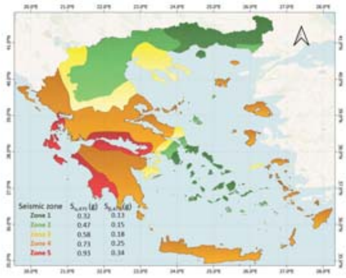
Site amplification factors

Intermediate period site amplification factor F_1

Site class	S_{sRP} (maximum response spectral acceleration at short period on site class A in g)					
	S_{sRP}<0.25	S_{sRP}=0.25	S_{sRP}=0.5	S_{sRP}=0.75	S_{sRP} =1.0	S_{sRP} ≥1.25
A	1.00	1.00	1.00	1.00	1.00	1.00
B1	1.10	1.10	1.10	1.10	1.10	1.10
B2	1.40	1.40	1.30	1.30	1.30	1.30
C1	1.50	1.50	1.40	1.40	1.40	1.40
C2	2.30	2.20	2.00	1.90	1.90	1.80
C3	2.40	2.30	2.10	2.00	2.00	1.90
D	4.00	3.50	3.00	2.70	2.40	2.30
E	1.20	1.10	1.10	1.10	1.10	1.10
X	-	-	-	-	-	-

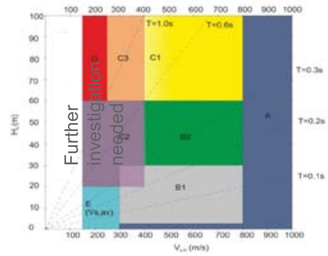
Proposition of the New Seismic Hazard Zones and Seismic Actions in Greece

Proposition of the New Seismic Hazard Zones and Seismic Actions in Greece



Pitilakis K, Riga E, Apostolaki S, Danciu L. (2024). Seismic hazard zonation map and definition of seismic actions for Greece in the context of the ongoing revision of EC8. Bull Earthquake Eng 22, 3753–3792. https://doi.org/10.1007/s10518-024-01919-8

Pitilakis K, Riga E, Anastasiadis A (2020) Towards the revision of EC8: Proposal for an alternative site classification scheme and associated intensity dependent amplification factors. In: 17th World Conference on Earthquake Engineering, Sendai, Japan 13–18 Sep 2020.



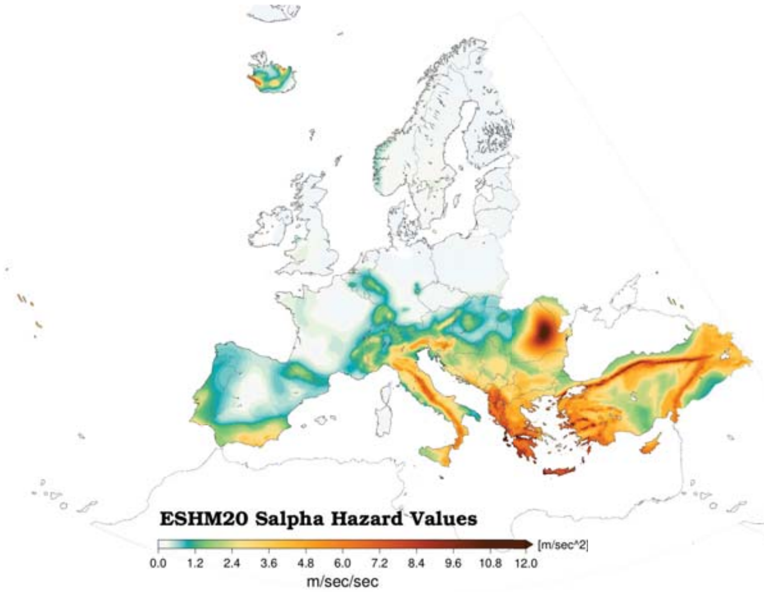
EC8 Part1 Annex A : informative European seismic hazard maps

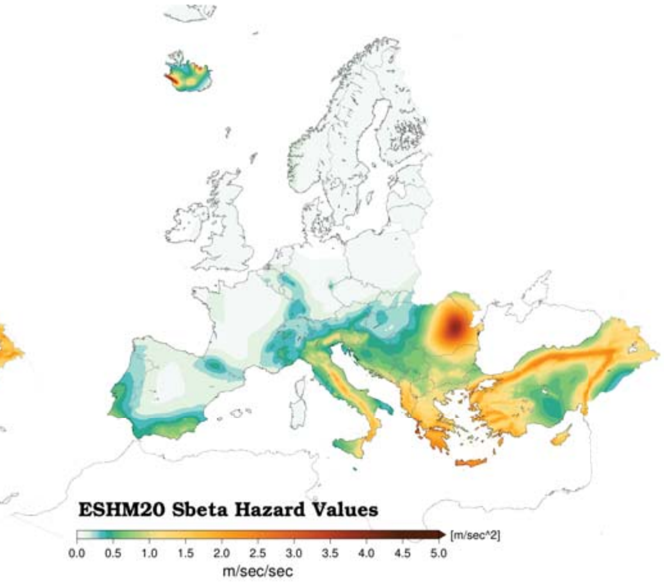
Informative Annex contains **small-scale maps of $S_{a,475}$ and $S_{\beta,475}$ values**

NOTE: The seismic hazard maps issued from deliverable of **ESHM20** research project, which received funding from the EU Horizon 2020 research and innovation program, in order to provide updated and homogeneous information on the seismic hazard in Europe.

ESHM20

Spectral acceleration maps (T=475y) for rock conditions





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European Seismic Hazard Model (ESHM20)

Main Input Datasets

EPICA: the European PreInstrumental earthquake CAtalogue: Historical Earthquake Catalogue (1000-1899)
Rovida and Antonucci, 2021

- contains 5703 earthquakes with either maximum observed intensity≥5 or Mw≥4.0, for the period 1000-1899.

Rovida A., Antonucci A. (2021). EPICA - European PreInstrumental Earthquake CAtalogue, version 1.1. Istituto Nazionale di Geofisica e Vulcanologia (INGV).Dataset. https://doi.org/10.13127/epica.1.1

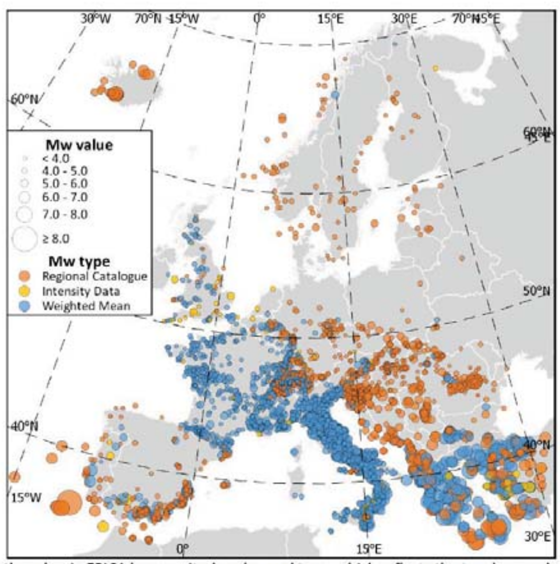


Fig. 1.2 Earthquakes in EPICA by magnitude value and type, which reflects the typology and reliability of supporting data.

European Seismic Hazard Model (ESHM20)

Main Input Datasets

EMEC: the instrumental European-Mediterranean Earthquake Catalogue

- contains 55,732 events with $M_w \geq 3.5$ in the period 1900 to the end of 2014

S. Lammers, G. Grünthal, G. Wheatherill, F. Cotton
GFZ Seismic Hazard and Risk Dynamics

Danciu L., Nandan S., Reyes C., Basili R., Weatherill G., Beauval C., Rovida A., Vilanova S., Sesetyan K., Bard P-Y., Cotton F., Wiemer S., Giardini D. (2021) - The 2020 update of the European Seismic Hazard Model: Model Overview. EFEHR Technical Report 001, v1.0.0, https://doi.org/10.12686/a15.

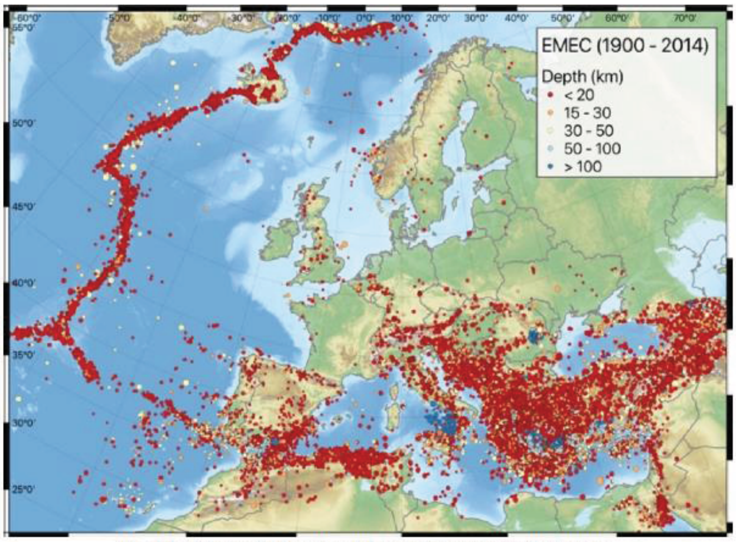


Fig. 1.4 The complete EMEC catalogue for the period 1900 – 2012.

European Seismic Hazard Model (ESHM20)

Main Input Datasets

European Fault-Source Model 2020 (EFSM20)

- An update of the European Database of Seismogenic Faults 2013 (EDSF13; Basili et al., 2013)
- two main categories of seismogenic faults are considered: 1) crustal faults; and 2) subduction zones

Danciu L., Nandan S., Reyes C., Basili R., Weatherill G., Beauval C., Rovida A., Vilanova S., Sesetyan K., Bard P-Y., Cotton F., Wiemer S., Giardini D. (2021) - The 2020 update of the European Seismic Hazard Model: Model Overview. EFEHR Technical Report 001, v1.0.0, https://doi.org/10.12686/a15.

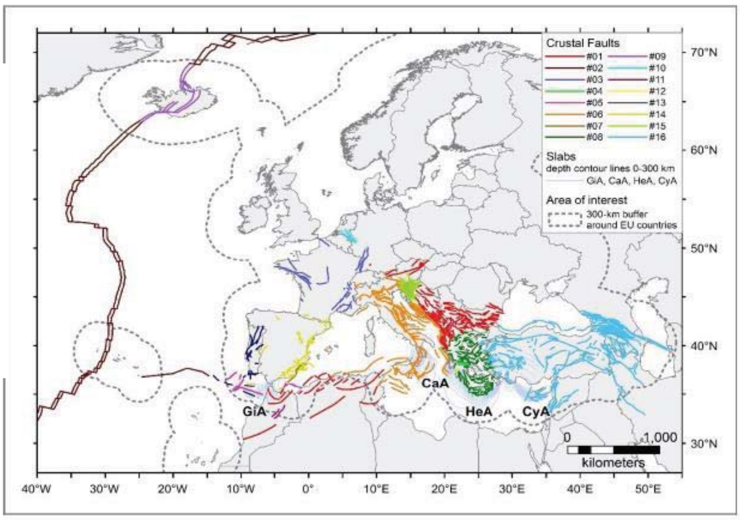


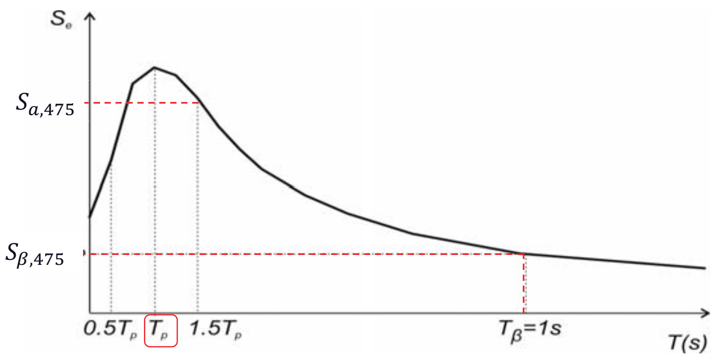
Fig. 1.6 Map of collated fault datasets for the development of the European Fault-Source Model 2020 (EFSM20). See text for the descriptions of the various datasets. From west to east, the subduction systems are: Gibraltar Arc (GiA); Calabrian Arc (CaA); Hellenic Arc (HeA); and Cyprus Arc (CyA).

European Seismic Hazard Model (ESHM20) in the revision of EC8

- The European Seismic Hazard Model ESHM20 (Danciu et al., 2021) was released, in 2021 developed within the EU funded project "Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe" (SERA).
- It is an update of the previous ESHM13 (Woessner et al., 2013), proposed in the framework of SHARE project (Giardini et al.,2013)
- It has been built upon recently compiled and fully cross-border harmonized datasets (i.e., earthquake catalogues, active faults, ground shaking recordings), information (tectonic and geological) and models (seismogenic sources, ground shaking).
- The source data, input models and output of ESHM20 are online available at the portal of the European Facilities for Earthquake Hazard and Risk (www.hazard.EFEHR.org)

European Seismic Hazard Model (ESHM20) in the revision of EC8

- Thanks to the **interaction of ESHM20 with CEN/TC250/SC8**, where our team in AUTH was strongly involved, ESHM20 additionally delivered maps of the **two seismic hazard parameters of Revised EC8 for T=475 years, $S_{\alpha,475}$ and $S_{\beta,475}$**
- $S_{\alpha,475}$ and $S_{\beta,475}$ maps are derived from ESHM20 median uniform hazard spectra (**UHS**) obtained across Europe **T=475 years**
- $S_{\beta,475}$ is directly the UHS value for a spectral period of 1 s.
- $S_{\alpha,475}$ is calculated as the average spectral value over the range of periods between $0.5T_p$ and $1.5T_p$



European Seismic Hazard Model (ESHM20) in the revision of EC8

Seismic hazard results of ESHM20 are provided at a grid covering the whole Europe and Turkey, and include:

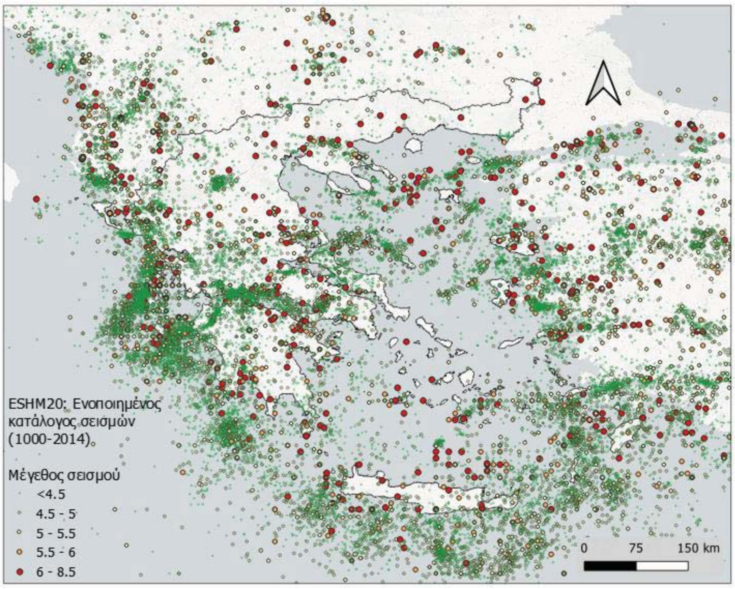
- ❑ hazard curves (5th, 16th, 50th, 84th and 95th percentiles) for specified IMs
- ❑ Uniform Hazard Spectra (UHS) (5th, 16th, 50th, 84th and 95th percentiles) for five mean return periods Tm (i.e., 50, 475, 975, 2500 and 5000 years).
- ❑ hazard maps for all intensity measure types and all return periods
- ❑ the two seismic hazard parameters for T=475 years, used in the revised Eurocode 8 to anchor the horizontal elastic response spectra for rock conditions, $S_{\alpha,475}$ and $S_{\beta,475}$ (interaction with CEN/TC250/SC8 for the development of the revised Eurocode 8)

ESHM20 seismogenic sources

The main seismogenic source model consists of four distinct source models:

- ❑ The **area sources model** is assumed to be the pan-European consensus model, incorporating the national area sources provided by local experts and fully cross-border harmonization.
- ❑ **Active faults and background smoothed seismicity**, a hybrid seismicity model that combines the updated active faults datasets with the background seismicity in regions where faults are identified.
- ❑ Subduction sources depicting both the subduction interface and in-slab of the Hellenic, Cyprian, Calabrian and Gibraltar Arcs.
- ❑ Non-subducting deep seismicity sources describe the nested seismicity with depth in Vrancea, Romania, and the cluster of deep seismicity in the southern Iberia Peninsula.

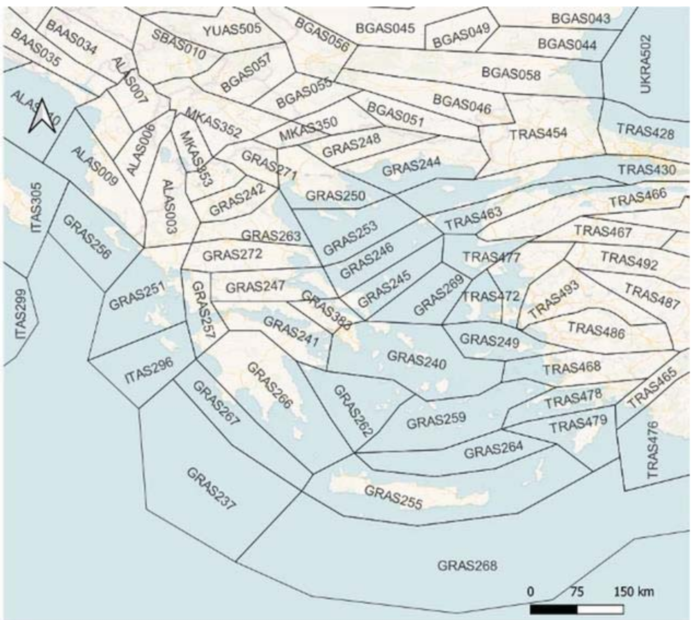
ESHM20 input datasets for Greece



Unified earthquake catalogue

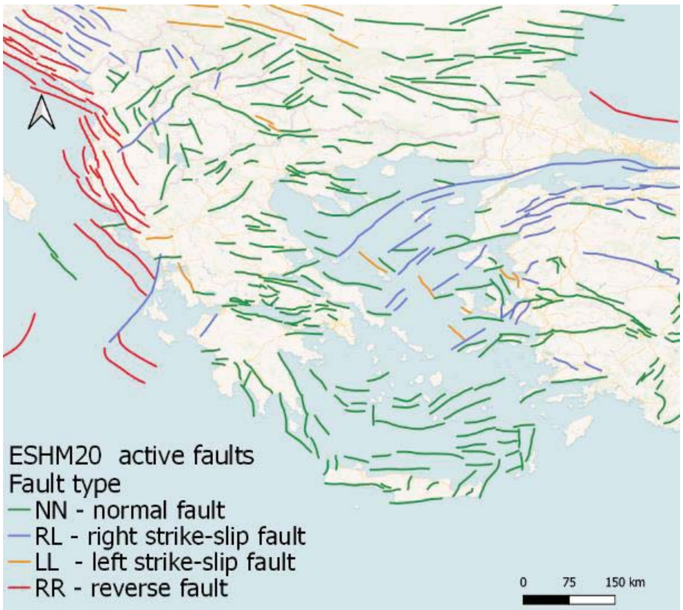
Epicenters and magnitudes of earthquakes included in the ESHM20 unified earthquake catalogue for Greece (Grünthal and Wahlström, 2012; Rovida and Antonucci, 2021; Danciu et al., 2021).

ESHM20 seismogenic sources



Area sources of ESHM20 for Greece

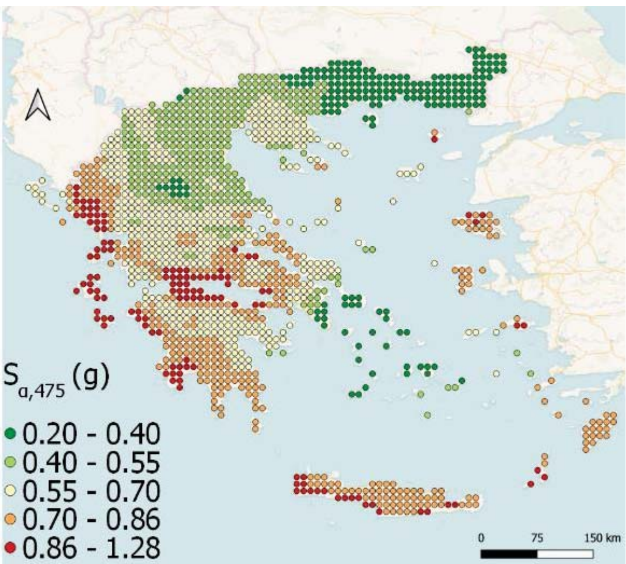
ESHM20 input datasets for Greece



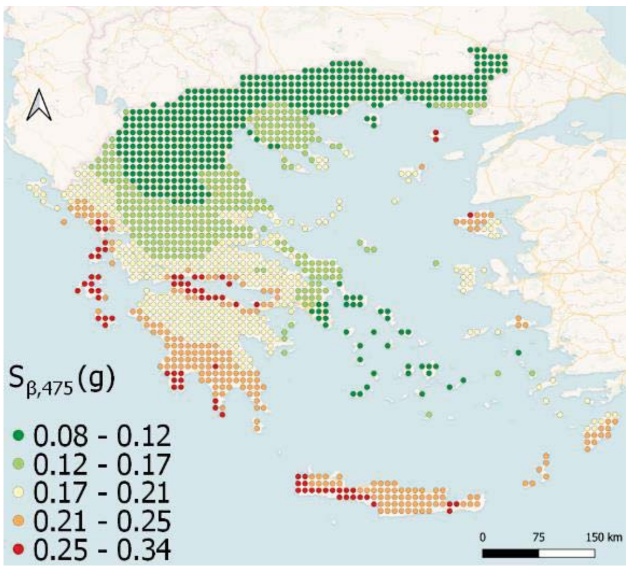
Active Faults and subduction zones database (EFSM20, Basili et al., 2020)

Application of ESHM20 in Greece according to the revision of EC8

median $S_{\alpha,475}$ for Greece (rock)

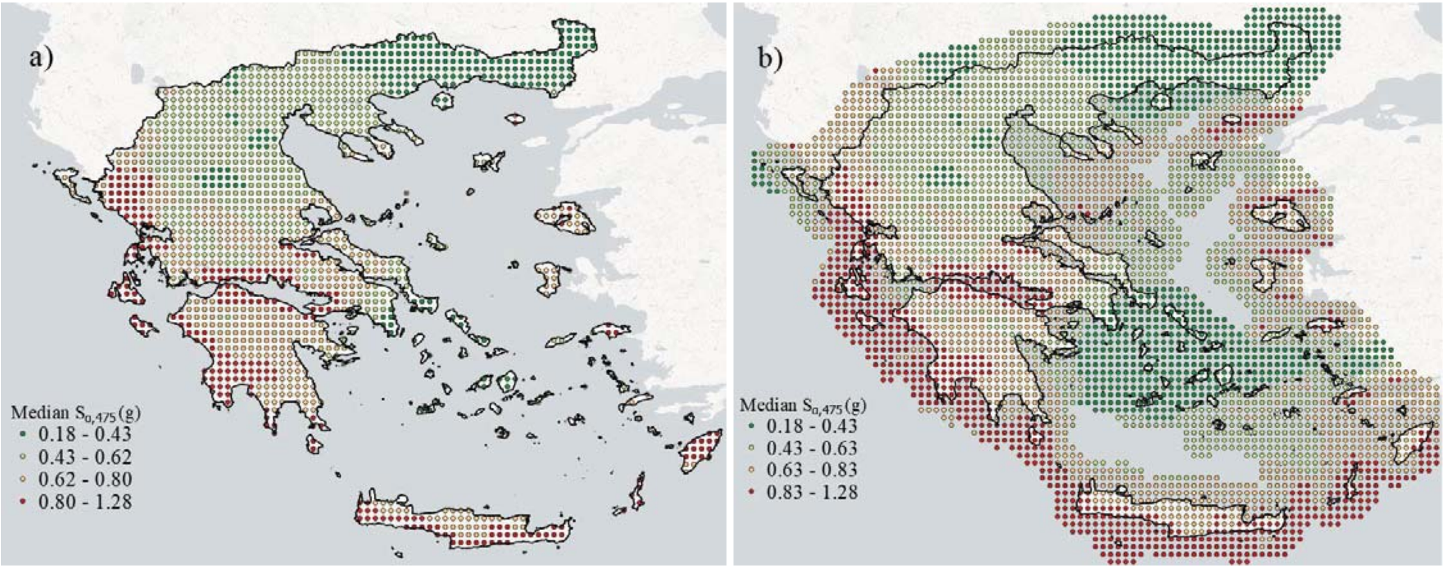


median $S_{\beta,475}$ for Greece (rock)



Application of ESHM20 in Greece according to the revision of EC8

Median $S_{\alpha,475}$ for Greece using Jenks classification method



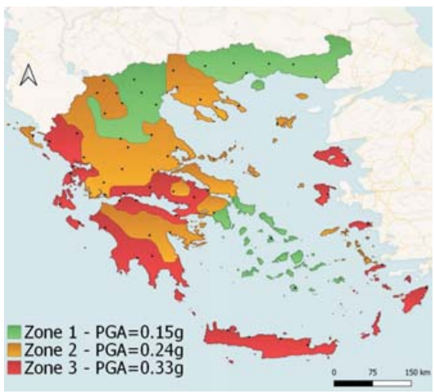
only terrestrial grid points | terrestrial and offshore grid points

Applying ESHM20 to propose a new seismic hazard map for Greece (1/4)

From grid points to zones

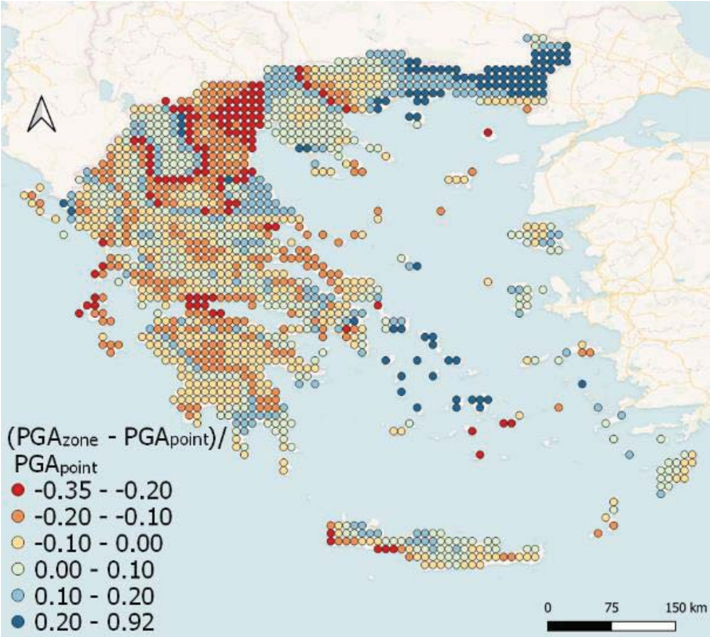
- **Three-seismic zones map** using the $S_{\alpha,475}$ parameter and Natural Breaks (Jenks 1967) algorithm
- Average $S_{\alpha,475}$/2.5 values of the grid points within each zone, **PGA_{zone}**
- $S_{\alpha,475}$/2.5 values for all terrestrial ESHM20 grid points in Greece, **PGA_{point}**
- **(PGA_{zone}-PGA_{point})/PGA_{point}** ratio for all grid points as a measure of the deviation between PGA_{zone} and PGA_{point}

3 Zones



Zone	PGA_{zone} (g)
1	0.15 ± 0.03
2	0.24 ± 0.03
3	0.33 ± 0.04

Applying ESHM20 to propose a new seismic hazard map for Greece (2/4)



3 Zones

Negative ratio values → the **zonation underestimates PGA** with respect to ESHM20 grid output

Positive ratio values → the **zonation overestimates PGA** with respect to ESHM20 grid output

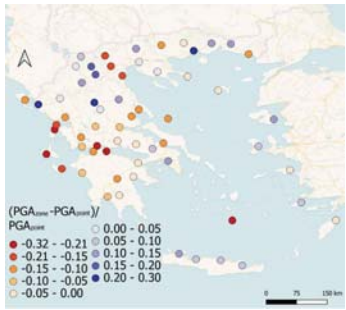
Applying ESHM20 to propose a new seismic hazard map for Greece (3/4)

- For the 63 major cities in Greece, maximum allowable thresholds are applied to the **(PGA_{zone}-PGA_{point})/PGA_{point}** ratio based on **population criteria**

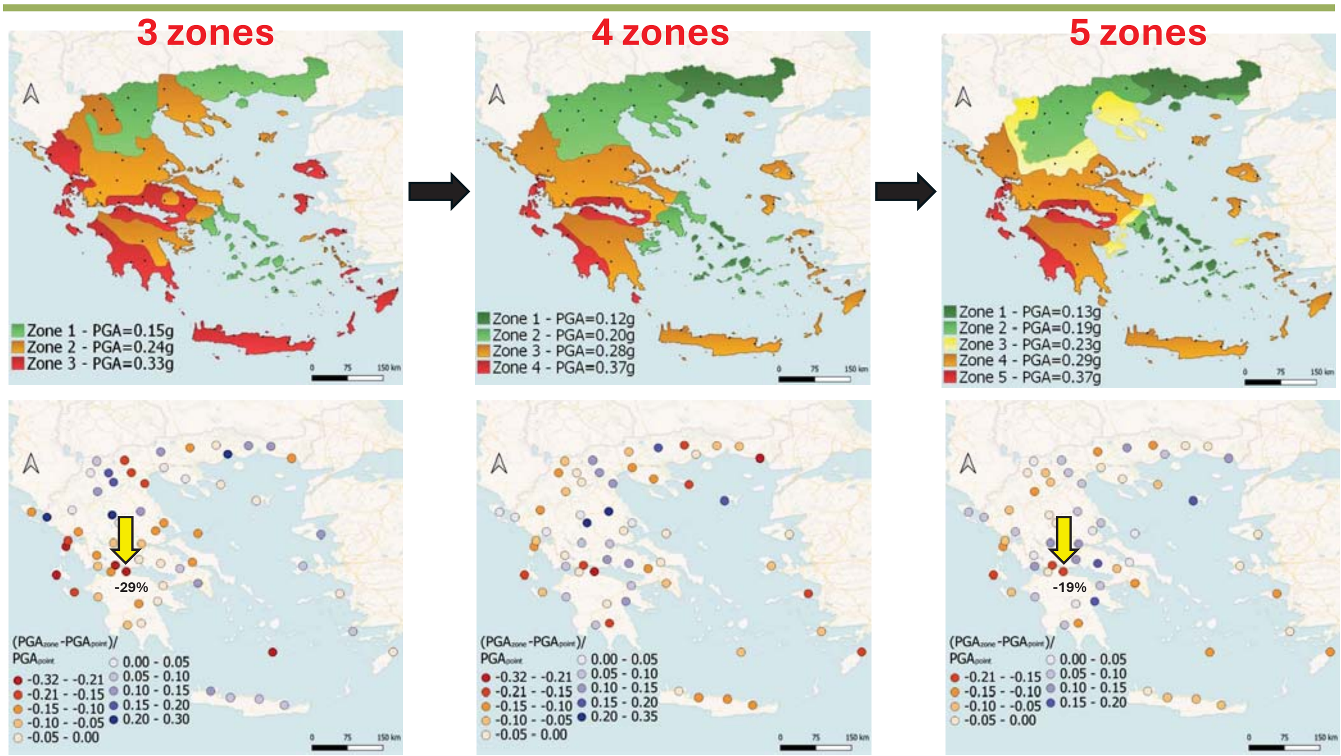
Population class	Threshold values for (PGA_{zone}-PGA_{point})/PGA_{point}
≤ 100,000	± 20%
100,000 – 500,000	± 15%
≥ 500,000 – 1,000,000	± 10%

- For the three-zone map, criteria are not met for some cities, so the number of zones ncreased and the process is iterated.
- Iterative process concluded with a seismic hazard zonation map consisting of **5 zones**

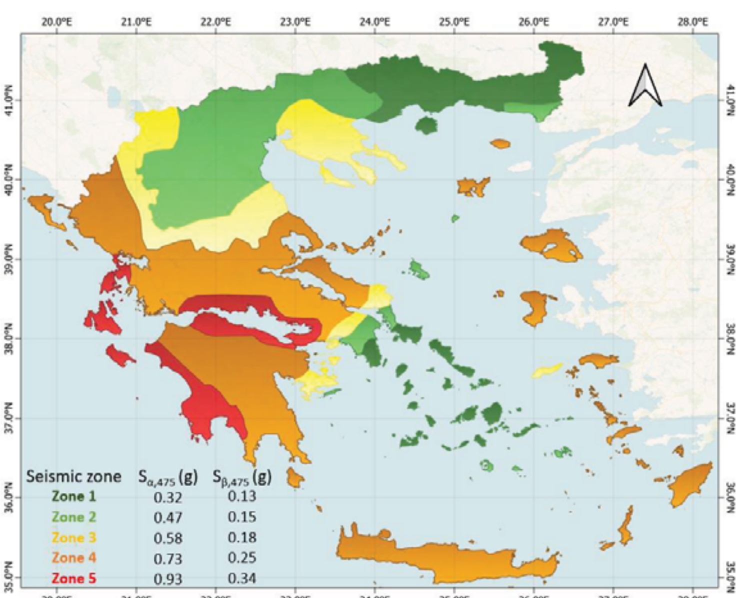
3 Zones



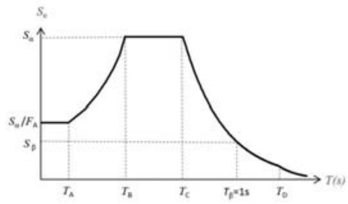
Applying ESHM20 to propose a new seismic hazard map for Greece (4/4)



Proposed ground shaking zonation for Greece (Rock conditions)



- ✓ Further **smoothing of the zone borders** so that they do not cross large urban areas
- ✓ **Harmonization** of specific areas belonging in the same administrative units
- ✓ **Same map for $S_{\alpha,475}$ and $S_{\beta,475}$**



Seismic zone	$S_{\alpha,475}$ (g)	$S_{\beta,475}$ (g)	PGA (g)	T_A (s)	T_B (s)	T_C (s)	T_D (s)
Zone 1	0.32	0.13	0.13	0.02	0.10	0.41	2.28
Zone 2	0.47	0.15	0.19	0.02	0.08	0.32	2.47
Zone 3	0.58	0.18	0.23	0.02	0.08	0.31	2.77
Zone 4	0.73	0.25	0.29	0.02	0.09	0.35	3.45
Zone 5	0.93	0.34	0.37	0.02	0.09	0.36	4.34

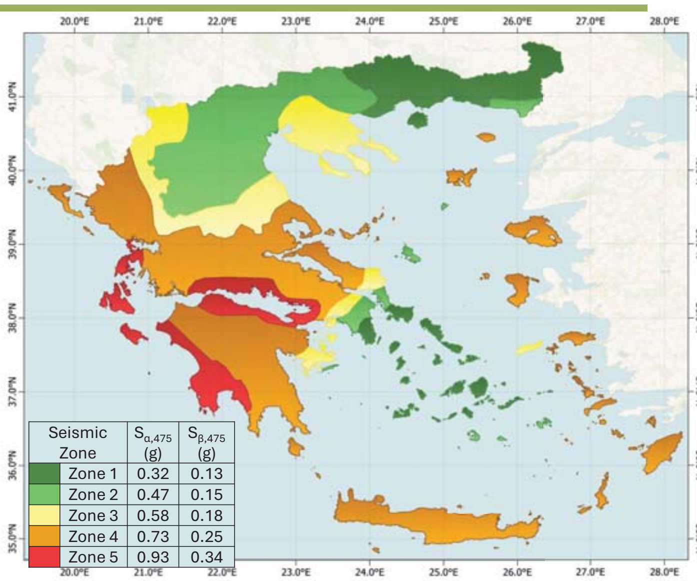
Intensity-dependent soil amplification factors

F_{α}

	1	2	3	4	5
A	1	1	1	1	1
B1	1.3	1.3	1.3	1.2	1.2
B2	1.3	1.2	1.2	1.2	1.2
C1	1.7	1.6	1.6	1.5	1.5
C2	1.5	1.3	1.3	1.2	1.1
C3	1.5	1.4	1.3	1.2	1.2
D	1.6	1.5	1.5	1.4	1.3
E	1.6	1.6	1.6	1.5	1.5

F_{β}

	1	2	3	4	5
A	1	1	1	1	1
B1	1.1	1.1	1.1	1.1	1.1
B2	1.4	1.3	1.3	1.3	1.3
C1	1.5	1.4	1.4	1.4	1.4
C2	2.1	2.0	2.0	1.9	1.9
C3	2.2	2.1	2.1	2.0	2.0
D	3.4	3.1	2.9	2.7	2.5
E	1.1	1.1	1.1	1.1	1.1



based on Pitilakis et al. (2020) site categorization and proposed amplification factors

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Earthquake & Geotechnical Engineering

March 27th, 2025 • Romania - Greece Seminar

Implication in the seismic risk assessment

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Earthquake & Geotechnical Engineering
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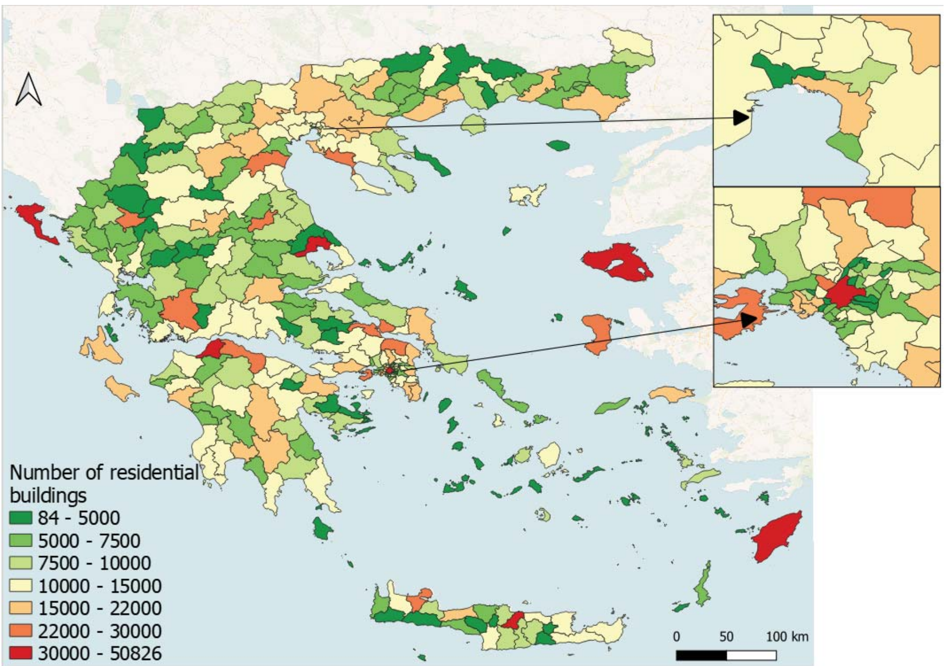
Exposure

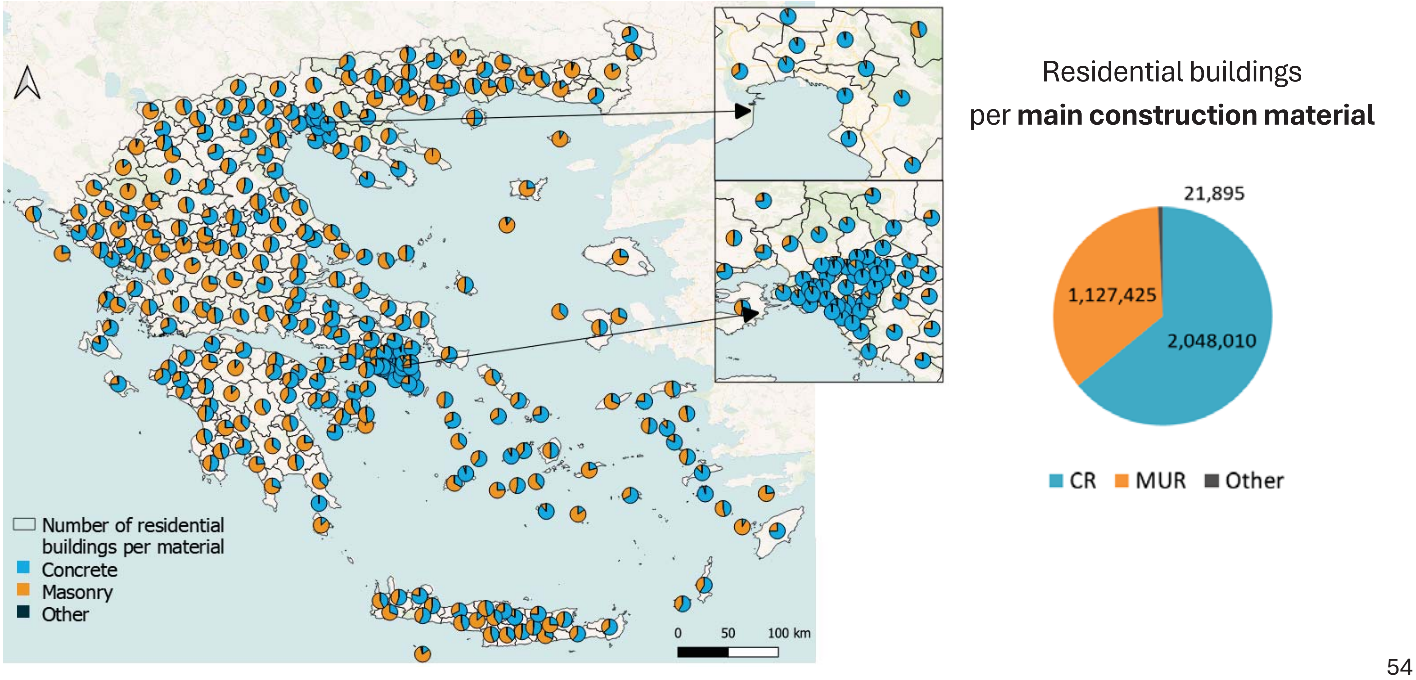
Number of residential buildings



3,197,330 residential buildings classified to 161 taxonomies

Municipality	Residential buildings
ΔΗΜΟΣ ΑΘΗΝΑΙΩΝ	**50,826**
ΔΗΜΟΣ ΛΕΣΒΟΥ	49,944
ΔΗΜΟΣ ΚΕΡΚΥΡΑΣ	49,586
ΔΗΜΟΣ ΠΑΤΡΕΩΝ	44,692
ΔΗΜΟΣ ΒΟΛΟΥ	40,463
ΔΗΜΟΣ ΗΡΑΚΛΕΙΟΥ	38,460
ΔΗΜΟΣ ΡΟΔΟΥ	36,547
ΔΗΜΟΣ ΣΑΛΑΜΙΝΟΣ	29,962
ΔΗΜΟΣ ΑΓΡΙΝΙΟΥ	28,884
ΔΗΜΟΣ ΧΑΛΚΙΔΕΩΝ	28,880
ΔΗΜΟΣ ΧΑΝΙΩΝ	28,612
ΔΗΜΟΣ ΝΕΑΣ ΠΡΟΠΟΝΤΙΔΑΣ	26,721
ΔΗΜΟΣ ΧΙΟΥ	26,392
ΔΗΜΟΣ ΛΑΡΙΣΑΙΩΝ	26,000
ΔΗΜΟΣ ΠΕΡΙΣΤΕΡΙΟΥ	24,674
ΔΗΜΟΣ ΑΙΓΙΑΛΕΙΑΣ	24,290
ΔΗΜΟΣ ΙΩΑΝΝΙΤΩΝ	23,321
ΔΗΜΟΣ ΚΑΤΕΡΙΝΗΣ	22,316
ΔΗΜΟΣ ΩΡΩΠΟΥ	22,262
ΔΗΜΟΣ ΠΕΙΡΑΙΩΣ	21,723
ΔΗΜΟΣ ΑΧΑΡΝΩΝ	21,487
ΔΗΜΟΣ ΘΕΣΣΑΛΟΝΙΚΗΣ	**21,402**

Number of residential buildings per main construction material



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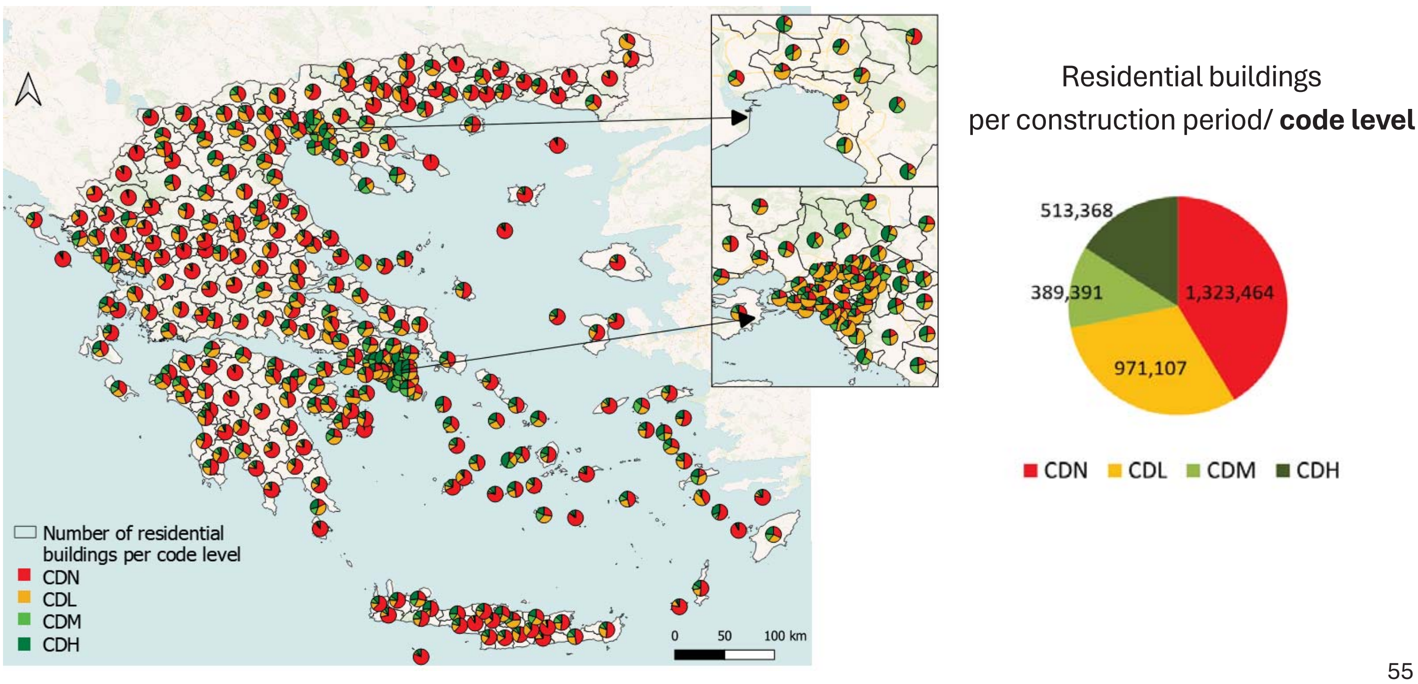
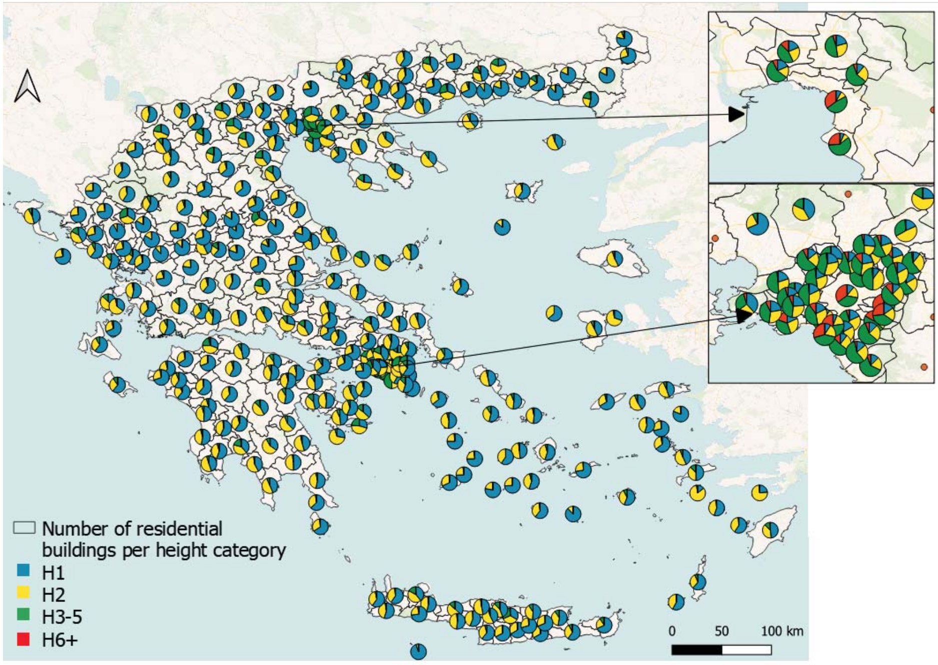
Number of residential buildings per code level

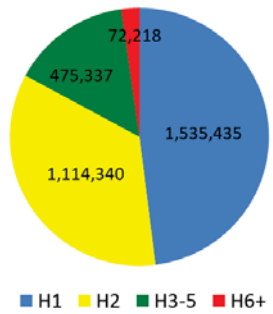


55

Number of residential buildings per height



Residential buildings
per **number of storeys**/ height



56

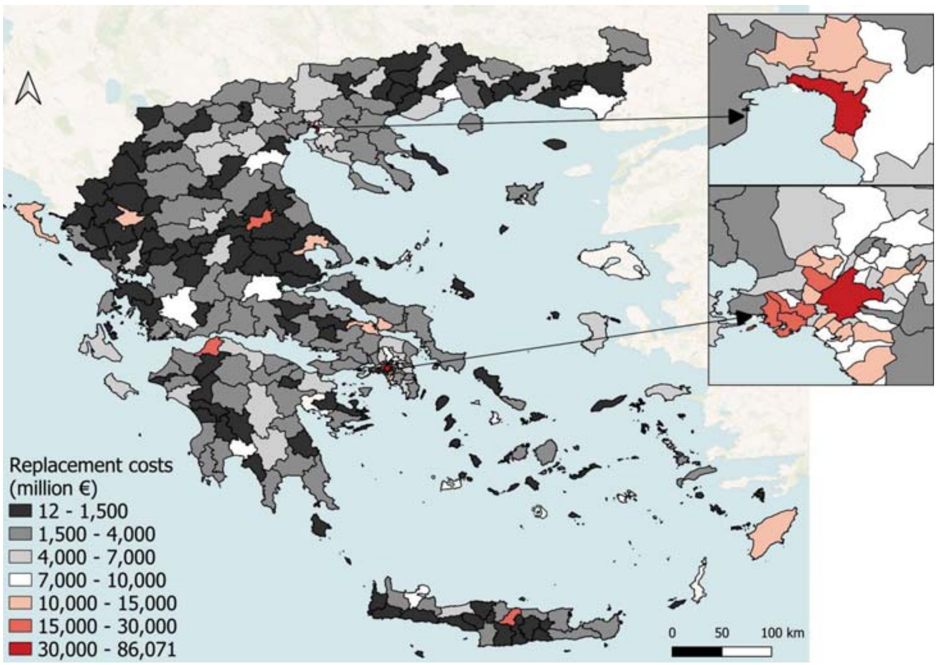
Replacement cost

Replacement cost:
- 1500 €/m^2 for Athens, Thessaloniki
- 1200 €/ m^2 for the rest municipalities

Number of storeys	Slab area (m^2)
H1	90
H2	90
H3	300
H4	300
H5	300
H6	300
H7	300
H8	300
H9	300

Total replacement cost:
1258 billion €

Athens → 86.07 billion €
Thessaloniki → 40,88 billion €



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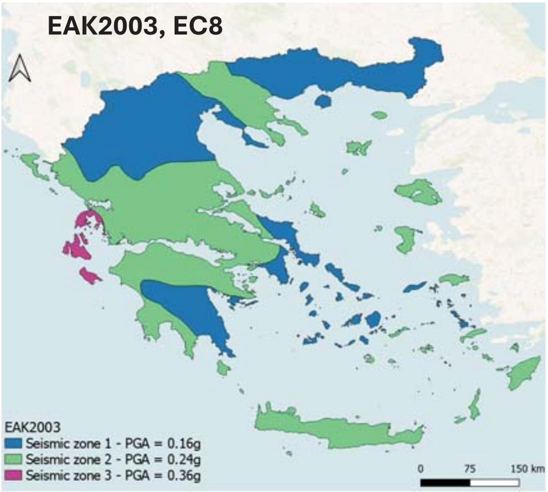
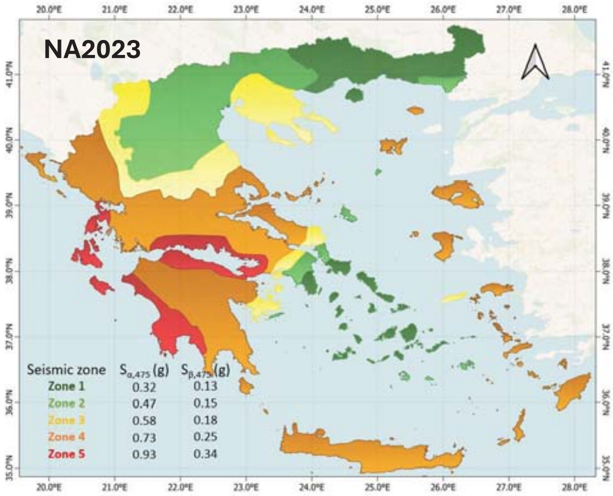
Seismic hazard – RP = 475y

Scenario-type analyses for 475y return period

- **Three scenarios for the seismic demand** (PGA, Sa-0.3s, Sa-0.6s, Sa-1.0s):
  - Current Greek Seismic Code (**EAK2003**):
    - PGA values of the zonation map in force
    - No site amplification factor
  - **Current EC8:**
    - PGA for rock conditions obtained from the zonation map in force (adopted by the Greek NA)
    - Site amplification factors of current EC8 for Type 1 seismicity.
  - Proposal for the Greek National Annex of revised EC8 (**NA2023**):
    - $S_{\alpha,475}$ and $S_{\beta,475}$ parameters for rock conditions for the five zones of the new map
    - Intensity-dependent site amplification factors F_α and F_β for the five zones of the new map

Seismic hazard at rock





Seismic Zone	PGA_{475y} (g)	$S_{\alpha,475}$ (g)	$S_{\beta,475}$ (g)
Zone 1	0.13	0.32	0.13
Zone 2	0.19	0.47	0.15
Zone 3	0.23	0.58	0.18
Zone 4	0.29	0.73	0.25
Zone5	0.37	0.93	0.34

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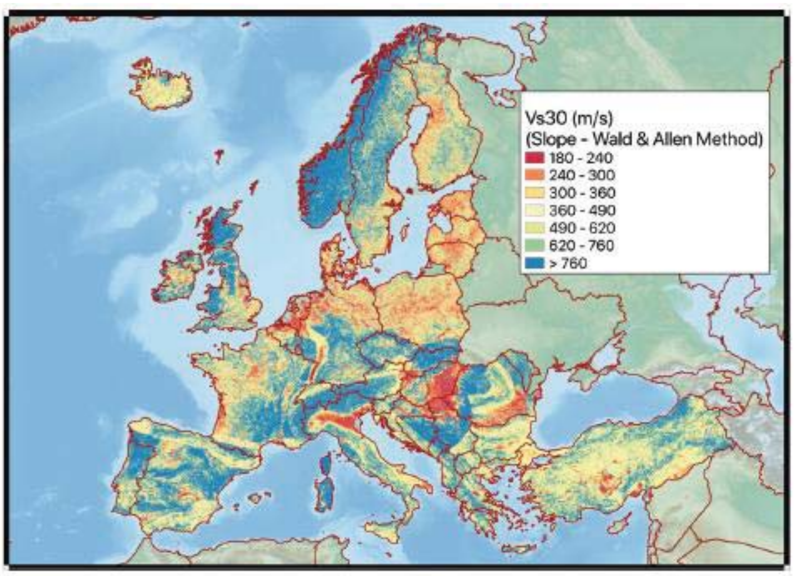
Modelling site conditions at European scale

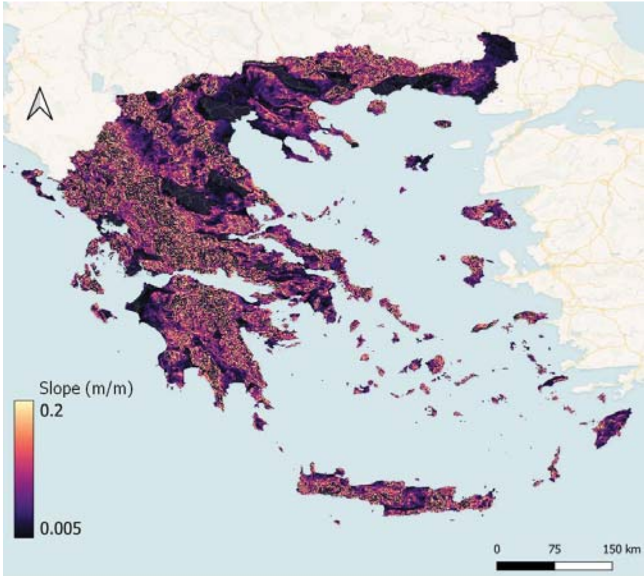


Vs30 inferred from GEBCO topography/bathymetry using the **Wald and Allen (2007**) correlation approach (*Weatherill et al., 2020)*

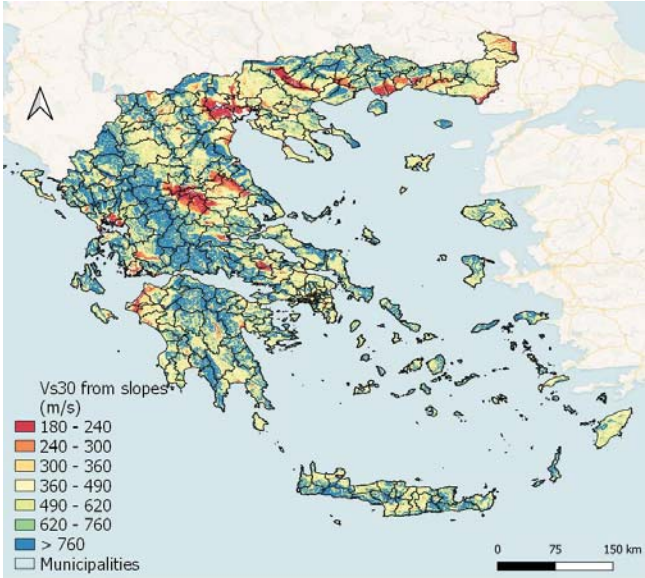
61

Local site conditions

Topographical slope (ESRM20) (*)



Vs30 model from slope



(*) plus geology

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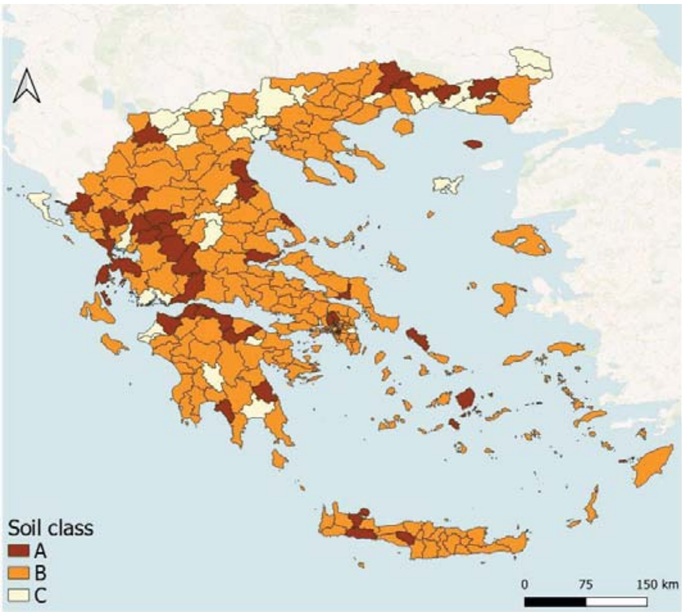
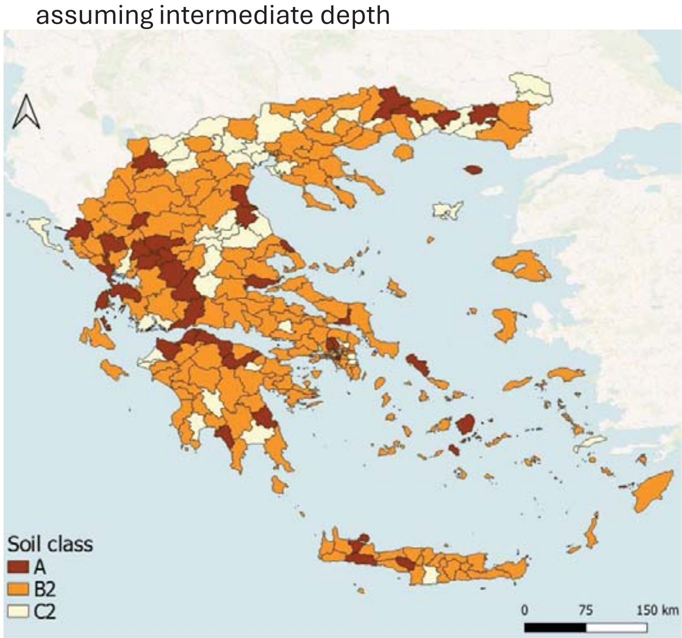
Soil categorization

EAK2003, EC8



NA2023 (Pitilakis et al., 2020)
assuming intermediate depth



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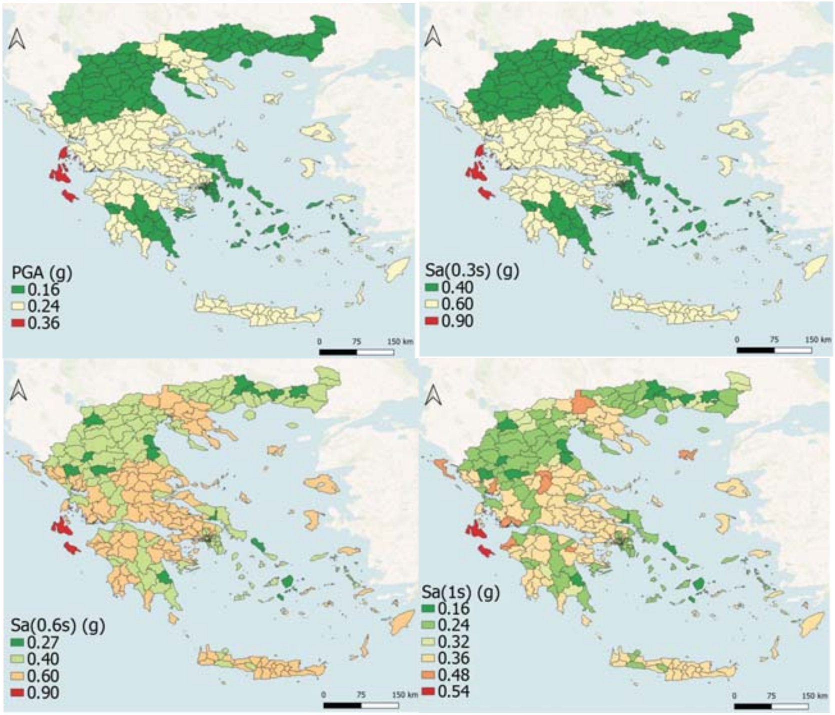
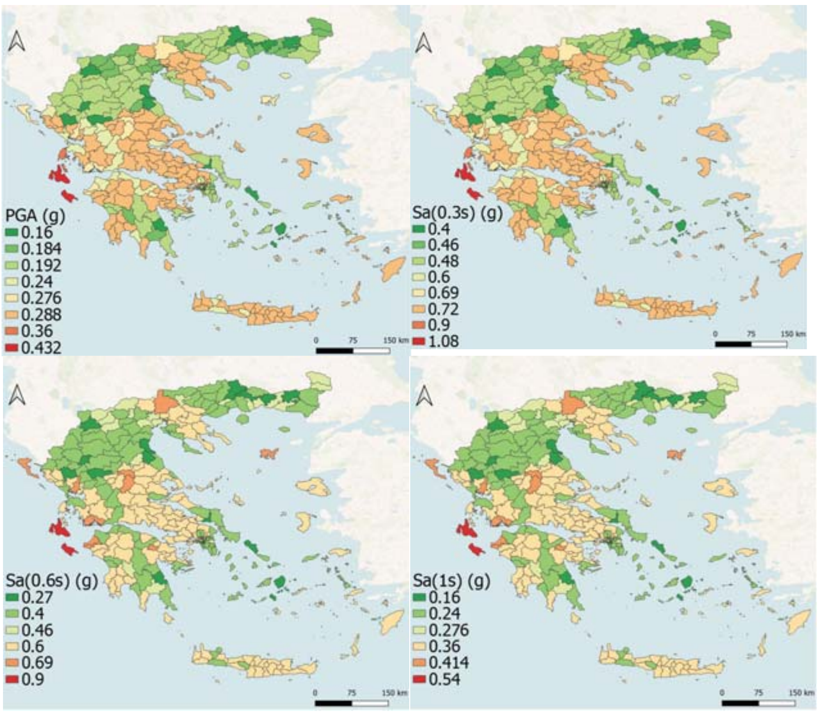
Ground motion fields at surface – EAK2003 (RP = 475y)

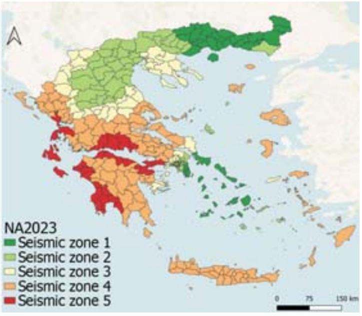


No extra site amplification factor

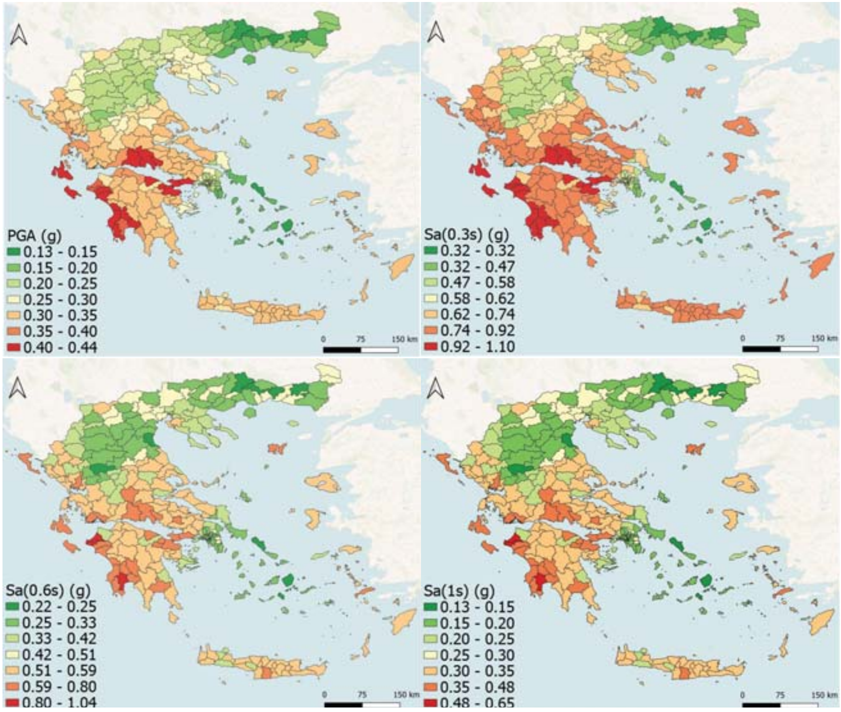
Ground motion fields at surface – EC8 (RP = 475y)



Ground motion fields – NA2023 (RP = 475y)



Seismic zone	area (km2)
1	28,735
2	47,174
3	34,827
4	90,208
5	17,703



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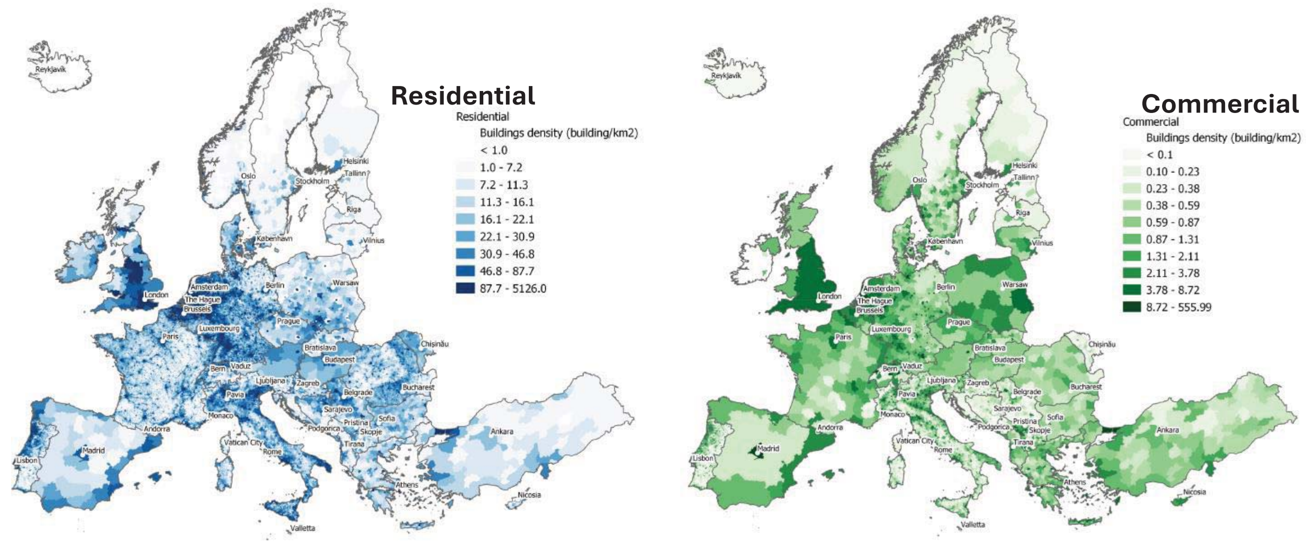
European Seismic Risk Model (ESRM20)

European Exposure Model



Crowley H., Dabbeek J., Despotaki V., Rodrigues D., Martins L., Silva V., Romão, X., Pereira N., Weatherill G. and Danciu L. (2021) European Seismic Risk Model (ESRM20), EFEHR Technical Report 002, V1.0.0, 84 pp, https://doi.org/10.7414/EUC-EFEHR-TR002-ESRM20

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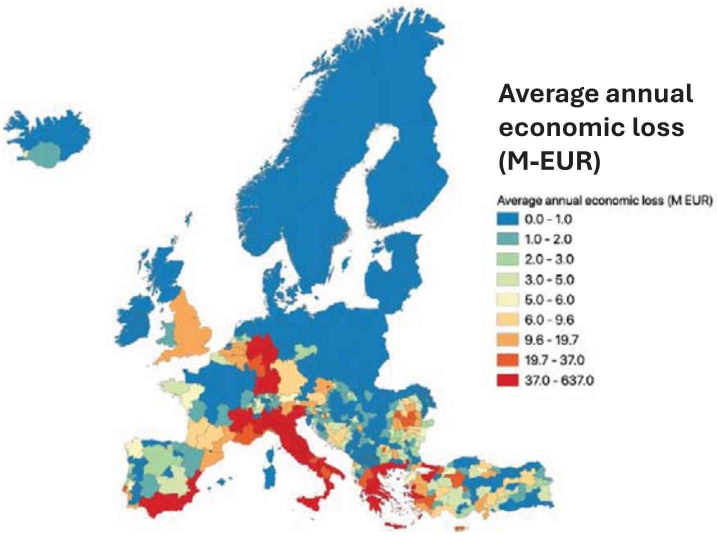
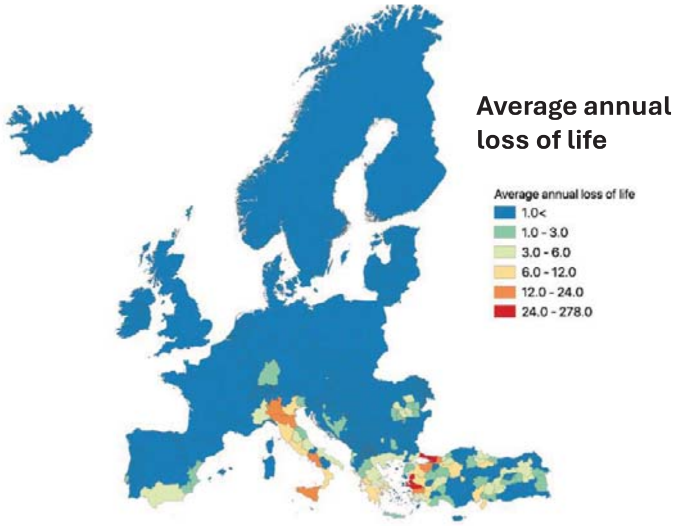
European Seismic Risk Model (ESRM20)

Risk outputs

Average annual economic loss (M-EUR)



Average annual loss of life



Crowley H., Dabbeek J., Despotaki V., Rodrigues D., Martins L., Silva V., Romão, X., Pereira N., Weatherill G. and Danciu L. (2021) European Seismic Risk Model (ESRM20), EFEHR Technical Report 002, V1.0.0, 84 pp, https://doi.org/10.7414/EUC-EFEHR-TR002-ESRM20

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Exposure model for the building stock of Greece

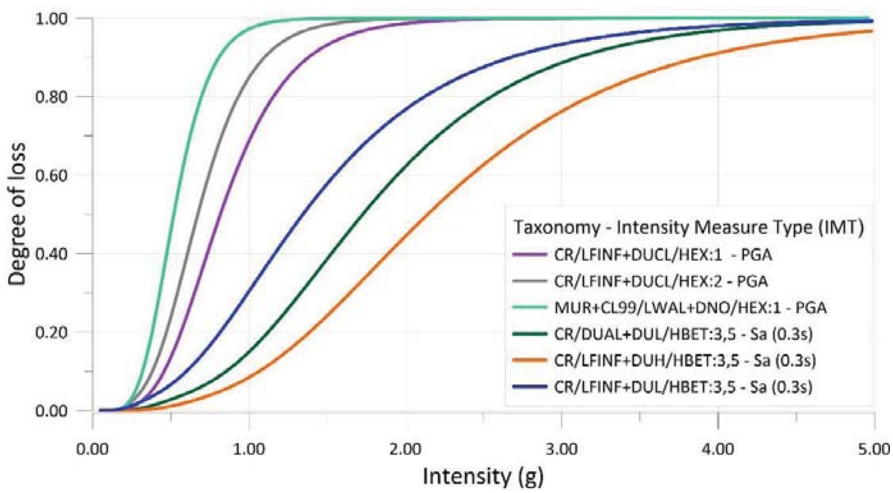
GED4ALL Building Taxonomy scheme (Silva et al., 2022)

Attribute	Element Code	Level 1 Value	Element Code	Level 2 Value
Material	CR	Concrete, reinforced		
	MUR	Masonry, unreinforced	CL99	Fired clay unit, unknown type
			CB99	Concrete blocks, unknown type
			STDRE	Stone
	S	Steel		
	W	Wood		
Lateral load-resisting system (LLRS)	LFM	Moment frame		
	LFINF	Infilled frame		
	LWAL	Walls and frames where the walls, due to their substantial lengths, resist the vast majority of the lateral load		
	LDUAL	Moment frames and shear walls acting together to resist seismic effects		
Ductility Level – Seismic Code Level	DNO or CDN	Non-ductile (Period of construction: before 1959)		
	DUCL or CDL	Ductile, low (Period of construction: 1960-1985)		
	DUCM or CDM	Ductile, medium (Period of construction: 1986-1995)		
	DUCH or CDH	Ductile, high (Period of construction: 1996-present)		
Height	H	Exact number of storeys above ground		
Lateral Force Coefficient	Number expressed in %	The value of the lateral force coefficient, i.e. the fraction of the weight that was specified as the design lateral force in the seismic design code (Applied to reinforced concrete moment and infilled frames only)		

Vulnerability curves

Loss ratio and economic losses

Vulnerability curves by Martins and Silva (2020)



Important source of uncertainties!

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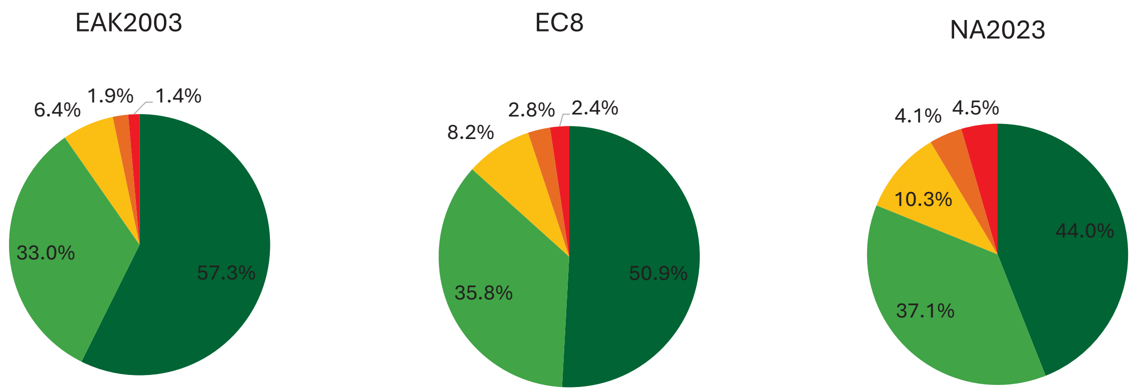
Earthquake & Geotechnical Engineering

March 27th, 2025 • Romania - Greece Seminar

Estimated damages - RP = 475y

Results of the scenario damage analyses - RP = 475y



Number of buildings in each Damage State

Seismic hazard	No damage	Slight	Moderate	Extensive	Complete
EAK2003	1,833,460	1,053,690	204,615	61,009	44,557
EC8	1,626,720	1,144,510	262,992	87,947	75,161
NA2023	1,406,820	1,186,690	330,028	130,535	143,263

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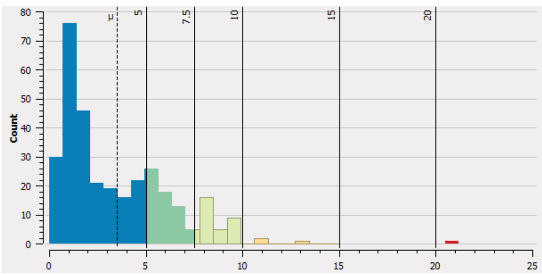
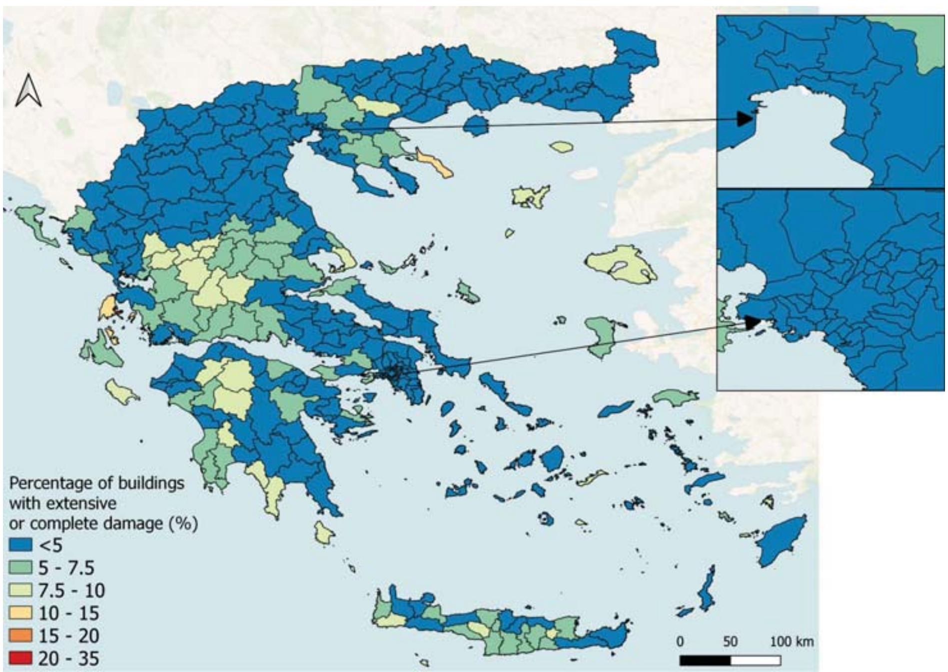
Results of the scenario damage analysis with EAK2003 - RP = 475y

Percentage of buildings with extensive or complete damage (%)



Athens → 1.5%
Thessaloniki → 1.6%



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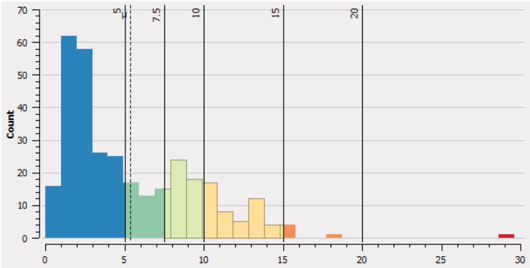
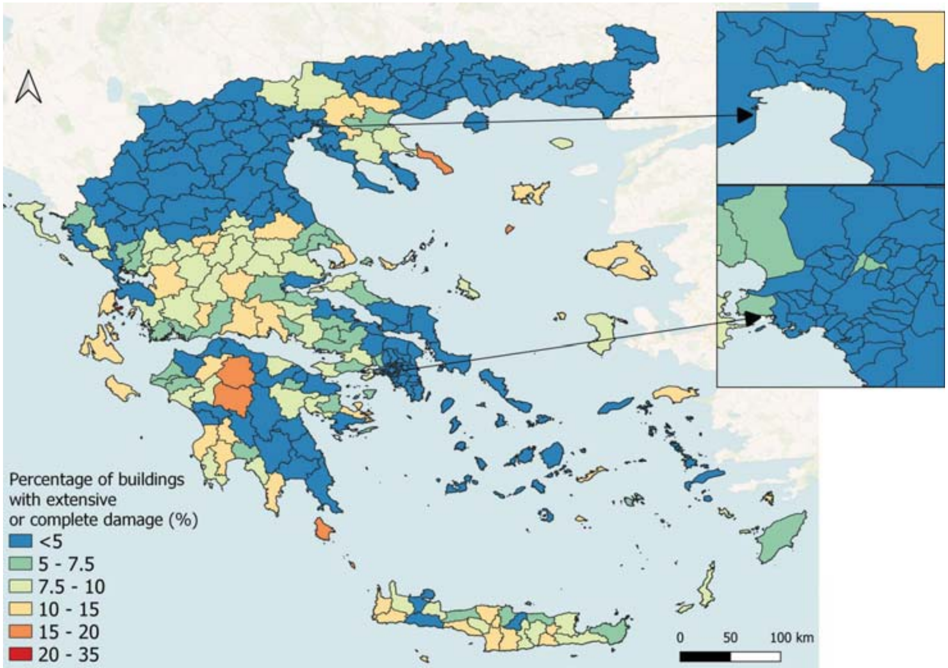
Results of the scenario damage analysis with EC8 - RP = 475y

Percentage of buildings with extensive or complete damage (%)



Athens → 1.8%
Thessaloniki → 1.8%



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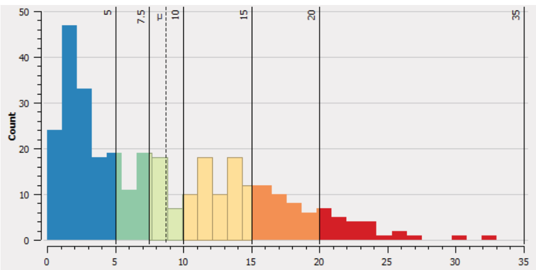
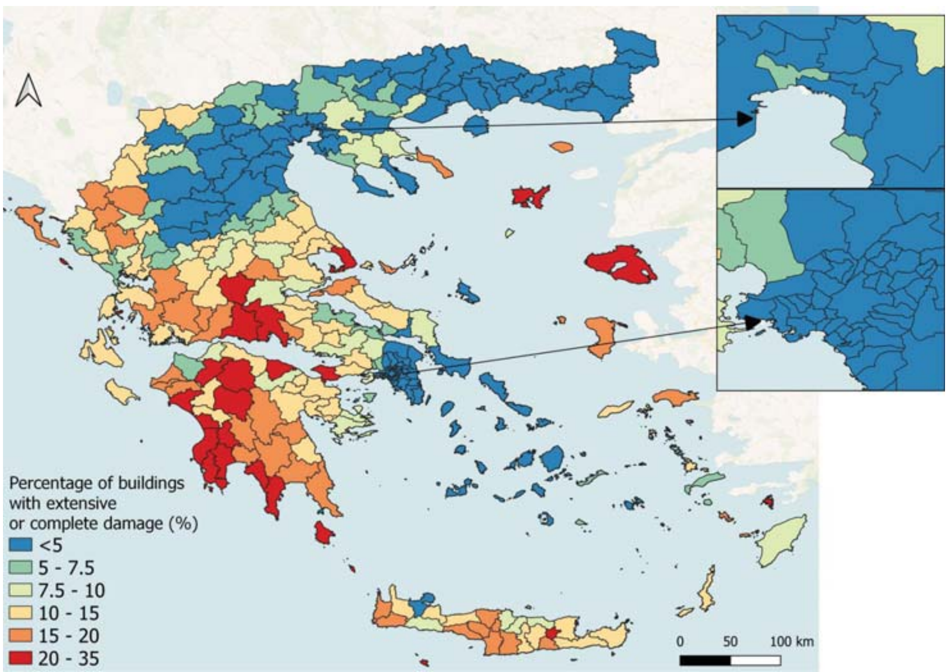
Results of the scenario damage analysis with NA2023 - RP = 475y

Percentage of buildings with extensive or complete damage (%)



Athens → 1.6%
Thessaloniki → 2.7%



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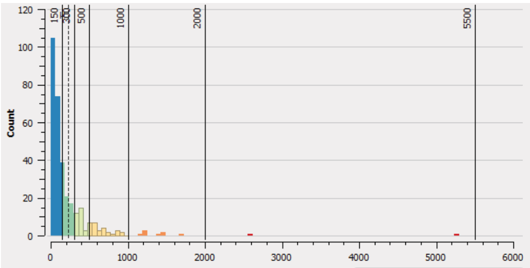
Economic losses and loss ratio – RP = 475y

Results of the scenario risk analyses - RP = 475y

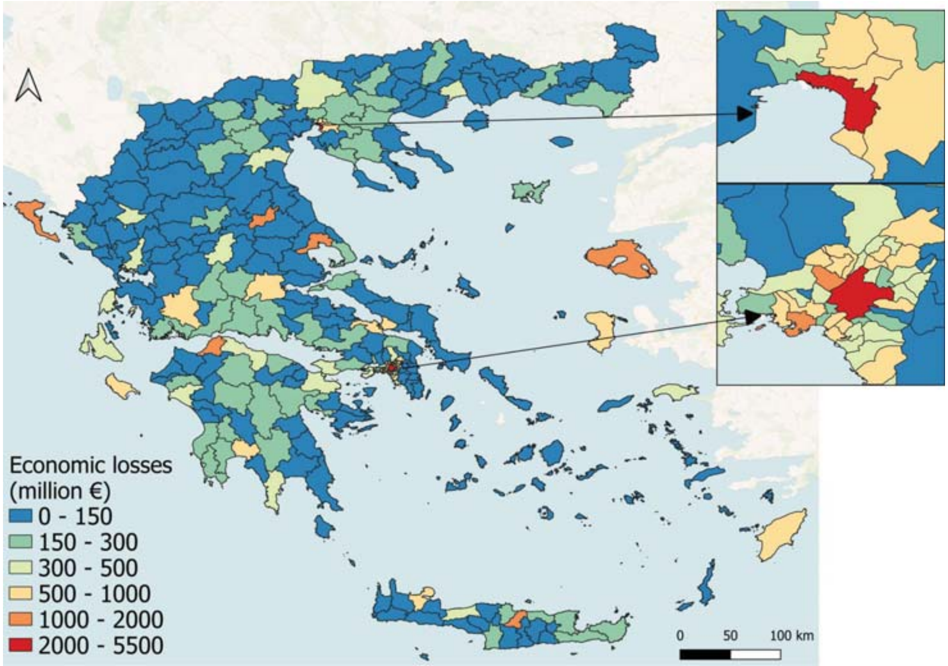
Seismic hazard	Economic losses	Loss ratio
EAK2003	73.19 billion €	0.0582
EC8	88.64 billion €	0.0704
NA2023	108.45 billion €	0.0862

Economic losses from the scenario risk analysis with EAK2003 - RP = 475y

Economic losses (million €):



Athens → 5295 million €
Thessaloniki → 2564.3 million €



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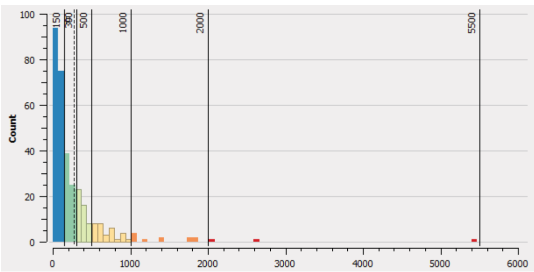
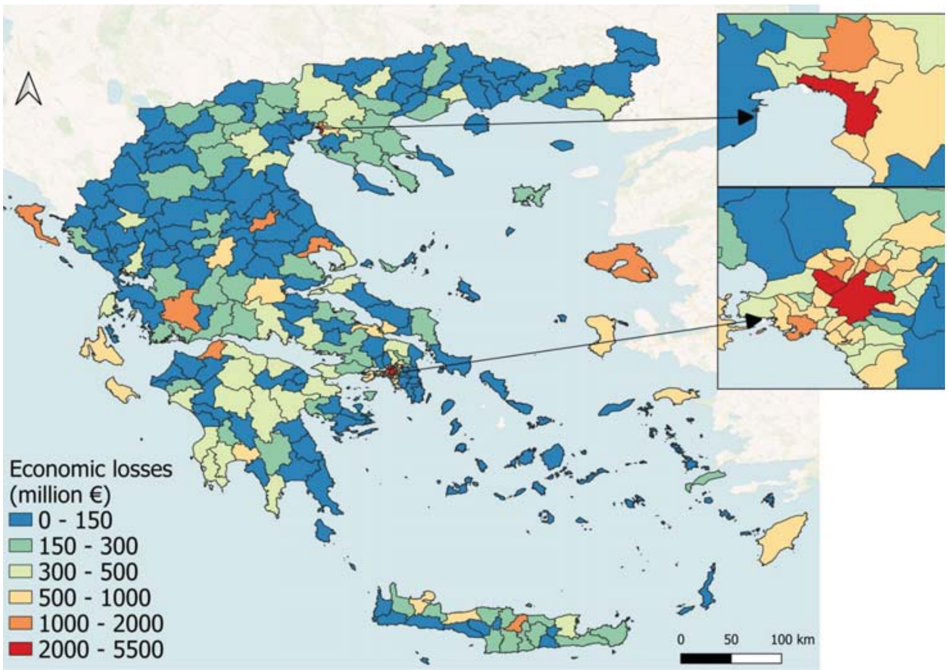
Economic losses from the scenario risk analysis with EC8 - RP = 475y

Economic losses (million €):



Athens → 5473.1 million €
Thessaloniki → 2623.3 million €



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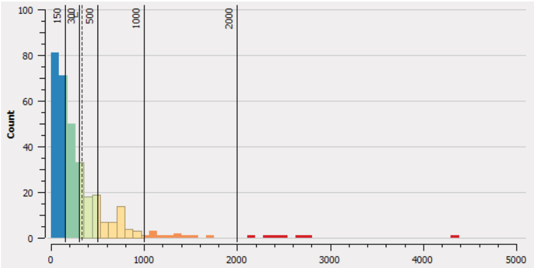
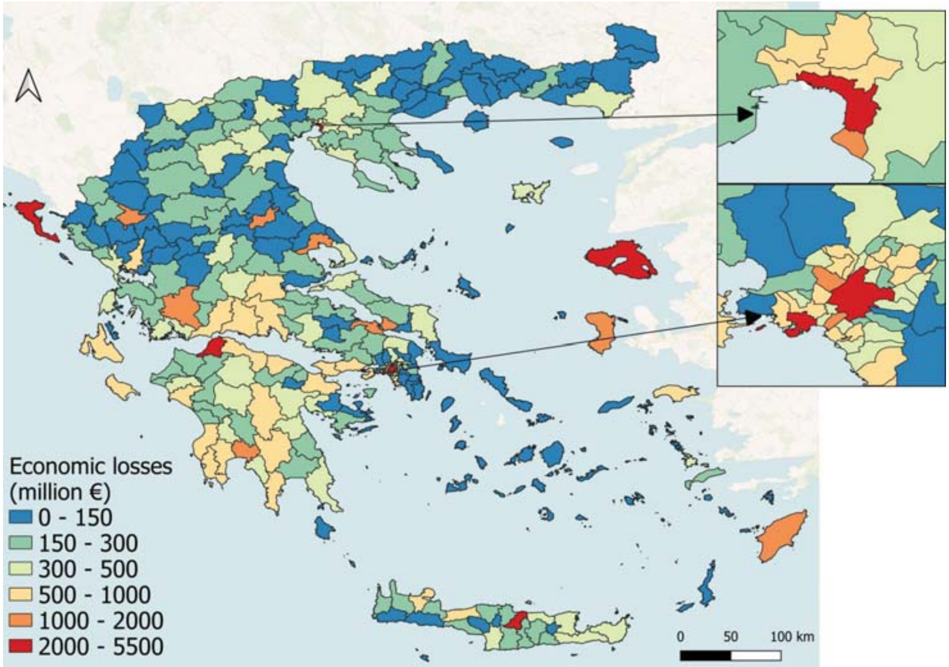
Economic losses from the scenario risk analysis with NA2023 - RP = 475y

Economic losses (million €):

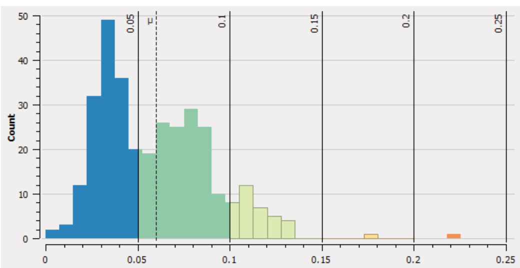


Athens → 4973.1 million €
Thessaloniki → 2724.7 million €

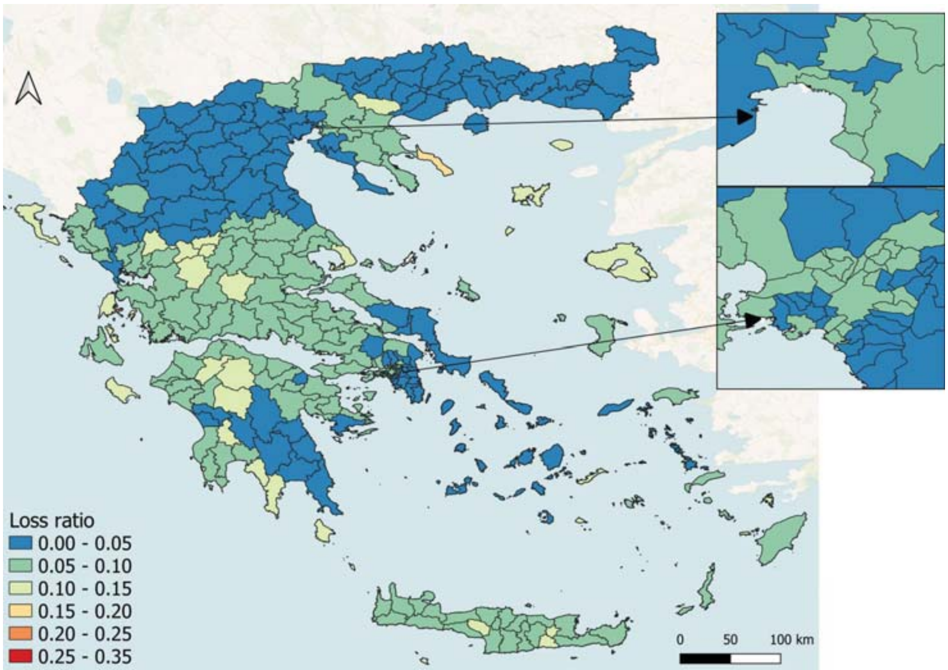


Loss ratio from the scenario risk analysis with EAK2003 - RP = 475y

Loss ratio values:

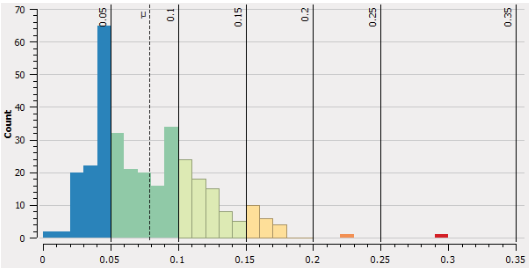


Athens → 0.062
Thessaloniki → 0.062

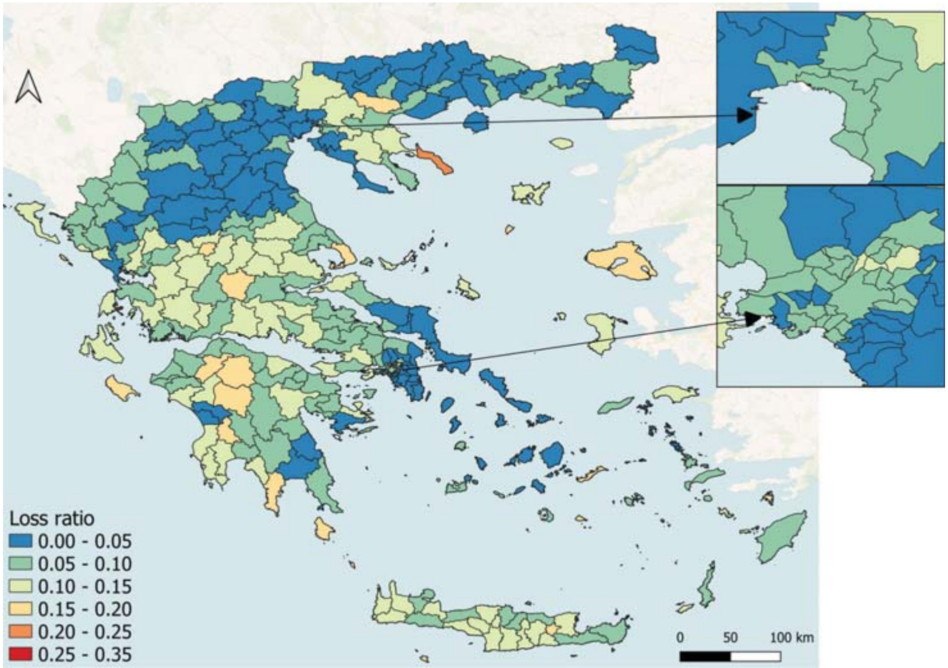


Loss ratio from the scenario risk analysis with EC8 - RP = 475y

Loss ratio values:



Athens → 0.057
Thessaloniki → 0.060



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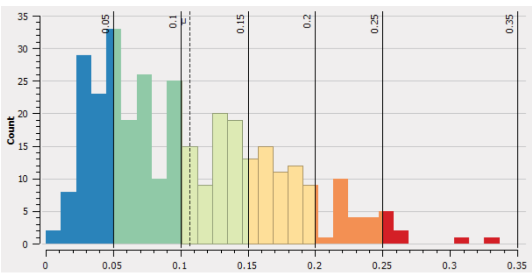
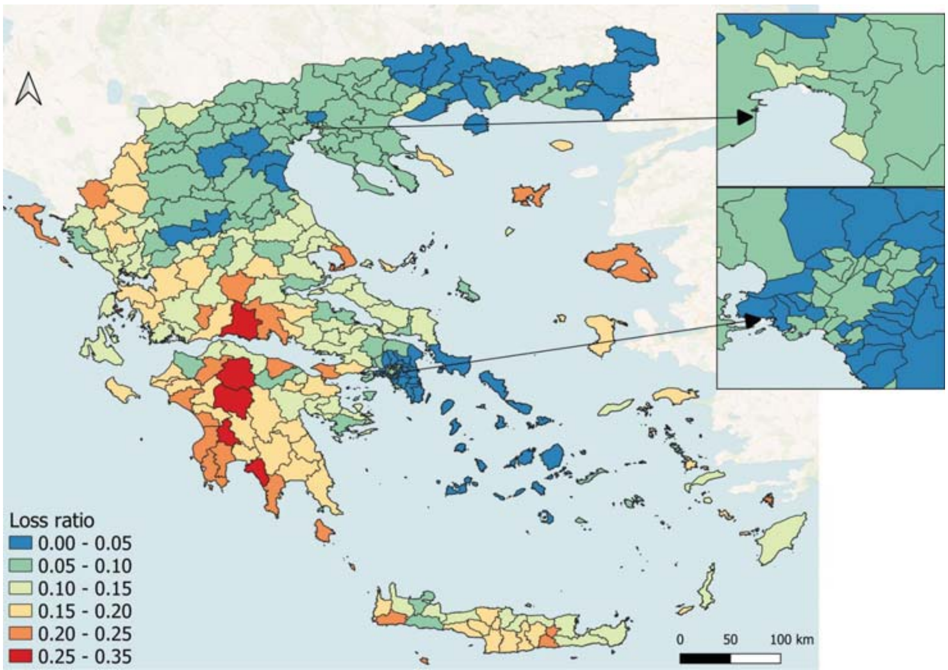
Loss ratio from the scenario risk analysis with NA2023 - RP = 475y

Loss ratio values:



Athens → 0.051
Thessaloniki → 0.066



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Multumesc

Ευχαριστώ

Thank you

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Earthquake & Geotechnical Engineering

March 27th, 2025 • Romania - Greece Seminar

Soil–Structure Interaction

and Foundation Design

in the new EC8-5

George Gazetas

G. Gazetas [http://www.civil.ntua.gr/gazetas/ or http://ssi.civil.ntua.gr/]

A. The mechanics of Soil–Foundation–Structure Interaction (SFSI)

Physics, Methods of analysis, Typical Results

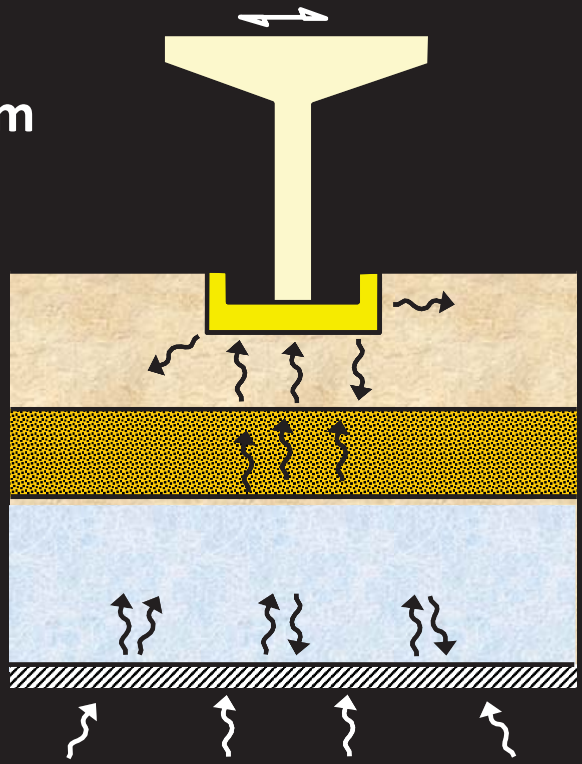
B. SFSI in the Seismic Eurocode (EC8-5): Chapter 8, Annex D

- **Kinematic and Inertia Response**
- **Force-based versus Displacement-based Methods**
- **Rules and Simplifications**

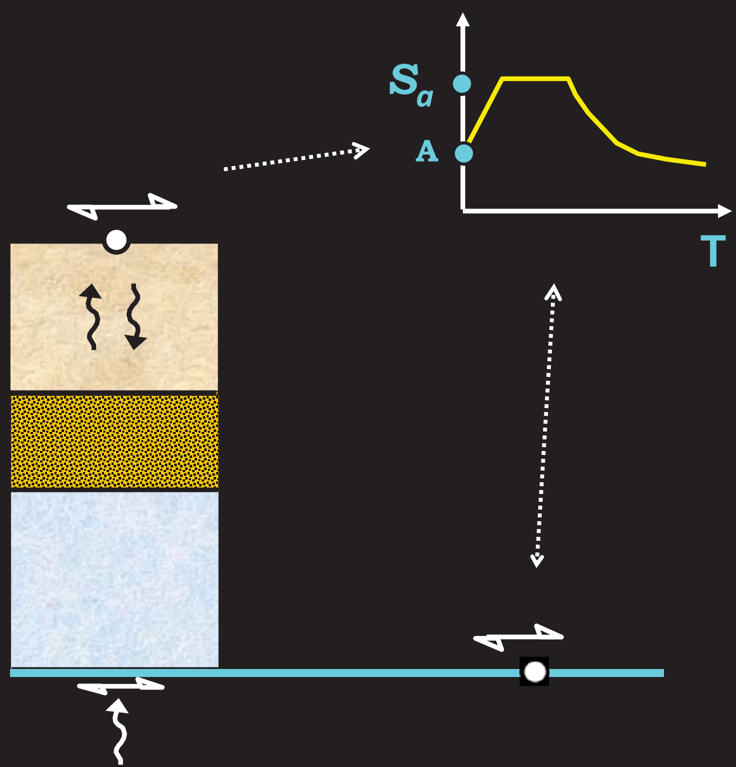
C. Foundation System in Eurocode (EC8-5): Chapter 9, Annex E

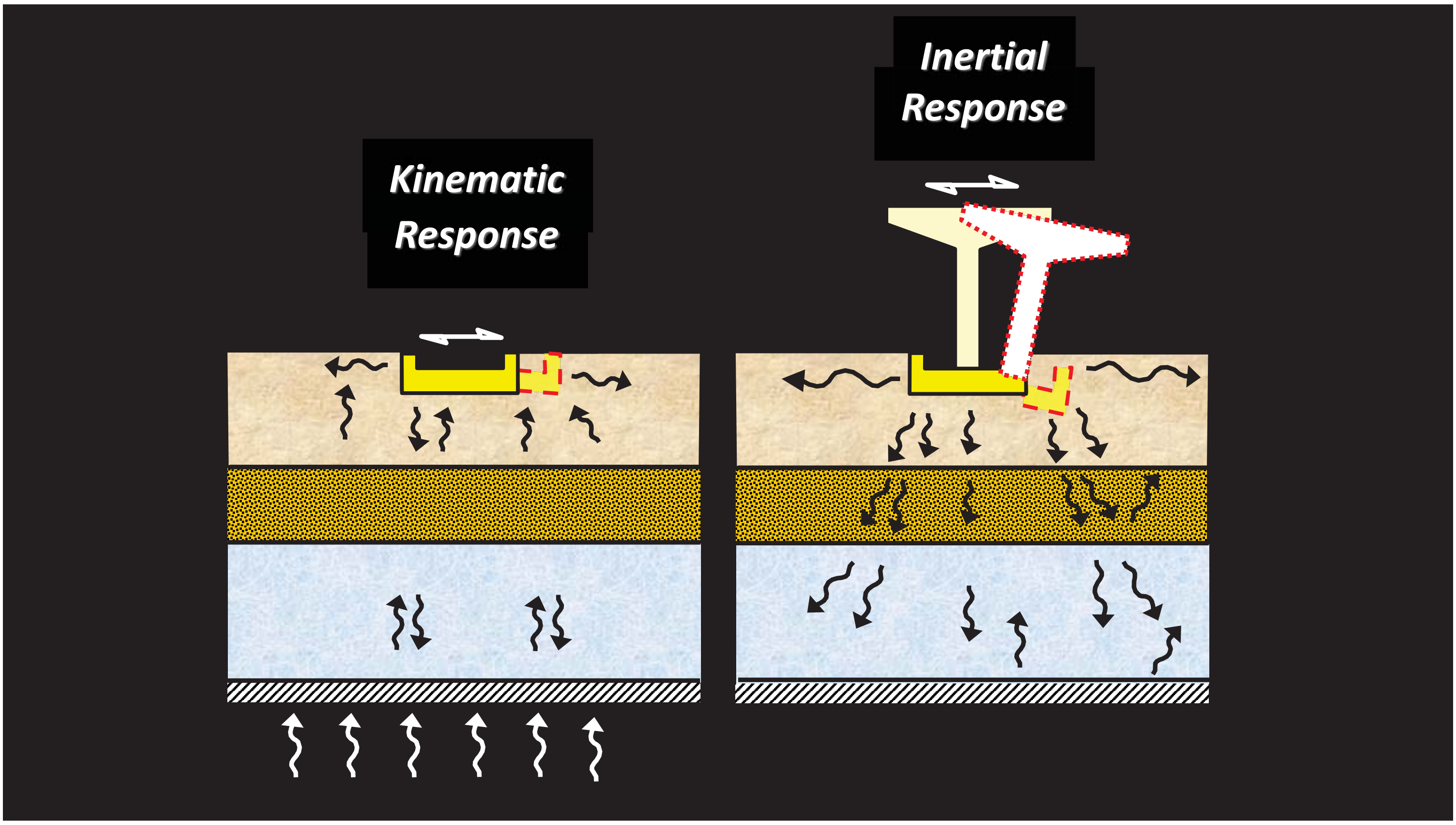
- **Shallow, Embedded, Deep Foundations**

The Seismic Problem



Usual Definition of the Design Motion



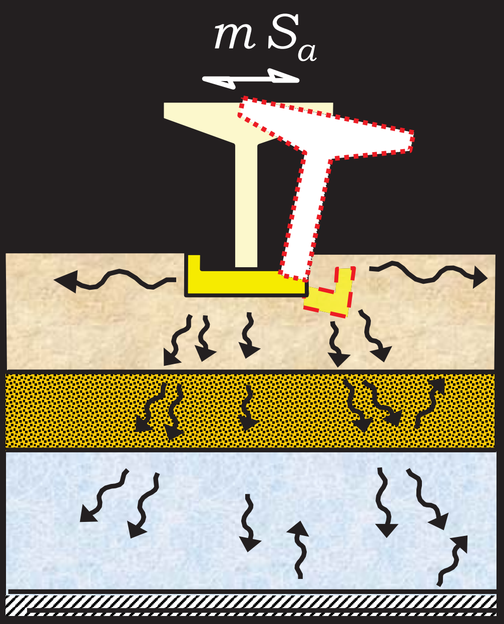


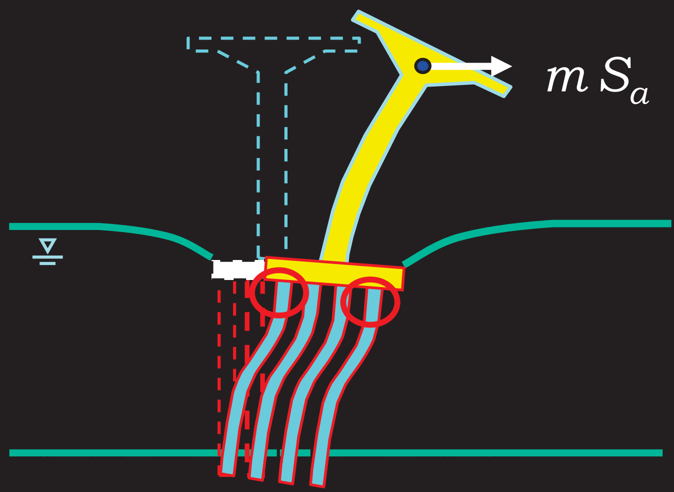
INERTIAL RESPONSE

Common (Conventional) Approach:

Inertial Forces and Soil–Foundation Compliance

Inertial Response





Inertial Response Effects

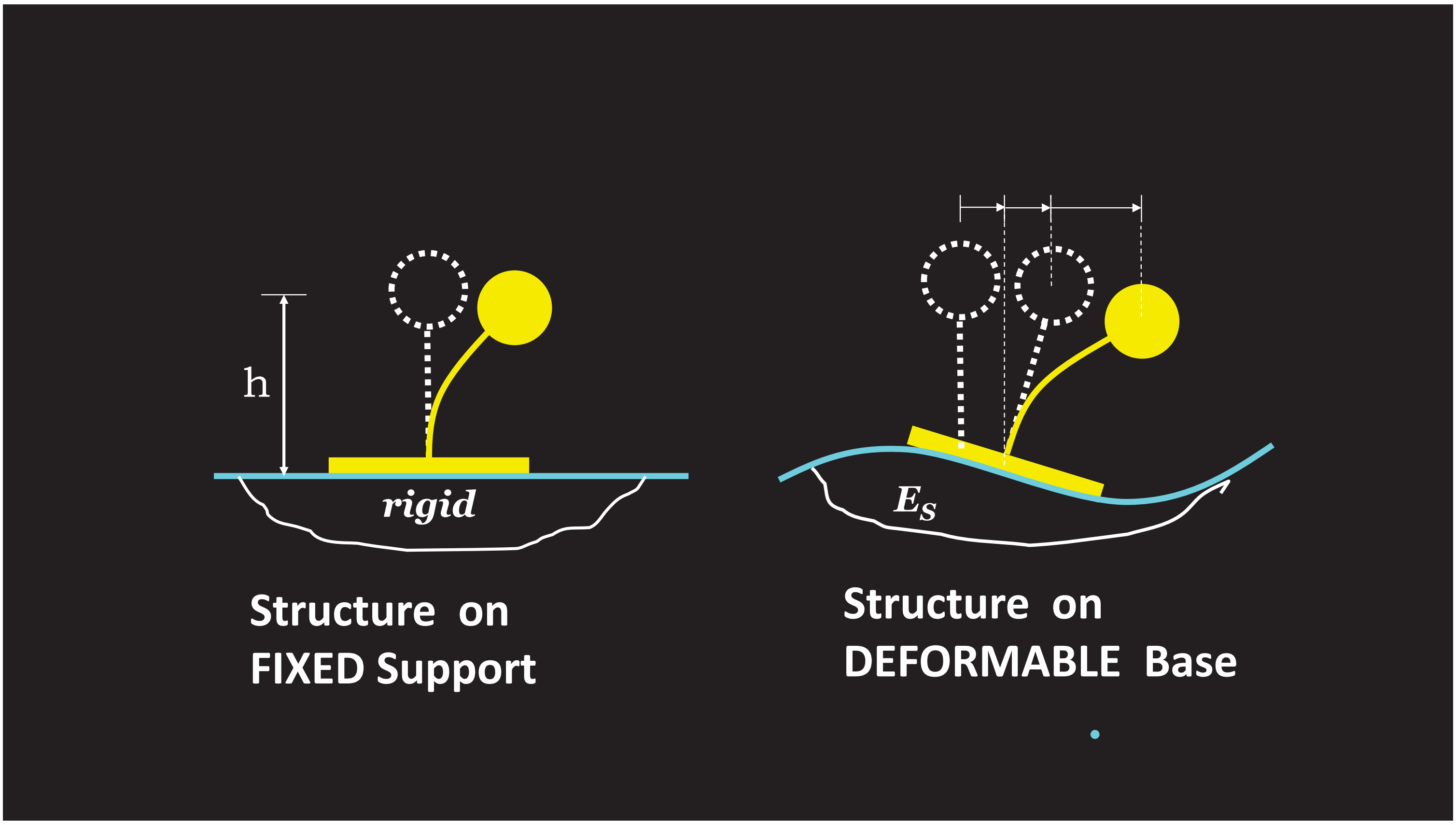
- Inertia forces induce:

  Shear Force, *V*, and Overturning Moment, *M*, onto the foundation

- *V and M cause additional horizontal and vertical stresses*

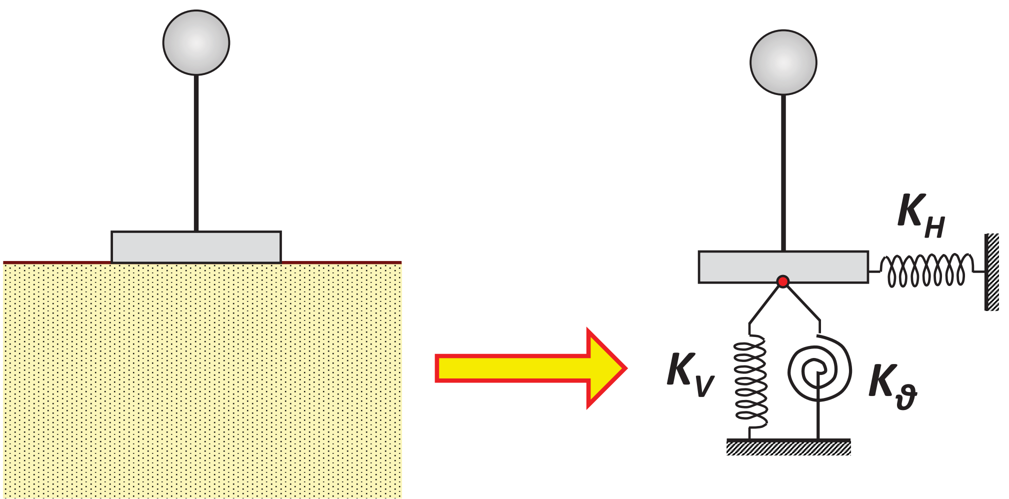
  in the soil due to its "flexibility" →

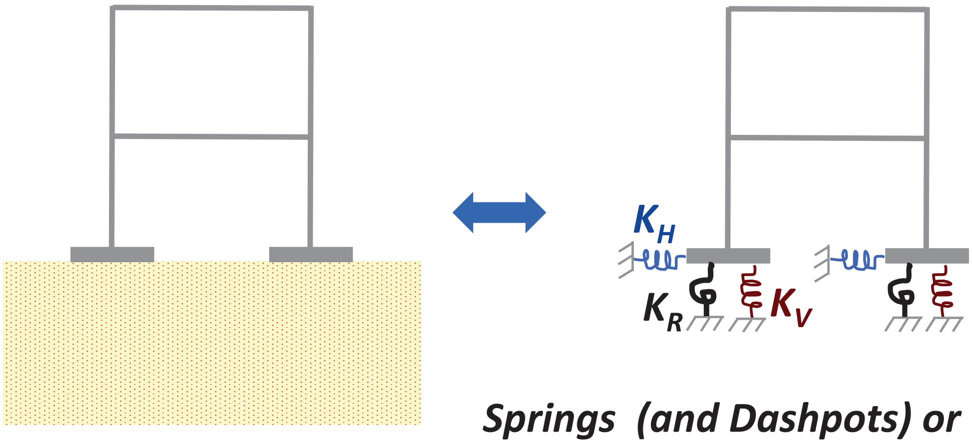
  lateral displacement, rotation



To analyse INERTIAL SFS Interaction:

Replace the soil with springs K_V K_H K_ϑ . . .

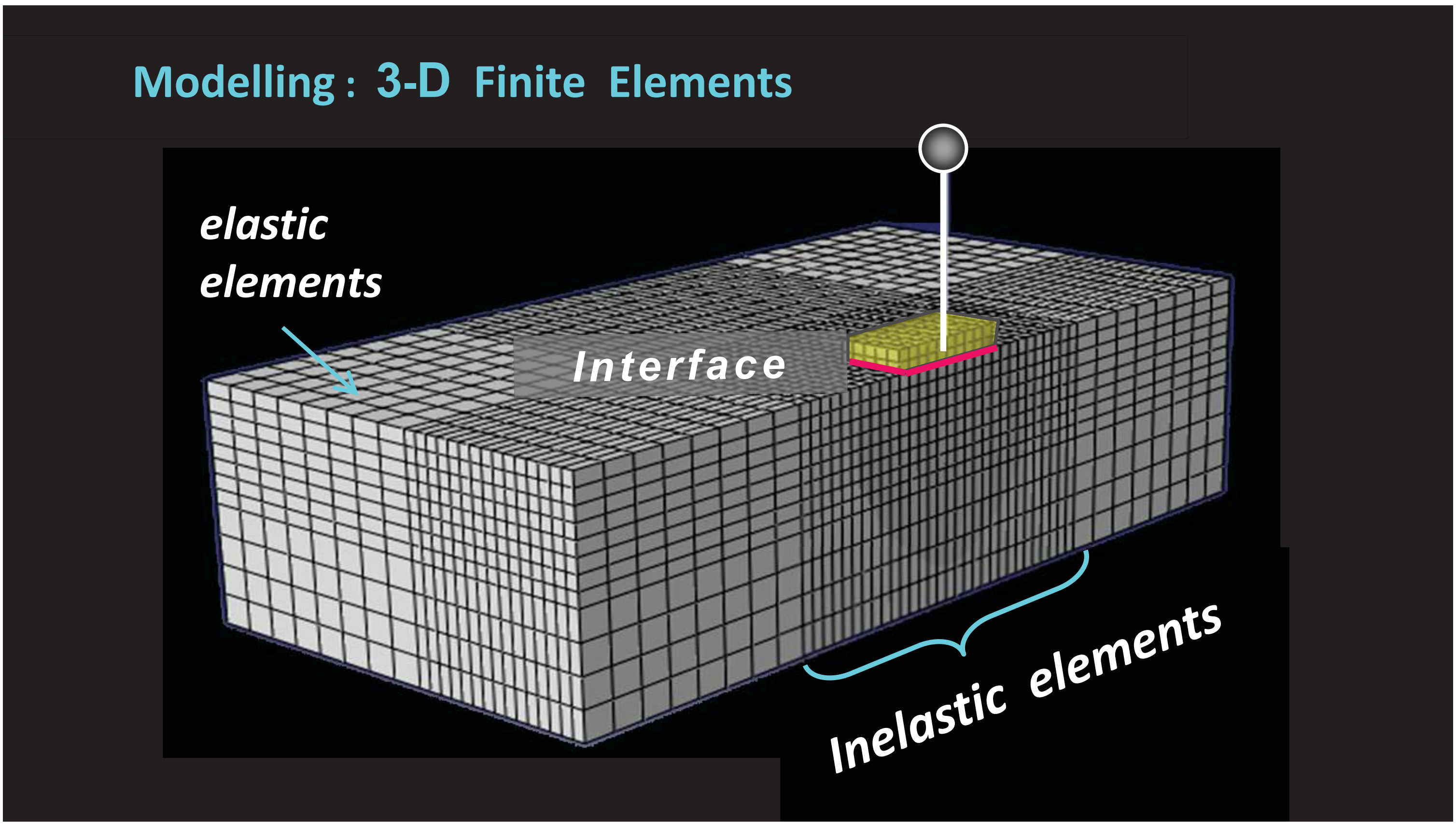


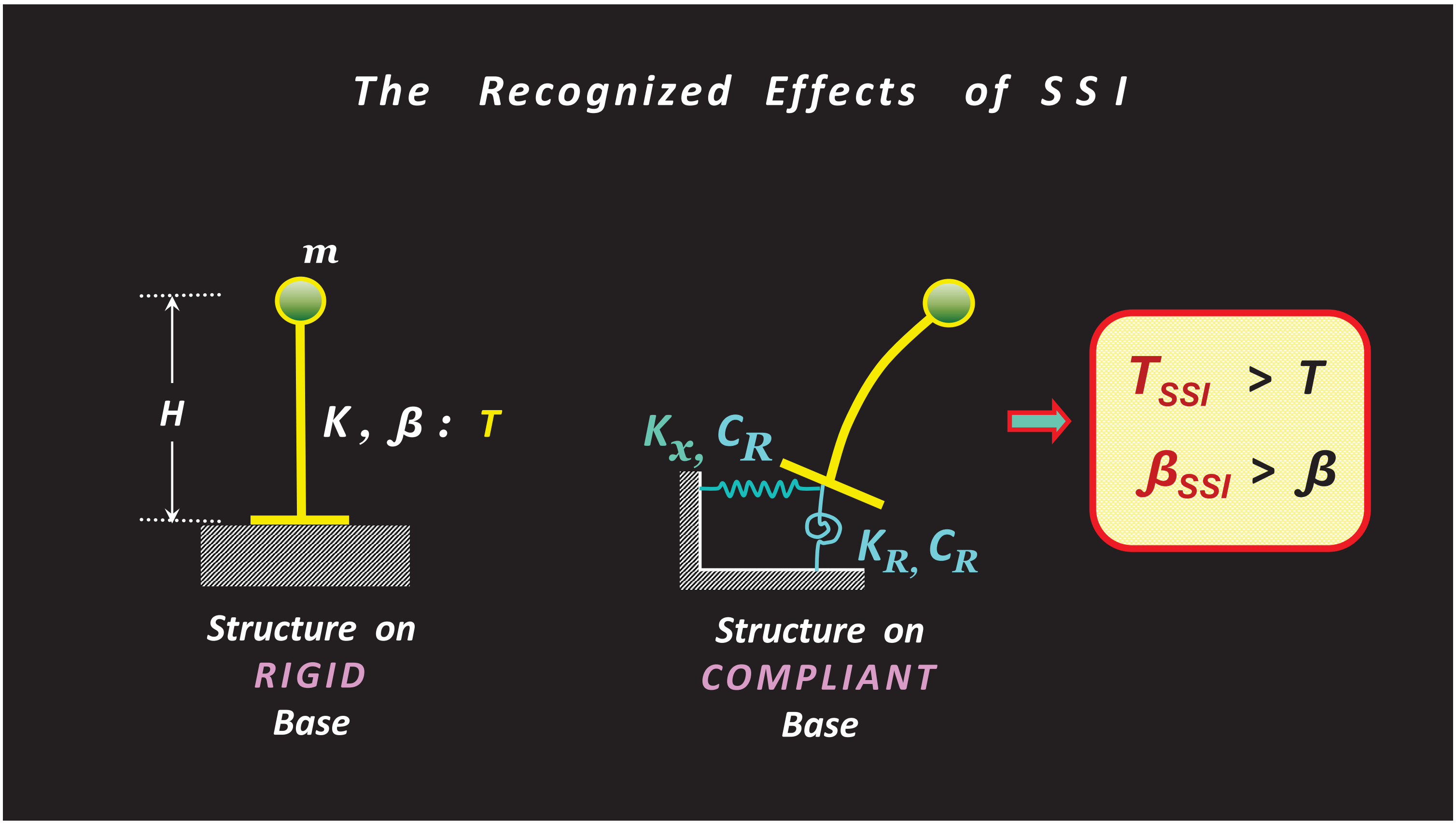


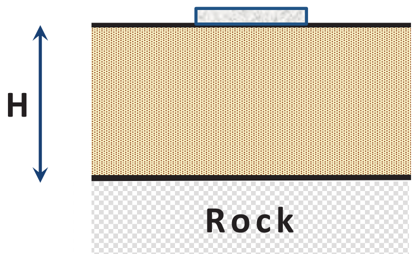
Springs (and Dashpots) or every footing

Spring Stiffness = Static Stiffness (K) · Dynamic Stiffness coefficient (k)

Dashpot Modulus (C) = Radiation "+" Material Damping







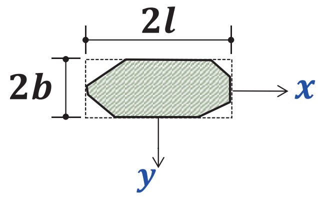
	CIRCLE	STRIP
K_v	$\approx \dfrac{4GR}{1-\nu}\left(1+1.3\dfrac{R}{H}\right)$	$\approx \dfrac{0.73}{1-\nu}\left(1+3.5\,\dfrac{b}{H}\right)$
K_h	$\approx \dfrac{8GR}{2-\nu}\left(1+\dfrac{1}{2}\dfrac{R}{H}\right)$	$\approx \dfrac{2G}{2-\nu}\left(1+2\dfrac{b}{H}\right)$
K_r	$\approx \dfrac{8GR^3}{3(1-\nu)}\left(1+\dfrac{1}{6}\dfrac{R}{H}\right)$	$\approx \dfrac{\pi G b^2}{2(1-\nu)}\left(1+\dfrac{1}{5}\dfrac{b}{H}\right)$
K_t	$\approx \dfrac{16}{3}GR^3$	—

Dynamic Stffness =

Static Stiffness, *K*, times a dynamic stiffness coefficient, $k = k(\omega)$

Shallow Foundations of Arbitrary Shape $(2b, 2l, Ab)$ on homogenous Halfspace

TRANSLATIONAL STIFFNESSES

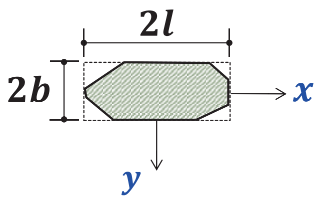


$$\chi = A_b/4l^2 \rightarrow b/l$$

Vertical K_z	$= \frac{2Gl}{1-\nu}\left(0.73 + 1.54\chi^{0.75}\right)$
Lateral K_y	$= \frac{2Gl}{2-\nu}\left(2 + 2.5\chi^{0.85}\right)$
Lateral K_x	$= K_y - \frac{0.2\,Gl}{0.75-\nu}(1 - b/l)$

Shallow Foundations of Arbitrary Shape $(2b, 2l, Ab)$ on homogenous Halfspace

ROTATIONAL STIFFNESSES

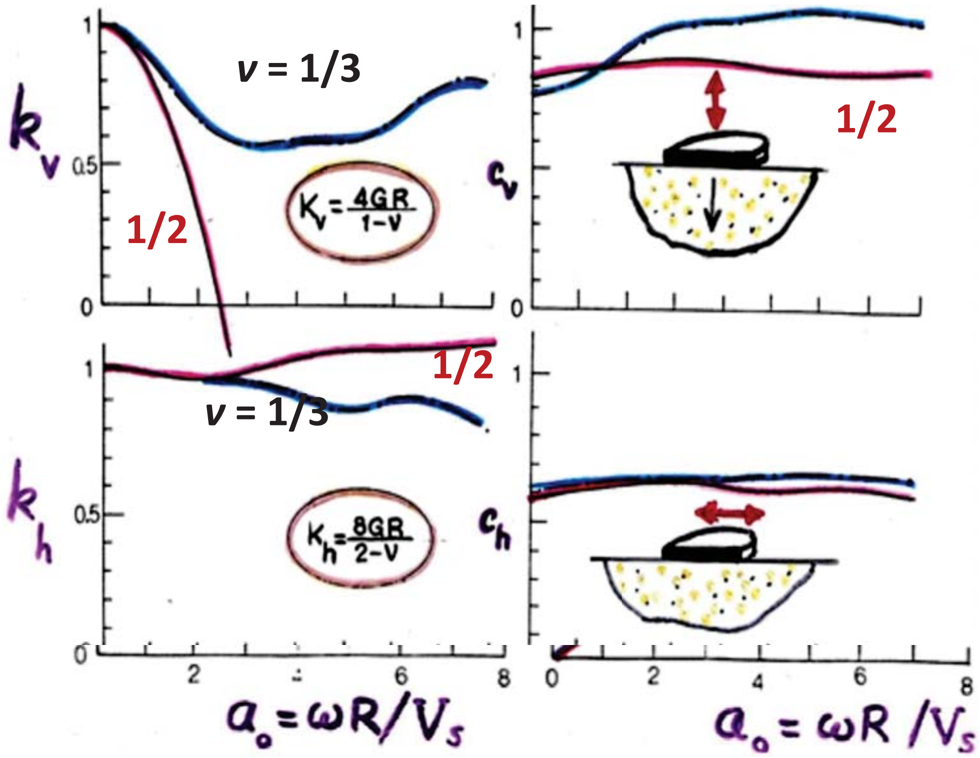


$$\chi = A_b/4l^2 \rightarrow b/l$$

Rocking K_{r_x}	$= \frac{G}{1-\nu} I_{bx}{}^{0.75}\left(\frac{l}{b}\right)^{0.25}\left(2.3 + 0.5\,{}^{b}/_{l}\right)$
Rocking K_{r_y}	$= \frac{G}{1-\nu} I_{bx}{}^{0.75}\left[2.8\left(\frac{l}{b}\right)^{0.25}\right]$
Torsional K_{r_z}	$= G J_t{}^{0.75}\left[4 + 11(1 - b/l)^{10}\right]$

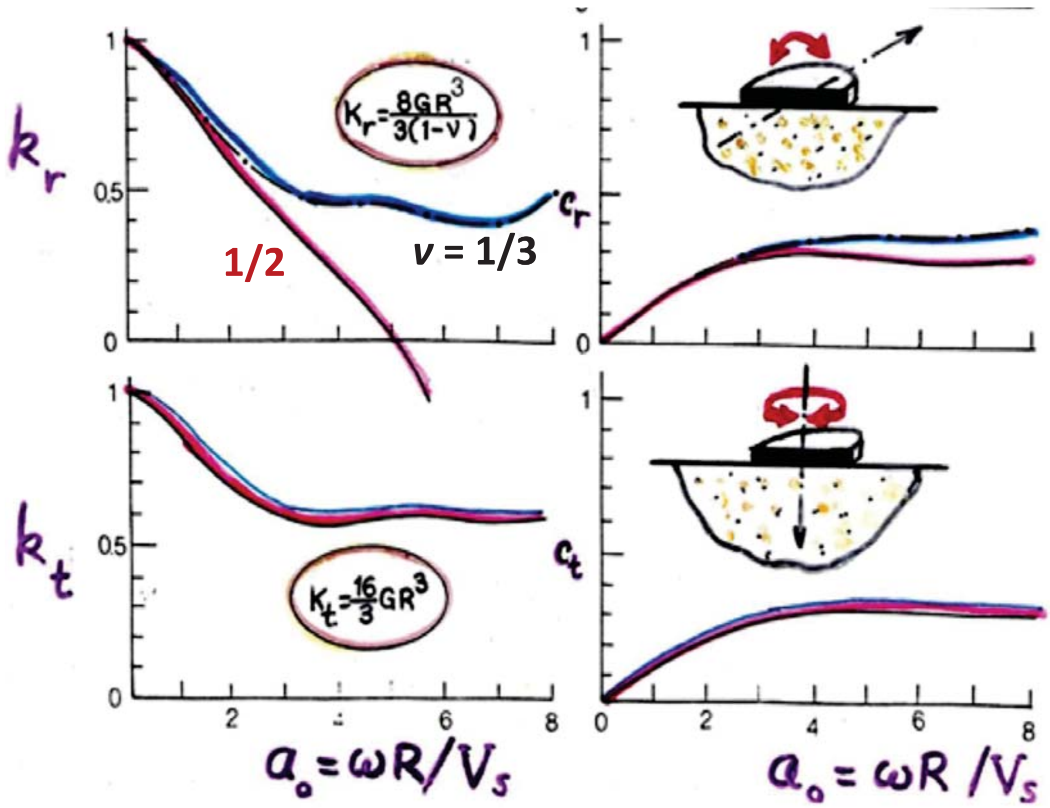
Vertical
Stiffness + Damping

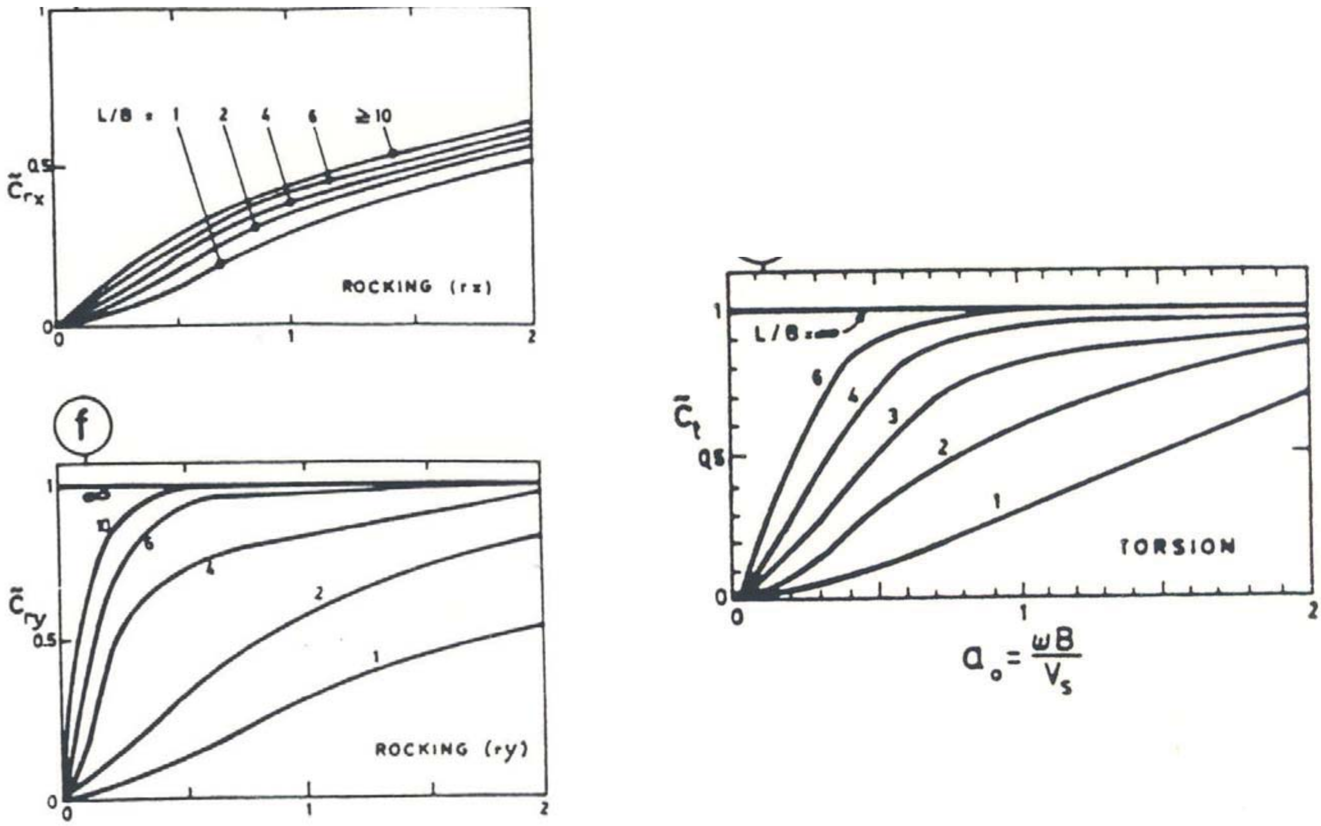
Horizontal
Stiffness + Damping

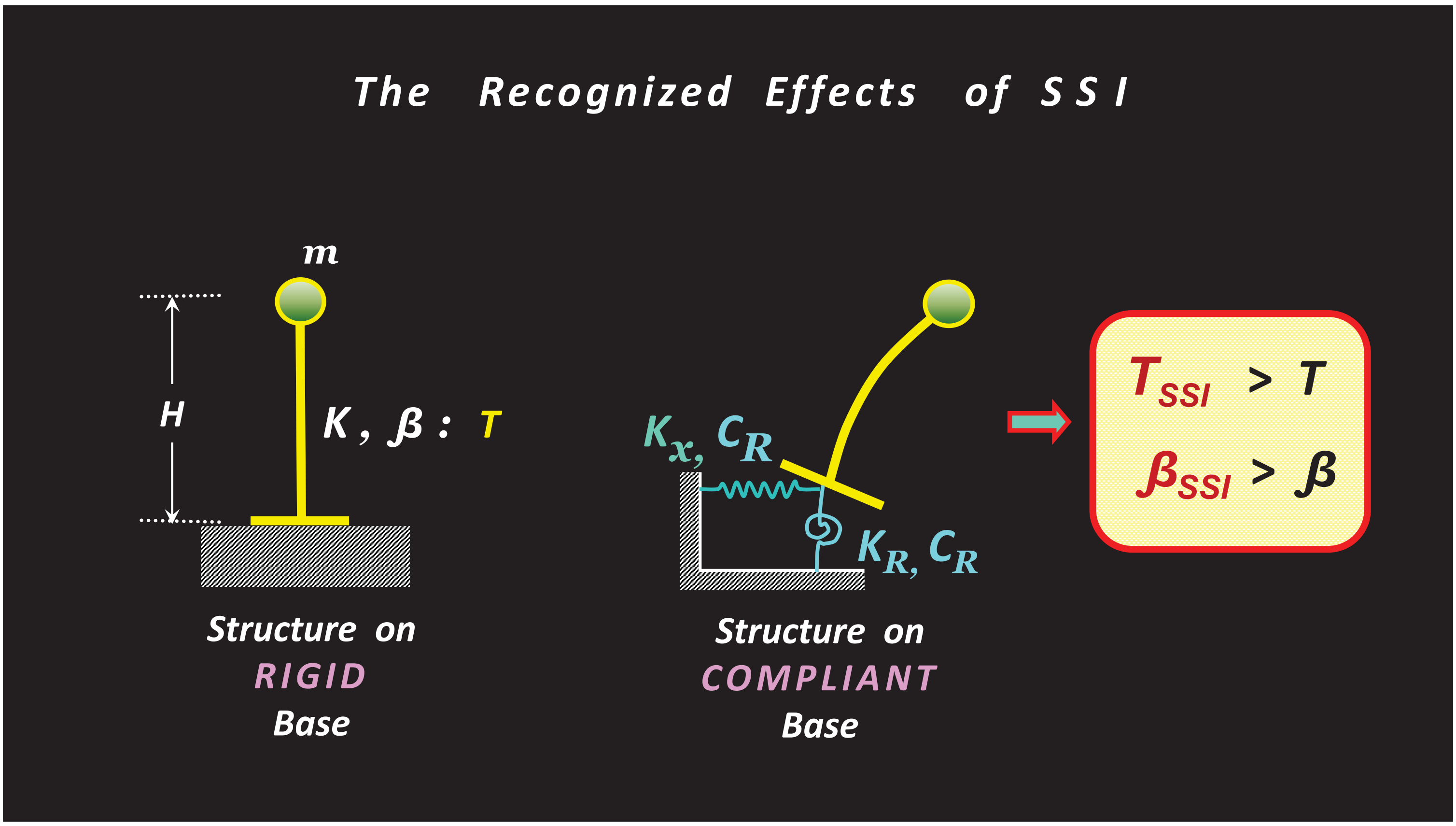


Rocking
Stiffness + Damping

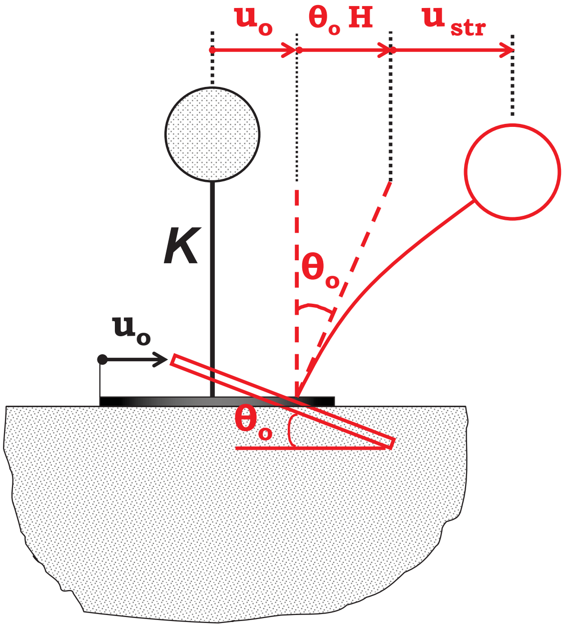
Torsional
Stiffness + Damping



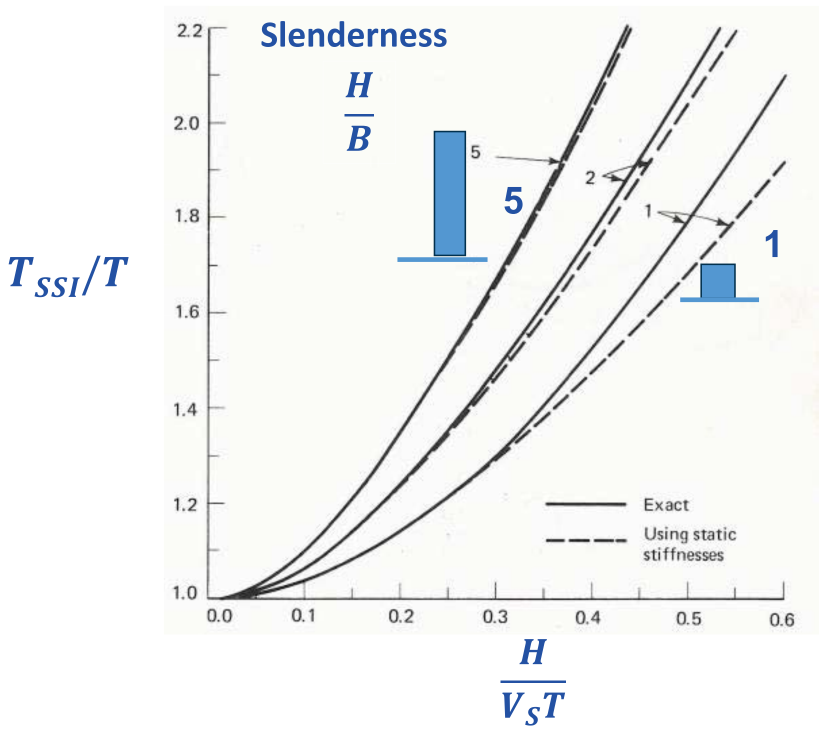

L/B = 1 2 4 6 ≥10
ROCKING (rx)
ROCKING (ry)
TORSION
f
a₀ = ωB/Vs


The Recognized Effects of SSI
m
H
K , β : T
Kx, CR
KR, CR
TSSI > T
βSSI > β
Structure on
RIGID
Base
Structure on
COMPLIANT
Base

$T_{SSI} > T$



$$T_{SSI} = T\sqrt{\left(1 + \frac{K}{K_H} + \frac{K}{K_R}H^2\right)}$$



EFFECTIVE (TOTAL) DAMPING

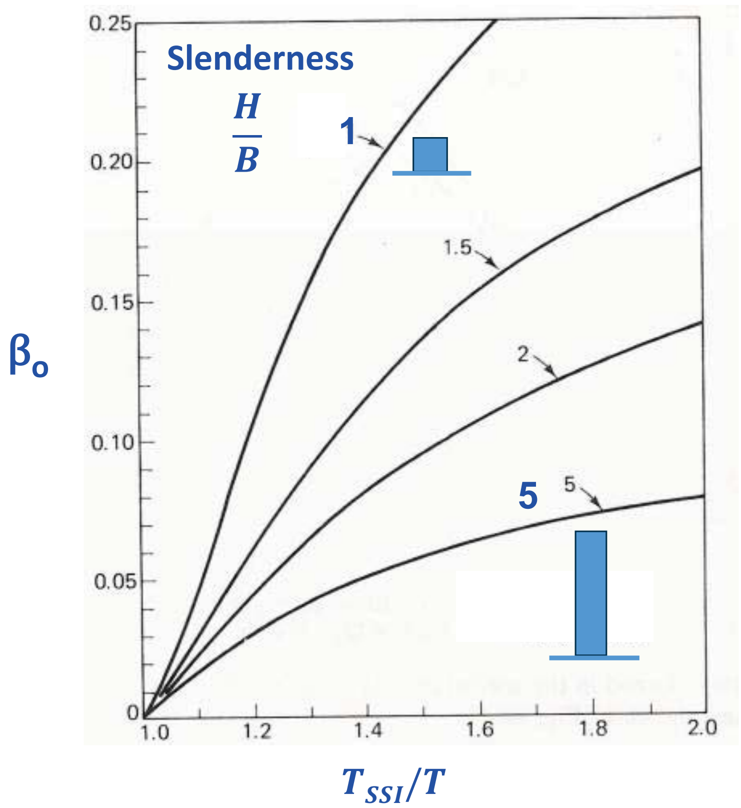
$$\tilde{\xi} = \{\beta_o + \xi\} + \frac{\xi}{(T_{SSI} + T)^3}$$

STRUCTURAL Damping

FOUNDATION Damping

↓

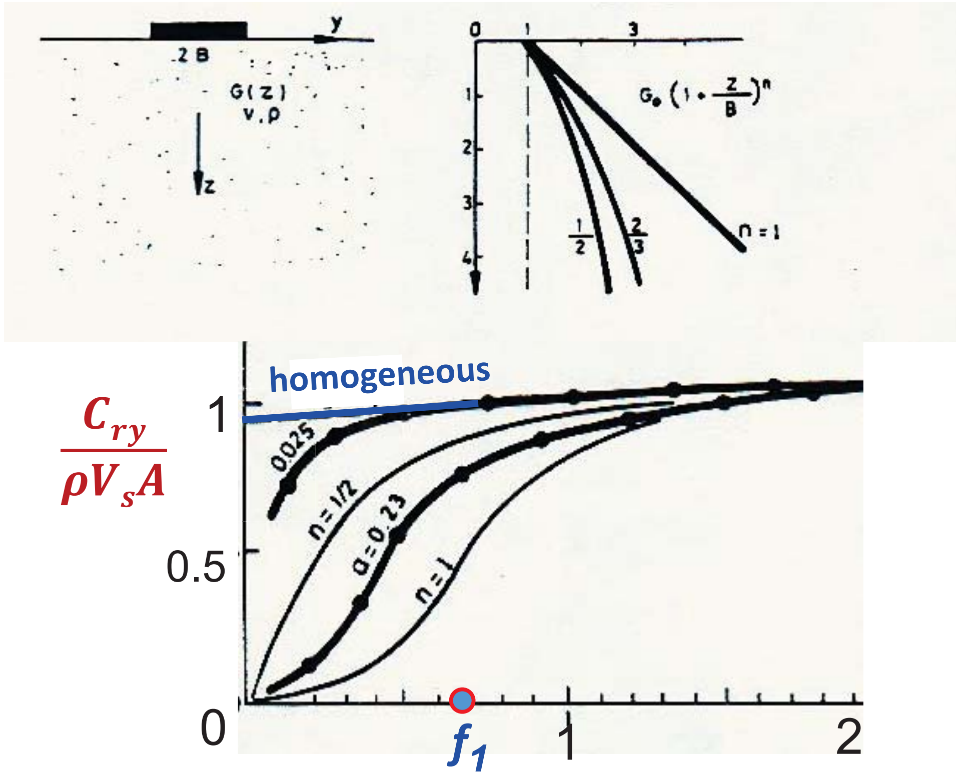
RADIATION (β_o) + INELASTITY $(\xi = \xi(\gamma))$

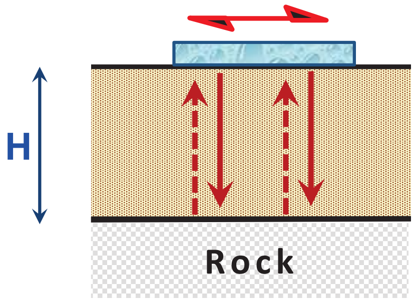


However:

These curves are only for a homogeneous halfspace!

The presence of very stiff layers, inhomogeneity, or rock at shallow depth, severely reduces radiation, and thus β_o is much smaller



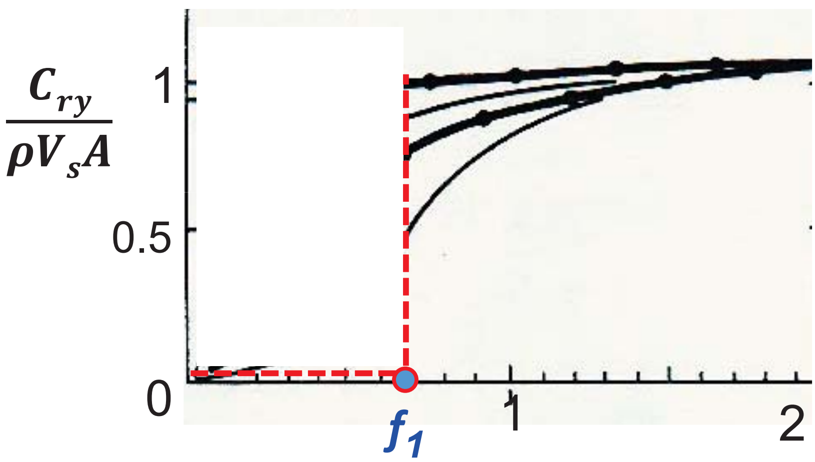


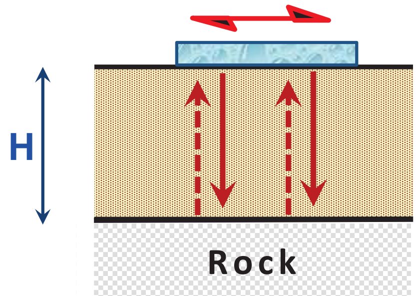
$C_{ry} = 0$ when

$f < f_1 = V_S /4H$

↑

Fundamental frequency in SHEAR





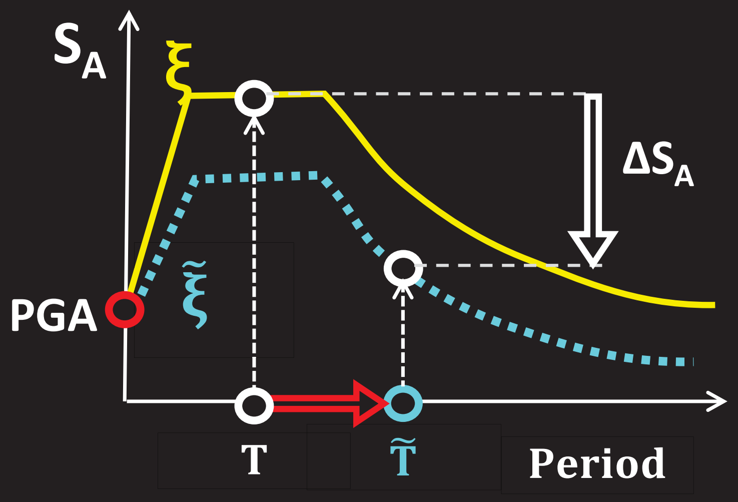
$C_{ry} = 0$ when

$f < f_1 = V_S /4H$

↑

Fundamental frequency in SHEAR

Conventional consequence of SSI



⇨ *Invariably Decreasing Base Shear*

But increasing displacements (~ T^2)

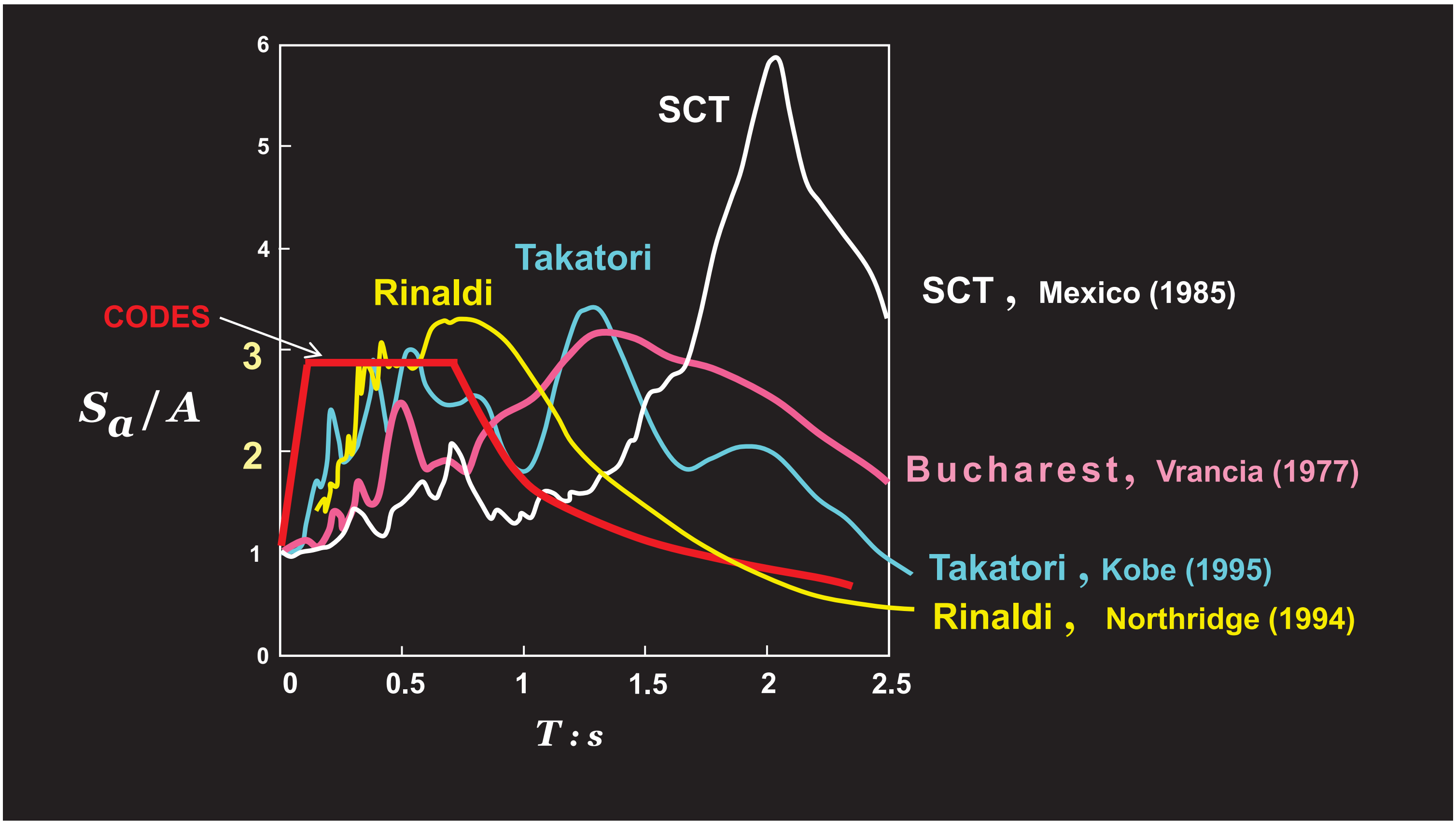
These (CODE) Spectrum SHAPES imply that

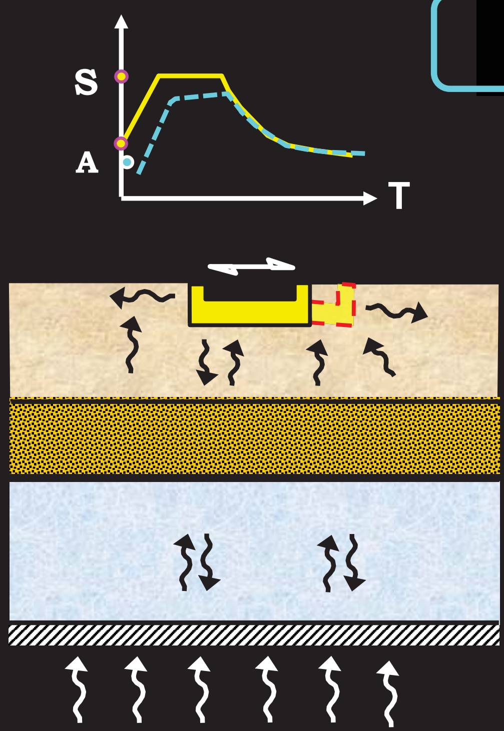
(even on very soft soils)

S S I is always beneficial for the base shear !

Yet , REALITY contradicts this conclusion

(Mexico , Bucharest , Loma Prieta , Kobe , Kocaeli , . .)





Kinematic Response

(1) Modifies the Ground Motion
(excitation of the structure,
Foundation Input Motion — FIM)

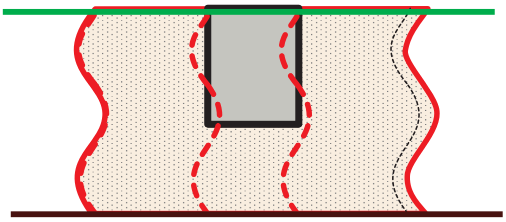
(2) Imposes Deformations & Stresses
in/on the foundation elements
(piles, basement walls)

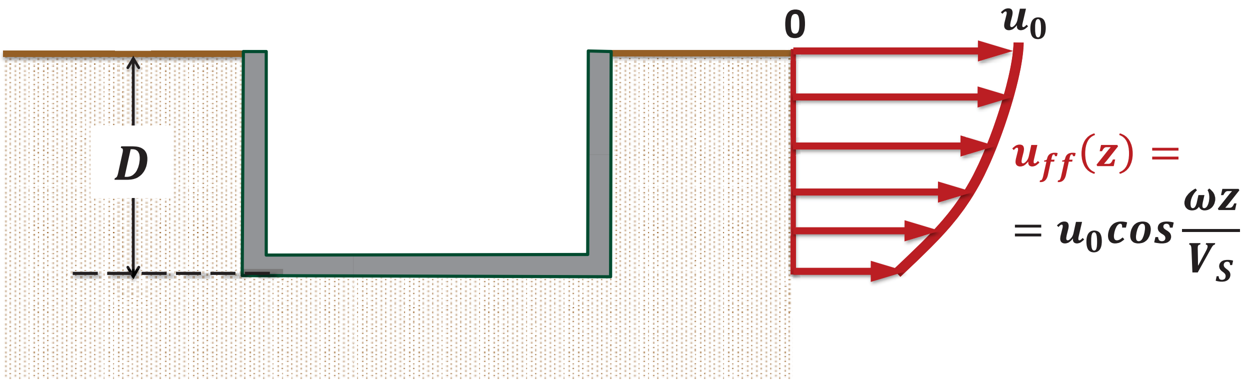
Kinematic Interaction:

the foundation oscillates, but can not follow exactly the motion of the surrounding soil arising from the wave propagation.

(a) the incident waves set the foundation in motion

(b) the free-field motion is being modified

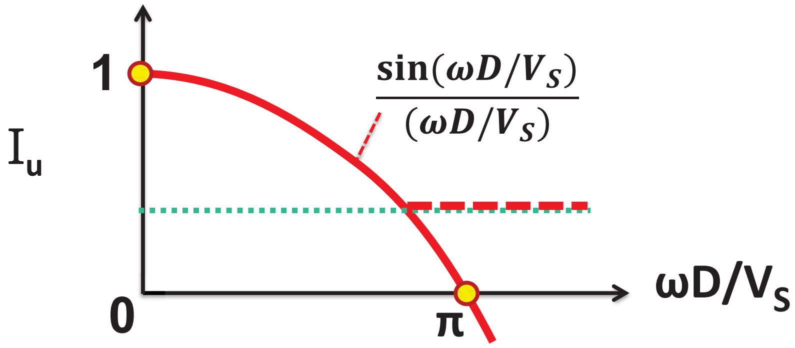




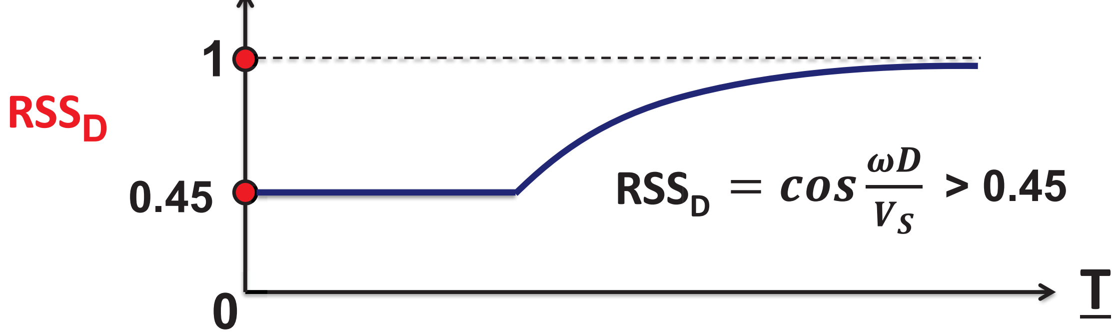
Crude conceptual estimate of kinematic FIM

$$u_{fdn} = \frac{1}{D}\int_0^D u_{ff}(z)dz$$

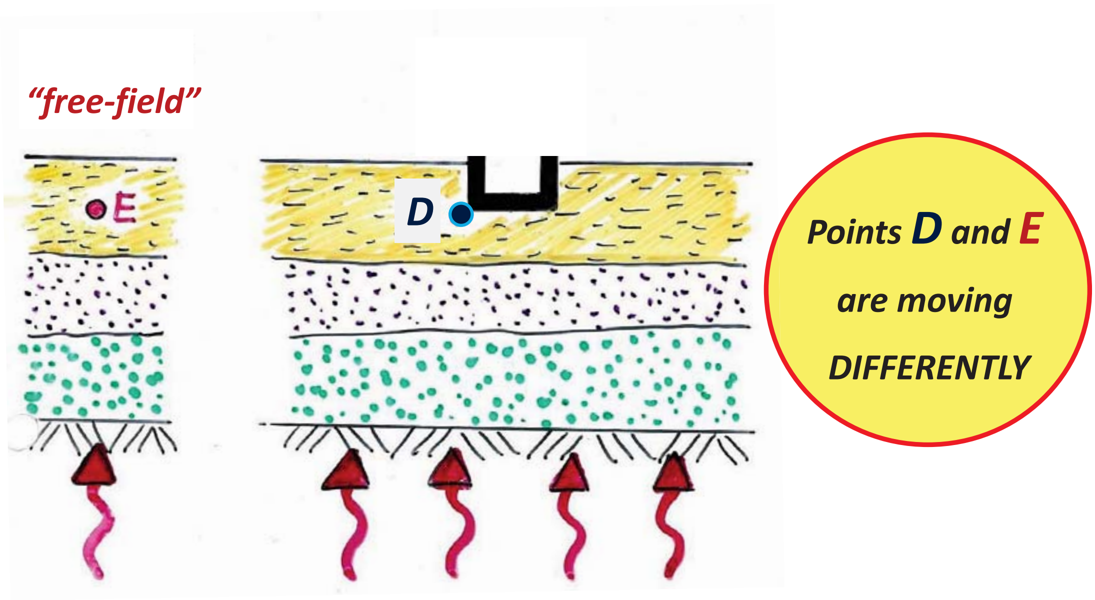
$$I_u = \frac{u_{fdn}}{u_0} = \frac{sin(\omega D/V_S)}{\omega D/V_S}$$



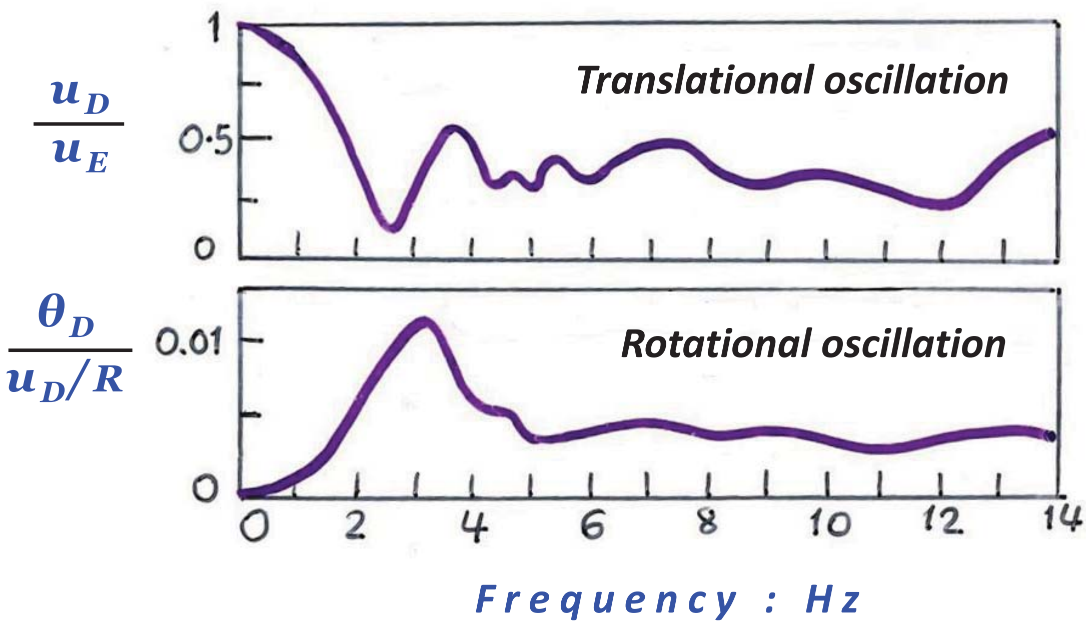
Ratio of Response Spectra for Embedment D : RSS_D



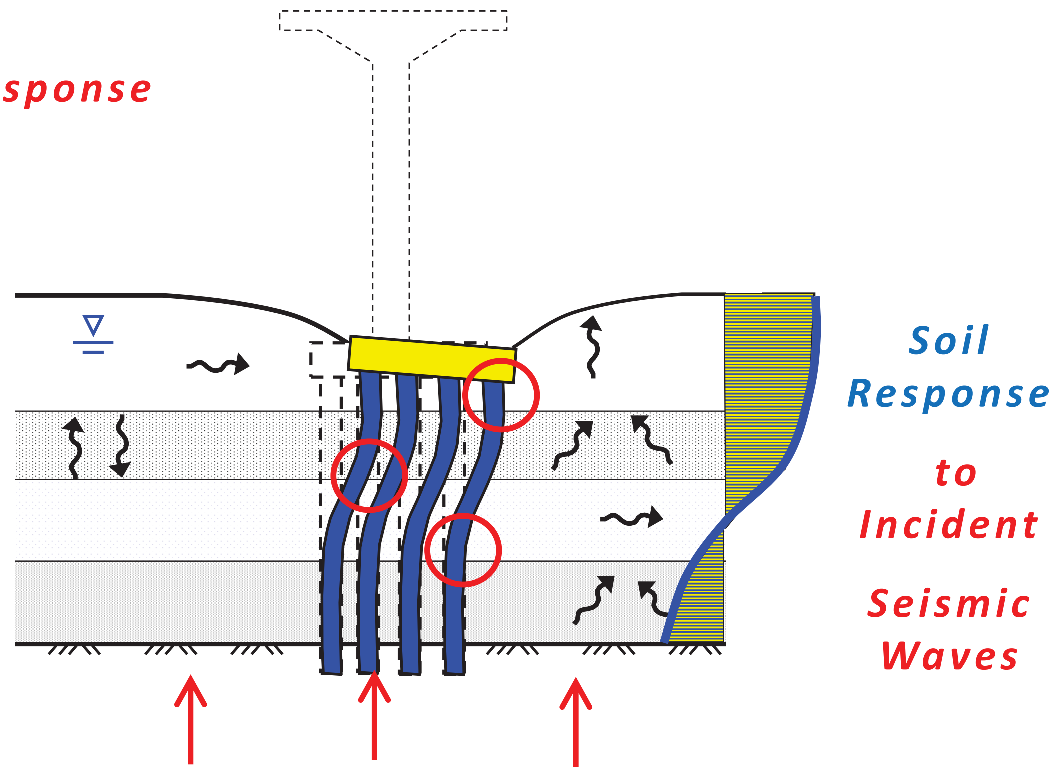
The fundamental problem
of seismic soil-structure-interaction



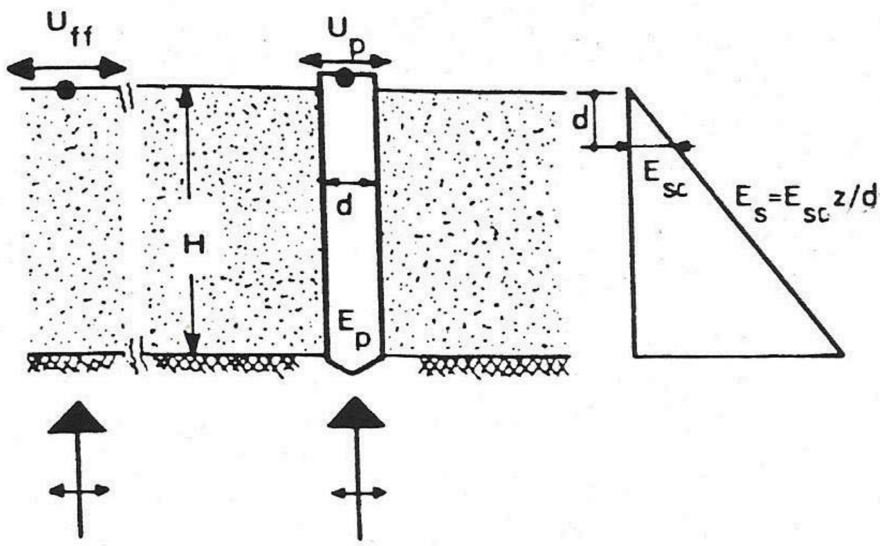
Example from a FE study



Kinematic Response

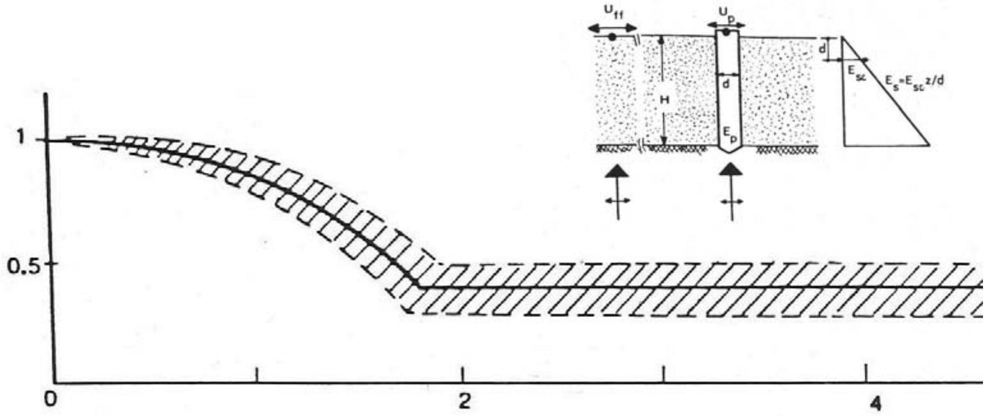


KINEMATIC Soil–Structure Interaction



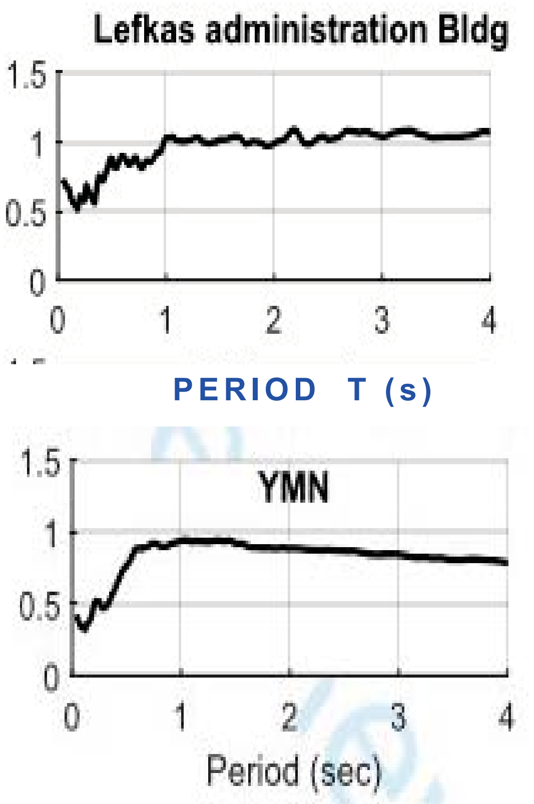
KINEMATIC SSI EFFECT
FIM decreases as a function of frequency

$$I_U = \frac{|U_p|}{U_{ff}}$$



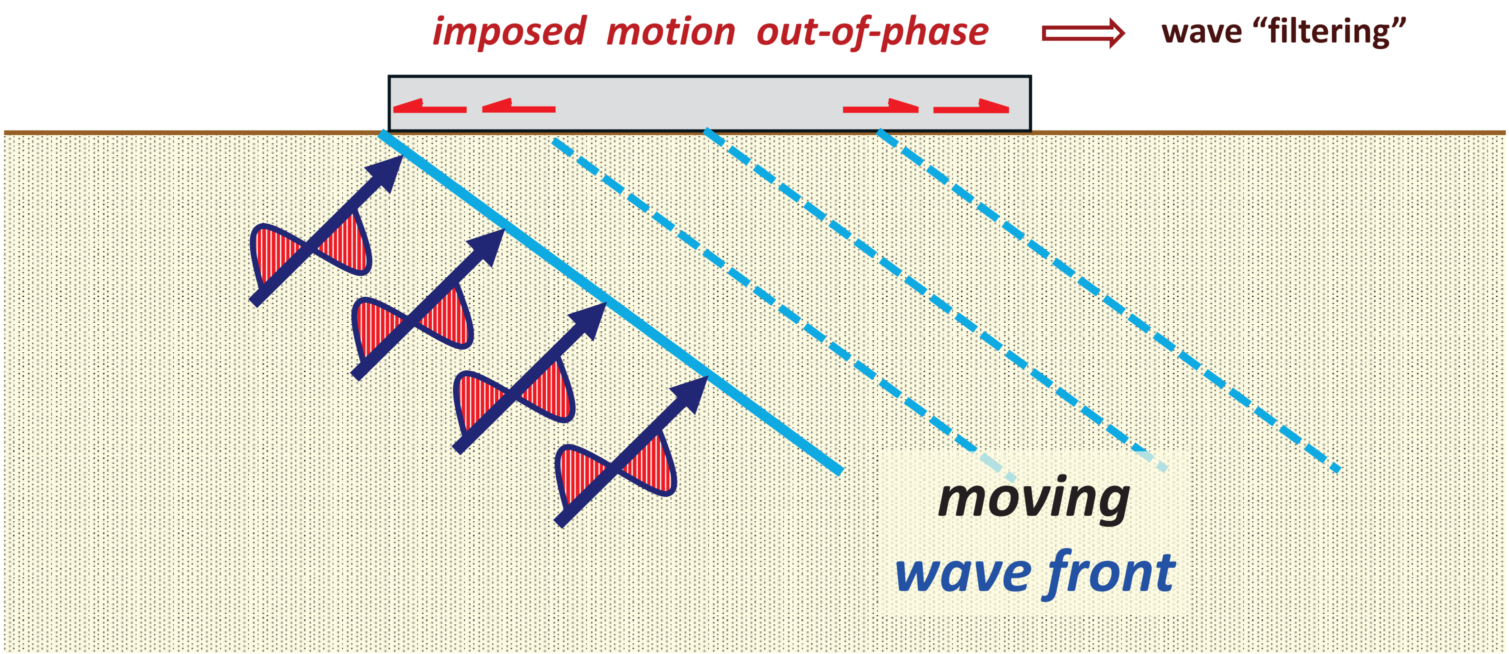
$$\frac{\omega d}{V_{sc}}$$

$$I_S = \frac{S_{a\ base}}{S_{a\ ff}}$$

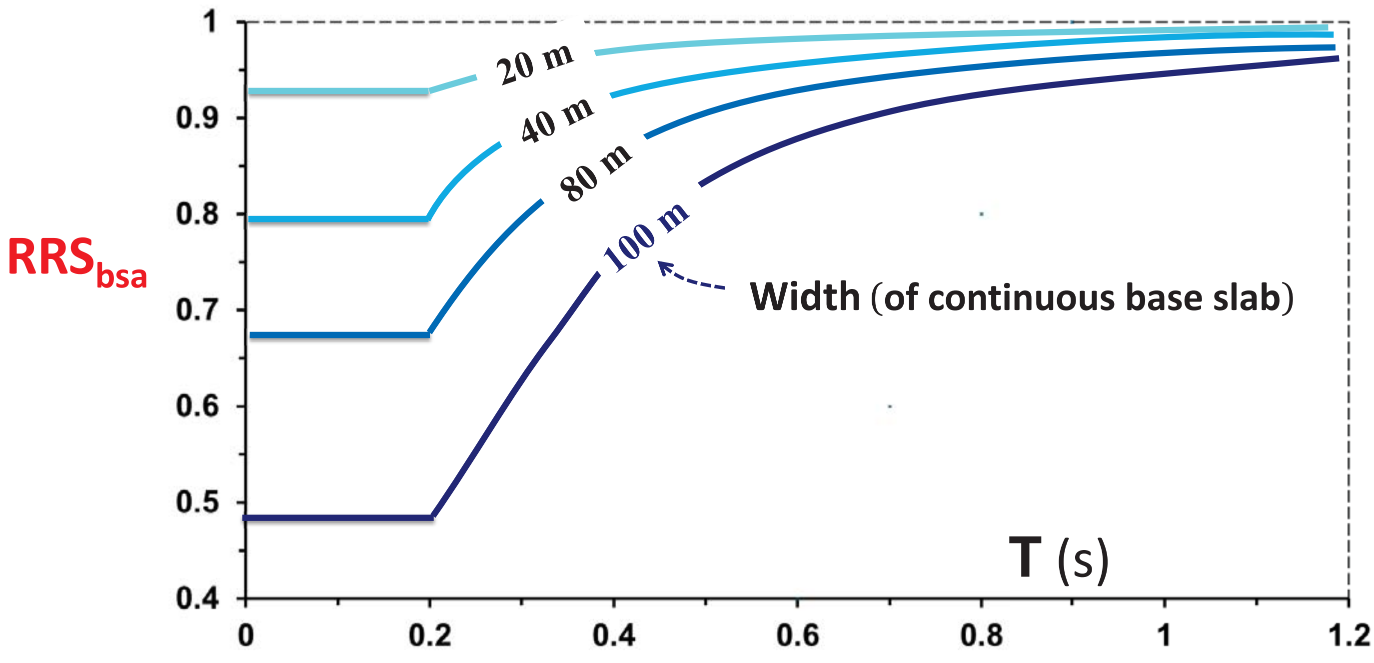


Cesaro + Di Laora, 2023

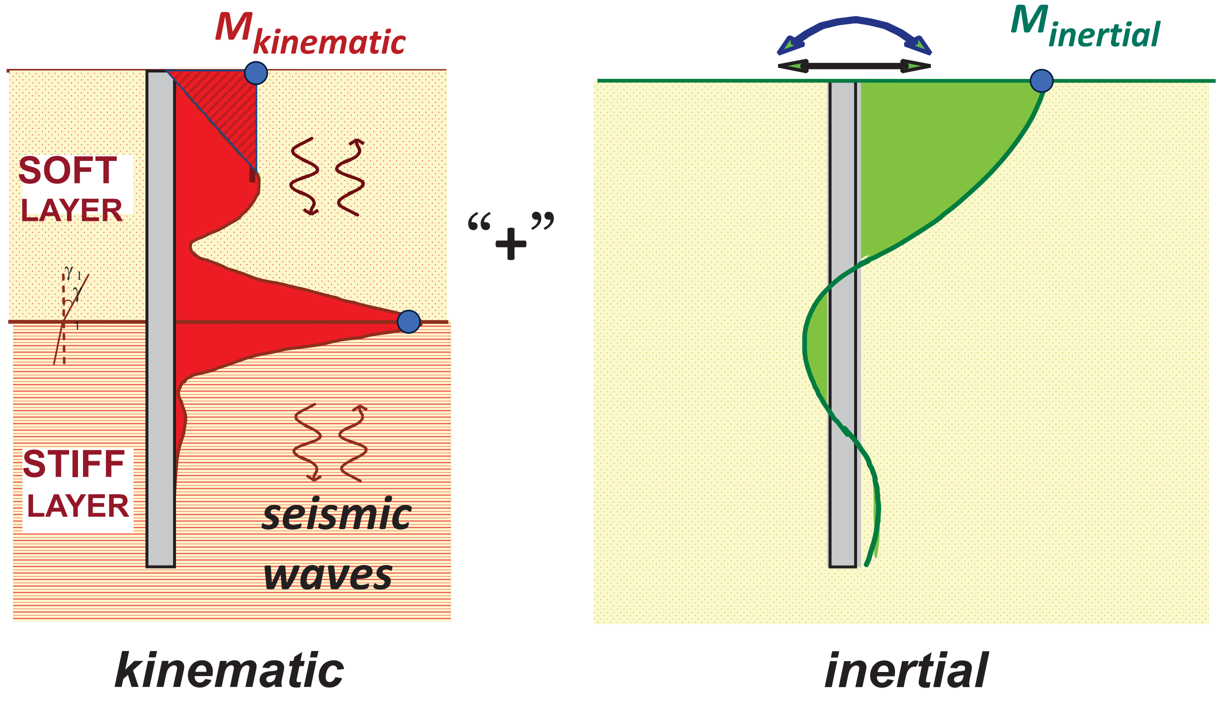
Base Slab Averaging
from non-vertical and incoherent incident waves



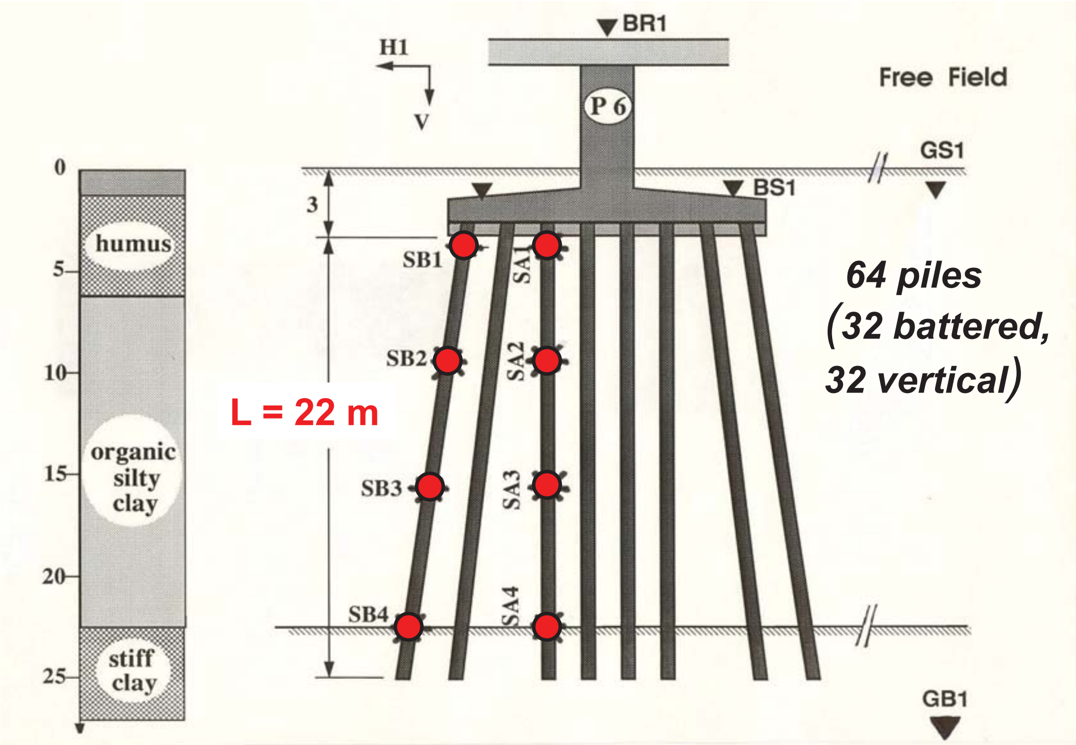
Ratio of Response Spectra for Base Slab Averaging: RSS_{bsa}

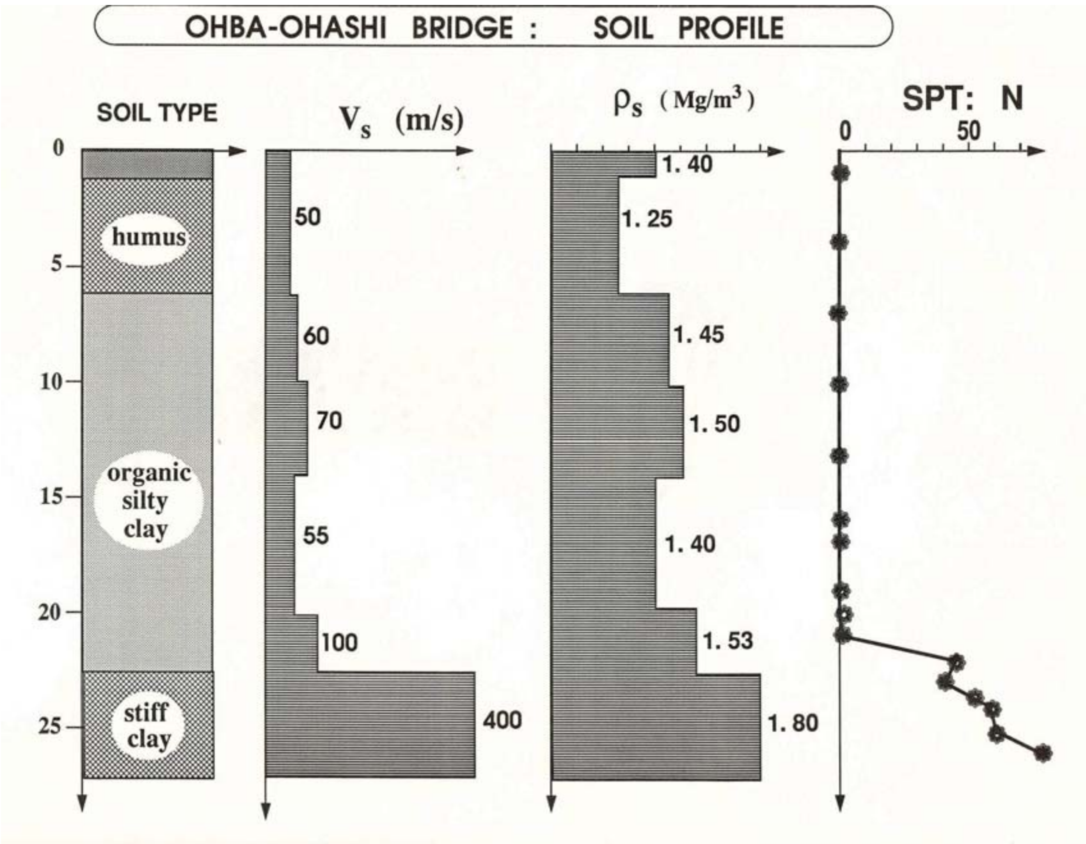


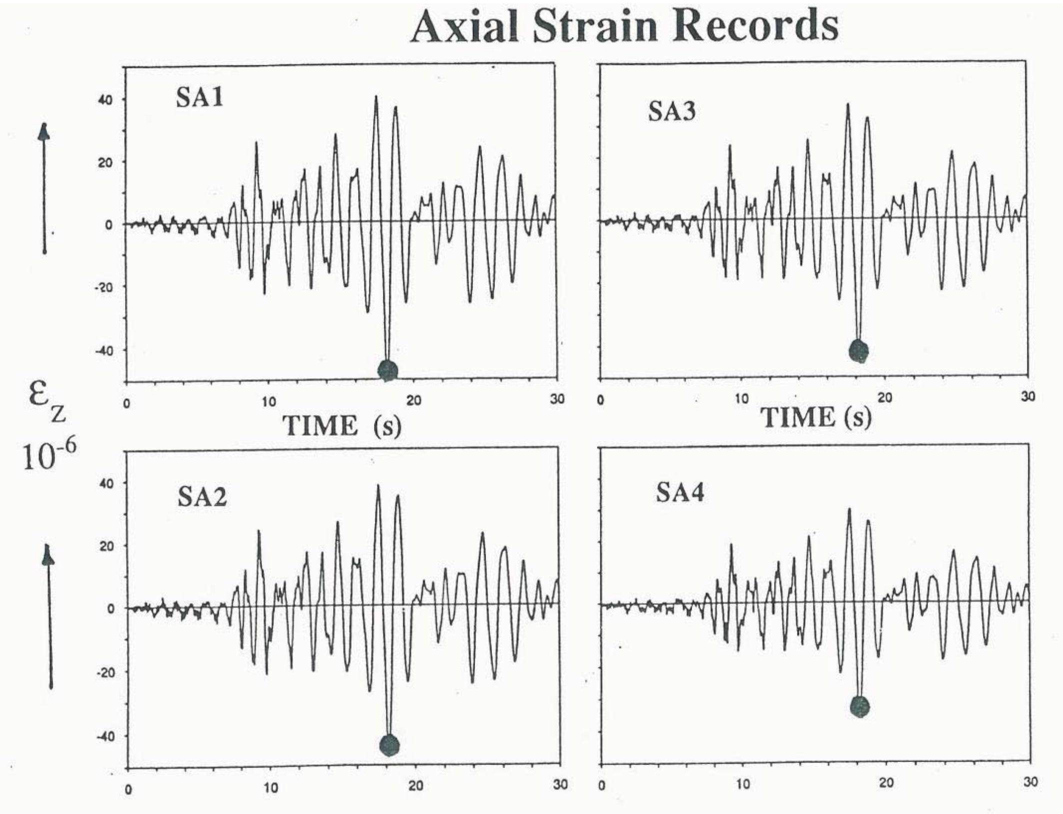
Lateral Loading of the Piles

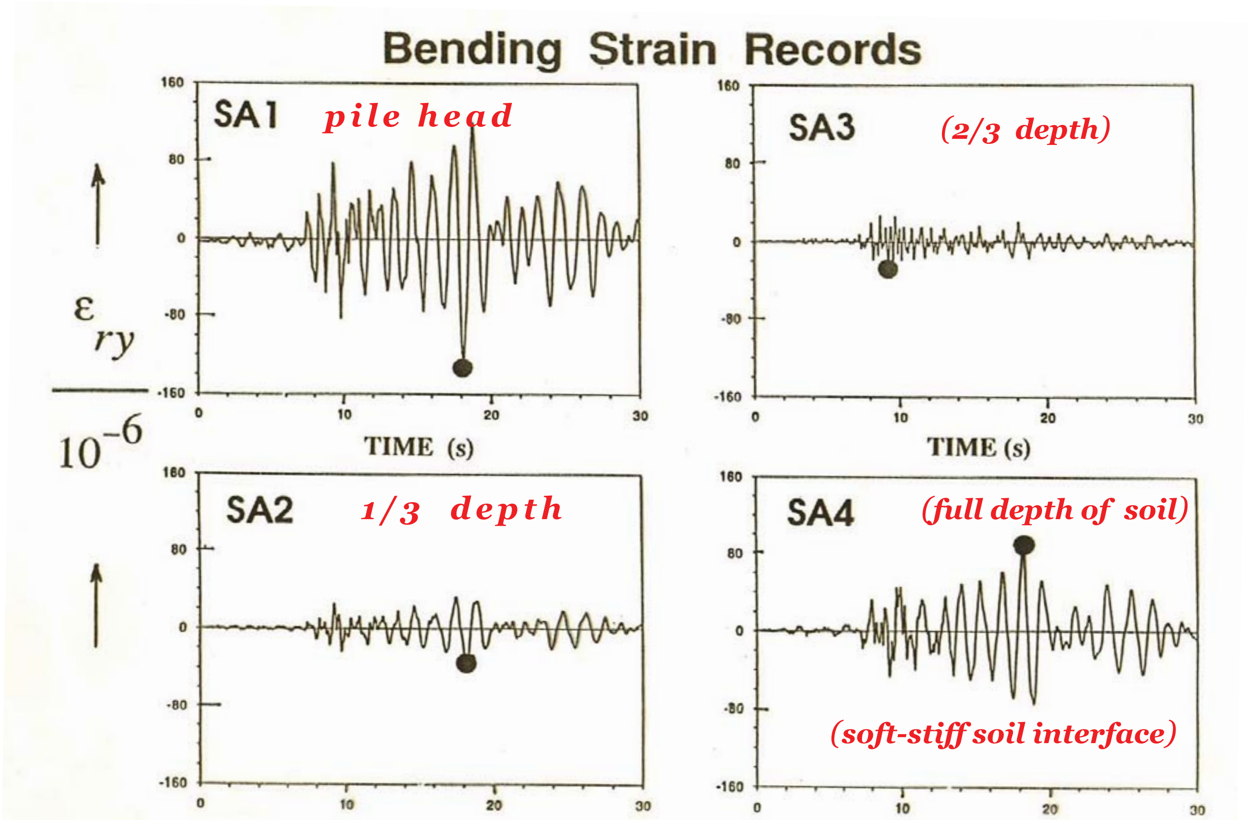


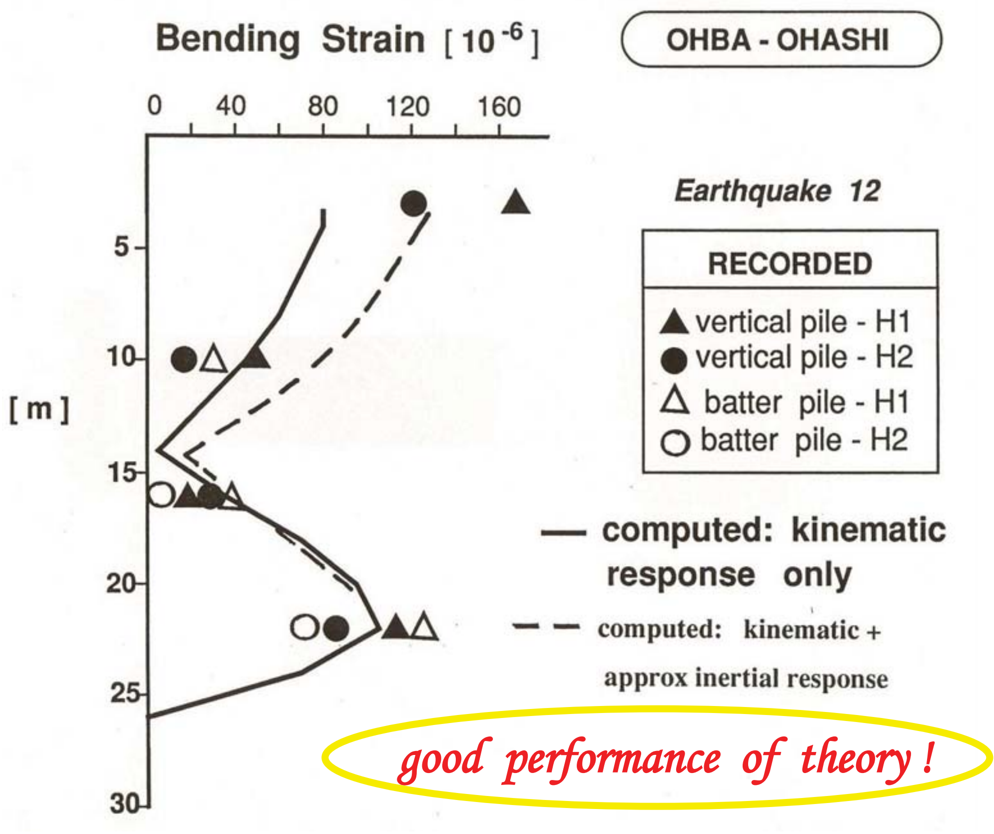
Ohba–Ohashi Bridge, Japan: Strain Gauges in two piles in Pier 6






Axial Strain Records
SA1
SA3
SA2
SA4
TIME (s)
TIME (s)
ε_z
10^{-6}

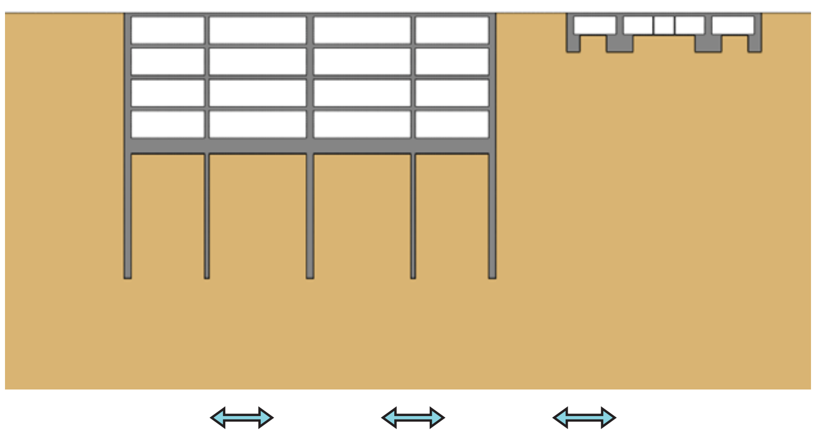

Bending Strain Records
SA1
pile head
SA3
(2/3 depth)
SA2
1/3 depth
SA4
(full depth of soil)
(soft-stiff soil interface)
TIME (s)
TIME (s)
$\frac{\varepsilon_{ry}}{10^{-6}}$



Not well understood problems on seismic response (kinematic and inertial):

Deeply Embedded Foundation (Multistory Basement) on Piles

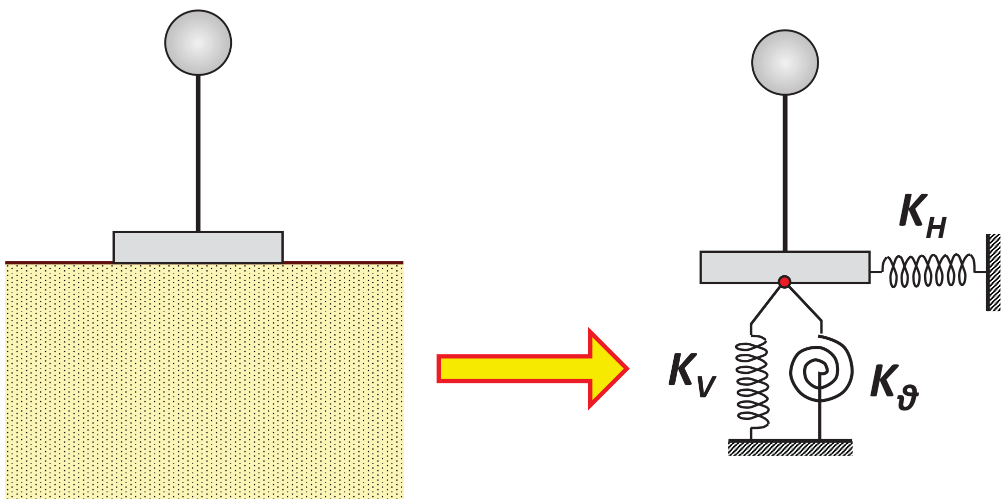
Example from Bucharest



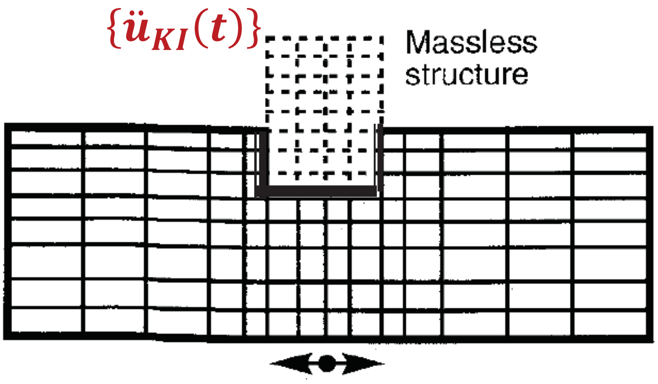
METHODS of ANALYSIS

To analyse INERTIAL SFS Interaction:

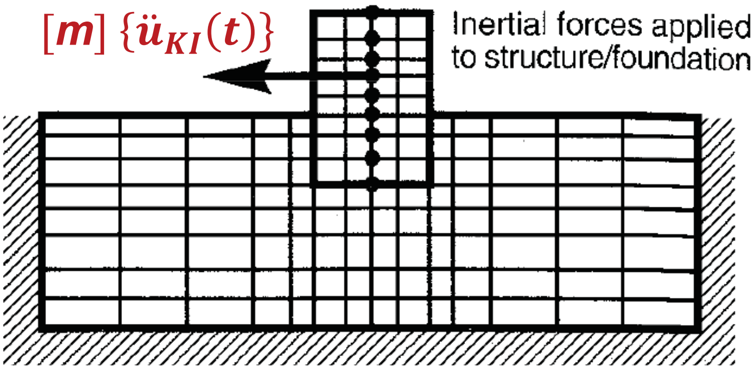
Replace the soil with springs K_V K_H K_ϑ . . .



Kinematic

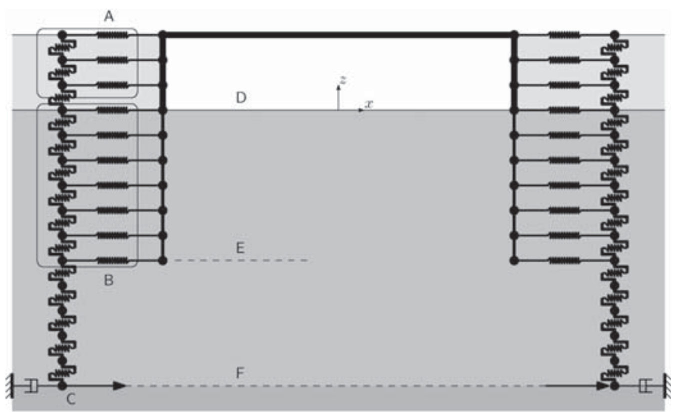


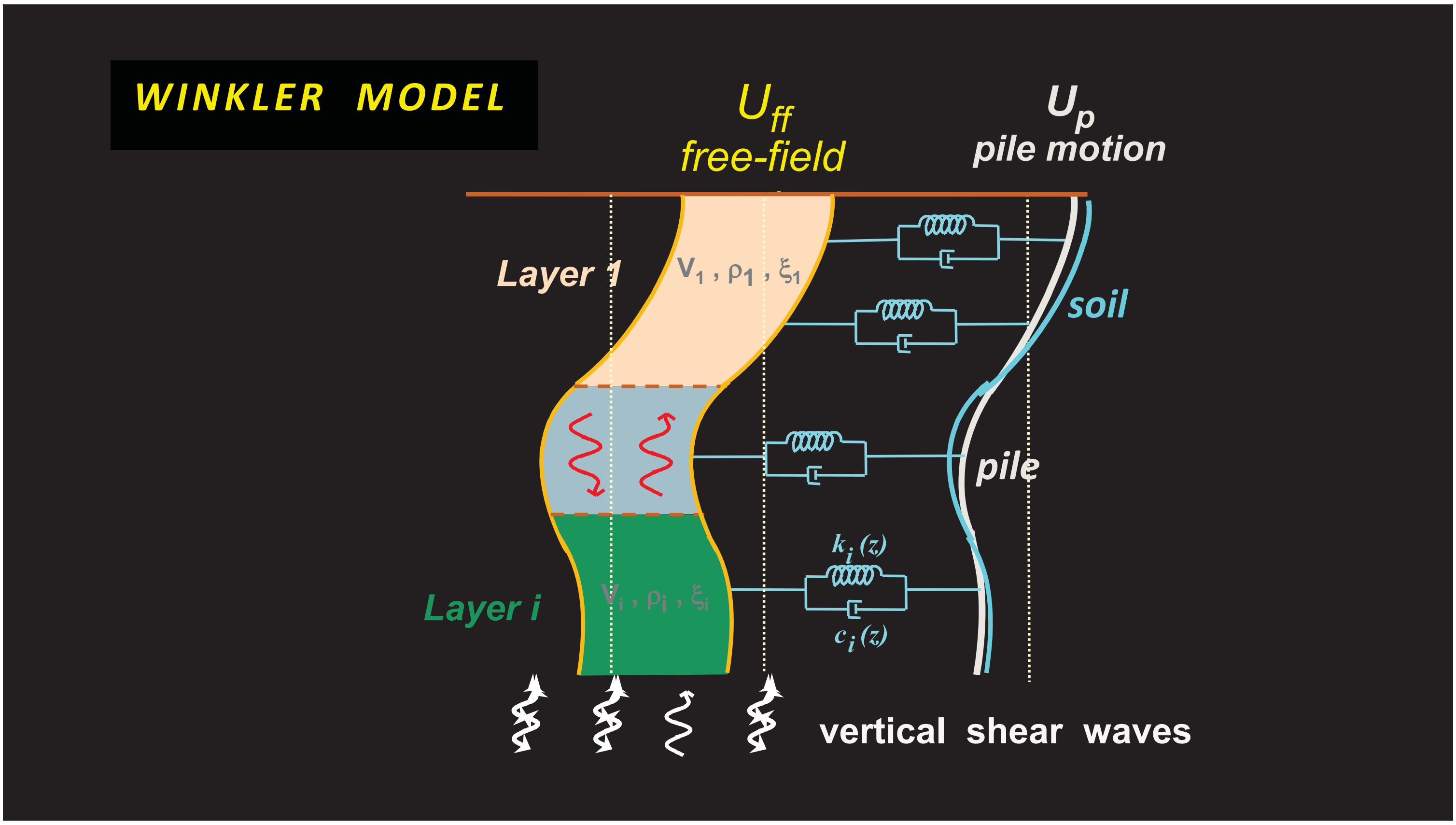
Inertial



Winkler-springs methods

DBA modelling example





How are all these phenomena and methods represented in the Code ??

B. SFSI in the Seismic Eurocode (EC8-5): Chapter 8, Annex D

- **Force-based versus Displacement-based Methods**
- **Kinematic and Inertia Response**
- **Rules and Simplifications**

Force-based approach: the seismic action is applied pseudo-statically.

The performance of a system evaluated based on the comparison of forces: *demand*, *F* , versus *capacity*, *R*,

$$F \leq R$$

Displacement-based approach: used when an evaluation of displacements is needed. It requires dynamic response-history analysis.

The performance of a system evaluated based on comparison of *computed displacements*, u , versus *acceptable displacements* , $u_{allowable}$,

$$u \leq u_{allowable}$$

4.5 Methods of analysis

(1) The seismic action effects should be calculated using either

- force-based approach
- displacement-based approach.

(2) In force-based, verifications should be in terms of generalized stresses.

(3) In displacement-based, compliance ... checked by comparing calculated permanent displacements to acceptable ones.

NOTE 1 Acceptable displacements agreed for a project by relevant parties.

5.2 Seismic action

Horizontal ground acceleration are defined as

$$a_H = \alpha_H g = \frac{Effective\ Ground\ Acceleration}{\chi_H}$$

coefficient χ_H = 1.25 ÷ 3 depending on accepted displacements in *force-based* approaches (*FBA*)

χ_H	1,25	1,5	1,75	2,00
permanent displacements (mm)	≤ 15	20 to 50	50 to 100	100-200

χ_H = 1 in *displacement-based* approaches (*DBA*)

8.1 General requirements

(1) The analysis of seismic SSI effects should consider two effects:

a) Inertial effects that modify the dynamic response of the structure by changing the fundamental period and damping of the soil-structure system.

b) Kinematic effects that modify the seismic excitation at the base of the structure with respect to the free-field, and produce loading of foundation elements.

8.1 (5) The inertial effects of SSI should be considered when at least one of the following applies:

a) When increasing the fundamental period increases spectral accelerations.

b) When the displacement of the structure controls the width of joints separating nearby buildings (existing or planned), or other performance criteria.

c) For structures supported on soft soils in which $v_s < 250$ m/s (velocity averaged over a depth equal to 3 times the maximum foundation width in case of footings or to the maximum width in case of a raft foundation).

d) Structures where geometric non-linearity (*$P - \Delta$ effect*) plays a significant role.

8.1 (6) Kinematic Modification of Foundation input motion should be considered:

a) in case of deep foundations (piles, caissons)

b) foundations embedded to a depth of at least 2 floors, or to a depth $> L/4$, if the foundation vertical surfaces is in full contact with the surrounding ground

c) abutments of bridges with large embankments, or integral bridges without specific provisions for minimizing SSI effects

d) very large foundations with *L or B* > 50 m consisting of a slab, or a single box foundation, or footings interconnected with tie beams.

8.1

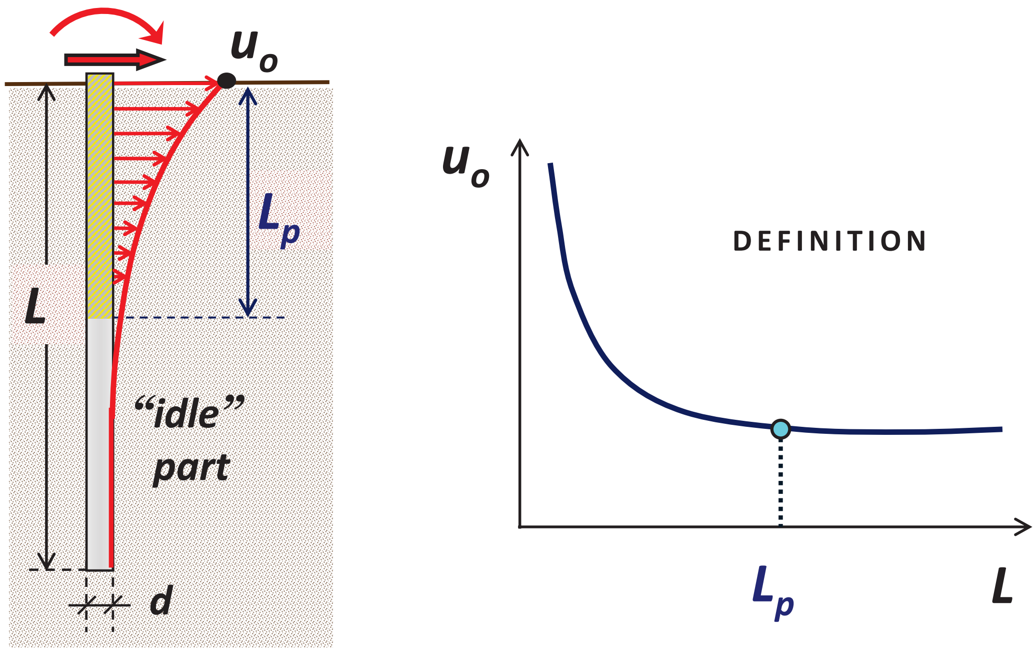
(7) For flexible pile foundations, modification of the free-field motion, as required in 8.1(6)a), may be neglected and the free-field motion may be used for the foundation input motion (FIM).

(8) A pile foundation may be considered as flexible when

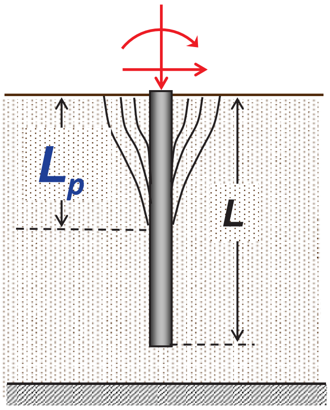
$$E_P / E_S \leq (L_P / 1{,}5\ d)^4 \quad \text{from} \quad L_P \geq L_c \approx 1{,}5\ d\ (E_p / E_s)^{0.25}$$

where L_p and d are the pile length and pile diameter.

active length L_p of flexible pile

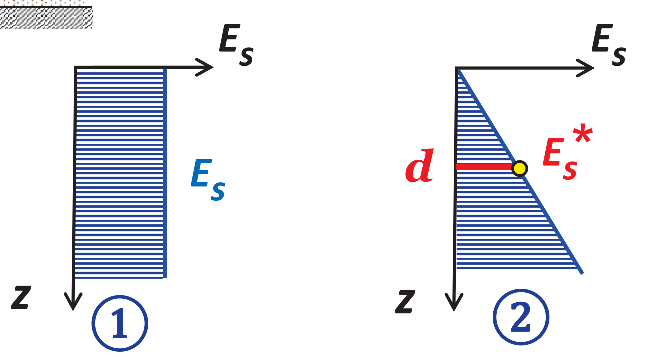


L_p of flexible pile



$$L_p \approx 1.5d(E_p/E_s)^{0.25} \quad (1)$$

$$L_p \approx 1.5d(E_p/E_s^{*})^{0.22} \quad (2)$$



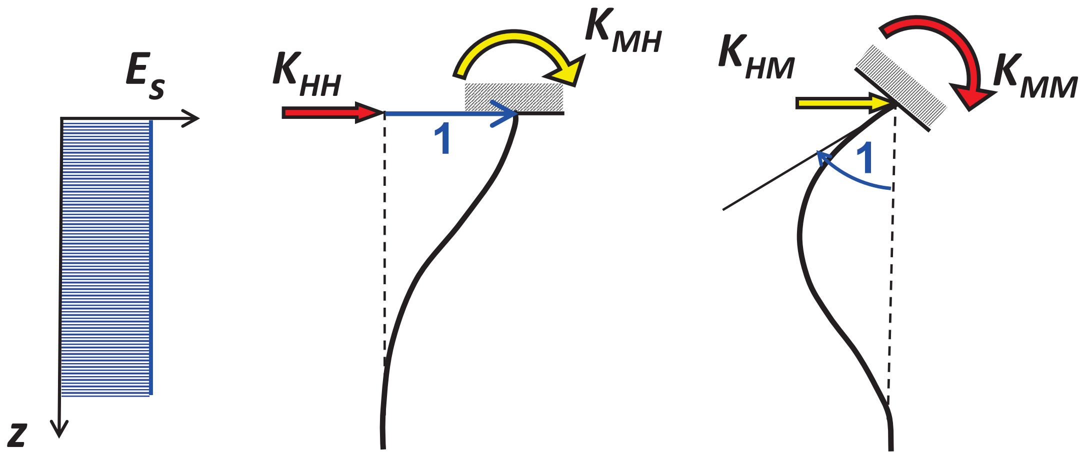
8.2 Analysis of inertial effects

(1) Seismic action effects ...should be determined with suitable model ......
The ground reaction may be represented by springs for all degrees of freedom.

NOTE A rigid foundation has 6 degrees of freedom, 3 translational (in x, y, z) and 3 rotational (rx, ry, rz , about the x, y and z axes).

(3) For some foundation shapes (circle, strip, rectangle), piles and ground profiles spring stiffnesses may be obtained from available elasticity-based solutions.

NOTE See Annex D for guidance to obtain stiffness and damping of foundations and piles.



Homogeneous Layer

$$K_{HH} \cong E_s d\,(E_p/E_s)^{0.21}$$

$$K_{MM} \cong 0.15\,E_s d^3\,(E_p/E_s)^{0.75}$$

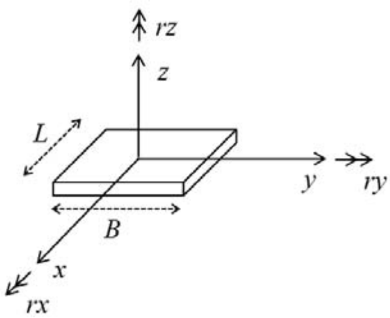
$$K_{MH} = K_{HM} \cong -0.22\,E_s d^2\,(E_p/E_s)^{0.50}$$

Footing B x L on Homogeneous halfspace

$$K_{xx} = \frac{GB}{2-\nu}\left[1{,}2 + 3{,}3\left(\frac{L}{B}\right)^{0{,}65}\right] \qquad K_{rx} = \frac{GB^3}{8(1-\nu)}\left[0{,}4 + 3{,}2\left(\frac{L}{B}\right)\right]$$

$$K_{yy} = \frac{GL}{2-\nu}\left[2 + 2{,}5\left(\frac{B}{L}\right)^{0{,}85}\right] \qquad K_{ry} = \frac{GB^3}{8(1-\nu)}\left[3{,}6\left(\frac{L}{B}\right)^{2{,}4}\right]$$

$$K_{zz} = \frac{GL}{1-\nu}\left[0{,}73 + 1{,}54\left(\frac{B}{L}\right)^{0{,}75}\right] \qquad K_{rz} = \frac{GB^3}{8}\left[4{,}1 + 4{,}2\left(\frac{L}{B}\right)^{2{,}45}\right]$$



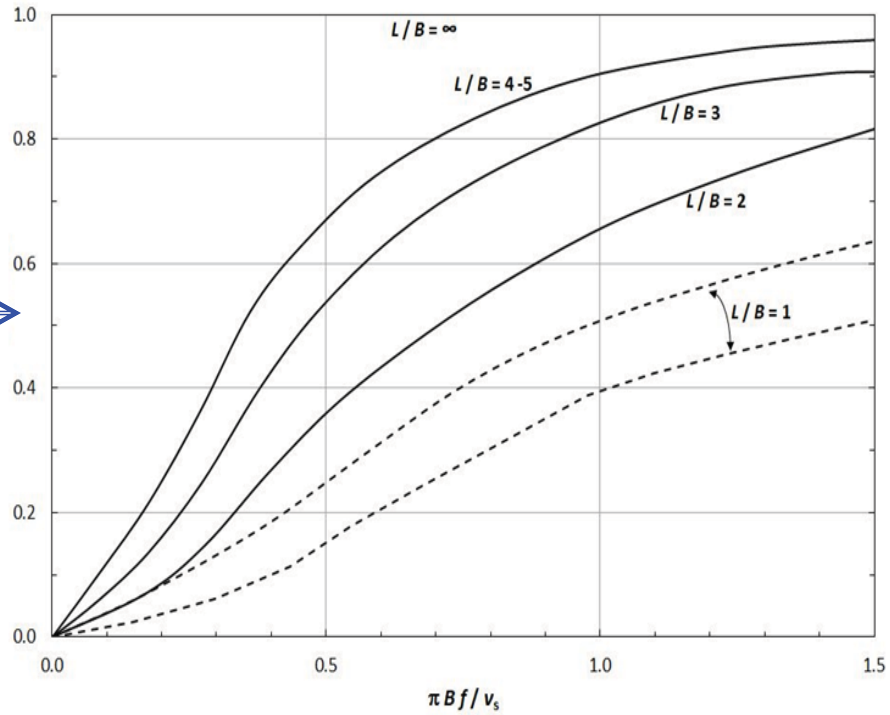
Translational modes

$$\boxed{C_{xx} = C_{yy} \approx \rho v_s A_b}$$

Rotational modes

$$C_{ry} = \frac{\rho v_s J_{by}}{1-\nu} c'_{ry}$$

On HOMOGENEOUS Halfspace



(4) Frequency-independent stiffness <u>may</u> be assigned to each spring, corresponding to the period of the fundamental mode, accounting for SSI in the considered direction. If this period is difficult to determine reliably, the <u>static stiffnesses</u> may be used instead.

(5) For design limit states SD and NC, the equivalent-linear stiffnesses for nonlinear springs to be used should be compatible with the amplitude of horizontal displacements and rotations of the foundation.

8.2.1 Force–based approach

(1) The effect of damping due to SSI should be neglected in FBA.

(2) Radiation damping <u>may</u> be used only for periods $T < T_1$ (the fundamental period of the soil deposit).

Radiation damping should be limited to 20 %,
unless supported by numerical calculations which model the layer properties down to a depth where $v_s > 600$ m/s,

8.2.2 Displacement–based approach

8.2.2.1 Nonlinear static analysis

(1) In non-linear static analysis …. translational and rotational ***inelastic*** springs may be used.

….

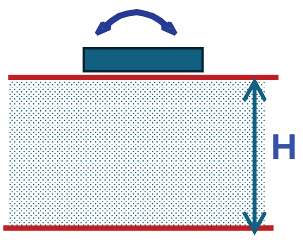
The possibility of uplift on the tension side of the foundation, as well as of slippage at the ground-foundation contact surface, <u>may</u> be included in the model.

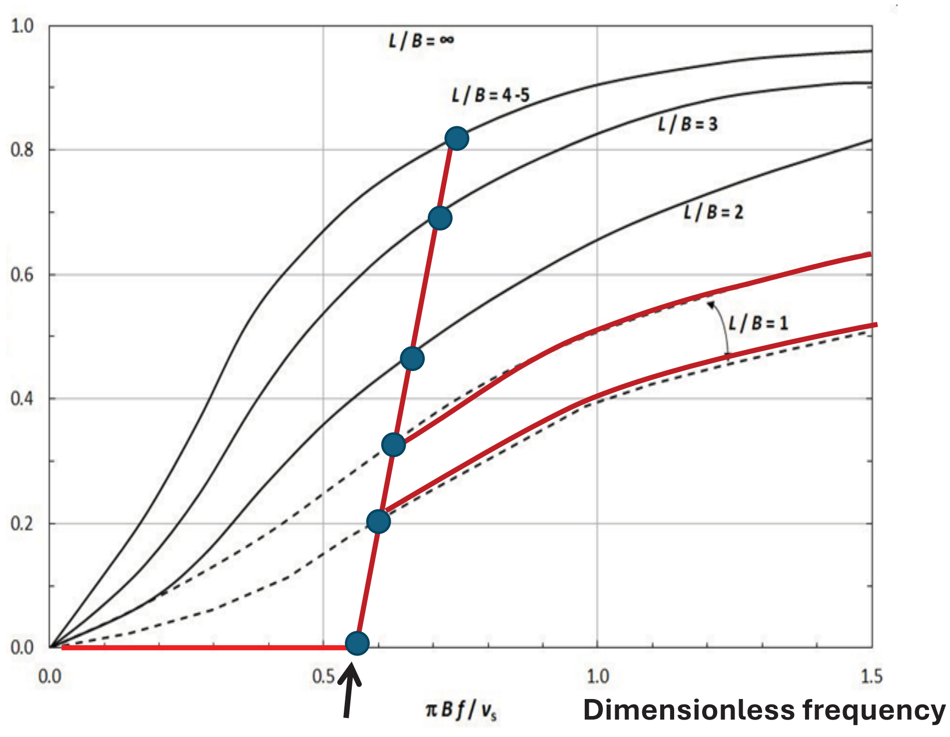
8.2.2.2 Time history analyses

(2) A frequency-independent stiffness value may be assigned to each spring, corresponding to the period of the fundamental mode, accounting for SSI in the considered direction.

NOTE Radiation damping is strongly affected by ground layering. Solutions for a homogeneous elastic half-space result in unrealistically large values of damping.

$$C_{\mathrm{ry}} = \frac{\rho v_{\mathrm{s}} J_{\mathrm{by}}}{1-\nu} c'_{\mathrm{ry}}$$





$f_o = 1 / T_o \approx v_s / 4H$

8.3 Modelling of kinematic effects

(1) Kinematic interaction effects may be calculated .... as part of the whole structure-foundation-soil system, or with a separate analysis where only foundation and the soil are included.

(2) For piles a suitable Winkler type model may be used with lateral soil springs and dashpots representing the action of the soil in contact with the foundation elements.

(3) In FE/FD of pile–soil system, the seismic excitation should be imposed at the base of soil stratum and lateral boundaries should be capable of deforming as the free-field.

8.4 Combination of inertial and kinematic effects for internal forces

(1) If inertial and kinematic effects are evaluated separately, the forces in the foundation elements from the two analyses may be combined according to either a) or b):

a) when the frequency of the mode contributing most to the SSI response differs by more than 15% from the fundamental frequency of the soil deposit, the action effects are combined with SRSS rule (square root of the sum of the squares)

b) when the condition in a) is not satisfied, the absolute values of the action effects of the two analyses are summed up.

Chapter 9. Foundation system

in the new EC8

- Shallow foundations
- Piles
- Design values and verifications

Annex E

9.2 Design values of the action effects

Force-based approach (FBA):

over-design = $(R_{di}/E_{di}) \leq q$

overstrength factor

$$E_{Fd} = E_{Fd,G} \text{ "+" } \frac{\Omega_d \gamma_{Rd}}{\chi_H} E_{Fd,E}$$

non-seismic action effects

design seismic action effects (S_a/q)

Depends on accepted permanent displacement.

$\Omega_d\ \gamma_{Rd} = 1{,}25\ q_R$: FBA Capacity Design

= 1 DBA

Sliding verification:

FBA

$$V_{\mathrm{Ed}} \leq V_{\mathrm{Rd,1}} + V_{\mathrm{Rd,2}} + 0{,}3\, V_{\mathrm{Rd,3}}$$

Action effects in superstructure without considering sliding

$$V_{\mathrm{Rd,1}} = F_{\mathrm{Rd}} = (N_{\mathrm{Ed}} - U)\frac{\tan\delta_f}{\gamma_\delta}$$

DBA (non-linear)

- **Sliding accepted at SD or NC**

  (If acceptable for the superstructure and lifelines)

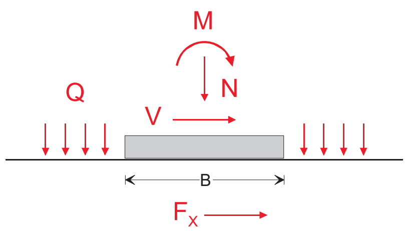
- χ_H = 1
- Full passive resistance

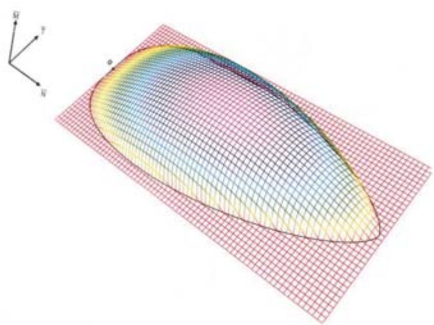
Resisting mechanisms for MOMENT *M*, SHEAR FORCE *V*, AXIAL FORCE *N*:

Bearing capacity verification:

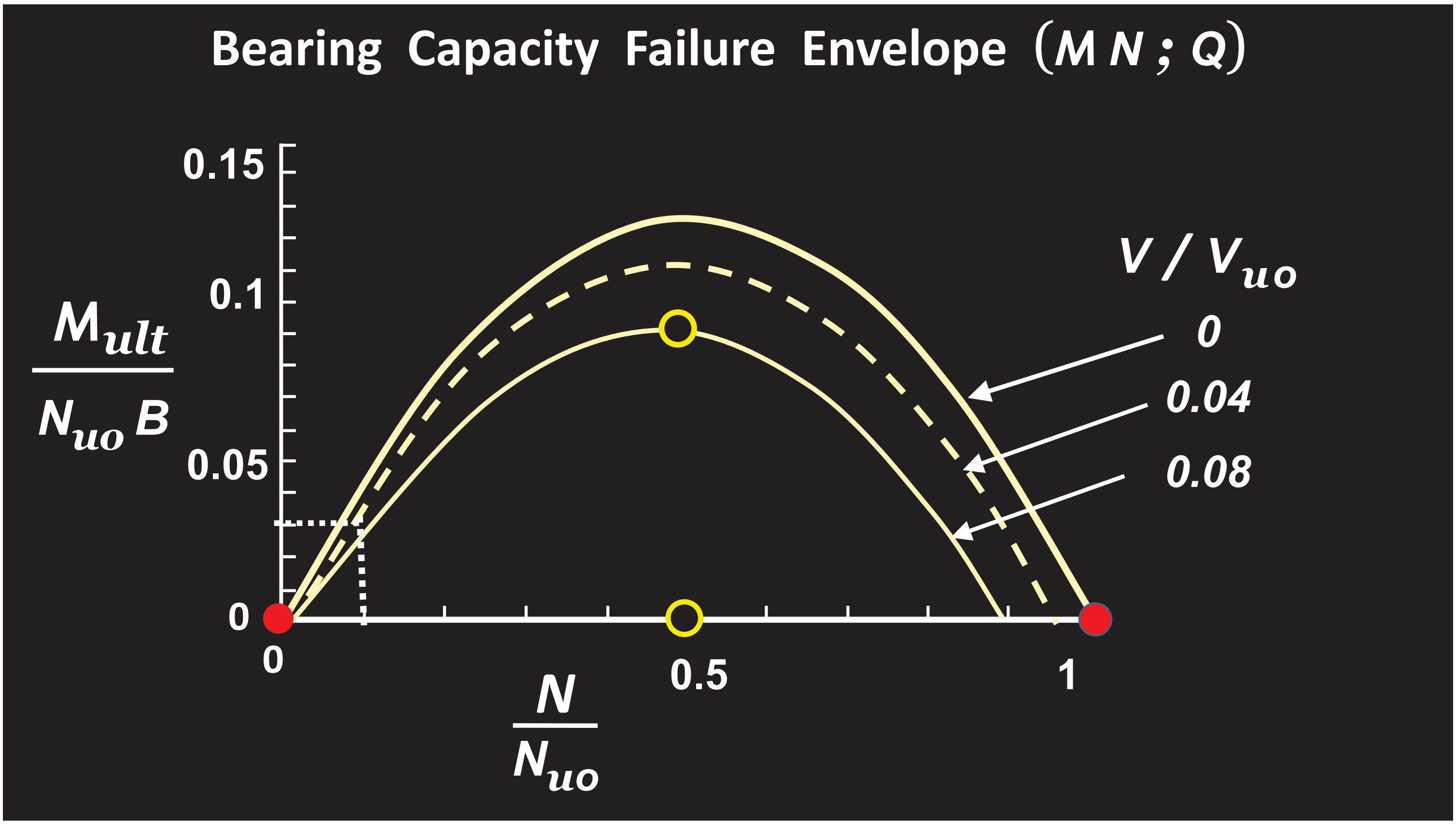
FBA

➢ Combination of N_{Ed}, V_{Ed}, M_{Ed}





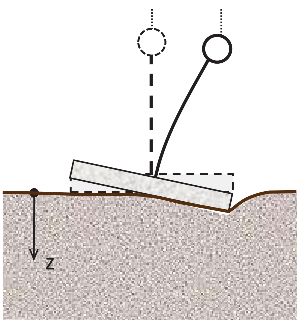
Annex E Interaction Surface (or Failure Envelope) including inertia forces in the ground



Rotational failure verification:

- in DBA: Uplifting (in rocking) is allowed always
  (if permanent rotations and settlements are acceptably small)

- in FBA if uplifted area is < 1/3



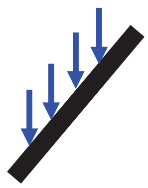
9.5 Pile foundations

Piles shall be designed to resist the following action effects:

a) **inertial forces and moments** transmitted by the superstructure

b) **kinematic action effects** of the deformation of the surrounding ground due to the passage of seismic waves.

Battered (inclined) piles are acceptable. Should be designed to carry **residual action effects** from vertical pressures



9.5.3 Methods of analysis

Group of piles:

- **Cap-base–soil interface** strength and stiffness should be limited to **30%** of full contact assumption
- But **no contribution from the contact** should be accepted if minimum pile spacing **s < 6 D**
- In FBA, limit to **30%** of horizontal **passive resistance** of ground in front of the cap

KINEMATIC analysis should give the bending moments at the **pile head** and at the **interface between layers of different stiffness.**

9.5.4 Design verifications

Earlier seismic codes demanded that piles remain structurally elastic.

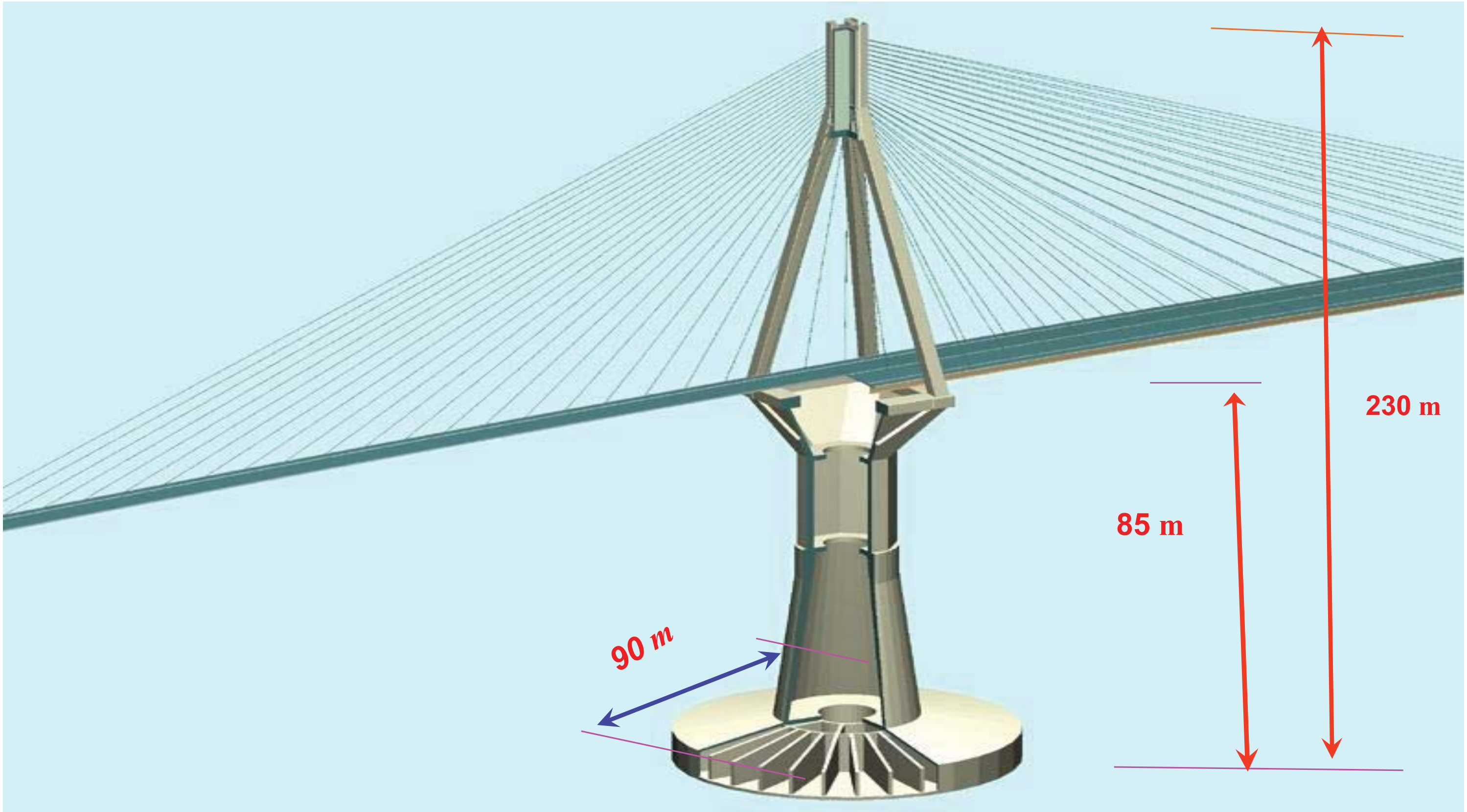
However:

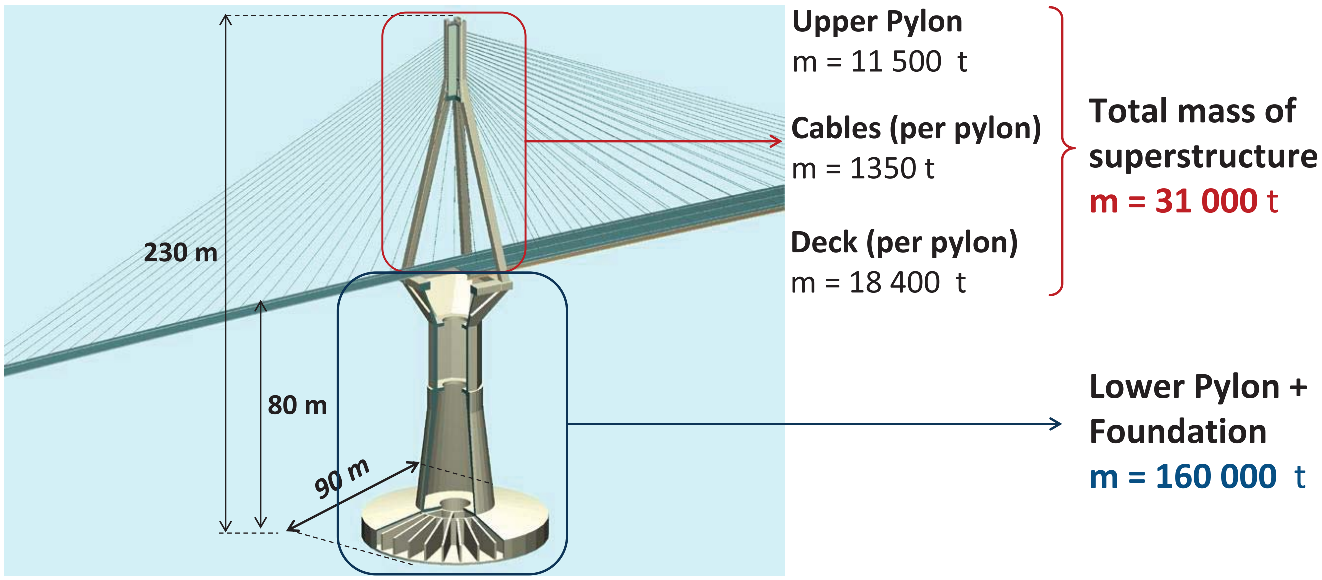
- **Pile yielding is not as concentrated as in columns, but instead distributed over a much greater length thanks to soil confinement. As a result, the plastic hinge rotation is likely too enough to be detrimental.**
- **Inelastic response of piles may have a beneficial effect on the overall response of the superstructure**

Sfârşit

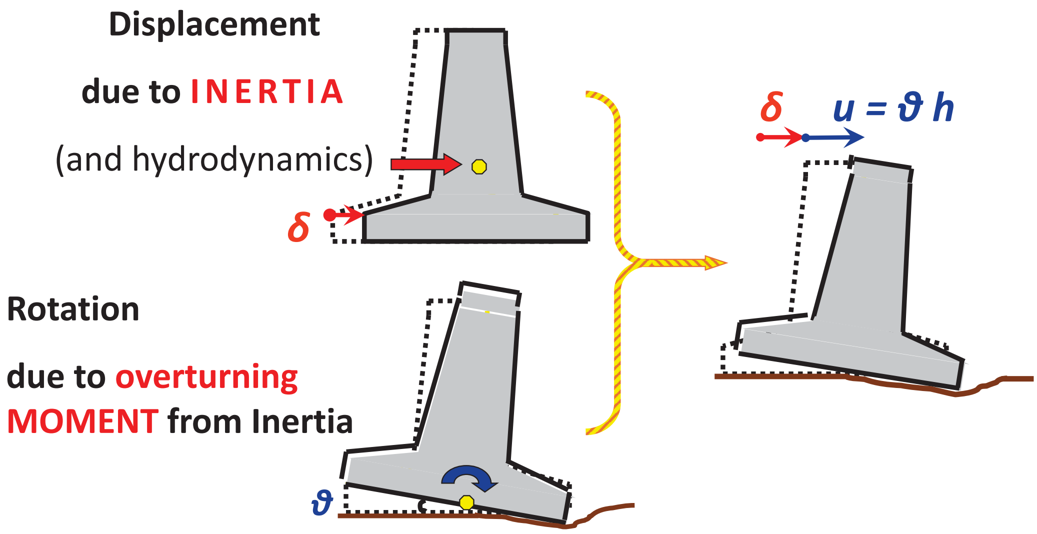
Vă mulțumesc foarte mult pentru atenție

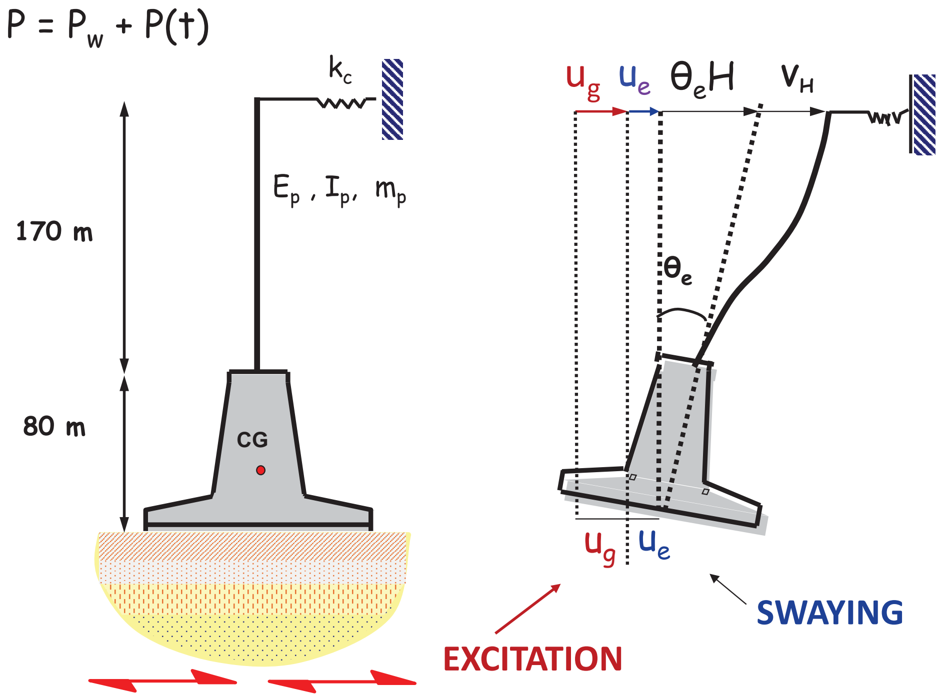

Rion–Antirrion Bridge


230 m
85 m
90 m

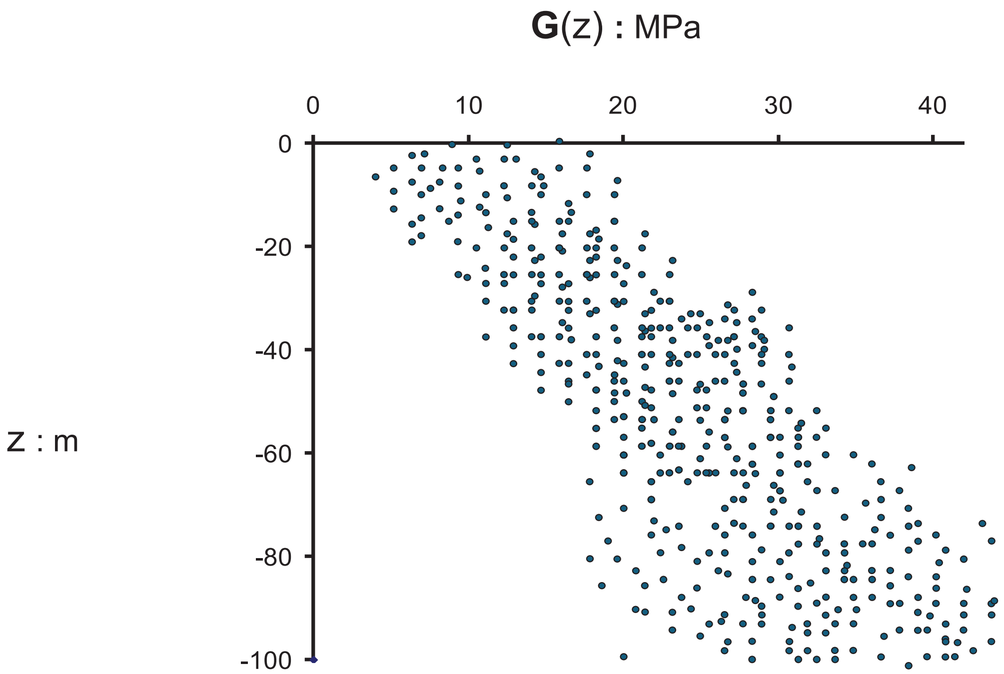


Mechanisms of PYLON–FOOTING Motion





P = P_w + P(t)
k_c
E_p , I_p, m_p
170 m
80 m
CG
u_g u_e θ_eH v_H
θ_e
u_g u_e
SWAYING
EXCITATION



G(z) : MPa
0 10 20 30 40
0
-20
-40
-60
-80
-100
z : m

With these $(K_\vartheta\ K_H)$ the

NATURAL PERIOD of the pylon is determined

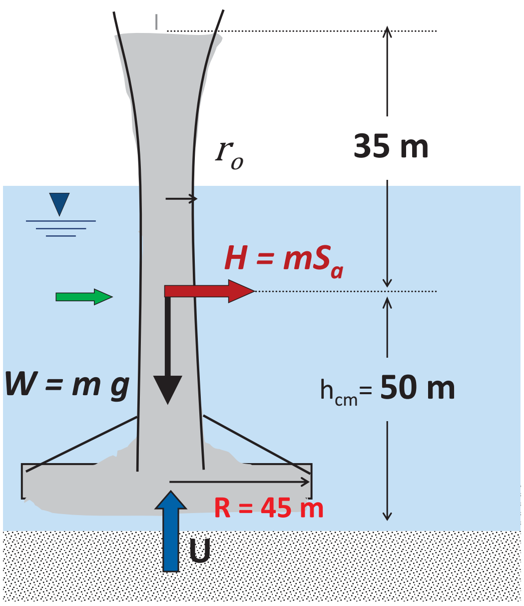
(approximately, rigid-body coupled saying–rocking).

Then, the effective SPECTRAL ACCELERATION is read.

Inertia + Hydrodynamic forces are computed and then the

SHEAR + MOMENT transmitted back onto the soil–footing interface

Seismic Actions



Structural weight: $\boldsymbol{W = 1\,900\ \mathrm{MN}}$

Uplift force: $\boldsymbol{U = 1\,100\ MN}$

→ Effective weight: $\overline{\boldsymbol{W}} = \boldsymbol{W} - \boldsymbol{U} = \mathbf{800}\ \mathrm{MN}$

→ Effective pressure: $\overline{\boldsymbol{q_{av}}} = \overline{\boldsymbol{W}}/\boldsymbol{\pi R^2} = \mathbf{125}\ \mathrm{kPa}$

Inertia force: $\boldsymbol{H = mS_a = (W/g)S_a = 550}\ \mathrm{MN}$

(acceleration is applied to the ***total mass***)

Hydrodynamic force: $\boldsymbol{H_{hydr} = 0.6\rho_w\,\pi\, r_o{}^2 = 300}\ \mathrm{MN}$

Base shear: $\boldsymbol{Q_{base} = 550 + 300 = 850}\ \mathrm{MN}$

Rough approximation: $h_{hydr} \approx h_{CM} \approx 50\ \mathrm{m}$

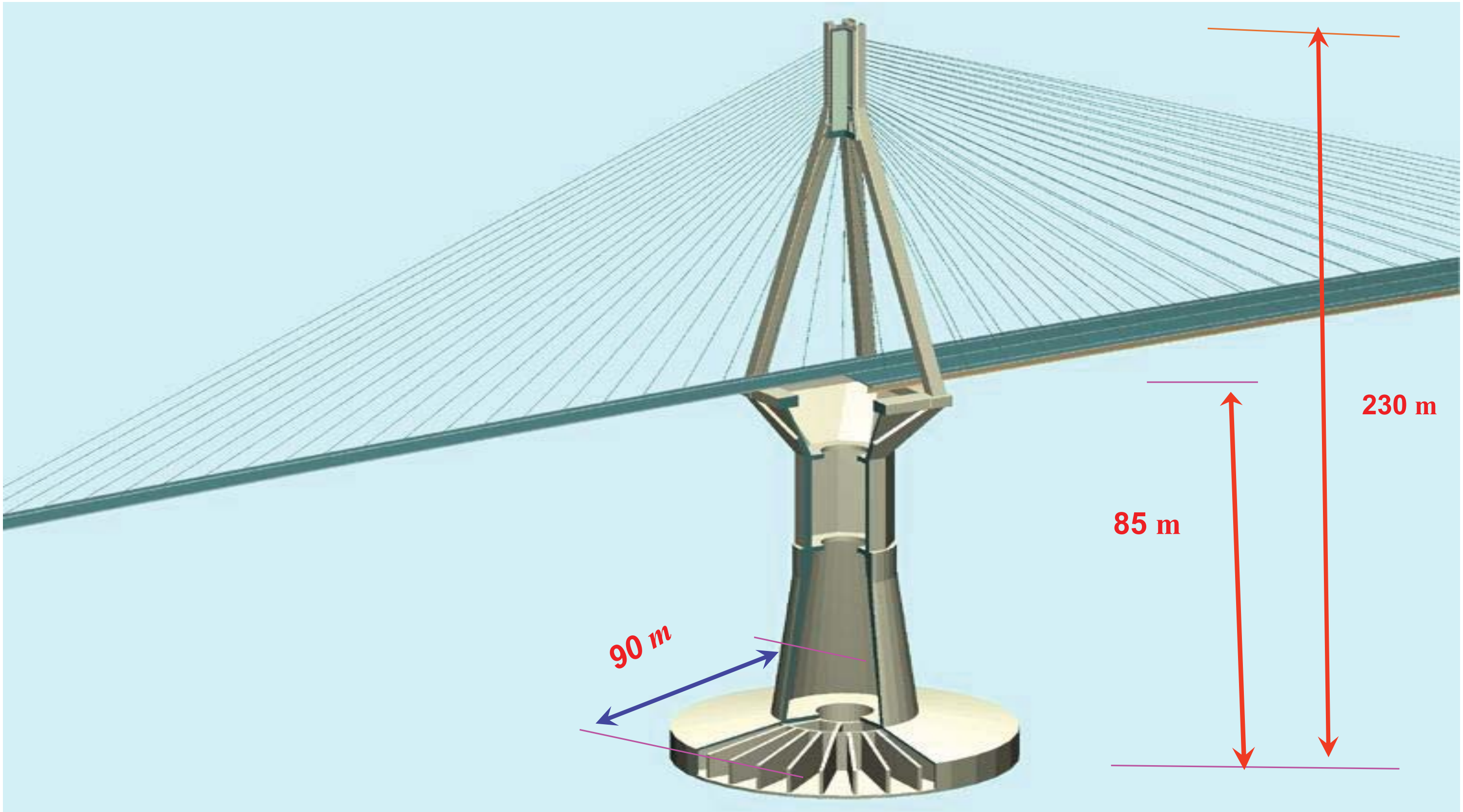
Base moment: $\boldsymbol{M_{base} = 850 \cdot 50 \approx 42\,500}\ \mathrm{MNm}$

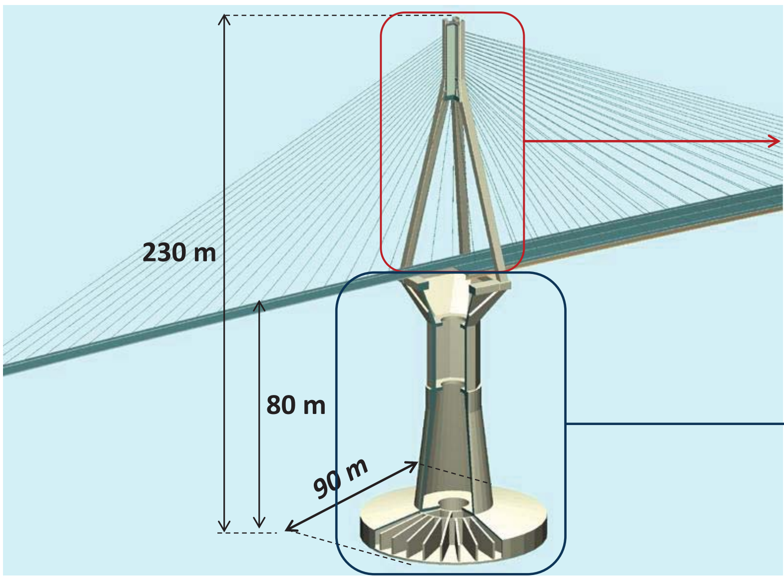
*The contribution of **SOIL COMPLIANCE** (K_M K_H) to the **NATURAL PERIOD** of the pylon is **so** decisive,*

*that it uniquely controls the effective **ACCELERATION** of the pylon,*

*and therefore the **FORCE + MOMENT** transmitted back onto the soil–footing interface*

And thus, it controls the seismic safety of the bridge!





Upper Pylon
m = 11 500 t

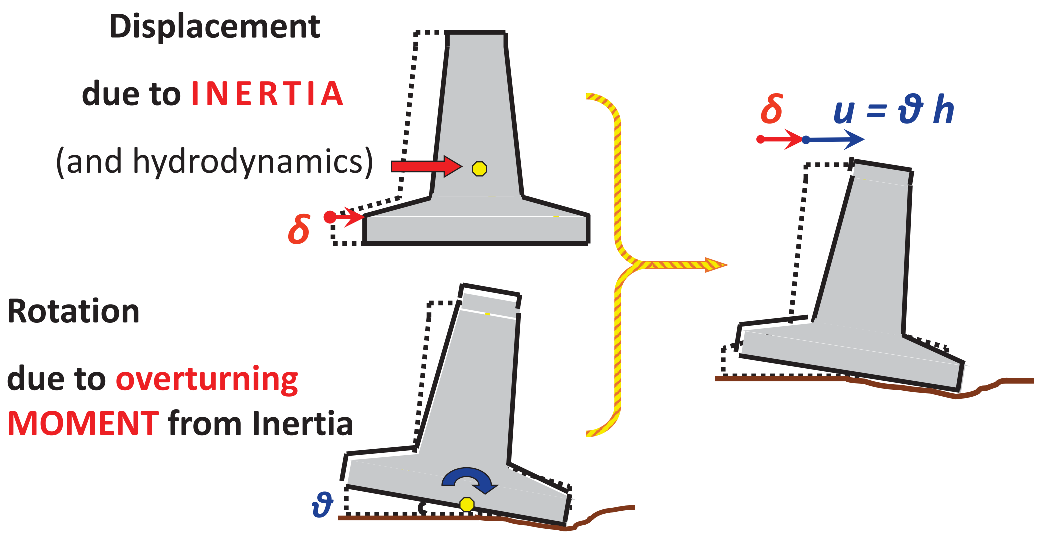
Cables (per pylon)
m = 1350 t

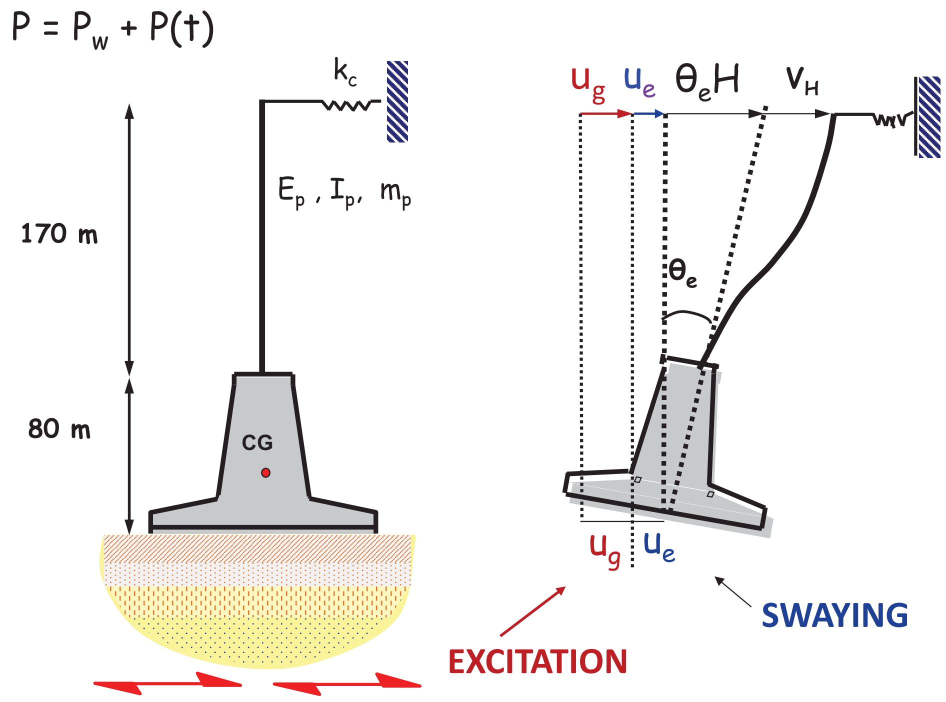
Deck (per pylon)
m = 18 400 t

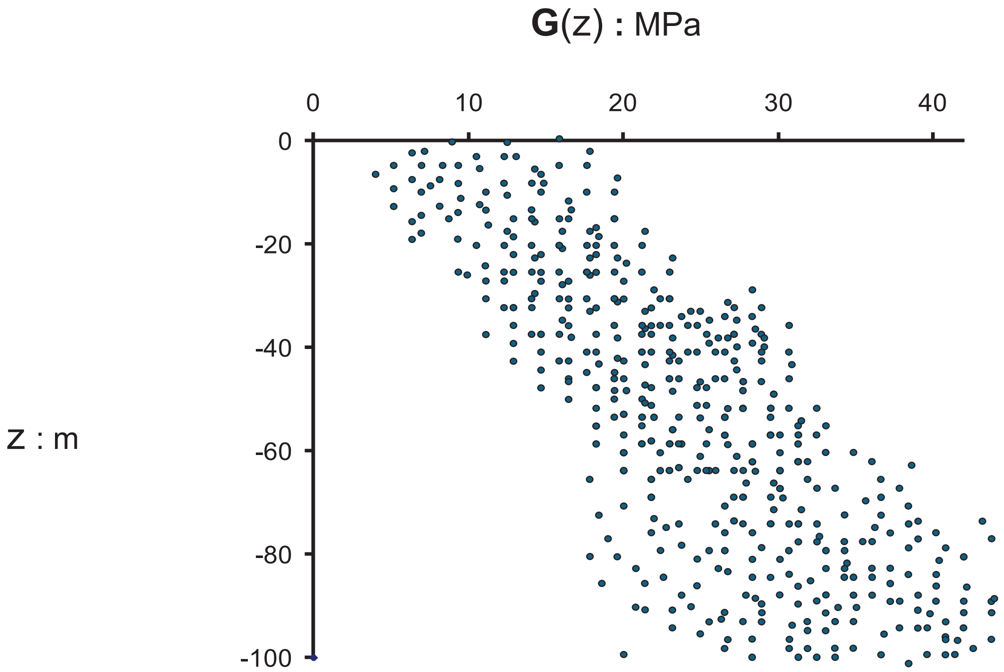
Total mass of superstructure
m = 31 000 t

Lower Pylon + Foundation
m = 160 000 t

Mechanisms of PYLON–FOOTING Motion







With these $\left(K_{\vartheta}\ K_{H}\right)$ the

NATURAL PERIOD of the pylon is determined

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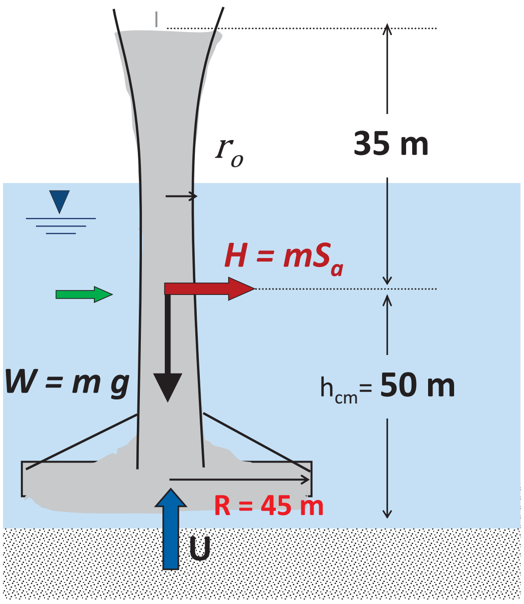

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Uplift force: $\boldsymbol{U} = \mathbf{1\,100}\ \boldsymbol{MN}$

→ Effective weight: $\overline{\boldsymbol{W}} = \boldsymbol{W} - \boldsymbol{U} = \mathbf{800}$ MN

→ Effective pressure: $\overline{\boldsymbol{q}_{av}} = \overline{\boldsymbol{W}}/\boldsymbol{\pi R^2} = \mathbf{125}$ kPa

Inertia force: $\boldsymbol{H} = \boldsymbol{mS_a} = (\boldsymbol{W}/\boldsymbol{g})\boldsymbol{S_a} = \mathbf{550}$ MN

(acceleration is applied to the ***total mass***)

Hydrodynamic force: $\boldsymbol{H_{hydr}} = \mathbf{0.6}\boldsymbol{\rho_w\, \pi\, r_o}^{\mathbf{2}} = \mathbf{300}$ MN

Base shear: $\boldsymbol{Q_{base}} = \mathbf{550} + \mathbf{300} = \mathbf{850}$ MN

Rough approximation: $h_{hydr} \approx h_{CM} \approx 50$ m

Base moment: $\boldsymbol{M_{base}} = \mathbf{850} \cdot \mathbf{50} \approx \mathbf{42\,500}$ MNm

The contribution of SOIL COMPLIANCE (K_M K_H) to the

NATURAL PERIOD of the pylon is so decisive,

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Ultimate Limit State Design Analysis of Foundations and Representative Strength of Soils in the new EC7

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SOMELAB
SOil MEchanics LABoratory - Civil Engineering Department

Bucuresti, March 2025

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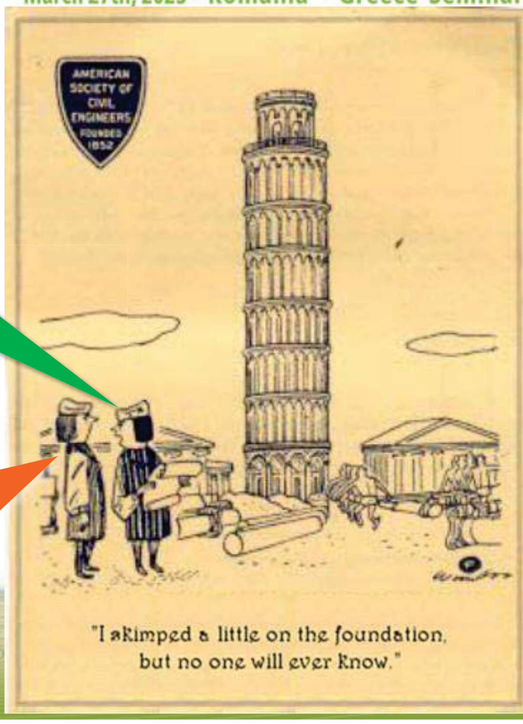
Eurocode 2nd Generation is for
<u>Structural and Geotechnical Design</u>

I skimped a little on the foundation, but no one will ever know
(ASCE, 1962)

How **<u>on earth</u>** does the foundation meet EN 1990 and EN 1997 reliability criteria?

?



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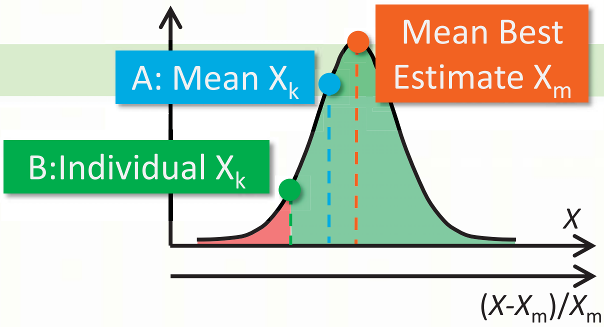

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Soil Properties – Basic Definitions (EN 1990 and EN 1997)

Characteristic Strength, X_k	The **95% probability** of the **mean or inferior/superior value** based on a hypothetical unlimited test series → **statistical method**
Nominal Strength, X_{nom}	A **cautious estimate** of the **mean or inferior/superior value** based on previous experience on similar conditions → **empirical method**
Representative Strength, X_{rep}	$= \eta \times X_k$ or $\eta \times X_{nom}$ η: conversion factor for scale effects, moisture/temperature effects, material ageing effects, and any other relevant parameter
Design Strength, X_d	$= X_{rep} / \gamma_M$ γ_M : partial material factor (for MFA approach)

New term!

harmonized with EN 1990!

Soil Strength – Definitions – Steps *(EN 1990 and EN 1997-1)*

Determination by **statistical analysis**	Selection by **knowledge** of the construction site and **experience** in comparable cases
Characteristic Value: X_k	NEW term! **Nominal** (empirical) **Value: X_{nom}**

$X_{rep} = X_k$ — **Representative Value: X_{rep}** — $X_{rep} = X_{nom}$

$X_{rep} = \eta X_k$ — conversion factor (usually, $\eta = 1$) — $X_{rep} = \eta X_{nom}$

Design Value: $X_d = X_{rep} / (\gamma_M k_M)$ … if needed

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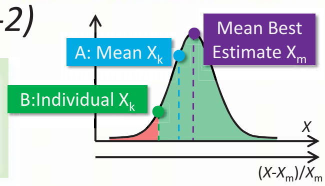
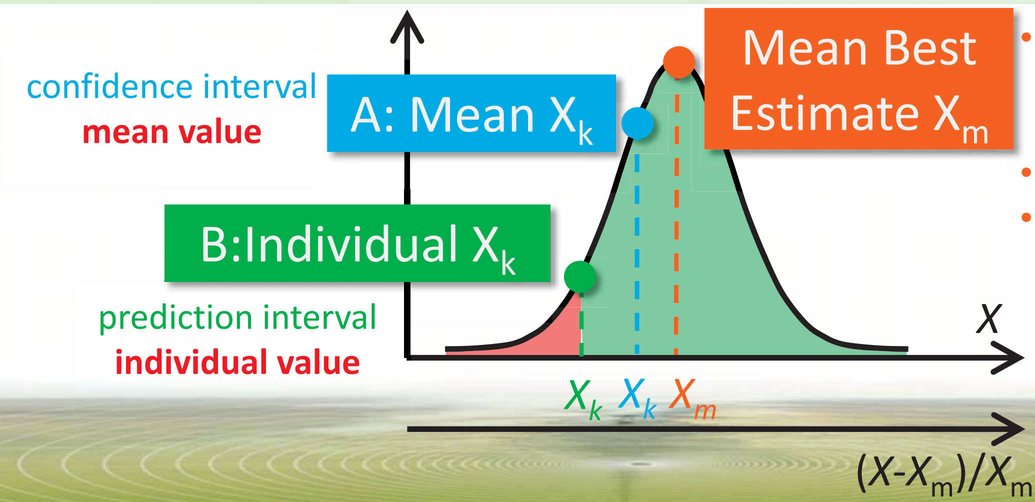
Soil Properties – Statistical Characteristic Strength *(EN 1990 and EN 1997-2)*



Two types of **characteristic values** X_k:

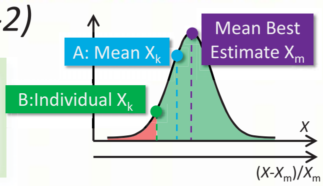
A: Characteristic value of the mean

B: Inferior (5%) or superior (95%) value



- Statistical methods (e.g. mean / median / most probable)
- Back analysis
- Empirical methods

Soil Properties – Statistical Characteristic Strength *(EN 1990 and EN 1997-2)*



Two types of characteristic values X_k:

A: Characteristic value of the mean	B: Inferior (5%) or superior (95%) value

General formula (EN 1997, adopted from EN 1990)

$$X_k(n) = X_m(1 \mp k_n V_X)$$

- Variation Coefficient (statistical sample value): $V_X = \dfrac{s_X}{X_m}$
- Statistical distribution coefficient for the required confidence level: k_n formula depends on V_x: "known", "assumed" or "unknown"
- Sample mean value of X (statistical sample value): $X_m = \dfrac{1}{n}\sum_{i=1}^{n} X_i$

➢ X_i: value of i-sample

➢ n: number of all the X_i sample derived values)

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Soil Properties – How is X_k Determined? *(EN 1990 and EN 1997-2)*

$$X_k(n) = X_m(1 \mp k_n V_X)$$

Three cases are considered: NEW!

Case 1 "V_X known": When V_X is known from prior knowledge

Might come from evaluation of previous tests in comparable conditions. The "comparable" can be determined by Engineering judgement

Case 2 "V_X assumed": When designer uses the indicative values in Table A.2, for ground parameters, or Table A.3, for test parameters

Case 3 "V_X unknown": When derived values for ground or tests parameters are used

Soil Properties – How is k_n Determined? *(EN 1990 and EN 1997-2)*

$$X_k(n) = X_m(1 \mp k_n V_x)$$

V_x Cases	statistical characteristics	X_k Types: A: mean value	X_k Types: B: inferior (5%) or superior (95%)
Case 1 "V_x known" Case 3 "V_x unknown"	• normal distribution • 95% confidence level • infinite degrees of freedom	$k_n = N_{95}\sqrt{\frac{1}{n}}$	$k_n = N_{95}\sqrt{1+\frac{1}{n}}$
Case 2 "V_x assumed"	• Student's t distribution • 95% confidence level • n-1 degrees of freedom	$k_n = t_{95,n-1}\sqrt{\frac{1}{n}}$	$k_n = t_{95,n-1}\sqrt{1+\frac{1}{n}}$
		confidence interval **mean value**	prediction interval **individual value**

Type A or B choice depends on which value dominates the behaviour of the problem

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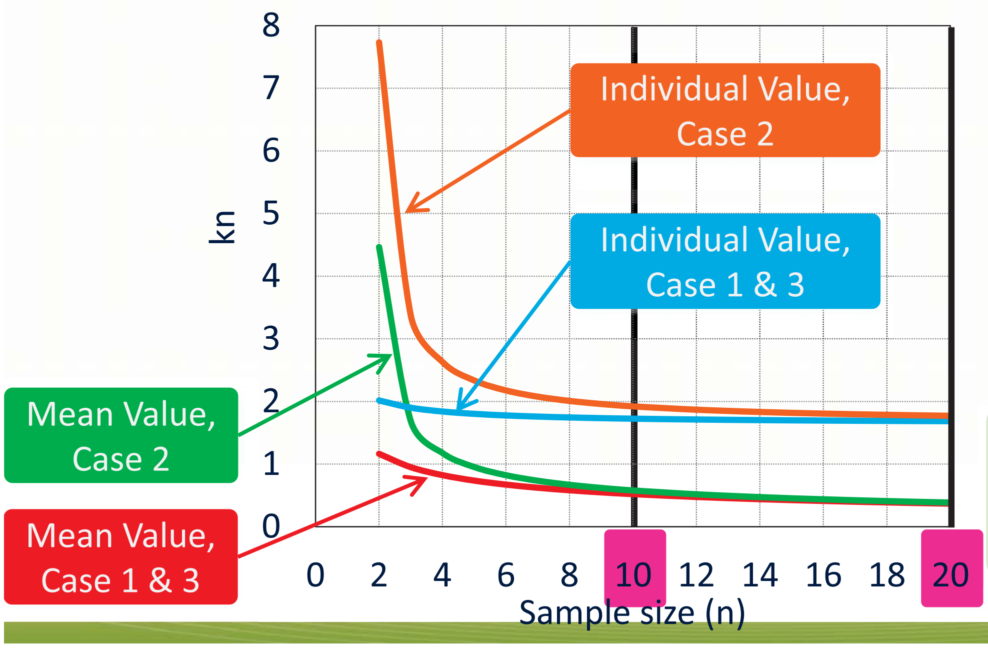
Soil Properties – k_n dependance on sample size n *(EN 1990 and EN 1997)*



$$X_k(n) = X_m(1 \mp k_n V_x)$$

sample size:

- n = 10 seems very good
- n =20 would be wonderful!

Soil Properties – How is V_x Determined? *(EN 1997-2)*

$$X_k(n) = X_m(1 \mp k_n V_x)$$

Case 2 “V_X assumed”

Indicative values for <u>ground properties</u>:

Soil / Rock Type	Ground property	Symbol	V_x (%)
All soils and rocks	Weight density	γ	5-10
Fine-grained soils	Undrained shear strength	c_u	30-50
All soils and rocks	Peak or residual cohesion	c_p' or c_r'	30-50
All soils and rocks	Angle of shearing resistance	ϕ	5-15
All soils and rocks	Unconfined compressive strength	q_u	20-80
All soils	Deformation modulus	E or G	20-70
Fine-grained soils	Vertical or horizontal consolidation coefficient	c_v or c_h	30-70

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Soil Properties – How is V_x Determined? *(EN 1997-2)*

$$X_k(n) = X_m(1 \mp k_n V_x)$$

Case 2 “V_X assumed”

Indicative values for <u>test parameters</u>:

Soil / Rock Type	Ground property	Symbol	V_x (%)
All soils	SPT blows	N_{SPT}	15-45
All soils	CPT cone resistance	q_c	5-15
All soils	CPT sleeve friction	f_s	5-15

Case 3 “V_X unknown”

By calculation from: $V_X = \dfrac{s_X}{X_{mean}}$

Mean value (X_{mean}): $X_m = \dfrac{1}{n}\sum_{i=1}^{n} X_i$

Standard deviation <u>of the sample</u> derived values (s_x): $s_x = SD_x = \sqrt{\dfrac{1}{n-1}\sum_{i=1}^{n}(X_i - X_{mean})^2}$

Single parameter model, e.g. undrained strength c_u

Soil Properties – How is X_k expressed in Terms of Standard Error?

Type A: mean value

$$X_k(n) = X_m(1 \mp k_n V_X) = X_m\left(1 \mp \frac{a_{95}}{\sqrt{n}} \frac{s_X}{X_m}\right) = X_m \mp a_{95}\frac{s_X}{\sqrt{n}} = X_m \mp a_{95} SE_{Xm}$$

A lower or upper Estimate of the **population mean value** for 95% probability of no exceedance

$a_{95} = N_{95}$ or $t_{95,n-1}$

Type B: inferior (5%) or superior (95%)

$$X_k(n) = X_m(1 \mp k_n V_X) = X_m\left(1 \mp a_{95}\sqrt{\frac{n+1}{n}} \frac{s_X}{X_m}\right) = X_m \mp a_{95}\sqrt{n+1}\frac{s_X}{\sqrt{n}} = X_m \mp a_{95}\sqrt{n+1} SE_{Xm}$$

An estimate of the lower or upper **individual value** for 95% probability of no exceedance

$a_{95} = N_{95}$ or $t_{95,n-1}$

! By knowing the **mean value** and its **standard deviation** we can also apply **probabilistic analysis**

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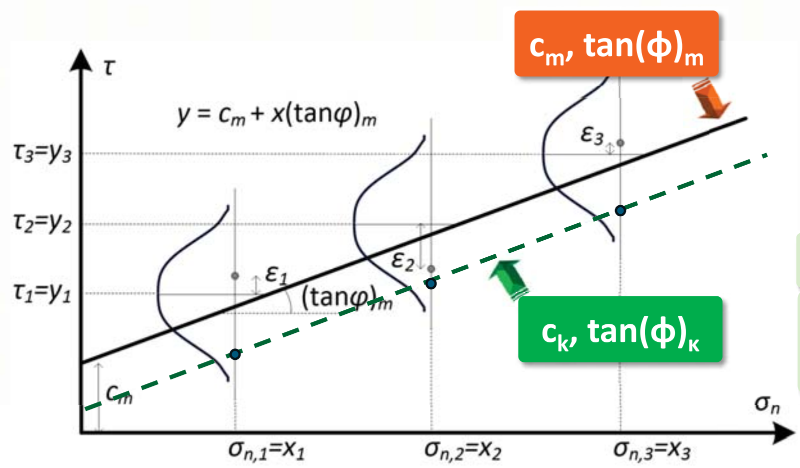
Soil Properties – Mohr – Coulomb Example on Direct Shear



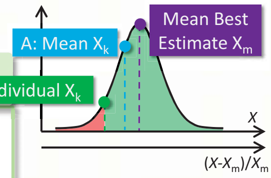
(figure based on Belokas, 2020)

M- C Failure Criterion: $\boldsymbol{\tau = c_i' + \sigma_{ni} \tan\varphi'}$

*The **sample** can be **treated as set** to give the derived characteristic values*
(e.g. Belokas, 2020)

Two parameters model	
variables	Independent (imposed): σ_n Measured (response): τ

Soil Properties – Mohr – Coulomb Example on Direct Shear



angle of shearing resistance

$$\left(\tan\varphi\right)_m = b_m = \frac{\sum_{i=1}^{n}\left(x_i - x_m\right)\left(y_i - y_m\right)}{\sum_{i=1}^{n}\left(x_i - x_m\right)^2} = \frac{\sum_{i=1}^{n} x_i y_i - \frac{1}{n}\sum_{i=1}^{n} y_i \sum_{i=1}^{n} x_i}{\sum_{i=1}^{n} x_i^2 - \frac{1}{n}\left(\sum_{i=1}^{n} x_i\right)^2}$$

$$SE_{\tan\varphi} = SE_b = \sqrt{\frac{1}{n-2}\frac{\sum_{i=1}^{n}\varepsilon_i^2}{\sum_{i=1}^{n}\left(x_i - \bar{x}\right)^2}}$$

cohesion

$$c_m = a_m = y_m - b_m x_m$$

$$SE_c = SE_a = SE_b\sqrt{\frac{1}{n}\sum_{i=1}^{n} x_i^2}$$

Type A: mean value	$\left(\tan\varphi\right)_k = \left(\tan\varphi\right)_m - \alpha_{95} SE_{(\tan\varphi)}$	$c_k = c_m - \alpha_{95} SE_c$	*Case 1, 3:* $a_{95} = N_{95}$ *or Case 2:* $t_{95,n-2}$
Type B: inferior (5%) or superior (95%)	$\left(\tan\varphi\right)_k = \left(\tan\varphi\right)_m - \alpha_{95}\sqrt{n+1}SE_{(\tan\varphi)}$	$c_k = c_m - \alpha_{95}\sqrt{n+1}SE_c$	

(adopted from Belokas, 2020 & Belokas, 2023)

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Soil Properties – Mohr – Coulomb Example on Direct Shear

Data of granular fill material from a quarry in Attica prefecture Greece.

The largest direct shear device in Greece and one of the largest in the world designed by Dr M. Bardanis and operated in EDAFOS SA – Laboratory

*(**data from** Bardanis, personal communication, **analysis in** Belokas, 2023)*


Photo by M. Bardanis


Photo by M. Bardanis


Photo by M. Bardanis

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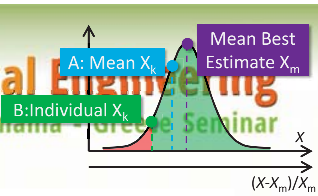
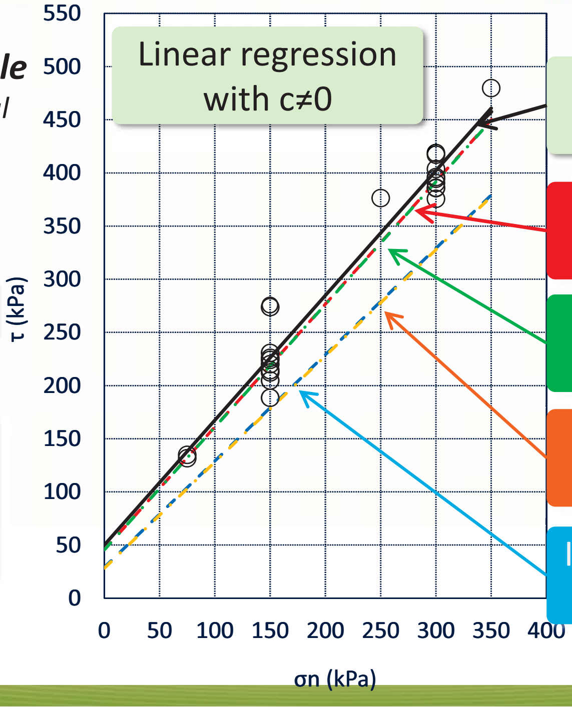


Soil Properties – Mohr – Coulomb Example

*(**data from** Bardanis, personal communication, **analysis in** Belokas, 2023)*

23 data points

practically insensitive of case, i.e. type of distribution



Mean value, c_m=50 kPa, ϕ_m=49.6°

Mean Char Value, Case 1 & 3: c_k=46 kPa, ϕ_k=53.1°

Mean Char Value, Case 2: c_k=46 kPa, ϕ_k=49.1°

Individual Char Value, Case 2: c_k=28 kPa, ϕ_k=47.2°

Individual Char Value, Case 1 & 3: c_k=29 kPa, ϕ_k=47.3°

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Soil Properties – Design values for MFA

$$X_d = X_{rep} / (K_M \gamma_M)$$

Revised!

Consequence Classes according to EN 1990

Qualitative Criterion	MIN	MAX
Loss of human life or personal injury	very low (CC0)	extreme (CC4)
Economic, social or environmental consequences	insignificant (CC0)	Huge (CC4)

EN 1997-1:
- **adopts** CC1, CC2, CC3 and CC4
- **gives classifications** for CC1, CC2 and CC3

NEW: harmonized with EN 1990!

You get

two CC

Change!

Most unfavourable CC

You choose

Soil Properties – Design values for MFA

$$X_d = X_{rep} / (K_M \gamma_M)$$

Revised!

Consequence Classes according to EN 1990

Qualitative Criterion	MIN	MAX
Loss of human life or personal injury	very low (CC0)	extreme (CC4)
Economic, social or environmental consequences	insignificant (CC0)	Huge (CC4)

NEW!

CC0: not used
CC1: $k_M = 0.9$
CC2: $k_M = 1.0$
CC3: $k_M = 1.1$

EN 1997-1:

- **adopts** CC1, CC2, CC3 and CC4
- **gives classifications** for CC1, CC2 and CC3

NEW: harmonized with EN 1990!

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Soil Properties – Guidelines

Many more details *can be found in this new publication by Joint Research Centre:*

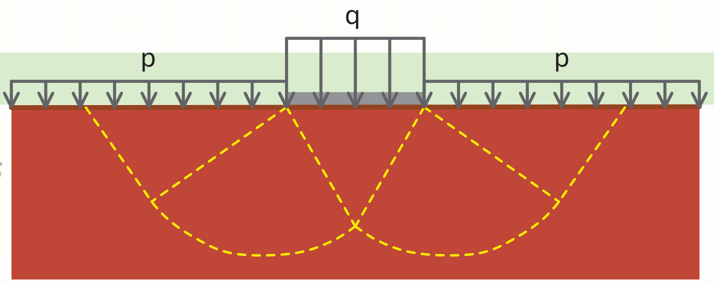
European Commission: Joint Research Centre, Orr, T., Sorgatz, J., Estaire, J., Prästings, A. et al., ***Determination of representative values from derived values for verification with limit states with EN 1997 – Guidelines for the application of the 2nd generation of Eurocode 7 – Geotechnical design***, Estaire, J(editor), Publications Office of the European Union, 2025

!

https://eurocodes.jrc.ec.europa.eu/learning-corner/publications

- *Currently 3 guidelines for EN 1997*
- ***Visit regularly*** *for the upcoming EN 1997 supporting material!*

Contents



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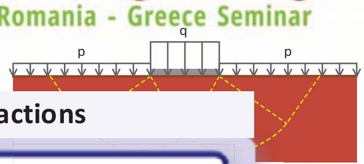
ULS Partial Factors – Definitions



Definitions of partial factors of safety approaches for strength and actions

Material Factor Approach (γ_M, M1, M2) **or** **Resistance Factor Approach** (γ_R)

$$R_d = R\{X_d; a_d; \Sigma F_{Ed}\} = R\{\eta X_k/\gamma_M; a_d; \Sigma F_{Ed}\}$$

$$R_d = R\{X_{rep}; a_d; \Sigma F_{Ed}\}/\gamma_R = R\{\eta X_k; a_d; \Sigma F_{Ed}\}/\gamma_R$$

γ_M "inside"

γ_M = value*K_M or value

RFA: γ_R = value*K_R or value

γ_R "outside"

Partial Factors on Actions (VC1, VC2, VC3) **or** **Partial Factors on Effects of Actions** (VC4)

$$E_d = E\{\Sigma F_d; a_d; X_{Rd}\} = E\{\Sigma(\gamma_F \psi F_k); a_d; X_{Rd}\}$$

$$E_d = \gamma_E E\{\Sigma F_{rep}; a_d; X_{rep}\} = \gamma_F E\{\Sigma(\psi F_k); a_d; X_{Rd}\}$$

Verification Cases: VC1, VC2, VC3

γ_F "inside"

$\gamma_F = \gamma_G$ or γ_Q

γ_G = value*k_F or value or γ_Q = value*k_F or value

Verification Case: VC4

γ_E = value*k_F or value (VC4)

γ_E "outside"

Chart in JRC Report. 2025. TG B2 – Design Examples (to be published)

MFA, RFA, VCs definitions new!

! *The general equations of R_d and E_d are **simplified** and **separate** for each **factor approach** and **verification case***

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ULS Partial Factors – MFA

Design Situations (1990-1: 5.2(3), 1997-1: 4.2.2, 1997-3: 5.2.1)

Persistent	Transient	Accidental	Seismic	Fatigue

Partial factors for Material Factor Approach (MFA) (1997-1: 4.4.1.3)

Persistent Design Situations: Table 4.8(NDP)

M1 (γ_M)	M2 (γ_M)	
γ_M = 1.00 *(all ground properties)*	*Soils:*	*Rock/Rock mass:*
	γ_{tf}=1.25k_M	γ_{tr} =1.25k_M
	$\gamma_{\tan\varphi,p}$=1.25k_M	γ_{qu} =1.4k_M
	$\gamma_{c,p}$=1.25k_M	*Rock Discontinuity:*
	$\gamma_{\tan\varphi,cs}$=1.1k_M	$\gamma_{\tau dis}$ =1.25k_M
	$\gamma_{\tan\varphi,r}$=1.1k_M	$\gamma_{\tan\varphi dis,r}$=1.1$k_M$
	$\gamma_{c,r}$=1.1k_M	*Interface:*
	γ_{cu} = γ_{qu} = 1.4k_M	$\gamma_{\tan\delta}$ =1.25k_M

Accidental Design Situations: (NDP)

$\gamma_{M,acc} = (\gamma_M)^{0.5}$

Transient Design Situations: (NDP)

$\gamma_{M,tr} = k_{tr}\gamma_M \geq 1$

(default: k_{tr}=1.0, optional by NA: k_{tr} < 1.0)

Notes:
- MFA not used for pile axial resistance of single piles
- X_d may be determined directly (EN 1990-1: 8.3.6(2))

Consequence Classes (EN 1997-1: Table 4.9(NDP))

CC3: k_M = 1.1, CC2: k_M = 1.0, CC1: k_M = 0.9

Chart in JRC Report. 2025. TG B2 – Design Examples (to be published)

k_M is NEW!

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ULS Partial Factors – RFA

γ_R values

- defined in EN1997-3
- depend on geostructure type

k_R not used

Chart in JRC Report. 2025. TG B2 – Design Examples (to be published)

Partial factors for Resistance Factor Approach (RFA) (EN 1997-3)

Fundamental (persistent and transient) **Design Situations (γ_R):** (NDP)

Values of γ_R are given in the various clauses of EN 1997-3

Accidental Design Situations: (NDP)

$\gamma_{R,acc} = (\gamma_R)^{0.5}$

Transient Design Situations: (NDP)

$\gamma_{R,tr} = k_{tr}\gamma_R \geq 1$

$\gamma_{R,acc}$, $\gamma_{R,tr}$ symbols are not included in Eurocode

γ_R values given in EN1997-3 for Fundamental (persistent and transient) **Design Situations:**

Slopes, cuttings, and embankments
- RFA not used for verification (4.6.3)
- Supporting elements (4.6.2 and clauses 6 to 11)
- **Foundations Bearing and Sliding Resistance** (5.6.6)
- Resistance: γ_{RN} = 1.40, γ_{RT} = 1.1, $\gamma_{RT,face}$ = 1.4 Table 5.2(NDP)

Single Pile Axial Compressive Resistance:
- Model factor $\gamma_{Rd,pile}$ for Verification by
  a) Calculation: for Ground Model Method (GMM) or Model Pile Method (MPM) by Table 6.4(NDP)
  b)Testing: for tests & soil type by Table 6.5(NDP)
- Resistance γ_R: for GMM or MPM by Table 6.9 (NDP), 6.10 (NDP) and 6.11 (NDP)
- Representative drag forces and transverse ground loads: load factors $\gamma_{F,drag}$ and $\gamma_{F,tr}$ (6.6.4.1 (2)) given by Table 6.9 (NDP), 6.10 (NDP) and 6.11 (NDP)

Pile Group & Raft Axial Compressive Resistance
- Model $\gamma_{Rd,group}$= 1.0 & $\gamma_{Rd,raft}$= 1.0 or by NA (6.6.3(4))
- Resistance: $\gamma_{R,group}$= 1.4 & $\gamma_{R,raft}$= 1.4 Table 6.12 (NDP)
- RFA not used for combined axial & transverse

Gravity walls
- Overall resistance (7.6.2): Model Factors (or by NA)
- $\gamma_{Rd,retain}$ = 1.2 (persistent, sensitive structures), 1.05 (transient) or 1.0 (deep failure mechanisms)

Gravity walls Bearing and sliding resistance
- Resistance: γ_{RN} = 1.40, γ_{RT} = 1.1 Table 7.2 (NDP)

Embedded walls Bearing/rotational resistance
- Resistance: γ_R = 1.40 (vertical), γ_{Re} = 1.4 (passive) Table 7.2 (NDP)
  Note: model factors for sloped ground (7.6.2(6)), supporting elements (7.6.7)

Basal heave of embedded walls
- Resistance: γ_R = 1.40 Table 7.2 (NDP) & (Annex D)

Reinforced fill structures Resistance
- Model factor for tensile resistances of: a) $\gamma_{Rd,gs}$ for geosynthetics (9.6.2.1), b) $\gamma_{Rd,pwm}$ for polymeric coated woven wire mesh (9.6.2.3)
- Resistance for pull-out and direct shear: $\gamma_{R,po}$ = 1.25, $\gamma_{R,po}$ = 1.25, Table 9.4 (NDP)
- Resistance for rupture of a) reinforcing element & b) connection to failures in Table 9.4(NDP) (MFA & RFA)

Anchors (Clause 8), **Soil nailed structures** (Clause 10, Annex G.3), **Rock bolts and rock surface support** (Clause 11, Annex H.3), **Ground Improvement** (Clause 12)

Consequence Classes: k_R **not used** in EN 1997-3

ULS Partial Factors – VCs

VC new definitions !

Verification Cases: partial factor on Actions (γ_F) and Effects (γ_E) (EN1990-1: Table A.1.8)

Structural and Geotechnical Design	**Static equilibrium** (EN1997-1: 8.1.3.1) **and uplift** (EN1997-1: 8.1.3.2)		**Geotechnical design**	
VC1	VC2a	VC2b	VC3	VC4
$\gamma_G = 1.35k_F$	$\gamma_G = 1.35k_F$	$\gamma_G = 1.00$	$\gamma_G = 1.00$	G_k not factored
$\gamma_{Gw} = 1.20k_F$	$\gamma_{Gw} = 1.20k_F$	$\gamma_{Gw} = 1.00$	$\gamma_{Gw} = 1.00$	
$\gamma_{G,stb}$ = not used	$\gamma_{G,stb} = 1.15$	$\gamma_{G,stb} = 1.00$	$\gamma_{G,stb}$ = not used	
$\gamma_{Gw,stb}$ = not used	$\gamma_{Gw,stb} = 1.00$	$\gamma_{Gw,stb} = 1.00$	$\gamma_{Gw,stb}$ = not used	
$\gamma_{G,fav} = 1.00$	$\gamma_{G,fav} = 1.00$	$\gamma_{G,fav} = 1.00$	$\gamma_{G,fav} = 1.00$	
$\gamma_Q = 1.50k_F$	$\gamma_Q = 1.50k_F$	$\gamma_Q = 1.50k_F$	$\gamma_Q = 1.30$	$\gamma_{Q,1}/\gamma_{G,1} = 1.11/k_F$
$\gamma_{Qw} = 1.35k_F$	$\gamma_{Qw} = 1.35k_F$	$\gamma_{Qw} = 1.35k_F$	$\gamma_{Qw} = 1.15$	$\gamma_{Qw} = 1.00$
$\gamma_{G,fav} = 0$	$\gamma_{G,fav} = 0$	$\gamma_{G,fav} = 0$	$\gamma_{G,fav} = 0$	$\gamma_{G,fav} = 0$
γ_E = not applied	γ_E = not applied	γ_E = not applied	γ_E = not applied	$\gamma_E = 1.35k_F$ $\gamma_{E,fav} = 1.00k_F$

Consequence Classes (EN 1990: Table A.1.9(NDP)): CC3: $k_F = 1.1$, CC2: $k_F = 1.0$, CC1: $k_F = 0.9$

Chart in JRC Report. 2025. TG B2 – Design Examples (to be published)

k_F is NEW!

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ULS Partial Factors – Summary of Combinations and Consequence Class

MFA

(a): VC1(γ_F) or VC4 (γ_E) + M1(γ_M=1)
and
(b): VC3(γ_F)+M2(γ_M)

(c): VC1(γ_F) + M2(γ_M)

RFA

(d): VC1(γ_F) + γ_R

(e): VC4(γ_E) + γ_R

Choice in National Annex

Actions: $F_d = F_k \times (k_F \times \gamma_F)$ **VC1**

$F_d = F_k \times \gamma_F$ **VC3**

Effects of Actions: $E_d = E_{rep} \times (k_F \times \gamma_E)$ **VC4**

! **no k_F on VC3**

Strength: $X_d = F_{rep} / (k_M \times \gamma_M)$ **MFA**

Resistance: $R_d = R_{rep} / \gamma_R$ **RFA**

! **no k_R on RFA**

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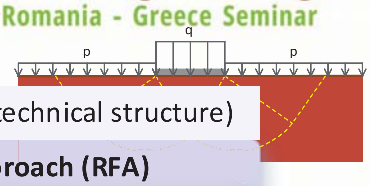
ULS Partial Factors – Combinations



Partial factors combinations for ULS design check (NA selects <u>one</u> for each geotechnical structure)

Material Factor Approach (MFA)		Resistance Factor Approach (RFA)	
(a): VC1(γ_F) or VC4 (γ_E) + M1(γ_M=1) <u>and</u> (b): VC3(γ_F)+M2(γ_M)	(c) VC1 (γ_F) + M2 (γ_M) (note: only for spread foundations and gravity retaining walls)	(d): VC1 (γ_F) + (γ_R)	(e) VC4 (γ_E) + (γ_R)

Improved definitions!

Chart in JRC Report. 2025. TG B2 – Design Examples (to be published)

Direct correspondence to 1st Generation

DA1	DA3	DA2	DA2*

Practically the same!

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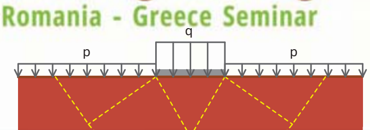
ULS Partial Factors – Consequence (Class) Factors



<u>CONSEQUENCE CLASS (CC)</u>: (EN 1997-1, Table 4.3)

CC3	CC2	CC1
foundations supporting public buildings, with high exposure	if CC1 & CC3 **not** applicable	foundations supporting buildings with low occupancy

<u>GEOTECHNICAL COMPLEXITY CLASS (GCC)</u>: (EN 1997-1, Table 4.3 and Annex C)

GCC3	GCC2	GCC1
if **any** applies: considerable ground uncertainty highly variable/difficult ground high ground and surface water sensitivity high ground – structure interaction complexity	if GCC1 & GCC3 **not** applicable	**all** following apply: negligible ground uncertainty uniform ground low ground and surface water sensitivity low ground – structure interaction complexity

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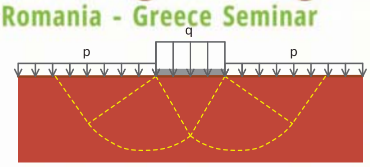
ULS Partial Factors – Consequence (Class) Factors



An example on CC and GCC:

foundation for public building → CC3

significant uncertainty of ground conditions → GCC3

EN 1997-1

Actions: $k_F = 1.1$

Strength (MFA): $k_M = 1.1$

Resistance (RFA): k_R = n/a

consequence factors

	GCC1 (lower)	GCC2 (normal)	GCC3 (higher)
CC3 (higher)	GC2	GC3	**GC3**
CC2 (normal)	GC2	GC2	GC3
CC1 (lower)	GC1	GC2	GC2

EN 1997 describes ways to ensure the **required reliability level**

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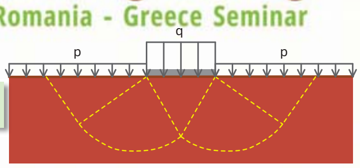
ULS Partial Factors – Summary of Combinations and Consequence Class



Example: Retaining walls and foundations – criteria

MFA: ($\gamma_{RT} = \gamma_{RN}$ = n/a)

(a) **VC4** (γ_G=1.0, γ_E=1.35k_F=1.485) + **M1** (γ_M=1.0) : $\gamma_E \times \gamma_M$ = 1.49

(b) **VC3** (γ_G=1.35k_F=1.485, γ_E=1.0) + **M2** ($\gamma_M = \gamma_{\tan\delta}$= 1.25$k_M$=1.375) : $\gamma_E \times \gamma_M$ = 2.04

(c) **VC1** (γ_G=1.35k_F=1.485, γ_E=1.0) + **M2** ($\gamma_M = \gamma_{\tan\delta}$= 1.25$k_M$=1.375) : $\gamma_E \times \gamma_M$ = 2.04

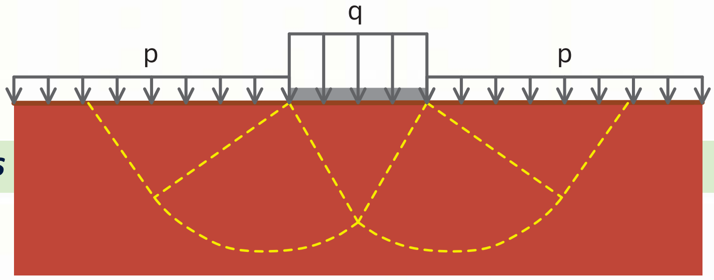
(a) & (b) concurrently or (c) alone

RFA: (γ_M = n/a, γ_{RT} = 1.1)

(e) **VC4** (γ_G=1.0, γ_E=1.35k_F=1.485) : $\gamma_E \times \gamma_{RT}$ = 1.63

(d) **VC1** (γ_G= 1.35k_F=1.485, γ_E=1.0) : $\gamma_E \times \gamma_M$ = 1.63

Contents



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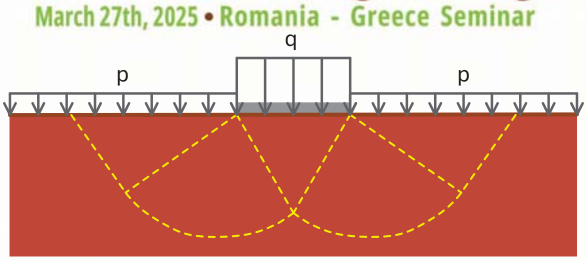
ULS by Bearing Capacity – MFA vs RFA



UNDRAINED $R = f\left[c_u, B/L, B, L, q\right]$

Almost Linear → MFA & RFA practically **equivalent**

DRAINED

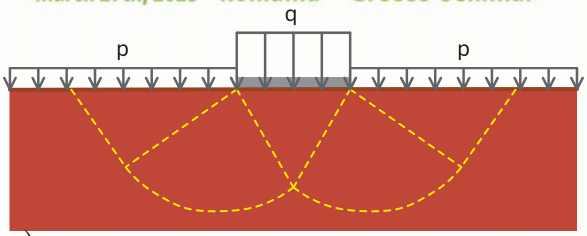
$$R = f\left[\underbrace{c', \tan\varphi', e^{\tan\varphi'}, \tan^2(45+\varphi'/2), \sin\varphi'}_{\text{Soil strength}}, \underbrace{B/L, B, L}_{\text{Geometry}}, \underbrace{q', 0.5\gamma'}_{\text{Geotechnical Loads (permanent)}}\right]$$

Non - linear → MFA may give **different results** from RFA !

See next slides (based on Kovaiou & Belokas, 2023)

ULS by Bearing Capacity – MFA vs RFA

Overdesign Factor: $ODF = R_d/E_d \geq 1$ Frank et al. 2004



UNDRAINED

$$\frac{ODF_{(VC3+M2)}}{ODF_{(VC4+\gamma R)}} = \frac{R\left(c_{u;rep}/\gamma_{M2}\right)/\left(1.00+1.30F_{Q;rep}/F_{G;rep}\right)}{\frac{1}{\gamma_R}R\left(c_{u;rep}\right)/\left(1.35+1.50F_{Q;rep}/F_{G;rep}\right)}$$

A COMPARISON VIA THE ODF

DRAINED

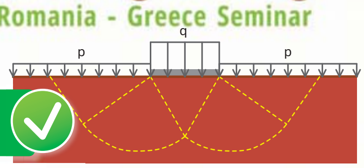
$$\frac{ODF_{(VC3+M2)}}{ODF_{(VC4+\gamma R)}} = \frac{R\left(c_d', \tan\varphi_d', e^{\tan\varphi_d'}, \tan^2\left(45+\varphi_d'/2\right), \sin\varphi_d'\right)/\left(1.00+1.30F_{Q;rep}/F_{G;rep}\right)}{\frac{1}{\gamma_R}R\left(c_{u;rep}\right)/\left(1.35+1.50F_{Q;rep}/F_{G;rep}\right)}$$

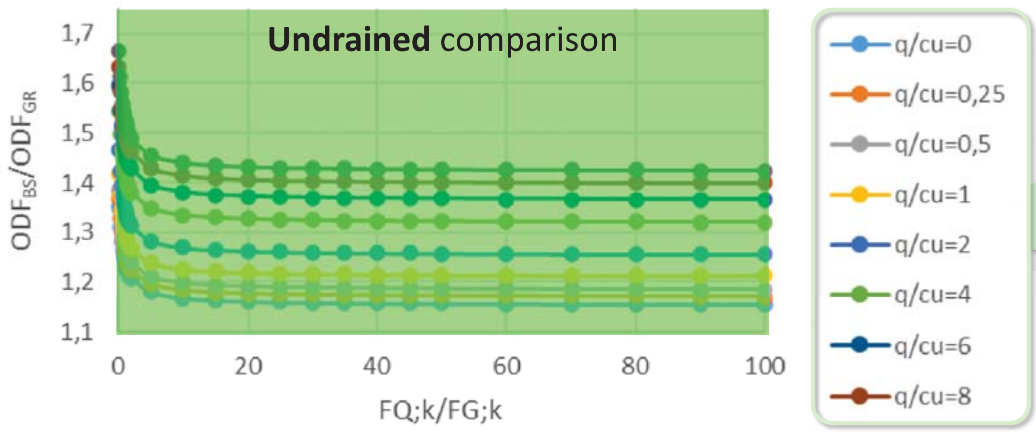
ULS by Bearing Capacity – MFA vs RFA

Graphs of $\frac{ODF_{(VC3+M2)}}{ODF_{(VC4+\gamma R)}}$ vs $\frac{F_{Q;rep}}{F_{G;rep}}$

$> 1 \Rightarrow$ VC3(γ_F) + M2(γ_M) higher capacity

$< 1 \Rightarrow$ VC4(γ_E) + (γ_R) higher capacity

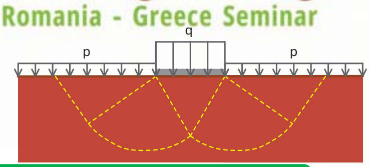




Normalized q/c_u

(based on Kovaiou & Belokas, 2023)

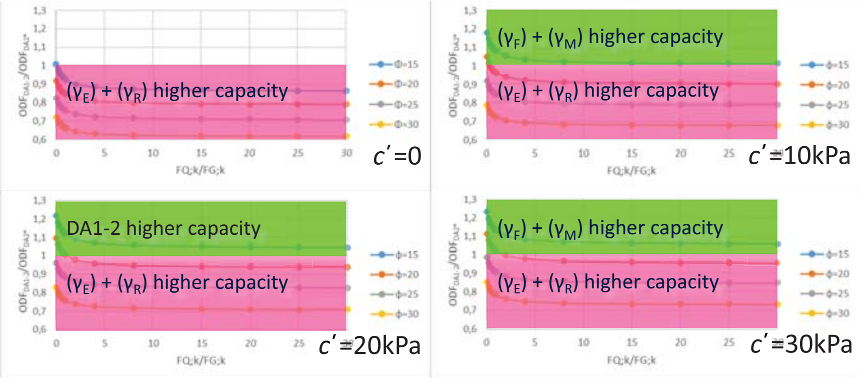
ULS by Bearing Capacity – MFA vs RFA



Graphs of

$$\frac{ODF_{(VC3+M2)}}{ODF_{(VC4+\gamma R)}} \text{ vs } \frac{F_{Q;rep}}{F_{G;rep}}$$

e.g. the **drained** BC comparison for q = 0 kPa:



q = 0 kPa (based on Kovaiou & Belokas, 2023)

VC3(γ_F) + M2(γ_M) gives higher capacity for:

- ϕ = 15 °, c > 0 & q = 0
- ϕ = 15 °, c ≥ 0 & q = 10kPa

VC4(γ_E) + (γ_R) gives higher capacity for:

- ϕ ≥ 20 ° & $F_{Q;k}/F_{G;k}$>1
- ϕ ≥ 25 ° in all cases

! it is the effect of the **non-linear equations**

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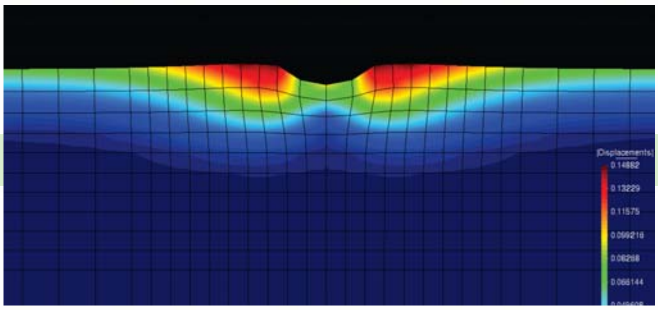
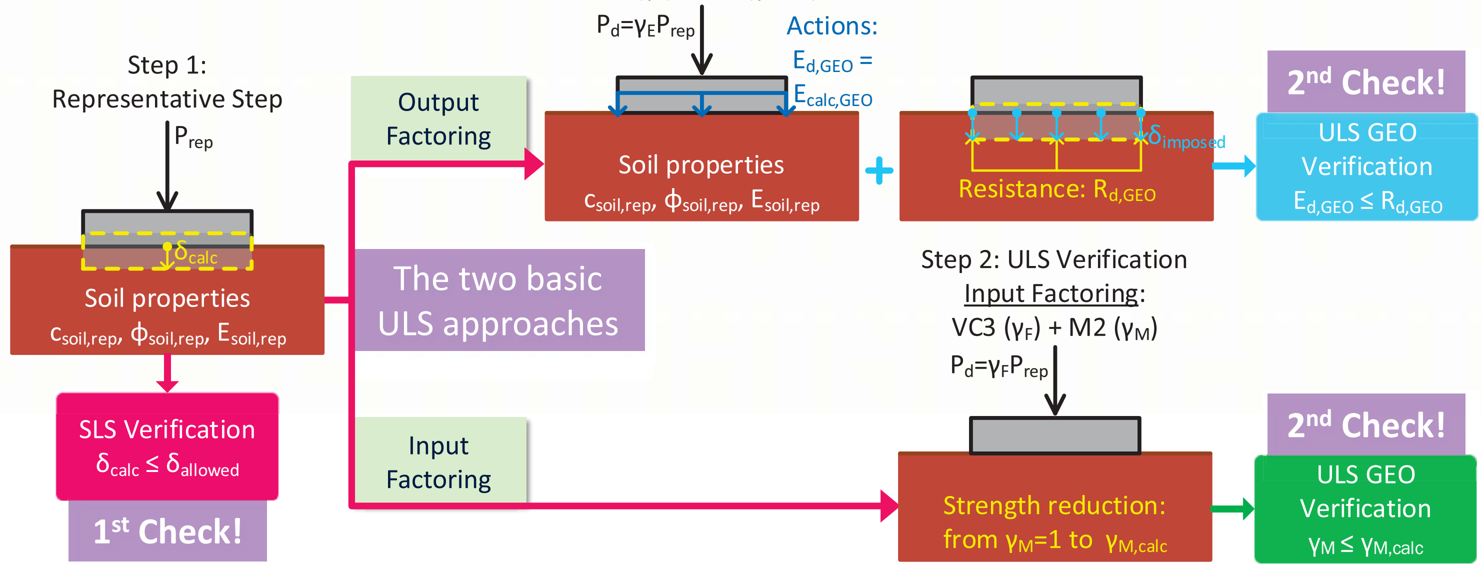
1. *Soil Properties*
2. *ULS Partial Factors of Safety*
3. *ULS by Bearing Capacity Calculation Models*
4. ***ULS by Numerical Models***
5. *Conclusions – Acknowledgements*



ULS by Numerical Models
(EN 1997-1 and EN 1997-3)



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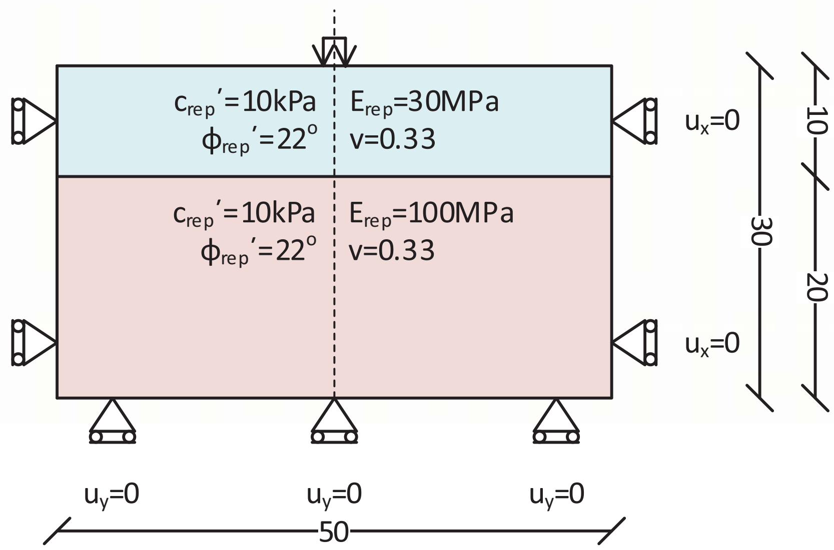
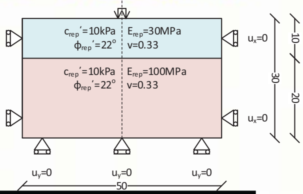
ULS by Numerical Models *(EN 1997-1 and EN 1997-3)*

Simple Bearing Capacity Example:



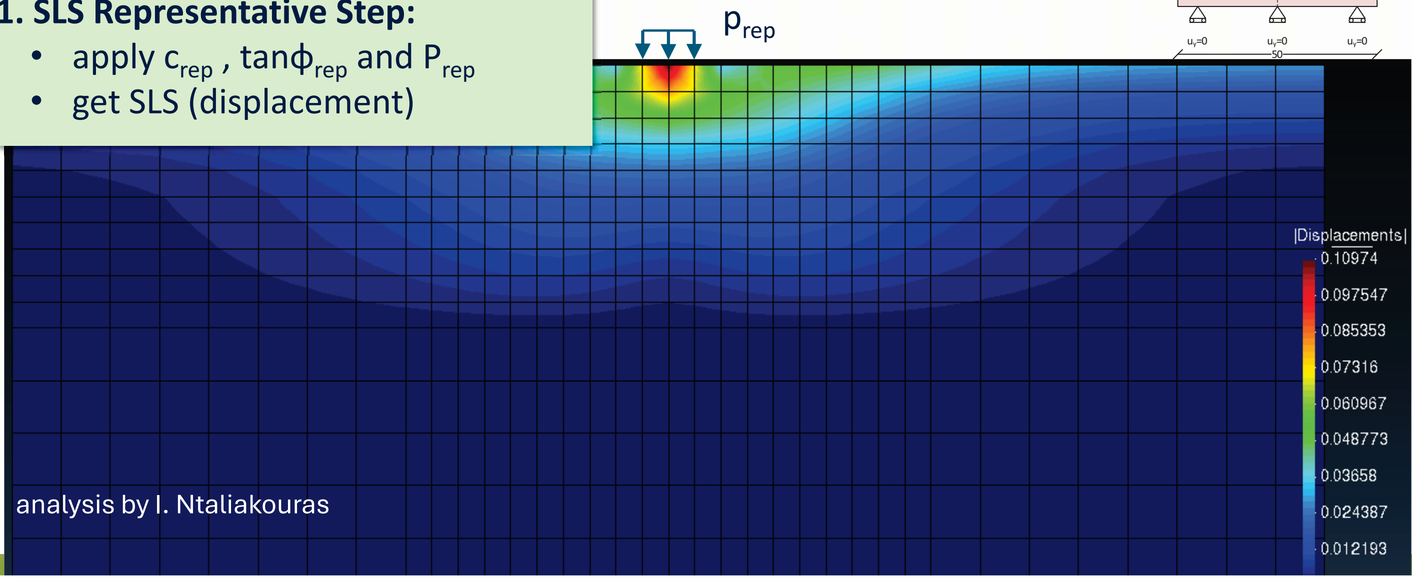
ULS by Numerical Models *(EN 1997-1 and EN 1997-3)*

Simple Bearing Capacity Example:



1. SLS Representative Step:

- apply c_{rep} , tanϕ_{rep} and P_{rep}
- get SLS (displacement)



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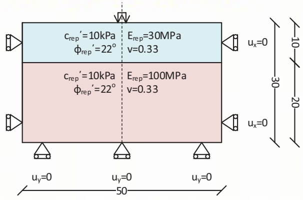
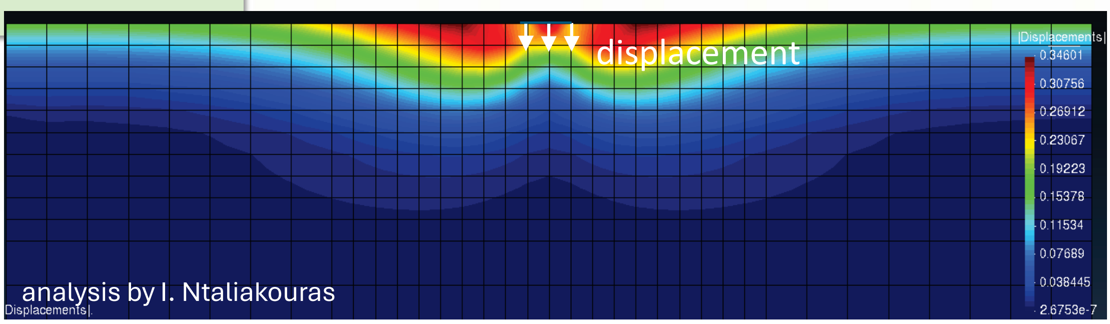
ULS by Numerical Models *(EN 1997-1 and EN 1997-3)*

Simple Bearing Capacity Example:



2. ULS Output Factoring Step

- Apply c_{rep} , tanϕ_{rep} and P_d
- Get load to structure E_d
- Apply displacement
- Get the R_d



Bearing failure mechanism and load by imposed displacement

ULS by Numerical Models (EN 1997-1 and EN 1997-3)

Simple Bearing Capacity Example:

3. ULS Input Factoring Step

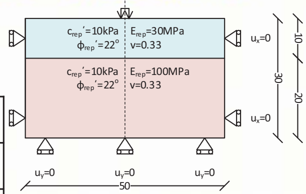
Strength reduction technique

Representative values

c_{rep}'	10	kPa
ϕ_{rep}'	22	o
$\tan\phi_{rep}'$	0.404026	
$E_{rep}'=$	30	Mpa

Example of strength evolution

	c_{calc}'	$\tan\phi_{calc}'$	ϕ_{calc}'
γ_M	c_{rep}'/γ_M	$\tan\phi_{rep}'/\gamma_M$	
1	10	0.4040	22.0
1.1	9.1	0.3673	20.2
1.2	8.3	0.3367	18.6
1.3	7.7	0.3108	17.3
1.4	7.1	0.2886	16.1
1.5	6.7	0.2694	15.1
1.6	6.3	0.2525	14.2
1.7	5.9	0.2377	13.4
1.8	5.6	0.2245	12.7
1.9	5.3	0.2126	12.0
2	5.0	0.2020	11.4



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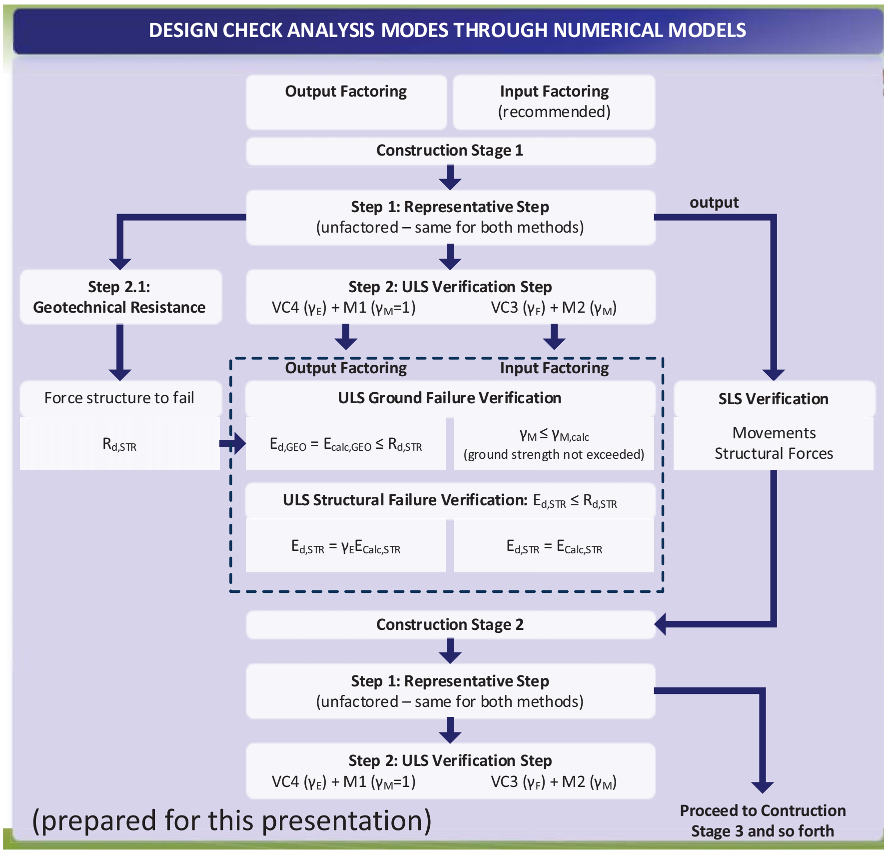

(prepared for this presentation)

ULS by Numerical Models (EN 1997-1 and EN 1997-3) – *Design Check Process*

Alternatives for Input Factoring

- Apply γ_E to strength to utilize load redistribution capacities

Alternatives for Input Factoring

- Use: VC4(γ_E) + M2(γ_M)
- Modify stiffness to counteract stiffness reduction due to γ_M
- Apply reduction to structural strength as well to identify combined failure mechanisms

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Conclusions

- *The 2nd Generation EN 1997 has **enhanced harmonization** with EN 1990, **easing** structural and geotechnical engineers' **communication***
- *This includes **clearer definitions** of X_{nom}, X_k, X_{rep}, X_d*
- *Knowing the statistical definitions and their determination provides **useful insight** on the **types and selection of the representative values***
- *While (MFA/RFA) + VC are **better definitions**, in practice they are **the same with** the DAs*
- *For ULS via limit analysis **care is required** on the selecting (MFA or RFA) + VC **when resistance R is nonlinear function** of strength properties*
- *The 2nd Generation of Eurocodes has **specific guidance on numerical analysis methods** that can be used even for ULS analysis*

References

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- Belokas, G. 2019. ***Probabilistic geotechnical engineering analysis based on first order reliability method***. *Frattura ed Integrità Strutturale. Vol. 13(50), 354–369. doi: 10.3221/IGF-ESIS.50.30.*
- *Frank, R., Bauduin, C., Kavvadas, M., Krebs Ovesen, N., Orr, T., and Schuppener, B. (2004). Designers' guide to EN 1997-1: Eurocode 7: Geotechnical design — General rules, London: Thomas Telford.*
- *European Commission: Joint Research Centre. 2025.* ***TG B2 – Design Examples***. *Joint Research Centre. European Commission. L. Batali and G. Belokas (editors). (editing in progress)*
- *Kovaiou M., Belokas G. 2023.* ***A comparative evaluation of design approaches and computational models for shallow foundations based on Eurocode 7***. *Proceedings 17th Danube European Conf. on Geotechnical Engrng Bucharest, România*

References

Mr **Ioannis Ntaliakouras** contributed in PhD student @ UNIWA contributed in the ULS by Numerical Models

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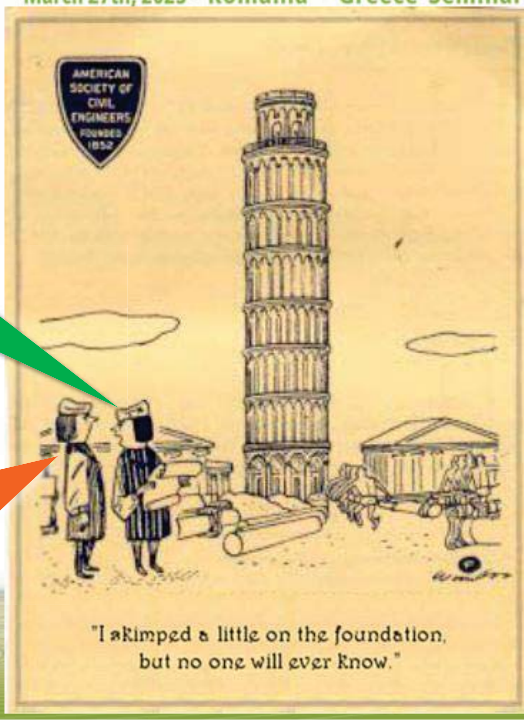
THANK YOU!

I skimped a little on the foundation, but no one will ever know

(ASCE, 1962)

How **on earth** does the foundation meet EN 1990 and EN 1997 reliability criteria?

?



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Seismic hazard assessment for calibration of seismic action in P100-1/2025 draft design code

Radu Văcăreanu, Alexandru Aldea, Cristian Arion, Florin Pavel
Technical University of Civil Engineering of Bucharest

1st Romania – Greece Seminar on Earthquake & Geotechnical Engineering

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4. Site conditions in design codes in Romania
5. Probabilistic Seismic Hazard Analysis (PSHA) results
6. P100-1/2025 – Seismic action
7. Acknowledgments

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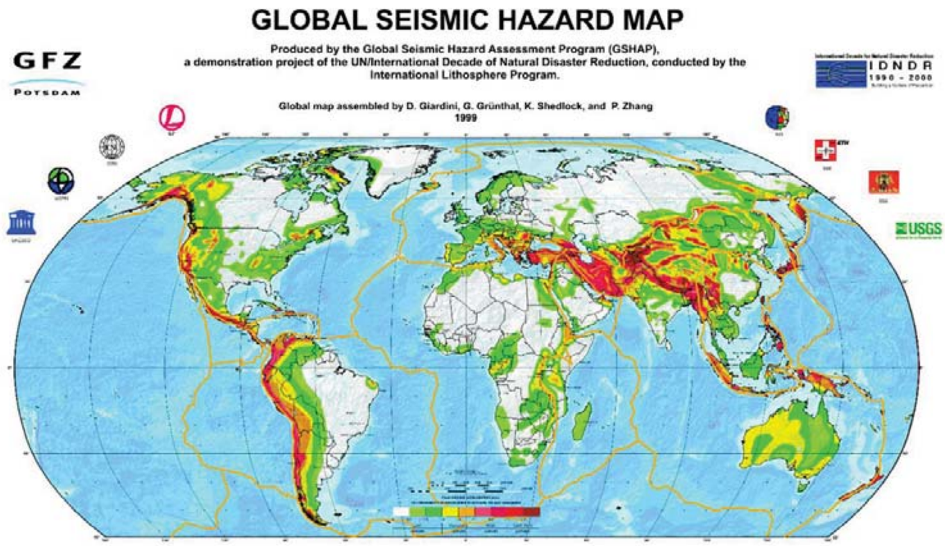
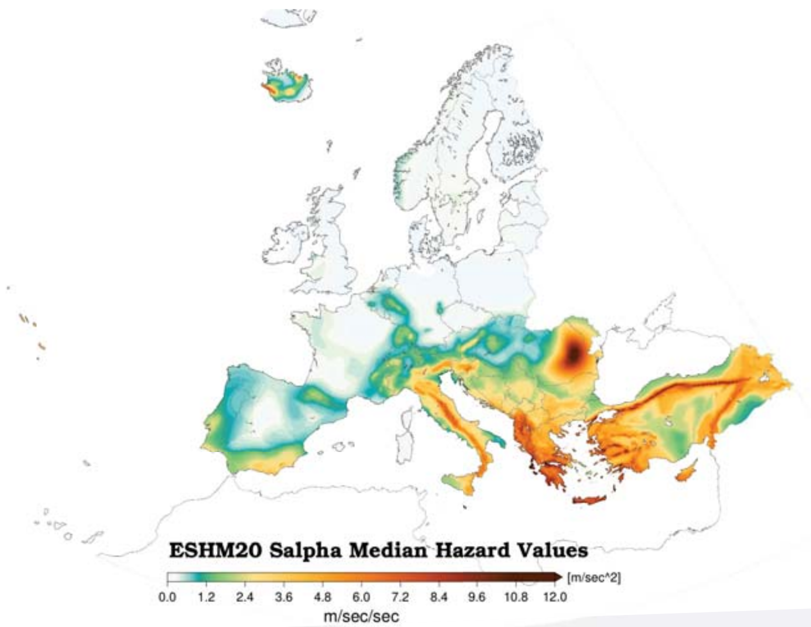
Introduction



Global Seismic Hazard Map – GSHAP
https://www.gfz-potsdam.de/en/GSHAP/

Introduction



European Seismic Hazard Map – ESHM20
http://hazard.efehr.org/en/hazard-data-access/hazard-maps/

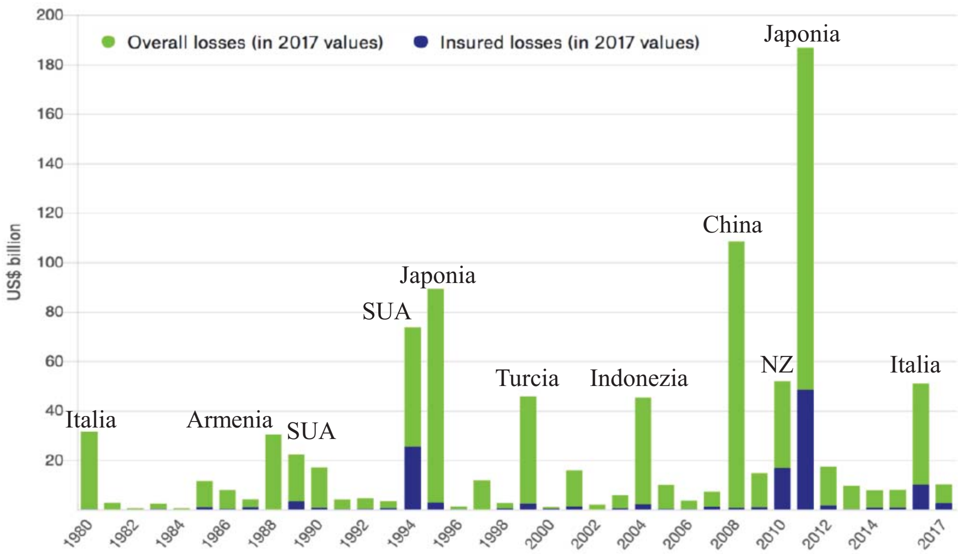
Introduction

- According to Munich Re (2016), earthquakes have caused average annual losses of $34.7 billion.
- The most expensive earthquakes were those in Japan in 2011 (Tohoku) and 1995 (Kobe), with losses of $210 billion and $100 billion, respectively.
- Earthquakes in Guatemala (1976), Nicaragua (1972), El Salvador (1986), and Haiti (2010) resulted in economic losses of 98%, 82%, 40%, and 120% of each country's GDP, respectively (Daniell et al., 2011).

Munich Re (2016). Loss events worldwide 1980–2015, 10 costliest events ordered by overall losses (as at March 2016), Munich Reinsurance Company

Daniell, J.E., Khazai, B., Wenzel, F., Vervaeck, A. (2011*). The CATDAT damaging earthquakes database*, Nat. Hazards Earth Syst. Sci., 11, 2235–2251, doi:10.5194/nhess-11-2235-2011

Introduction



Total and insured seismic losses - 1980-2017 (Silva et al., 2019)

Silva, V., Pagani, M., Schneider, J., Henshaw, P. (2019). *Assessing seismic hazard and risk globally for an earthquake resilient world*, Contributing Paper to GAR 2019

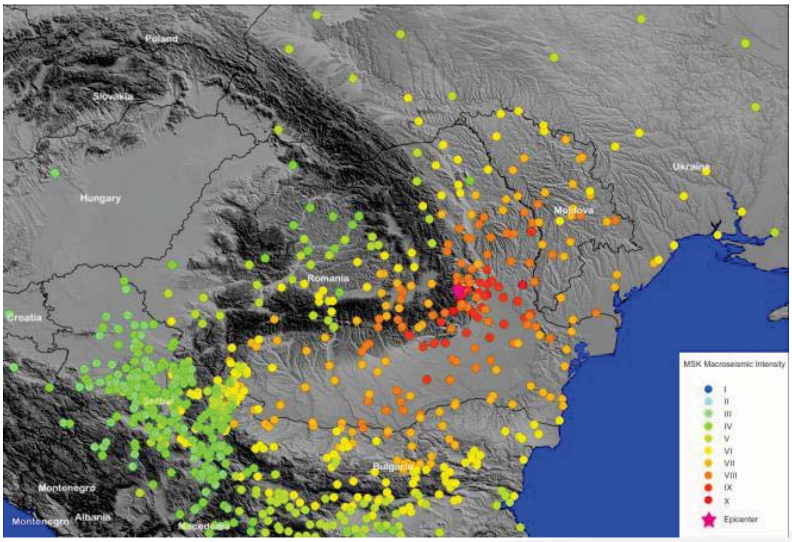
Major Earthquakes in Romania in the 20th Century

- If we refer to the 20th century and the intermediate-depth seismic source in Vrancea, the most powerful seismic events are those of November 10, 1940 (Mw 7.7) and March 4, 1977 (Mw 7.4).
- During the earthquake on November 10, 1940, 593 people lost their lives (140 of them in Bucharest), and over 60,000 homes were destroyed or heavily damaged (Georgescu & Pomonis, 2012).
- The earthquake on March 4, 1977, although it had a lower magnitude than the one in 1940, caused much more dramatic effects, both in Bucharest and throughout the country.

Georgescu E.S., Pomonis A. (2012). *Building damage vs. territorial casualty patterns during the Vrancea (Romania) earthquakes of 1940 and 1977*. In: Proceedings of the 15th World Conference of Earthquake Engineering. Lisbon, Portugal, 2012

Major Earthquakes in Romania in the 20th Century

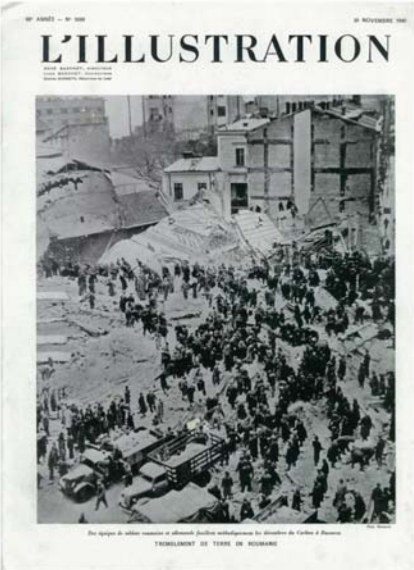
November 10, 1940, Mw=7,7, h=150 km



Macroseismic intensities MSK

Major Earthquakes in Romania in the 20th Century

November 10, 1940, Mw=7,7, h=150 km



Carlton Building (l'Illustration, 1940)

Major Earthquakes in Romania in the 20th Century

November 10, 1940, Mw=7,7, h=150 km

Ruinele oraşului Panciu (Putna).

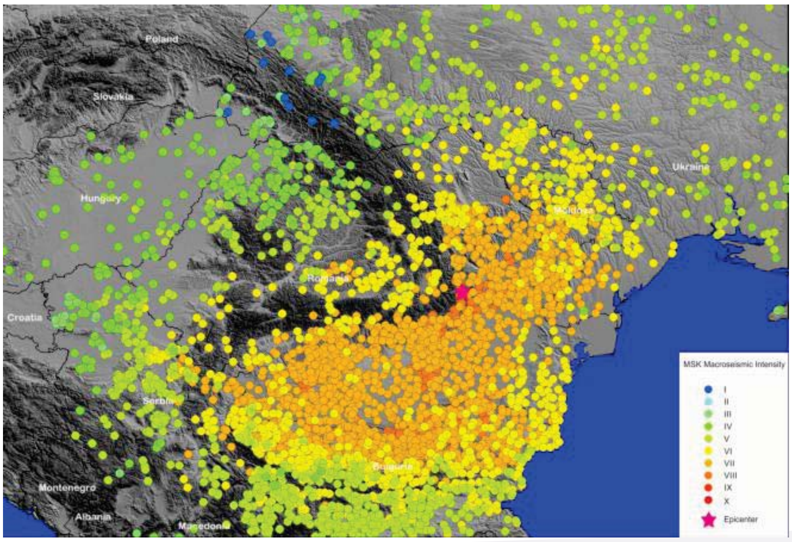
Biserică din Focşani

Abb. 8. — Zerstörung eines einstöckigen Hauses inmitten von Parterrehäusern in Adjud. Bezirk Putna.

https://www.monitoruldevrancea.ro/

Major Earthquakes in Romania in the 20th Century

March 4, 1977, Mw=7,4, h=94 km



Macroseismic intensities MSK

Major Earthquakes in Romania in the 20th Century

March 4, 1977, Mw=7,4, h=94 km

- 1,578 deaths (1,424 in Bucharest)
- 11,221 injured (7,598 in Bucharest)
- 32 collapsed buildings in Bucharest
- 33,000 homes destroyed or severely damaged
- Total losses: $2.05 billion (over 6% of 1977 Romania GDP)

World Bank (1978). *Report and Recommendation of the President of the International Bank for Reconstruction and Development to the Executive Directors on a Proposed Loan to the Investment Bank with the Guarantee of the Socialist Republic of Romania for a Post Earthquake Construction Assistance Project*, Report No. P-2240-RO

Major Earthquakes in Romania in the 20th Century

March 4, 1977, Mw=7,4, h=94 km

Major Earthquakes in Romania in the 20th Century

March 4, 1977, Mw=7,4, h=94 km

Major Earthquakes in Romania in the 20th Century

March 4, 1977, Mw=7,4, h=94 km


建築研究資料

Kenchiku Kenkyu Shiryo

No.20 January 1978

ルーマニア地震ディジタル記録

Digitized Data of Strong-Motion Earthquake Accelerograms in Romania (March 4, 1977)

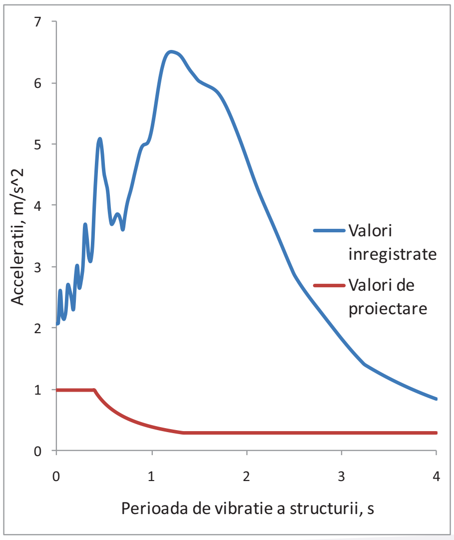
強震観測委員会

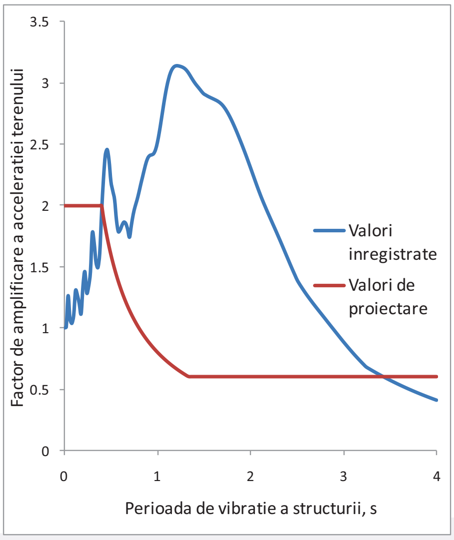
by

Observational Committee of Strong Motion Earthquake

建設省建築研究所

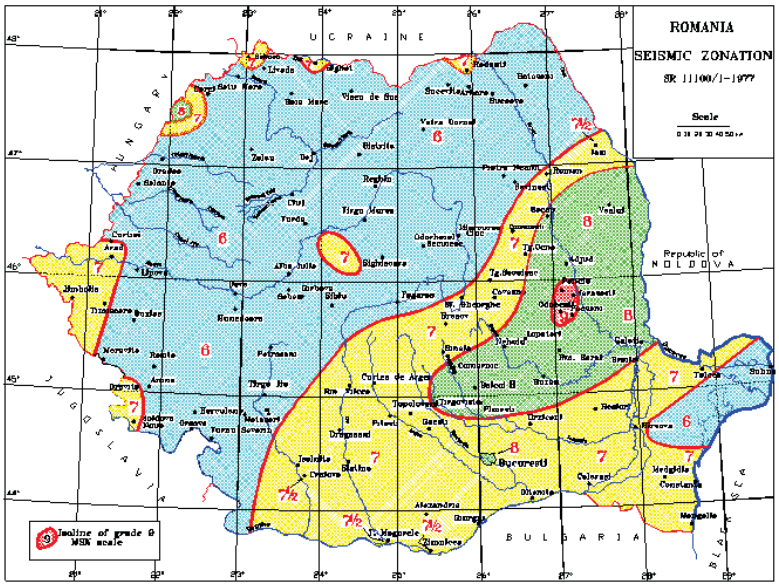
Building Research Institute, Ministry of Construction





Digitization of motion (left), response spectrum of absolute accelerations (centre), and normalized response spectrum of absolute accelerations (right) – blue – recorded values; red – values as of P13-70 seismic regulation

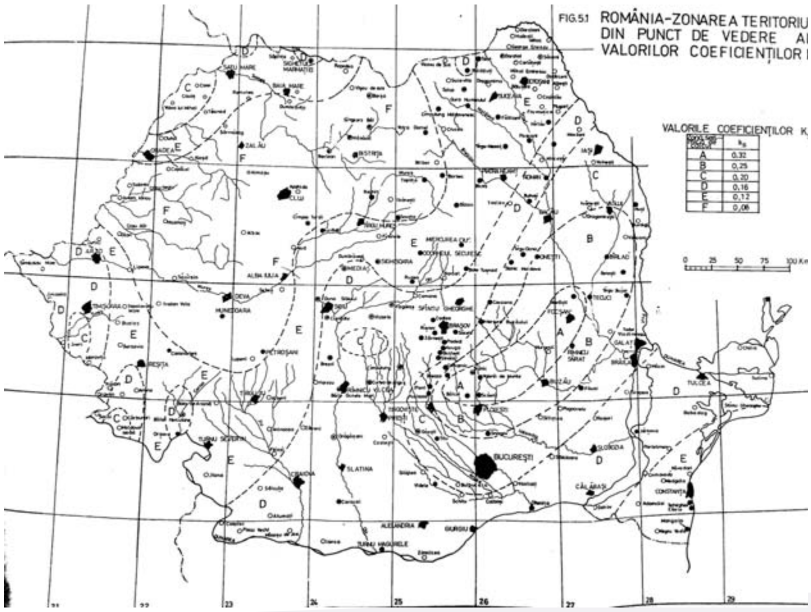
Seismic zonation– P100/78/81 design code



Macroseismic intensity (MSK)	*PGA* (‘g)
6	0.07
6 ½	0.09
7	0.12
7 ½	0.16
8	0.20
8 ½	0.26
9	0.32

Macroseismic intensities MSK

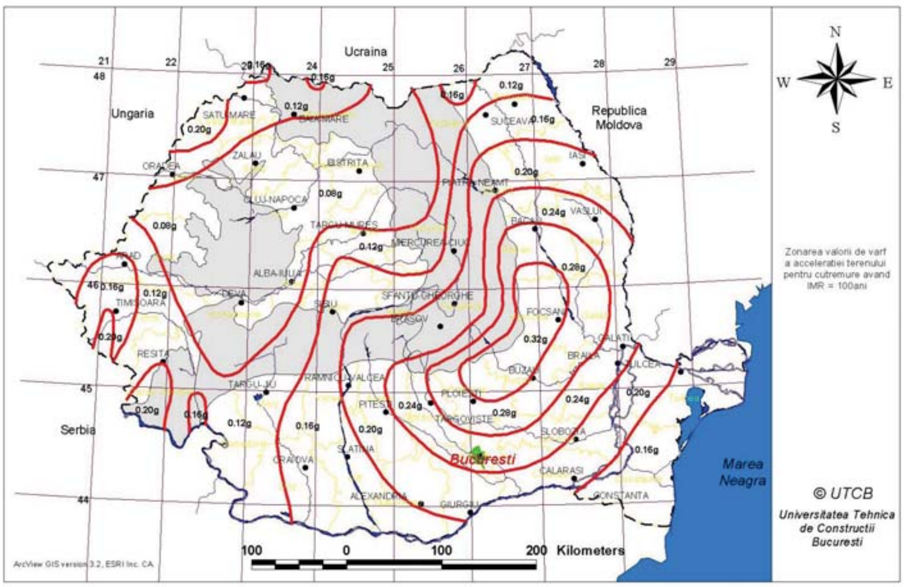
Seismic zonation – P100/92 design code



Seismic zone	*PGA* ('g)
A	0.32
B	0.25
C	0.20
D	0.16
E	0.12
F	0.08

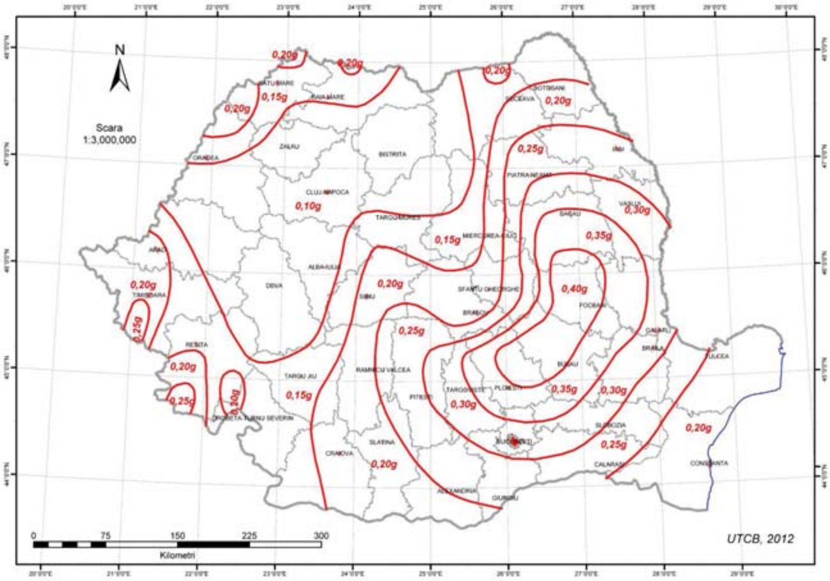
Seismic zonation of Romania (*PGA* values with 63% exceedance probability in 50 years– *MRP*=50 years)

Seismic zonation – P100-1/2006 design code



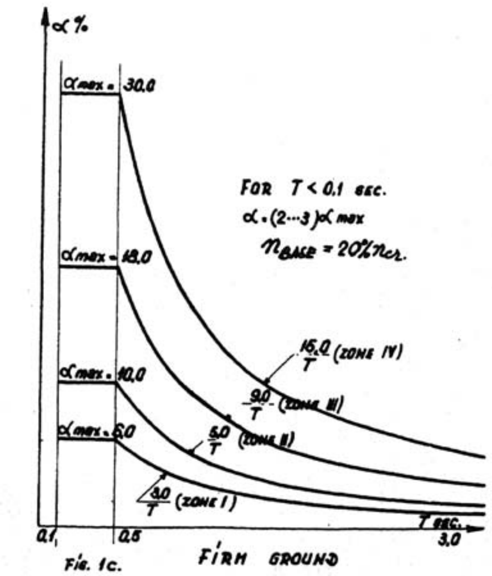
Seismic zonation of Romania (*PGA* values with 39% exceedance probability in 50 years– *MRP*=100 years)

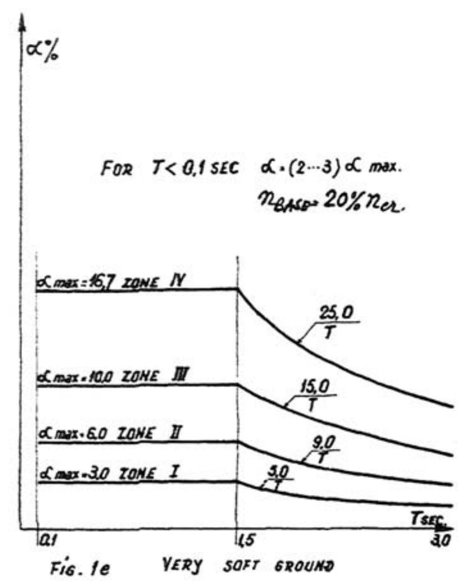
Seismic zonation – P100-1/2013 design code



Seismic zonation of Romania (*PGA* values with 20% exceedance probability in 50 years– *MRP*=225 years)

Site Conditions in Seismic Design Codes

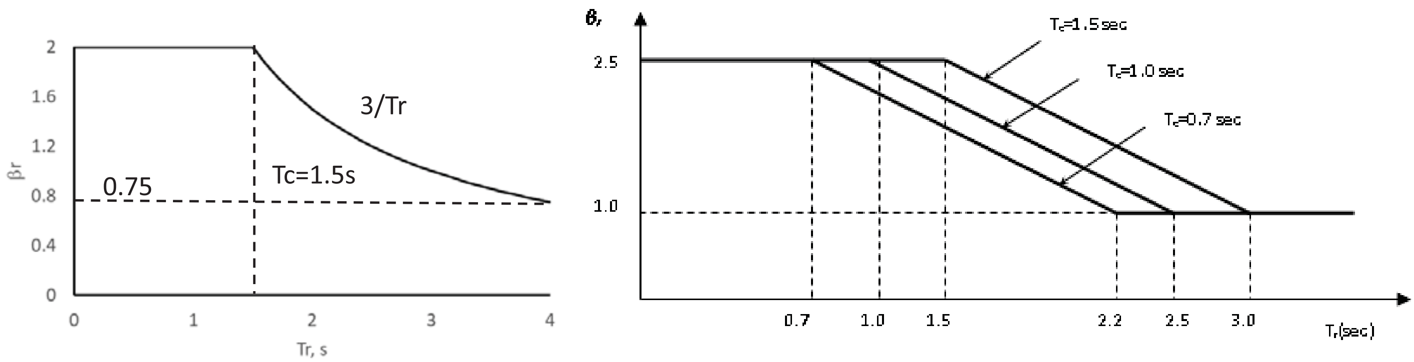




- The proposal by Țițaru & Cișmigiu in a paper presented at 2WCEE (1960), not included in P13-63,considers an increase in the corner period Tc for soft soils.
- For areas with deep Quaternary deposits in the Romanian Plain, the proposed Tc value is 1.5s, very close to that observed on March 4, 1977, at INCERC Bucharest.

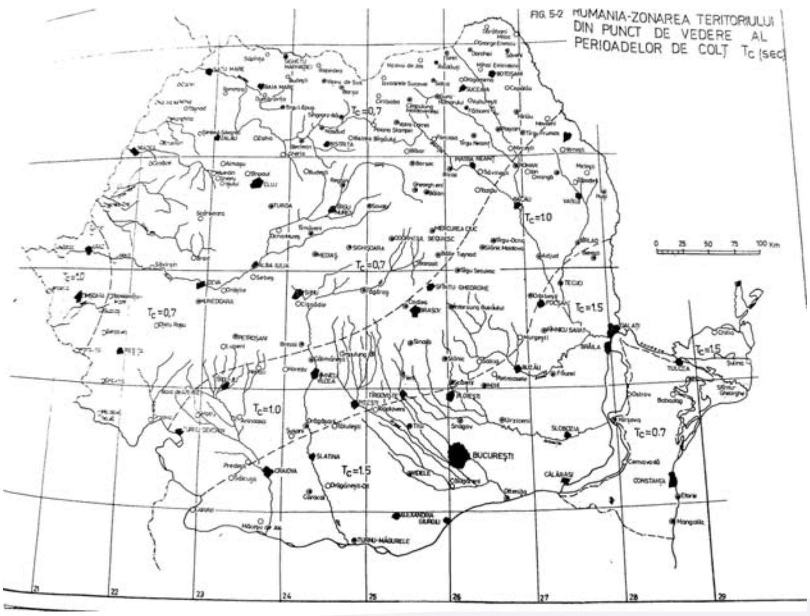
Elastic response spectra
(Țițaru & Cișmigiu, 1960)

Site Conditions in Seismic Design Codes



Normalized elastic response spectra in P100/78/81 (left, valid for the whole territory of Romania) and P100/92 (right)

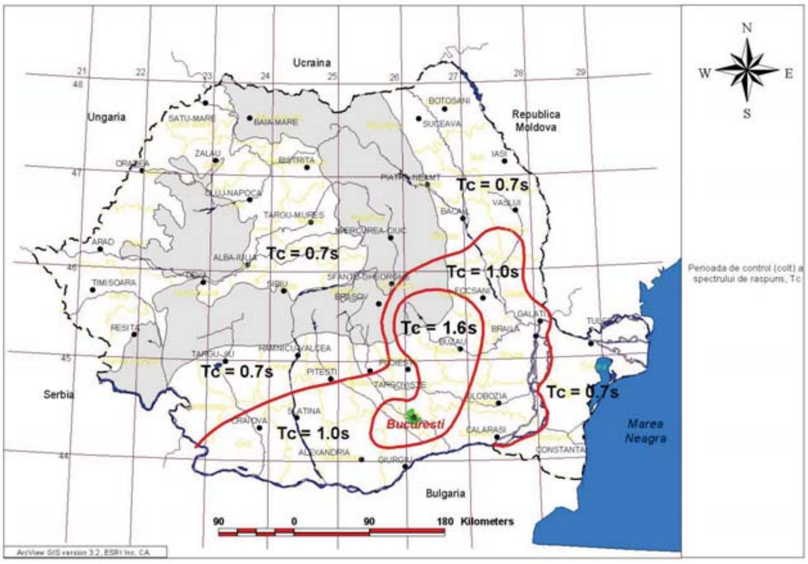
Site Conditions in Seismic Design Codes

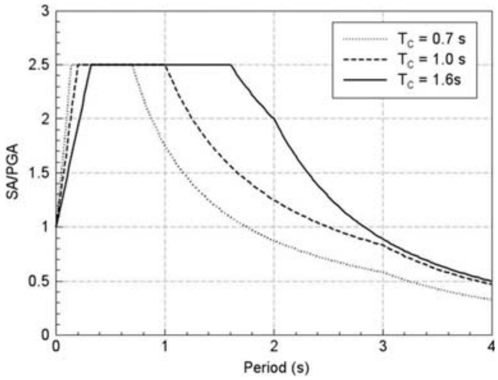


T_C (s)	T_D (s)
0.7	2.2
1.0	2.5
1.5	3.0

Zoning of the corner period Tc in P100/92

Site Conditions in Seismic Design Codes





T_C (s)	T_D (s)
0.7	3.0
1.0	3.0
1.6	2.0

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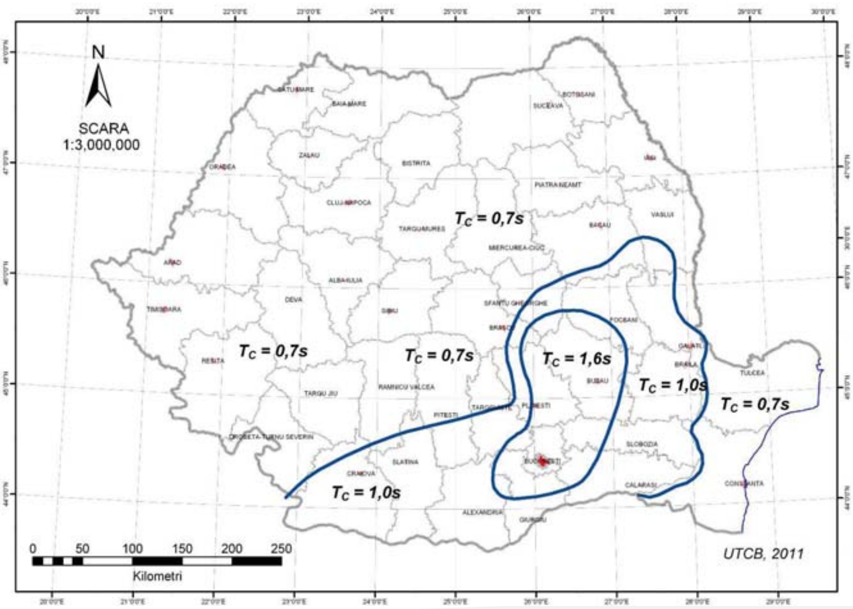
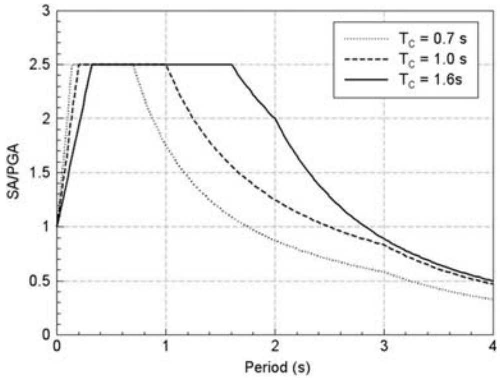
Zoning of the corner period Tc in P100-1/2006

UT CB Technical University of Civil Engineering Bucharest

Site Conditions in Seismic Design Codes





T_C (s)	T_D (s)
0.7	3.0
1.0	3.0
1.6	2.0

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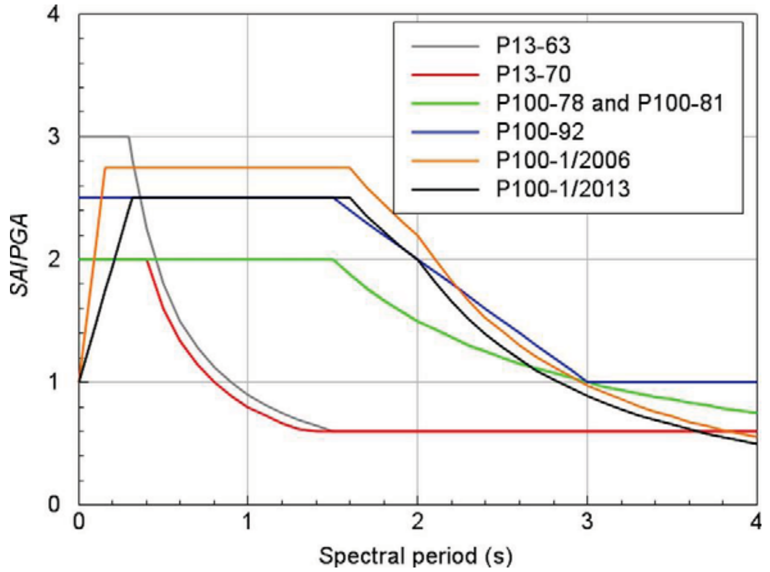
Zoning of the corner period Tc in P100-1/2013

UT CB Technical University of Civil Engineering Bucharest

Site Conditions in Seismic Design Codes



Evolution of normalized response spectra for Bucharest (1963–2013)

PSHA – Seismicity analysis

- PSHA for Romanian territory - BIGSEES (2012 – 2016) și RO-RISK (2016)
- Revised seismicity analysis (shallow sources + Vrancea intermediate depth); develop two new GMPEs (Văcăreanu et al. 2015, 2016) for Vrancea intermediate depth source for spectral ordinates SA(T) and macroseismic intensities I_{MSK}; testing GMPES; random and epistemic variability accounted for.

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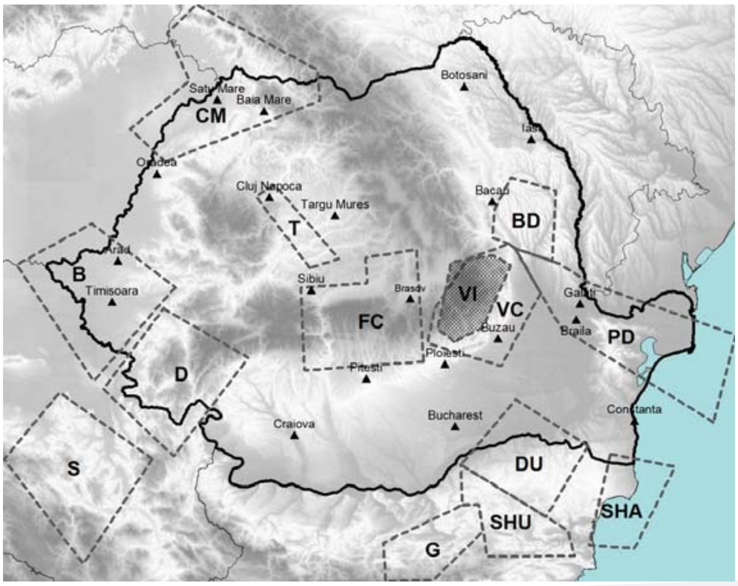
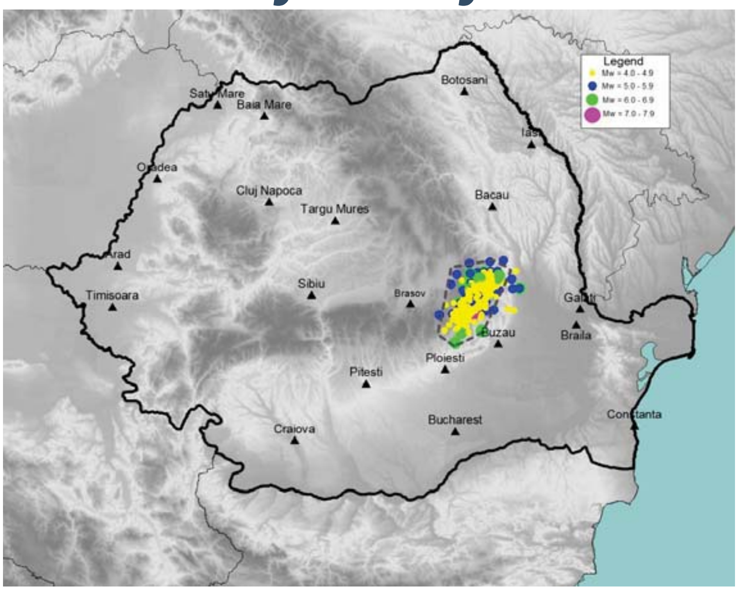
PSHA – Seismicity analysis



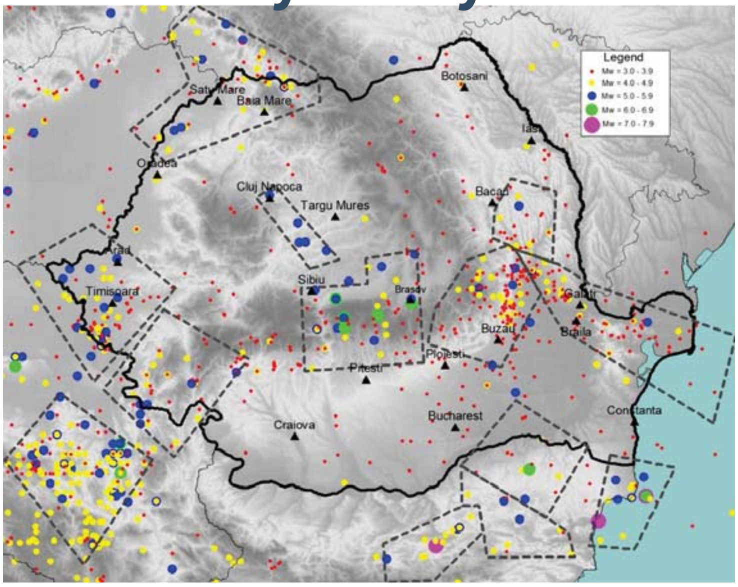
Seismic sources considered in PSHA

PSHA – Seismicity analysis



Epicentres of intermediate depth earthquakes considered in PSHA

PSHA – Seismicity analysis

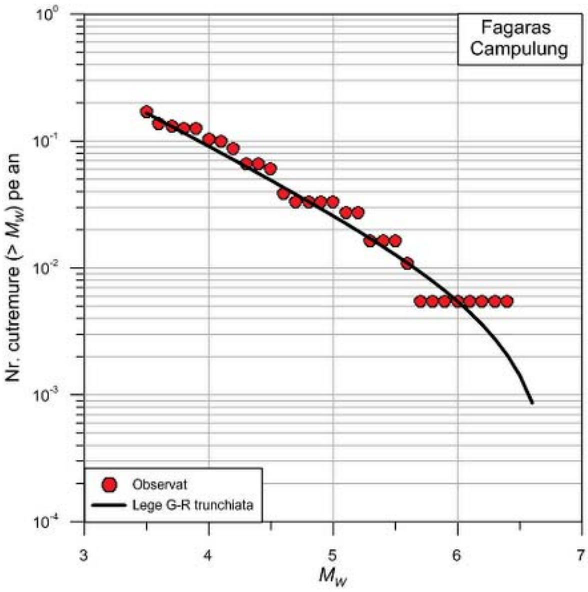


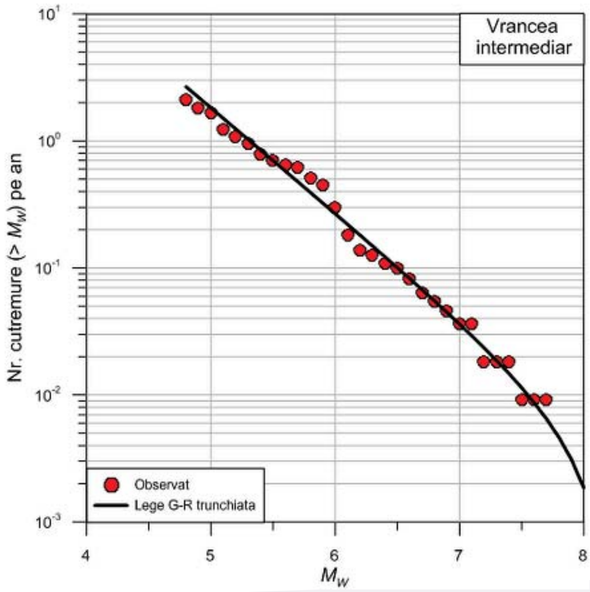
Epicentres of shallow earthquakes considered in PSHA

PSHA – Seismicity analysis

Seismic source	Completeness year	No. of earthquakes	Completeness magnitude	Maximum magnitude considered
Banat	1843	57	3.8	6.4
Bârlad Basin	1894	40	3.2	5.8
Crișana	1823	57	3.5	6.6
Danubius	1879	54	3.2	6.0
Făgăraș - Câmpulung	1826	31	3.5	6.8
Pre-Dobrogea Basin	1900	54	3.1	5.7
Serbia	1901	122	4.2	6.6
Transilvania	1523	11	4.5	6.2
Vrancea crustal	1893	40	3.8	6.2
Vrancea intermediate	1802	97	5.7	8.2
Dulovo	1892	21	3.2	7.1
Shabla	1900	17	4.5	7.8
Gorna	1900	46	4.1	7.4
Shumen	1850	19	4.5	6.7

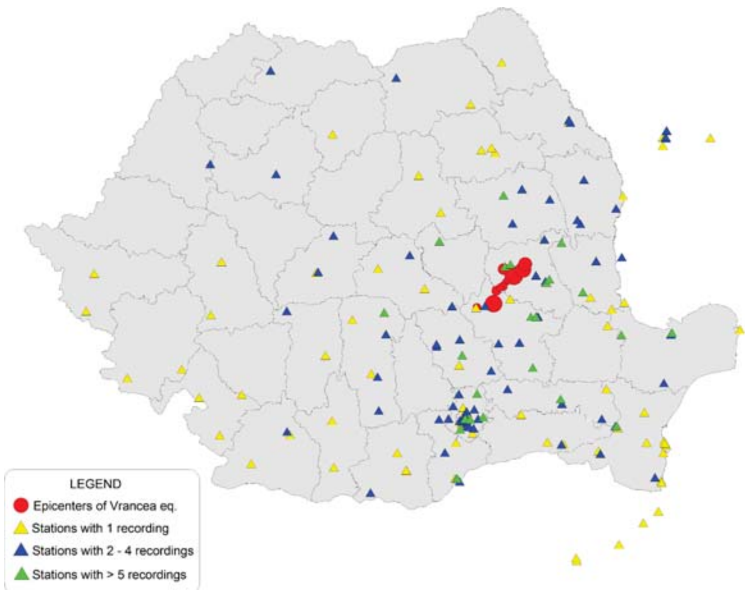
PSHA – Seismicity analysis





Gutenberg-Richter magnitude-frequency relations

PSHA - GMPEs

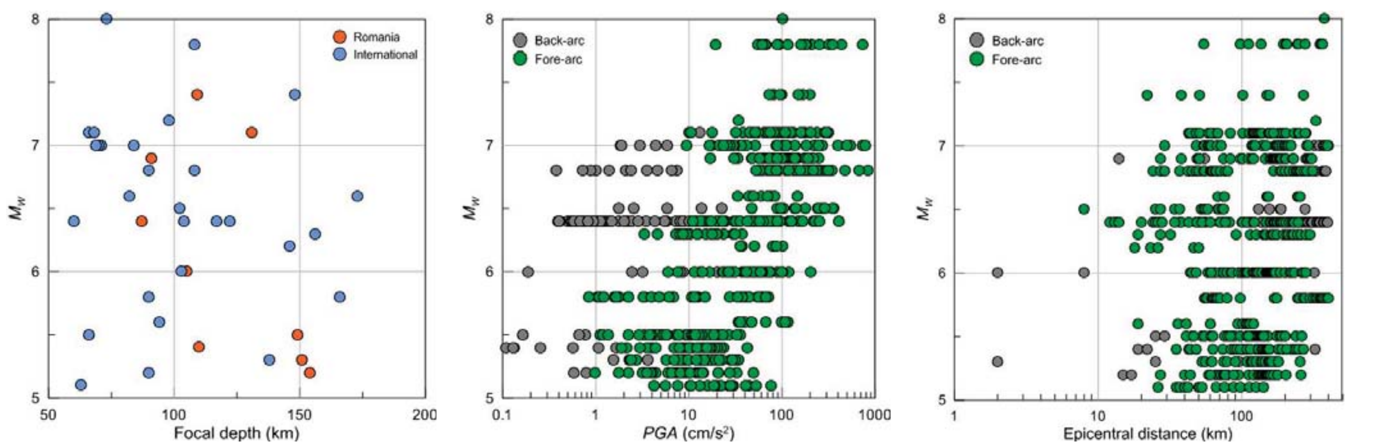


Epicentres of earthquakes and seismic stations with records for development and testing GMPEs for Vrancea intermediate source

PSHA - GMPEs

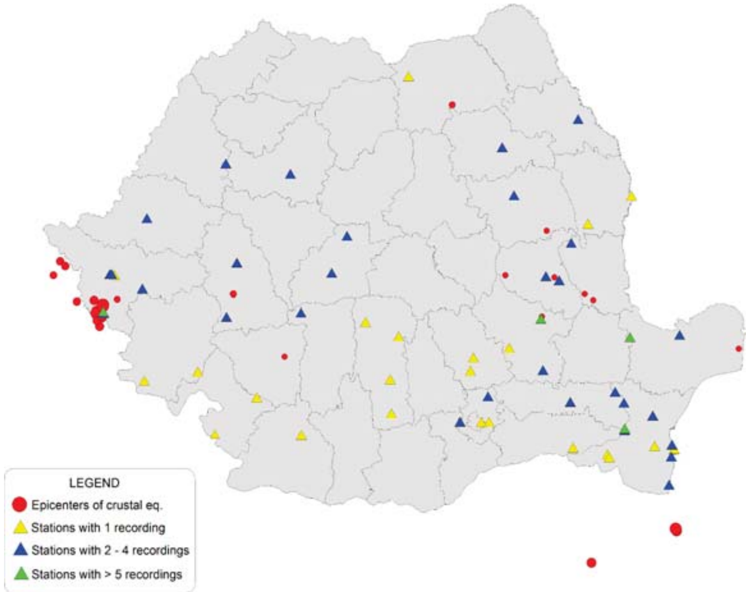
- Accelerometric database for the development and testing of GMPEs for intermediate-depth Vrancea earthquakes – 704 accelerograms, of which 344 represent ground motions generated by Vrancea intermediate seismic source; the remainder consists of accelerograms recorded in Japan, New Zealand, Mexico, Chile, India, Martinique, and Peru during intermediate-depth earthquakes.

PSHA - GMPEs



Structure of accelerometric database for testing GMPEs for Vrancea intermediate depth source

PSHA - GMPEs



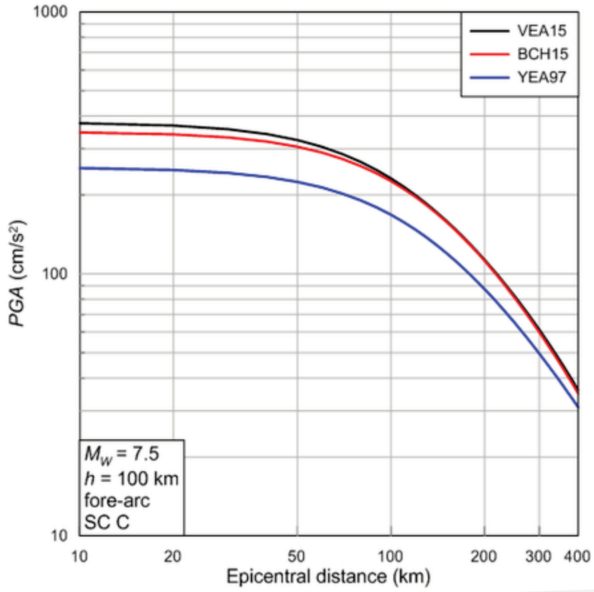
Epicentres of earthquakes and seismic stations with records for testing GMPEs for shallow sources

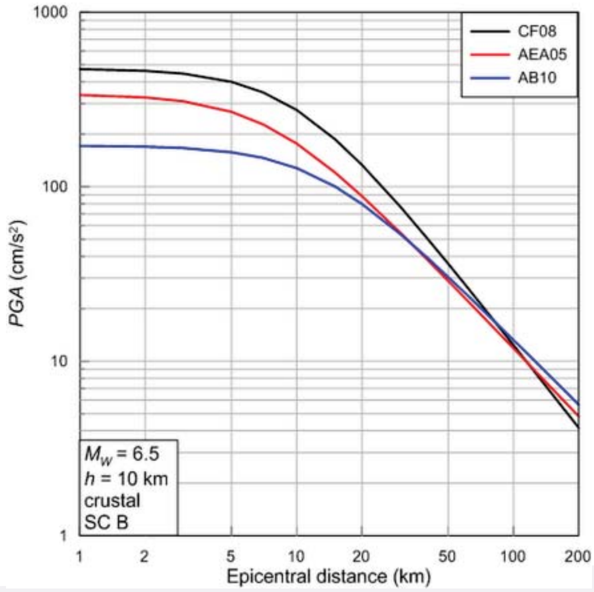
PSHA - GMPEs

- GMPEs employed in PSHA – VEA15a (Văcăreanu et al. 2015a), BCH15 (Abrahamson et al. 2015), YEA97 (Youngs et al. 1997), AB03 (Atkinson și Boore, 2003), CF08 (Cauzzi și Faccioli, 2008), AEA05 (Ambraseys et al. 2005) and AB10 (Akkar și Bommer, 2010)

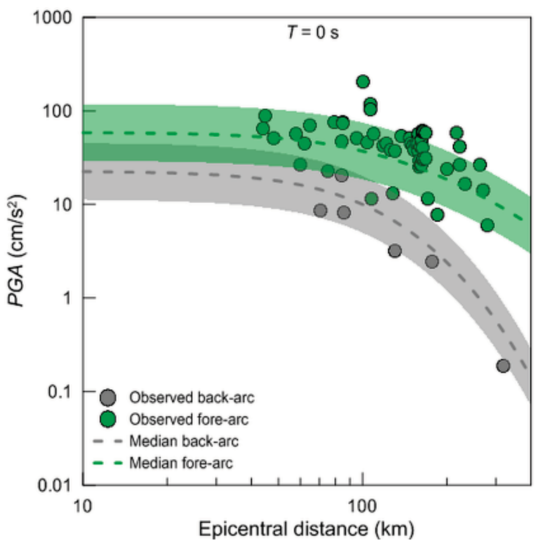
Vrancea fore-arc		Vrancea back-arc		Crustal	
Relație de atenuare	Pondere	Relație de atenuare	Pondere	Relație de atenuare	Pondere
VEA15a	0.40	VEA15a	0.40	CF08	0.40
BCH15	0.30	BCH15	0.40	AEA05	0.40
YEA97	0.30	AB03	0.20	AB10	0.20

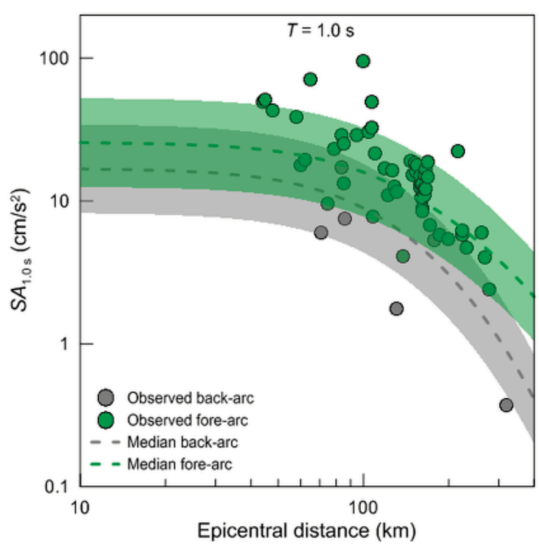
PSHA - GMPEs





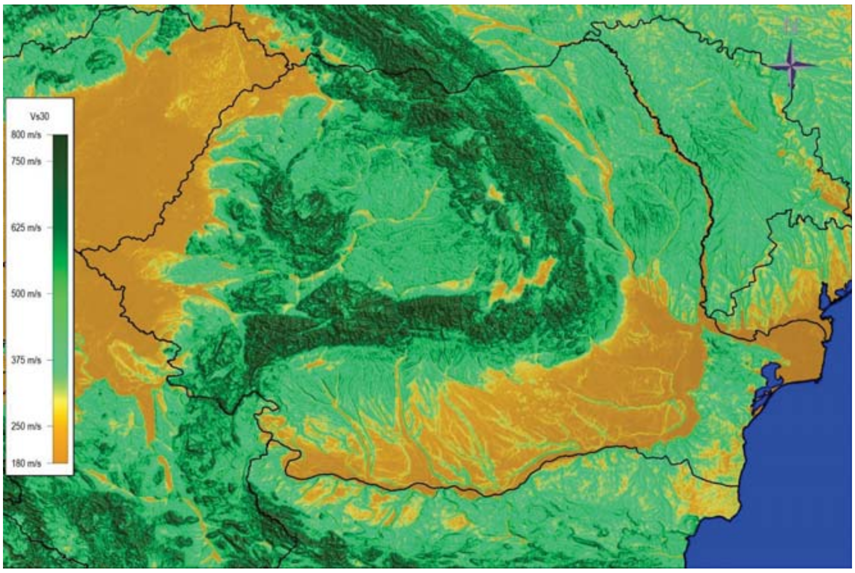
PSHA - GMPEs





Comparison of observed vs. analytical values (68% confidence bounds) – Vrancea intermediate depth earthquake, October 27 2004, M_W = 6

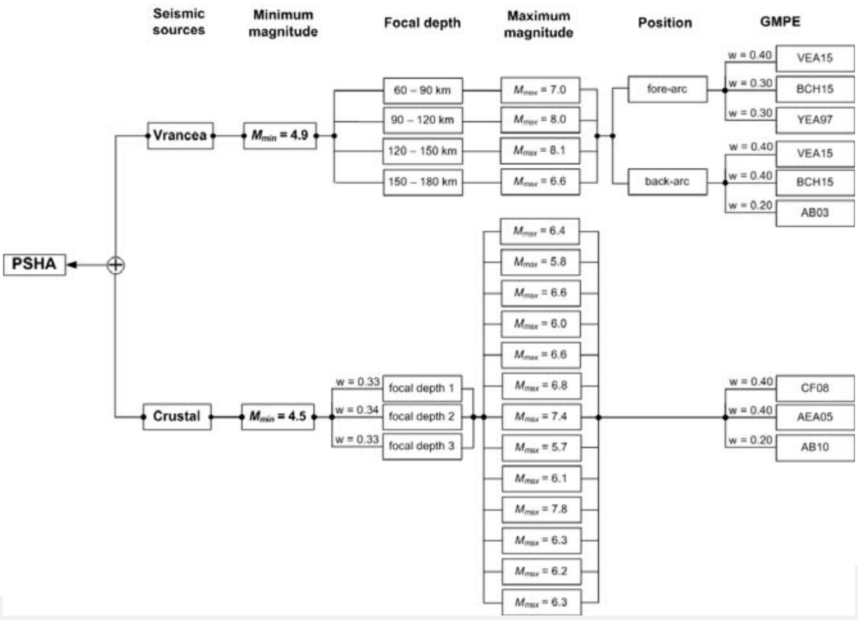
PSHA – site conditions



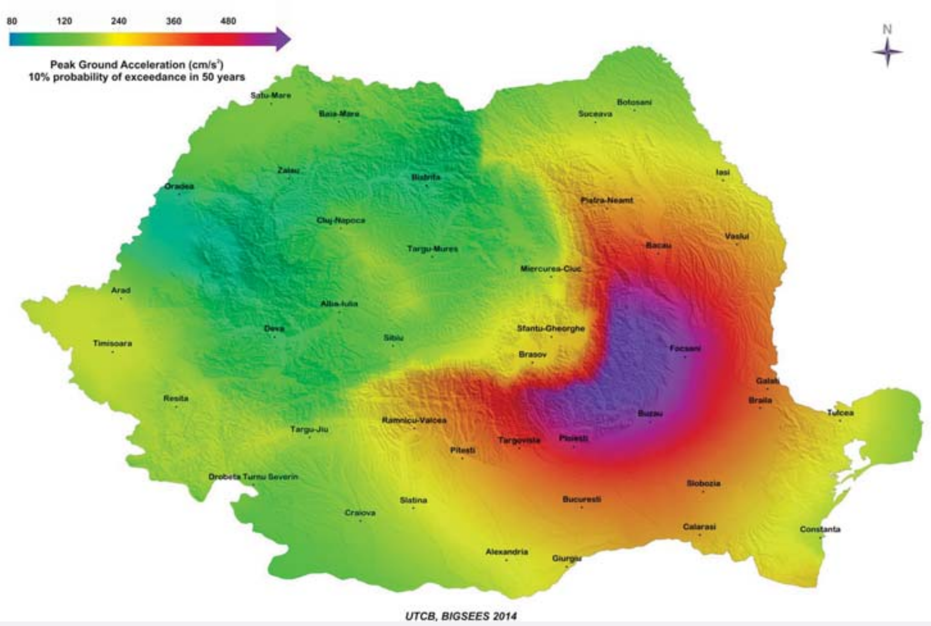
$V_{s,30}$ (m/s) (national and USGS proxy data)

PSHA – epistemic uncertainties

Logic tree used in PSHA for Romania

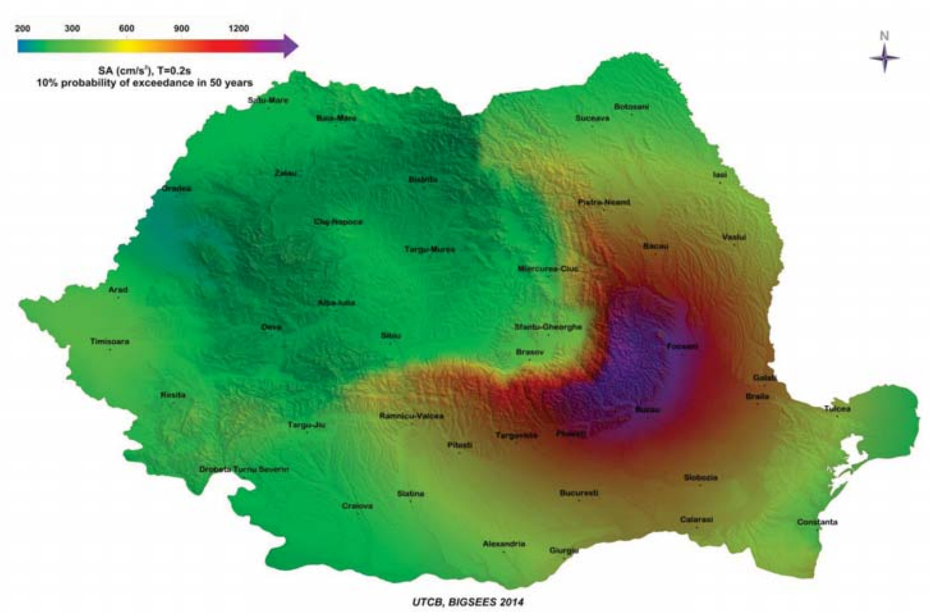


PSHA – results



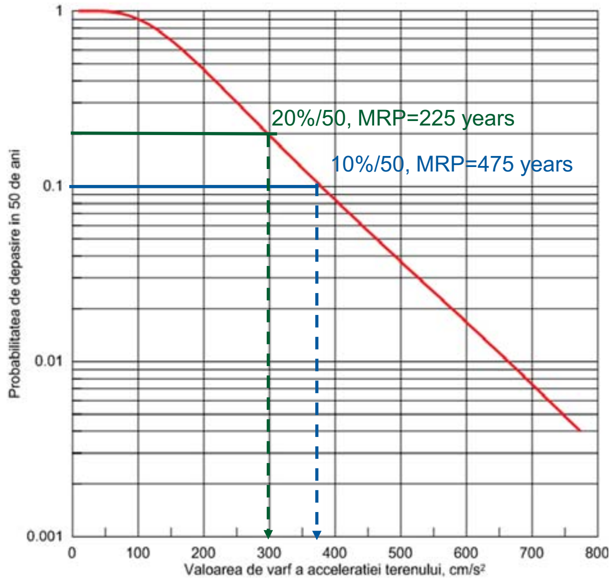
www.utcb.ro PGA values with 10% exceedance probability in 50 years

PSHA – results

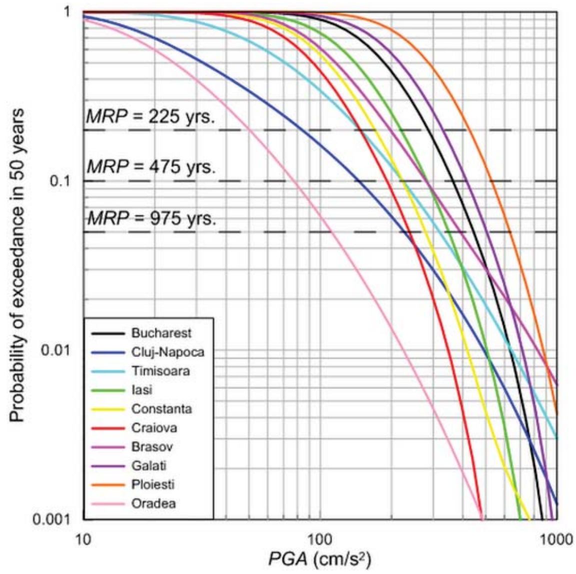


www.utcb.ro SA(T=0,2s) values with 10% exceedance probability in 50 years

PSHA – results

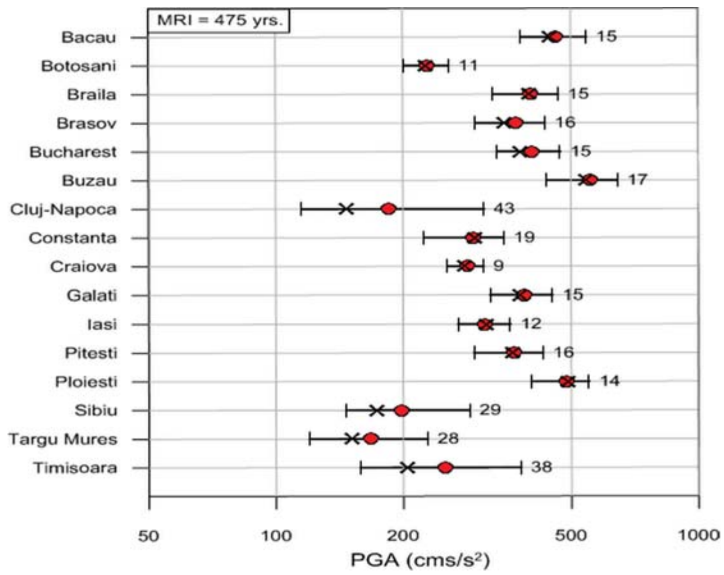


Seismic hazard curve for Bucharest
PGA values (cm/s^2)



Seismic hazard curves for ten most populous cities in Romania
PGA values (cm/s^2)

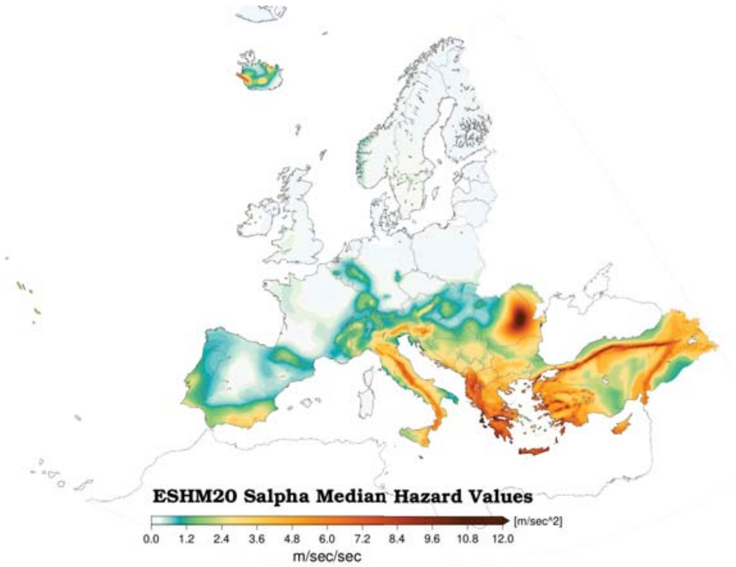
PSHA results

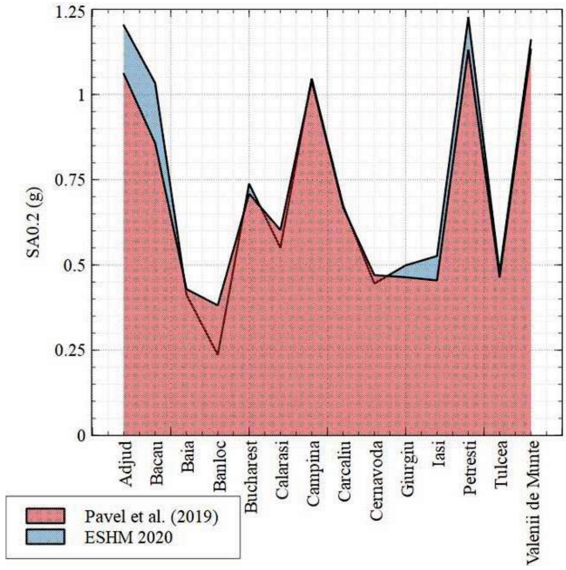


o – mean values
x – median values
| – 68% confidence bounds

Evaluation of uncertainties of PSHA results

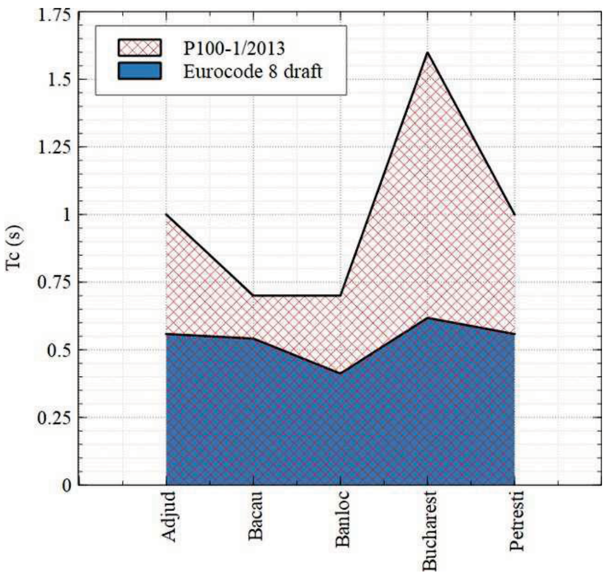
PSHA results

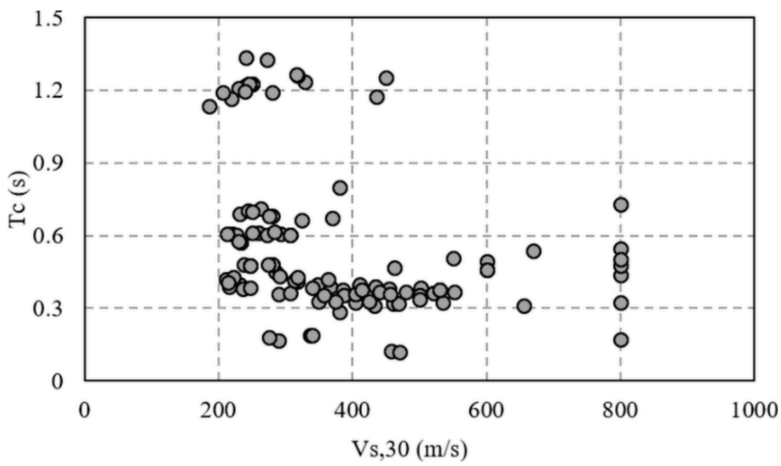




Median Sα values with 10% exceedance probabilities in 50 years (ESHM20) (left) and comparison with the values of national model (right) for bedrock spectral values

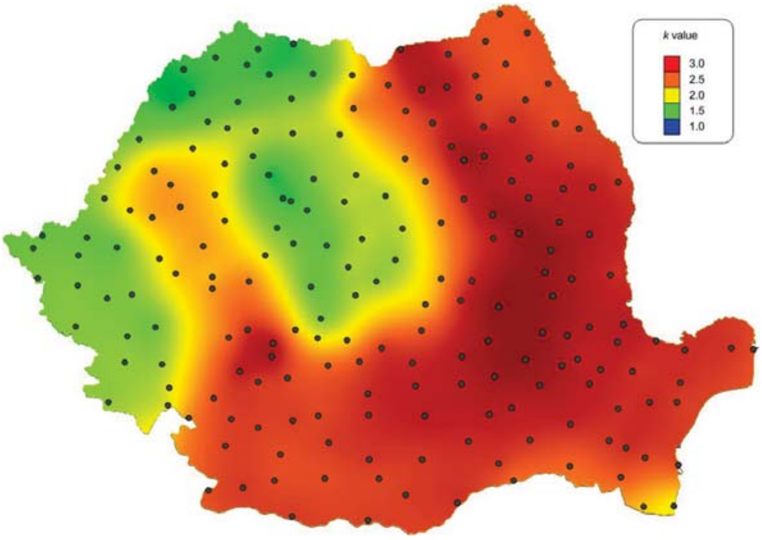
PSHA results





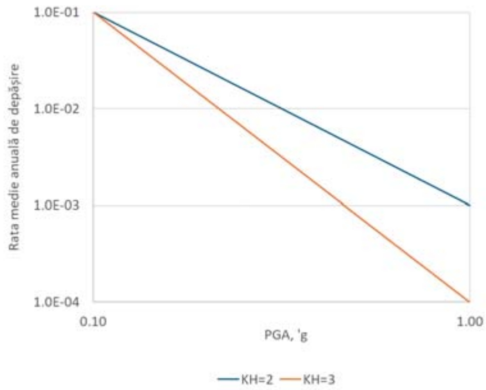
Comparison of control period Tc (left) and observed Tc values vs Vs,30 values (right)

PSHA results



Values of the slope of seismic hazard curves, K_H

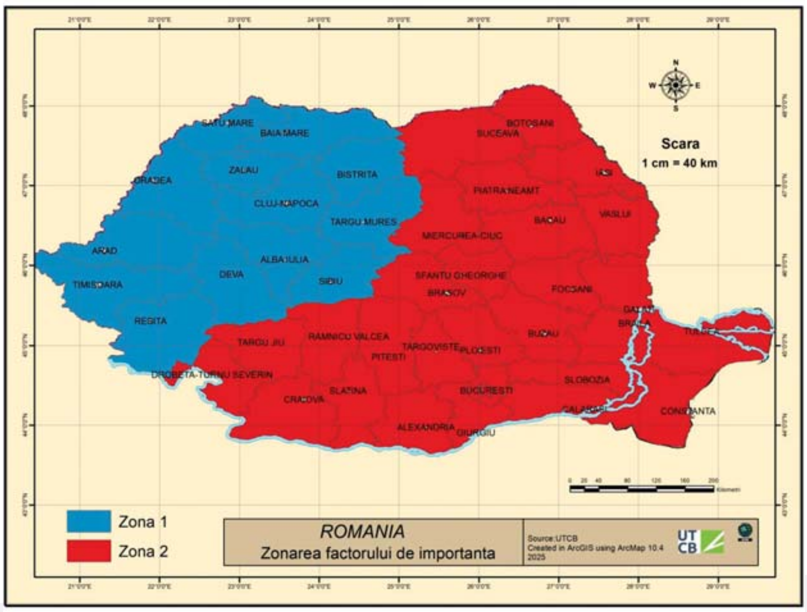
Seismic hazard curves for $SA = K_1 \cdot SA^{-KH}$



100/10	3.10	2.10
1000/100	3.16	2.19
225/40	2.35	1.75
475/40	3.45	2.25
475/225	1.47	1.29
1000/475	1.42	1.28

Ratios of seismic hazard values for different MRPs

P100-1/2025 – Seismic action

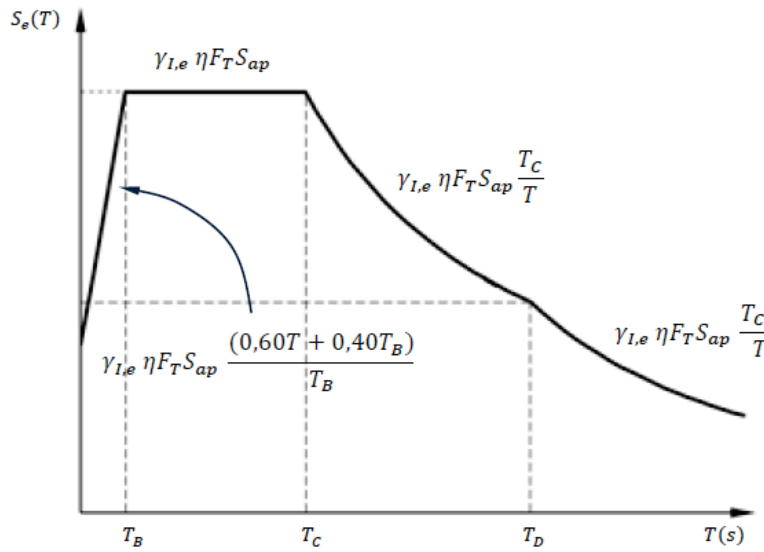


Classification of counties in two zones according to the seismic hazard curve slope

P100-1/2025 – Seismic action

Importance & exposure class of the building/ Probability of exceedance	ULS		SLS	
	For sites in counties in zone 1 (K_H=2)	For sites in counties in zone 2 (K_H=3)	For sites in counties in zone 1 (K_H=2)	For sites in counties in zone 2 (K_H=3)
I / 5%/50 ULS & 40%/50 SLS	1.50	1.25	1.55	1.35
II / 7,5%/50 ULS & 55%/50 SLS	1.15	1.10	1.25	1.15
III / 10%/50 ULS & 70%/50 SLS	1.00	1.00	1.00	1.00
IV / 20%/50 ULS & 90%/50 SLS	0.70	0.80	0.75	0.80

P100-1/2025 – Seismic action

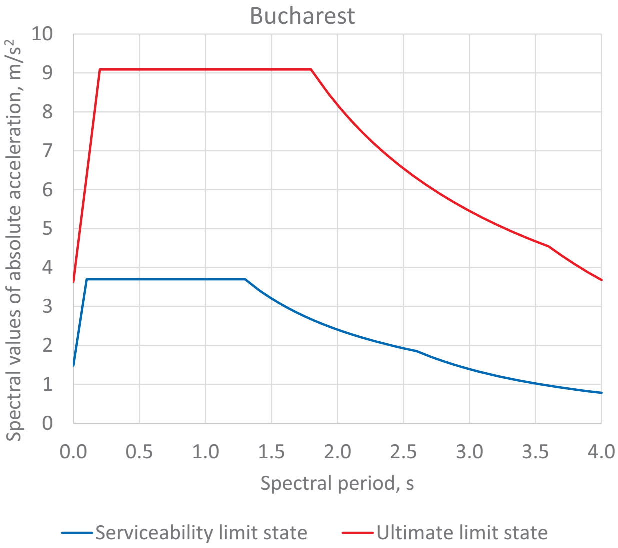


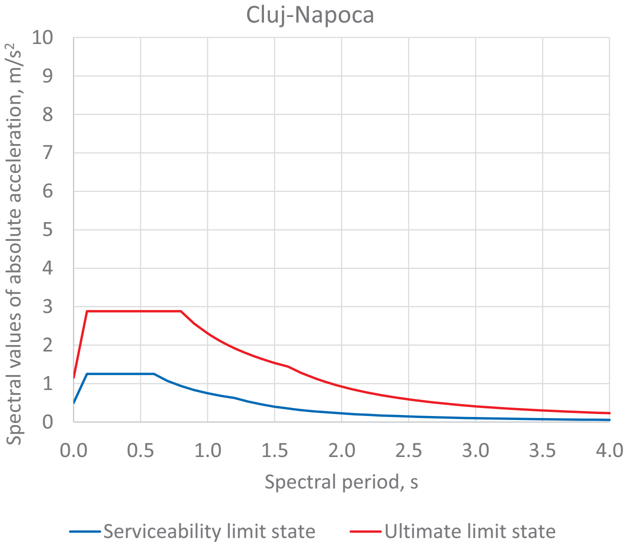
Elastic acceleration spectrum

P100-1/2025 – Seismic action

No.	County	City	$S_{ap,h}^{SLS}$, m/s²	$S_{ap,h}^{SLU}$, m/s²	T_C^{SLS}, s	T_C^{SLU}, s	Seismicity
695	BUCUREȘTI	BUCUREȘTI	3.70	9.09	1.3	1.8	High
941	CLUJ	CLUJ-NAPOCA	1.25	2.88	0.6	0.8	Low

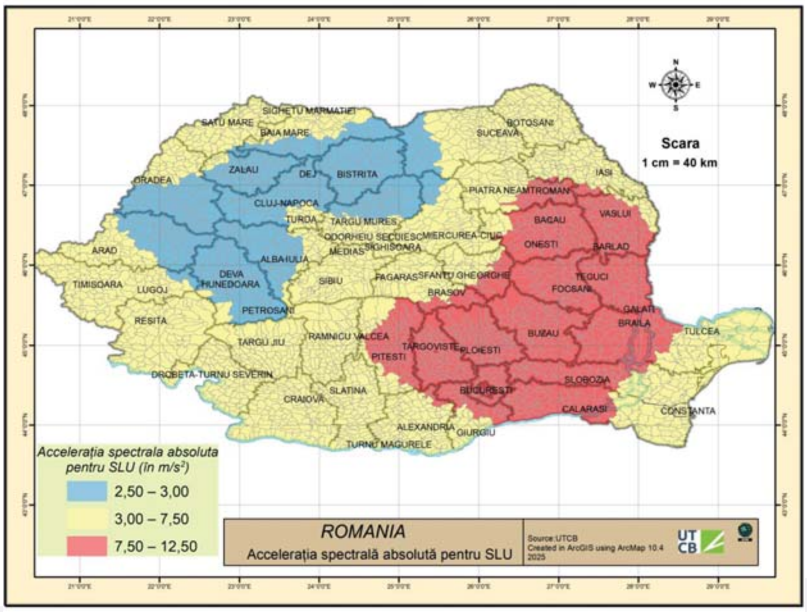
P100-1/2025 – Seismic action





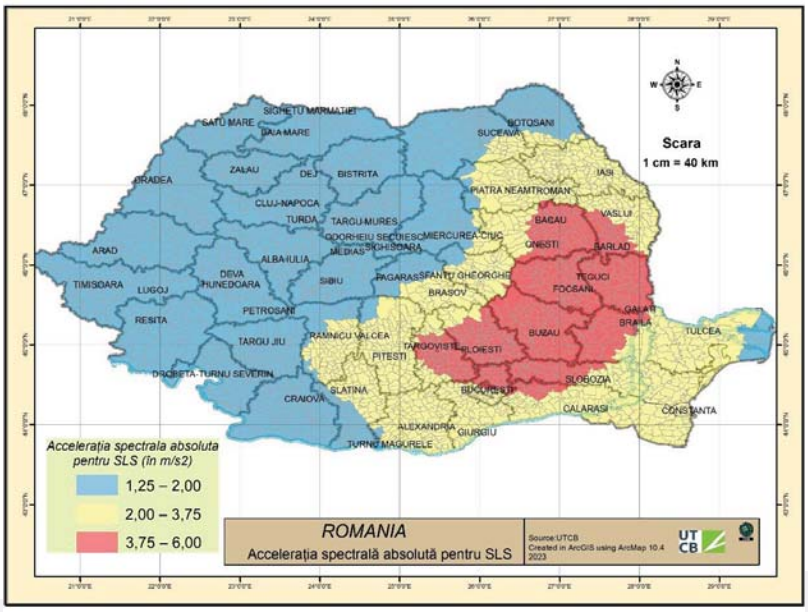
Elastic acceleration spectra for Bucharest and Cluj-Napoca

P100-1/2025 – Seismic action



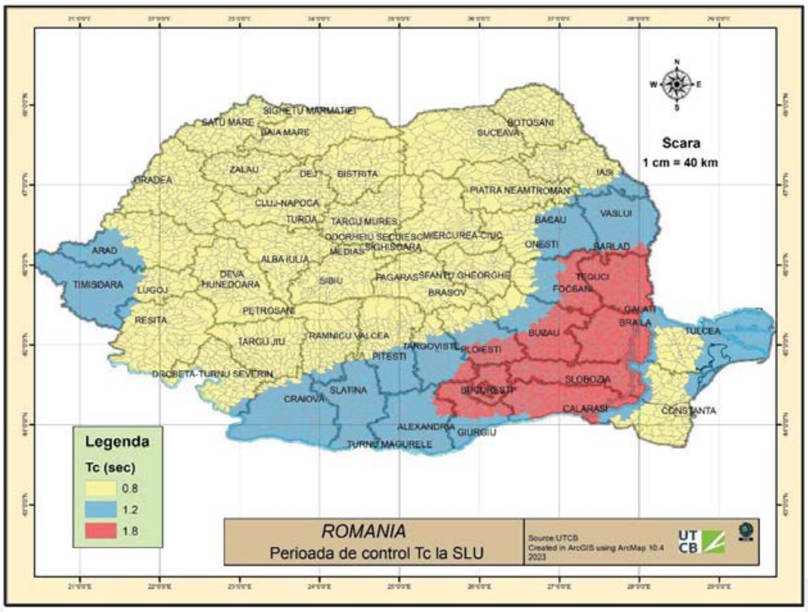
Maximum spectral acceleration values with 10% exceedance probability in 50 years for verification at Ultimate Limit State (ULS)

P100-1/2025 – Seismic action



Maximum spectral acceleration values with 70% exceedance probability in 50 years for verification at Serviceability Limit State (SLS)

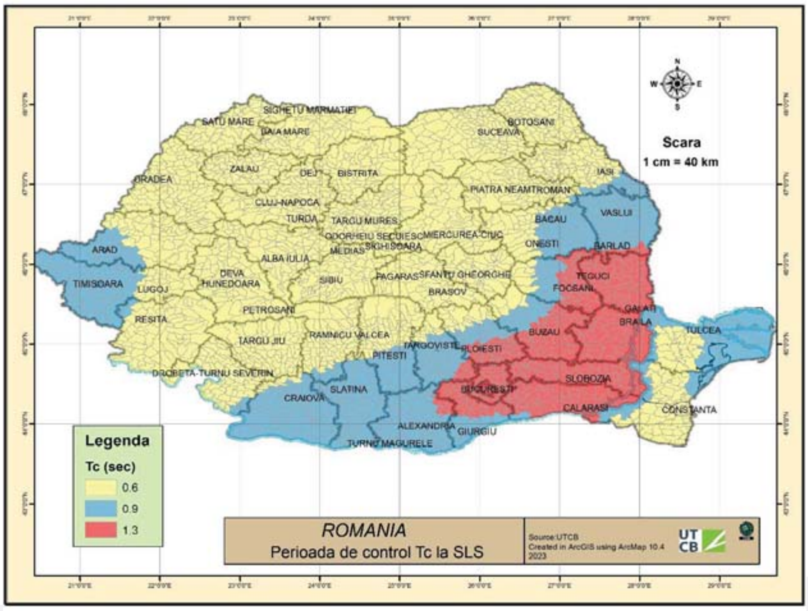
P100-1/2025 – Seismic action



$1{,}16 \cdot T_{C,20\%/50} \approx T_{C,10\%/50}$

Control period Tc values for verification
at Ultimate Limit State (ULS)

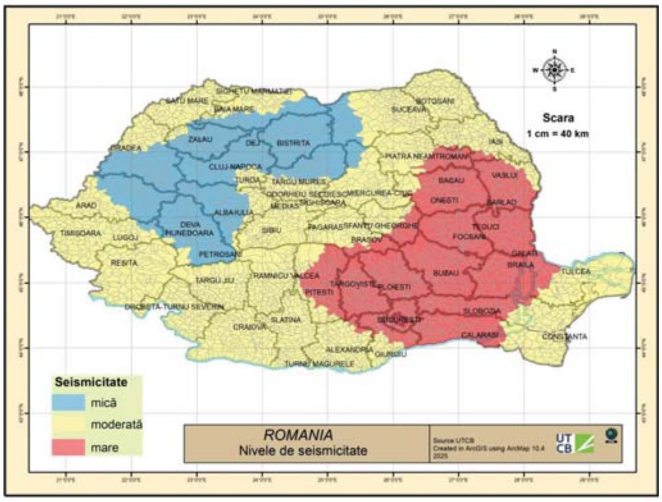
P100-1/2025 – Seismic action



$1{,}60 \cdot T_{C,70\%/50} \approx T_{C,10\%/50}$

Control period Tc values for verification
at Serviceability Limit State (SLS)

P100-1/2025 – Seismic action



Low seismicity (blue) – $S_{ap,h,SLU} \leq 3.00$ m/s^2
Moderate seismicity (yellow) – 3.00 m/s^2 < $S_{ap,h,SLU}$ < 7.50 m/s^2
High seismicity (red) – $S_{ap,h,SLU} \geq 7.50$ m/s^2

Zoning of areas with low, moderate, and high seismicity

Acknowledgement

- The results were obtained in two research projects, BIGSEES (Bridging the Gap between Seismology and Earthquake Engineering) & RO-RISK (National Risk Assessment), by the research group in UTCB: Alexandru Aldea, Cristian Arion, Florin Pavel, Radu Văcăreanu.
- The results contributed to the revision of Chapter 3 of the draft document P100-1/2025 – *Seismic design code – Part I – Design Provisions for buildings*

Thank you for your attention!

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Earthquake & Geotechnical Engineering
March 27th, 2025 • Romania - Greece Seminar

Direct and proxy seismic site characterization in Romania

Cristian Arion
Technical University of Civil Engineering, Bucharest

2

Strong earthquakes originating from Vrancea source affect large areas from Romania, Republic of Moldova, Bulgaria and Ukraine.
In the last 200 years the strong Vrancea events produced significant damage and triggered on large areas geological induced hazards.

October 26th, 1802 Vrancea Earthquake

Liquefaction was reported in many places in Bucharest and also in south-eastern Transylvania.

November 26th, 1829 Vrancea Earthquake

Liquefaction was reported in Bucharest (cellars filled with water).

January 23rd, 1838 Vrancea Earthquake

Liquefaction in Bucharest, in epicentral region and in several locations in Moldova and Romanian plain. The event was investigated in the epicentral region by engineer Gustav Schuller, Counselor of the Great Duke of Saxa. Schuller reported many sites with abundant liquefaction, water blowing out up to half meter. He also reported ground collapse on limited surfaces and a large number of ground cracks (many of them reaching 1 km length, the maximum length being 1.7 km) especially in the rivers meadows, that damaged peasants houses. The majority of the cracks were accompanied by liquefaction.

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3

November 10th, 1940 Vrancea Earthquake

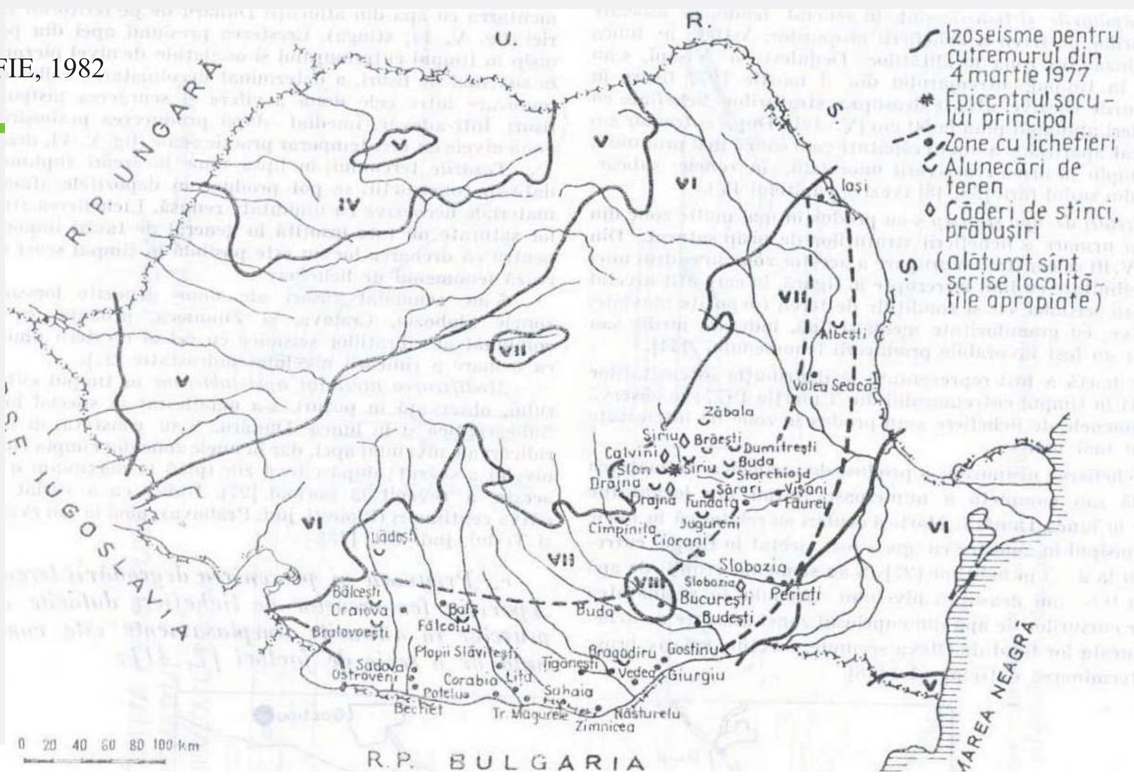
The earthquake triggered liquefaction at many sites in South Moldavian Plain and in Romania Plain including Bucharest, the water blowing out up to 1m height. The apparition of mud volcanoes with diameters up to 1.5 m and heights up to 15 cm was reported in the epicentral area. At many sites in Romanian Plain and South of Moldova the earthquake produced ground cracks along river meadows (rivers Prut, Siret, Trotuş, Putna, Ialomita, Prahova, Arges and Dâmboviţa). The majority of these cracks were accompanied by liquefaction. Their dimensions reached 250 m length and 2-3m opening.

March 4th, 1977 Vrancea Earthquake

March 4, 1977 earthquake induced geological hazards (liquefaction, landslides, rock falls, etc.) on large surfaces in Romania, Republic of Moldavia and Bulgaria. Liquefaction was reported along Danube river, in Danube Delta, in Bucharest, in Buzau and in the meadows of Prut, Barlad, Ialomita and Olt rivers. Witness reported that water blow was between 0.5-5 m and stopped quickly after the earthquake. In Bucharest liquefaction was observed at several locations along Dâmboviţa and Colentina rivers, with no damaging effects on buildings.

One of the most alarming aspects of the ground collapse in the plain area of the Danube River near the town of Giurgiu was the formation of several massive sinkholes.

@MONOGRAFIE, 1982



Map of March 4, 1977 earthquake induced geological hazards

Rockfalls in the Buzău Mountains (Eastern Carpathians) resulted from the earthquake on March 4, 1977 (M = 7.2).

https://www.ngdc.noaa.gov/hazardimages/#/earthquake/41

https://www.ngdc.noaa.gov/hazardimages/#/earthquake/41

The sliding of a massive rock was triggered by the earthquake on March 4, 1977, in the Eastern Carpathians. This landslide destroyed houses in the village of Colți in the Buzău Mountains.

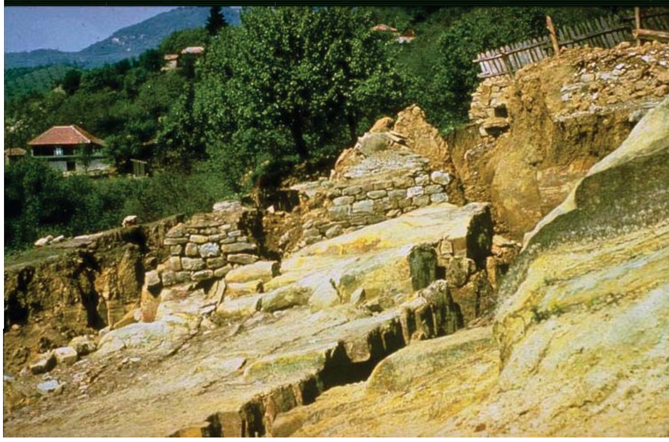


Fig.2.33 Alunecare de teren in Muntii Buzaului provocata de cutremurul de la 4 Martie 1977
Foto © D. Balteanu, Academia Romana

A landslide approximately 30-40 meters deep blocked a valley from March to April 1973. The landslide occurred on the southern slope of Blidișel Hill in the Buzău Subcarpathians. The rupture was during the earthquake on March 4, 1977, in the Eastern Carpathians.

https://www.ngdc.noaa.gov/hazardimages/#/earthquake/41

Fig. 2.31 Lichefiere - cutremurul din 4 Martie 1977: Bucuresti (stanga) si Buzau (dreapta)
Foto © Cutremurul de pamant din Romania de la 4 Martie 1977, Monografie, 1982

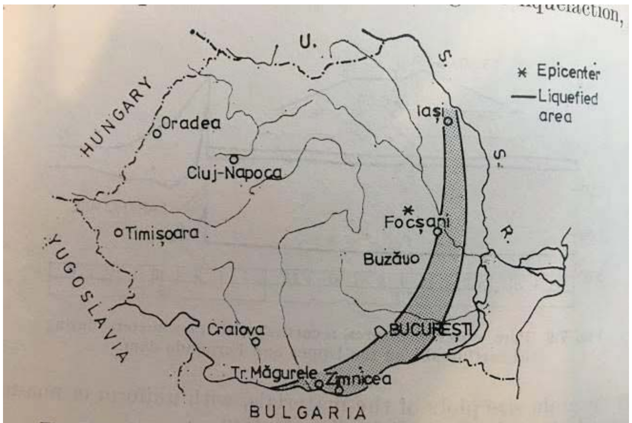


Fig. 7.11. Liquefaction of sandy soils, found after the earthquake of March 4, 1977.

LIQUEFACTION-ASSOCIATED GROUND DAMAGE DURING· THE VRANCEA EARTHQUAKE OF MARCH 4, 1977
KENJI ISHIHARA and VLAD PERLEA

SOILS AND FOUNDATIONS Vol.24, No.1; Mar.1984

After:
- 4 March 1977, Vrancea earthquake
- 20 June 1978, Thessaloniki earthquake
- 15 April 1979, Montenegro earthquake

Dynamic behaviour of soils, soil amplification and soil structure interaction. Final Report
Coordinator: Kenji Ishihara, Attila Ansal.

report published by United Nations Educational, Scientific and Cultural Organisation - UNESCO in 1982 about Bulgaria, ***Greece, Romania***, Turkey and Yugoslavia. Office of the United Nations Disaster Relief

Dynamic behaviour of soils, soil amplification and soil structure interaction
Final Report

Greece

- rely largely on SPT and less on CPT
- only few seismic refraction method used for evaluation soil properties
- no downhole method implemented
- dynamic triaxial & resonant columns (2 universities)

Romania

- DPT (German cone) and CPT
- seismic refraction method is used
- no SPT
- dynamic triaxial & resonant columns (2 research centres)

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Earthquake & Geotechnical Engineering
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Dynamic behaviour of soils, soil amplification and soil structure interaction
Final Report

Romania

2.2 In-situ testing techniques

Penetration tests are widely used for the determination of relative density, which is the main factor controlling liquefaction potential. Since the Standard Penetration Test is not suitable for liquefiable soils, the preferred methods are the Static (cone) Penetration Test and Dynamic (cone) Probing. Correlations between cone penetration resistance and relative density have been established, with regional validity.

Important events in geotechnical seismic engineering

Turkey Flat (California) experiment - 1986

12 organizations from the USA and Japan participated in this experiment. Laboratory and in situ geotechnical tests were carried out to determine lithology, velocity, density, electrical properties and dynamic properties of the ground (damping, shear modulus).

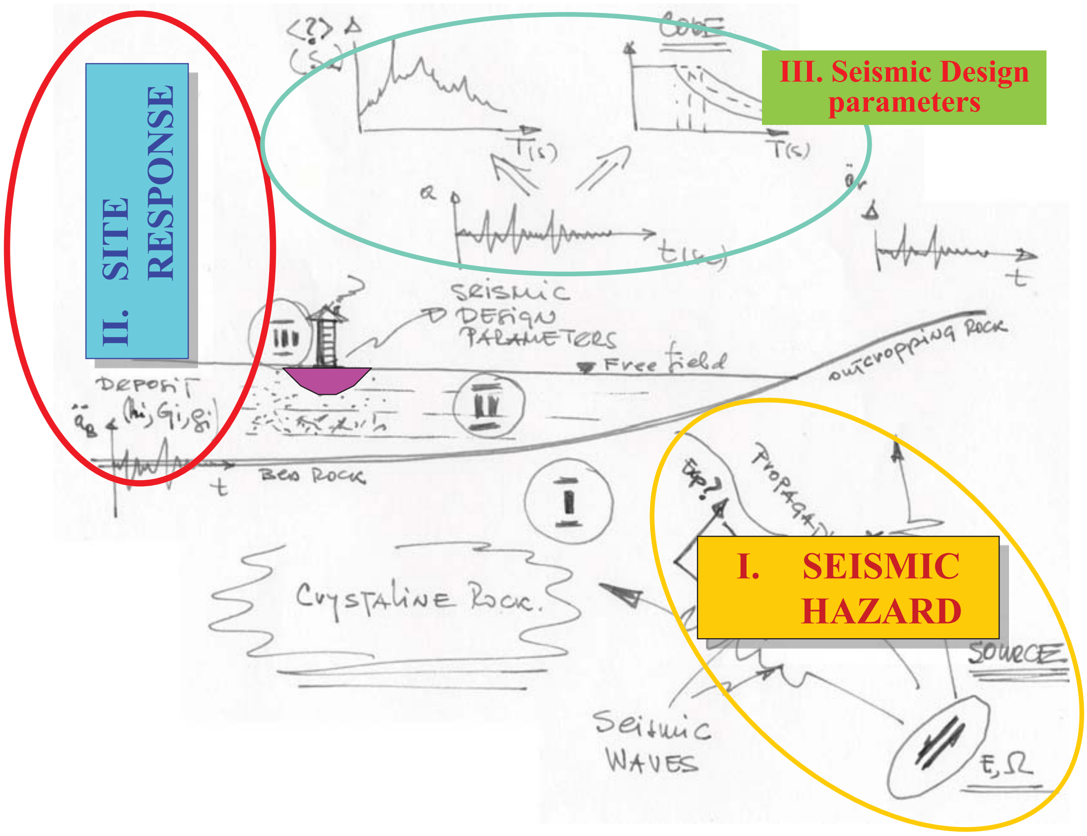
The results of the Tukey Flat experiment suggest that the accuracy of estimating light seismic movements depends more on the ***accuracy of the geotechnical model*** *used in the analysis than on the method used in calculating the response.*

Ashigara Valley in Kanto District, 80km Southwest of Tokyo experiment - 1992

"No significant difference is found in comparison of predicted spectral ratios by 1-D modelling (SHAKE) to those by a 2-D/3-D method".

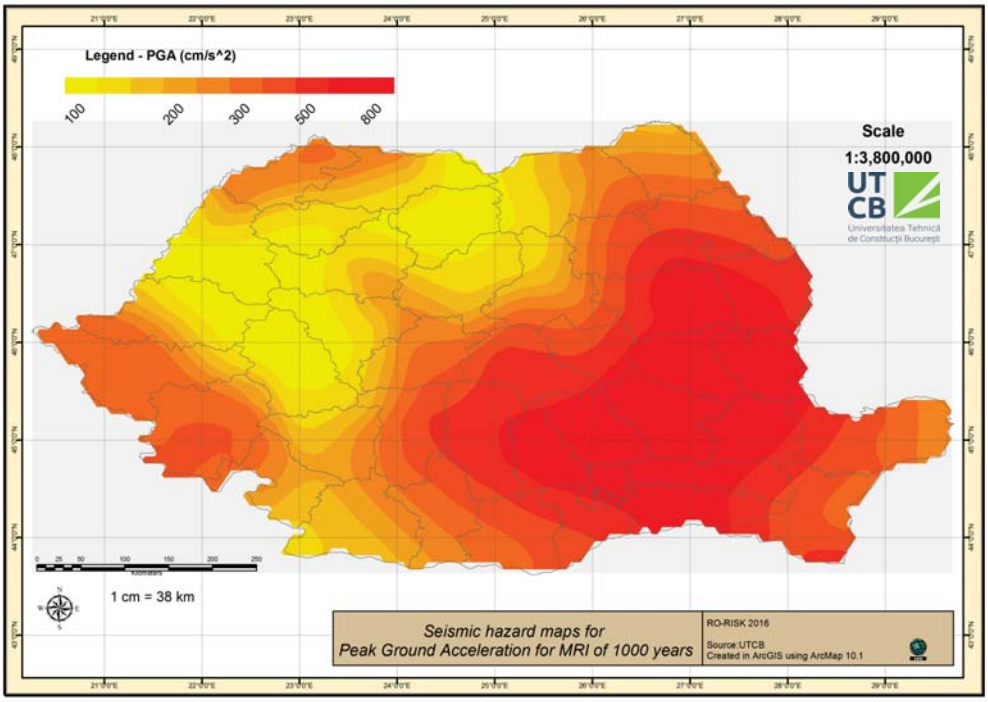
Strong historical ground motions have outlined that variability and specific parameters of layered ***unconsolidated sedimentary young deposits*** represents one of key component in site-response analysis.

To predict seismic effects of near-surface soils, comprehensive surveys are needed for the estimation of ***dynamic behavior*** *and* ***site characterization.***



16

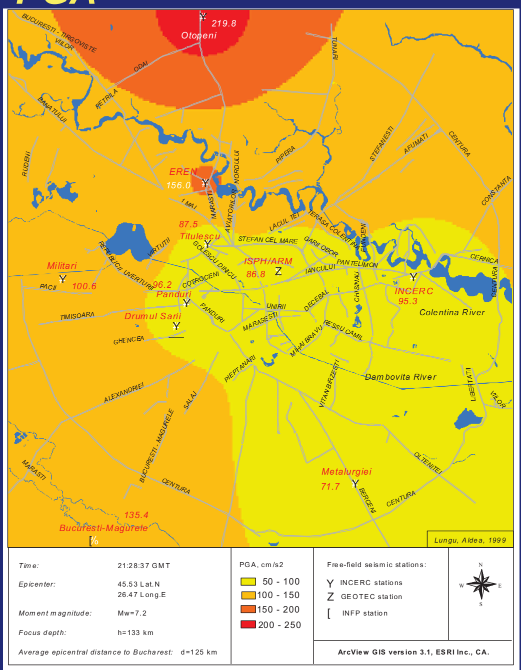
Seismic hazard of Romania – Map for the Mean Recurrence Interval of 1000 yr.



Seismic Investigation

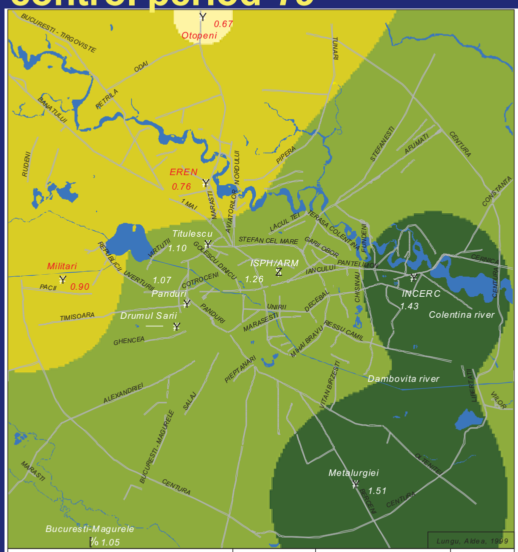
Microzonation studies

Bucharest. Aug. 30, 1986 Shake map for *PGA*



Bucharest. Aug. 30, 1986 Microzonation of control period *Tc*

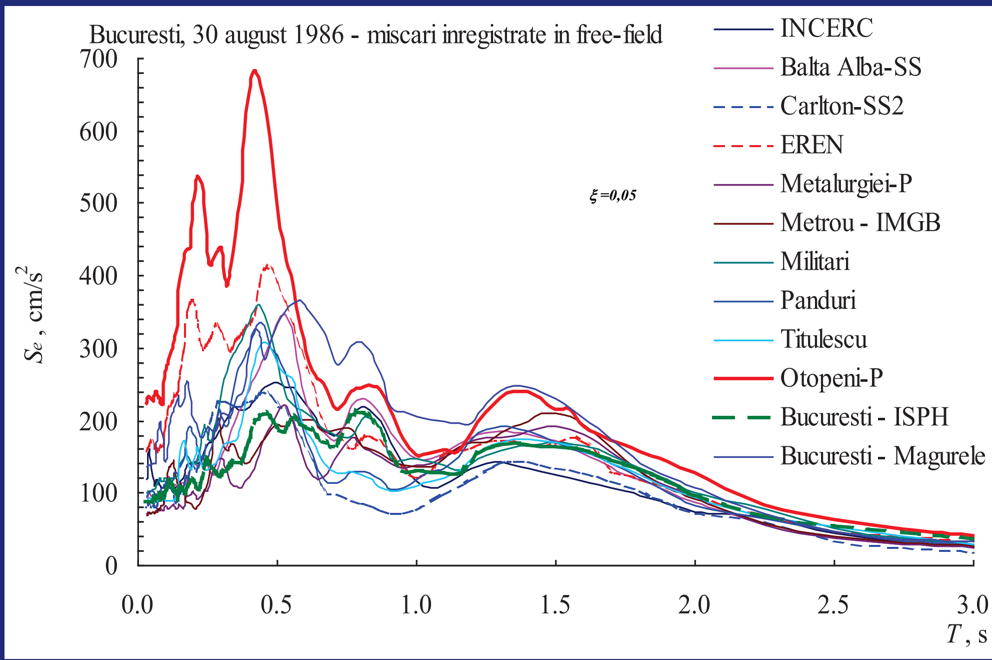
II. Site response



Lungu, D., Aldea, A., Demetriu, S., Arion, C., 2000. Zonation of seismic hazard for the city of Bucharest. Technical Report prepared for Association Française du Génie Parasismique, 59p.

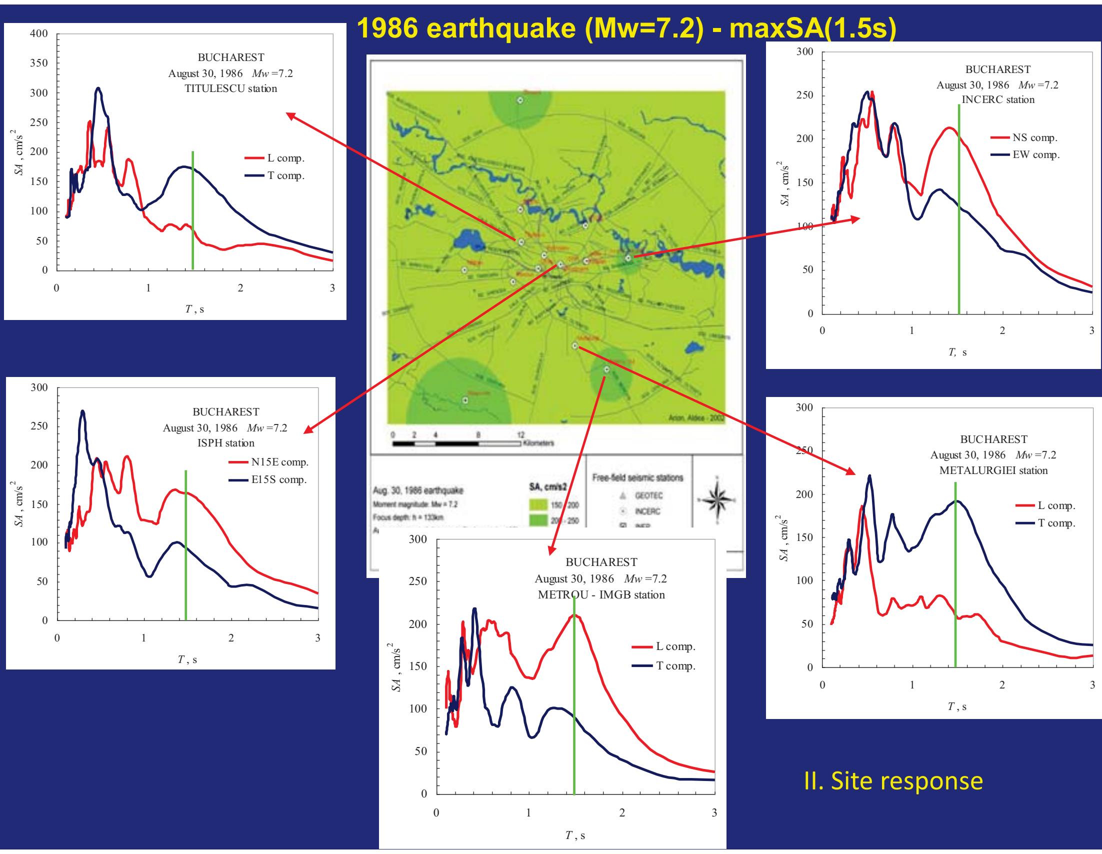
Elastic response spectra for August 30, 1986 free-field seismic records in Bucharest

II. Site response



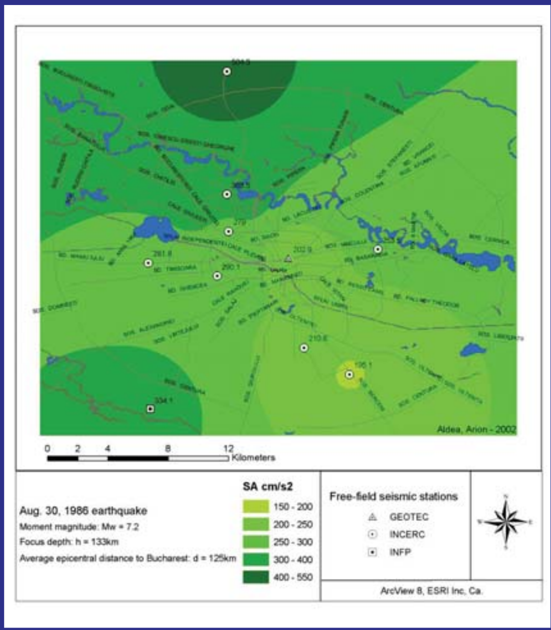
20

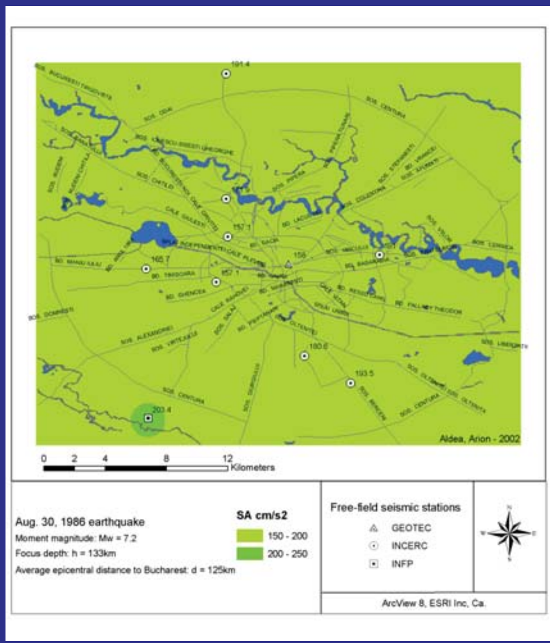
1986 earthquake (Mw=7.2) - maxSA(1.5s)



II. Site response

21

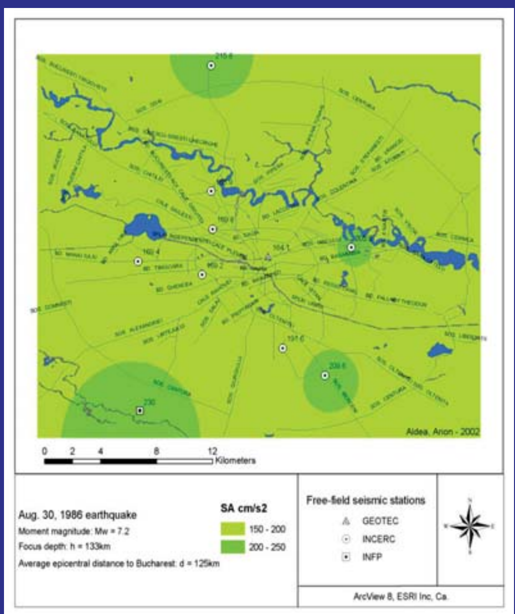


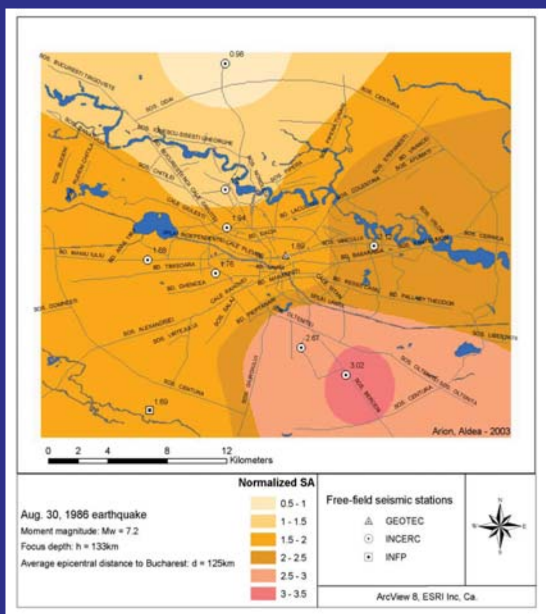


1Hz (0.5 sec) **0.625Hz (1.6 sec)**

Microzonation of spectral acceleration for different frequencies (periods)
30 August 1986 event

II. Site response





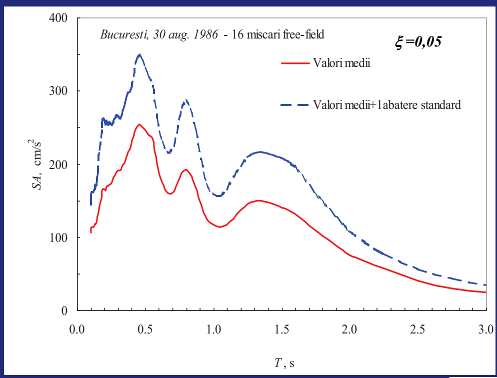
Microzonation of spectral acceleration at T=1.5s

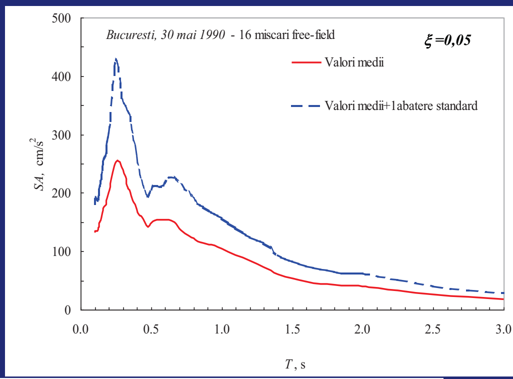
Microzonation of normalized spectral acceleration at T=1.5s

30 August 1986 event

II. Site response

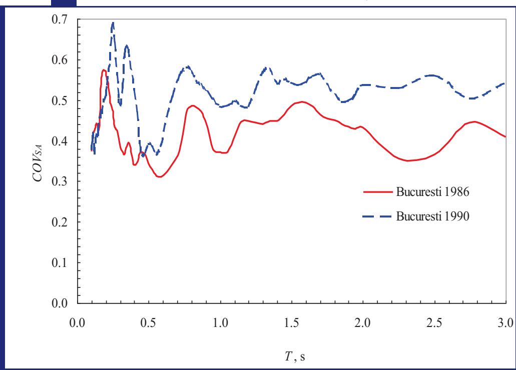
Variability of free-field seismic records in Bucharest





Mean and mean +1Stdev of the spectral acceleration recorded in Bucharest in 1986 and 1990 earthquakes

(16 free-field records)



Coefficient of variation of the spectral acceleration recorded in Bucharest in 1986 and 1990 earthquakes

II. Site response

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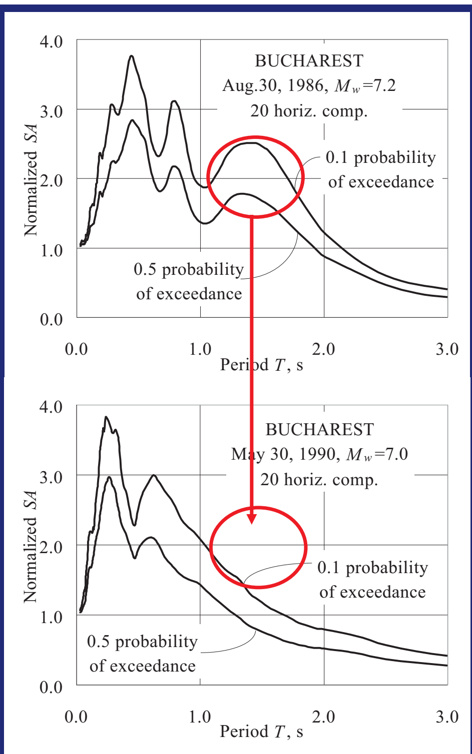
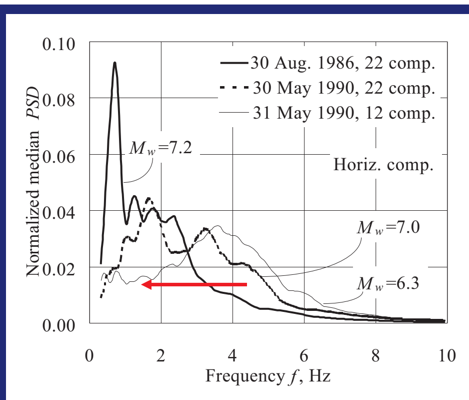


Mobility of average normalized *SA*



Mobility of average normalized *PSD*

Stronger earthquake

Narrower frequency band shifted towards lower frequencies

II. Site response

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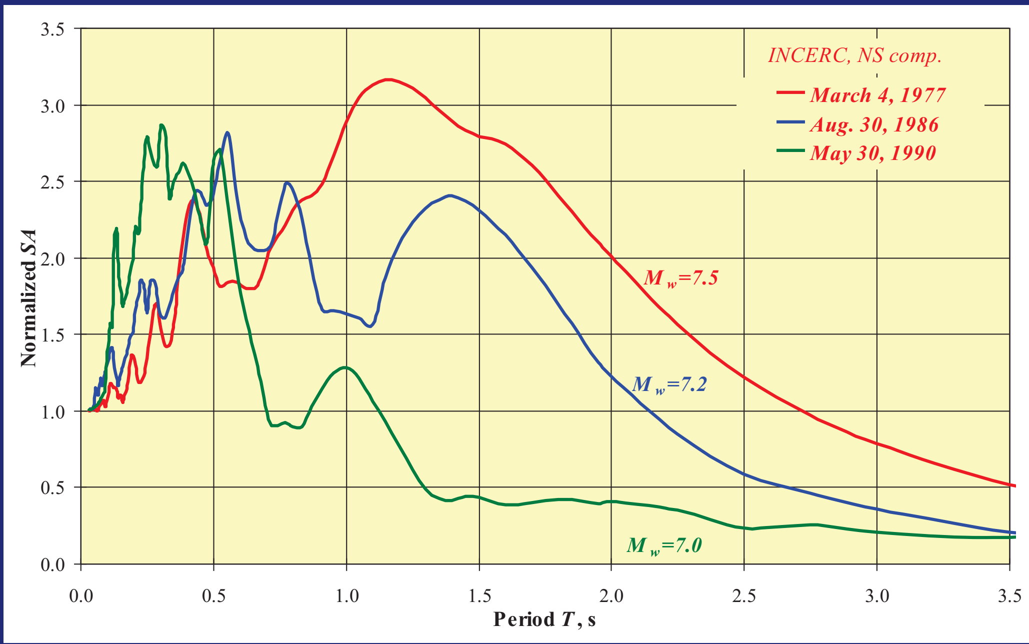
Mobility with magnitude of normalized SA spectrum for INCERC seismic station, East of Bucharest



II. Site response

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Soil testing and Investigation at CCERS

From 2003, new seismic networks, significant number of shallow and deep boreholes tests, standard penetration tests, non-invasive field techniques investigations have been carried out in Romania by National Center for Seismic Risk Reduction, NCSRR (now https://ccers.utcb.ro) by UTCB staff.

Parameters (shear modulus, G_{max} and damping, h) required for the dynamic response of geomaterials due to dynamic loads, such as traffic loads, earthquakes and machine vibrations, are being evaluated by using laboratory tests at small strain level and from in-situ seismic tests.

27

Modelling the local site conditions and estimation of seismic effects

The modern codes for earthquake resistant design, EC8, ASCE, UBC, etc., classify the soils based on

qualitative indicators (description of stratigraphy, soil type, layer thickness etc.)

and

quantitative indicators (average shear wave velocity, Standard Penetration Test results, etc.).

Objectives *for soil testing and investigation:*

✓data collection on ground motion to examine the characteristics of earthquakes

✓soil condition investigation and seismic hazard investigation

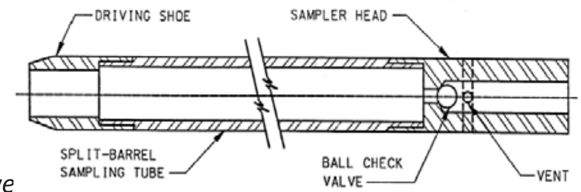
✓data collection on seismic building response to examine the buildings behavior

✓revision of strong ground motion design parameters and developing new models for strong ground motion simulation

In situ prospecting methods implemented at CCERS

Standard Penetration Test (SPT) – from 2003

- *most used geotechnical method for in situ soil investigation (<30 m);*
- *used to identify the soil stratification, the layer thickness, strength soil characteristics and other engineering properties of soil layers;*
- *easy and possible to apply to different soil types (most often used in granular material);*
- *results of SPT measurements (N_{SPT} - number of blows required to affect a segment of penetration) are used to estimate the relative firmness or consistency of cohesive soils or density of cohesionless soils;*
- *The disadvantage of the method consists in the limited shallow depth investigation (up to ~30 m) and soil disturbance, being considered an invasive geotechnical technique.*



EN ISO 22476-3

In situ prospecting methods

EN ISO 22476-3

Standard Penetration Test (SPT) – from 2003

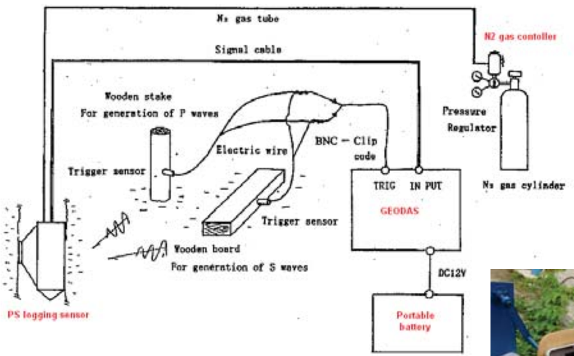




In situ prospecting methods – from 2003

Geophysical measurements: seismic downhole method

Adancime foraj, m	Site	$V_{s,30}$	T_g	$V_{s,51}$	T_g
140	INCERC	271	0.449	301	0.677
69	SPITAL	246	0.495	279	0.731
110	Victoriei	285	0.427	309	0.660
78	UTCB	310	0.393	325	0.627
66	INSTALATII	289	0.421	317	0.643
68	PRC	294	0.414	308	0.662
51	Primarie	224	0.544	264	0.772





$V_s / V_p = ((1 - 2\nu) / (2(1 - \nu))^{½}$

$G_{max} = \rho V_s^2$

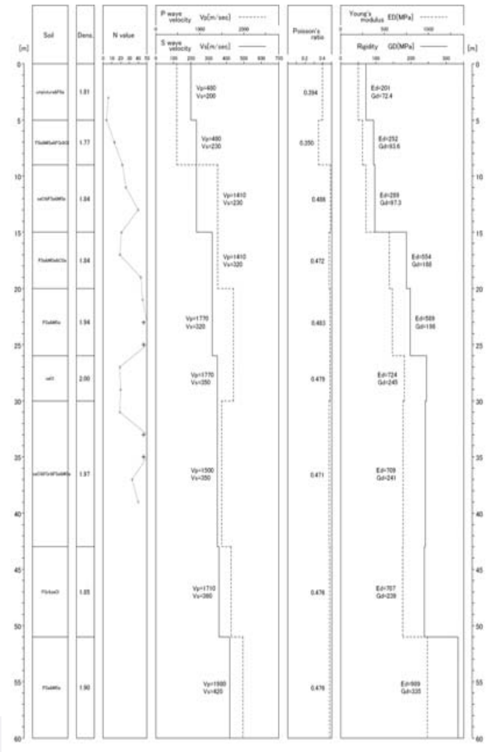
$E_{max} = 2 G_{max} (1 + \nu)$

ASTM D7400-08 "Standard Test Methods for Downhole Seismic Testing"

In situ prospecting methods – from 2003

Geophysical measurements: downhole method (PS Logging)

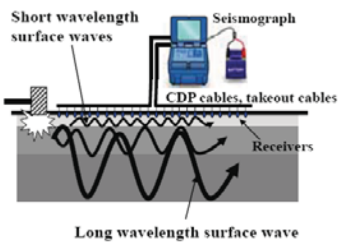
- *use the measurement of seismic waves arrival times generated by an impulse source at surface and waves travel to a sensor placed at a specific borehole depth;*
- *results of PS Logging: seismic velocity profiles (travel time data correlated with soil stratigraphy) and soil dynamic parameters*

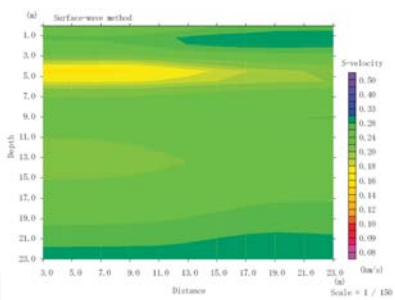


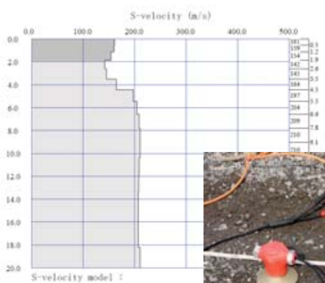
In situ prospecting methods – from 2003

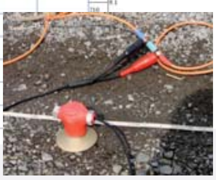
Geophysical measurements: Surface wave method (SASW or Rayleigh wave)

- *non-invasive seismic exploration method in which the dispersion character of the surface-waves is analyzed;*
- *elastic waves propagating along the surface and its energy concentrates near the ground surface;*
- *surface-wave velocity of propagation strongly depends on S-wave velocity.*









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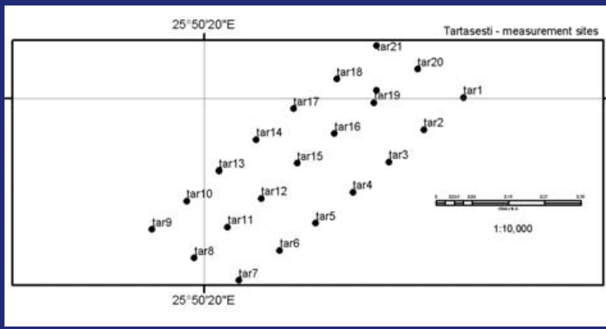

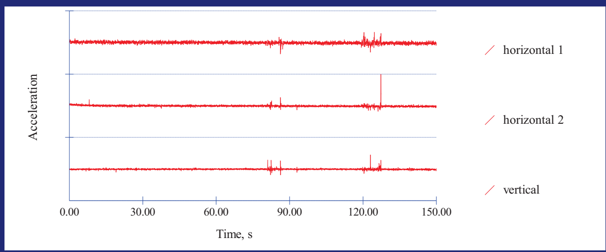

System
for Vibration measurement
(microtremors)

Arion, 2025

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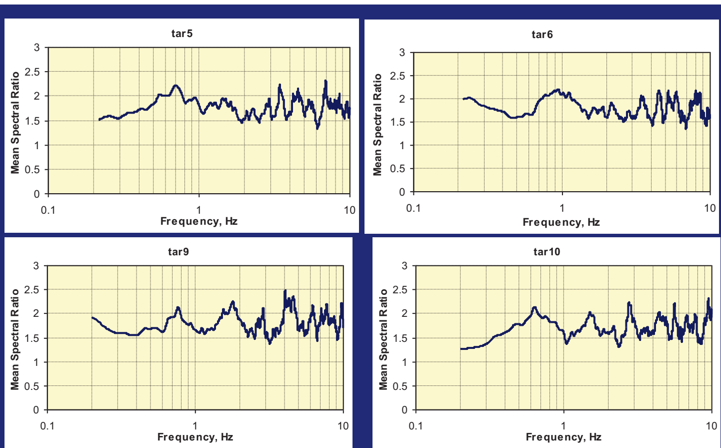
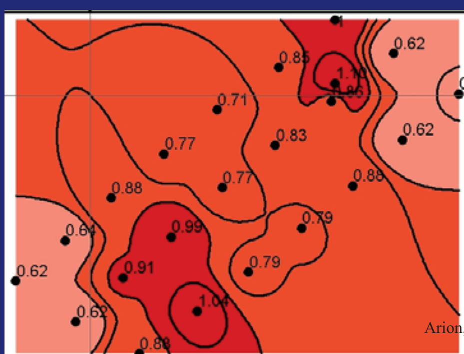


Average transfer Functions



Predominant period
of the ground vibration
average measurements~ 0.7sec

Arion, 2025

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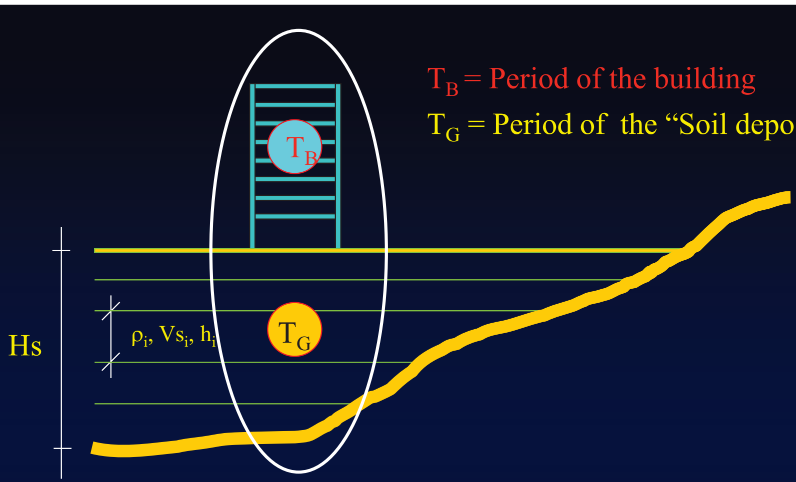
terminology



T_B = Period of the building

T_G = Period of the "Soil deposits"

$T_B \approx T_G$ Resonance domain (Bigger building response, increased damage)

$T_B \neq T_G$ Non-resonant domain (Lower building response, Lower damage)

$$T_G = 4 \cdot \sum_{i=1}^{n} \frac{H_i}{V_{si}}$$

T_G: Traits of ground (sec), i.e. fundamental natural period of ground

Specifications for Highway Bridges - Part V - Seismic Design, 1990 (Japan)

$$T^{(1)} = 4 \cdot \sqrt{\frac{2}{g} \cdot \sum_{i=1}^{n-1} \frac{H_i}{G_i} \cdot \left(\sum_{j=1}^{i} \gamma_j H_j - \frac{1}{2} \gamma_i H_i \right)}$$

γ_i – unit weight of the layer i

Hi – layer thickness i

Gi – shear modulus of the layer i

V_{si} – shear wave velocity of the layer i

g – acceleration of gravity

UT CB

Ground types – seismic classification

UT CB Universitatea Tehnică de Construcţii Bucureşti

A.3. Caracterizarea seismică a condiţiilor de teren

(1) Pentru construcţiile încadrate clasa I de importanţă-expunere şi pentru clădirile încadrate în clasa II de importanţă-expunere care au înălţimea totală supraterană mai mare de 45m se vor efectua studii specifice pentru caracterizarea seismică a condiţiilor de teren în amplasament. Aceste studii trebuie să conţină:

(i) Profilul vitezei undelor de forfecare V_s şi al undelor de compresiune V_p, pentru toate stratele de teren de la suprafaţă ... simplificat şi convenţional, profilul poate fi de ... adâncime;

(ii) Stratigrafia amplasamentului (grosimea, densitate ...

(iii) Valoarea medie ponderată a vitezei undelor de fo... considerată, $\overline{V_s}$:

P100-1/2013, Anexa A, pag.261

(2) Pe baza valorilor vitezei medii ponderate – in stratigrafia superficială cu grosime de 30m - $\overline{V_s}$, condiţiile de teren se clasifică în următoarele 4 clase:

Clasa A, teren tip roca	$\overline{V_s} \geq 760$ m/s,
Clasa B, teren tare	$360 < \overline{V_s} < 760$ m/s,
Clasa C, teren intermediar	$180 < \overline{V_s} \leq 360$ m/s,
Clasa D, teren moale	$\overline{V_s} \leq 180$ m/s.

Table 3.1: Ground types

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	–	–
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	–	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

EN1998-1:2004, pag.34

ASCE/SEI 7-16

Minimum Design Loads and Associated Criteria for Buildings and Other Structures

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50 blows/ft	>2,000 lb/ft²
D. Stiff soil	600 to 1,200 ft/s	15 to 50 blows/ft	1,000 to 2,000 lb/ft²
E. Soft clay soil	<600 ft/s	<15 blows/ft	<1,000 lb/ft²
	Any profile with more than 10 ft of soil that has the following characteristics: — Plasticity index $PI > 20$, — Moisture content $w \geq 40\%$, — Undrained shear strength $\bar{s}_u < 500$ lb/ft²		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

Note: For SI: 1 ft = 0.3048 m; 1 ft/s = 0.3048 m/s; 1 lb/ft² = 0.0479 kN/m².

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2020 NEHRP Provisions

Table 20.2-1 Site classification

Site Class	$\bar{v}_s$ Calculated Using Measured or Estimated Shear Wave Velocity Profile	
A. Hard rock	> 5,000 ft/s	> 1524 m/s
B. Medium hard rock	> 3,000 to 5,000 ft/s	> 914 m/s
BC. Soft rock	>2,100 to 3,000 ft/s	> 640 m/s
C. Very dense sand or hard clay	>1,450 to 2,100 ft/s	> 442 m/s
CD. Dense sand or very stiff clay	>1,000 to 1,450 ft/s	> 305 m/s
D. Medium dense sand or stiff clay	>700 to 1,000 ft/s	> 213 m/s
DE. Loose sand or medium stiff clay	>500 to 700 ft/s	> 152 m/s
E. Very loose sand or soft clay	< 500 ft/s	< 152 m/s
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.2.1	

Note: For SI: 1 ft = 0.3048 m ; 1 ft / s = 0.3048 m / s

National Earthquake Hazards Reduction Program (USA)

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$V_{S,30}$ is now one of the standard indicators for mapping seismic site conditions in most building codes of earthquake-prone countries.

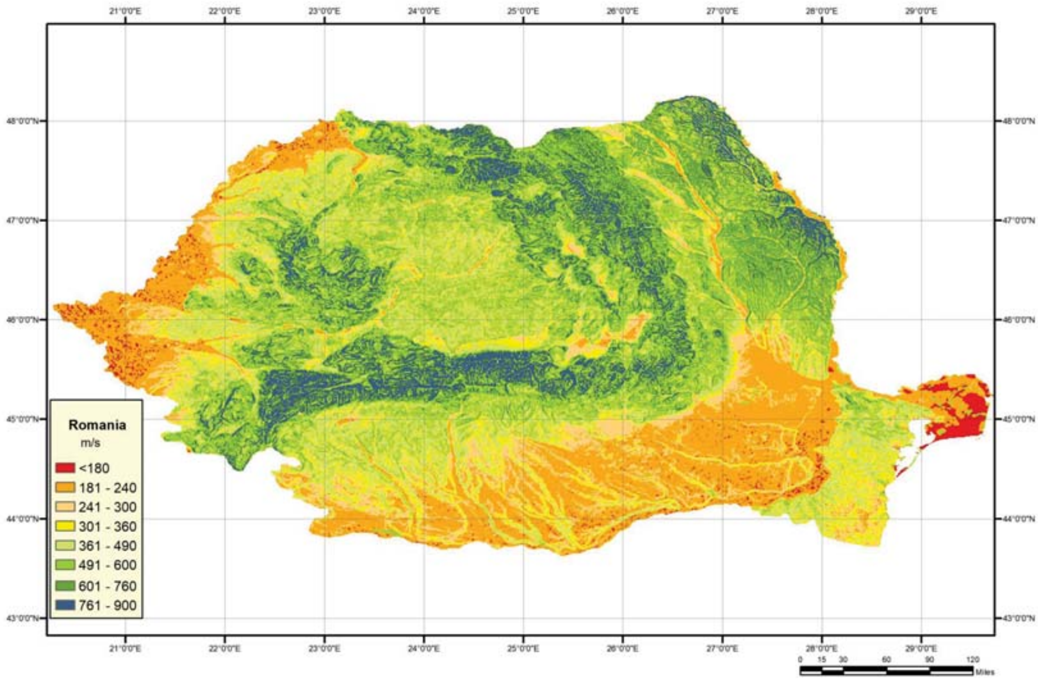
Soil classification (Allen, T.I., and Wald, D.J., 2007) and (Heath et al. 2020) proposed a methodology that correlate topographic slope data obtained from 30 arc-sec (SRTM30 – Shuttle Radar Topography Mission 30 arc-sec) topographic data recorded in 2000 by space shuttle Endeavor and $V_{S,30}$ values obtained from in-situ measurements from different sites.

The quality and density of $V_{S,30}$ measurements vary from one region to another.

Investigating the proposed $V_{S,30}$ map for Romania with the Bucharest measurement, we notice the differences between seismic downhole measured values and topographical slope estimated values of vary between −23% and +28% with a 14% mean value.

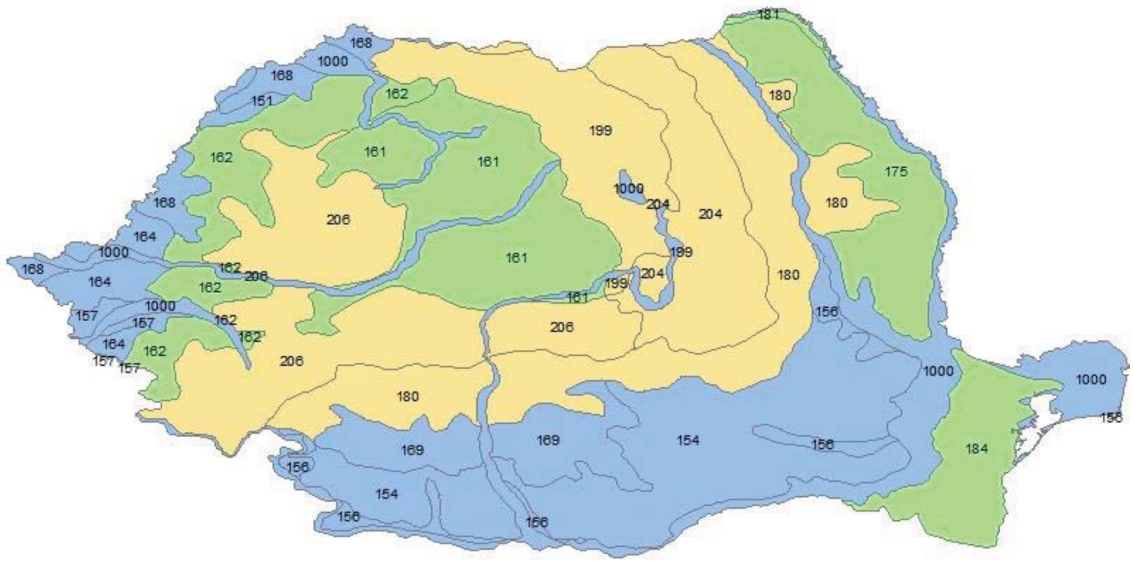
All Bucharest sites in ground type "C" according to Eurocode 8 ground type classification (shear wave velocity between 180 and 360 m/s).

The $V_{S,30}$ map for Romania created with data from 2022 USGS website



Average shear wave velocity Vs,30

Title: Soil Regions of the European Union and Adjacent Countries 1 : 5 000 000



Romania from EUSR5000 (dominant parent material, elevation class and slope class)
First stage in the corner period of response spectra zone proposal

BGR [Bundesanstalt für Geowissenschaften und Rohstoffe] (2005). Soil Regions Map of the European Union and Adjacent Countries 1:5,000,000 (Version 2.0). Special Publication, Ispra. EU catalogue number S.P.I.05.134.

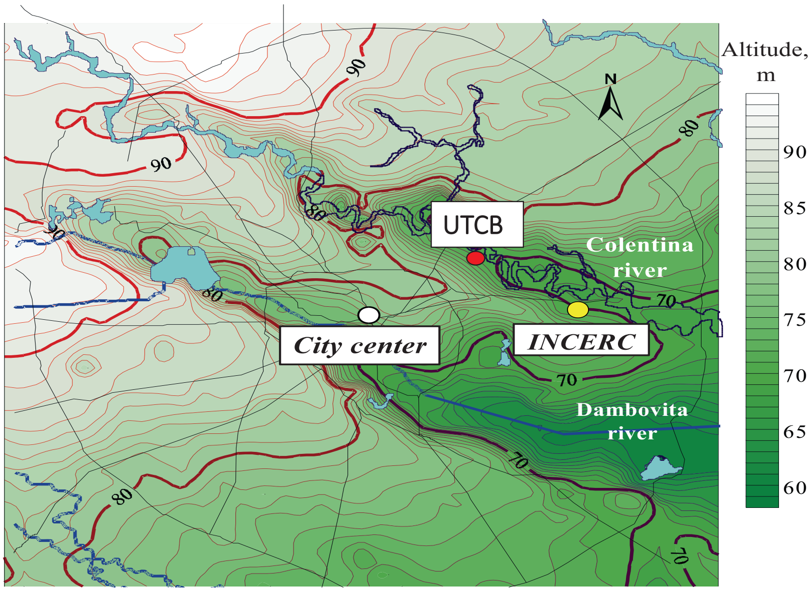
Results for Bucharest

1856 – first Bucharest photo
(by Ludwig Angerer, Austrian military pharmacist)

1950 – Tineretului, Caramidari, Serban Voda (Cocioc)

2022

Bucharest topography

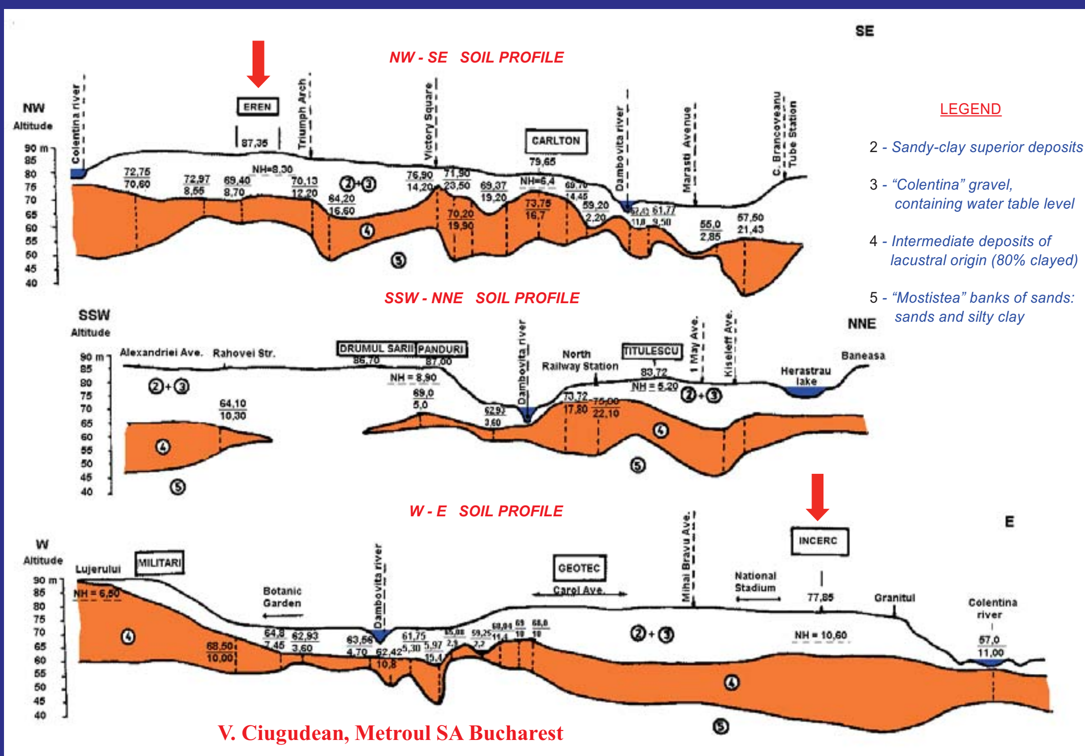


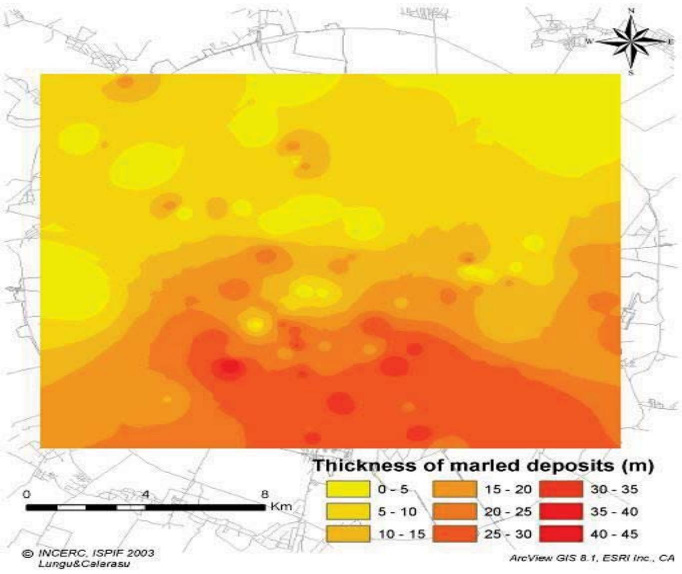
South of Romania belongs to Moesic Platform and at the north of Danube this zone it called "Romanian Plain"(Liteanu 1952, Lungu et. al, 1998).

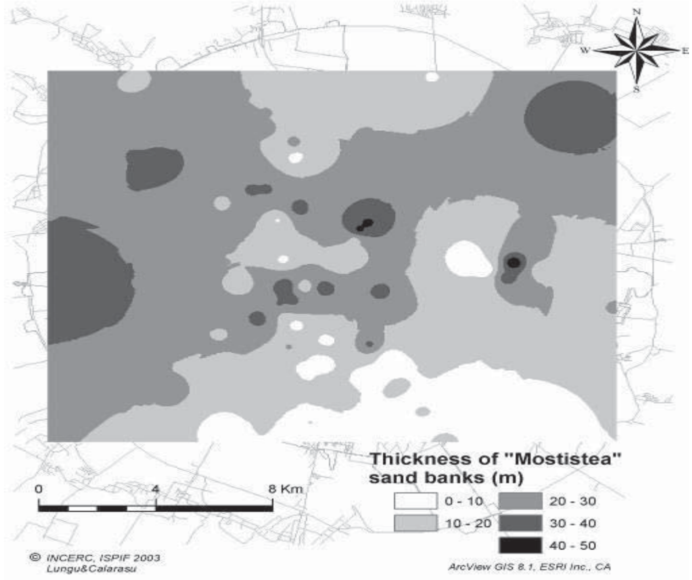
The surface geological deposits from Bucharest area are composed from unconsolidated alluvial layers of cohesive and cohesionless soils with a significant variability in thickness and spatial distribution.

The relative heterogeneity of young formations in an alluvial basin explains the peculiar site response during Vrancea strong motions

Geological zonation of Bucharest







Thickness distribution of lacustral (marled) deposits in the first 60 m (layer 4)

Thickness distribution of "Mostistea" sand banks in the first 60 m (layer 5)

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Shear wave velocity averaged on different depths (30m, 50m, 70m,100m)

UTCB conducted seismic downhole tests in more than 60 sites in Bucharest with an investigated borehole depth up to more than 140 m.

	$V_{s,30}$	$V_{s,50}$	$V_{s,70}$	$V_{s,100}$
No. of boreholes	62	41	10	4
Mean values (m/s)	281.8	314.5	336.0	357.0
Standard deviation (m/s)	22.17	17.74	18.1	14.4
Minimum value (m/s)	219	264	303	341
Maximum value (m/s)	316	348	365	379

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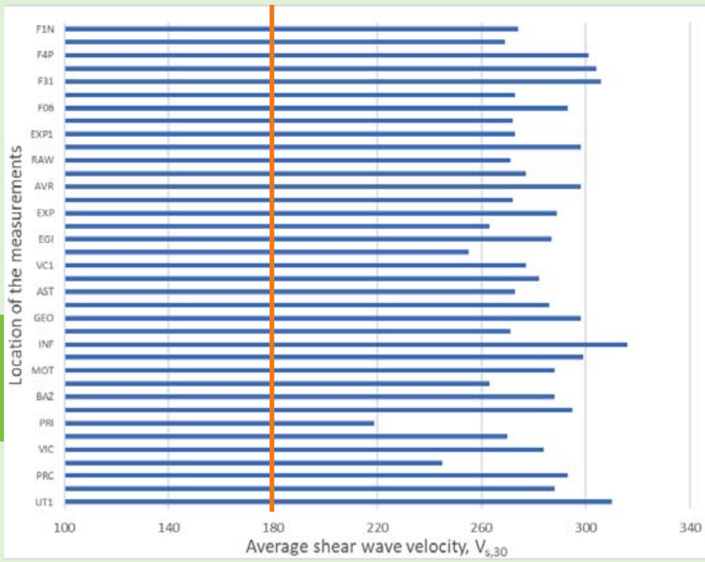
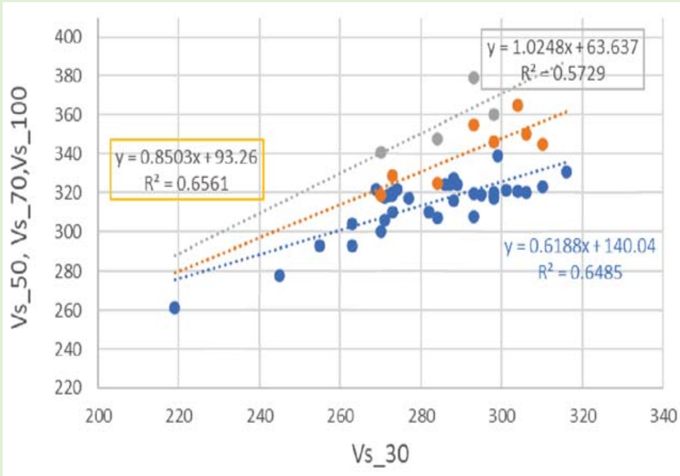


Shear-wave velocities, $\mathbf{V_{s30}}$ values from down-hole measurements

Seismic downhole tests results



Linear regression of $\mathbf{V_{s,30}}$ versus shear wave velocity averaged on different depths

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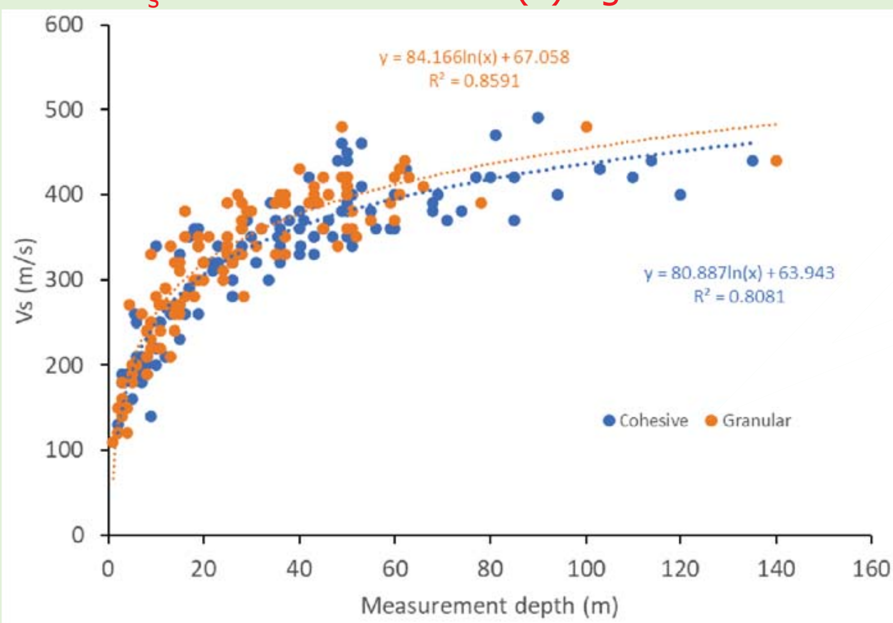
V_s = 63.943 + 80.887ln(h) cohesive soils

V_s = 67.058 + 84.166ln(h) granular soils



Seismic downhole test

Bucharest - Shear waves velocities versus depth

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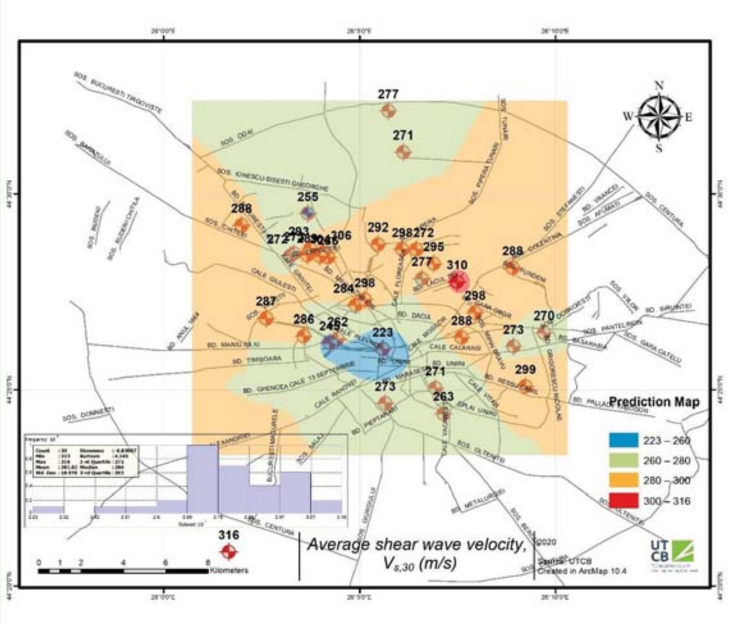
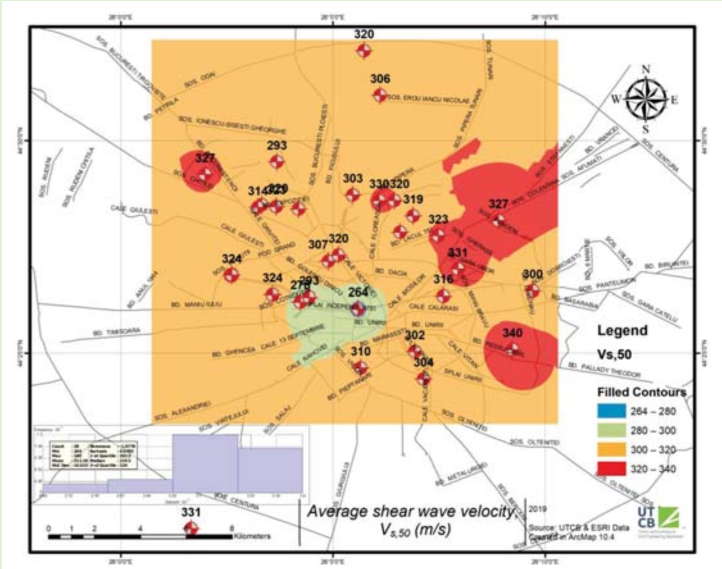


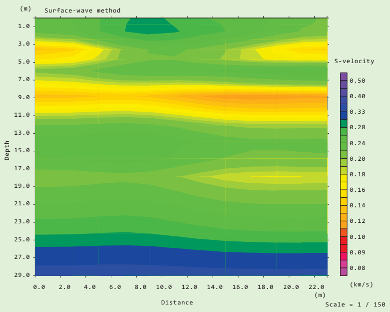
Bucharest:
Map for shear wave velocity (m/s)
averaged on 30m
(223 m/s– 316 m/s)

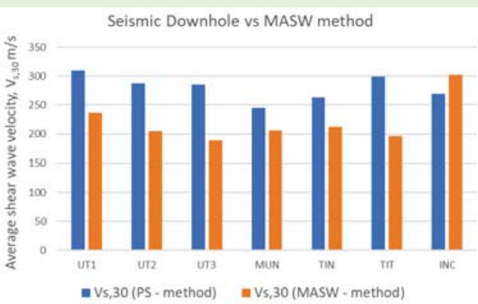
Bucharest:
Map for shear wave velocity (m/s)
averaged on 50m
(264 m/s – 340 m/s)



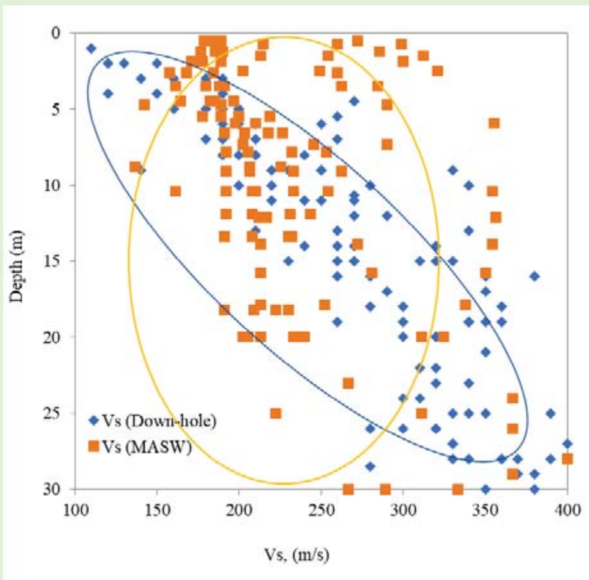
50

An alternative technique to obtain S-wave velocity profile at near-surface is the multi-channel analysis of surface waves (MASW) method.





V_S geo-data comparison:
down-hole (green) and MASW (magenta)



Differences between $V_{s,30}$ values obtained in down-hole and MASW surveys are ranging from 15-35%

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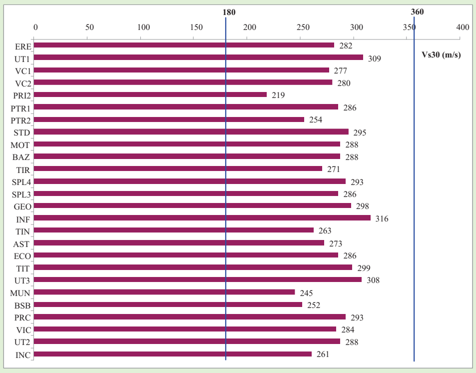
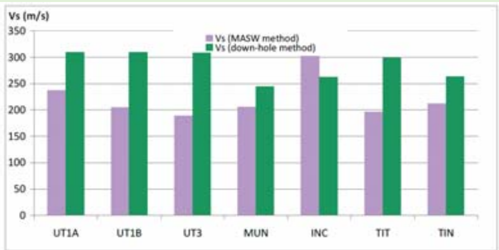
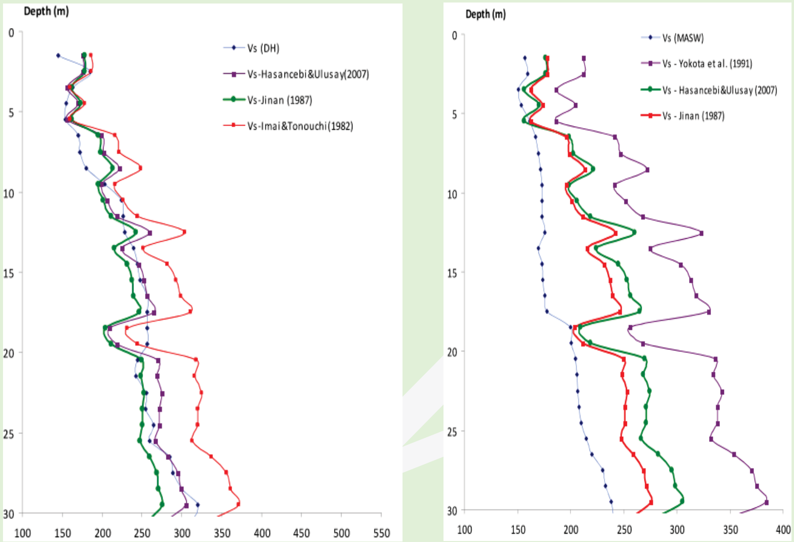


Shear-wave velocities, V_{s30} values from down-hole measurements



V_S geo-data comparison:
down-hole (green) and MASW (magenta)



Distribution of experimental and predicted shear wave velocity, V_S based on N_{60} values

Our proposed equation:

$$V_s = 101,07 \cdot N_{60}^{\;0,255}$$

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Corelations with Standard Penetration Test (SPT)

The N-value is the guideline of hardness and softness of soil.

Developed correlations for Bucharest sites between V_S and N_{SPT}

$$V_s = 101,07 \cdot N_{60}^{\;0,255}$$

$$V_s = 88,26 \cdot N_{1(60)}^{\;0,327}$$

Calarasu et al, 2018

Selected S-wave velocities values from PS logging provided 316 pairs of data for all types of soils (from 23 locations in Bucharest) and selected S-wave velocities values from MASW logging provided 99 pairs of data (from 5 locations) at depths nearest to the ones where N-SPT value was recorded.

$$V_s = 213.7 + 4.74h + 0.51N \quad \text{(Downhole)}$$

$$V_s = 162.5 + 3.77h + 0.32N \quad \text{(MASW)}$$

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$$\nu = \frac{\left[\left(\frac{V_p}{V_s}\right)^2 - 2\right]}{2\left[\left(\frac{V_p}{V_s}\right)^2 - 1\right]};$$

$$V_s = \sqrt{\frac{G}{\rho_b}}$$

$$E = \rho_b V_P$$

$$G = \frac{E}{2(1+\nu)}$$

$$V_p = \frac{2G(1-\nu)}{\rho(1-2\nu)}$$

$$G_0 = \rho \cdot V_S^2 = \frac{\gamma_t \cdot V_S^2}{98}$$

$$\frac{V_p}{V_s} = \sqrt{\frac{2(1-\nu)}{(1-2\nu)}}$$

$$E_{max} = 2\, G_{max}\,(1+\nu)$$

Table 2.: Correlations V_S – SPT values (examples)

Researcher	Proposed correlation	Soil type
Yokota et al. (1991)	$V_s = 121 \cdot N^{0,27}$	All soils
Hasancebi&Ulusay (2007)	$V_s = 104,79 \cdot N^{0,25}$	All soils
Imai & Tonouchi (1982)	$V_s = 96,9 \cdot N^{0,314}$	All soils

Calarasu et al, 2018

$$V_s = 101,07 \cdot N_{60}^{\;0,255}$$
$$V_s = 88,26 \cdot N_{1(60)}^{\;0,327}$$

Arion et al, 2022

$$V_s = 213.7 + 4.74h + 0.51N \quad \text{(Downhole)}$$

V_s = 63.94 + 80.89 ln(h) cohesive soils

V_s = 67.06 + 84.17 ln(h) granular soils

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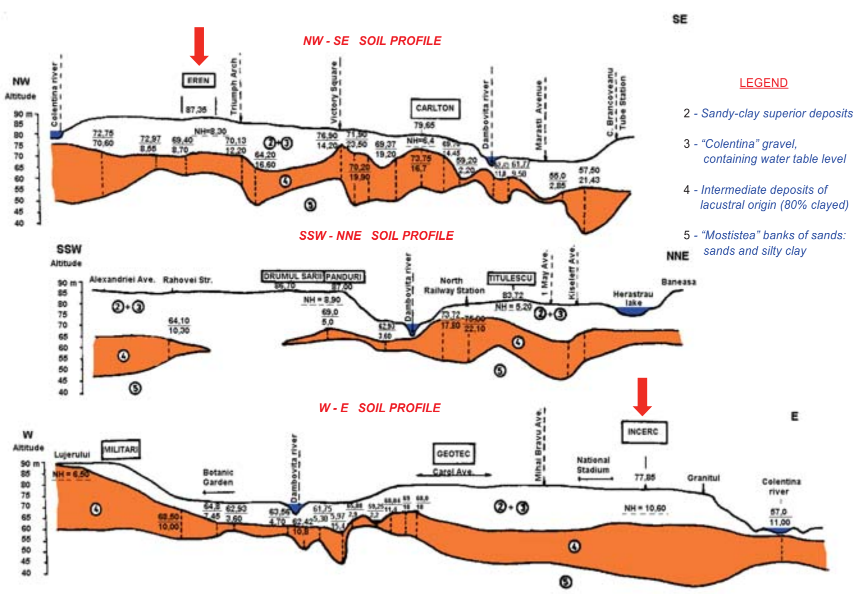
Surface geology in Bucharest



The Bucharest soil profile generally contains the following 7 typical layers (having various thickness from site to site).

Recommendation for the Vs values: for the Bucharest soil profile 7 typical layers

(1) Backfill,
(2) Sandy-clay superior deposits: shear wave velocity Vs ≅ 150m/s,
(3) "Colentina" gravel: Vs ≅ 360m/s,
(4) Intermediate deposits of lacustral origin, 80% clayed: Vs ≅ 250m/s,
(5) "Mostistea" banks of sand: Vs ≅ 290m/s,
(6) Lacustral deposits of marled clay and fine sand with some lime: Vs ≅ 340-390m/s,
(7) "Fratesti" gravel: Vs ≥ 400m/s.

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Conclusions

- over 60 sites located in Bucharest are characterized by using the shear wave velocity $V_{s,30}$, in order to obtain a comprehensive database to be used in site response analysis
- Different predicting equations for estimating the shear wave velocity V_s are proposed
- Recommendation for the Vs values for the Bucharest soil profile are offered
- SPT is also suitable for "Romanian" cohesive soils
- CPT test with electric cones due to the presence of gravely soils are difficult to be performed
- MASW surveys provide low confidence/reliable results (to many variables involved in the method). "**PLEASE DO NOT USE - E Pericoloso Sporgersi"**

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Problems related with…STRUCTURAL design optimization

- Larger values of shear wave velocities near the ground surface!!!

Why?

Increase the soil shear resistance near the surface to decrease the section of the vertical supports (decrease the shear wall and the columns transversal section/diameters) – decrease the construction cost

Bucharest city center example

Small investment in field measurements --- design optimization and large savings in construction cost

- Application of MASW in the case of Karamarmaras EQ example -

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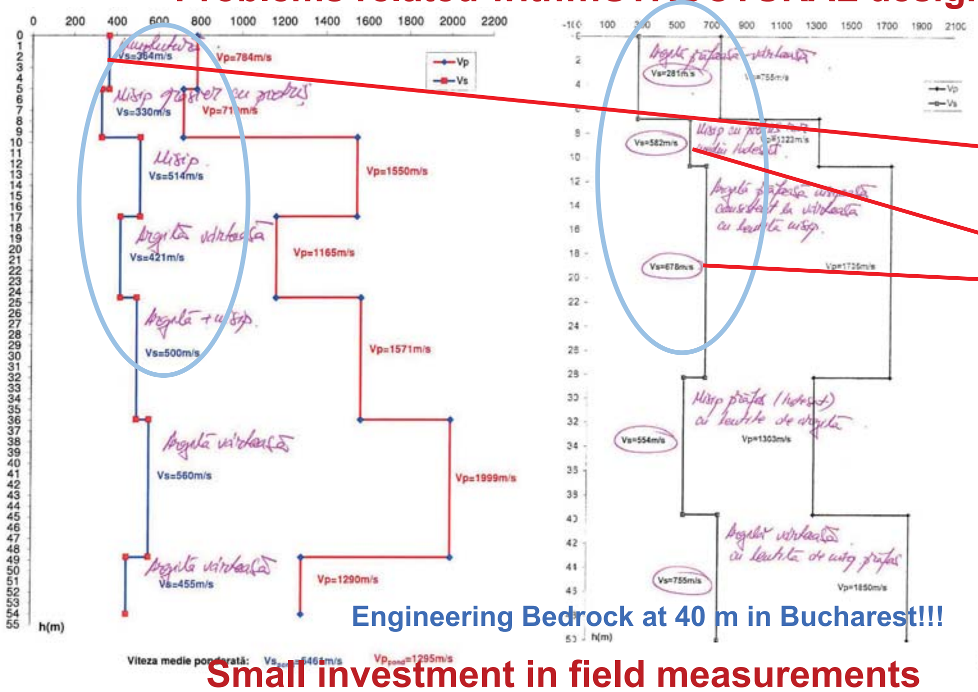
Bucharest city center example

Problems related with…STRUCTURAL design "optimization"



Vs (m/s)	Gd (MPa)
150	43
200	73
350	230
400	312
500	488
550	590
680	902

Engineering Bedrock at 40 m in Bucharest!!!

Small investment in field measurements
large savings in construction cost

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EN 1998 – part 1-1

Problems related with…costs

(5) The profile of the shear wave velocity v_s in the ground should be regarded as the most reliable predictor of the site-dependent characteristics of the seismic action at stable sites.

direct measurements should be preferred, performed **either** through ***invasive*** (in-hole measurements) or ***non-invasive*** (e.g. analysis of surface waves) techniques.

"PLEASE DO NOT USE - E Pericoloso Sporgersi"

SEE EN 1997-2
clause 10.4. and Table 10.2

MASW (surface waves) measurement – cost **x** euro

Downhole measurement – cost **4x** euro

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Starting with 2004 we have been carried out dynamic triaxial laboratory tests at ***Seismic Risk Assessment Research Center*** https://ccers.utcb.ro) by UTCB staff.

We managed the all stages of the investigations:
- Sampling (doble core barrel – split or full tube),
- Preservation and transportation of the samples,
- Preparation of the specimens,
- Testing,
- Reporting.

All the staff was trained in Japan at Tokyo Soil Research, Tokyo University, Waseda University and Istanbul Technical University.

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The equipment's, the data acquisition and processing systems and triaxial testing equipment is located at Seismic Risk Assessment Research Center, Technical University of Civil Engineering formerly installed at National Center for Seismic Risk Reduction Bucharest Romania (NCSRR) was made by Seiken and donated from Japan International Cooperation Agency (JICA) through the Technical Cooperation Project on the Reduction of Seismic Risk for Buildings and Structures in Romania.

The NCSRR triaxial equipment can solve the dynamic problems with soils subjected to a strain level as small as 10^{-6} (used to evaluate the soil strength in comparison with stresses induced by external loading and the settlement of ground or structures associated with the deformation of soils.

Japonese standard - Standards of Japanese Geotechnical Society for Laboratory Shear Tests – JGS 0542-2000 „Method for Cyclic Triaxial Test to Determine Deformation Proprieties of Geomaterials"

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Tipuri de încercari ce pot fi realizate cu echipamentul triaxial

Încercare	Test	Tip de teren analizat
STATICĂ	Test triaxial neconsolidat nedrenat (*UU* Test) Test triaxial consolidat nedrenat (*CU* Test) Test triaxial consolidat nedrenat cu măsurarea presiunii apei din pori ($\overline{CU}$ Test) Test triaxial consolidat drenat (*CD* Test)	**Nisipuri, Argile**
CICLICĂ	Test triaxial ciclic nedrenat (lichefiere) Test triaxial ciclic pentru determinarea proprietăţilor de deformaţie ale terenului	**Nisipuri, Argile**

Wave Propagation Test (ultrasonic pulse test/bender element)

Production of the excitation signal (the -wave synthesizer) and Signal reception (high-resolution oscilloscope)

Elementele de tip bender
By GEONOR

$$V_s = \frac{L_t}{T_t}$$

$$G = \rho \cdot V_s^{\,2}$$

The wave propagation test in the triaxial cell is used to directly obtain the dynamic shear modulus G.

Simulation of natural or artificial phenomenon in the laboratory tests

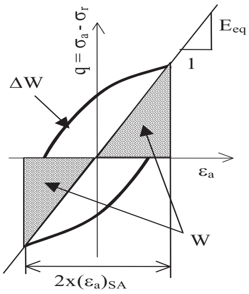
Number of cycles	Phenomenon	Duration of loading	Effect
1	Dropping bombs or blasting	10^{-3} – 10^{-2} seconds	Impulse or shock load
10 – 20 with different amplitudes	Earthquakes (the period of each impulse is between 0.1 to 3 seconds)	0.02 – 1 seconds	
100 – 1000	Pile driving, vibro-compaction	Frequency of the loads 10 – 60 Hz	
10^4 - 10^5	Machine foundation (for compressors, electric generator)	Frequency of the loads 10 – 60 Hz	
Very large	Parking, water waves, pavements of railroads,	0.1 – few seconds	Fatigue Repetition effect

Variation of soil properties with strain

	Magnitude of strain	10^{-6}	10^{-5}	10^{-4}	10^{-3}	10^{-2}	10^{-1}
	Phenomena	Wave propagation, vibration		Cracks, differential settlement		Slide, compaction, liquefaction	
	Mechanical characteristics	Elastic		Elasto-Plastic		Failure	
	Effect of load repetition						
	Effect of rate of loading						
	Constants	Shear modulus, Poisson's ratio, damping				Angle of internal friction, cohesion	
In situ measurements	Seismic wave method						
In situ measurements	In-situ vibration test						
In situ measurements	Repeated loading test						
Laboratory measurements	Wave propagation, precise test						
Laboratory measurements	Resonant column, precise test						
Laboratory measurements	Repeated loading test						
	Analytical Model	Linear elastic model		Viscoelastic model		Load history tracing type model	
	Method of response analysis	Linear method		Equivalent linear method		Step-by-step integration method	

Laboratory investigation

Dynamic Triaxial test

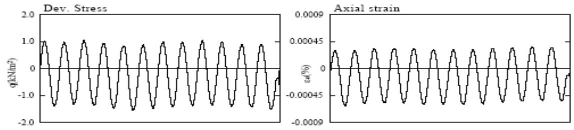


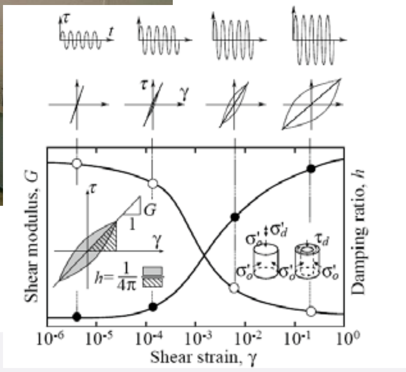
Method for Cyclic Triaxial Test to Determine Deformation Properties of Geomaterials (Loading process)

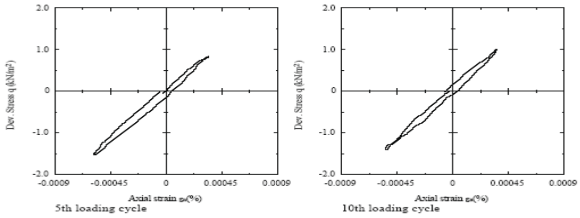
Project — Date 2006/10/07

Sample(Depth) 28.50 - 29.00 — Tested by Ing. Neagu C.

Soil name		[illegible]		Sample No.	P20	Stage	1
Before of loading	Ax.Disp ΔH_L cm	0.000		S.A.Dev.stress σ_d kN/m²	S.A.Ax.strain $(\varepsilon a)_{SA}$ %	E_{eq} MN/m²	h %
	Drainge ΔV_L cm³	0.000					
	Height H_i cm	10.000	2	1.161	0.00044	261.5	4.97
	Volume V_i cm³	193.92	3	1.215	0.00045	267.0	4.11
	Area A_i cm²	19.392	4	1.199	0.00046	261.5	2.95
			5	1.188	0.00047	255.1	4.52
			6	1.180	0.00045	260.3	4.28
After	Ax.Disp ΔH_L cm	0.001	7	1.205	0.00045	265.0	4.41
	Drainge ΔV_L cm³	-0.010	8	1.181	0.00046	258.9	4.44
			9	1.192	0.00045	262.8	4.64
			10	1.207	0.00045	268.8	3.93







$$E_{eq} = \frac{\sigma_d}{(\varepsilon_a)_{SA}} \times \frac{1}{10}$$

Sampling, preparation and testing of the soil samples

UTCB
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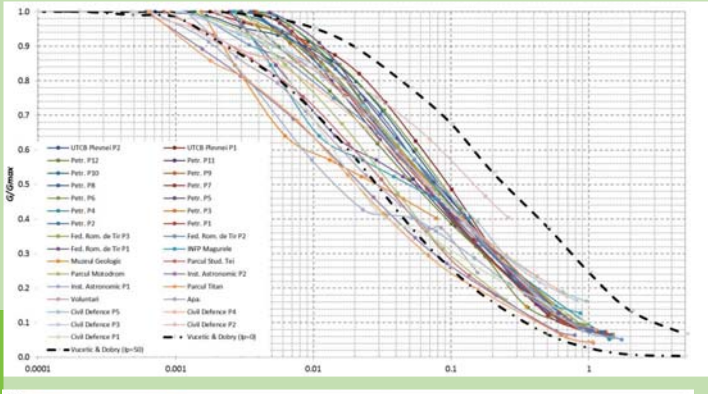
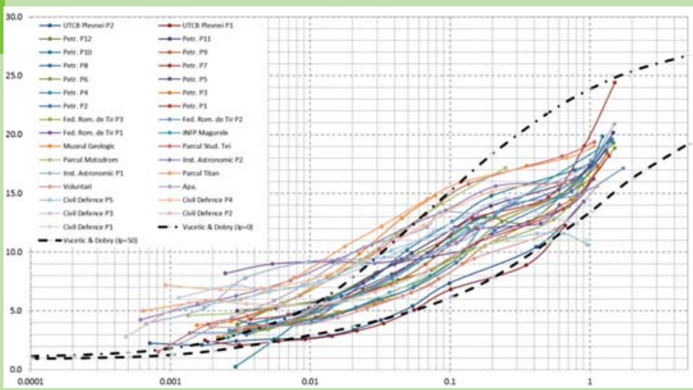
Test results on clayed samples and comparison with analytical model curves





shear modulus ratio G/G_{max} versus shear strain

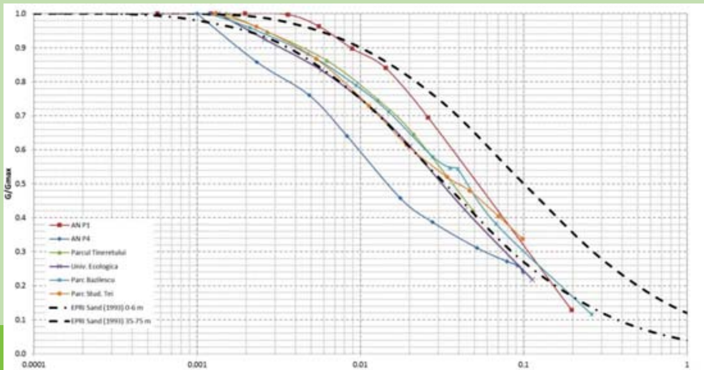
and

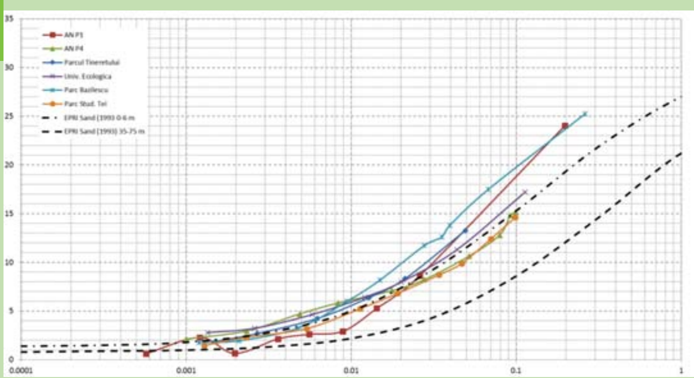
the strain dependent damping

The deepest clayed soils samples were taken at 67 m by using double core barrel sampler

UTCB
Universitatea Tehnică de Construcții București

Test results on sandy samples and comparison with analytical model curves



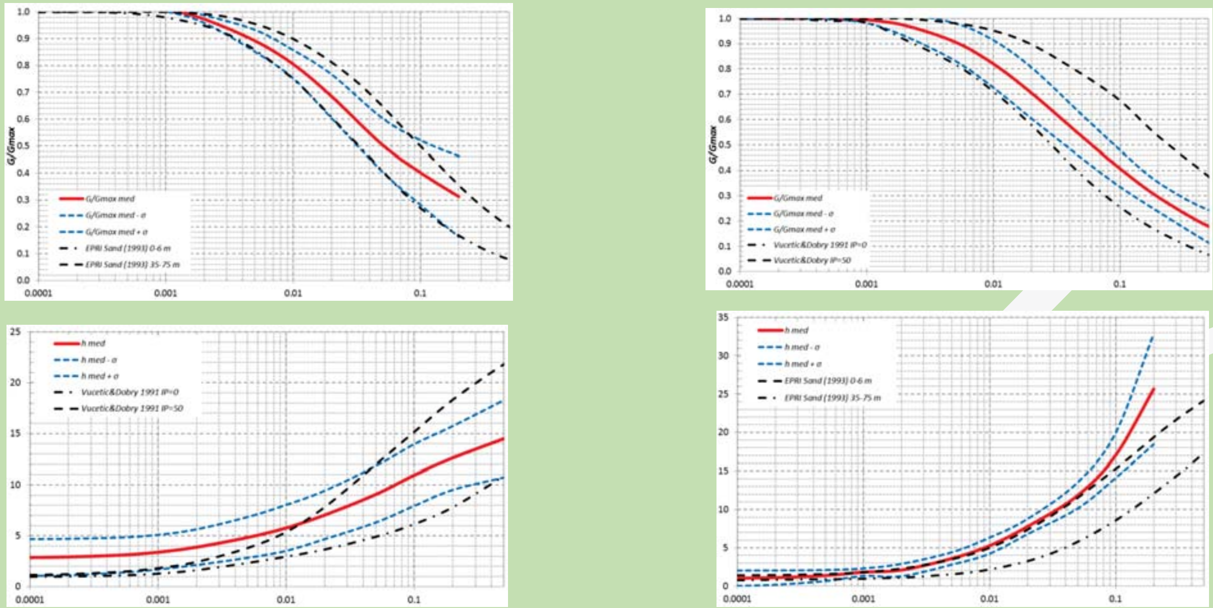


shear modulus ratio G/G_{max} versus shear strain

and

the strain dependent damping

The deepest sandy soils samples were taken at 39 m by using double core barrel sampler



Proposed curves for clayed soils	Proposed curves for sandy soils

mean curves (med) and med $\pm\sigma$ for the shear modulus ratio G/G_{max} versus shear strain
and
the strain dependent damping
and comparison with analytical model curves

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points	Cyclic shear strain	Statistical parameters of h		Statistical indicators of G/G_{max}	
	γ_a (%)	m_h	σ_h	$m_{G/Gmax}$	$\sigma_{G/Gmax}$
1	0.0001	1.032	1.020	1.044	0.026
2	0.0002	1.120	0.945	1.040	0.024
3	0.0005	1.386	0.736	1.029	0.018
4	0.001	1.828	0.503	1.011	0.010
5	0.002	2.104	0.816	0.973	0.015
6	0.005	3.611	0.821	0.892	0.041
7	0.01	5.289	1.058	0.805	0.053
8	0.02	7.794	1.003	0.685	0.082
9	0.05	11.852	1.705	0.505	0.099
10	0.1	17.057	2.934	0.402	0.120
11	0.2	25.668	7.205	0.313	0.150

Statistical indicators (Average, *m* and standard deviation, σ values) for the shear modulus ratio G/G_{max} and damping *h* and corresponding cyclic shear strain for
Bucharest sandy soils

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Technical committee, TC29 (Stress-strain and Strength Testing of Geomaterials) of International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE), has started international parallel test on the measurement of G_{max} using bender elements (BE).

The report of the test result was prepared based upon the submissions from 23 institutions worldwide. The participated institutions were total of 15 from Asia (Japan-11, China-1 & Korea-1), 9 from Europe (France-2, Italy-2, Finland-1, Holland-1, Portugal-1, ***Rumania-1*** & England-1) and one from North America (Canada-1).

Yamashita, Satoshi & Kawaguchi, Takayuki & Nakata, Yukio & Mikami, Takeko & Fujiwara, Teruyuki & Shibuya, Satoru. (2009).
Interpretation of International Parallel Test on the Measurement of G_{max} Using Bender Elements.
SOILS AND FOUNDATIONS. 49. 631-650. 10.3208/sandf.49.631.

*To many uncertainties in performing the laboratory test to predict the G_{max} by using the **low cost** equipment/method.*

Problems related with damping measurements at very small strains ($<10^{-5}$)

- Effects of bedding error at the top and bottom ends of specimen (the deformation of the loading piston and specimen cap),
- Frequency of loading
- Time gap; example (1/100 sec) between the force and displacement measurement can increase the damping.
- Noise of the power supply
- Sensitivity of the pickup (output of the pickup) (A/D –converter); 16 bit (2^{16}) or 12bit (2^{12})
- Signal/noise ratio – S/N ratio
- Cell pressure capacity (bigger – small resolution; small capacity- fine resolution) – (Question: what is BIG and what is SMALL)
- Type of soil specimen (stiff clay, soft rock – more problems in measurements)

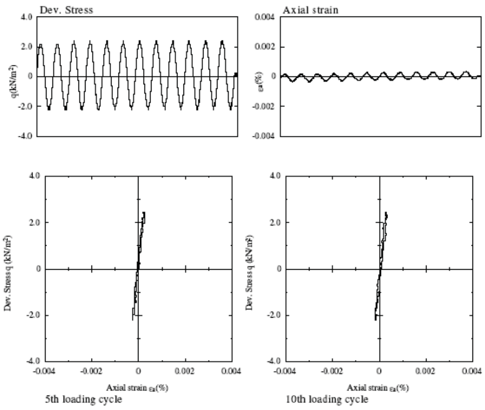
Problems related with damping measurements at very small strains ($<10^{-5}$)

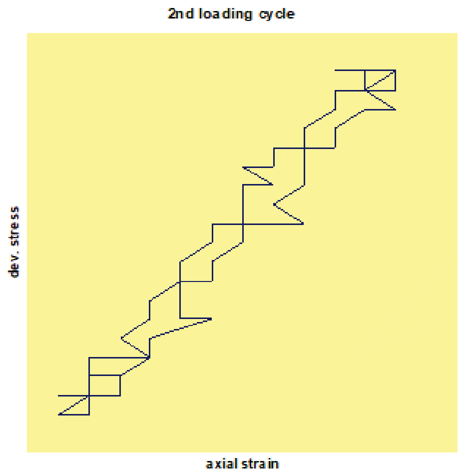
Method for Cyclic Triaxial Test to Determine Deformation Properties of Geomaterials (Loading process)

Project Hazard Seismic Bucuresti   Date 2003/12/05

Sample(Depth) 15.3 - 15.7   Tested by

	Soil name	argila slab nisipoasa	Sample No.	13	Stage	1
Before of loading	Ax.Disp ΔH_L cm	0.000		S.A.Dev.stress σ_d kN/m²	S.A.Ax.strain $(\varepsilon a)_{SA}$ %	E_{eq} kN/m² / h %
	Drainge ΔV_L cm³	0.000				
	Height H_i cm	10.102	2	2.198	0.00027	826.8 / 5.63
	Volume V_i cm³	201.99	3	2.320	0.00024	960.0 / 6.22
	Area A_i cm²	19.995	4	2.320	0.00027	872.7 / 4.87
			5	2.320	0.00024	960.0 / 6.06
			6	2.320	0.00027	872.7 / 5.18
After	Ax.Disp ΔH_L cm	0.000	7	2.320	0.00027	872.7 / 5.18
	Drainge ΔV_L cm³	-0.269	8	2.320	0.00024	960.0 / 4.31
			9	2.320	0.00027	872.7 / 6.70
			10	2.320	0.00027	872.7 / 5.48





Example:
- damping **h= 5.63%**
- axial strain 0.00027%
- dev. stress 2.3kN/m²

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Problems related with damping measurements at very small strains ($<10^{-5}$)

Proposed solutions:

- recommend the use of LDT type sensors instead of measurement system made with gap sensors.

This comment might be questionable since the gap sensors system is used in many places in Japan (Waseda Univ.- Akagi Lab., Tokyo Soil Research, Tokyo Univ., Science University of Tokyo).

Accepted solution:

- Changed the cell pressure capacity (from 2KN to 500N)
- Changed the A/D Convertor to 16bit

73

International disasters

1. Norway microzonation of quick clay
2. Kahramanmaraş Turkey, 2023 (site classification of seismic station)

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1. Case study- Norway – microzonation of quick clay!!!

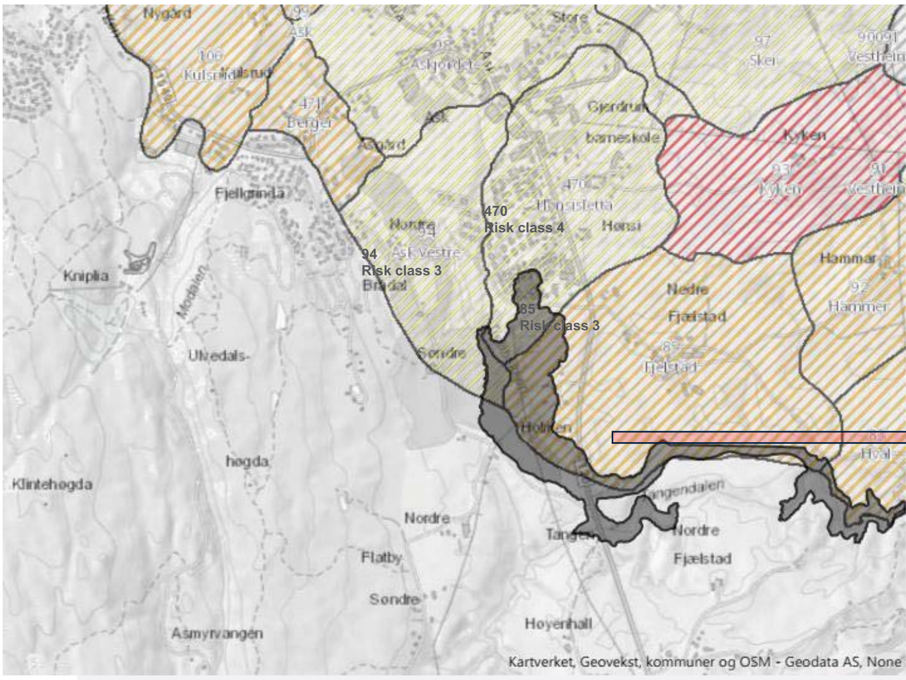
www.utcb.ro

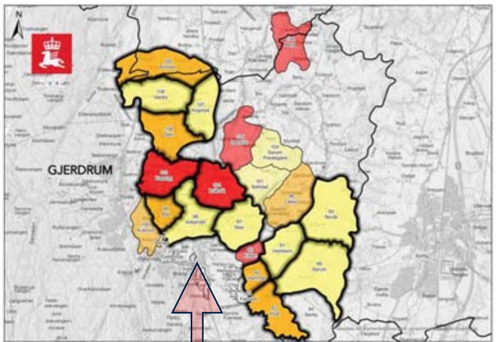
"There have never been landslides here"

75

Ten people lost their lives in the disaster on the night of December 30, 2020. An area of about 300 by 700 meters collapsed, taking with it more than 30 residential units in the Nystulia housing estate. 1,600 residents were evacuated.

In February 2022, Gjerdrum Municipality was charged with failing to follow up on warnings and for failing to take action. In November 2022, it became clear that the state attorney had dropped the case.





NVE mapping tool of quick clay

www.utcb.ro

Landslide due to quick clay in Gjerdrum

NVE-The Norwegian Water Resources and Energy Directorate

https://www.nve.no/

The Norwegian Water Resources and Energy Directorate (NVE) has a mapping tool where you can check areas with quick clay. Regulation plans can also be found at the municipalities.

Mapping of quickclay in Norway started - **after** 29 April 1978, **Rissa** the largest quick clay **landslide** in Norway in the 20th century, and is ongoing, and new zones are created every year, according to NVE.

The mapping of quick clay zones gives the municipality and private individuals a good tool, including in land use planning.

Over 100,000 people live in mapped quick clay zones in Norway. Over 2,300 zones have been mapped to date. NVE will continue to prioritize the mapping of quick clay zones in the areas where the risk of landslides is greatest.

The mapping includes both updating existing maps and developing new caution maps for quick clay landslides. These maps will help identify areas that require further investigations and potential protective measures.

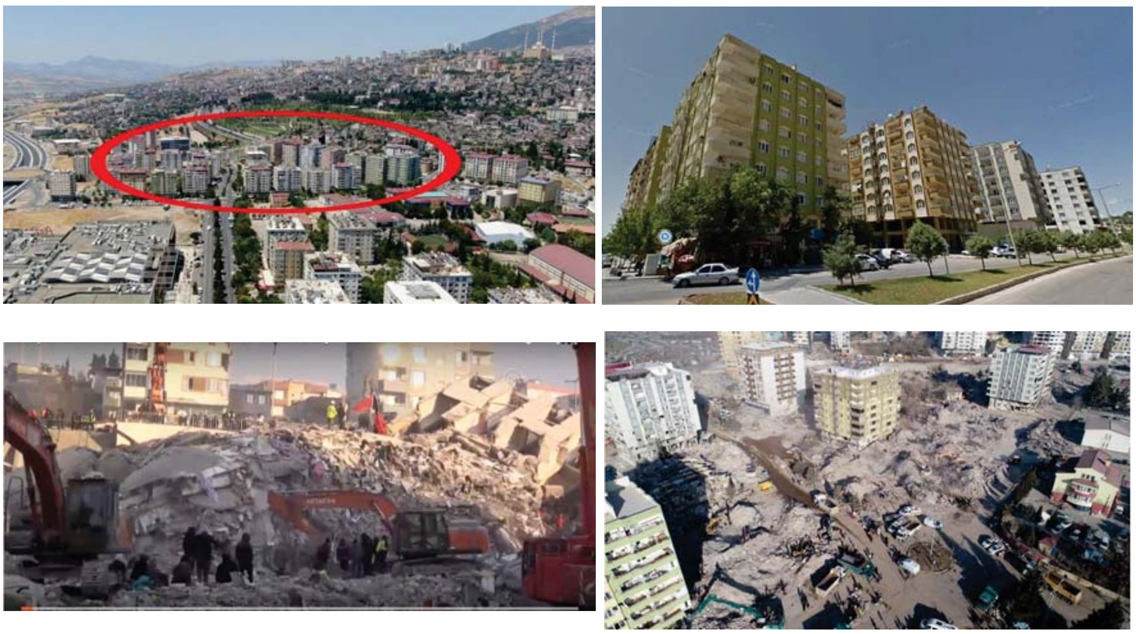
If Norway did not succeed with microzonation maps for quick clay (relatively low-cost investigation method) after 40 years of investigations done by NGI (the program **started after 29 April 1978, Rissa** the largest quick clay landslide in Norway in the 20th century) how about low budget countries…

think BIG decide SMALL.

2. Case study- Turkey

"There has never been a riverbed here"

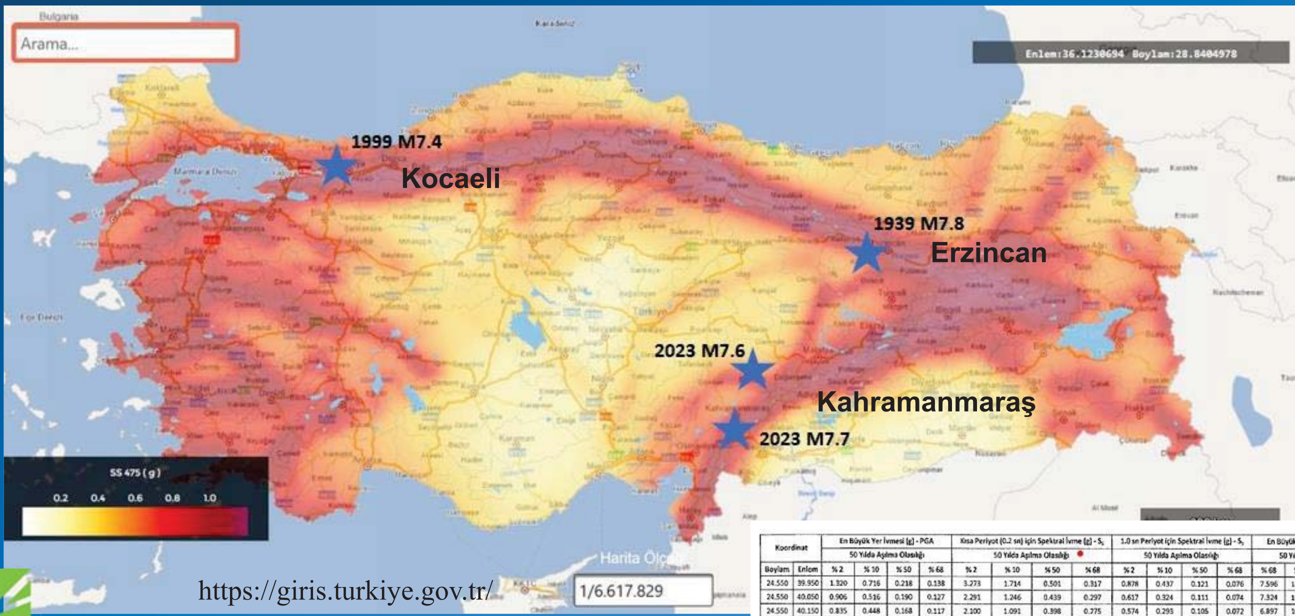
Ebrar Sitesi, Kahramanmaraş 14 blocks (10-stories) built 1997-2013- 1400 people –approx. 1.000 deaths



Former river bed in Kahramanmaraş

Devastating earthquakes in Turkey –in the last 100 yrs

The new Earthquake Hazard Map and Building Earthquake Regulation was published on 18 March 2018



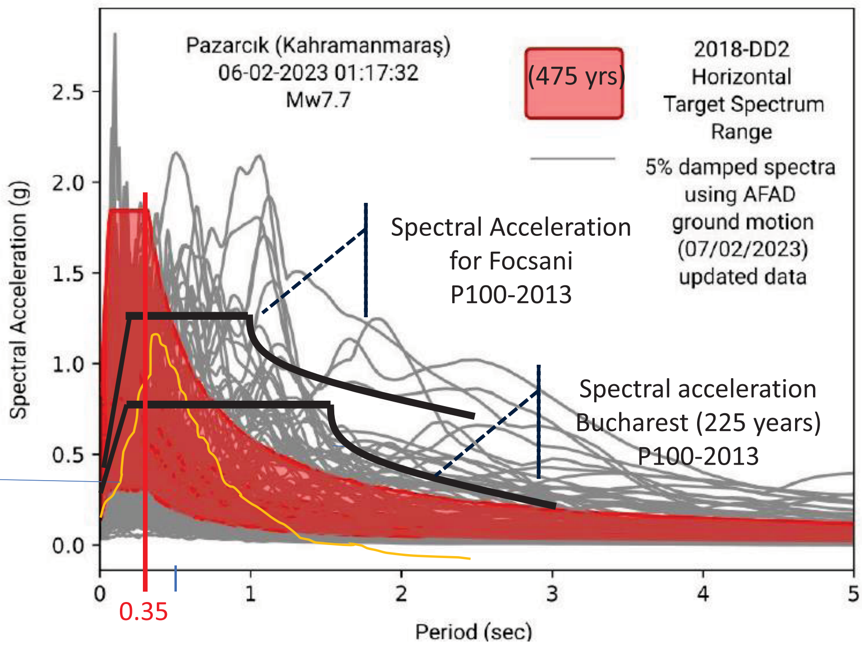
Koordinat		En Büyük Yer İvmesi (g) - PGA				Kısa Periyot (0.2 sn) için Spektral İvme (g) - S_S				1.0 sn Periyot için Spektral İvme (g) - S_1				En Büyük Yer Hızı (cm/sn) - PGV			
		50 Yılda Aşılma Olasılığı				50 Yılda Aşılma Olasılığı				50 Yılda Aşılma Olasılığı				50 Yılda Aşılma Olasılığı			
Boylam	Enlem	% 2	% 10	% 50	% 68	% 2	% 10	% 50	% 68	% 2	% 10	% 50	% 68	% 68	% 50	% 10	% 2
24.550	39.950	1.320	0.716	0.218	0.138	3.273	1.714	0.501	0.317	0.878	0.437	0.121	0.076	7.596	12.321	44.717	83.452
24.550	40.050	0.906	0.516	0.190	0.127	2.291	1.246	0.439	0.297	0.617	0.324	0.111	0.074	7.324	11.010	31.600	57.991
24.550	40.150	0.835	0.448	0.168	0.117	2.100	1.091	0.398	0.275	0.574	0.293	0.105	0.072	6.897	10.118	28.146	53.509
24.550	40.250	0.736	0.397	0.155	0.110	1.851	0.961	0.363	0.255	0.502	0.261	0.098	0.068	6.505	9.299	24.540	46.518
24.550	40.350	0.741	0.386	0.147	0.103	1.845	0.931	0.342	0.236	0.476	0.244	0.092	0.064	6.099	8.724	23.049	44.843
24.650	39.750	0.654	0.386	0.160	0.114	1.637	0.924	0.373	0.265	0.471	0.257	0.099	0.069	6.707	9.559	23.999	42.787

https://giris.turkiye.gov.tr/

https://vividmaps.com/seismic-risk-map-turkey/
www.jmo.org.tr

Seismic records

1999 Athens Eq.
M5.9
(4% loss of GDP)



Kurtulus Atasever & Oguz Cem Celik (Istanbul Technical University)
Anastasios Sextos (Bristol University), Arion Cristian

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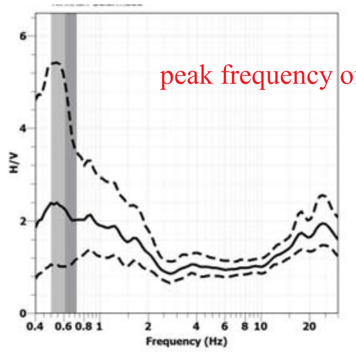
DETERMINATION OF SOIL PARAMETERS OF TURKISH NATIONAL STRONG MOTION STATIONS PROJECT - GEOPHYSICAL SURVEY REPORT
(MASW, REMI, MICROTREMOR)

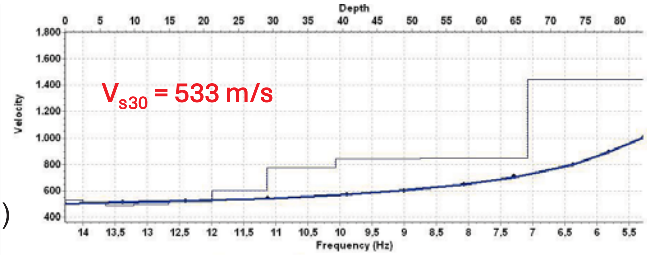
Lithology: Alluvial Fan
Age: Quaternary

a) MASW - multichannel analysis of surface waves method
b) REMI - Refraction Microtremor (seismic surface wave method)
c) AMBIENT NOISE MEASUREMENT

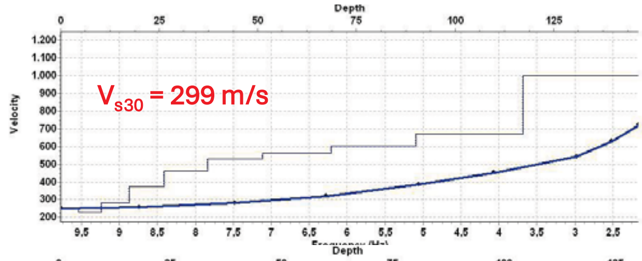


The site dominant frequency was identified from the average horizontal-to-vertical (H/V) spectral ratios of the 5%-damped

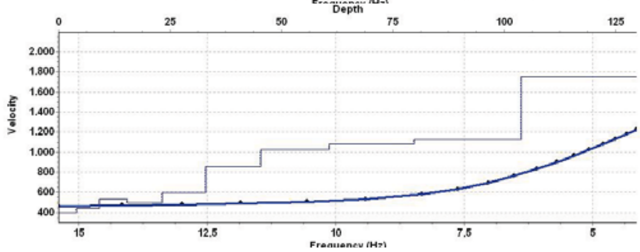
MASW



REMI



MASW +REMI



DETERMINATION OF SOIL PARAMETERS OF TURKISH NATIONAL STRONG MOTION STATIONS PROJECT - GEOPHYSICAL SURVEY REPORT
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SITE CLASSIFICATION OF THE SEIEMIC STATION

Subsoil classification based on Vs profiles (Vs30), obtained from MASW survey, according to NEHRP and EC8.

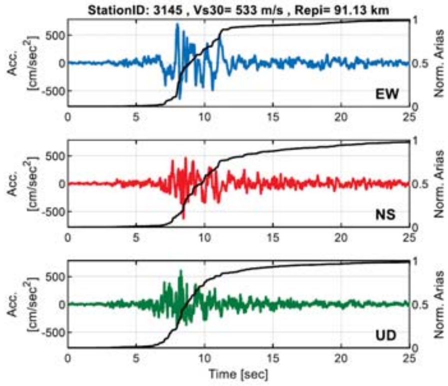
STATION CODE	Vs30 (m/s) (MASW)	SITE CLASS	
3145	533	NEHRP (MASW)	Eurocode-8 (MASW)
CLASS		C	B
DESCRIPTION		Very dense soil and soft rock	Deposits of very dense sand, gravel or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth

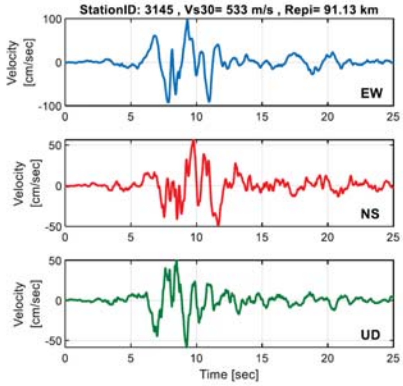
Spectral classification based on predominant period, obtained from microtremor measurement, according to Rodriguez-Marek et.al., 2001 and Di Alessandro et al. 2012 based on microtremor study

STATION CODE	SPECTRAL CLASSIFICATION	SITE DESCRIPTION	PREDOMINANT PERIOD (Tp)	PREDOMINANT FREQUENCY	EXPLANATIONS
3145	(Rodrigez-Marek et al. 2001) C-2	Shallow Stiff Soil	0.47	2.1	Soil depth > 6 m and < 30 m
	(Di Alessandro et al. 2012) CL-V	Tg not idenfiable (flat H/V and amplitude <2), Generic rock			

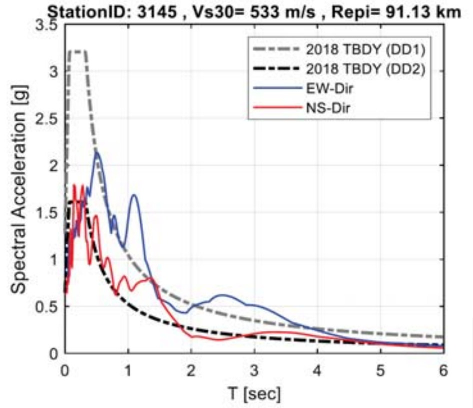
Subsoil classification of 3145 station based on Vs profiles (Vs30), obtained from MASW survey, according to Turkish Building Earthquake Code (TBEC, 2018).

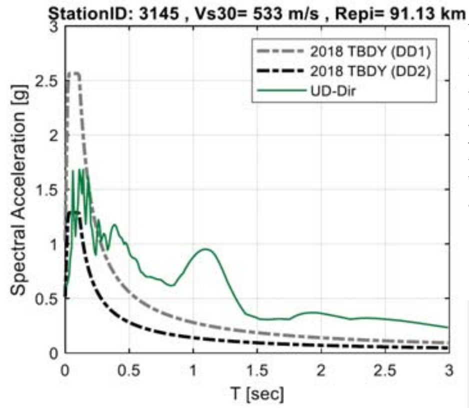
STATION CODE	SITE CLASS	SITE DESCRIPTION	Vs30 (m/s) (MASW)
3145	ZC	Very stiff sand, gravel and hard clay layers or latered, very cracked weak rocks	533





The building code-based spectra, are provided for average return periods of 2,475 years (DD-1), 475 years (DD-2), 72 years (DD-3), and 43 years (DD-4).
Residential buildings are generally designed considering the 475-year return period spectrum. (2018 earthquake-resistant design code of Türkiye (TBDY, 2018)





Boğaziçi University
Kandilli Observatory and Earthquake Research Institute
Department of Earthquake Engineering
Strong Ground Motion and Building Damage Estimations
Preliminary Report (v6) - 2023

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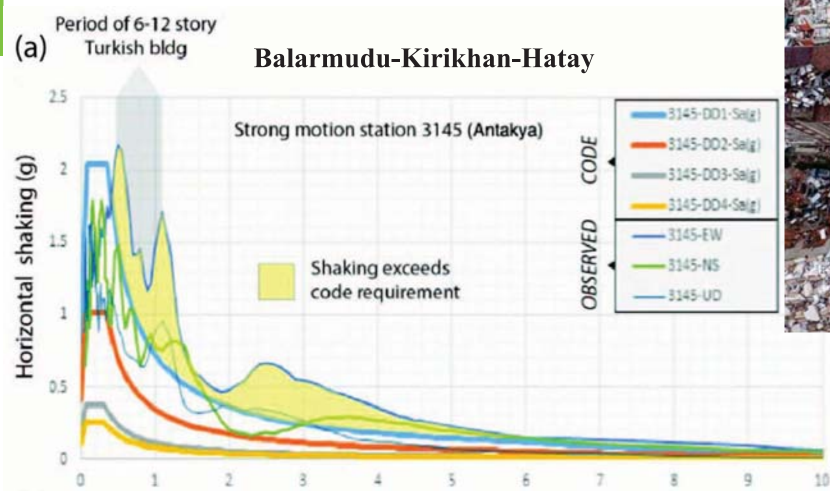
https://temblor.net/temblor/preliminary-report-2023-turkey-earthquakes-15027/



Comparison of the 5%-damped acceleration response spectra (thin curves) with TBDY (2018) code-based spectra (bold curves) at the station of (a) 3145.
Credit: Erdik, Tümsa, Pınar, Altunel, and Zülfikar

The building code-based spectra, are provided for average return periods of 2,475 years (DD-1), 475 years (DD-2), 72 years (DD-3), and 43 years (DD-4).
Residential buildings are generally designed considering the 475-year return period spectrum. (2018 earthquake-resistant design code of Türkiye (TBDY, 2018)

Problems related with design codes

- Advanced Design codes must be implemented taking into consideration the technical, financial capabilities of the local administrations.
- Low values in the Seismic Hazard Map for PGA in areas with evidence of large historical events
- Accepting local site characterization might lead to worst DAMAGE case - high rise buildings are designed/constructed with low levels of seismic design
- Low corner periods of design spectra associated to quaternary soil deposits (EC8, ASCE …
- US regulations are established based comprehensive site investigations

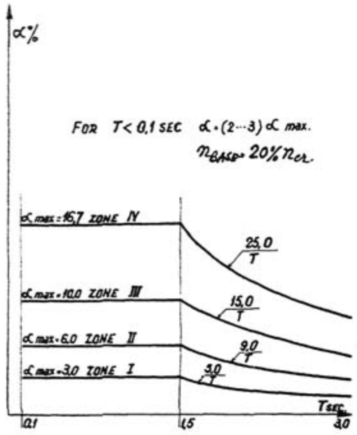


FIG. 1e VERY SOFT GROUND

ROMANIA before 1977

- Proposal made by Cișmigiu and Țițaru in a paper sent in 1960 to the 2WCEE; the proposal was **not included in P13-63 (first Romanian seismic code)**
- They consider the increase of the Tc corner period for soft soil
- The proposed Tc value for the Romanian Plain, with deep Quaternary deposits, is 1.5 s, a value close to that observed on March 4, 1977 in Bucharest and **included in P100/78**

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European Problems related with…

- Research capabilities of seismic European countries are limited
- Continuous budgetary allocation (example: CNRRS)
- Low interest in Field or laboratory investigation in some countries/cases
- Reduced number of researchers/laboratory technicians
- Limited involvement of large construction companies in the research
- Norway example of zoning the quick clay (after more than 40 years of investigation financed by NVE) shows the limitation of microzoning
- Design codes accepting local site characterization based on low reliability investigations methods might lead to extensive (UNEXPECTED!?) damages
- Low corner periods of design spectra sometimes associated to quaternary soil deposits

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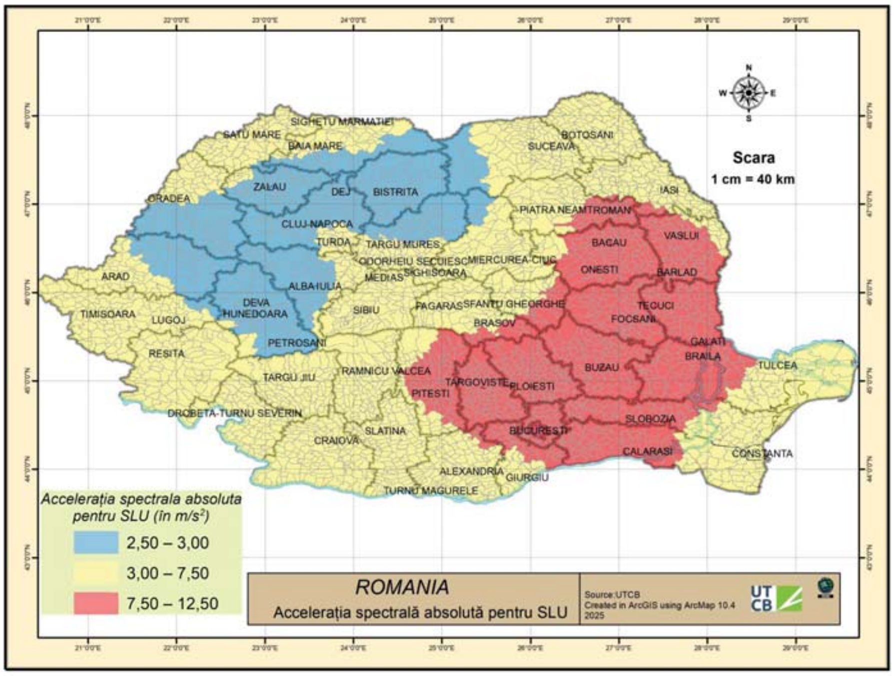
Final proposal on seismic zonation of Romania

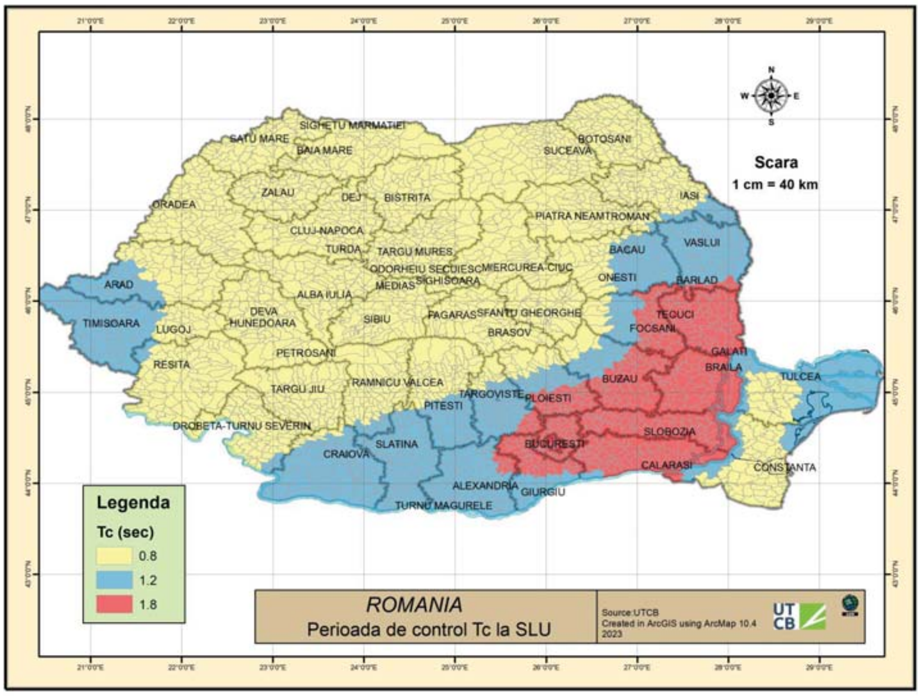
2025



Nr. crt.	Județ	Localitate	$S_{ap,h}^{SLS}$	$S_{ap,h}^{SLU}$	T_C^{SLS}	T_C^{SLU}	Seismicitate
1	ALBA	ABRUD	1.25	2.5	0,6	0,8	Mică
2	ALBA	AIUD	1.25	2.87	0,6	0,8	Mică
3	ALBA	ALBA IULIA	1.25	2.5	0,6	0,8	Mică
4	ALBA	ALBAC	1.25	2.5	0,6	0,8	Mică

The values of the horizontal acceleration spectra for ultimate limits states for all administrative-territorial unit (UAT) of Romania are provided (3186 UAT’s)

Revision of P 100-1
Seismic Design Code of Romania
(To be released in 2025)



The values of the control periods of the spectra for ultimate limits states for all administrative-territorial unit (UAT) of Romania are provided (3186 UAT's)

0,8s; 1,2s and 1,8s

Revision of P 100-1
Seismic Design Code of Romania
(To be released in 2025)

Nr. crt.	Judet	Localitate	$S_{ap,h}^{SLS}$	$S_{ap,h}^{SLU}$	T_C^{SLS}	T_C^{SLU}	Seismicitate
1	ALBA	ABRUD	1.25	2.5	0,6	0,8	Mică
2	ALBA	AIUD	1.25	2.87	0,6	0,8	Mică
3	ALBA	ALBA IULIA	1.25	2.5	0,6	0,8	Mică
4	ALBA	ALBAC	1.25	2.5	0,6	0,8	Mică

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Universitatea Tehnică de Construcții București

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NEVER give up!

2004 Niigata –Showa Bridge

91

We kindly acknowledge the support of Building Research Institute (BRI)-Japan , Tokyo Soil Research (TSR) and Oyo Corporation, as well as the generous funding provided by Japan International Cooperation Agency (JICA).
The authors acknowledge the involvement of our former colleagues from NCSRR: Roxana Oprea, Aurora Bucataru, Caterina Negulescu, Raluca Radoi, Natalia Poiata.

We acknowledge the cooperation of Loretta Batali from UTCB, and of the companies: Arup, Fugro, Saint-Gobain, Popp & Asociatii, Geosond,.

The TEAM:
Cristi Neagu, Elena Calarasu, Alexandru Aldea, Radu Vacareanu, Florin Pavel, Bogdan Gheorghe, Cristian Arion

Thank You!

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UTCB site response based on 20 years of observation

Alexandru Aldea (UTCB), Florin Pavel (UTCB),
Etienne Bertrand (Université Gustave Eiffel)

Earthquake & Geotechnical Engineering
March 27th, 2025 • Romania - Greece Seminar

Japan International Cooperation Agency JICA

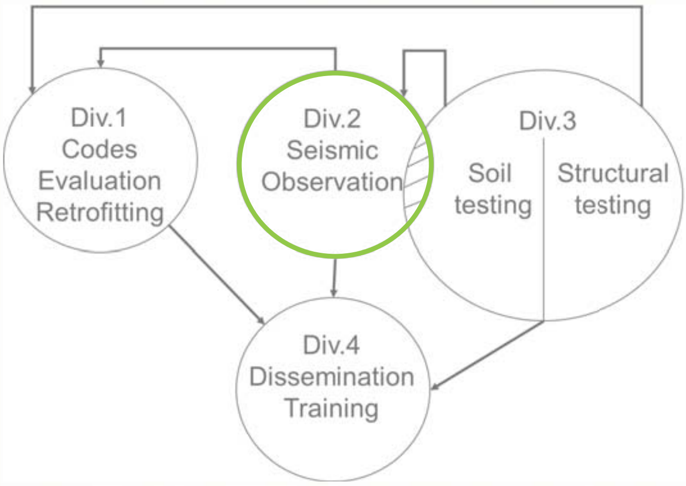
Project Type of Technical Cooperation in the field of Housing and Building

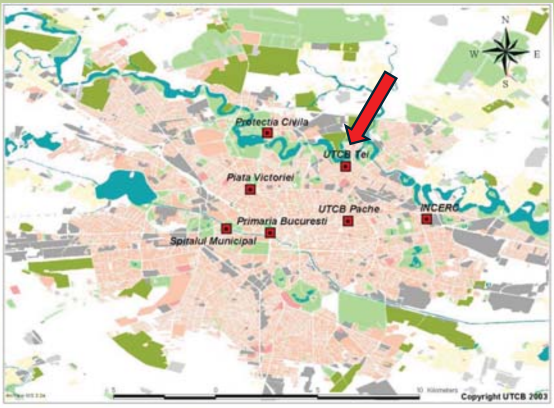
"Reduction of Seismic Risk for Buildings and Structures"

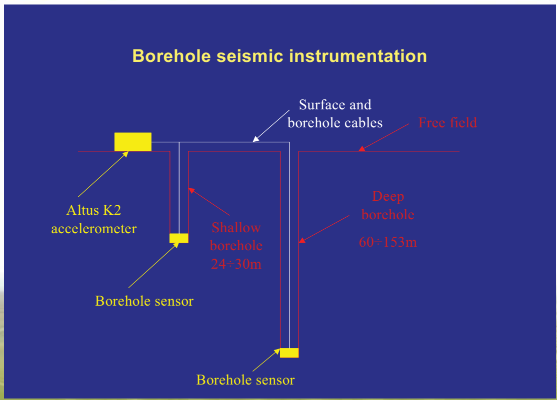
2002-2008

implementing agency

CNRRS
National Center for Seismic Risk Reduction







Seismic stations for site effect assessment

No.	Site	Station ID	Surface sensor location	Depth of sensor in shallow borehole, m	Depth of sensor in deep borehole, m
1	UTCB Tei	UTC1	free field	-28	-78
2	UTCB Pache	UTC2	1 storey bldg.	-28	-66
3	NCSRR/INCERC	INC	1 storey bldg.	-24	-153
4	Civil Protection	PRC	1 storey bldg.	-28	-68
5	Piata Victoriei	VIC	free field	-28	-151
6	City Hall	PRI	free field	-28	-52
7	Municipal Hospital	SMU	free field	-30	-70

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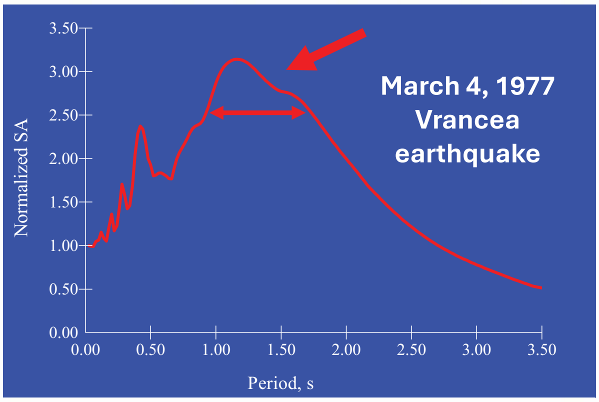
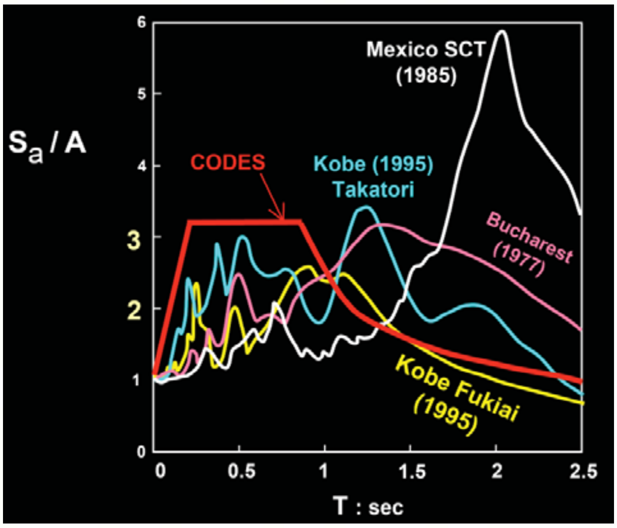
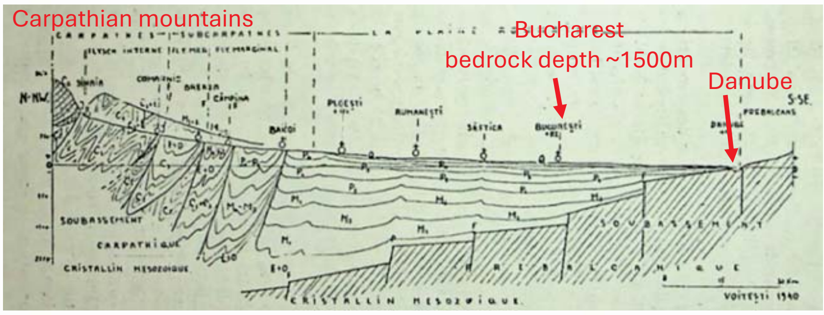




Gazetas, 2006,1st ECEES

"It is indeed fortunate that at least one reliable observation of the ground motion was made in Bucharest. It appears to be a very interesting one which may modify the concepts of standard response spectra." [EERI, 1977].



Romanian Plain geologic profile (Voitesti, 1941)

Map of the separation limit between quaternary and tertiary deposits (Liteanu, 1961)

Quaternary layer in Bucharest – between 200m-300m

SOCIETATEA ROMÂNĂ DE GEOTEHNICĂ ȘI FUNDAȚII
UTCB Universitatea Tehnică de Construcții București
ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ
Earthquake & Geotechnical Engineering
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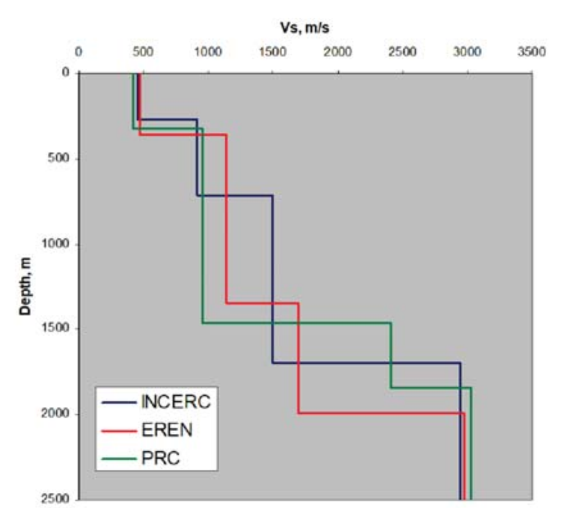
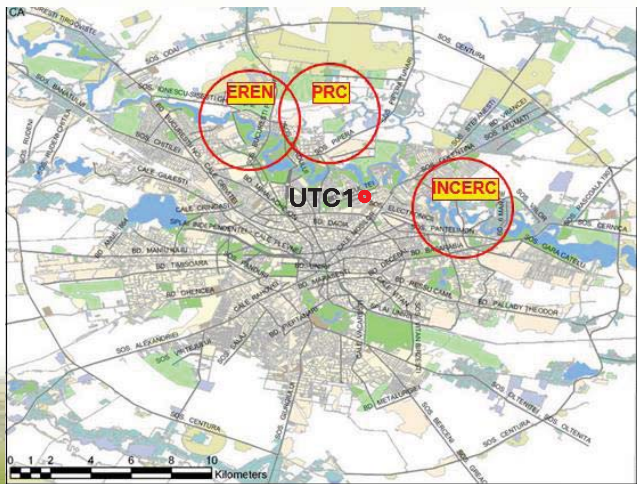
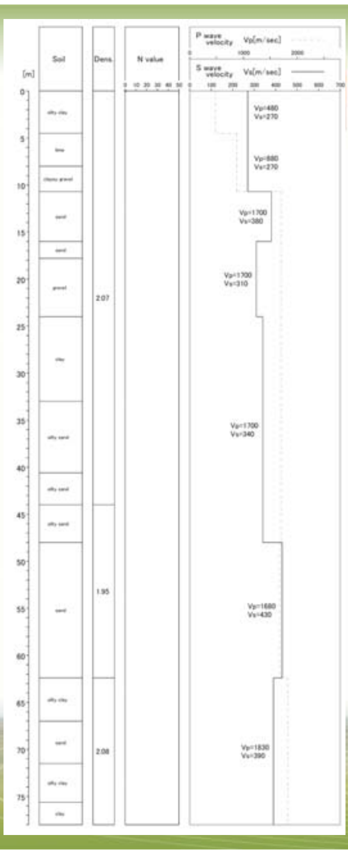


Aldea, Yamanaka and Takahashi, 2006, 1st ECEES

Soil thickness to the seismic bedrock from array microtremor measurements:

- 1696 m at INCERC
- 1993 m at EREN
- 1884 m at PRC





2003 – Down-hole tests
NCSRR & Tokyo Soil Co., Ltd.

Station	PRI	UTC2	PRC	SMU	UTC1	INC	VIC
Average (30m) Vs, m/s	219	288	293	245	**309**	270	284
Average (52m) Vs, m/s	258	318	309	281	**326**	302	310
Average Vs, m/s (whole borehole depth, m)	258 (52m)	332 (66m)	324 (68m)	303 (69m)	**349 (78m)**	364 (140m)	354 (110m)

Eurocode 8 class C
"Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of m"

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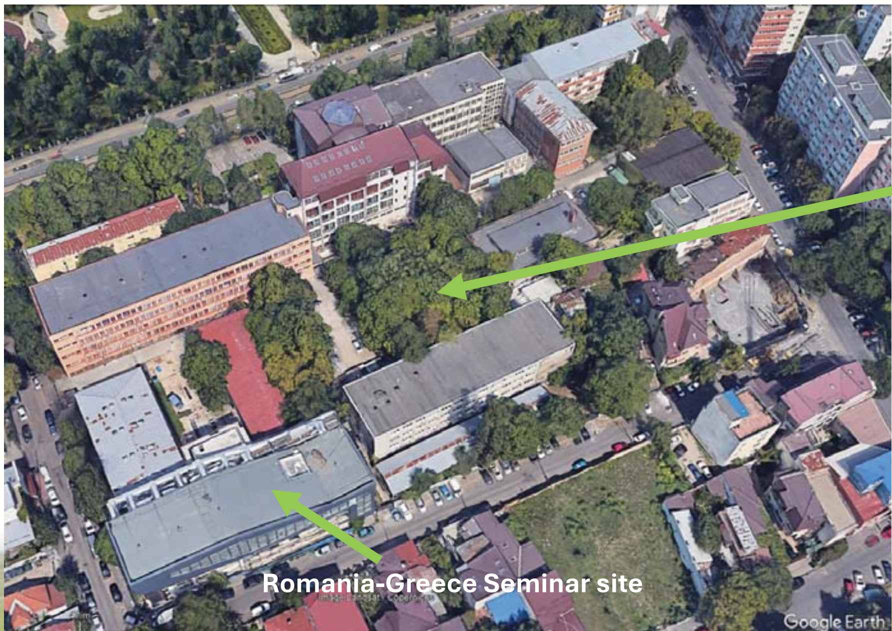

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July 16th, 2003
Kobayashi & Kanehira (OYO),
Aldea, Vacareanu, Radoi, Negulescu,
Poiata (NCSRR)

Kinemetrics K2 station with
Episensor & FBA23-DH

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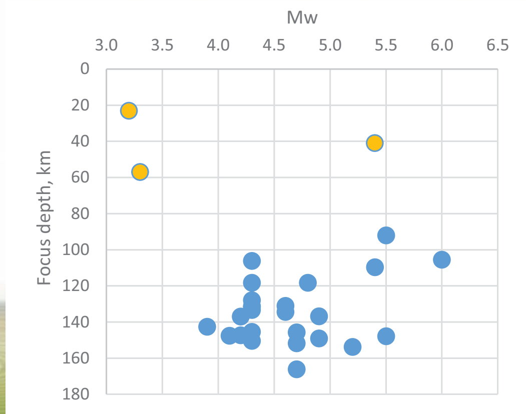
No	Date	Seismic source	Depth km	Magnitude Mw
1	2003.10.05	Vrancea intermediate	145.6	4.7
2	2004.07.10		150.4	4.3
3	2004.09.27		166.1	4.7
4	2004.10.27		105.4	6.0
5	2005.06.18		153.7	5.2
6	2005.12.13		136.8	4.9
7	2005.12.18	Vrancea surface	57.0	3.3
8	2006.03.06	Vrancea intermediate	151.7	4.7
9	2007.01.17		131.6	4.3
10	2009.04.25		109.6	5.4
11	2014.03.29		134.4	4.6
12	2014.04.03		127.9	4.3
13	2014.08.24		147.3	4.2
14	2014.09.10		106.1	4.3
15	2014.11.22	Vrancea surface	40.9	5.4
16	2015.03.16	Vrancea intermediate	118.2	4.3
17	2015.03.29		145.4	4.3
18	2016.09.23		92.0	5.5
19	2017.08.02		131.0	4.6
20	2018.03.14		136.9	4.2
21	2018.04.25		147.6	4.1
22	2018.10.28		147.8	5.5
23	2020.01.31		118.2	4.8
24	2021.05.25		131.2	4.3
25	2022.05.11		142.6	3.9
26	2022.09.18	Făgăraş surface	23.0	3.2
27	2022.11.03	Vrancea intermediate	149.0	4.9
28	2023.12.03		133.3	4.3

+ 5 records with no date (GPS malfunction)

Event recording (triggering threshold)

UTC1 station management
2003-2010 NCSRR
2010-2014 INCERC
2014 - UTCB



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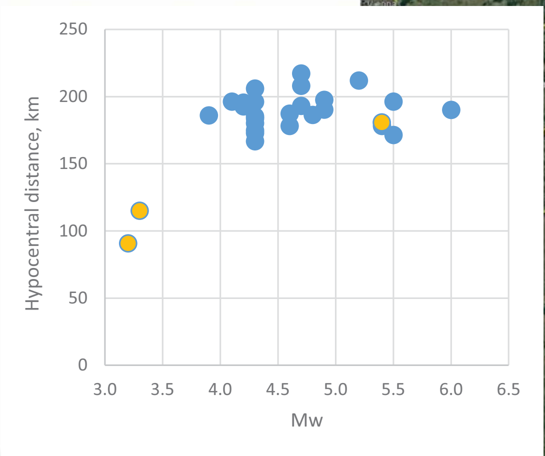

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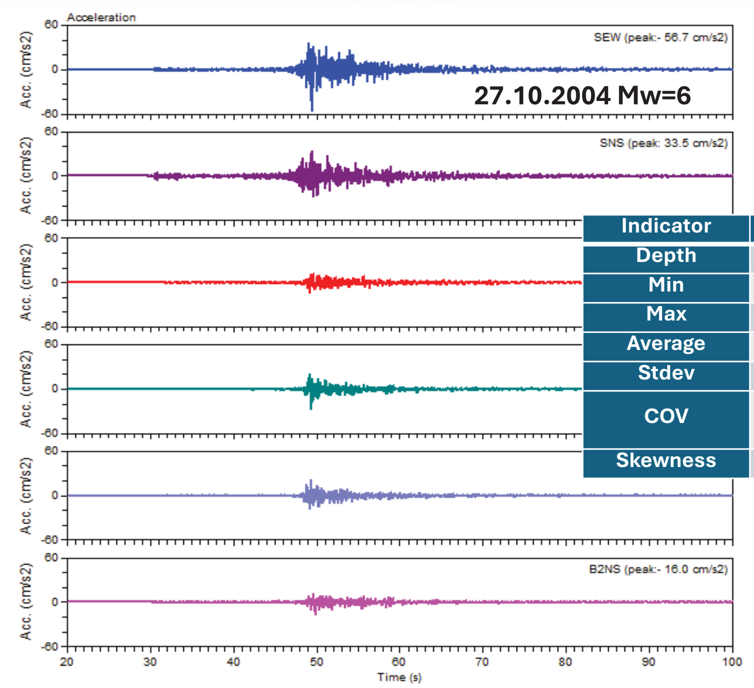

Peak Ground Accelerations PGA

Indicator	PGA horizontal (cm/s^2)			PGA vertical (cm/s^2)		
Depth	0m	-28m	-78m	0m	-28m	-78m
Min	0.7	0.3	0.2	0.4	0.2	0.1
Max	58.4	28.4	23.1	34.1	11.4	9.8
Average	7.04	2.59	2.65	4.46	1.68	1.41
Stdev	10.76	4.38	4.37	6.57	2.42	2.12
COV	152.9%	169.1%	164.8%	147.3%	143.6%	150.2%
Skewness	3.37	4.23	3.38	3.63	3.11	3.26

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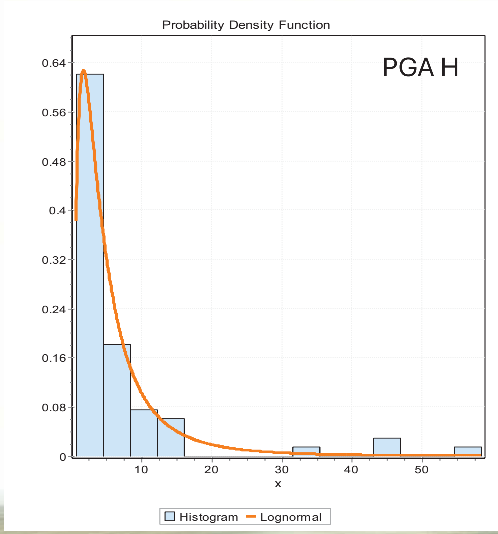
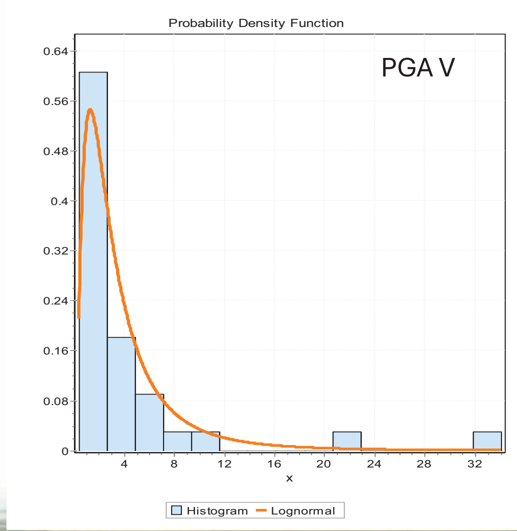
Peak Ground Accelerations PGA





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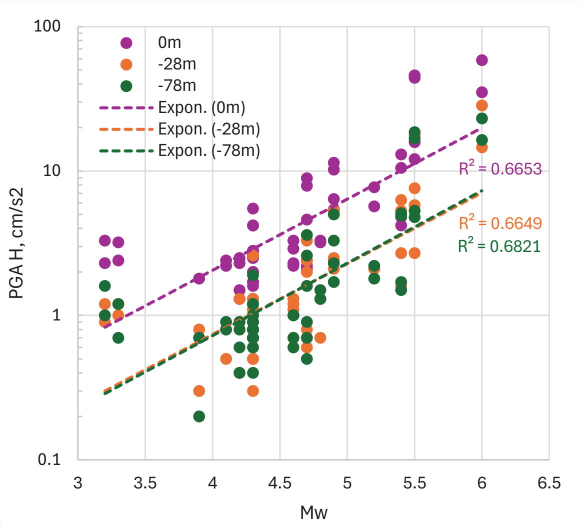
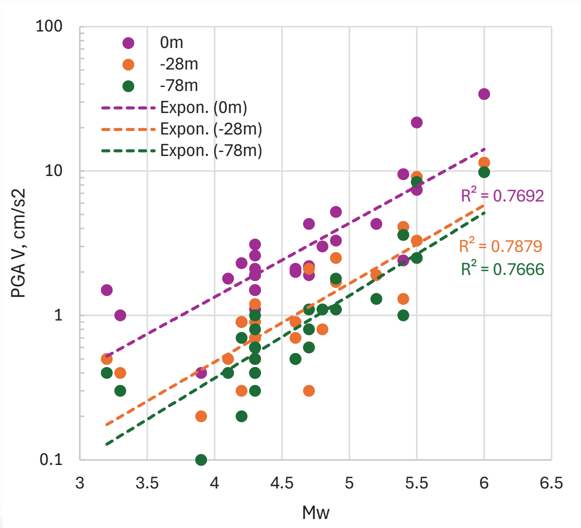
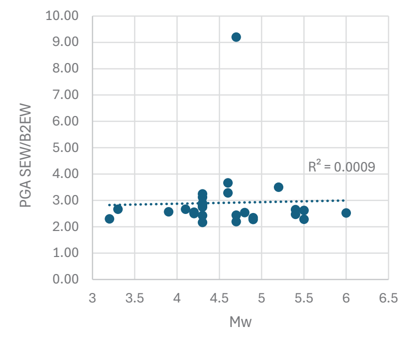


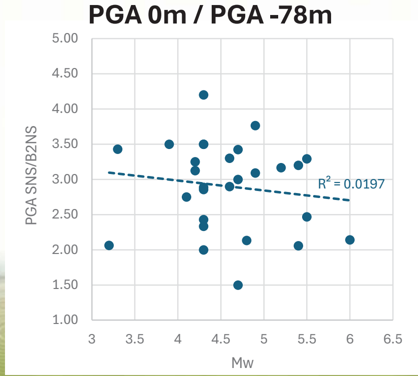


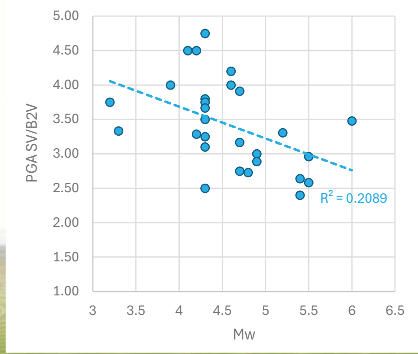
PGA ratios

Direction	EW			NS			Vertical		
Rapport des PGA	0m/ 28m	0m/ 78m	28m/ 78m	0m/ 28m	0m/ 78m	28m/ 78m	0m/ 28m	0m/ 78m	28m/ 78m
Moyenne	3,87	2,93	0,82	2,33	2,96	1,41	3,01	3,43	1,26
Ecart-type	1,34	1,26	0,34	0,70	0,65	0,68	1,30	0,65	0,40
COV	34,7%	43,0%	41,9%	30,1%	21,8%	48,3%	43,2%	18,9%	31,3%

PGA 0m / PGA -78m







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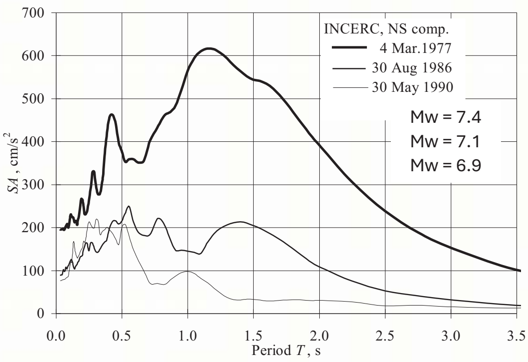
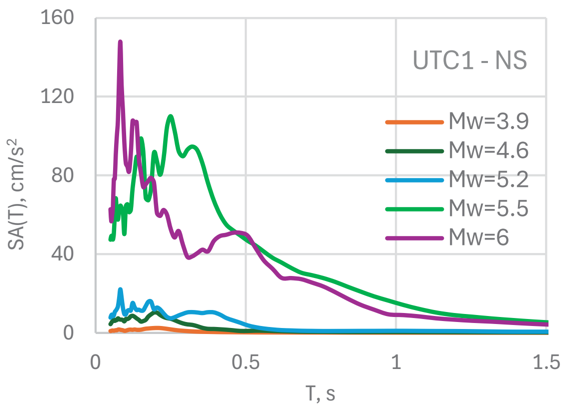
Mobility with magnitude of response spectra



Lungu et al., 2000



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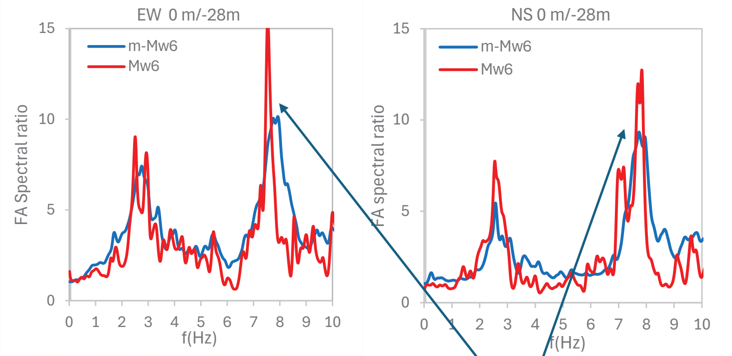
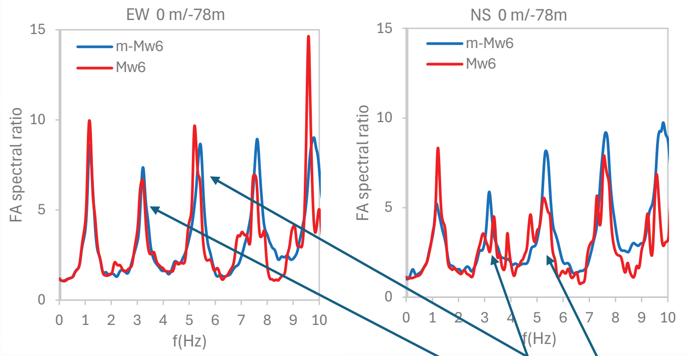
Response of the upper 28m



Direction	EW	NS
FA ratio	0m/28m	0m/28m
f_1 (Hz) from average FA ratios	2.75	2.57
f_1 (Hz) from Mw6 FA ratio	2.50 2.92	2.54
f_1 (Hz) average	2.66	
f_1 (Hz) from average Vs	2.94	

$f_2 \approx 8.0$Hz
$f_2 \approx 8.8$Hz

Response of the upper 78m



Direction	EW	NS
FA ratio	0m/78m	0m/78m
f_1 (Hz) from average FA ratios	1.17	1.16
f_1 (Hz) from Mw6 FA ratio	1.15	1.21
f_1 (Hz) average	1.17	
f_1 (Hz) from average Vs	1.12	

$f_2 \approx 3.5$Hz $f_3 \approx 5.8$Hz
$f_2 \approx 3.6$Hz $f_3 \approx 5.6$Hz

$$T_P = 4\,h\,/\,V_{s,1}$$

$$T_{n+1} = T_P\,/\,(2n+1) \quad n=0,\ 1,\ 2,\ 3,\ \ldots$$

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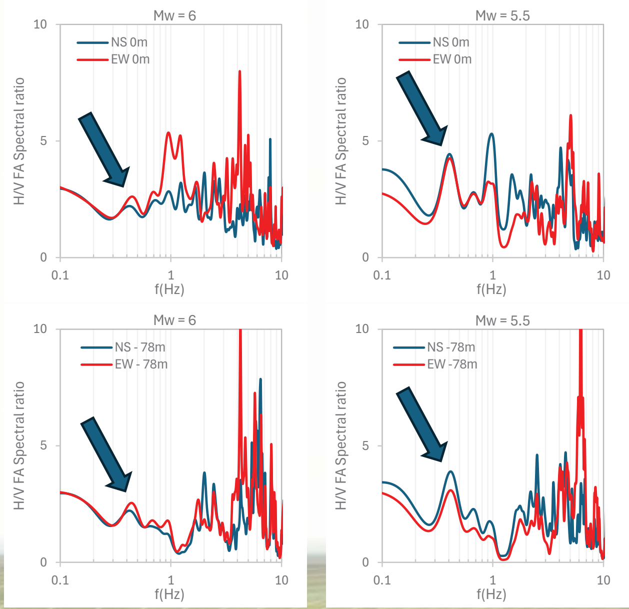
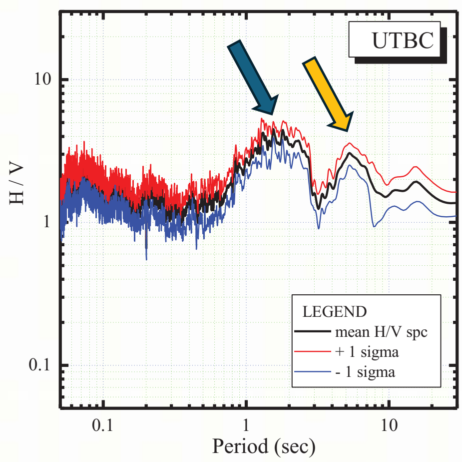


H/V seismic records



H/V ambient vibrations
Aldea et al., 2007

Concluding remarks

Considering the limited amount of data available (number, magnitude range):

- Recorded data showed that the upper 28m of soil are the main contributor for the PGA amplification
- PGA amplification had no correlation with earthquake magnitude
- Surface over borehole spectral ratios captured the vibration characteristics of the soil layers between the recording points (even the higher modes)
- Soil behaviour remained elastic
- H/V spectral ratios from seismic records display a lower frequency peak that can be associated to the deep geology (the peak is visible also at -78m depth)

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Acknowledgements

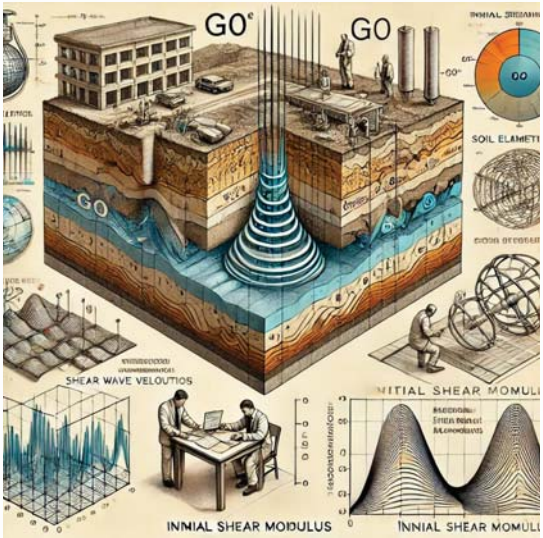
JICA not only for the seismic instrumentation, but also for the long-term support that included working periods of Romanian NCSRR staff in Japan and of Japanese experts in Romania.

The support of staff from **BRI, Tokyo Soil Research Co., Ltd., Tokyo Institute of Technology,** Japan

Japan and **Japanese tax-payers** for their generosity for international help within JICA activities in developing countries.

Assessment of soils stiffness for small strains by in situ tests and correlations for sites in Bucharest

Alexandra Ene



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G_0 = initial shear modulus = small strain shear modulus

- *useful in the analysis of **soil-structure interaction** of structures in case of seismic actions*

Advanced design of civil structures – soil-structure interaction → Analytical approach; Numerical approach

Numerical approach → DEPENDENT ON THE DEFORMABILITY PARAMETERS OF THE FOUNDATION GROUND → G_0

G_0 → In situ tests; Laboratory tests

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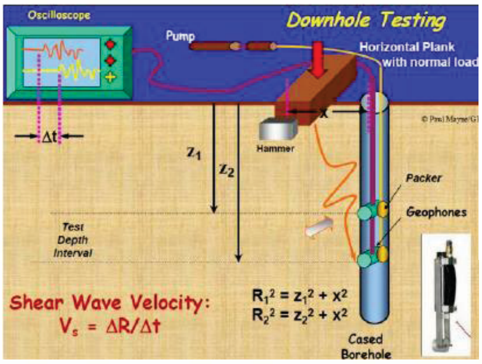

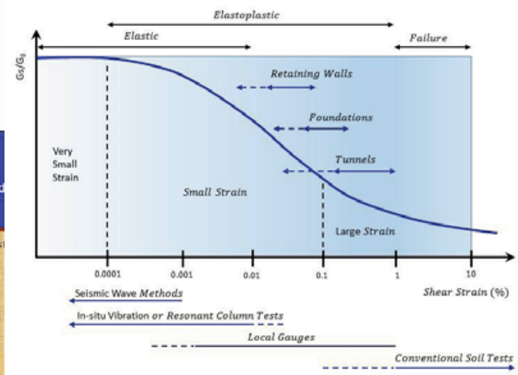

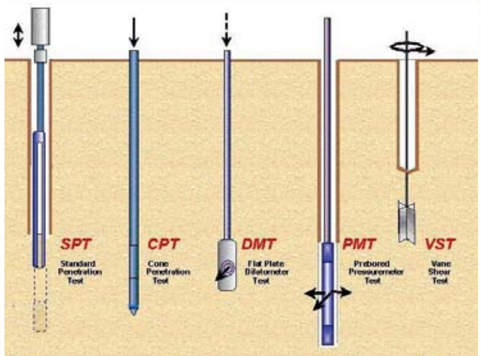

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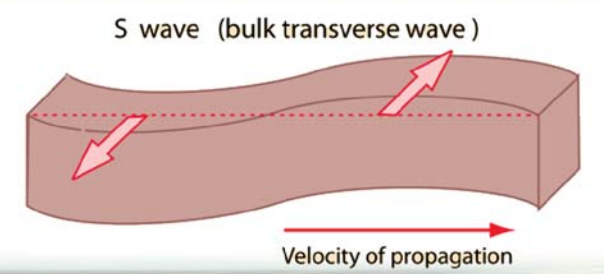
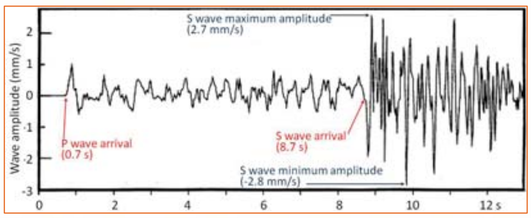
IN SITU TESTS – preferred method

- *disturbance of the soil samples is minimal*
- *better coverage of the investigated soil volume*
- ***indirect**, based on theoretical and empirical correlations*

- *The most common tests carried out in Romania: **Downhole, sCPT** & **sDMT***

Generating shear waves (in depth or at the ground surface) → **Measuring the velocity of S-waves (using sensors)**





Assessment of soils stiffness for small strains by in situ tests and correlations for sites in Bucharest

Soil stratification specific to Bucharest area:

1. *Old and new fillings in surface - various sources and periods;*
2. *Upper sandy clayey complex, "Clays of Bucharest" or "Bucharest Loam" comprised of **silty-clayey soil deposits and pockets of clayey sands** (stiff);*
3. *Upper sandy complex "Colentina Gravels" comprised of **sands and small gravels** (medium dense);*
4. *Intermediate lacustrine complex consisting in general of **clays or silty-clays with bounding surfaces** (stiff);*
5. *Intermediate sandy complex, "Mostiștea Sands" consisting of **medium and fine sands**, sometimes **with clayey or sandy inserts** (medium dense);*
6. *Inferior lacustrine complex, consisting of **fine clays and sands**;*
7. *Frăteşti layers, the oldest quaternary formations in the area, at relatively high depth (approximately 100-180 m) consisting of **sands and gravels with clayey inserts** (medium dense to dense).*

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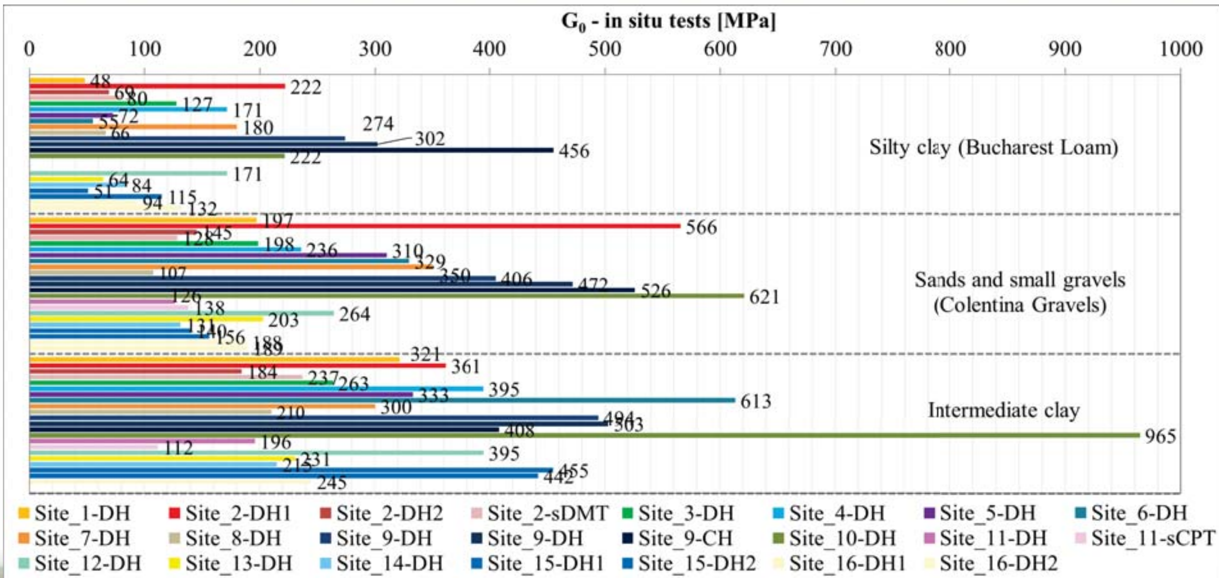

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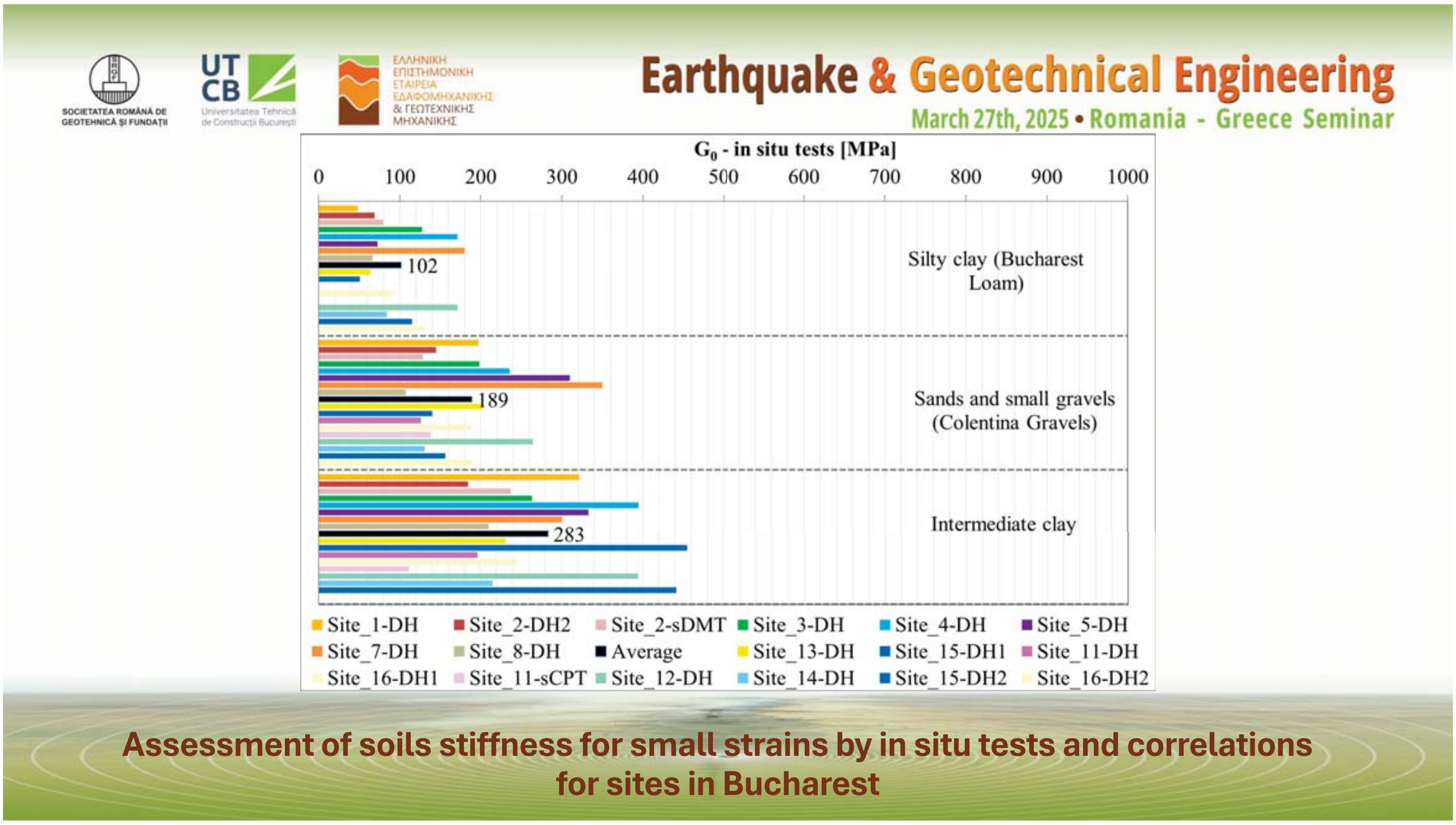
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G0 - in situ tests [MPa]
0 100 200 300 400 500 600 700 800 900 1000
102
Silty clay (Bucharest Loam)
189
Sands and small gravels (Colentina Gravels)
283
Intermediate clay
Site_1-DH Site_2-DH2 Site_2-sDMT Site_3-DH Site_4-DH Site_5-DH
Site_7-DH Site_8-DH Average Site_13-DH Site_15-DH1 Site_11-DH
Site_16-DH1 Site_11-sCPT Site_12-DH Site_14-DH Site_15-DH2 Site_16-DH2
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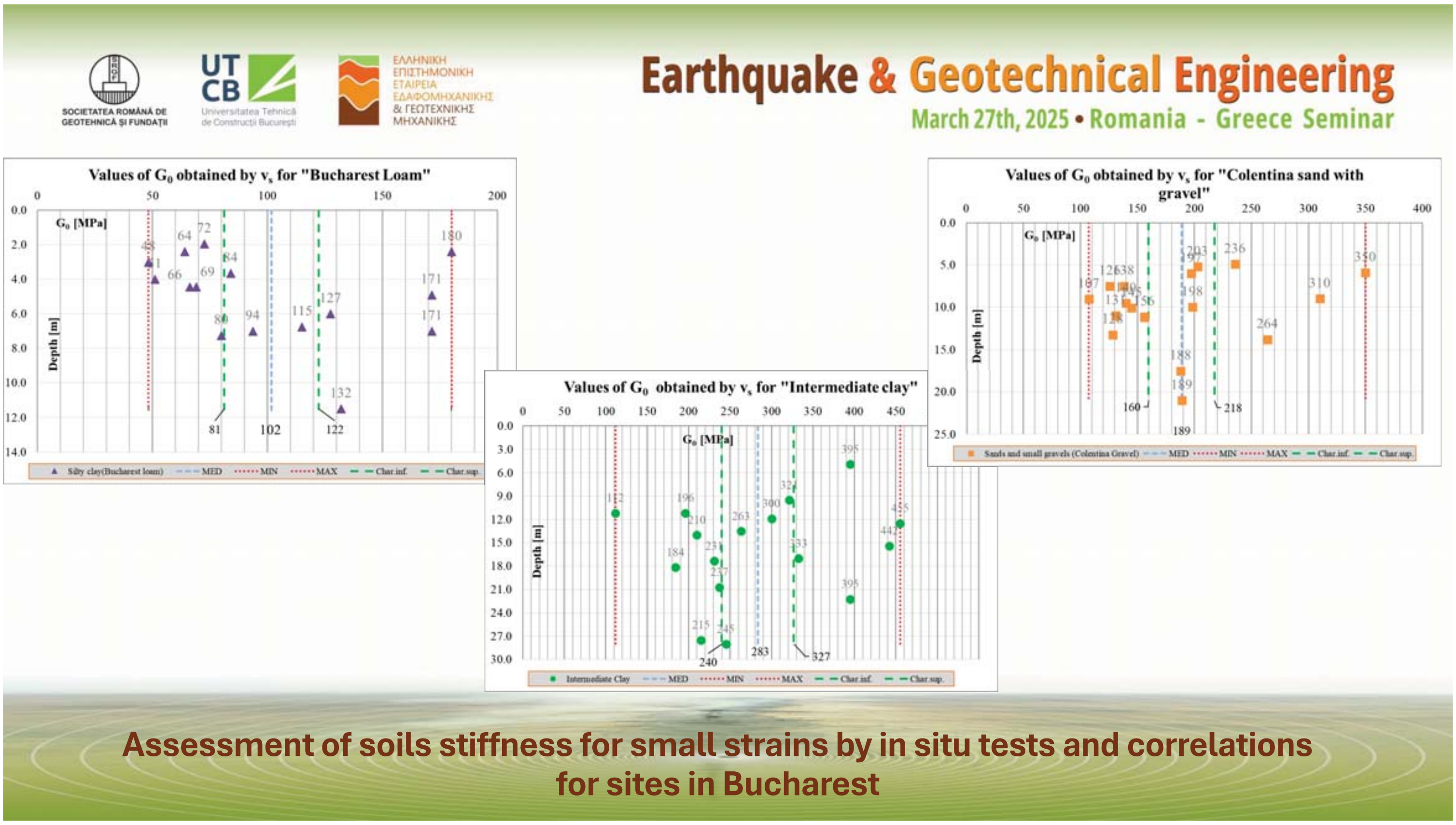
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Values of G0 obtained by vs for "Bucharest Loam"
G0 [MPa]
Depth [m]
81
102
122
Silty clay(Bucharest loam) MED MIN MAX Char.inf. Char.sup.
Values of G0 obtained by vs for "Intermediate clay"
G0 [MPa]
Depth [m]
240
283
327
Intermediate Clay MED MIN MAX Char.inf. Char.sup.
Values of G0 obtained by vs for "Colentina sand with gravel"
G0 [MPa]
Depth [m]
160
189
218
Sands and small gravels (Colentina Gravel) MED MIN MAX Char.inf. Char.sup.
Assessment of soils stiffness for small strains by in situ tests and correlations for sites in Bucharest

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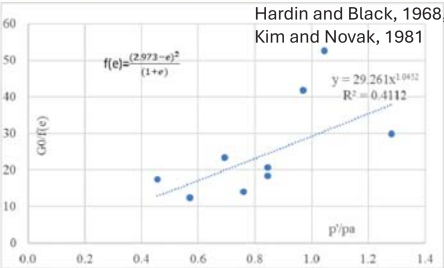
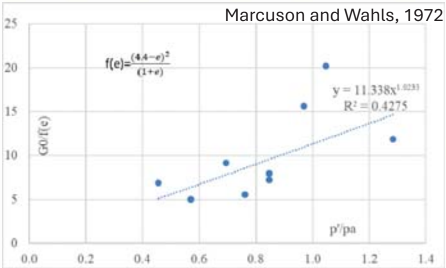
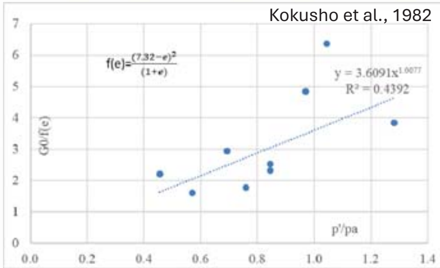
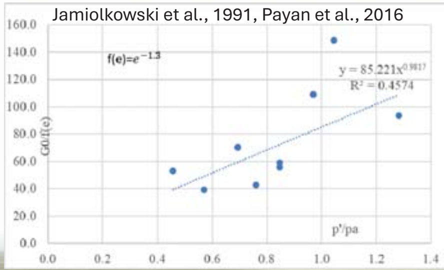
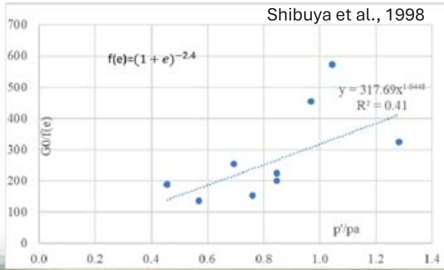






No proper correlation with the void ratio





Assessment of soils stiffness for small strains by in situ tests and correlations for sites in Bucharest

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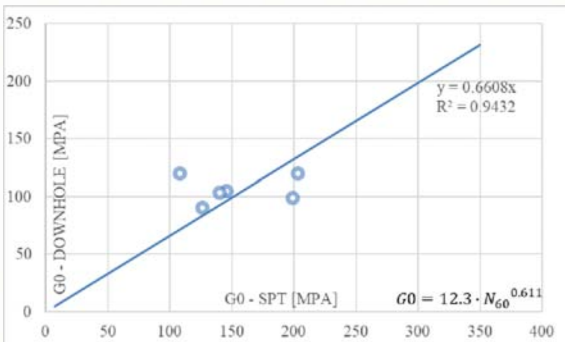

Good correlation with SPT

Crespellani and Vannucchi, 1991
(r=0,671)

Assessment of soils stiffness for small strains by in situ tests and correlations for sites in Bucharest

Conclusions

- *Variation of the soil initial modulus even in relatively small area (all within the margins of Bucharest city) is very important, thus precaution is needed in estimating this parameter or comparing with experience.*
- *Further research on the correlations between the initial soil stiffness and simple soil parameters is needed*
- *Besides from the differences in soil layers thickness and properties, it can also be intuited that also the variation of the testing procedures and equipment might lead to such discrepancies .*

Local site conditions in hybrid strong ground motion simulation

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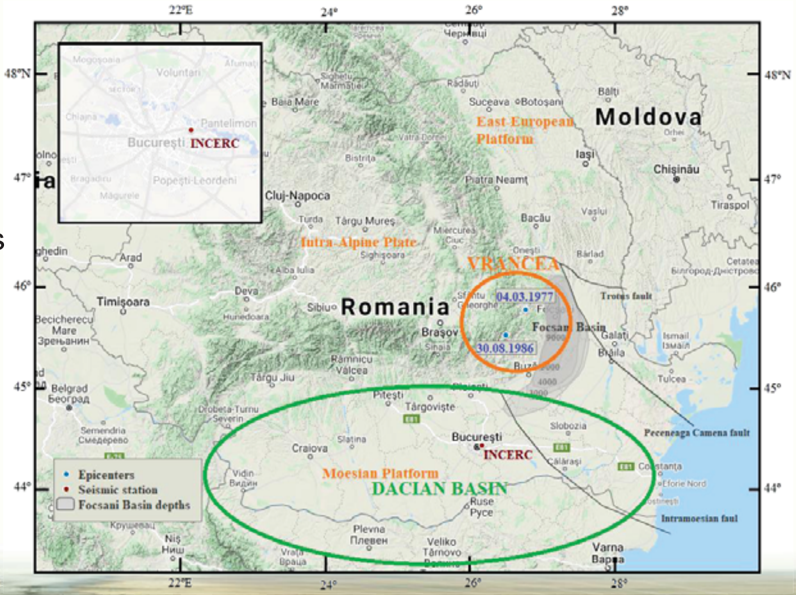
Content

- Introduction
- Simulation method
- Local site conditions – general considerations
- Local site conditions – issues
- Conclusions



The map with the locations of the INCERC seismic station and the March 4th 1977, August 30th 1986 and scenarios earthquakes epicenters; the outlined area (after Borleanu et al. 2011) is an approximation of the Focsani Basin area, and the contours underline the depth of the sedimentary layers in meters. The map in the upper corner emphasizes the location of the INCERC station in Bucharest

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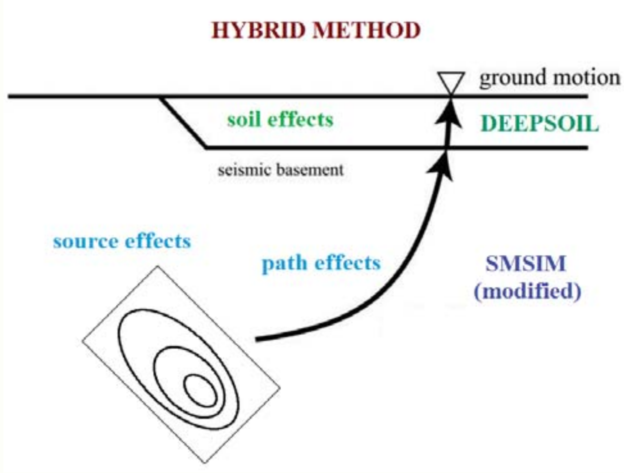
Introduction

Time domain ground motion simulation

- **The empirical method of Green functions**
- **Stochastic simulation of ground motion**
- **Numerical (physical based) modeling**
- **Hybrid methods**



Proposed hybrid method

The use of a certain method is dependent on the applicability to the seismogenic zone and the level of knowledge concerning the processes that generate and modificate of the seismic waves.

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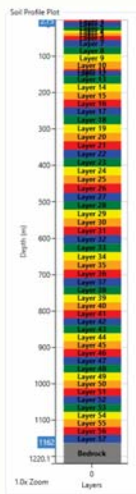
Simulation method - Hybrid method

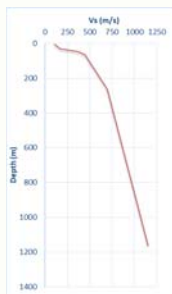
Spectrum of ground motion (**SMSIM** - Boore, 2003)
$Y(M_0,R,F) = E(M_0,f)P(R,f)G(f)I(f)$

Shaping window (Cotovanu and Vacareanu, 2020b)

Souce equation $E(M_0) = CM_0S(M_0,f)$	Path equation $P(R,f) = Z(R)\exp[-\pi fR/Q(f)cQ]$	Local site equation $G(f) = A(f)D(f)$
Finite source vs point source	Geometrical spreding and attenuation	Crustal amplification and site diminution
Single corner vs double corner source spectrum	**Path duration** (Cotovanu and Vacareanu, 2021)	Linear effects vs nonlinear effects, liquefaction, topography
Share-wave velocity and density in source vicinity, radiation path, corner frequencies, stress, magnitude, source dimensions, strike, dip, rake, source duration	Depth, reference distance, distance-dependent geometrical spreding function, frequency-dependent attenuation funciton,	Soil profile, seismic impedance, share-wave velocity, density for each type of soil, frequency-dependent diminution function, k diminution function



DEEPSOIL
(Hashash, et al., 2016)



Velocity of local site profile of INCERC (Bucharest) station after Constantinescu and Enescu (1985)

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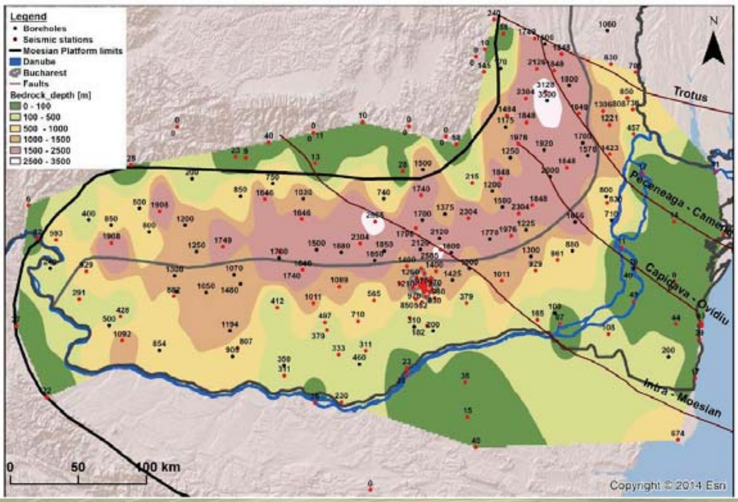
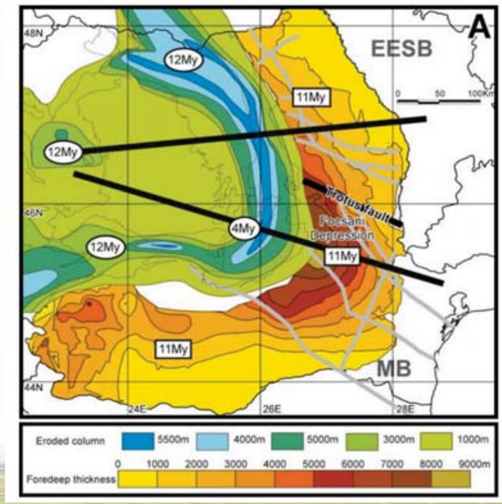
Local site conditions – *General considerations*



Bedrock depth under the Moesian Platform (Manea et al, 2020)



Spatial variations in uplift and erosion along the Romanian Carpathians and thickness of foredeep sediments (Cloetingh et al, 2004)

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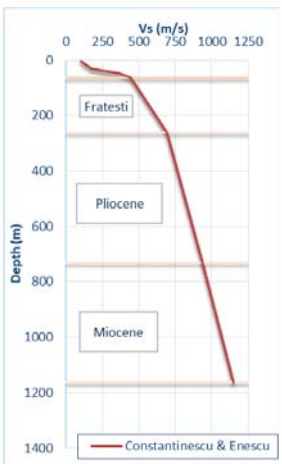
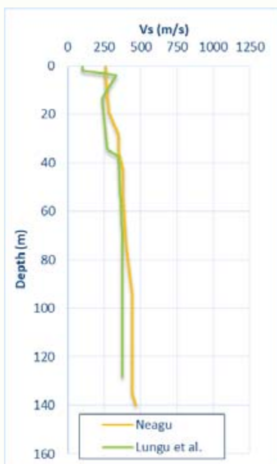
Local site conditions - *issues*

- DEPTH
- PROFILE DEFINITION
- MAGNITUDE DEPENDENCE
- DISCRETIZATION
- DINAMIC PARAMETHERS



Velocity of INCERC (Bucharest) station after Constantinescu and Enescu (1985)



Velocity of INCERC (Bucharest) station after Neagu (2015) after Aldea et al. (2006) and Lungu et al. (1998)

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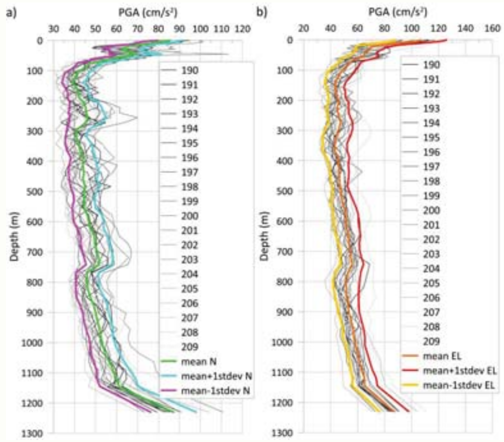
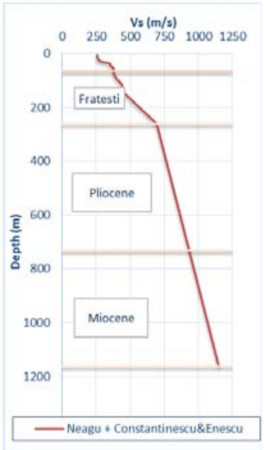
Local site conditions - *depth*





August 30, 1986 Vrancea earthquake		
Parameters	Parameter values/functions	Reference
Mw	7.1	ROMPLUS (Radulian et al. 2019)
Epicenter	45.52º lat. N 26.49º long. E	ROMPLUS (Radulian et al. 2019)
h (km)	131.0	ROMPLUS (Radulian et al. 2019)

- Simulations for Vrancea 1986 scenario
- Nonlinear analyses (N) and equivalent-linear (EL) analyses
- Attenuation from 1162 m to approximately 150 m
- Obvious amplifications in the last 150 m

Should it be considered as local site conditions only the last 150 m?

Peak ground acceleration (PGA) evolution over the stratification depth for each simulation and the statistical descriptors for both nonlinear (N) and equivalent linear (EL) analyses for Vrancea 1986: mean, mean – 1 standard deviation (stdev) and mean + 1 standard deviation

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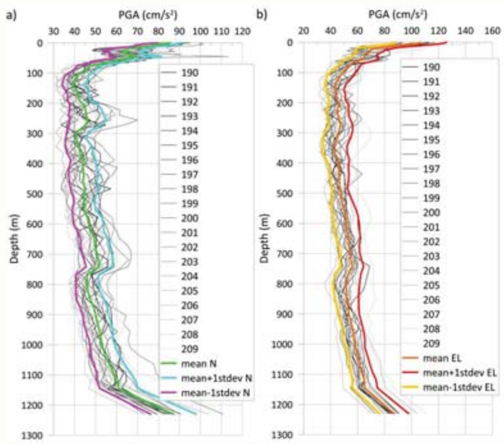
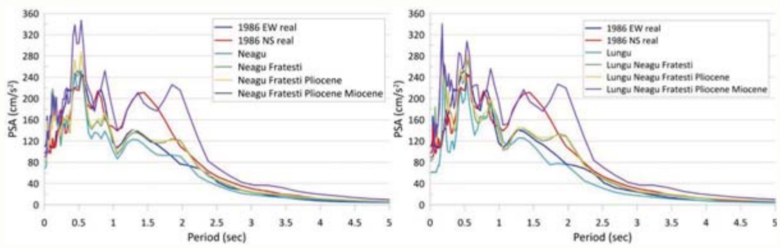
Local site conditions - *depth*





The influence of the geological stratification depth - site effects simulations employing the profiles Lungu et al. (1998) – 128 m and Neagu (2015)/Aldea et al. (2006) – 153 m gradually completed with layers from Constantinescu and Enescu (1985) – 1162 m

Should it be considered as local site conditions only the last 150 m?
NO – shorter profiles do not simulate the long period amplifications

Peak ground acceleration (PGA) evolution over the stratification depth for each simulation and the statistical descriptors for both nonlinear (N) and equivalent linear (EL) analyses for Vrancea 1986: mean, mean – 1 standard deviation (stdev) and mean + 1 standard deviation

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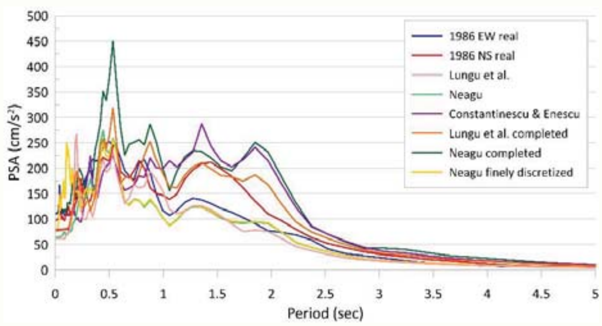
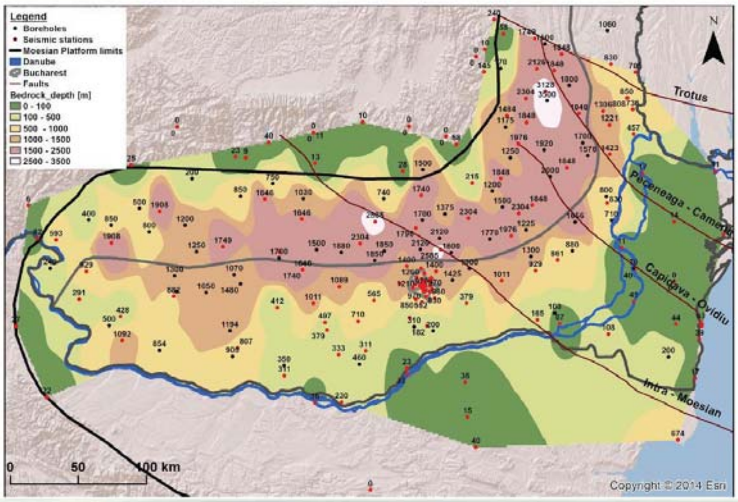
Local site conditions – *profile definition*



Comparison between pseudo-acceleration spectra emphasizing different local site conditions effects depending on the stratification definition (Cotovanu, 2020)



Bedrock depth under the Moesian Platform (Manea et al, 2020)

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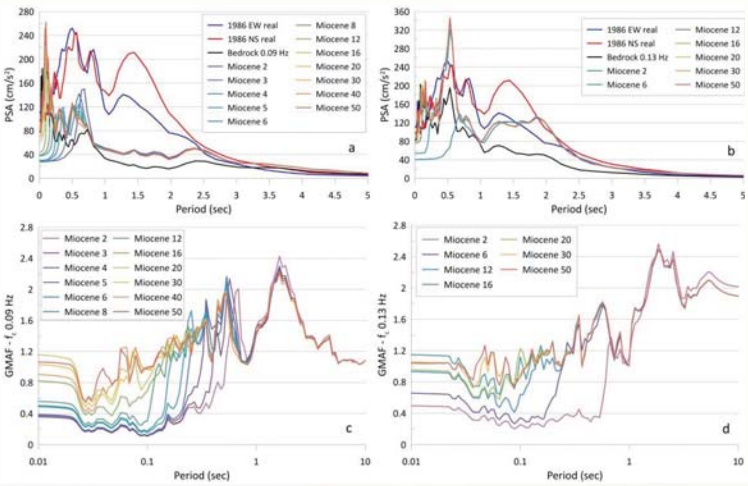
Local site conditions – *magnitude dependence and discretization*



The influence of the discretization used in defining geological stratification. Comparisons (GMAFs - c, d and PSA - a, b) between simulations of Miocene layer effects when divided into variants of equal elements (2 - 50 elements) and subjected to bedrock motions generated with source corner frequencies of 0.09Hz (a, c) and 0.13Hz (b, d)

Two sets of simulations were performed for the deepest layer (Miocene layer) with several discretization variants, using two simulated motions at the bedrock with different source spectrum corner frequency:

- 0.09Hz (11s period – lower magnitude/stress drop)
- 0.13Hz (7.7s period – higher magnitude/stress drop)

"Lower magnitudes" do not "activate" long period amplifications - the local stratification response is dependent on the source characteristics

Discretization as fine as possible improves simulations, especially in the short period domain (0.02-0.7s)

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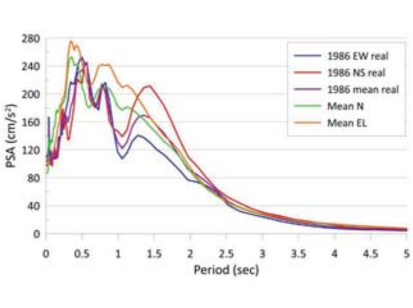
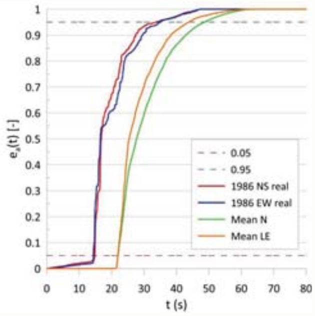
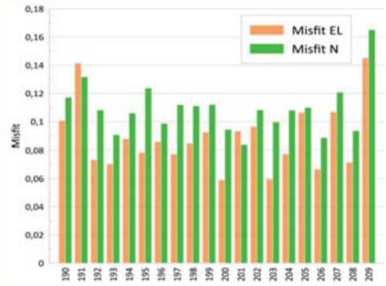
Local site conditions – *dynamic parameters*

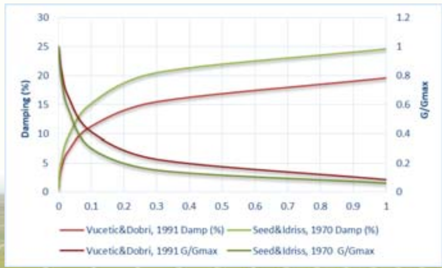




Comparison between the recorded ground motions at the INCERC station during the earthquake of August 30, 1986, and the simulations considering a nonlinear analysis (N) and an equivalent linear analysis (EL) regarding the mean simulation spectra, and the mean normalized cumulative energy Cotovanu (2020)



Comparison between the recorded ground motions at the INCERC station during the earthquake of August 30, 1986, and the simulations considering a nonlinear analysis (N) and an equivalent linear analysis (EL) in terms of mean misfits after Karimzadeh (2019) – PGA, PGV, PGV/PGA, CAV, Ia, teff, ASI, HI, VSI, PSA (Cotovanu, 2020)



General functions of shear modulus and damping curves

There is a need for dynamic characteristics for deeper strata. The differences between nonlinear (N) and equivalent linear analysis (EL) is determined probably by the general curves employed

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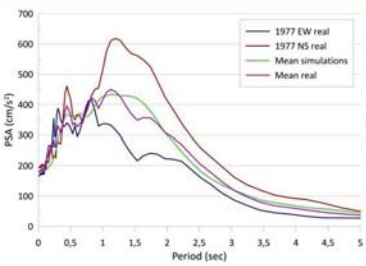
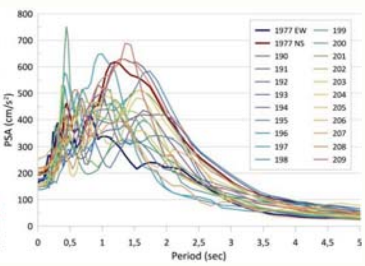
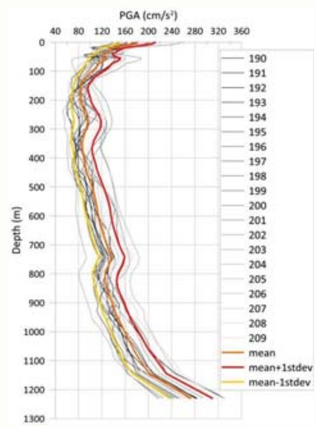
Local site conditions - *validation*

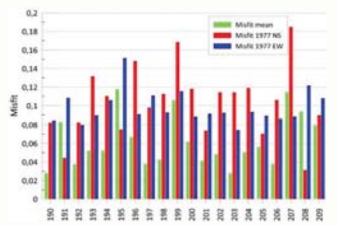




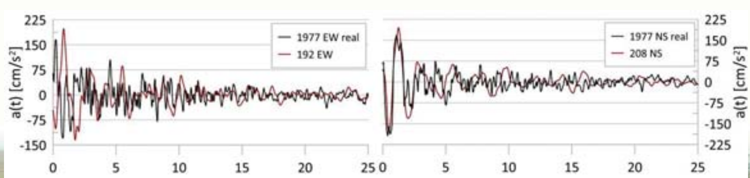
Comparison between the mean response spectra of simulations, each simulation separately and the INCERC ground motion recordings of the March 4, 1977 seismic event



Peak ground acceleration (PGA) evolution over the stratification depth for each simulation and the statistical descriptors: mean, mean – 1 standard deviation (stdev) and mean + 1 standard deviation



Mean misfits between the simulations and the recordings (green - in relation to the mean of the two directions, red - in relation to the NS component, blue - in relation to the EW component) (Cotovanu, 2020)



Simulations with the best misfits to the ground motions produced by March, 1977, Vrancea earthquake at the INCERC station

Conclusions

- The use of a shallow depth geological profile does not generate the changes specific to the local site conditions: the long period amplifications are produced by the entire sedimentary profile that extends to approximately 1 km
- The local stratification response is dependent on the source characteristics (magnitudes, stress drop)
- The discretization controls the recognition of the short period components of the motion
- Due to the considerable high thicknesses of superficial geology in the Southern and Eastern parts of Romania, the parameters of the layers in different locations are incompletely researched.
- There is a need of dynamic characterization of the soil profiles. The differences between nonlinear and equivalent linear simulations should be further researched (the spectral characteristics are better simulated by the nonlinear analysis, while the others by the equivalent linear analysis)

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Questions?

Thank you for your attention

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