INTERNATIONAL SCHOOL AND WORKSHOP.
Nonlinear Mathematical Physics and Natural Hazards

NONLINEAR SEISMOLOGY, THE ACTUAL SEISMOLOGY IN THIS CENTURY !

by
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SOFIA. ***November 28 - December 02,2013*. BULGARIA**

Motto 1:

The instability is the rule,
The stability is the exception !
Tullio Levi-Civita.

Motto 2:

The nonlinear seismology is the rule,
The linear seismology is the exception !
Paraphrasing Tullio Levi-Civita.

N.B. ***All generalizations are false, including each one…?***
(Mark Twain)

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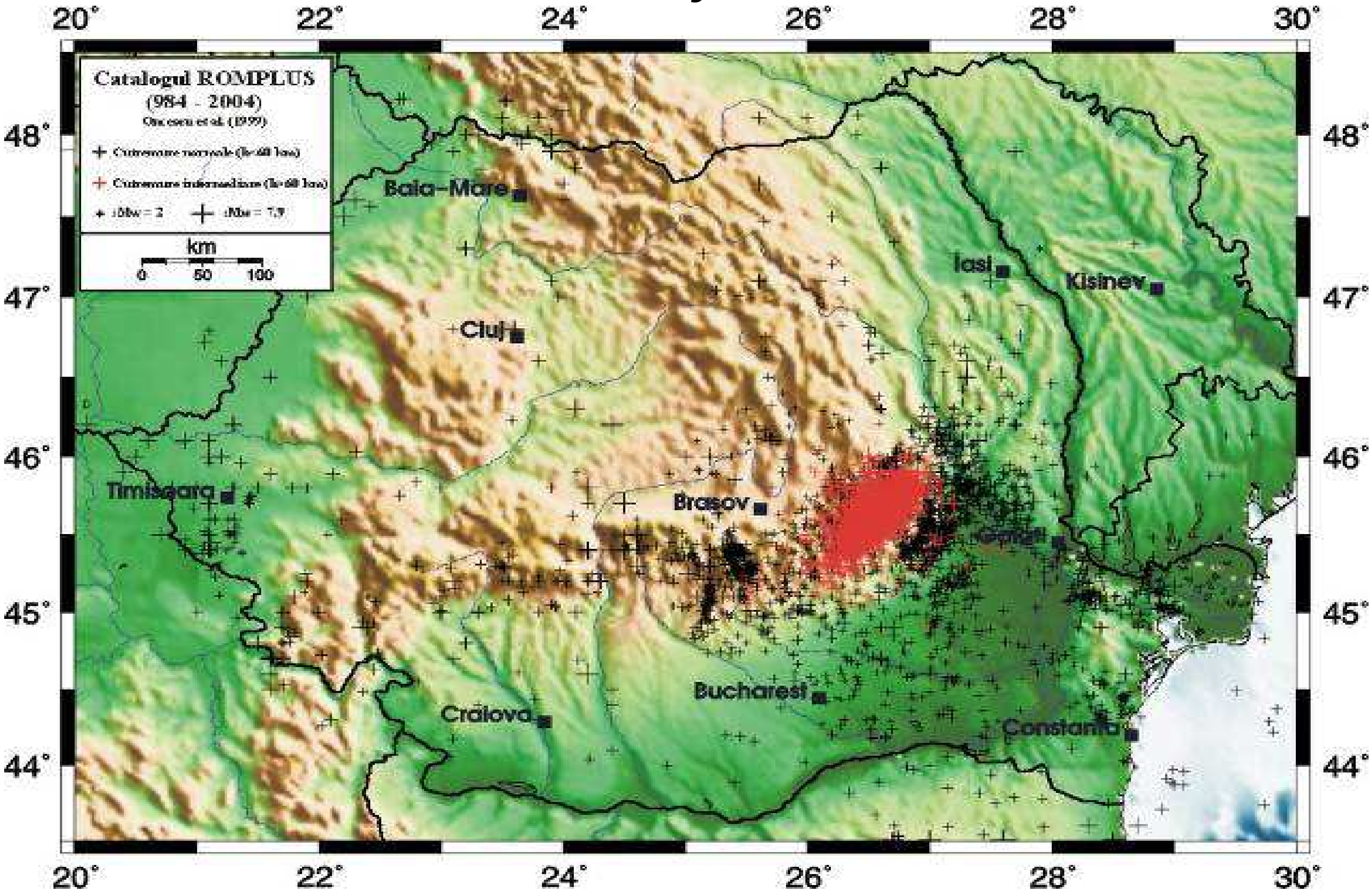

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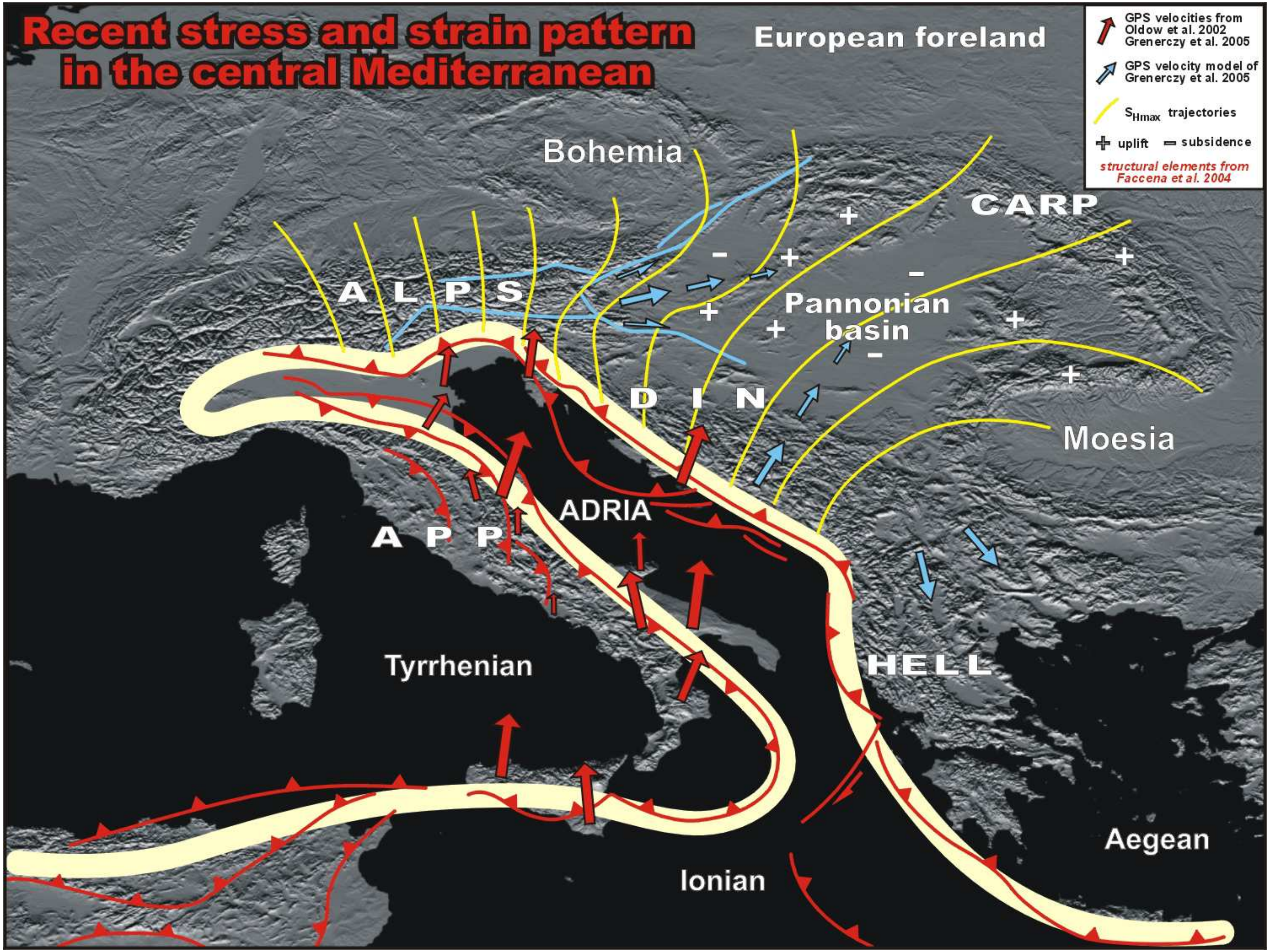
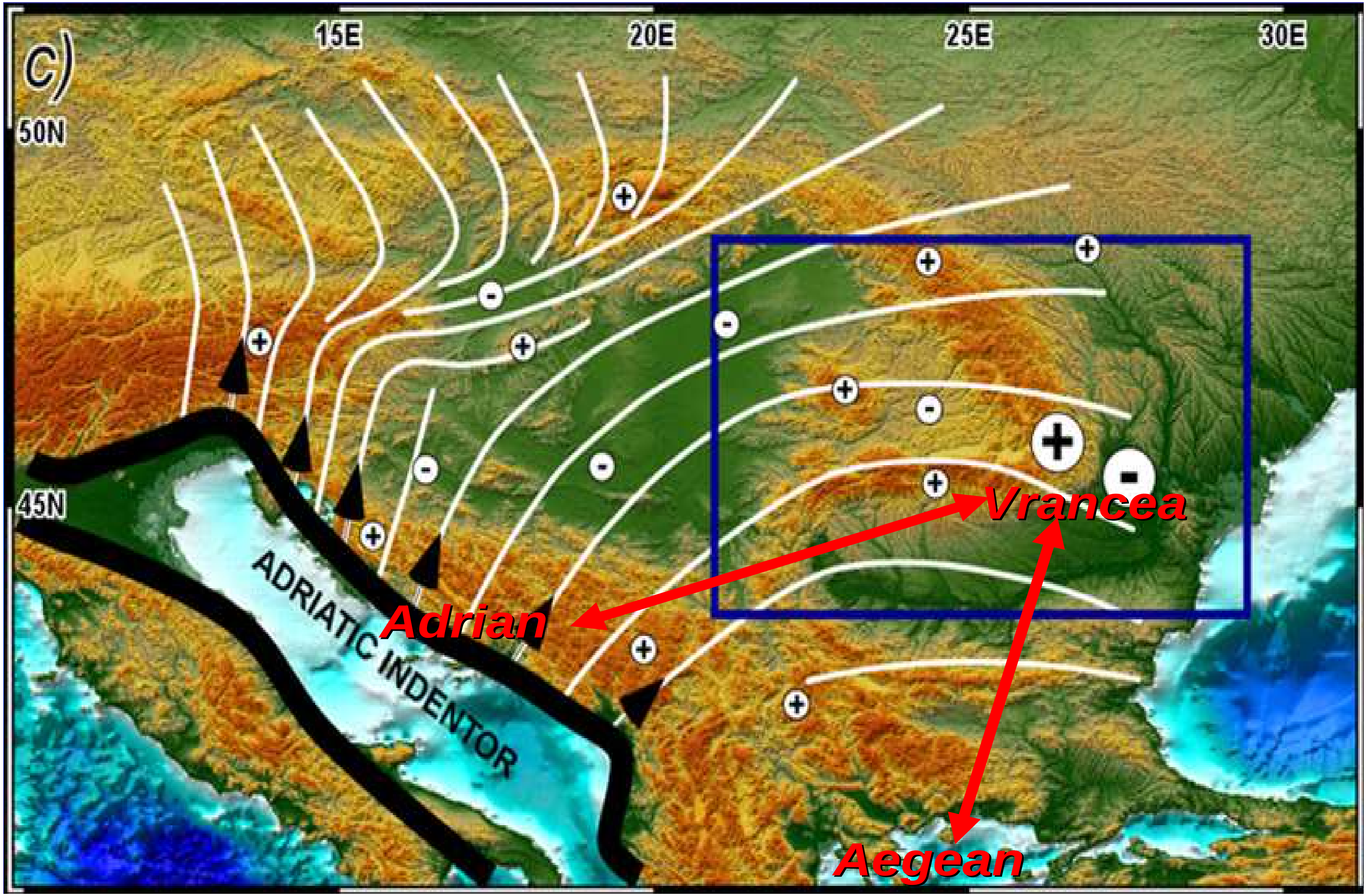


Recent stress and strain pattern
in the central Mediterranean
European foreland
GPS velocities from
Oldow et al. 2002
Grenerczy et al. 2005
GPS velocity model of
Grenerczy et al. 2005
S_{Hmax} trajectories
uplift
subsidence
structural elements from
Faccena et al. 2004
Bohemia
CARP
ALPS
Pannonian
basin
DIN
Moesia
ADRIA
APP
Tyrrhenian
HELL
Aegean
Ionian



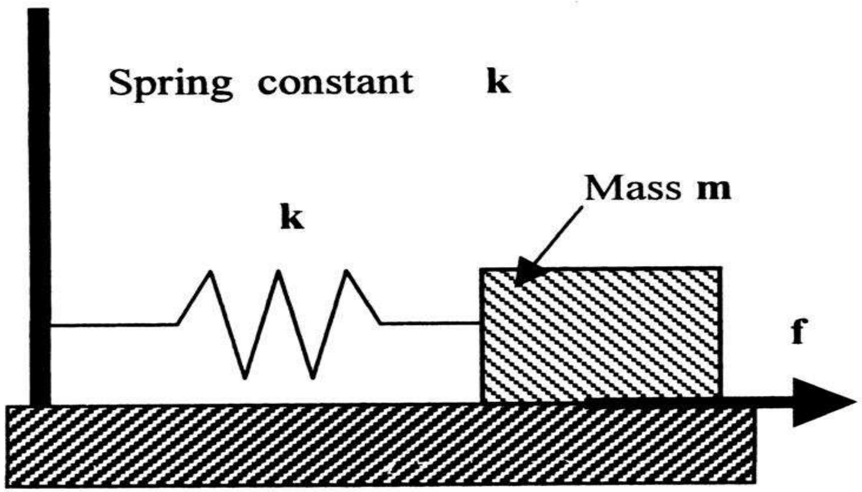
Strain transfer from the active Adriatic, Aegean and Vrancea deformation fronts through the ALCADI- Pannonia System[7]

The leading question : how many cities, villages, metropolitan areas etc. in seismic regions are constructed on rock sites ? Most of them are located on alluvial deposits/ sediments,on Quaternary layers , in river valleys…In last book of Prof.Peter M. Shearer[8,11], we can find… among others , the following concepts :

(i)- Strong ground accelerations from large earthquakes can produce a non-linear response in shallow soils;

(ii)-When a non-linear site response is present, then the shaking from large earthquakes cannot be predicted by simple scaling of records from small earthquakes;

(iii)-This is an active area of research in strong motion and engineering seismology !



A mechanical linear oscillator of mass m, a spring with spring constant k, a single degree of freedom and the attenuation is introduced by adding a damping force f, proportional to the velocity, c.x(t), as a friction between the moving mass and the underlying surface. In the case of "source-free motion"(also called transient, natural, homogeneous, complementary) the equation of motion can be written as:

$$m\ddot{y} + kx + cy + c\acute{y} = \text{or}, F_0\cos\omega t \quad \longrightarrow \quad \ddot{y}(t) + 2\alpha\omega_0.\acute{y} + \omega_0^2.y(t) = 0$$

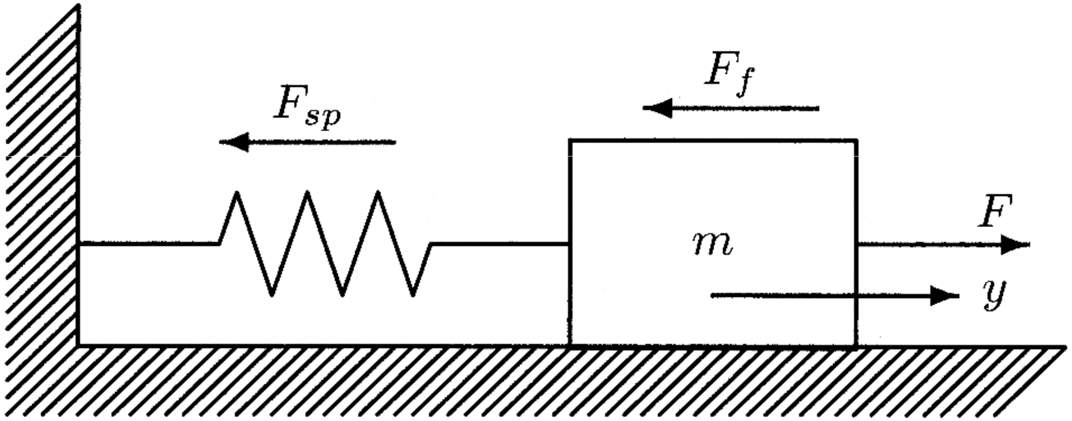
where $k/m=\omega_0^2$;$c_0/m=2\alpha\omega_0$;where c_0 represents the critical viscous damping coefficient and α is the coefficient of friction (dimensionless; if $\alpha=0$,no attenuation).The value of damping coefficient for an $\omega \neq \omega_0$,known as angular frequency of the perturbatory force, is

$$c = 2m\alpha\omega$$

The ratio between damping coefficient (c) and the critical one (c_0) is a dimensionless parameter named damping ratio or fraction of critical damping (D):

$$D\% = c/c_o \ ; \ \xi\,\% = c/c_0.$$

The simplest description of nonlinearity and instability of the whole composed soil-structure system is the mass spring mechanical oscillator of mass „m” sliding on a horizontal surface and attached to a vertical surface through a spring. The mass is subjected to an external force F.



Mass-spring mechanical system

We define y as the displacement from a reference position and write Newton’s law of motion:

$$m\ddot{y} + F_f + F_{sp} = F$$

where: F_f = resistive force due to friction;

F_{sp}=restoring force of the spring and we assuming that it is a function only of displacement y; that is ,F_{sp}=g(y) with $g(0)=0$.The external force *F* is at our disposal, for example: seismic action etc. Depending upon F,F_f , and g, several interesting autonomous and non-autonomous second-order models arise.

For relatively *small displacements*, the restoring force of the spring can be modeled as a linear function: g(y)=ky, k=spring constant.

For *large displacements*, the restoring force may depend nonlinearly on y .For large displacements, the restoring force may depend nonlinearly on y. For example:

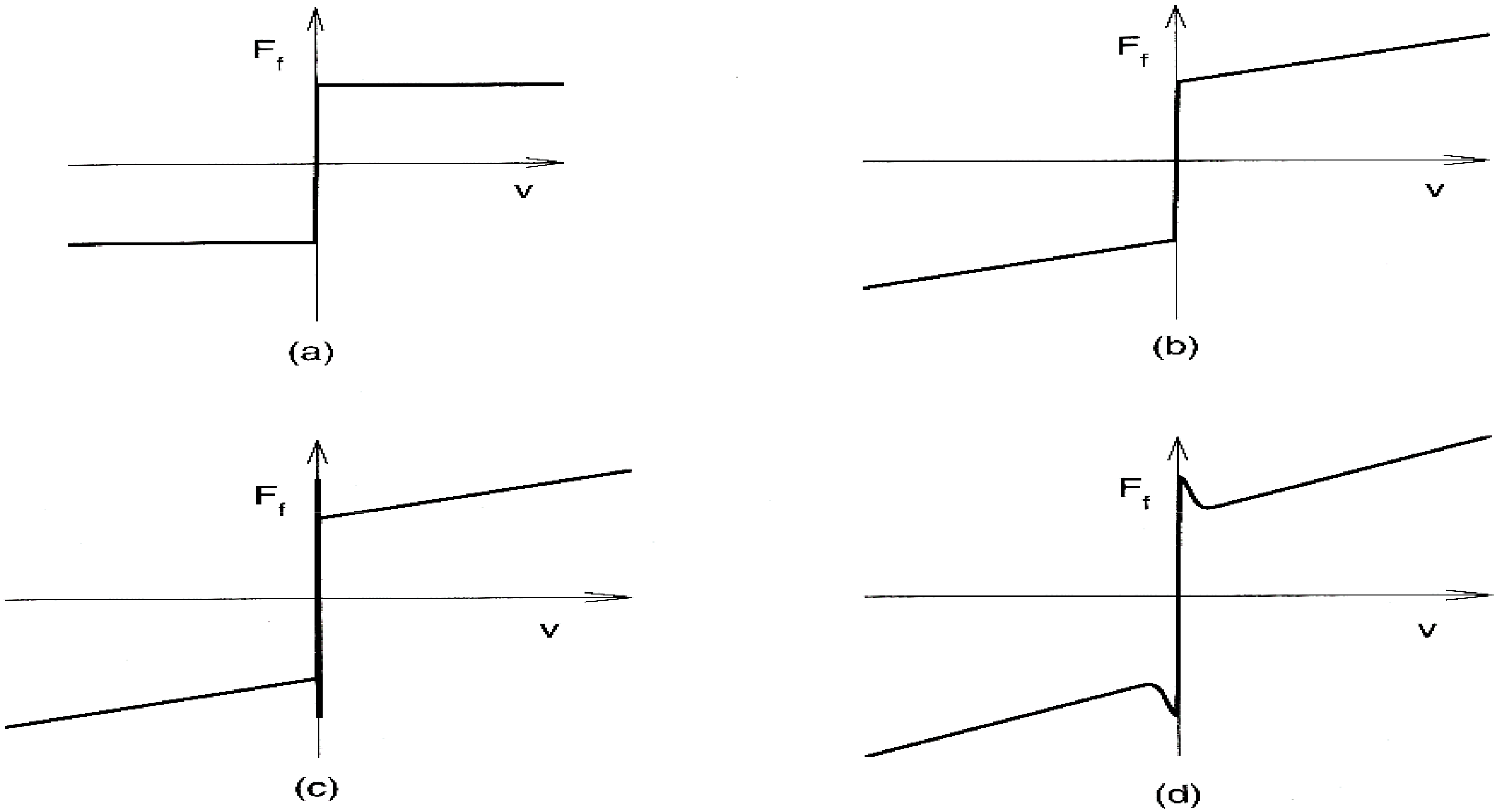
(i)- $g(y)= k(1-a^2y^2)y$, $|ay| < 1$,models so-caled *softening spring*, where, beyond a certain displacement, a large displacement increment produces a small force increment;

(ii)- $g(y)=k(1+\alpha^2y^2)y$, models so-called *hardening* spring, where, beyond a certain displacement, a small increment produces a large force increment…

$\dot{y}$

The resistive force F_f may have components due to: (i)- static; (ii)-*Coulomb*, and: (iii)-*viscous friction.* Once motion has started, for example, an earthquake, the resistive force F_f ,which acts in the direction opposite to motion, is modeled as a function of the sliding velocity v= ý…As the mass moves in a viscous medium, such as air, soil etc. , there will be a frictional force due to viscosity and this force is usually modeled as a nonlinear function of the velocity; that is, F_v =h(v). where h(0)=0.For small velocity, we can assume that F_v = cv. Figures (a)&(b)-examples of friction models for Coulomb friction and Coulombs olus linear viscous friction,respecti-vely. For last one we can apply Boltzmann's *superposition* principle;

Figure (c)-example where the static friction is higher than the level of Coulomb friction; Figure (d)-a similar situation,but with the force decreasing continuosly with increasing velocity,the so-called *Stribeck effect.*



Friction models :(a)-Coulomb friction ;(b)-Coulomb plus linear viscous friction; (c)-static, Coulomb, and linear viscous friction; (d)-static, Coulomb, and linear viscous friction- Stribeck effect, that is, force decreasing continuously with increasing velocity.

The combination of a hardening spring, linear viscous friction, and a periodic external force F=A cos ωt results in the Duffing's equation

$$m\ddot{y} + c\acute{y} + ky + k\,a^2y^3 = A\cos\omega t$$

which is a classical example in the study of periodic excitation of non-linear systems. A combination of a linear spring, static friction, Coulomb friction, linear viscoelastic friction, and zero external force results in:

$$m\,\ddot{y} + ky + c\acute{y} + \eta(y,\acute{y}) = 0$$

where:

$$\eta(y,\acute{y}) = \begin{cases} \mu_k mg\,\mathrm{sign}(\acute{y}), & \text{for } |\acute{y}| > 0 \\ -ky, & \text{for } \acute{y}=0 \text{ and } |y| \le \mu_s mg/k \\ -\mu_s mg\,\mathrm{sign}(y), & \text{for } \acute{y}=0 \text{ and } |y| > \mu_s mg/k \end{cases}$$

where μ_k is the kinetic friction coefficient and μ_s is the static friction coefficient, $0 < \mu_s < 1$. *When* the mass is at rest, there is a static friction force F_s, that act parallel to the surface and is limited to $\pm\,\mu_s$ mg.

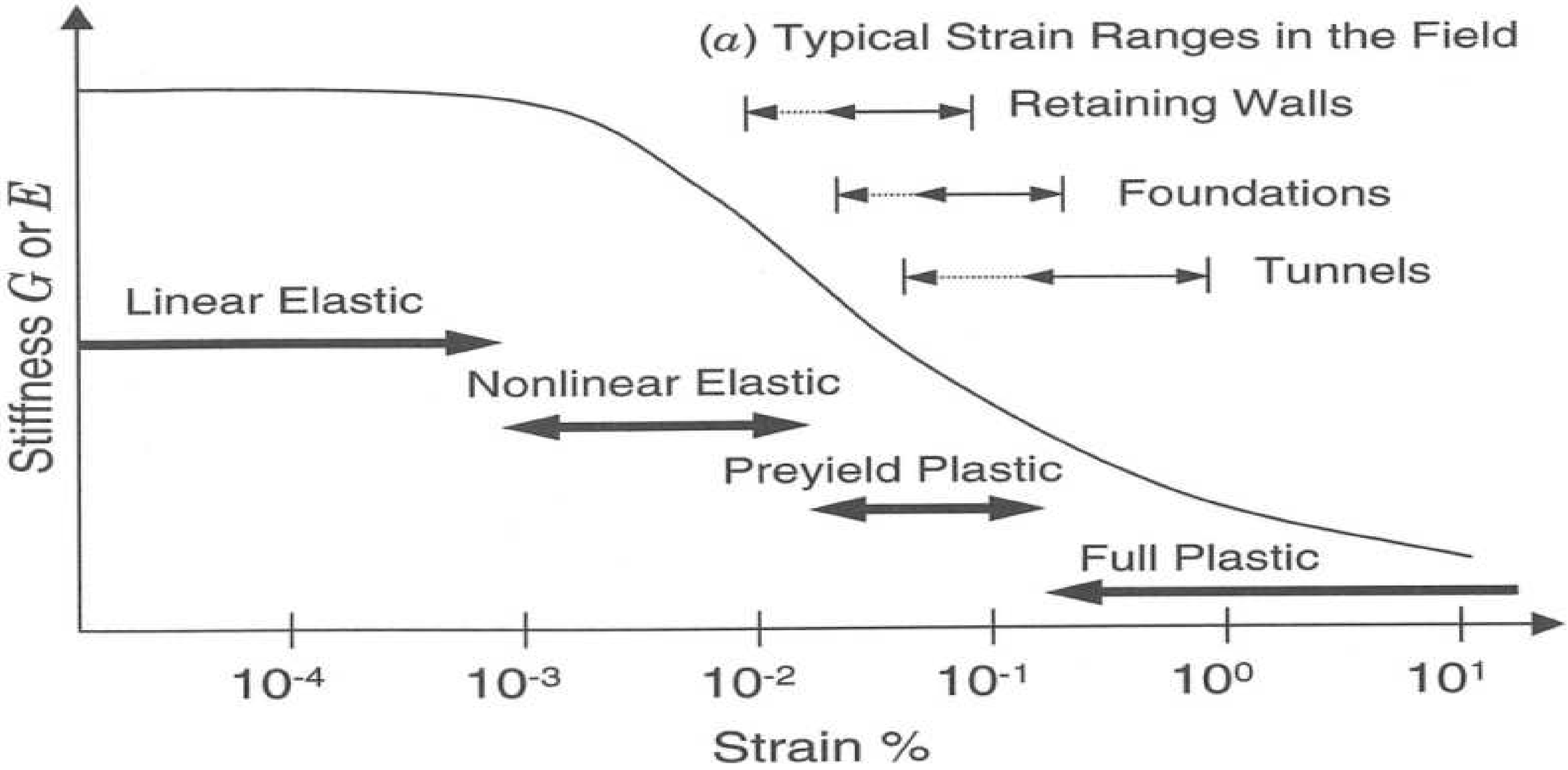
In the elastic method of modal analysis viscosity is introduced until the later stage of the computation at which it is introduced as a fraction **, β**, of critical damping for each mode.

This implies that the damping introduced is not associated with any particular element. This procedure may be satisfactory foe structural analysis, ***but is hardly acceptable for soil-structure analysis where the damping ratio in the soil is several times higher than the structural damping .For large earthquakes there are values for internal damping of 18-55 % in soils …***

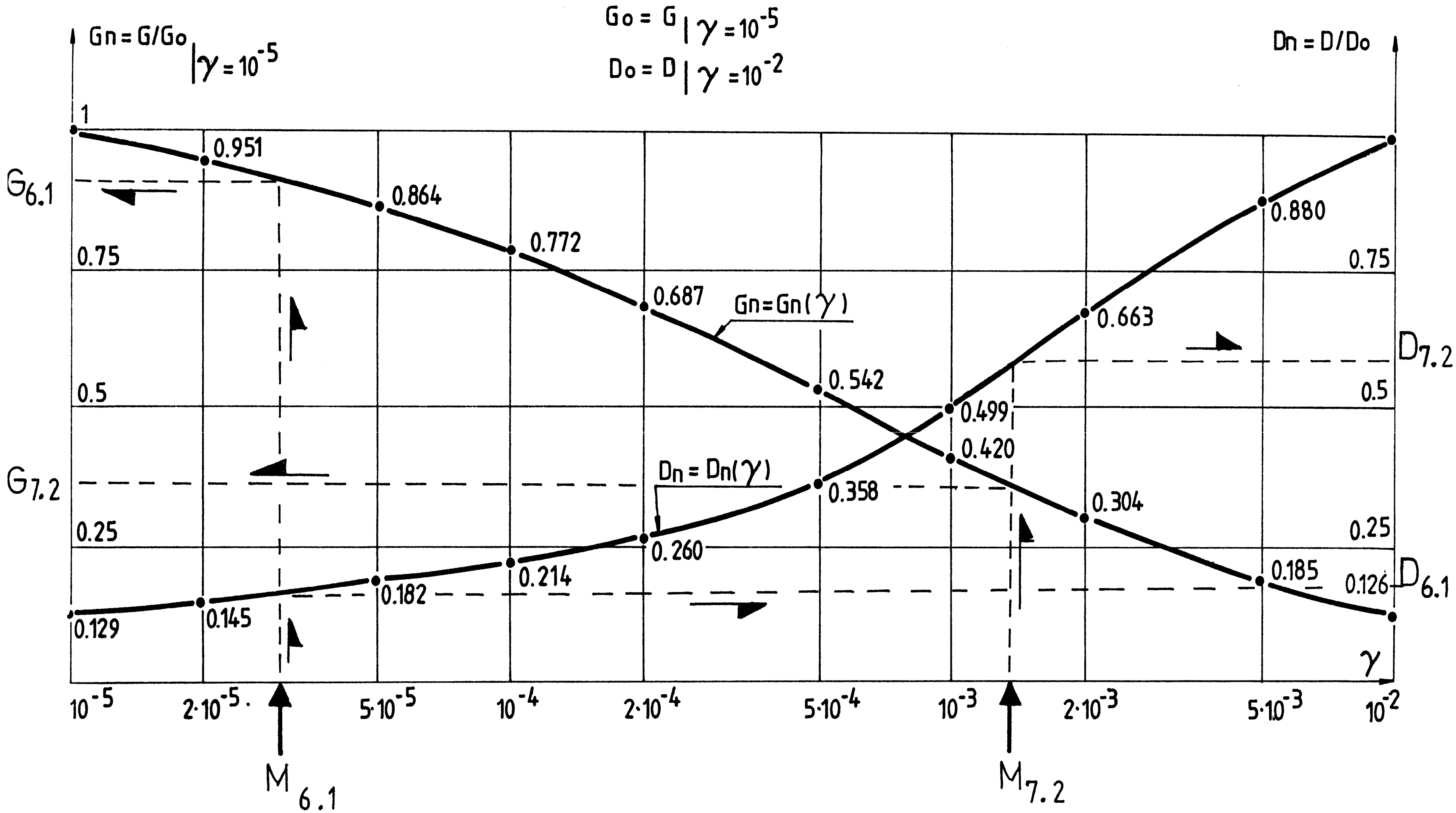
Soils exhibit a strong non-linear behavior under cyclic loading conditions. In the elastic zone, soil particles do not slide relative to each other under a small stress increment, and the stiffness is at its maxim value. The stiffness begins to decrease from the linear elastic value as the applied strains or stresses increase, and the deformation moves into the nonlinear elastic zone [3,4,9].

Stress and strain states are not enough to determine the mechanical behavior of soils. It is necessary, in addition, to model the relation between stresses and strains by using *specific constitutive laws to soils.*

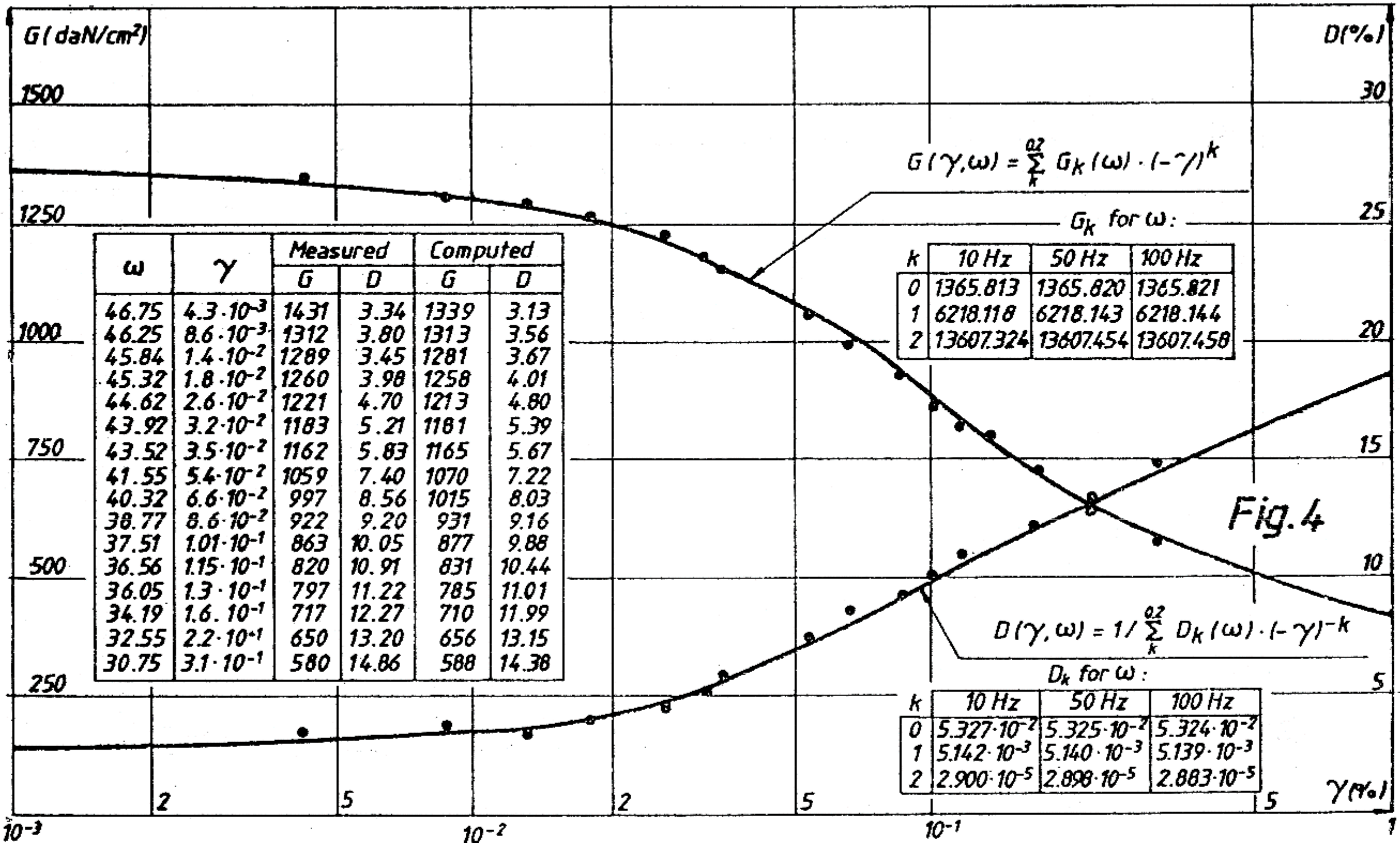
Currently, there are not constitutive laws to describe all real mechanical behaviors of deformable materials like soils.



Stiffness *degradation* curve in terms of shear modulus G and Young's modulus E plotted against logarithm of typical strain levels observed during construction of typical geotechnical structures [7,10].



The variation of *dynamic torsion modulus function (G, daN/cm2*) and *torsion damping function (G%)* of specific strain (γ%) for *sand and gravel samples with normal humidity obtained in Hardin & Drnevich resonant columns (USA patent) from* NIEP, Laboratory of Earthquake Engineering. Normalized values [5-9].



ω	γ	Measured G	Measured D	Computed G	Computed D
46.75	$4.3\cdot10^{-3}$	1431	3.34	1339	3.13
46.25	$8.6\cdot10^{-3}$	1312	3.80	1313	3.56
45.84	$1.4\cdot10^{-2}$	1289	3.45	1281	3.67
45.32	$1.8\cdot10^{-2}$	1260	3.98	1258	4.01
44.62	$2.6\cdot10^{-2}$	1221	4.70	1213	4.80
43.92	$3.2\cdot10^{-2}$	1183	5.21	1181	5.39
43.52	$3.5\cdot10^{-2}$	1162	5.83	1165	5.67
41.55	$5.4\cdot10^{-2}$	1059	7.40	1070	7.22
40.32	$6.6\cdot10^{-2}$	997	8.56	1015	8.03
38.77	$8.6\cdot10^{-2}$	922	9.20	931	9.16
37.51	$1.01\cdot10^{-1}$	863	10.05	877	9.88
36.56	$1.15\cdot10^{-1}$	820	10.91	831	10.44
36.05	$1.3\cdot10^{-1}$	797	11.22	785	11.01
34.19	$1.6\cdot10^{-1}$	717	12.27	710	11.99
32.55	$2.2\cdot10^{-1}$	650	13.20	656	13.15
30.75	$3.1\cdot10^{-1}$	580	14.86	588	14.38

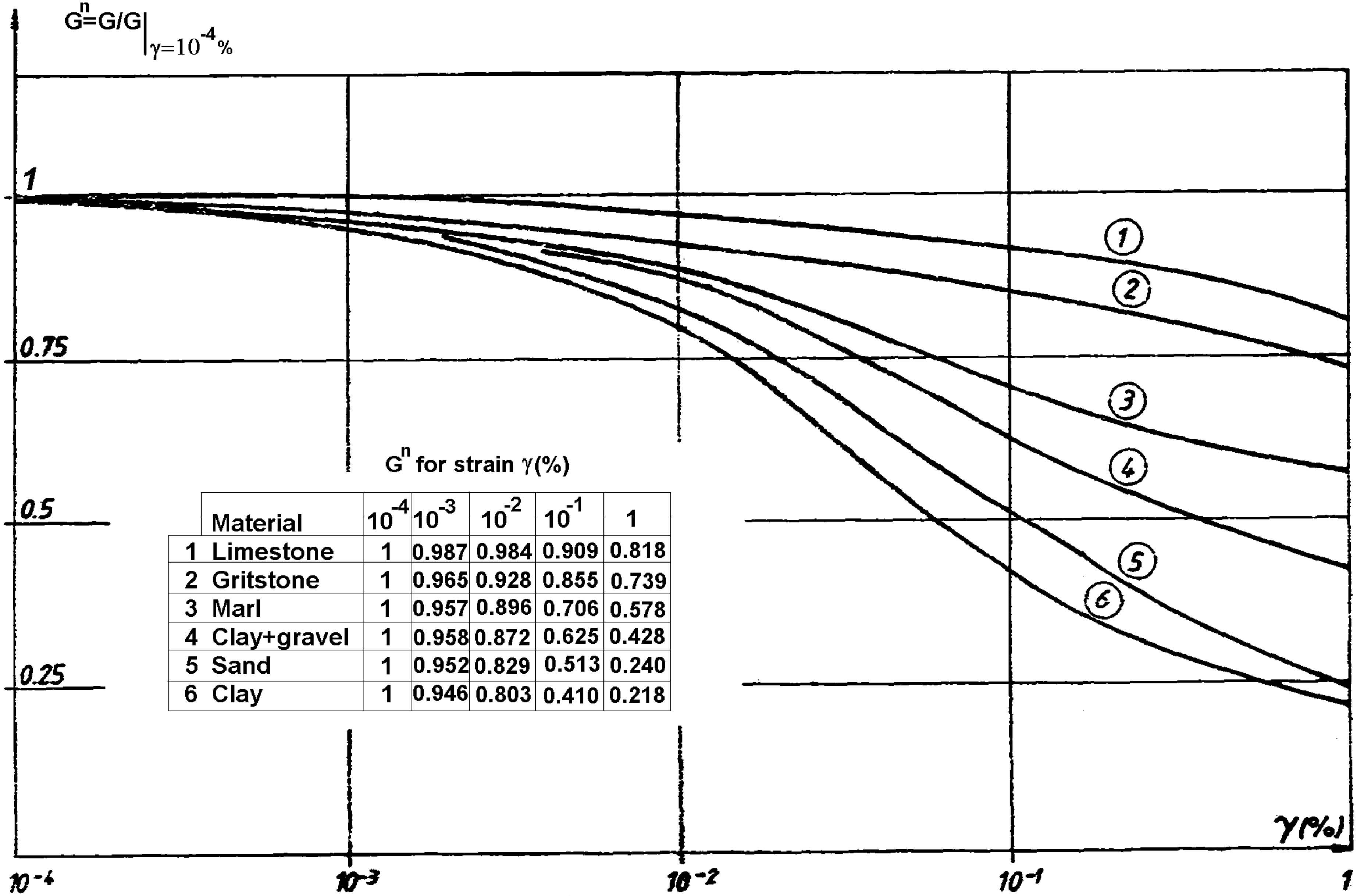
G_k for ω:

k	10 Hz	50 Hz	100 Hz
0	1365.813	1365.820	1365.821
1	6218.118	6218.143	6218.144
2	13607.324	13607.454	13607.458

D_k for ω:

k	10 Hz	50 Hz	100 Hz
0	$5.327\cdot10^{-2}$	$5.325\cdot10^{-2}$	$5.324\cdot10^{-2}$
1	$5.142\cdot10^{-3}$	$5.140\cdot10^{-3}$	$5.139\cdot10^{-3}$
2	$2.900\cdot10^{-5}$	$2.898\cdot10^{-5}$	$2.883\cdot10^{-5}$

The variation of *dynamic torsion modulus function (G, daN/cm2*) and *torsion damping function (G%)* of specific strain(γ%) for *marl samples obtained in Hardin & Drnevich resonant columns (USA patent) from* NIEP, Lab.of Earthquake Engrg.



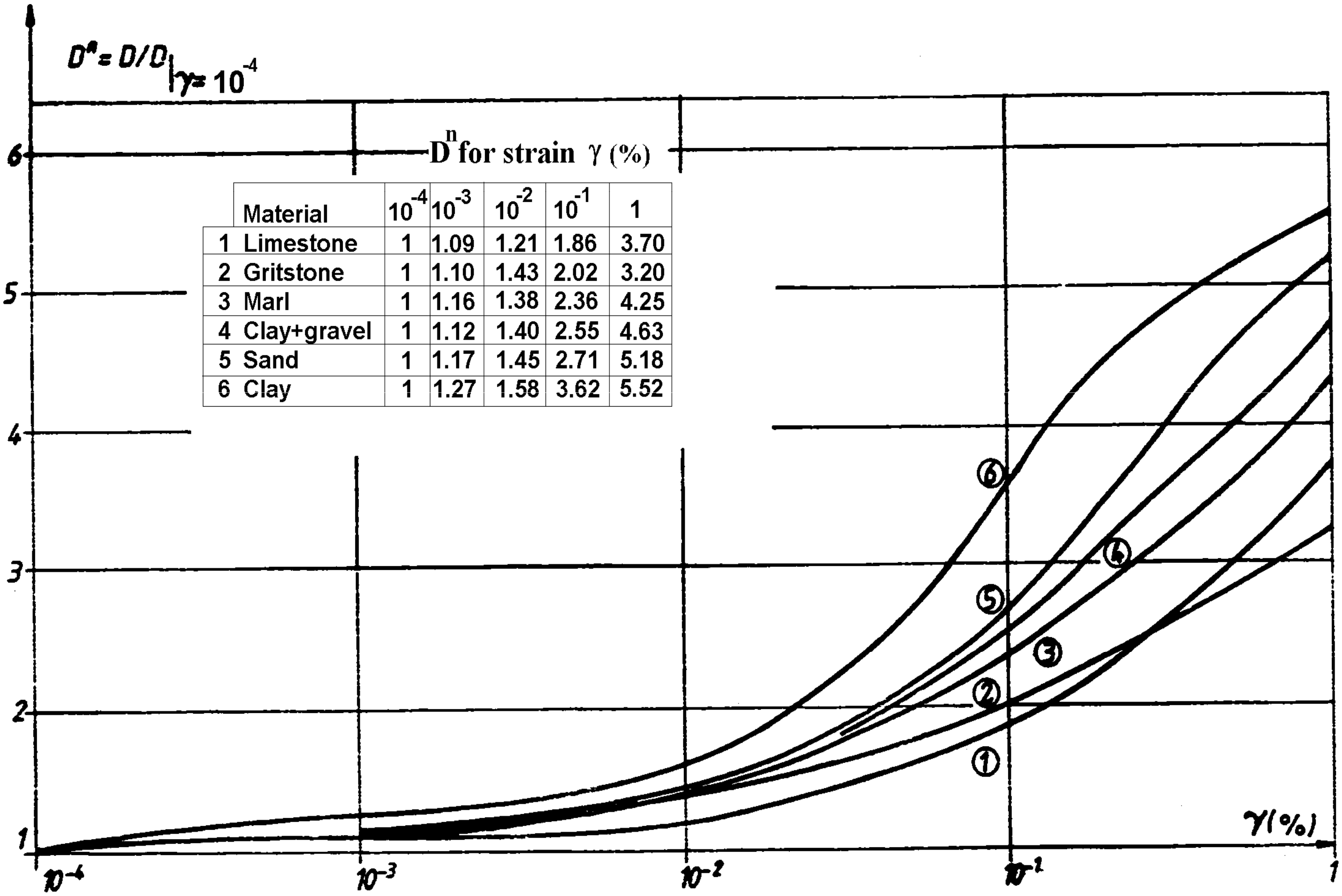
G^n for strain γ(%)

	Material	10^{-4}	10^{-3}	10^{-2}	10^{-1}	1
1	Limestone	1	0.987	0.984	0.909	0.818
2	Gritstone	1	0.965	0.928	0.855	0.739
3	Marl	1	0.957	0.896	0.706	0.578
4	Clay+gravel	1	0.958	0.872	0.625	0.428
5	Sand	1	0.952	0.829	0.513	0.240
6	Clay	1	0.946	0.803	0.410	0.218

Nonlinear relation between dynamic torsion modulus function (G%) and shear-strain(γ%) -experimental data from resonant colums

γ

Nonlinear relation between torsion dumping function (D%) and shear-strain (γ%) – experimental data from resonant columns



	Material	10^{-4}	10^{-3}	10^{-2}	10^{-1}	1
1	Limestone	1	1.09	1.21	1.86	3.70
2	Gritstone	1	1.10	1.43	2.02	3.20
3	Marl	1	1.16	1.38	2.36	4.25
4	Clay+gravel	1	1.12	1.40	2.55	4.63
5	Sand	1	1.17	1.45	2.71	5.18
6	Clay	1	1.27	1.58	3.62	5.52

Keiti Aki **[1993]:** ***„Nonlinear amplification at sediments sites appears to be more pervasive than seismologists used to think. Any attempt at seismic zonation must take into account the local site conditions and this nonlinear amplification”*****[1]!**

From mechanical behavior point of view there are two main groups of main importance: *sands* and *clays*. These soils, although have many common mechanical properties require the use of different models to describe behavior difference. Soils are simple materials with memory: sands are „*rate-independent*” type and clays are „*rate-dependent*”one,names used in mechanical deformable bodies.

However the complexity of these "*simple*" models exceeds the possibility of solving and requires to introduce of simplifying assumptions or conditions which are restricting the loading conditions which makes additional permissible assumptions.

Sands typically have low rheological properties and can be modeled with an acceptable *linear elastic model* and clays which frequently presents significant changes over time can be modeled by a *nonlinear viscoelastic model*

Viscoelastic material behavior could be characterized using Boltzmann's formulation of the constitutive law[2].Theory of viscoelasticity is approaching completion…Boltzmann's formulation of the constitutive relation between stress and strain as expressed by the convolution integrals (1)&(2) is general in the sense that all linear behavior may be characterized by such a relation. Conversely, if the response is characterized by one of the convolution integrals then the Boltzmann's *superposition* principle is valid.

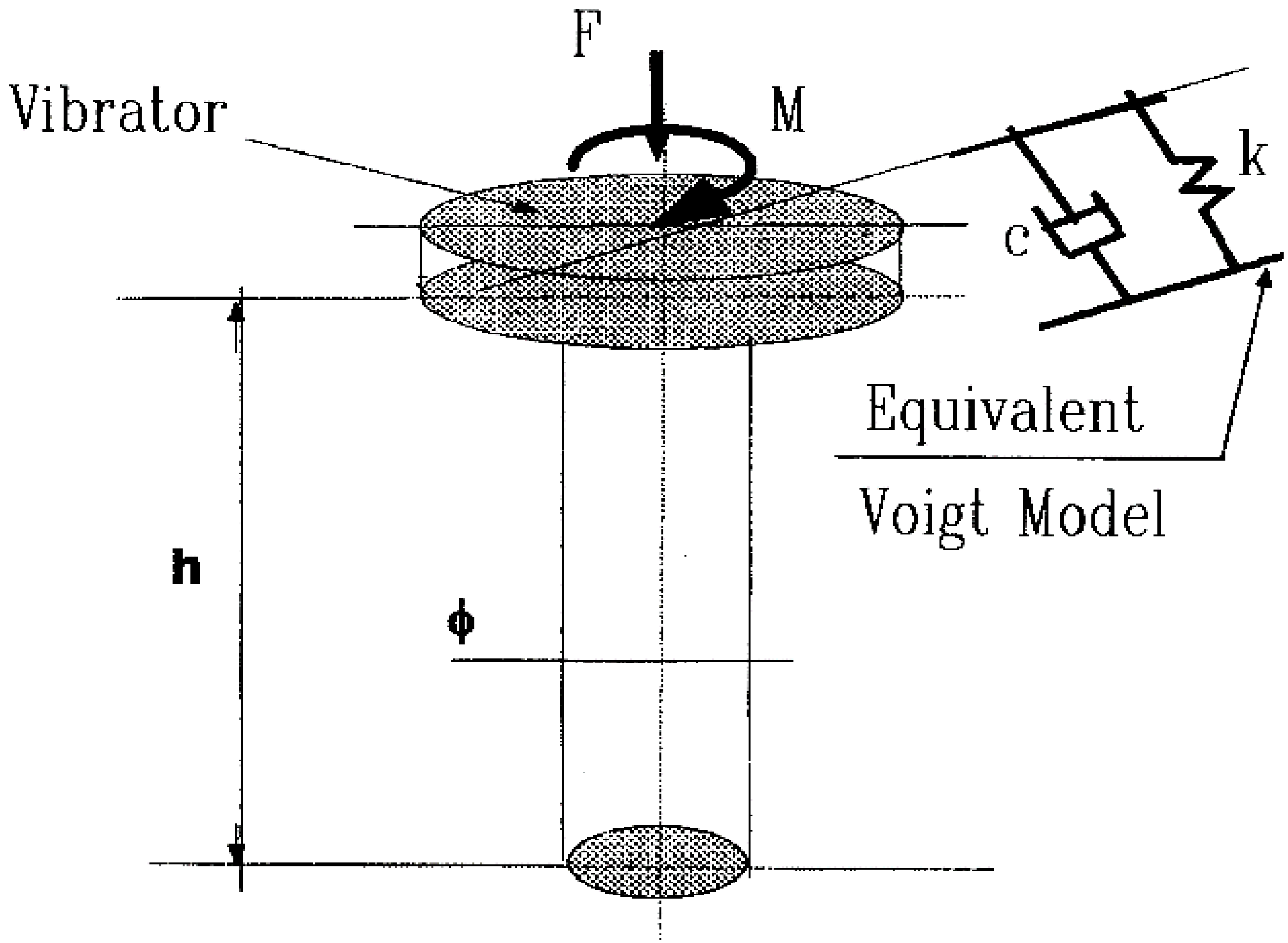
$$p(t) = \int_{-\infty}^{\infty} r(t-\tau)\,de(\tau) \quad (1) \quad \& \quad e(t) = \int_{-\infty}^{\infty} c(t-\tau)\,dp(\tau) \quad (2)$$

In terms of convolution operators as: *p= r* ◘ *de* (*1'*) ; ***e*** *=c* ◘ *dp* (*2'*)
If the material response is characterized by one of the convolution integrals, then Boltzman's superposition principle is valid !

Nonlinear viscoelastic model

Displacement vector u , the tensors T & E for tension and strain , in case of nonlinear viscoelastic materials, are function of position x and time t, functions that define the *viscoelastic body state.* For a given time and set t = ct. these functions will define a state elastic body. The reduction of viscoelastic states to elastic states is observed experimentally in samples of clay behaviour subjected to a *triaxial creep tests* ;the isochronous $\sigma(\varepsilon) = \sigma(\varepsilon,t)|_{t=ct}$ and /or $\tau(\gamma) = \tau(\gamma,t)|_{t=ct}$ being tension-strain curves which can be modelled with a linear elastic model.

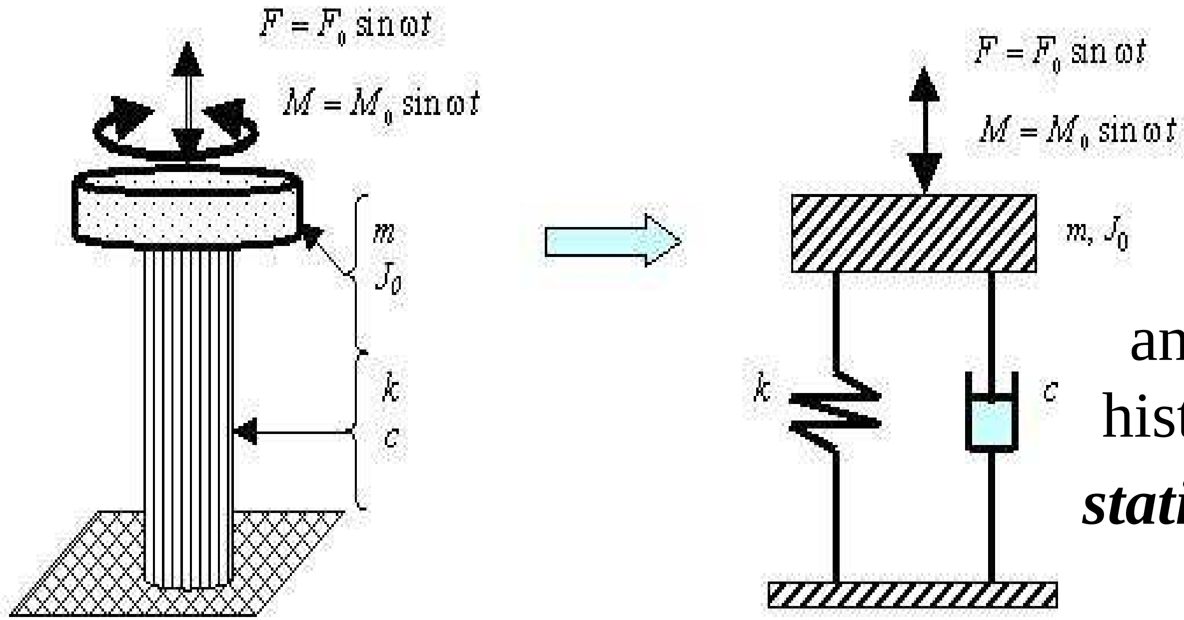
The model presented here is based on reducing *viscoleastic states to elastic states* and the nonlinear relaxation functions K=K(ε,t) and G=G(γ) are reduced to nonlinear elastic modulus functions, K = K (ε) and G = G (γ) [3,5].



The mechanical model of resonant column

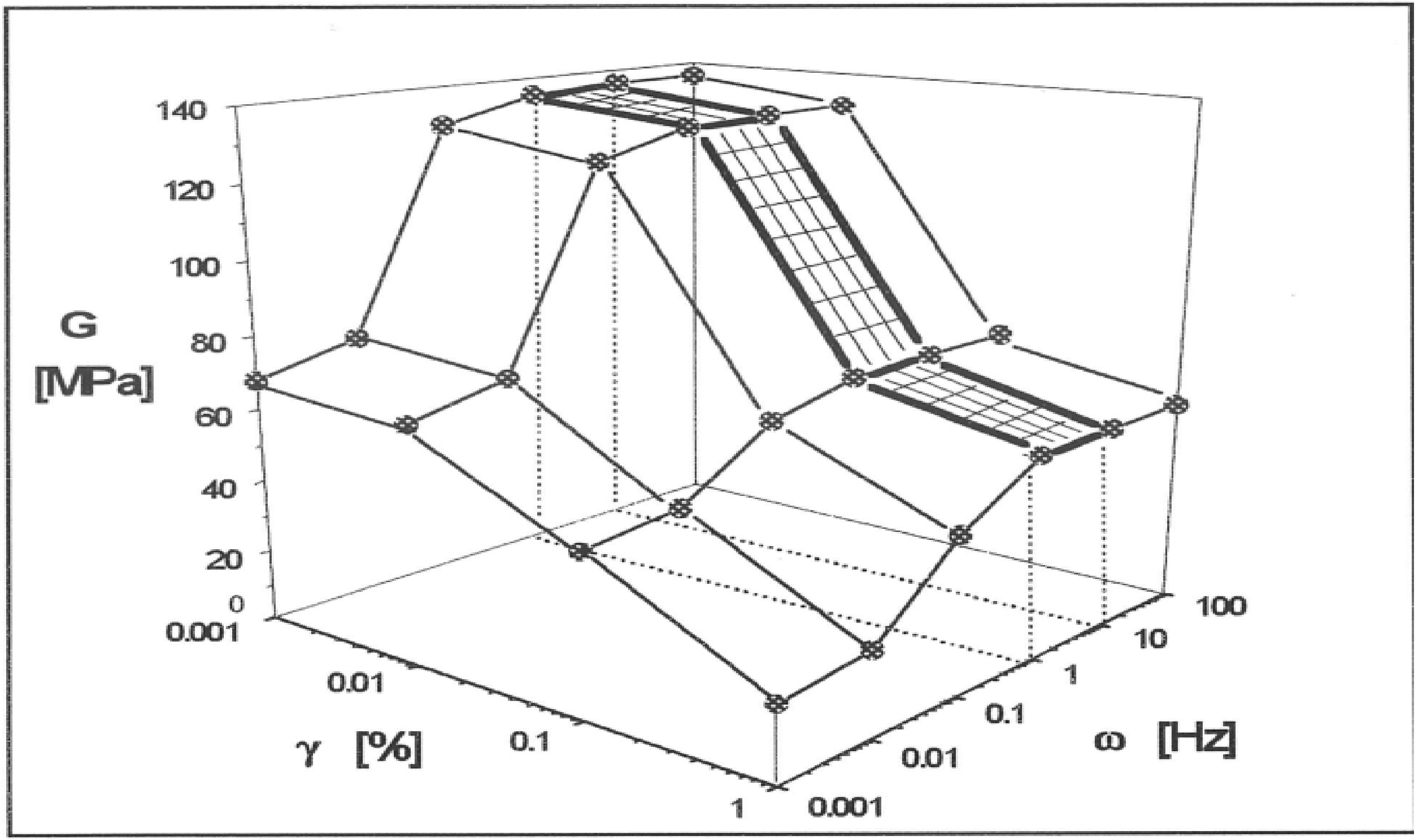
Relaxation functions of the nonlinear viscoelastic soil along the time variable „t" should contain as arguments the strain tensor invariants,K = K (ε, t) and G=G(γ). Under these conditions the nonlinear viscoelastic constitutive equations for soils take the form[3,4]:

$$\sigma(t)=\int_0^t K(\varepsilon,t-\varepsilon)\cdot\dot{\varepsilon}(s)\cdot ds \qquad \tau(t)=\int_0^t G(\gamma,t-\gamma)\cdot\dot{\gamma}(s)\cdot ds$$

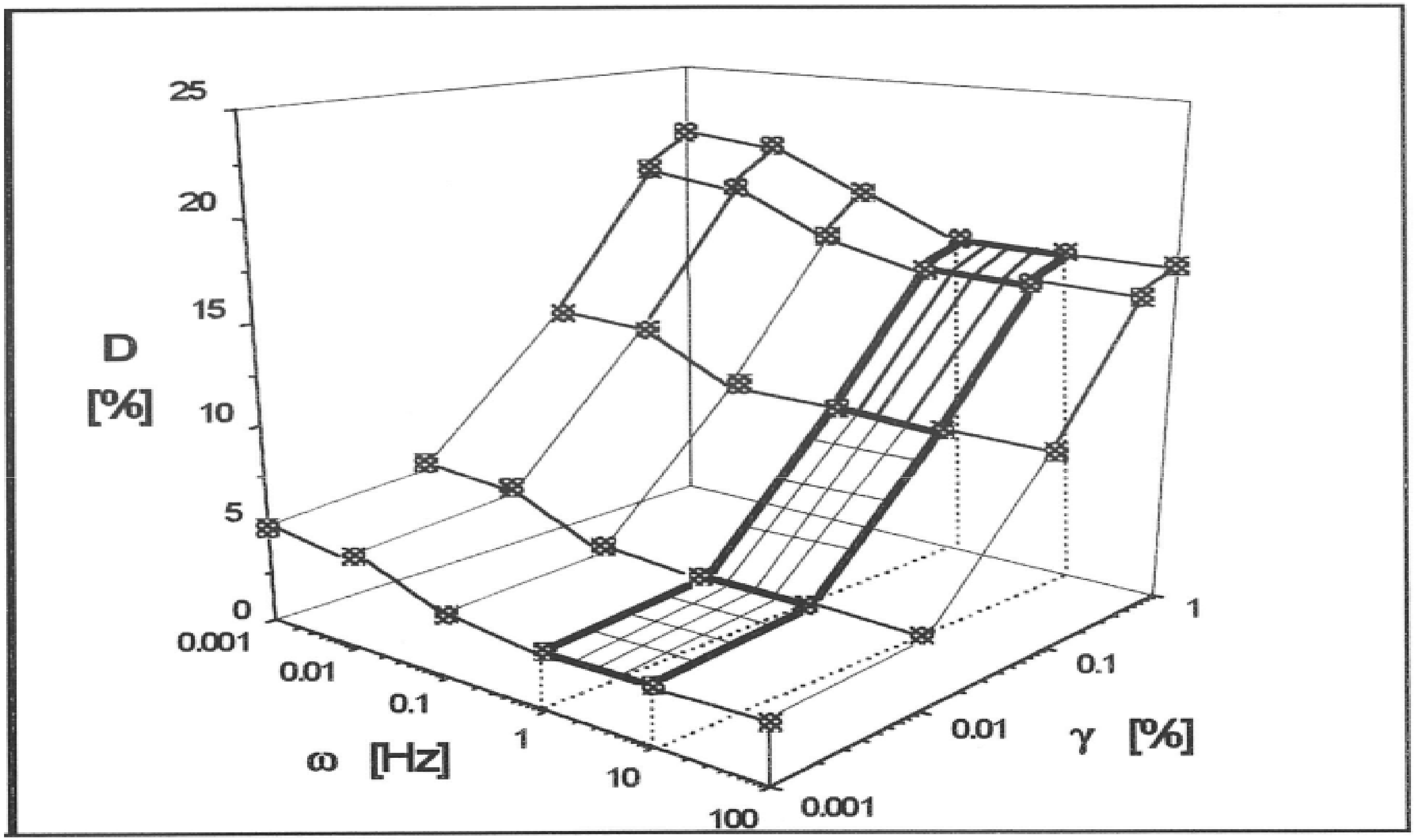


In these constitutive equations: K(ε,t) and G(γ,t) are the nonlinear relaxation functions…

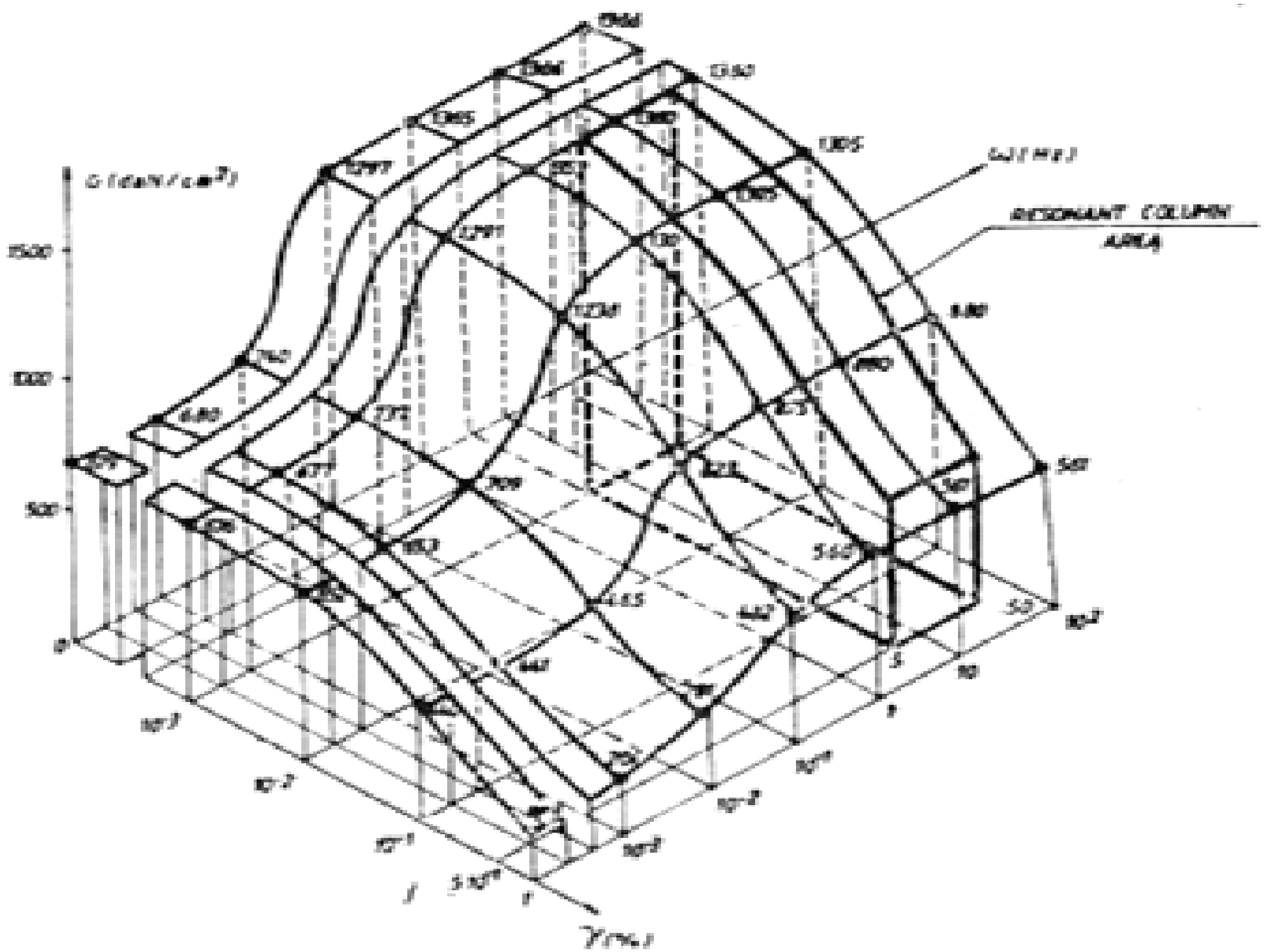
and we can accept a strain-history of form (***harmonic & stationary***):**ε(t)=ε_o. exp(-iωt); γ(t)=γ_o. exp(-iωt)**



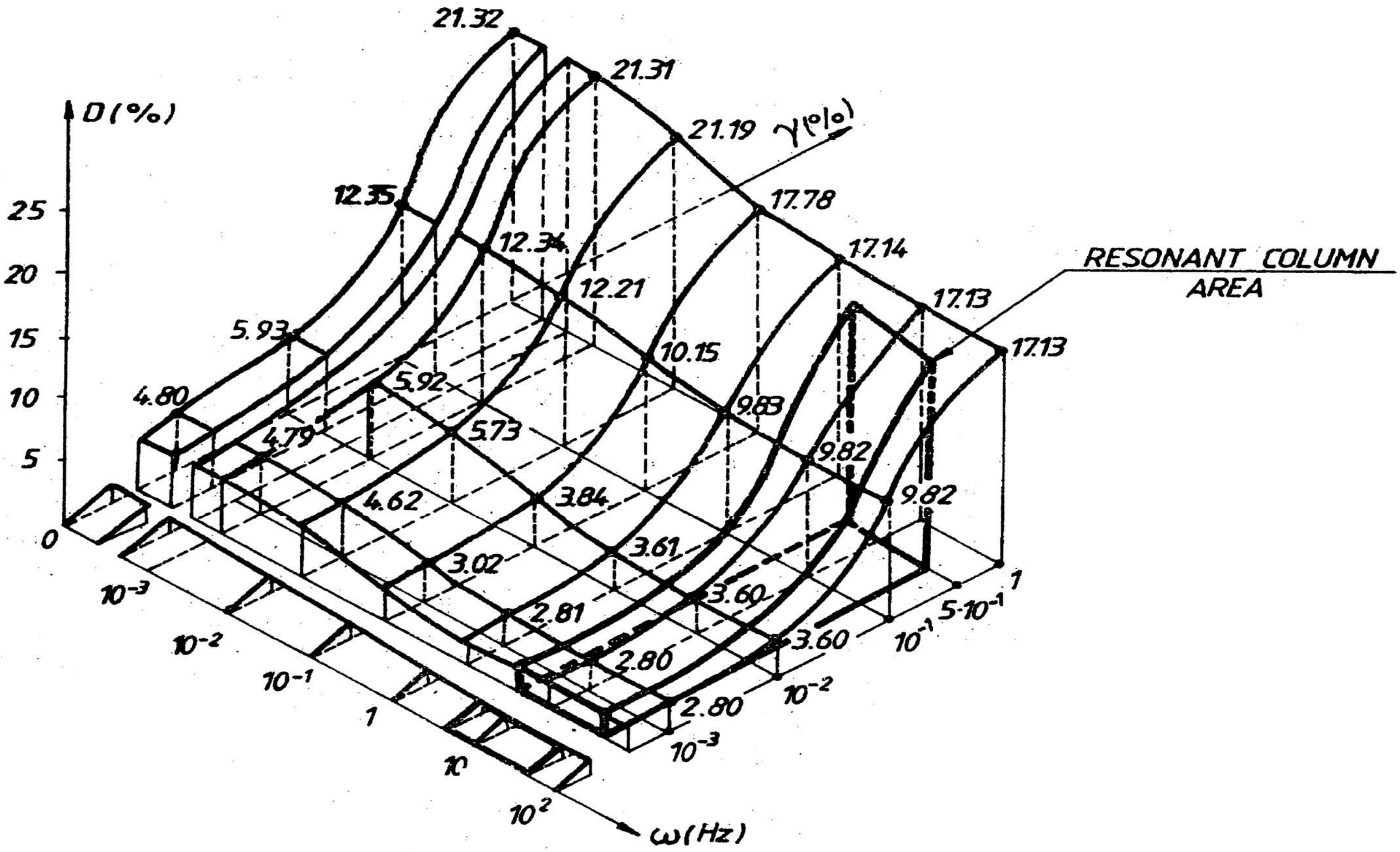
Dependence of dynamic torsio modulus function (G, daN/cm2) with shear strains(γ%) and frequency (ω)[3,5]



Dependence of torsion damping function (D%) with shear strains (γ%) and frequency (ω,Hz) [3,5]



The variation of dynamic torsion modulus function(G, daN/cm^2) of specific strain(γ %) and frequency (Hz) for clay obtained in Hardin and Drnevich resonant columns from NIEP. Absolute values.



The variation of torsion damping function(D%) of specific strain(γ %)and frequency (ω, Hz) for clay obtained in Hardin and Drnevich resonant columns from NIEP. Absolute values.

1.Sand with gravel : **Gn = 0.344 +0.656/ (1+14.651 $\gamma^{0.716}$)**
Dn = 1.428 -1.212/ (1+2.43 $\gamma^{0.682}$);

2.Loess : **Gn = 0.107+0.903/ (1+13,12 $\gamma^{0.682}$**
Dn = 1.556 -1.367/ (1+1.780 $\gamma^{0.655}$);

3.Diluvian clay : **Gn = 0.176+0.824/(1+27.357 $\gamma^{0.986}$)**
Dn = 1.085 -0.888/(1+10.674 $\gamma^{0.950}$);

4.Grey marl : **Gn = 0.542 +0.468/ (1+18.724 $\gamma^{0.73}$)**
Dn = 1.711-1.476/(1+1.41 $\gamma^{0.593}$);

5.Limestone : **Gn = 0.737+0.263/(1+3.974 $\gamma^{0.456}$)**
Dn = 1.902-1.627/(1+0.732 $\gamma^{0.691}$).

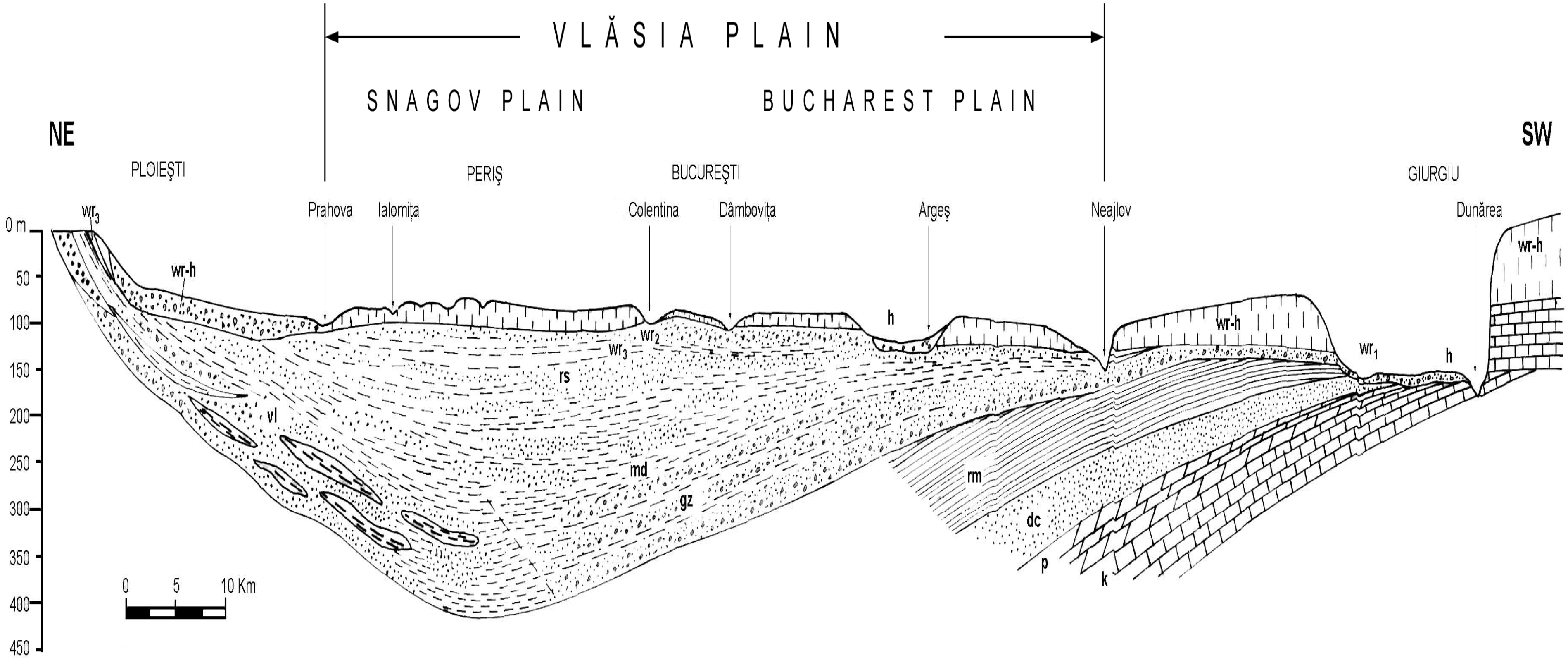
In engineering applications they are interested in the soil behavior to earthquakes dangerous frequencies, that are between 0.1 and 10 Hz. In this domain we can consider G_k and D_k to be constant in relation to frequency and will depend of shear strain γ%. Then ,the dynamic functions are:

$$G(\gamma)=\sum_{k}^{0.2} G_k.(-\gamma)^k \quad ; \quad D(\gamma)=1/\sum_{k}^{0.2} D_k.(-\gamma)^k$$

and all of them are function of shear strains (γ%) developed during of strong earthquakes…

Geological cross-section in the eastern part of the Romanian Plain

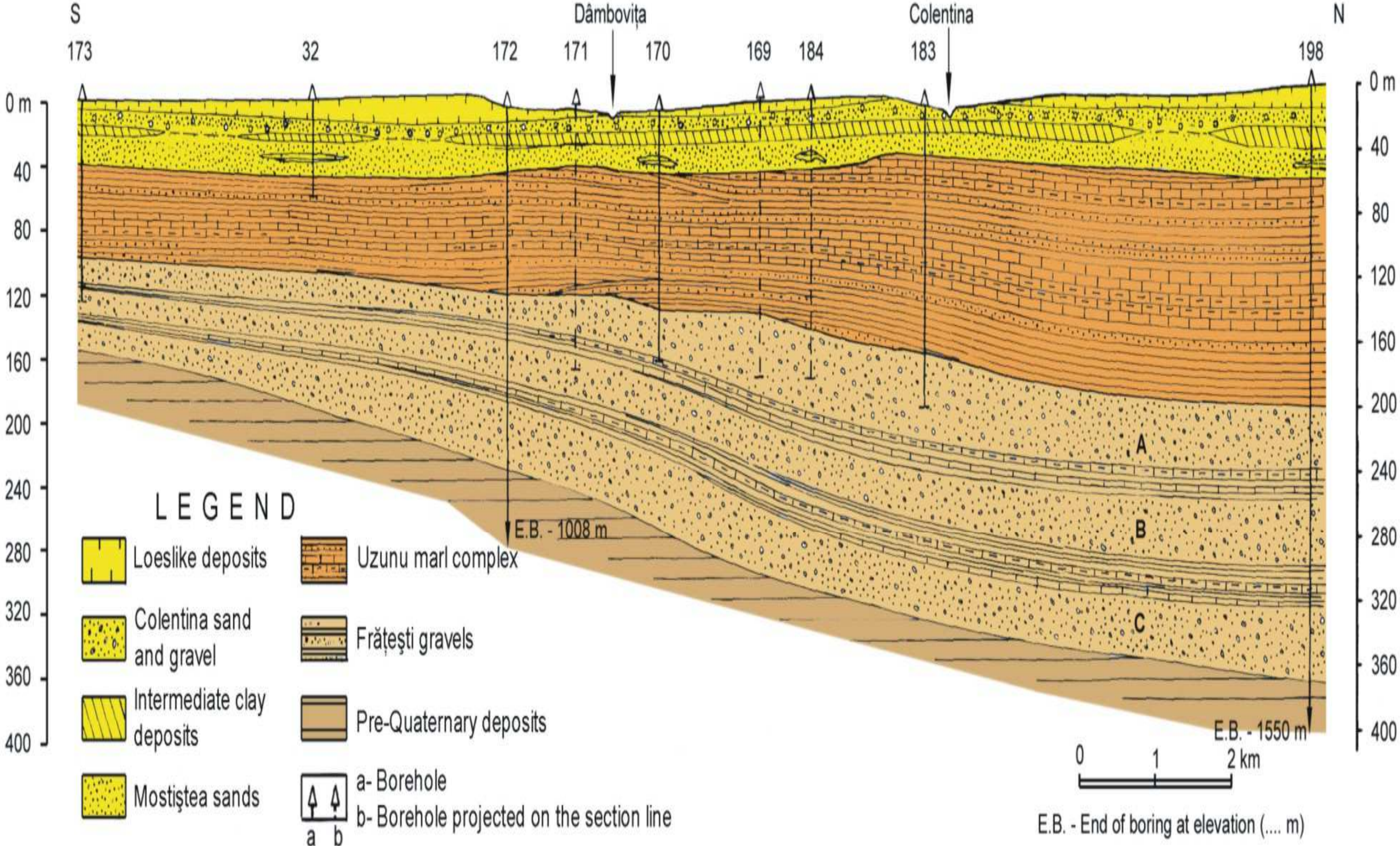
(after E. Liteanu and C. Ghenea, 1969, with modifications)



LEGEND

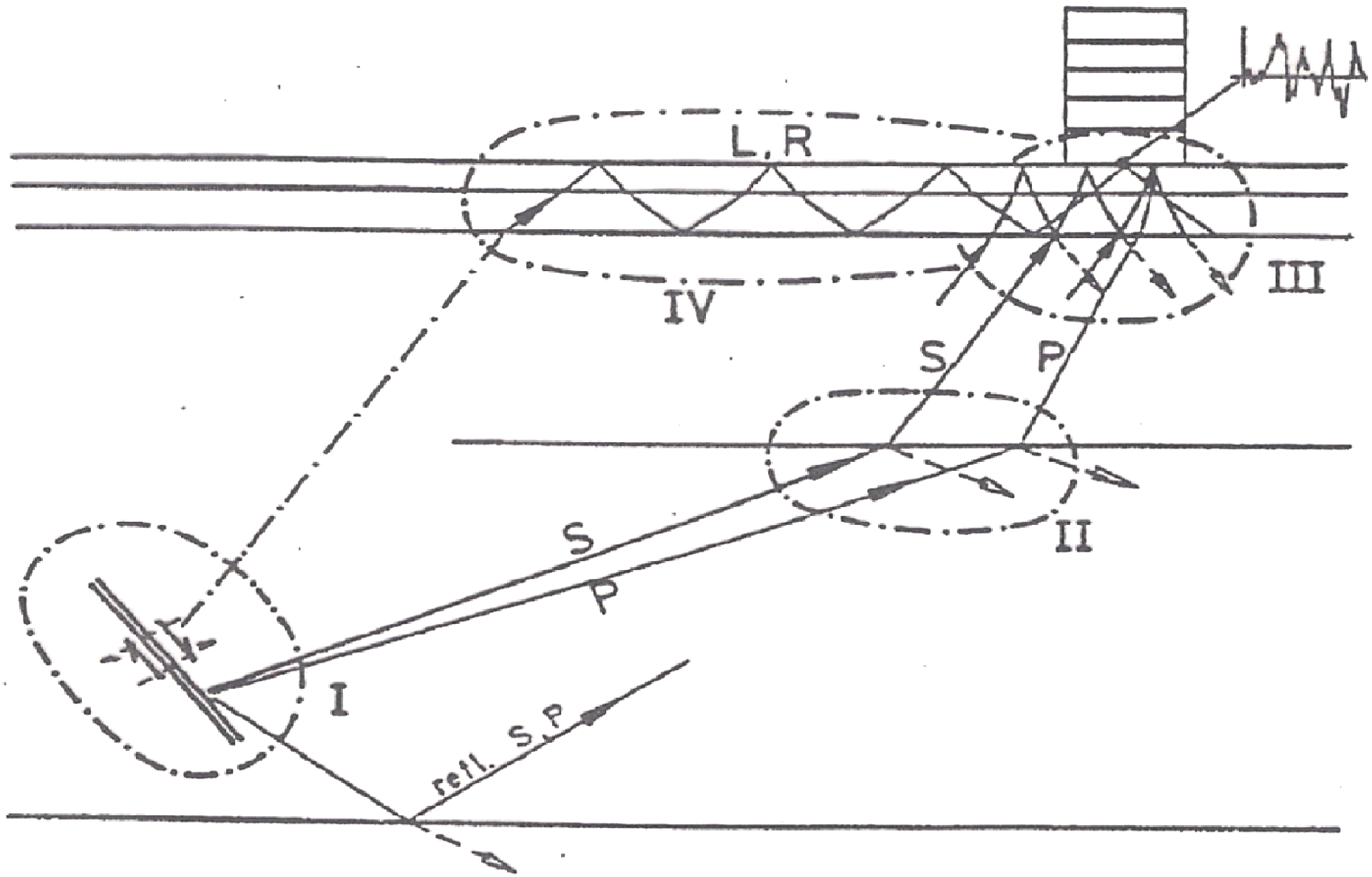
k- Cretaceous; pm- Pontian; dc- Dacian ; rm- Romanian; lower Pleistocene: vl- Villafranchian (Cândesti layers); gz- Günz (Frătesti layers); middle-upper Pleistocene: md- Mindel (marl complex); rs-Riss (Mostistea sands); wr- Würm (wr_1- low: terace; wr_2- Colentine gravel; wr_3- red clay); wr-h- Würm-Holocene (löesslike deposits); h- Holocene- alluvium deposits.

Geological cross-section in the eastern part of the Romanian Plain (NE - SW) (Vrancea-Ploiesti-Bucharest- Giurgiu-Danube river)



The geological structure under Bucharest. Isobars are generally oriented East-West with slope of 8‰ down from South to North. In the same direction , the thickness of layers becomes larger[7].

To avoid these uncertainties we are coming with a new way. In fact from response spectra we can find all nonlinearities from source to free field for each strong (Vrancea) earthquake.



The seismic model from source to free field !

The authors , in order to make **quantitative evidence** of large nonlinear effects, used /introduced and developed the concept of the nonlinear spectral amplification factor (SAF) as ratio between maximum spectral absolute acceleration (S**a**), relative velocity (S**v**),relative displacement (S**d**) from response spectra for a fraction of critical damping (ζ %) at ***fundamental period or any period*** and peak values of acceleration (a_{max}) , velocity (v_{max}) and displacement (d_{max}) , respectively, from processed strong motion records, that are:(SAF)a= **Sa**/a_{max} ; (SAF)v= **Sv**/ v_{max} ; (SAF)d= **Sd**/d_{max} **,**where: $a_{max} = \ddot{y}(t)_{max}$; $v_{max} = \dot{x}(t)_{max}$ and $d_{max} = x(t)_{max}$**[5]**

The concept was used also for last Stress Test asked by IAEA Vienna for Romanian Cernavoda Nuclear Power Plant, where we recorded last three deep strong Vrancea earthquakes: August 30,1986 (M_W = 7.1),May 30(M_W = 6.9) and May 31,1990 (M_W = 6.4).

Table 1.Bucharest-INCERC Seismic Station(E-W Comp.): $\Phi^0 = 44.442$; $\lambda^0 = 26.105$

Earthquake	$a_{max}(cm/s^2)$ (recorded)	S_a^{max} (β=5%)	S_a^{max}/a_{max} (SAF	c	$S_a^*(g)$ (β=5%)	a^*	%
04.03,1977	188,4	440 cm/s^2	2.33	1,214	1025.2	228.7	**21.4%**
08.30,1986	109.1	249 cm/s^2	2.28	1.241	309.0	135.4	**24.1%**
05.30,1990	98,9	280 cm/s^2	2.83	1.000	280.0	98.9	-

Table 2.Bucharest-INCERC Seismic Station(N-S Comp.): $\Phi^0 = 44.442$; $\lambda^0 = 26.105$.

Earthquake	$a_{max}(cm/s^2)$ (recorded)	S_a^{max} (β=5%)	S_a^{max}/a_{max} (SAF)	c	$S_a^*(g)$ (β=5%)	a^*	%
04.03,1977	206,90	650 cm/s^2	3.14	1,322	859.3	273.5	**32.2%**
08.30,1986	96.96	255 cm/s^2	2.62	1.583	403.6	153.4	**58.3%**
05.30,1990	66,21	275 cm/s^2	4.15	1.000	275.0	66.2	-

Table 3.Bucharest-Balta Albă Seismic Station(E-W Comp.): $\Phi^0 = 44.413$; $\lambda^0 = 26.169$

Earthquake	$a_{max}(cm/s^2)$ (recorded)	S_a^{max} (β=5%)	S_a^{max}/a_{max} (SAF)	c	$S_a^*(g)$ (β=5%)	a^*	%
08.30,1986	89.08	345 cm/s^2	3.87	1,217	419.86	104.41	**21.7%**
05.30,1990	63.13	270 cm/s^2	4.27	1.103	297.81	69.63	**10.3%**
05.31,1990	15.90	75 cm/s^2	4.71	1.000	75.00	15.90	-

Table 4.Bucharest-Bolintinu Vale Seismic Station(N155E Comp.):Φ^0 =44.444;λ^0=25.757

Earthquake	a_{max} (recorded)	S_a^{max} (β=5%)	S_a^{max}/a_{max} (SAF)	c	$S_a^*(g)$ (β=5%)	$a^*(g)$	%
08.30,1986	83.7 cm/s^2	295 cm/s^2	3.52	1,235	364.3	103.3	**23.5%**
05.30,1990	215.0 cm/s^2	800 cm/s^2	3.72	1.169	935,2	251.3	**16.9%**
05.31,1990	35.6 cm/s^2	155 cm/s^2	4.35	1.000	155.0	35.6	-

Table 5.Bucharest- Brăneşti Seismic Station(N107W Comp.): Φ^0 =44.460; λ^0=26.329

Earthquake	a_{max}(cm/s^2) (recorded)	S_a^{max} (β=5%)	S_a^{max}/a_{max} (SAF)	c	$S_a^*(g)$ (β=5%)	a^*	%
08.30,1986	89.08cm/ss	345 cm/s^2	3.87	1,217	419.86	104.4	**21.%**
05.30,1990	63.13cm/ss	270 cm/s^2	4.27	1.103	297.81	69.6	**10.%**
05.31,1990	15.90cm/ss	75 cm/s^2	4.71	1.000	75.00	15.9	-

Table 6.Bucharest-Metalurgiei Seismic Station(N127W Comp.): Φ^0 =44.376; λ^0=26.119

Earthquake	a_{max}(cm/s^2) (recorded)	S_a^{max} (β=5%)	S_a^{max}/a_{max} (SAF)	c	$S_a^*(g)$ (β=5%)	a^*	%
08.30,1986	71.07cm/s^2	220 cm/s^2	3.06	1,483	326.26	105,39	**48.3%**
05.30,1990	55.4 cm/s^2	220 cm/s^2	3.97	1.143	251.46	63,32	**14.3%**
05.31,1990	12.1cm/s^2	55 cm/s^2	4.54	1.000	55.00	12.10	-

Table 7.Bucharest-Panduri Seismic Station(N131E Component): Φ^0 =44.426; λ^0=26.065

Earthquake	$a_{max}(cm/s^2)$ (recorded)	S_a^{max} (β=5%)	S_a^{max}/a_{max} (SAF)	c	S_a^*(g) (β=5%)	a^*	%
08.30,1986	89.4cm/s^2	295 cm/s^2	3.29	1,513	446.33	135.26	**51.3%**
05.30,1990	131.3cm/s^2	590 cm/s^2	4.49	1.109	654.31	145.61	**10.9%**
05.31,1990	33.0 cm/s^2	160 cm/s^2	4.98	1.000	160.00	33.00	-

Table 8.Bucharest-Titulescu Seismic Station(N145W Component): Φ^0 =44.452; λ^0=26.080

Earthquake	$a_{max}(cm/s^2)$ (recorded)	S_a^{max} (β=5%)	S_a^{max}/a_{max} (SAF)	c	S_a^*(g) (β=5%)	a^*	%
08.30,1986	87.54	395 cm/s^2	4.51	1,142	451.09	99.97	**14.2%**
05.30,1990	56.80	210 cm/s^2	3.69	1.395	292,95	78.91	**39.5%**
05.31,1990	10.67	55 cm/s^2	5.15	1.000	55.00	10.67	-

Table 9.Bucharest-Carlton Seismic Station(N75E Comp.): Φ^0 =44.436; λ^0=26.102

Earthquake	$a_{max}(cm/s^2)$ (recorded)	S_a^{max} (β=5%)	S_a^{max}/a_{max} (SAF)	c	S_a^*(g) (β=5%)	a^*	%
08.30,1986	79.60	240 cm/s^2	3.015	1,276	306.24	101.64	**27.6%**
05.30,1990	114.7	305 cm/s^2	2.659	1.447	210.78	165.97	**44.7%**
05.31,1990	19.48	75 cm/s^2	3.850	1.000	75.00	19.48	-

Table 10.Galaţi-IPJ(GLT2)Seismic Station(N97WE Comp.):Φ^0 =45.430; λ^0=28.058

Earthquake	a_{max}(cm/s^2) (recorded)	S_a^{max} (β=5%)	S_a^{max}/a_{max} (SAF)	c	S_a^*(g) (β=5%)	a^*	%
08.30,1986	69.10	220 cm/s^2	3.183	1,334	293.48	92.17	**33.4%**
05.30,1990	74.23	250 cm/s^2	3.368	1.260	315.00	93.53	**26.0%**
05.31,1990	47.11	200 cm/s^2	4.245	1.000	200.00	47.11	-

Tabel 11.Iaşi-Centru(IAS2)Seismic Station(N-S Comp.):Φ^0 =47.160; λ^0=27.570

Earthquake	a_{max}(cm/s^2) (recorded)	S_a^{max} (β=5%)	S_a^{max}/a_{max} (SAF)	c	S_a^*(g) (β=5%)	a^*	%
08.30,1986	64.10	190cm/s^2	2.964	1.363	563.16	87.36	36.3%
05.30,1990	109.5	390cm/s^2	3.561	1.135	442.65	124.28	13.5%
05.31,1990	45.76	185cm/s^2	4.042	1.000	185.00	45.76	-

Table 12.Iaşi-Copou(IAS2)Seismic Station(N-S Comp.):Φ^0 =47.193; λ^0=27.562

Earthquake	a_{max}(cm/s^2) (recorded)	S_a^{max} (β=5%)	S_a^{max}/a_{max} (SAF)	c	S_a^*(g) (β=5%)	a^*	%
08.30,1986	68.18	225 cm/s^2	3.300	1.293	290.92	88.15	**29.3%**
05.30,1990	97.22	395 cm/s^2	4.063	1.050	414.75	102,08	**13.5%**
05.31,1990	49.44	211 cm/s^2	4.267	1.000	211.00	49.44	-

Table 13.Bucharest-Măgurele Seismic Station(E-W Comp.):Φ^0 =47.347; λ^0=26.030

Earthquake	$a_{max}(cm/s^2)$ (recorded)	S_a^{max} (β=5%)	S_a^{max}/a_{max} (SAF)	c	$S_a^*(g)$ (β=5%)	a^*	%
08.30,1986	113.80	307 cm/s^2	2.6982	1.329	408.6	151.46	**32.9%**
05.30,1990	90.25	324 cm/s^2	3.5869	1.000	324.0	90.25	**-**

Table **14.Ploieşti-(PLS)Seismic Station(N100E Comp.):Φ^0 =44.930; λ^0=26.020**

Earthquake	$a_{max}(cm/s^2)$ (recorded)	S_a^{max} (β=5%)	S_a^{max}/a_{max} (SAF)	c	$S_a^*(g)$ (β=5%)	a^*	%
08.30,1986	207.2	730 cm/s^2	3.523	1.124	820.5	232.89	**12.4%**
05.30,1990	72.6	235 cm/s^2	3.236	1.224	287.6	88.86	**22.4%**
05.31,1990	16.4	65 cm/s^2	3.963	1.000	65.00	16.40	**-**

Table 15.Bacău-(BAC2)Seismic Station(E-W Comp.):Φ^0 =46.567; λ^0=26.900

Earthquake	$a_{max}(cm/s^2)$ (recorded)	S_a^{max} (β=5%)	S_a^{max}/a_{max} (SAF)	c	$S_a^*(g)$ (β=5%)	a^*	%
08.30,1986	72.20	292 cm/s^2	4.0443	1.457	425.44	105.19	**45.7%**
05.30,1990	132.43	684 cm/s^2	5.1649	1.141	780.44	151.10	**24.1%**
05.31,1990	63.07	372 cm/s^2	5.8942	1.000	372.00	63.07	**-**

Table 16.Cernavoda -(CVD2)Seismic Station(E-W Comp.):Φ^0 =44.340; λ^0=28.030

Earthquake	$a_{max}(cm/s^2)$ (recorded)	S_a^{max} (β=5%)	S_a^{max}/a_{max} (SAF)	c	$S_a^*(g)$ (β=5%)	a^*	%
08.30,1986	62.78	256 cm/s^2	4.0777	1.420	363.52	89.14	**42.0%**
05.30,1990	100.06	475 cm/s^2	4.7471	1.219	579.02	121.97	**21.9%**
05.31,1990	49.73	288 cm/s^2	5.7912	1.000	288.00	49.73	**-**

Table 17.Craiova-(CRV) Seismic Station (N05E Comp.):Φ^0 =47.321; λ^0=23.798

Earthquake	$a_{max}(cm/s^2)$ (recorded)	S_a^{max} (β=5%)	S_a^{max}/a_{max} (SAF)	c	$S_a^*(g)$ (β=5%)	a^*	%
08.30,1986	140.70	690 cm/s^2	4.9040	1.1435	789.01	160.89	**14.4%**
05.30,1990	62.41	350 cm/s^2	5.6080	1.000	350.00	62.41	**-**

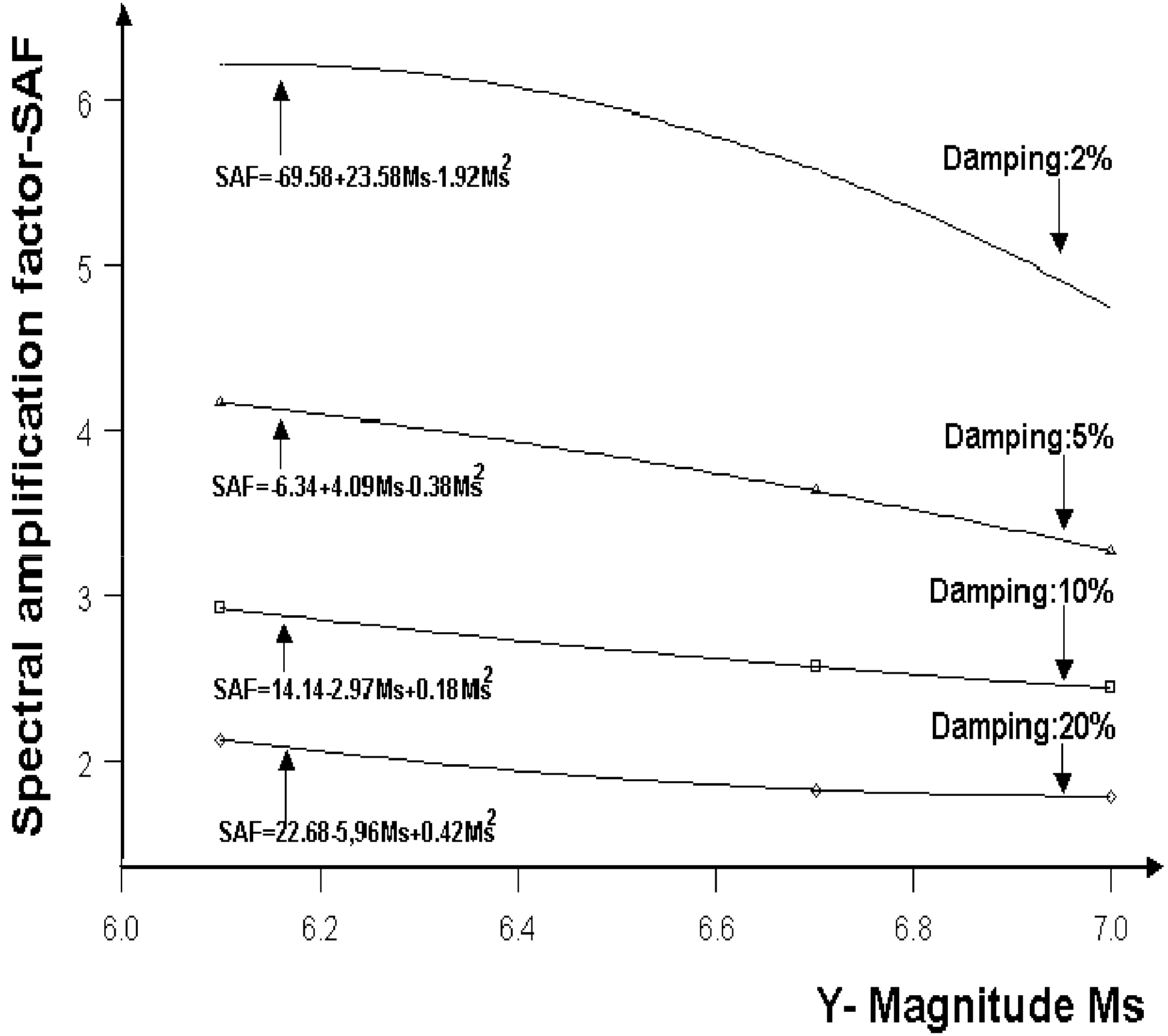
Table 18.Râmnicu Sărat -(RMS2)Seismic Station(N55E Comp.):Φ^0 =45.380; λ^0=27.040

Earthquake	$a_{max}(cm/s^2)$ (recorded)	S_a^{max} (β=5%)	S_a^{max}/a_{max} (SAF)	**c**	$S_a^*(g)$ (β=5%)	a^*	%
08.30,1986	140.3	400 cm/s^2	2.8510	1.215	486.0	170.46	**21.5%**
05.31,1990	66.4	230 cm/s^2	3.4638	1.000	230.0	66.40	**-**

At the same seismic station, for example at *Bucharest-Panduri Seismic Station* (*Table* 7) and Figure 3, close to borehole 172, for horizontal components and β=5% damping, the values of the SAF for accelerations are: 3.29 for August 30,1986 Vrancea earthquake (M_W=7.1); 4.49 for May 30, 1990 (M_W=6.9) and 4.98 for May 31, 1990 (M_W =6.4). Vrancea earthquake on May 31,1990 (M_W=6.4) could be assumed that the response is still in elastic domain and then we have the possibility to compare to it. In R.G. 1.60 ,SAF= 3.13 and is constant at all…

Table 19. Median values of (SAF) for last three strong Vrancea earthquakes

Damping	August 30, 1986 (M_S=7.0; M_w=7.1)		May 30,1990 (M_S=6.7 ; M_w =6.9)		May 31,1990 (M_S=6.2; M_w =6.4)	
ξ%	S_a^{max}/a_{max}	S_v^{max}/v_{max}	S_a^{max}/a_{max}	S_v^{max}/v_{max}	S_a^{max}/a_{max}	S_v^{max}/v_{mav}
2%	4.74	3.61	5.58	3.72	6.22	4.84
5%	**3.26[3.13]**	2.69	**3.63[3.13]**	2,95	**4.16[3.13]**	3.48
10%	2.43	1.99	2.56	2,14	2.92	2.69
20%	1.78	1.50	1.82	1,58	2.13	1.86



On the other hand, from Tables 1-19 and Figure 4 we can see that there is a strong *nonlinear* dependence of the spectral amplification factors(SAF) on earthquake magnitude for other seismic stations on Romanian territory on extra-Carpathian area (Iasi, Bacau, Focsani, Bucharest-NIEP, NPP Cernavoda, Bucharest-INCERC etc.).

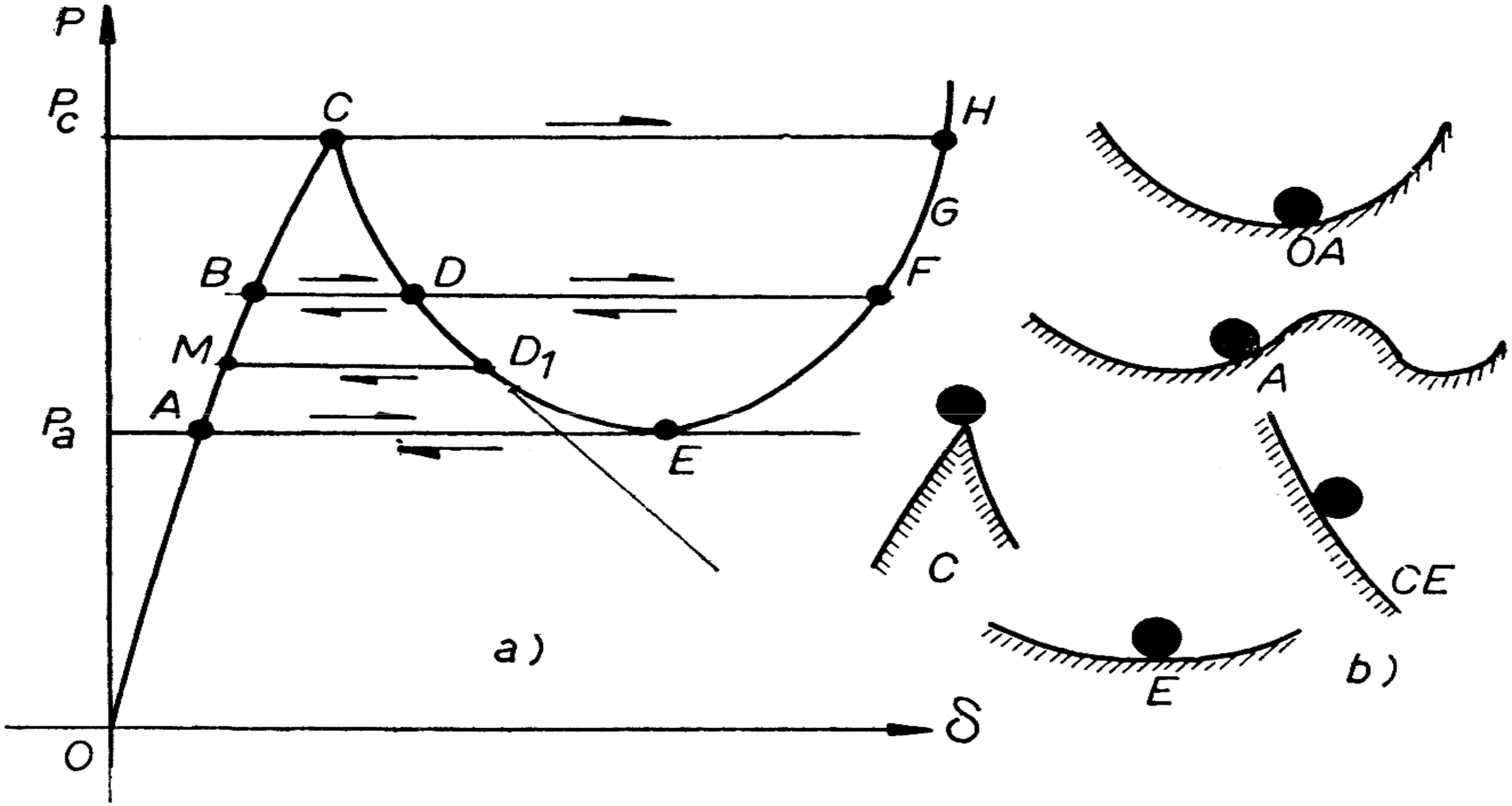
SAF=3.13

(Regulatory Guide 1.60 of the US Atomic Commission) & IAEA

Stability theory is playing a central role in systems theory and engineering. There are different kinds of stability problems that arise in the study of dynamical systems. Stability of equilibrium points is usually characterized in the sense of Lyapunov, a Russian mathematician and engineer who laid the foundation of the theory.

"An equilibrium point is stable if all solutions starting at nearby points stay nearby, otherwise is unstable". Near the equilibrium point, the quadratic and higher order terms are much smaller than the linear terms, and so they can be neglected.

The loss of stability of any structural system occurs under certain characteristic circumstances always following a dynamic process.



A diagram load (P)-displacement (δ) for a certain system[4]

Let y(x ,t)=the prevailing deflection of the <u>structure</u> ;x=the vector of the spatial coordinates ; and t=the time, then the motion of the structure is given mathematically by the differential equation:

$$m\ddot{y} + Ey + \lambda P(x,t,y) + D\acute{y} = F(,t) \qquad (1)$$

and by initial boundary conditions.E,P and D=linear differential operators with respect to x;F= afunction of the given arguments;m=the mass density;λ=a load parameter. Operators P and D may have various possible forms depending on load and damping, and operator E is once and for all defined by the fact that the structure is supposed to be elastic.

In the sense of Lyapunov's theory of the stability of motion, a perturbated motion,y +u,is being considered and its deviation from the unperturbated motion, y, is studied…

For the purpose ,y+u is being substituted into Eq.(1) and the boundary conditions in place for y.Eq.(1) become:

$$m\ddot{u} + Eu + \lambda\, P(x,t,u) + D\acute{u} = 0 \qquad \textbf{(2a)}$$

$$[Uu]_B = 0 \qquad \textbf{(2b)}$$

results for the perturbations, u, i.e., an "equation of variation"(Eq.21),and boundary conditions(Eq.2b).These equations are all homogeneous equations,they represent a boundary- eigenvalue problem, the load parameter,λ,being one of the eigenvalues.

The particular solution of Eqs.2 that coresponds to the steady-state response is:

$$u(x,t) = e^{i\omega t}\,\psi(x) \qquad \textbf{(3)}$$

In which i=the imaginary unit;w=the frequency of the steady-state motion;and ψ(x)=the mode form of this vibration.Using Eq.3 in Eq.2a,Eq.2b yields:

Let y(x ,t)=the prevailing deflection of the <u>structure</u> ;x=the vector of the spatial coordinates ; and t=the time, then the motion of the structure is given mathematically by the differential equation:

$$m\ddot{y} + Ey + \lambda P(x,t,y) + D\acute{y} = F(,t) \qquad (1)$$

and by initial boundary conditions.E,P and D=linear differential operators with respect to x;F= afunction of the given arguments;m=the mass density;λ=a load parameter. Operators P and D may have various possible forms depending on load and damping, and operator E is once and for all defined by the fact that the structure is supposed to be elastic.

In the sense of Lyapunov's theory of the stability of motion, a perturbated motion,y +u,is being considered and its deviation from the unperturbated motion, y, is studied…

$$-\mu\omega^2\psi + E\psi + \lambda P(x,t,\psi) + i\omega\, Du = 0 \qquad (4a)$$

$$[Uu]_B = 0 \qquad (4b)$$

and this formulation reveals clearly the dynamic nature of any stability investigation, as the limit of stability is determined by the behavior of two eigenvalues,i.e.,ω and λ, the first being a frequency and the second a load parameter.

If the damping is not considered,Eq.4a involving operator D vanishes.Consequenrly,the frequency equation becomes:

$$F(\omega^2,\lambda) = 0 \qquad (5)$$

which is a relationship between ω^2 and λ.In a corresponding ω^2,λ space and Eq.5 represents a curve (or a hypersurface , if several parameters are being considered). Its projection on the frequency axis, together

with the points of intersection with this axis yield the limit of stability of the system.

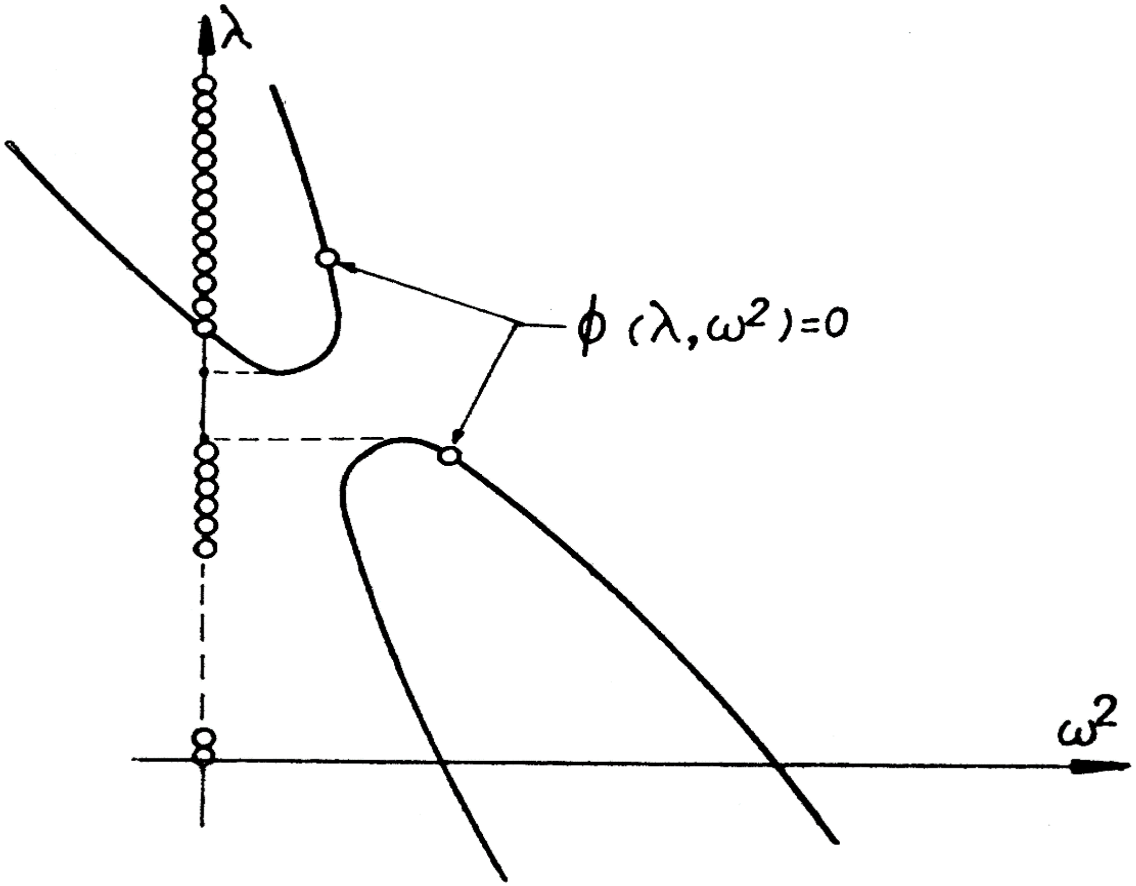
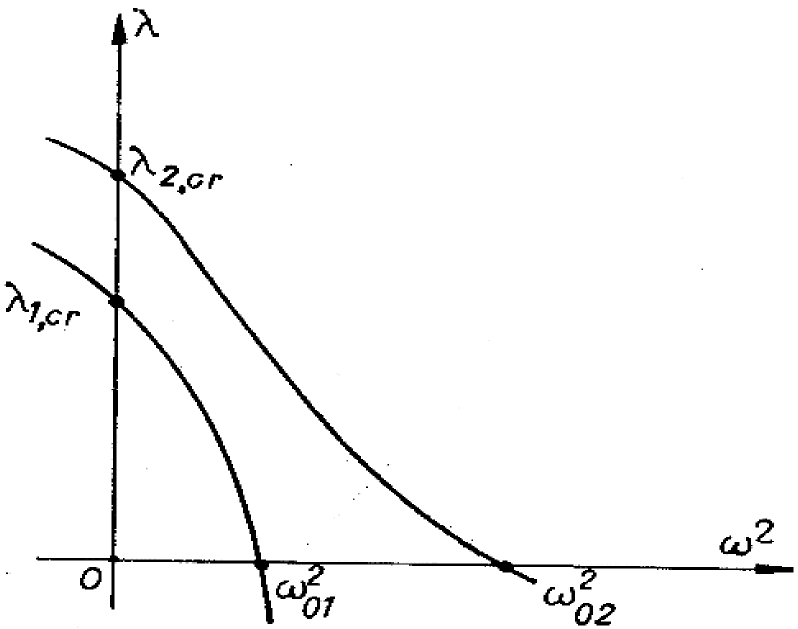
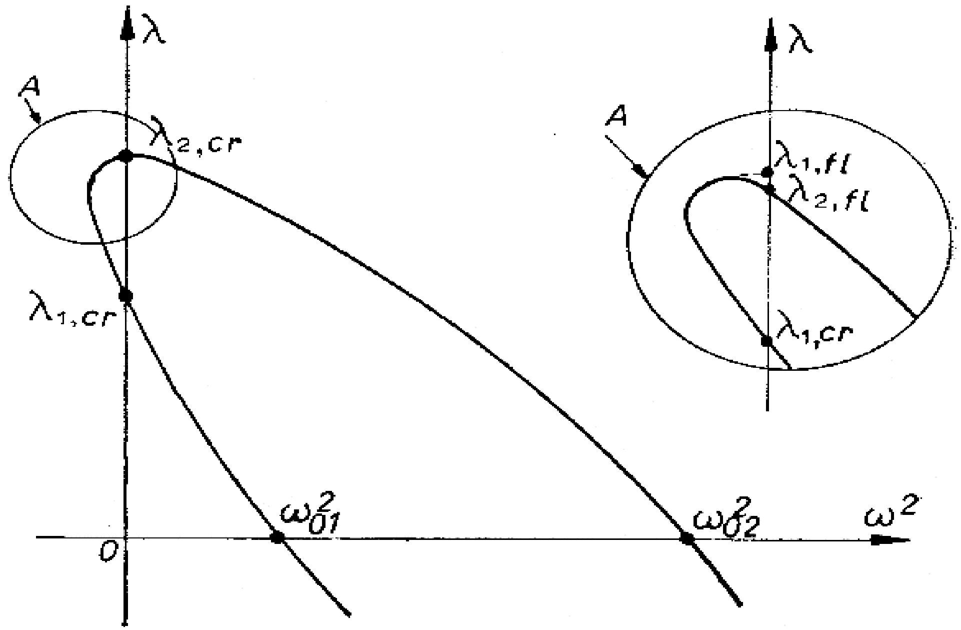


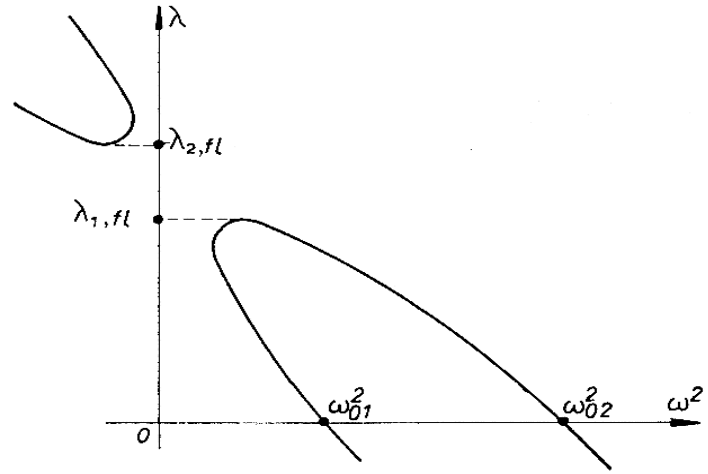
Fig.1.Relationship between λ and ω^2 [4]

Considering again an un-damped structure, and therefore, working with Eq.5,frequency-load curves (eigenvalue curves) will be found that, in principle, belong to one of three classes shown in the following Figures. In the case of a divergence type structure[Fig.2(a)],the branches of the eingenvalue curve intersect the load axis ,λ,thus yielding the critical loads,$\lambda_{i,cr}$,i=1,2,… In this case ,the transition of the structure from stability to instability occurs at ω^2=0,i.e.,in passing through a nontrivial equilibrium position .In the other two cases ,i.e.,a hybrid type structure [Fig.2(b)] and a flutter type structure[Fig.2(c)], the so-called flutter loads,$\lambda_{i,n}$,i=1,2,…,may cause or will cause instability at ω^2 values that are different from zero, i.e., the instability of the system consists in vibrations with unboundedly increasing amplitudes





a).Divergent type of structure; b). Hybrid type of structure



c).Flutter type structure[4]

The differential equation describing many nonlinear oscillators can be written in the form:

$$d^2x/d^2t + f(x, dx/dt)=0 \qquad (1)$$

A convenient way to treat eq.(1) is to rewrite it as a system of two first order o.d.e.'s:

$$dx/dt=y \;,\quad dy/dt=-f(x,y) \qquad (2)$$

and eqs.(2) may be generalized in the form:

$$dx/dt =F(x,y) \;,\quad dy/dt =G(x,y) \qquad (3)$$

A point which satisfies *F(x,y)* =0 and *G(x,y)* =0 is called an *equilibrium point* and a solution to (3) may be pictured as a curve in the x-y *phase plane* passing through the point of initial conditions (x_0 , y_0)…

Structural Stability

If an equilibrium point is hyperbolic, then we say that the linear variational equations correctly represent the nonlinear system locally, as far as Lyapunov stability goes. When we discuss about structural stability ,we are concern about the relationship between the dynamics of a given dynamical system, say for example eqs.(3),and the dynamics of a neighboring system, for example:

$$dx/dt = F(x,y) + \varepsilon F1(x,y) \; ;$$

$$dy/dt = G(x,y) + \varepsilon G1(x,y)$$

where ε is a small quantity and where F1 and G1 are continuous. A system S is said to be structurally stable if all nearby systems are topologically equivalent to S. Specifically ,eqs(3) are structurally stable if there exist homeomorphism taking motions of (3) to motions of (14) for some ε.

Note the similarity between Lyapunov stability and structural stability : Both involve a given dynamical object ,and both are concerned with the effects of a perturbation off of that object.

Note the similarity between Lyapunov stability and structural stability :Both involve a given dynamical object, and both are concerned with the effects of a perturbation off of that object.

On the other hand, a point is said to be *wandering* if it has some neighborhood which leaves and never (as $t \to \infty$) returns to intersect its original position…

Now, the problem of nonlinear damping developed in any system during of strong earthquakes…What is happened during of strong earthquake in the vicinity/ neighbourhood of resonant frequency of the system ?

If we consider the system

$$dx/dt=f(t,x)+G(t,x)[u+\delta[t,x,u] \qquad (1)$$

where x€R is the state and u€ R is the control input .The functions F,G, and δ are defined for (t ,x ,u) € [0,∞) x S x R ,where D is included in R and is a domain that contains the origin .Also f ,G , δ are piecewise continuous in t and locally Lipschitz in „x” and „u”.

The functions f and G are known precisely ,while the function δ is an unknown function that lumps together various uncertain terms due to modal simplification, parameter uncertainty, and so on. The uncertain term δ satisfies the matching condition . A nominal model of the system can be taken as:

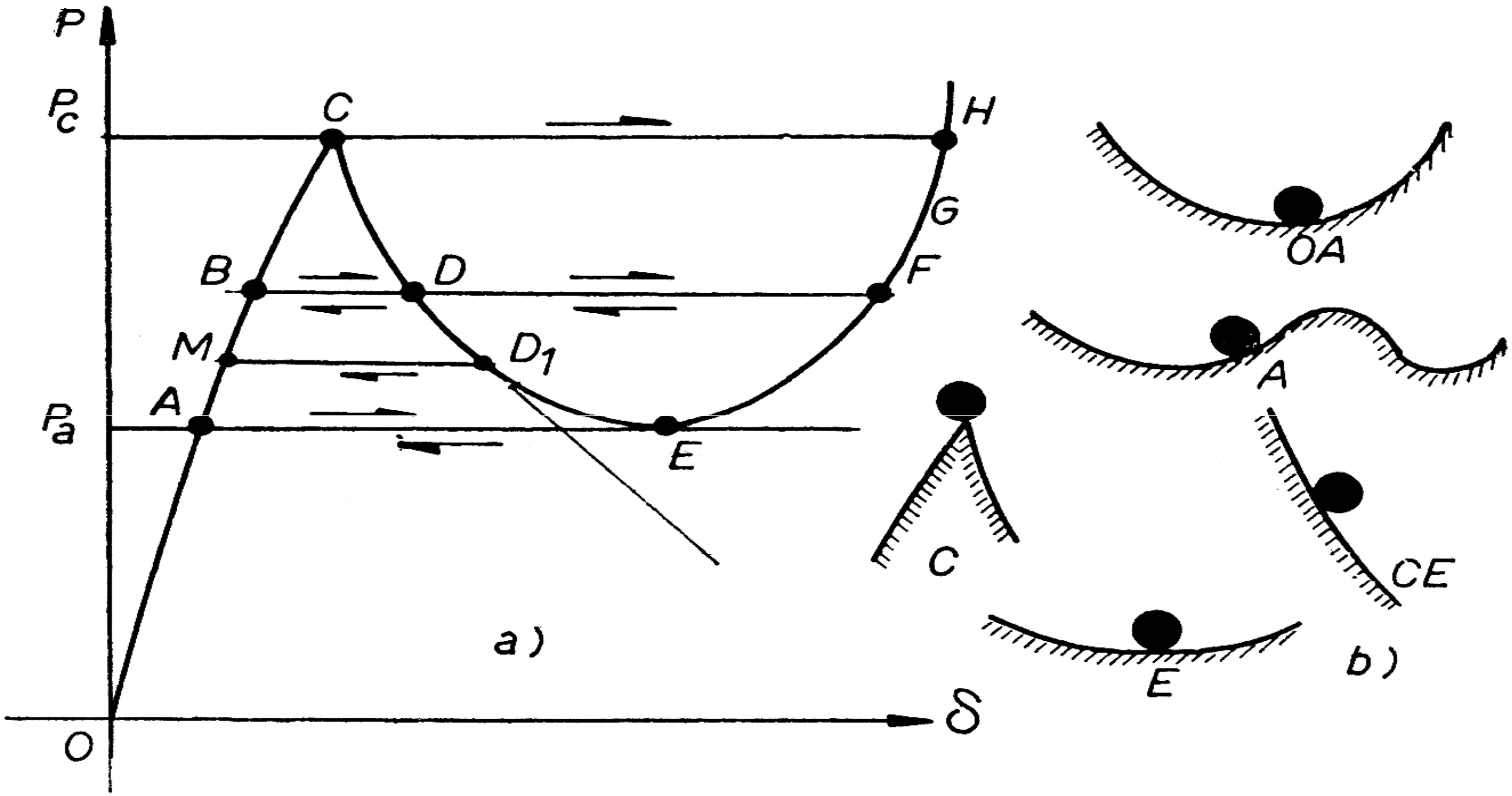
$$dx/dt = f(t,x) + G(t,x)\,u \quad (2)$$

We proceed to design a stabilizing state feedback controller by using this nominal model .Suppose we have succeeded to design a feedback control law u=ψ (t ,x) such that the origin of the nominal closed-loop system

$$dx/dt = f(t,x) + G(t,x)\psi(t,x) \quad (3)$$

is uniformly asymptotically stable…

In final, we meet at Lyapunov the term "redesign" which is called *nonlinear damping*…,which is found in any test from resonant columns…


P
Pc
Pa
C
H
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a)
OA
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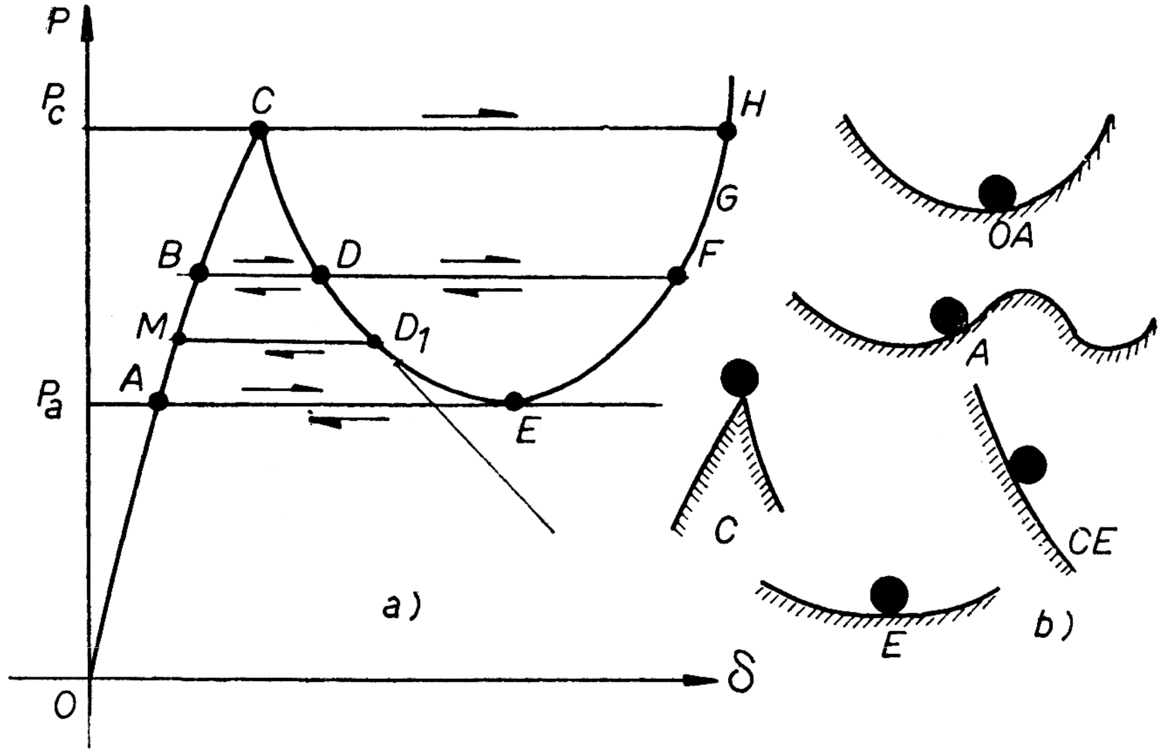

OA-system stable; AC in B-the system is still stable for small perturbations and is going back to original configuration ; for slightly large perturbations the system is going in D and function of ration between load and displacement , position of D cold be stable or unstable. It is possible to go in F. If the load is increasing , we got C , where P=Pcr and the system is strong unstable and usually the system will not remain in C and will go somewhere in G…On portion CE ,the stability of the system is function of many parameters (load , the earthquake magnitude ,the response of the soil and structure etc. In point E , the system is in a neutral equilibrium for small perturbations ,but stable for large one… [4].

On the other hand, nonlinear damping in the soil and structure system during a strong earthquake is playing a prominent part in stability of dynamical systems. The loss of stability of any system occurs under certain characteristic circumstances always following a dynamic process. Stability of equilibrium points is usually characterized in the sense of Lyapunov's theorem..

What is happened in the system near of soil fundamental period/frequency of vibration?

We found that the fundamental period of soil is strong dependence of earthquake magnitude and type of soil. In the sense of Lyapunov's theory of stability of motions during a large earthquake, a perturbed motion and its deviation from the unperturbed motion could be studied now and to consider the Lyapunov redesign ,called nonlinear damping etc.

CONCLUSIONS:

- **The central question of the discussion was in last time whether soil amplification is function of earthquake amplitude dependent. The dependence of soil response on strain amplitude become a standard assumption in the geotechnical field , in earthquake engineering and engineering seismology.**
- **Laboratory data shows a typical stiffness degradation curve ,in term of G modulus and increasing of damping along with strain levels developed during strong earthquakes. In other words, a variation of dynamic torsion modulus function (G, daN/cm^2) and torsion damping function (G%) of specific shear strain (γ%).**

- **Stress and strain states are not enough to determine the mechanical behavior of soils. It is necessary, in addition, to model the relation between stresses and deformations by using *specific constitutive laws to soils. Currently, there are not constitutive laws to describe all real mechanical behaviors of deformable materials like soils.***
- **Soils, although have many common mechanical properties require the use of different models to describe behavior difference. Soils are simple materials with memory: sands are „*rate-independent*" type and clays are „*rate-dependent* ".**

- **Sands typically have low rheological properties and can be modeled with an acceptable *linear elastic model* and clays which frequently presents significant changes over time can be modeled by a *nonlinear viscoelastic model.***
- **Viscoelastic material behavior could be characterized using Boltzmann's formulation of the constitutive law;**
- **Displacement vector u , the tensors T & E for tension and strain , in case of nonlinear viscoelastic materials, are function of position x and time t, functions that define the *viscoelastic body state…***

From resonant columns: between 0.1 and 10 Hz, dynamic functions G(γ) and D(γ) are constant and functions of shear strains (γ%)…

- **To avoid these uncertainties we are coming with a new way. In fact from response spectra we can find all nonlinearities from source to free field for each strong Vrancea earthquake.**
- The ***quantitative evidence*** of large nonlinear effects, used /introduced and developed the nonlinear spectral amplification factor (SAF) concept as ratio between: (SAF)a= $\mathbf{Sa/a_{max}}$; (SAF)v= $\mathbf{Sv/\ v_{max}}$; (SAF)d= $\mathbf{Sd/d_{max}}$ ***at fundamental periods or at any one***; where: $\mathbf{a_{max}} = \ddot{y}(t)_{max}$; $\mathbf{v_{max}} = \dot{x}(t)_{max}$ and $\mathbf{d_{max}} = x(t)_{max}$

- **From Tables 1-18 and 19 for median values, we can see that there is a strong nonlinear dependence of the spectral amplification factors (SAF) for absolute accelerations on earthquake magnitude for all records made on extra-Carpathian area from Iasi to Craiova for last strong Vrancea earthquakes, inclusively for NPP Cernavoda site;**
- **The amplification factors are decreasing with increasing the magnitudes of deep strong Vrancea earthquakes and this values are far of that given by Regulatory Guide 1.60 of the U. S. Atomic Energy Commission. The spectral amplification factors(SAF) and, in fact, the nonlinearity, are functions of Vrancea earthquake magnitude. The amplification factors decrease as the magnitude increases**

- ***Stability theory is playing a central role in systems theory and engineering. There are different kinds of stability problems that arise in the study of dynamical systems. Stability of equilibrium points is usually characterized in the sense of Lyapunov, a Russian mathematician and engineer who laid the foundation of the theory.***
- ***The loss of stability of any structural system occurs under certain characteristic circumstances always following a dynamic process.***
- **When we discuss about structural stability ,we are concern about the relationship between the dynamics of a given dynamical system and the dynamics of a neighboring system**
- **Note the similarity between Lyapunov stability and structural stability :Both involve a given dynamical object, and both are concerned with the effects of a perturbation off of that object.**
- **A point is said to be *wandering* if it has some neighborhood which leaves and never (as $t \rightarrow \infty$) returns to intersect its original position;**

- **We meet at Lyapunov the term "redesign" which is called *non-linear damping*…,which is found in any test from resonant columns;**
- **Nonlinear damping in the soil and structure system during a strong earthquake is playing a prominent part in stability of dynamical systems. The loss of stability of any system occurs under certain characteristic circumstances always following a dynamic process;**
- ***What is happened in the system near of soil fundamental period/frequency of vibration?***
- **We found that the fundamental period of soil is strong dependence of earthquake magnitude and type of soil. In the sense of Lyapunov's theory of stability of motions during a large earthquake, a perturbed motion and its deviation from the unperturbed motion could be studied now and to consider the Lyapunov redesign ,called nonlinear damping etc.**

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Thank you for your attention !