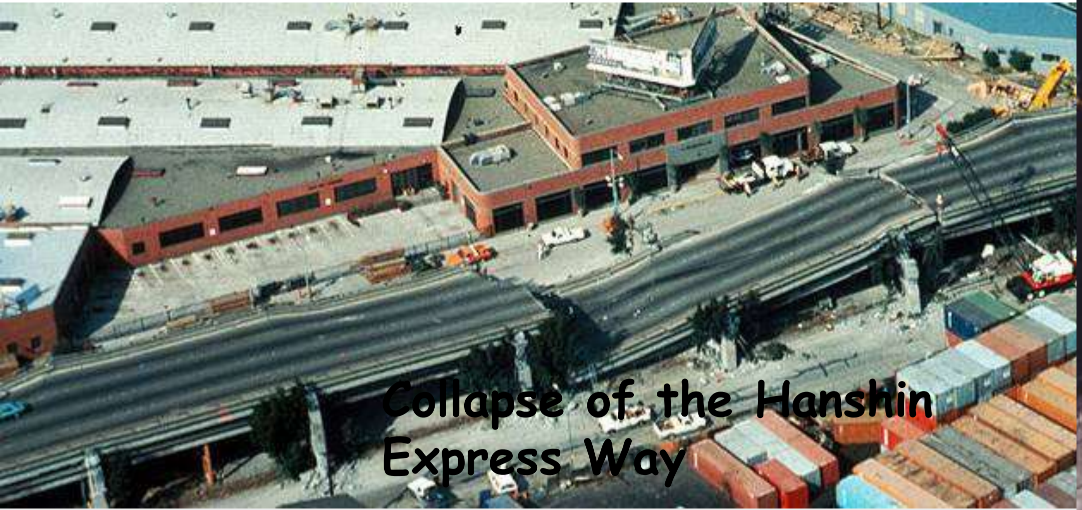
Re-visiting Earthquake Resistant Design of Bridges

G. Michele Calvi

IUSS Pavia and Eucentre Foundation

In a few cases only road networks in general and bridges in particular have been severely and extensively damaged.

- San Fernando (1971)
- Loma Prieta (1989)
- Kobe (1995)



Collapse of the Hanshin Express Way

L'Aquila 2009
Mw 6.3
300 deaths

What is the explanation of this apparent logical inconsistency?

- Is the nature of earthquake ground motion more selective for bridges than for buildings?
- Or does the reason lie in a less uniform distribution of bridge structures over the territory?
- Or rather bridge structures tend be associated to very steep fragility functions, such that the correlation between demand and damage is far from being proportional?

Whatever the answer is, it bears relevant consequences in terms of earthquake resistant design and assessment of bridges.

Roadway and bridge construction in the last century

A brief critical historical note on our heritage

First half of the XX century

Ordinary bridge construction dominated by concrete arch

Ponte dell'Impero
(today della Libertà)
Pavia
Spans of 45 m
Completed 1936

… but,

arch bridge construction continued into the XXI century

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Galena Creek Bridge
Main span 212 m
Completed 2012

Second half of the XX century

Ordinary bridge construction dominated by prestressed concrete

Bridge over the Po River
Pieve P. Morone
Spans ~ 80 m
60'

Second half of the XX century

Ordinary bridge construction dominated by prestressed concrete

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the Po River
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60'

Second half of the XX century

Ordinary bridge construction dominated by prestressed concrete

Bridge over the Po River Casalmaggiore
Spans of 65 m
1955 - 1958

Second half of the XX century

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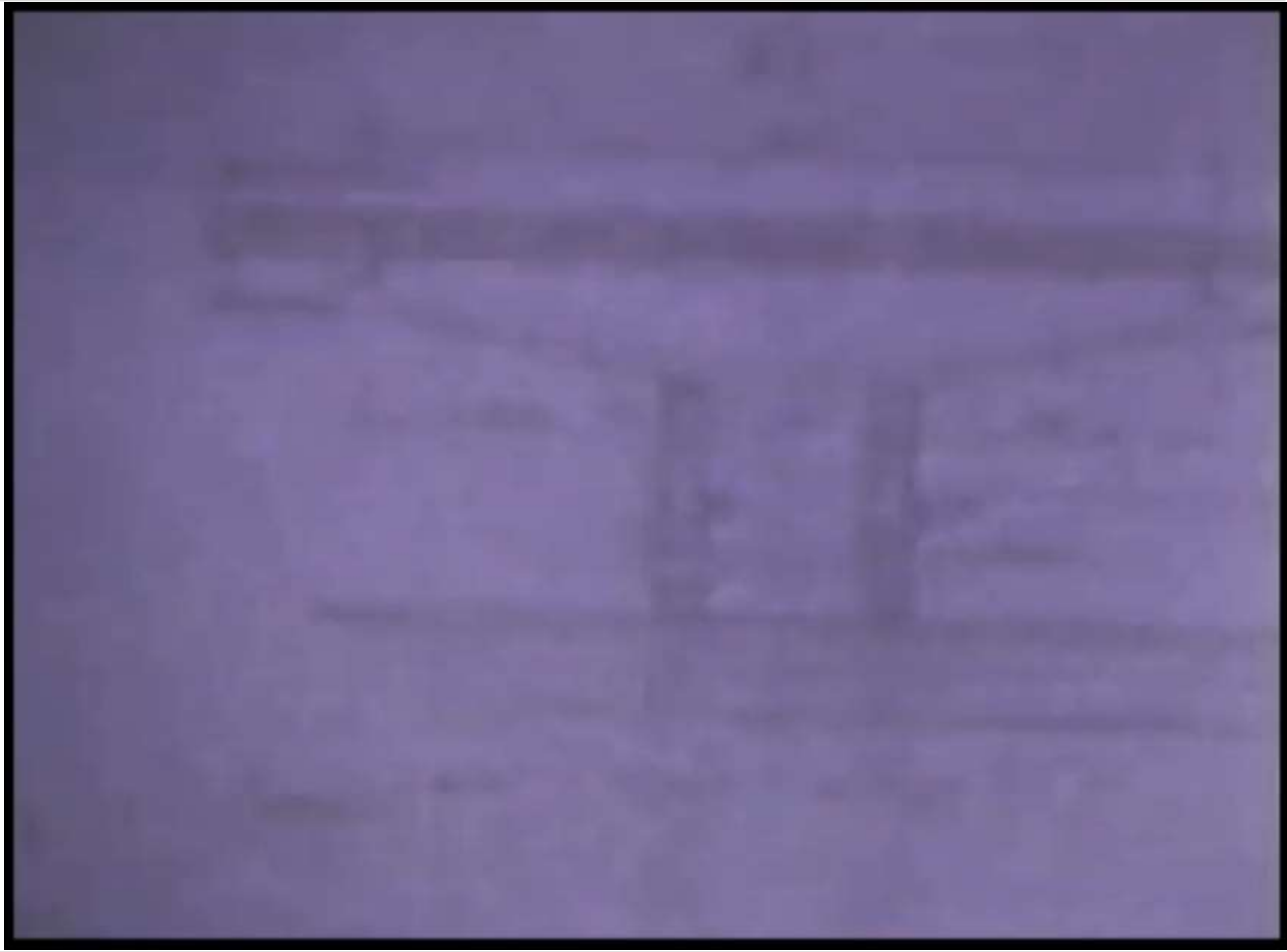
1955 – 1958



Construction of road infrastructure in the 1960s

US

1956 President Eisenhower signs the Federal-Aid Highway Act
the "*Greatest Decade*": 60,000 km, 5,000 km/y until 1960

1961 President Kennedy maintains the project and $ 0.01 gas tax per gallon

1966 29,000 km completed; $ 25 billion spent

Complete project:

- *12,957 interchanges requiring 22,252 individual structures*
- *20,748 other highway grade-separation structures*
- *4,361 railroad grade separations*
- *14,806 other bridges and tunnels*

Construction of road infrastructure in the 1960s

Europe

1920s	First freeways in Italy
1930s	4,000 km in Germany
	Eisenhower: "*Germany had made me see the wisdom of broader ribbons across the land*"
1960s	Most European countries construct the backbone of the modern freeway system
1956-1964	"Autostrada del Sole": 853 bridges, 572 overpasses

2018 The New York Times:

- In France, the highway system comprising 12,000 bridges is in a state of chronic underinvestment, with 7% having damage that could eventually result in collapse if not addressed
- In Germany, of the 39,621 bridges monitored by the Federal Government, 10.6% are in a condition that is not satisfactory and 1.8 % are in "inadequate" condition
- Similar examples reported for other European countries

The Kobe Earthquake 1995
(Hyogoken Nambu, Great Hanshin)

Lessons on bridges

Collapsed bridges in Japan before Kobe

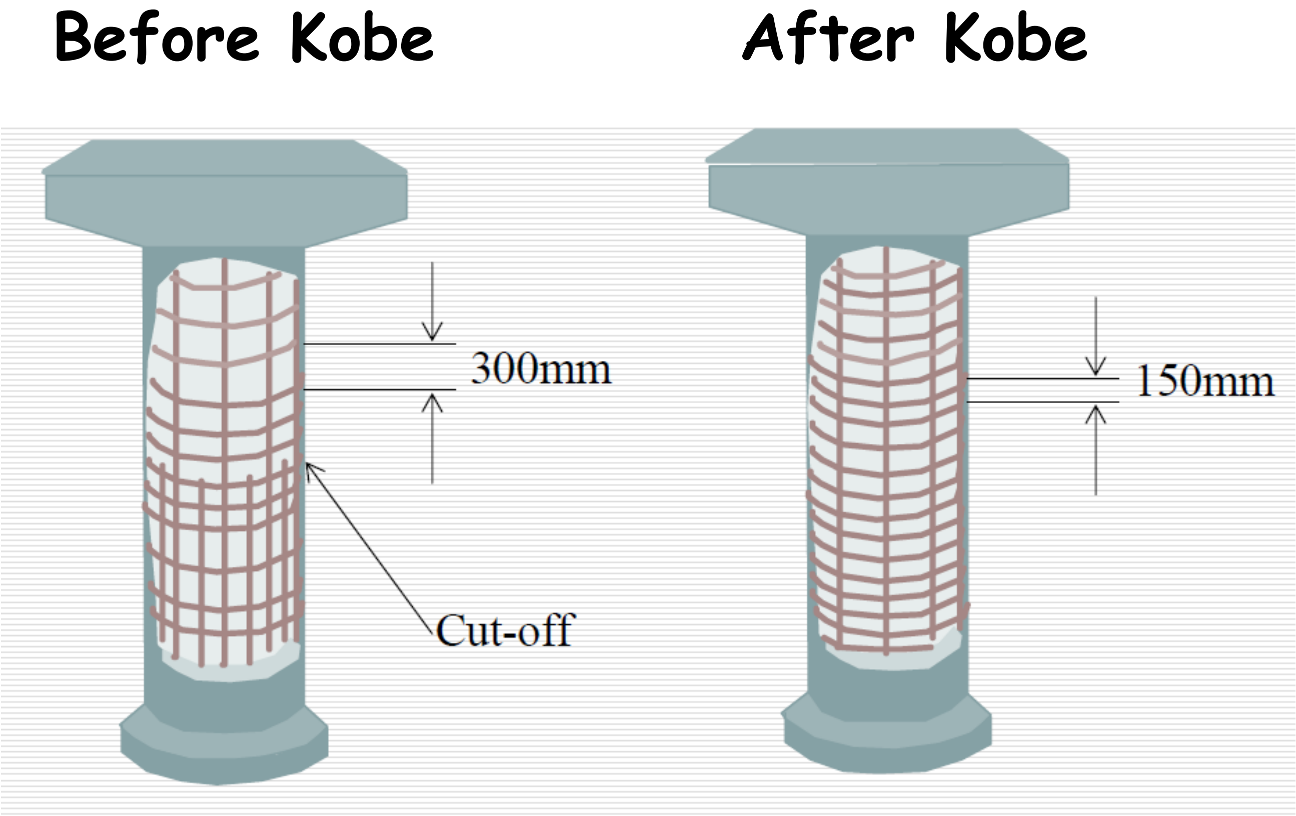
• 1923 Kanto M7.9	6
• 1946 Nankai M8.1	1
• 1948 Fukui M7.3	4
• 1964 Niigata M7.5	3
• 1978 Miyagi-ken-oki 7.4	1

Typical problems:

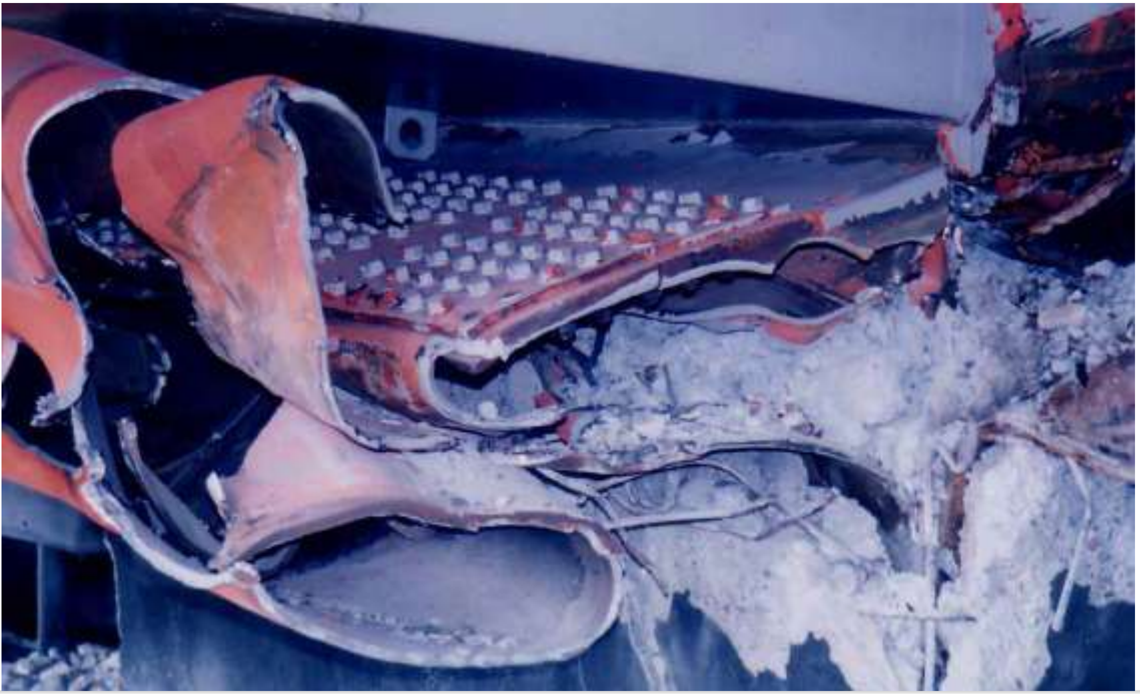
- Foundation
- Liquefaction
- Unseating
- Columns strength

(K. Kawashima)

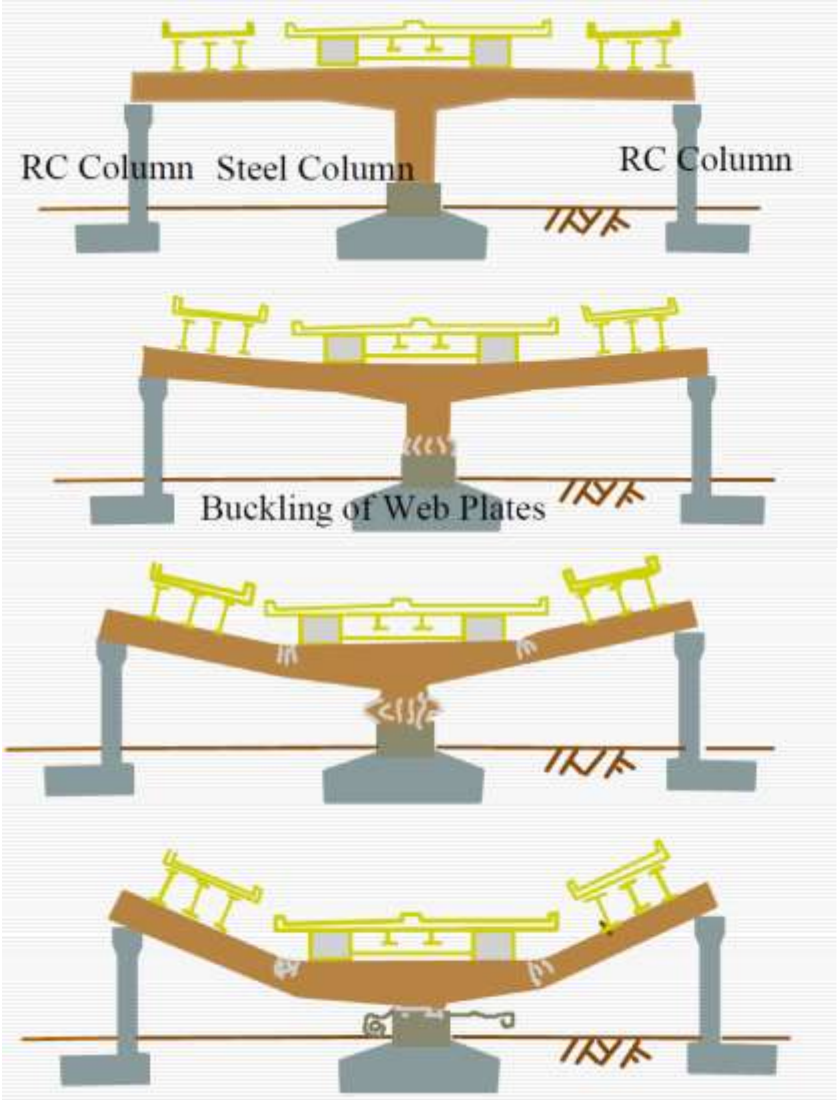
Following Kobe: Shear and lap splices



(K. Kawashima)

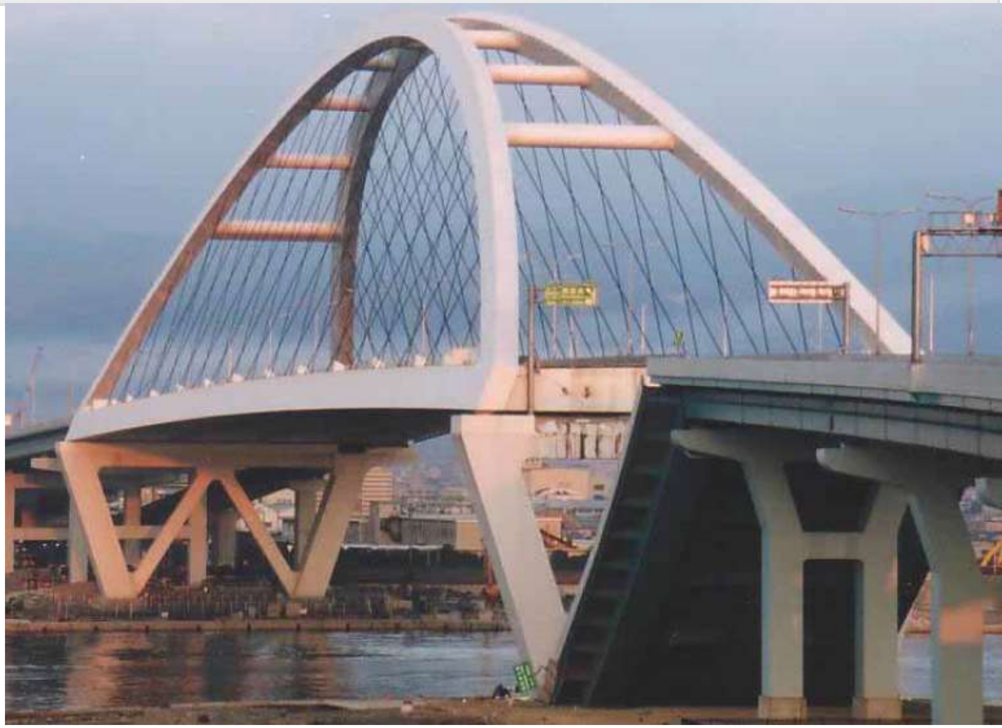


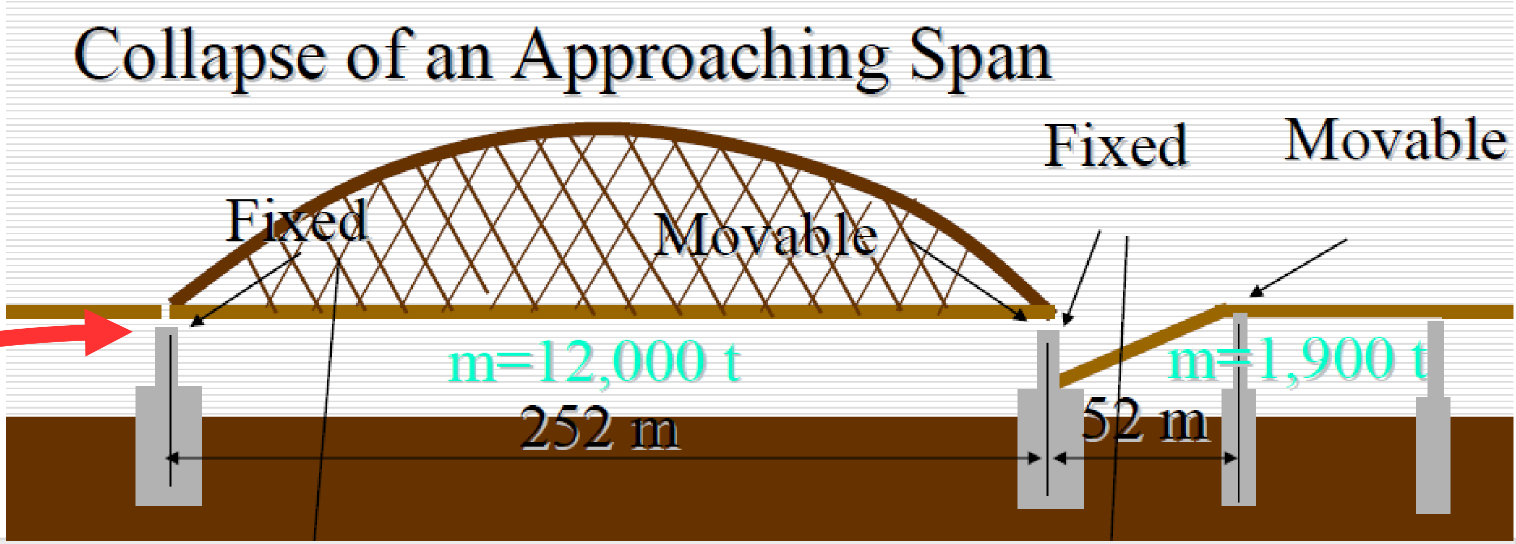
Following Kobe: Steel columns



(K. Kawashima)

Following Kobe: bearings

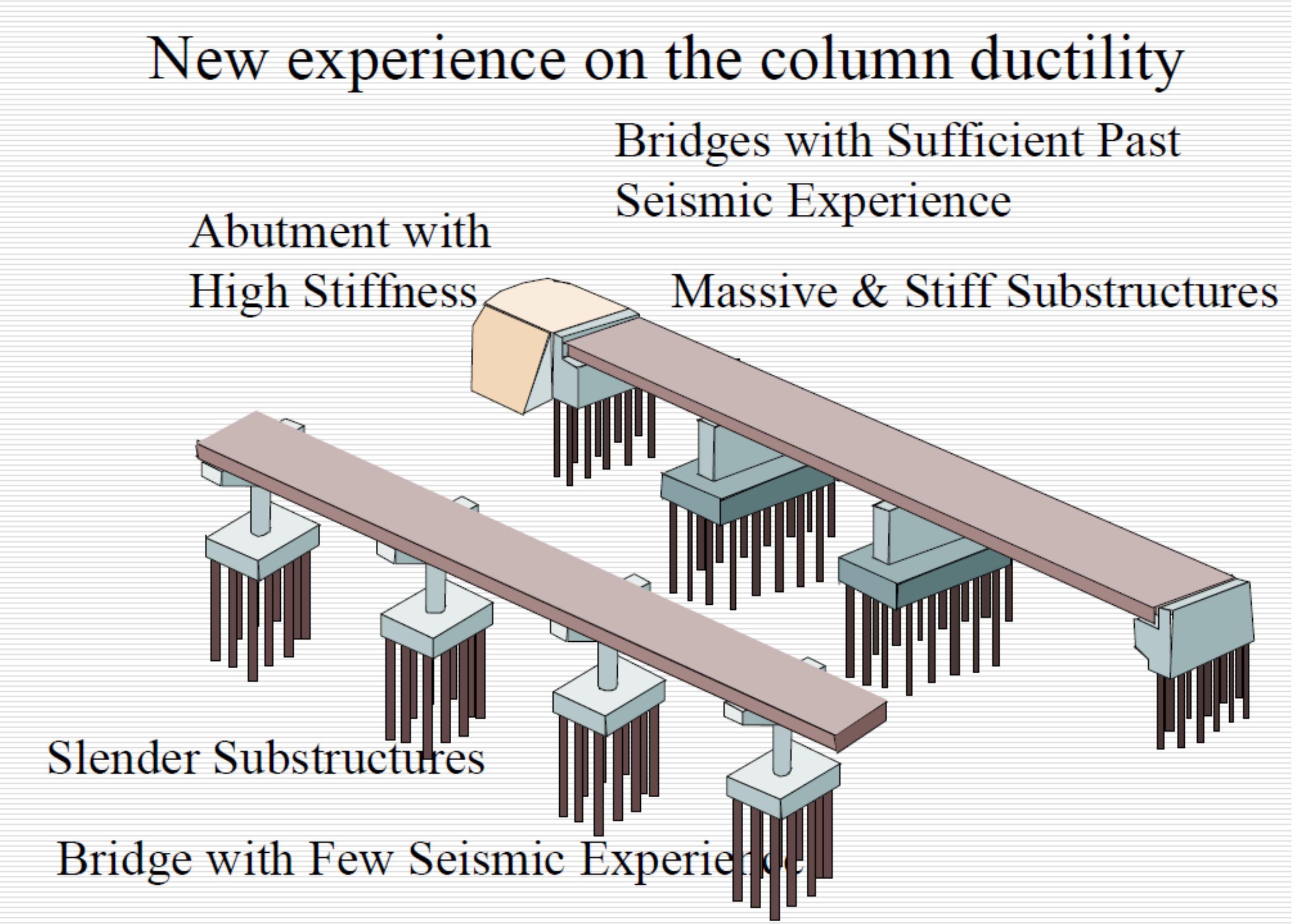




(K. Kawashima)

Following Kobe: residual drift

(K. Kawashima)



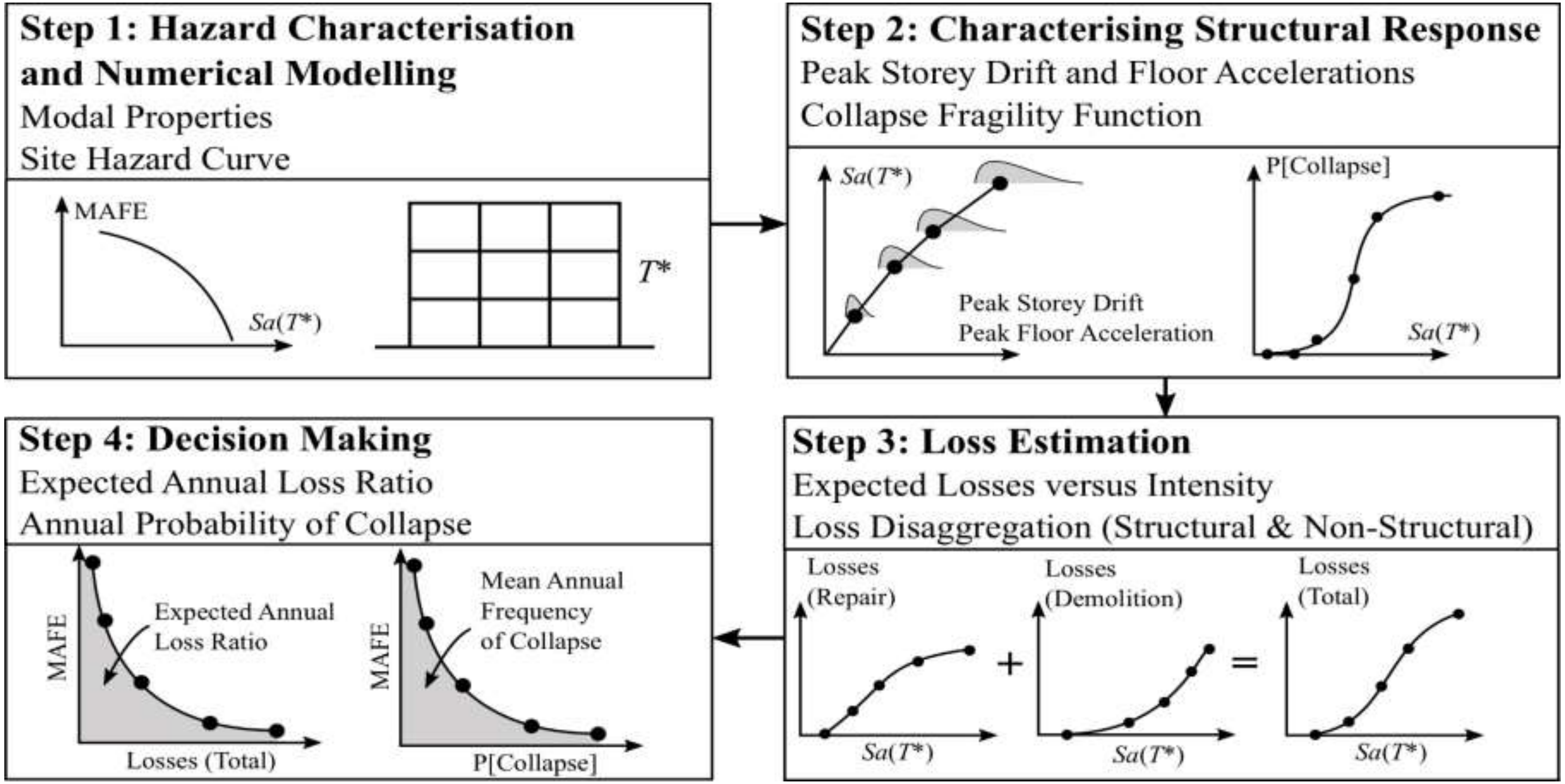
The real lessons from Kobe

Losses induced by bridge damage
are disproportionate
to the number of collapses

How can we use this in practice for design, assessment and strengthening?

A possible reference framework: FEMA P58 (PEER PBEE methodoly)

Focus on assessment of existing buildings



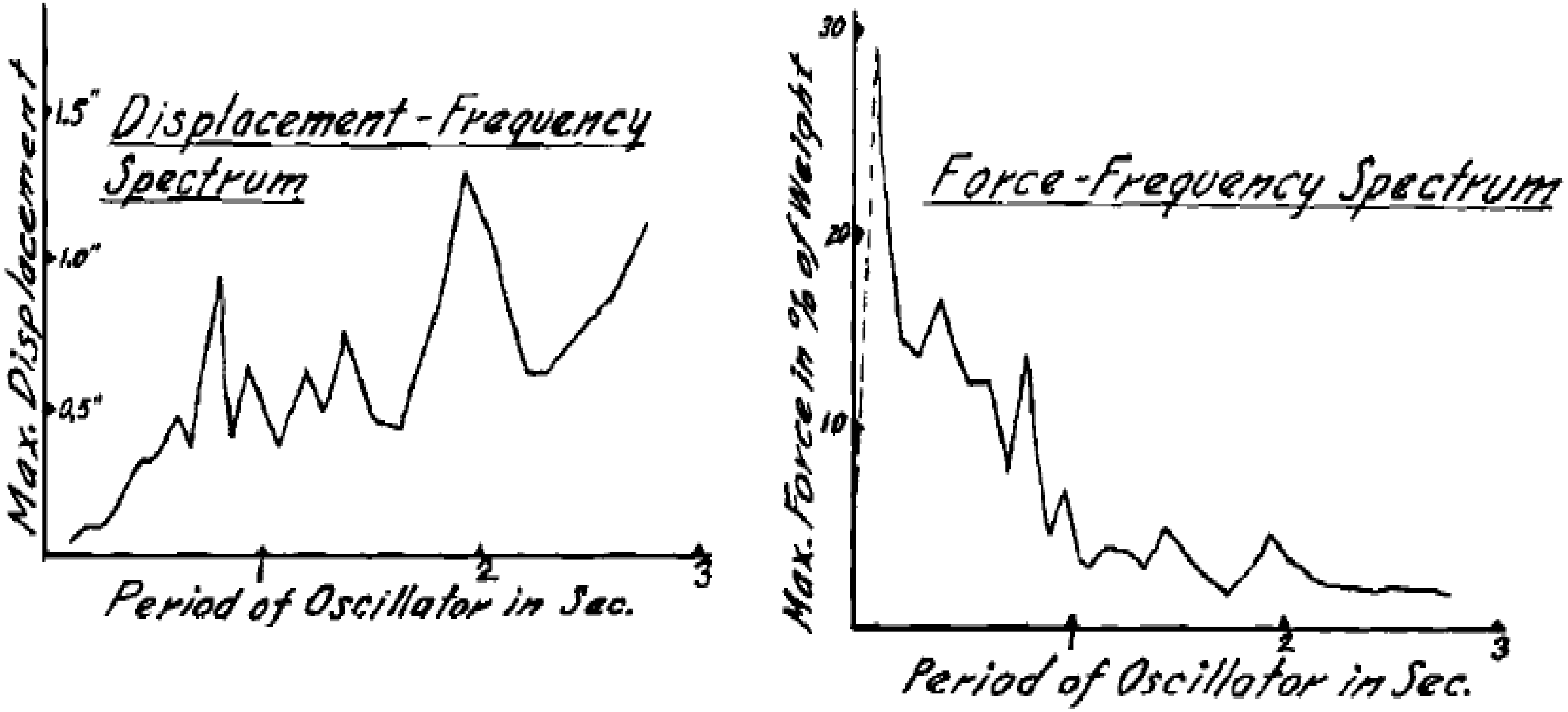
1. Proper definition of input

2. Selection of structural system

3. Consistent design of structural and non-structural elements (bearings, joints, barriers, …)

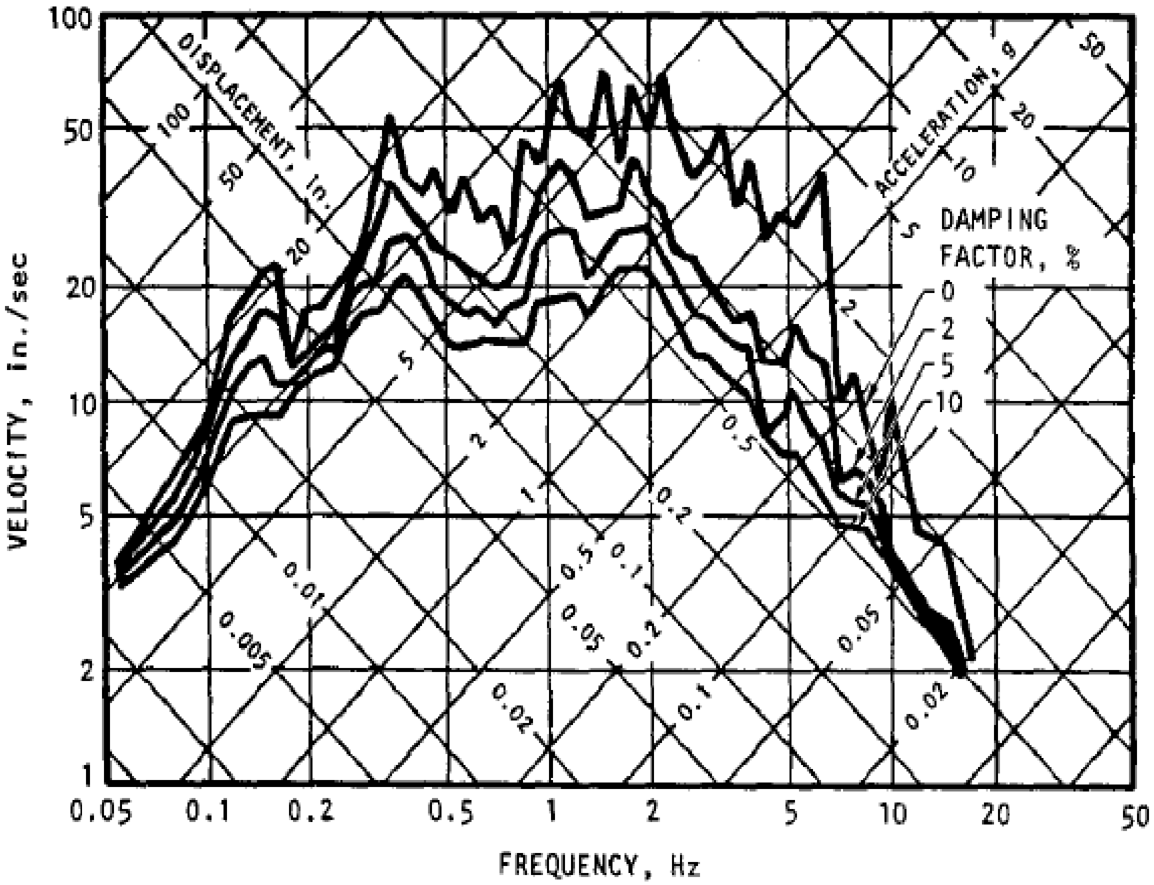
Definition of seismic input

in the form of appropriate design spectra

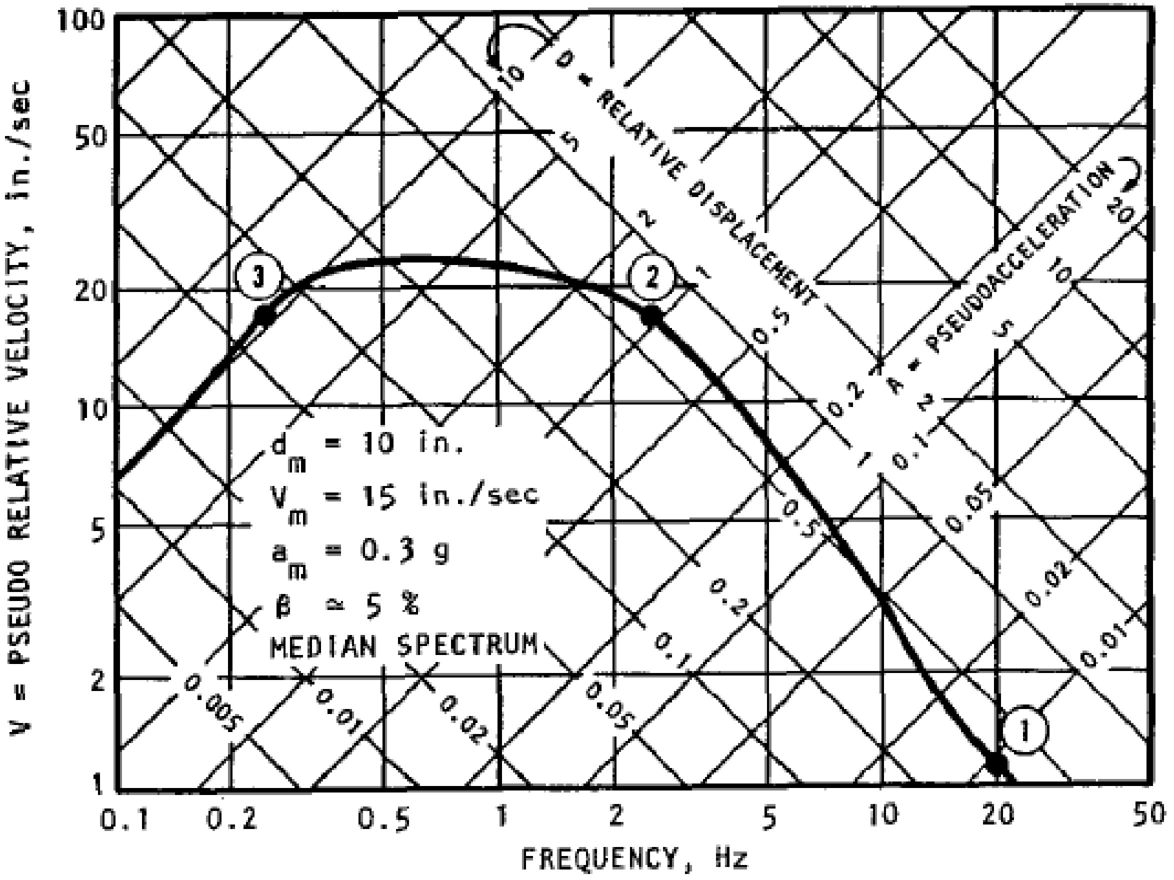


Displacement and acceleration response spectra for a component of the Los Angeles earthquake of 2 October 1933

Housner (1941)

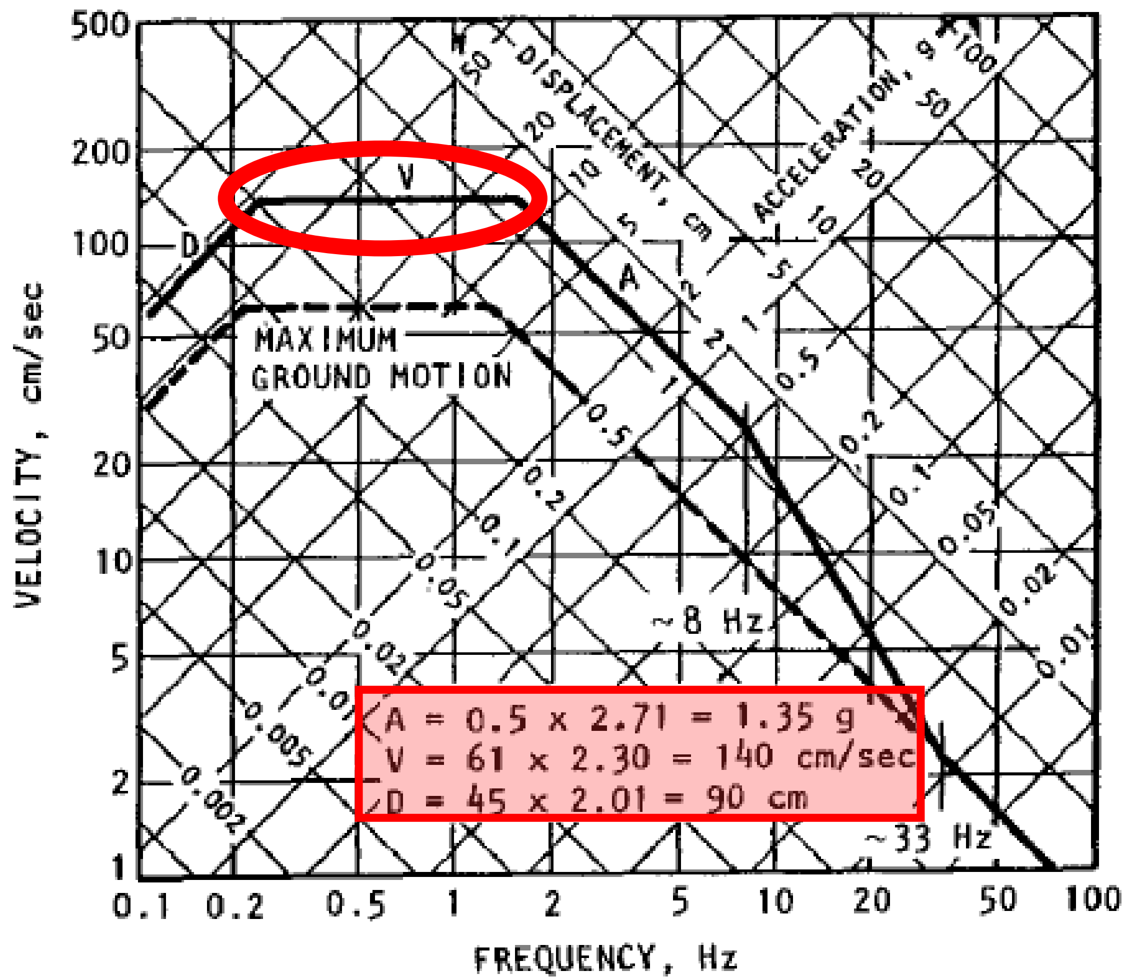


Response spectra from the NS El Centro record



"Typical response spectrum"

Newmark and Hall, 1982



Why constant velocity?

Elastic design spectrum for 0.5 g PGA, 5% damping and one sigma cumulative probability

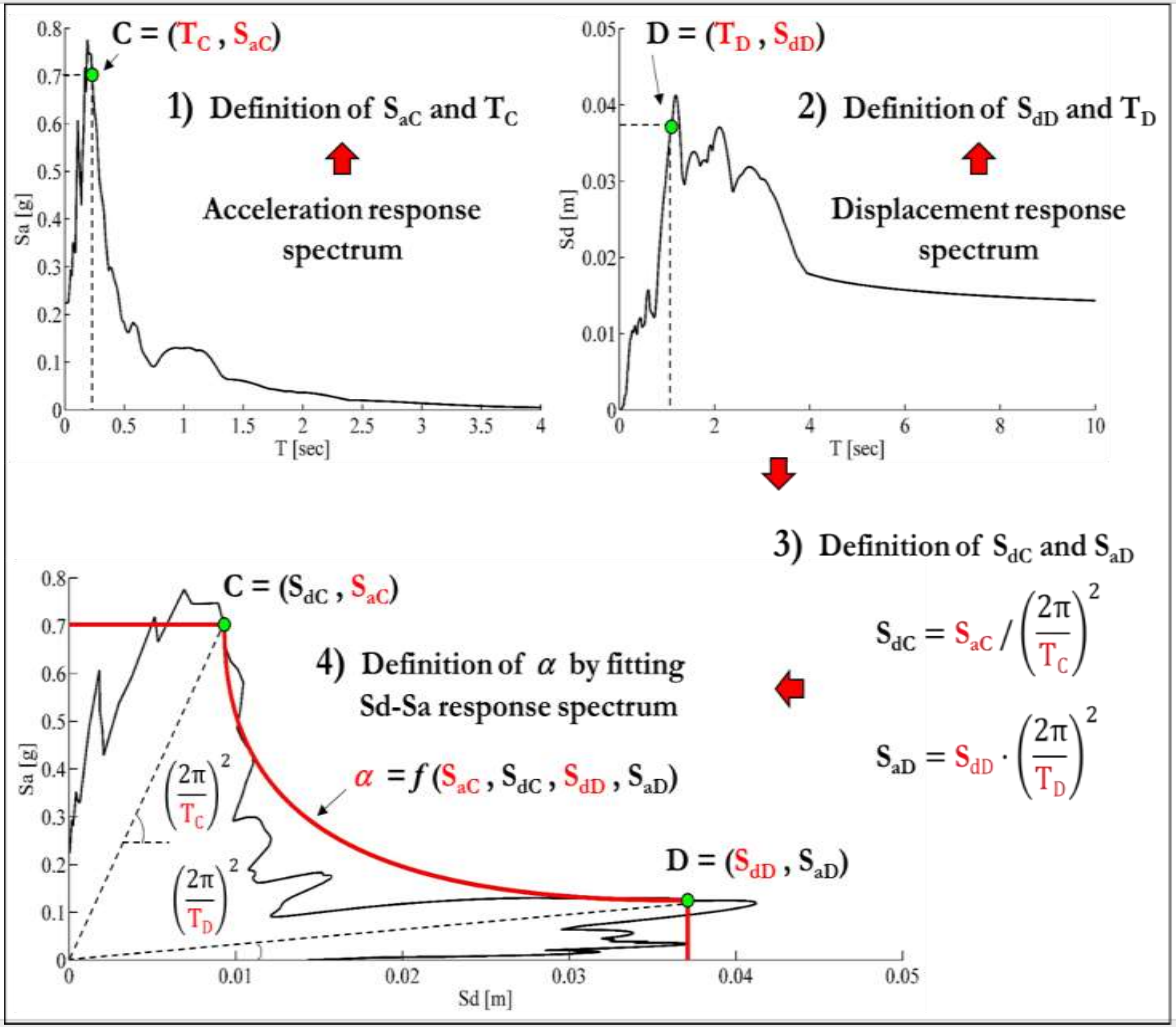
Newmark and Hall (1982)

Definition of design and assessment spectra

Point C (T_C, S_{aC})

Point D (T_D, S_{dD})

α: shape of curve between C and D



The assumption of constant velocity force the position of points C and D

Formulation of the parameter α

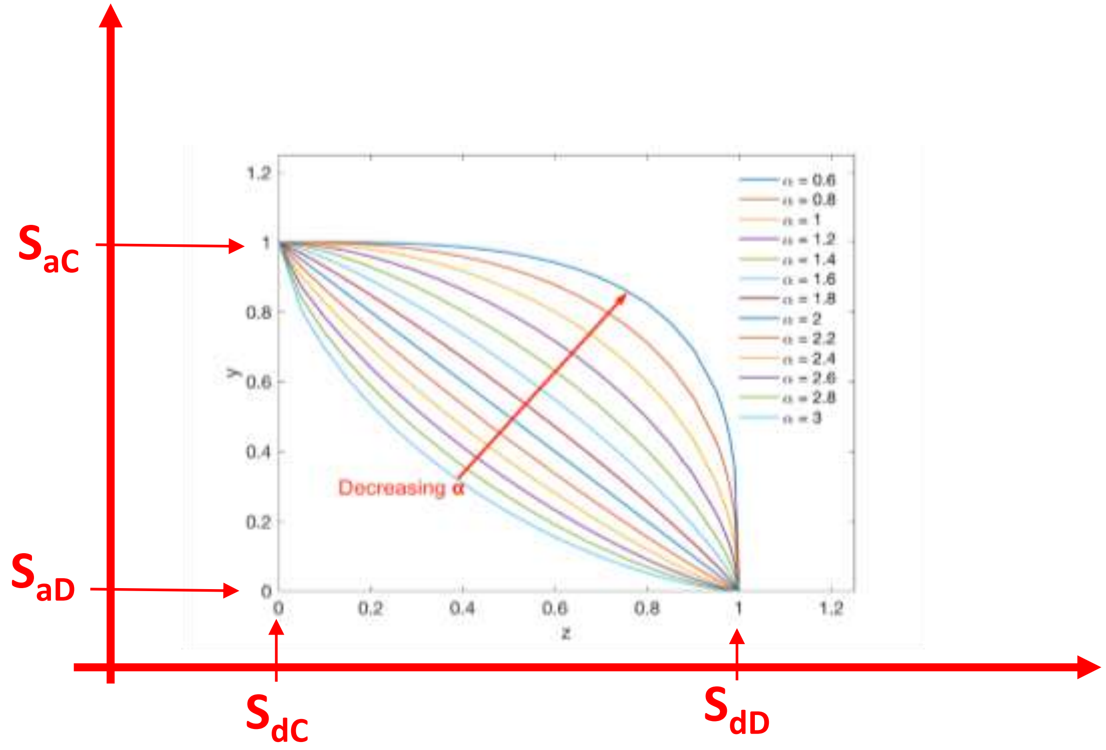
analogy with the function that modifies the shape of a force – displacement curve of a viscous damper

$$y = \sin^{\alpha} t$$

$$z = \cos^{\alpha} t$$

$$y = \sin^{\alpha}\left(\cos^{-1}\left(z^{1/\alpha}\right)\right)$$

$$S_{aD} = S_{dD}\frac{4\pi^2}{T_D^2}$$



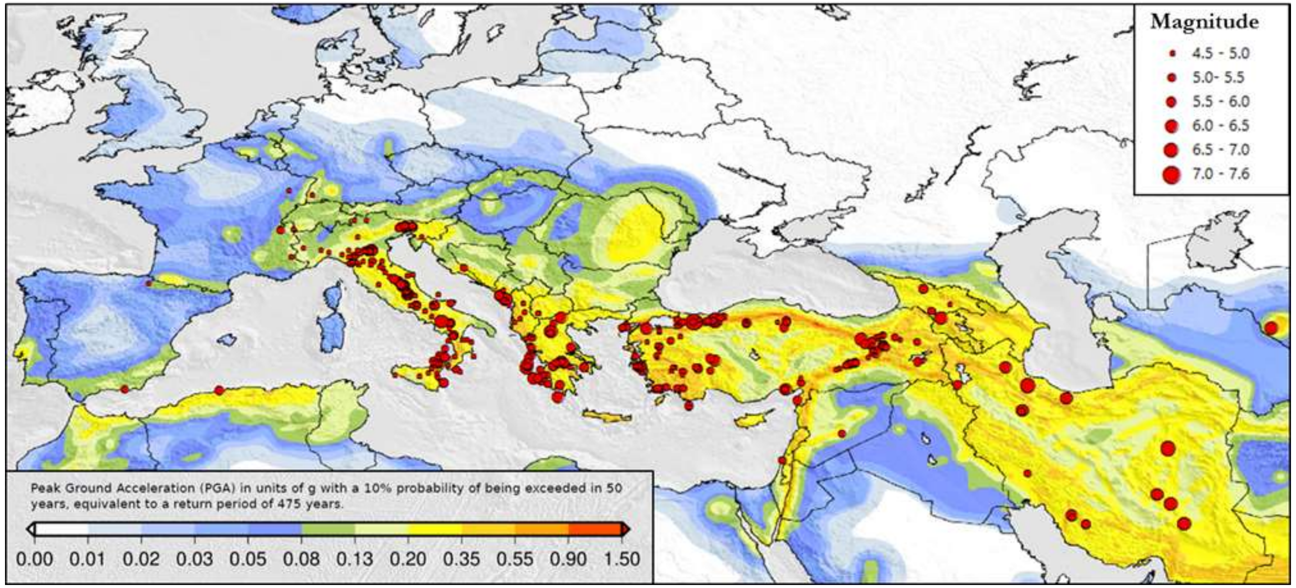
$$S_{dC} = S_{aC}\frac{T_C^2}{4\pi^2}$$

A simple transformation of coordinates:

$$S_a = S_{aD} + \left(S_{aC} - S_{aD}\right)\cdot \sin^{\alpha}\left(\cos^{-1}\left(\frac{S_d - S_{dC}}{S_{dD} - S_{dC}}\right)^{\frac{1}{\alpha}}\right)$$

Assumed key parameters:

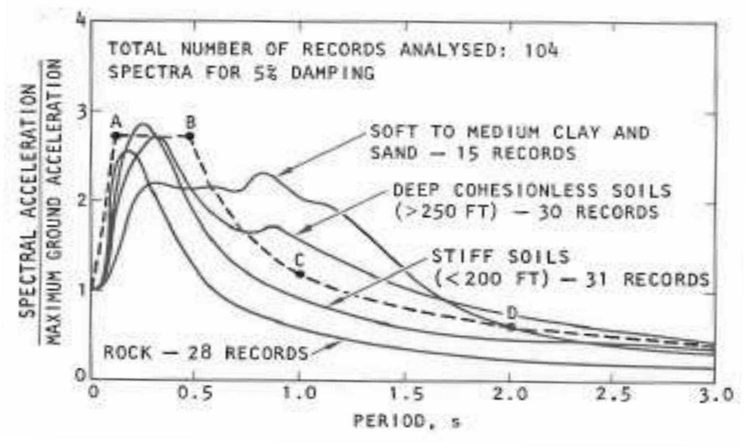
Magnitude
Distance
Soil type

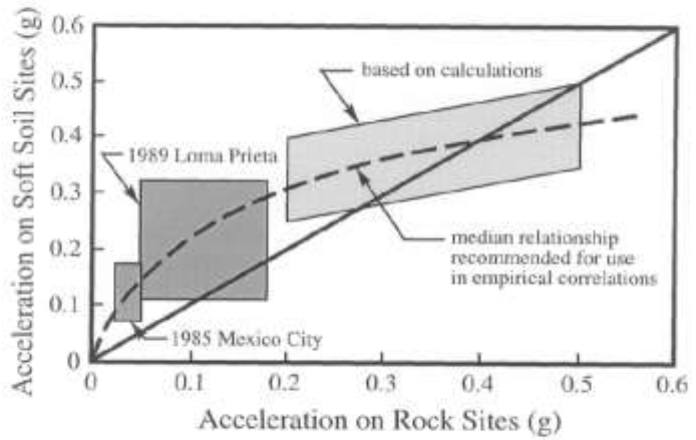


3433 couples of records from 387 events

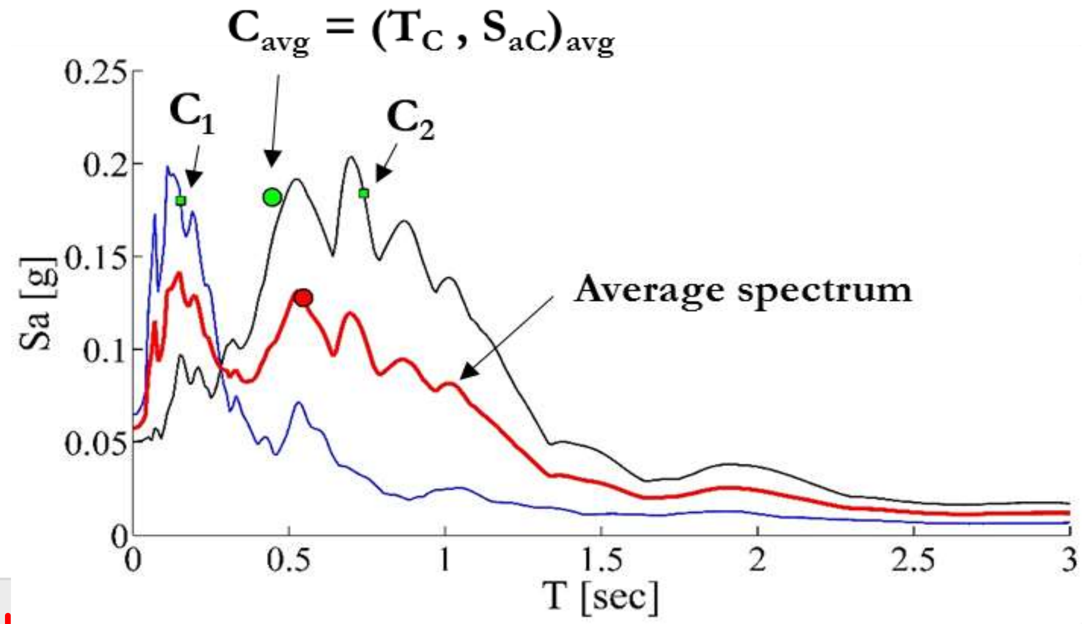
Soil class	M_w	r (km)						
		<10	10-20	20-30	30-40	40-50	50-60	>60
A	7.0-7.6	1	-	-	-	-	-	-
	6.5-7.0	1	2	1	1	-	-	-
	6.0-6.5	6	3	1	4	2	1	7
	5.5-6.0	4	5	11	5	11	6	19
	5.0-5.5	11	14	25	12	19	14	41
	4.5-5.0	11	19	31	27	35	34	61
B	6.5-7.0	5	1	3	1	1	1	2
	6.0-6.5	19	18	6	13	6	7	17
	5.5-6.0	18	25	34	22	22	22	32
	5.0-5.5	44	74	50	70	54	49	109
	4.5-5.0	57	114	103	75	86	80	156
C	7.0-7.6	1	-	1	-	-	-	-
	6.5-7.0	1	-	1	-	-	-	-
	6.0-6.5	6	3	4	14	4	5	7
	5.5-6.0	27	21	23	18	14	11	19
	5.0-5.5	25	34	43	37	43	32	45
	4.5-5.0	35	78	58	50	67	41	79
D	6.5-7.0	1	-	-	-	-	-	-
	6.0-6.5	-	1	-	1	2	-	-
	5.5-6.0	3	1	2	1	1	5	2
	5.0-5.5	2	1	1	4	2	8	13
	4.5-5.0	9	11	2	3	3	7	12
E	6.5-7.0	-	-	-	-	1	-	1
	6.0-6.5	2	-	-	3	-	2	1
	5.5-6.0	2	2	5	1	2	1	4
	5.0-5.5	3	9	6	10	1	1	7
	4.5-5.0	3	5	8	9	2	3	5

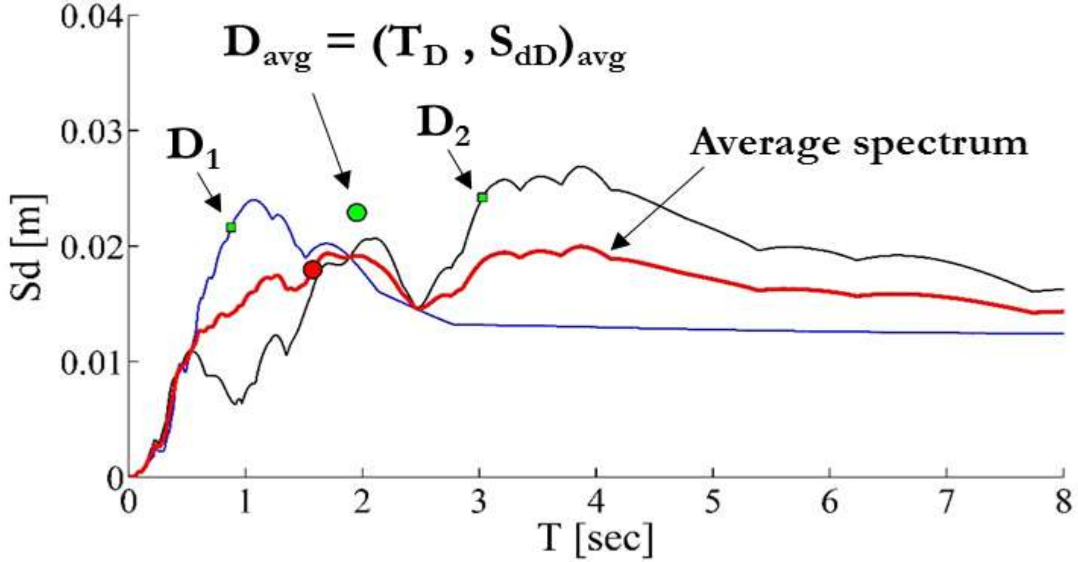
Effect known but not appropriately recognized in codes





Average of peaks is different from peak of average

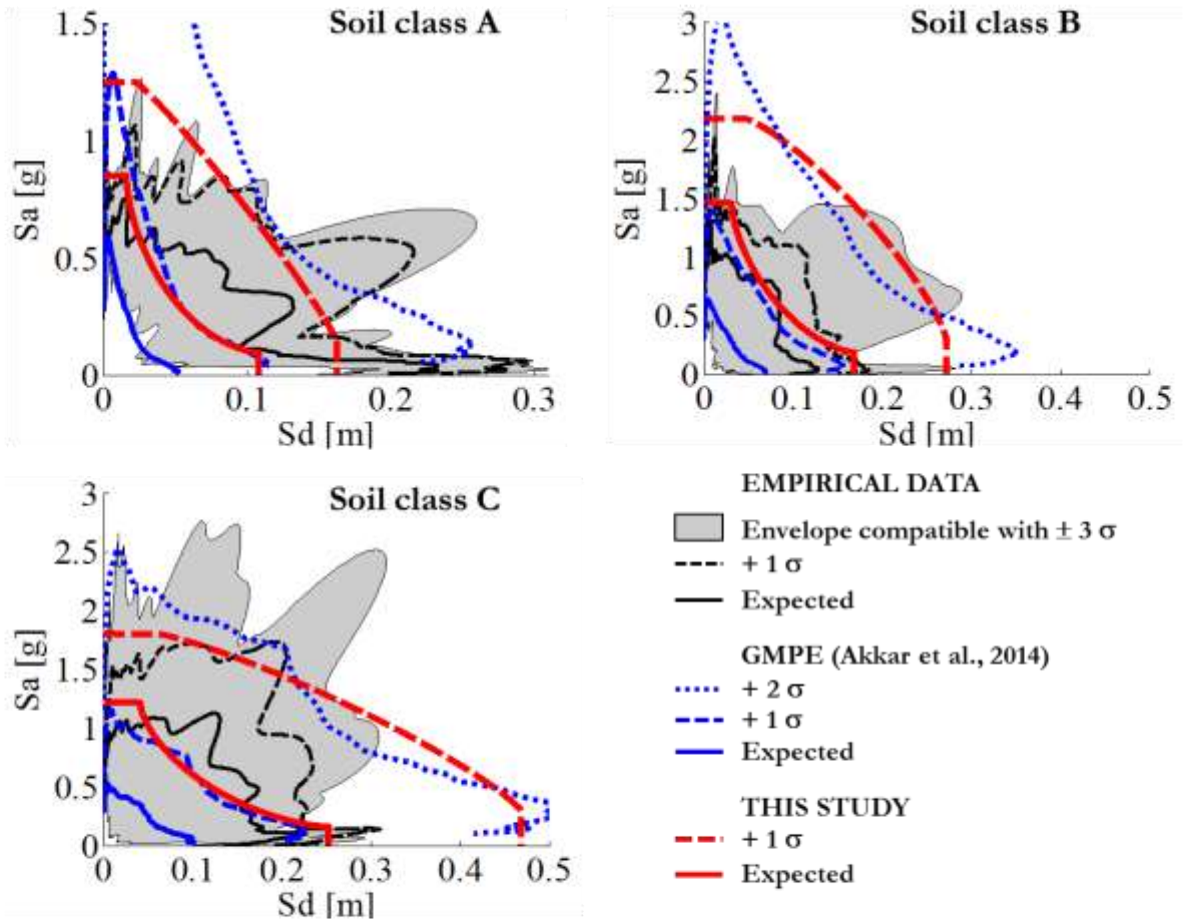




Resulting spectra

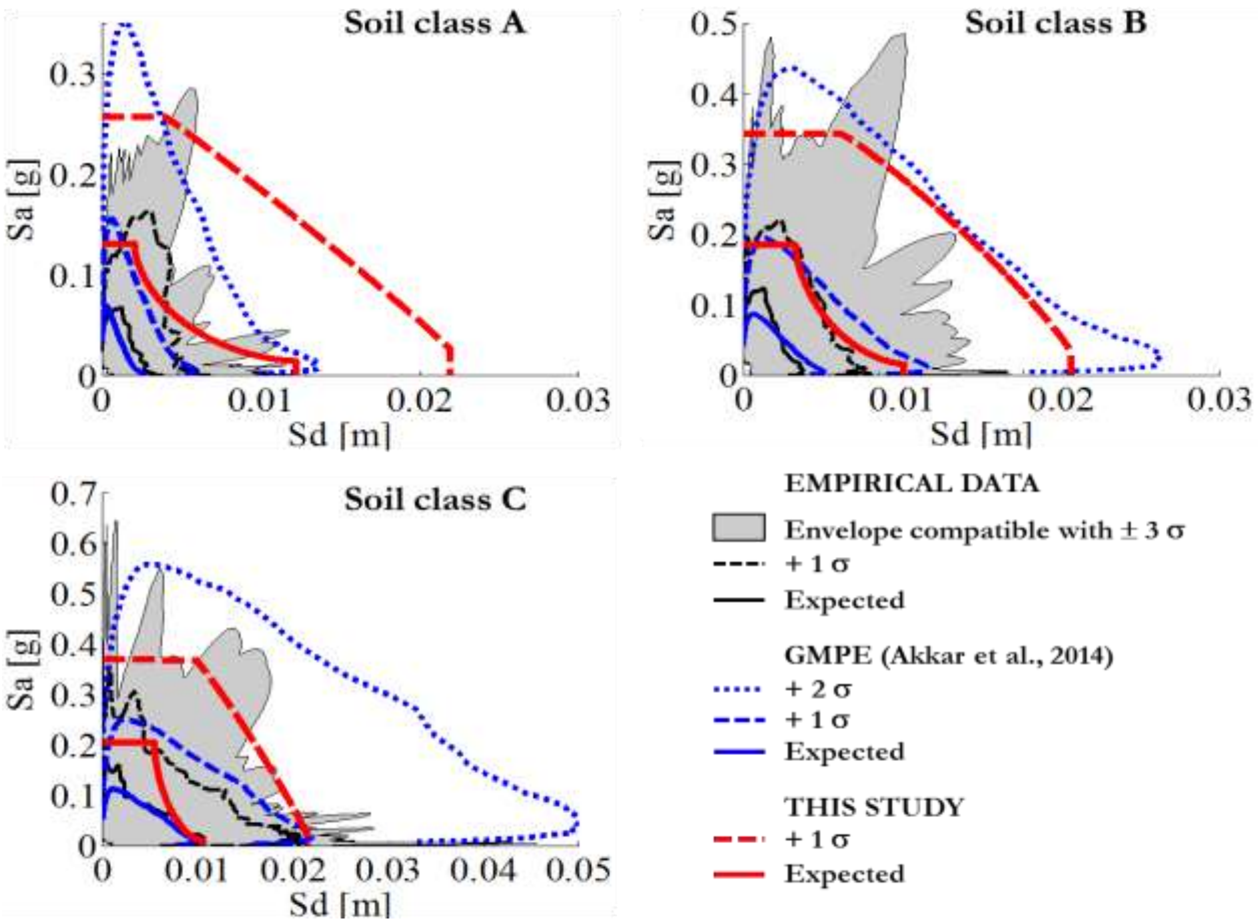
Distance: r < 10 km

Magnitude: 6.0 < M < 6.5



Distance: 20 < r < 30 km

Magnitude: 5.0 < M < 5.5

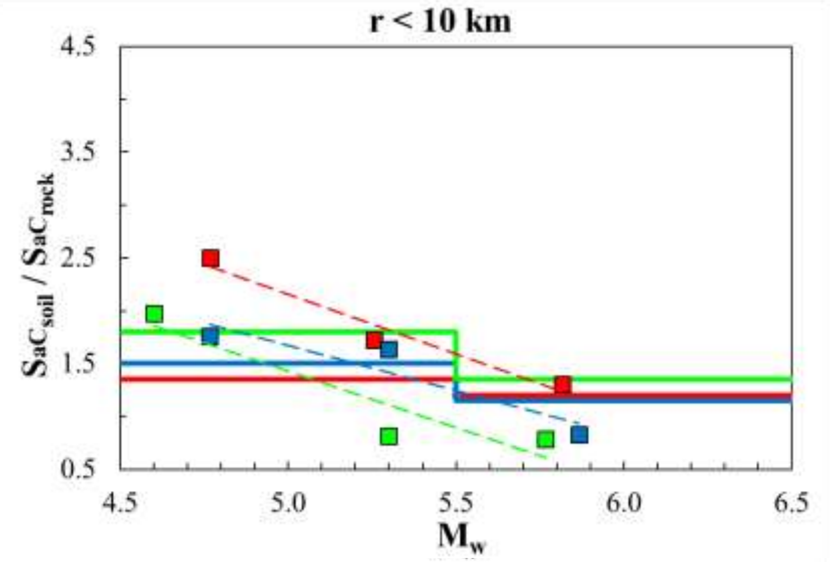


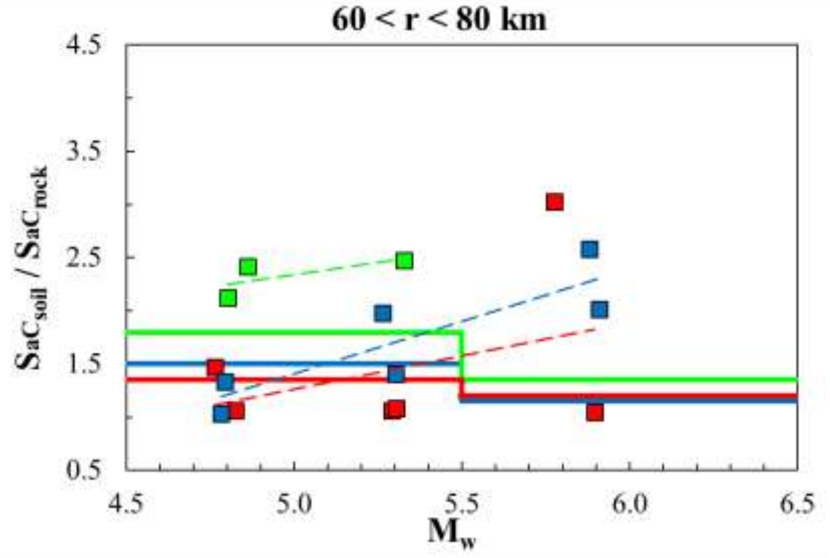
Soil amplification

Depends on Magnitude Distance

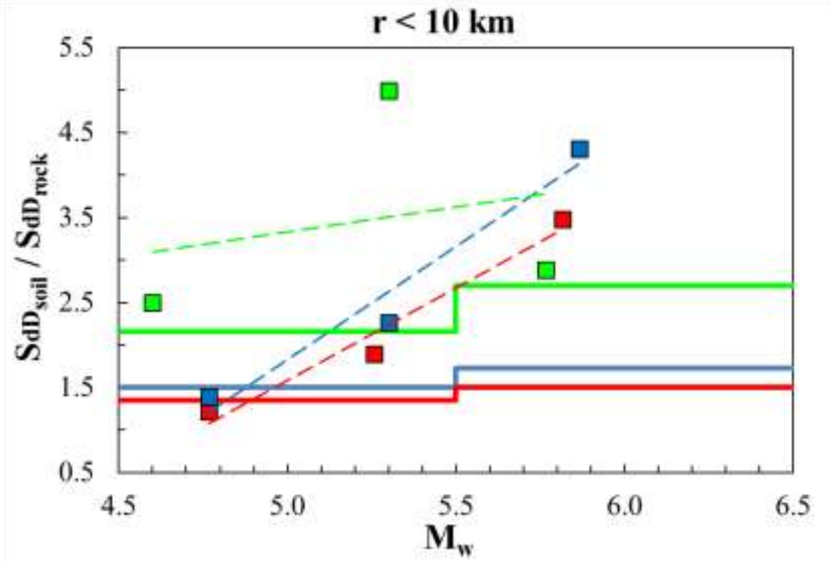
is different on acceleration and displacement

Maximum spectral acceleration (S_{aC})





Maximum spectral displacement (S_{dD})



Calvi and Andreotti (2019)

- Soil class B (stiff soils)
- Soil class C (soft soils)
- Soil class D (very soft soils)

Eurocode 8 (CEN, 2006)

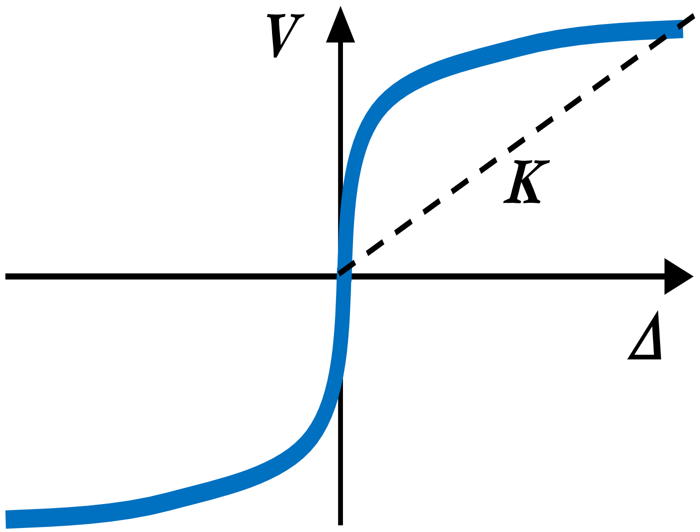
- Soil class B
- Soil class C
- Soil class D

Accounting for energy dissipation

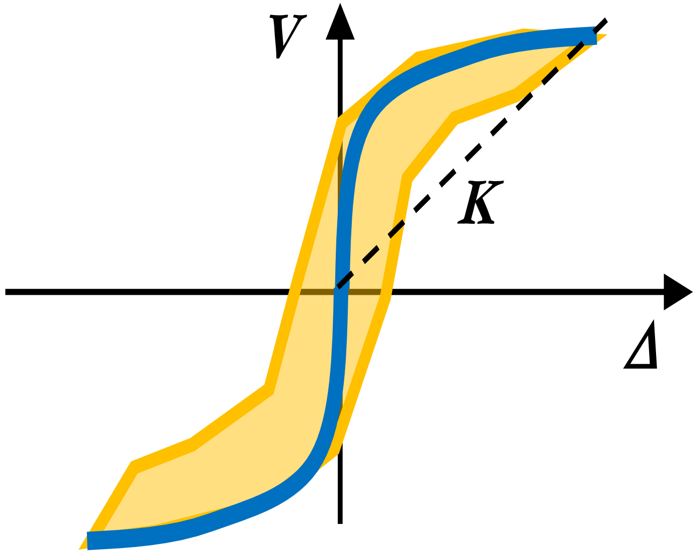
why a structural parameters is accounted for on the demand side?

An increased dissipation capacity reduces the expected displacement demand for the same sets of ground motions

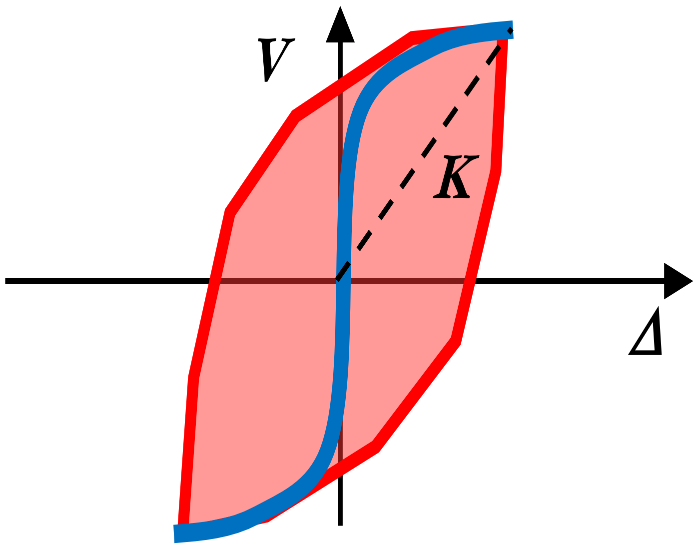
Non-linear elastic response
(ξ_e = 5%, η_x = 1)



Moderately dissipative structure
(ξ_e = 15%, η_x = 0.75)



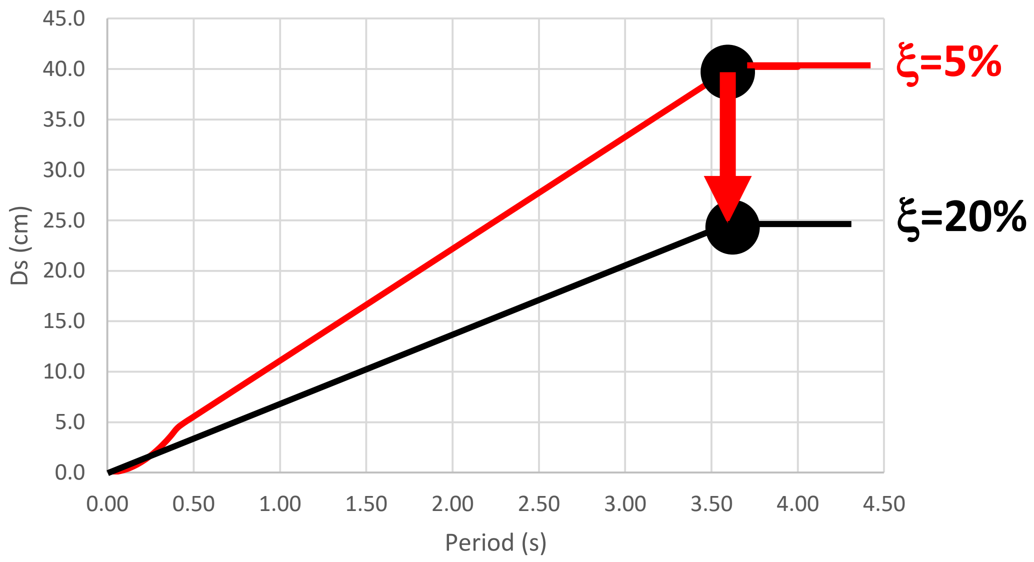
Highly dissipative structure
(ξ_e = 26%, η_x = 0.5)



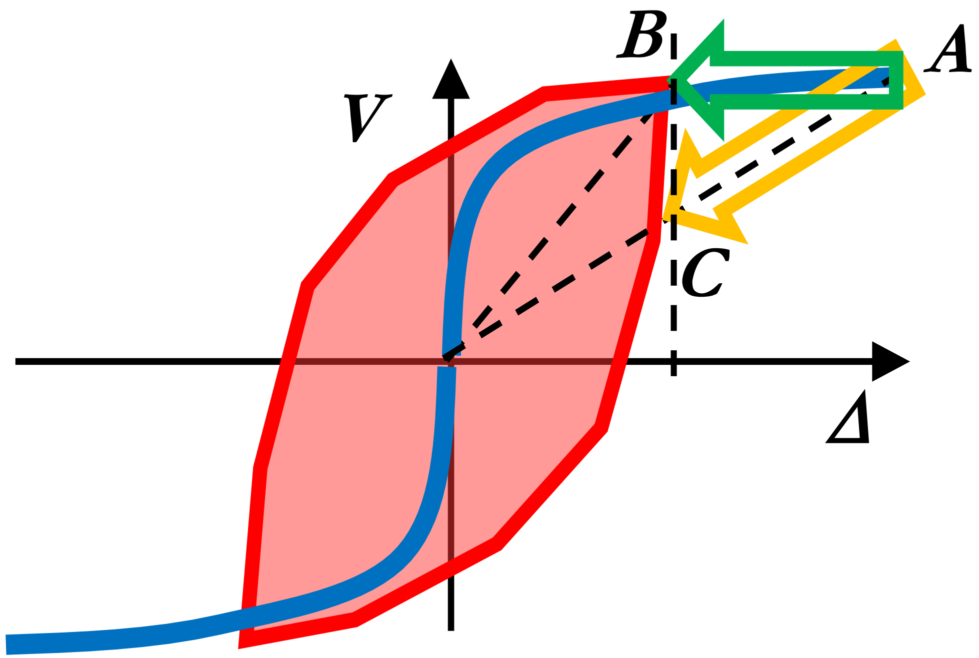
Current practice:

- ❑ Reduce displacement
- ❑ Conserve period
- ❑ Acceleration reduces proportionally to displacement:

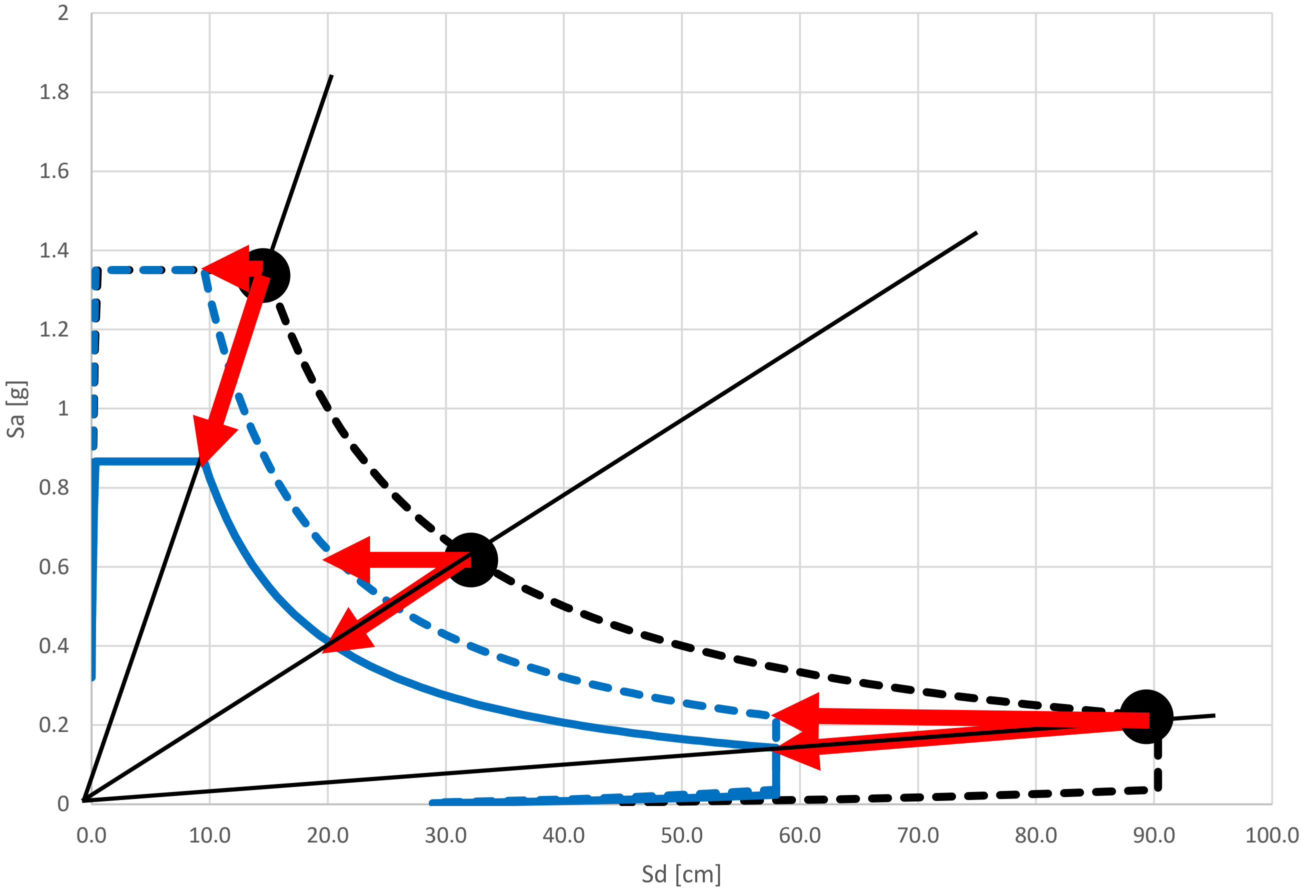
$$S_a = \frac{4\pi^2}{T^2} S_d$$

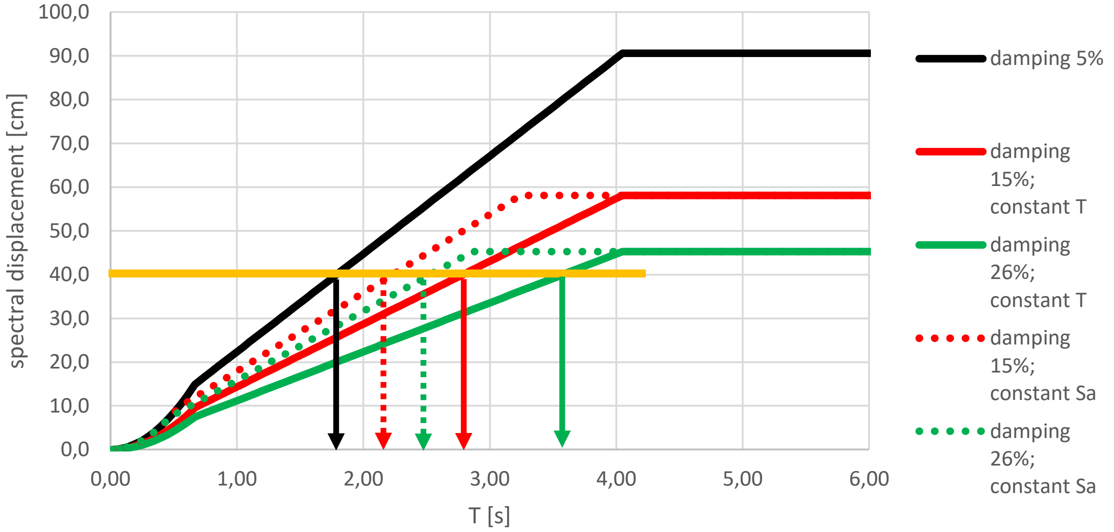


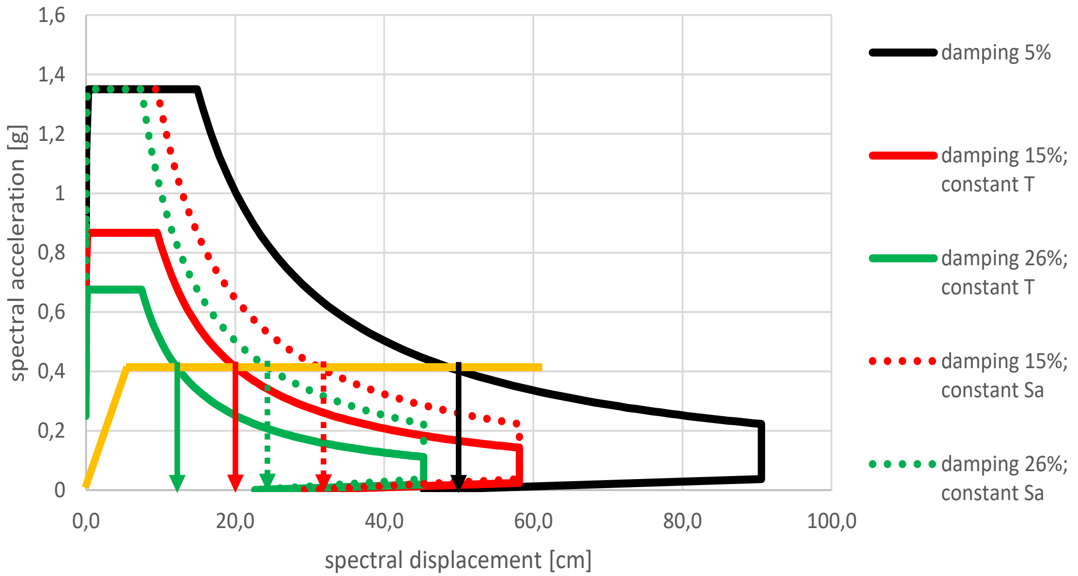
Is it correct to reduce displacement conserving period or acceleration?



Relevant effects on spectrum shape, design period and displacement capacity







Defining "acceptable" performances

Design for life safety
and check for damage limitation?

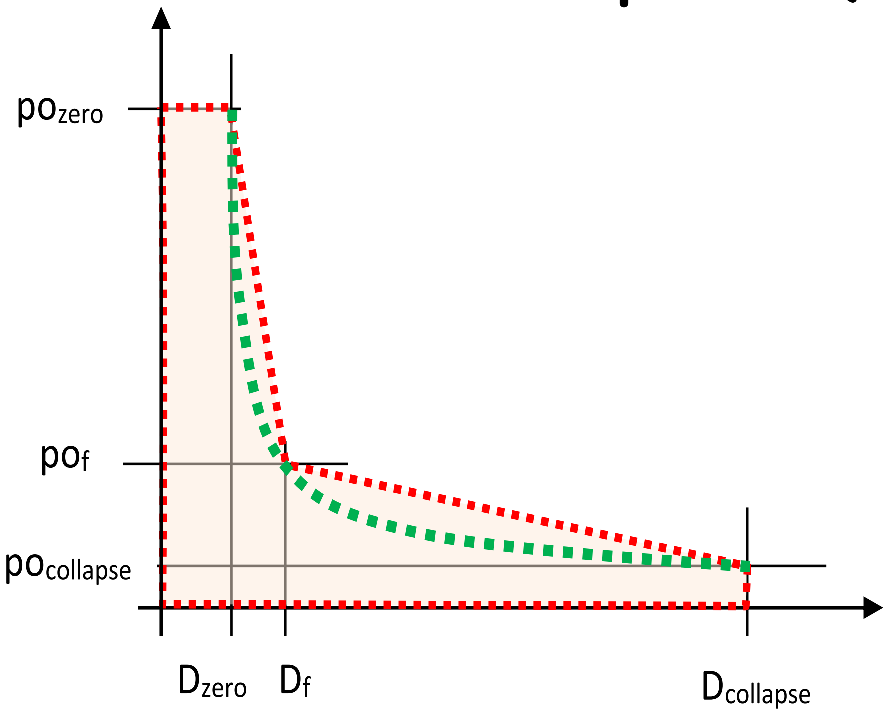
Or

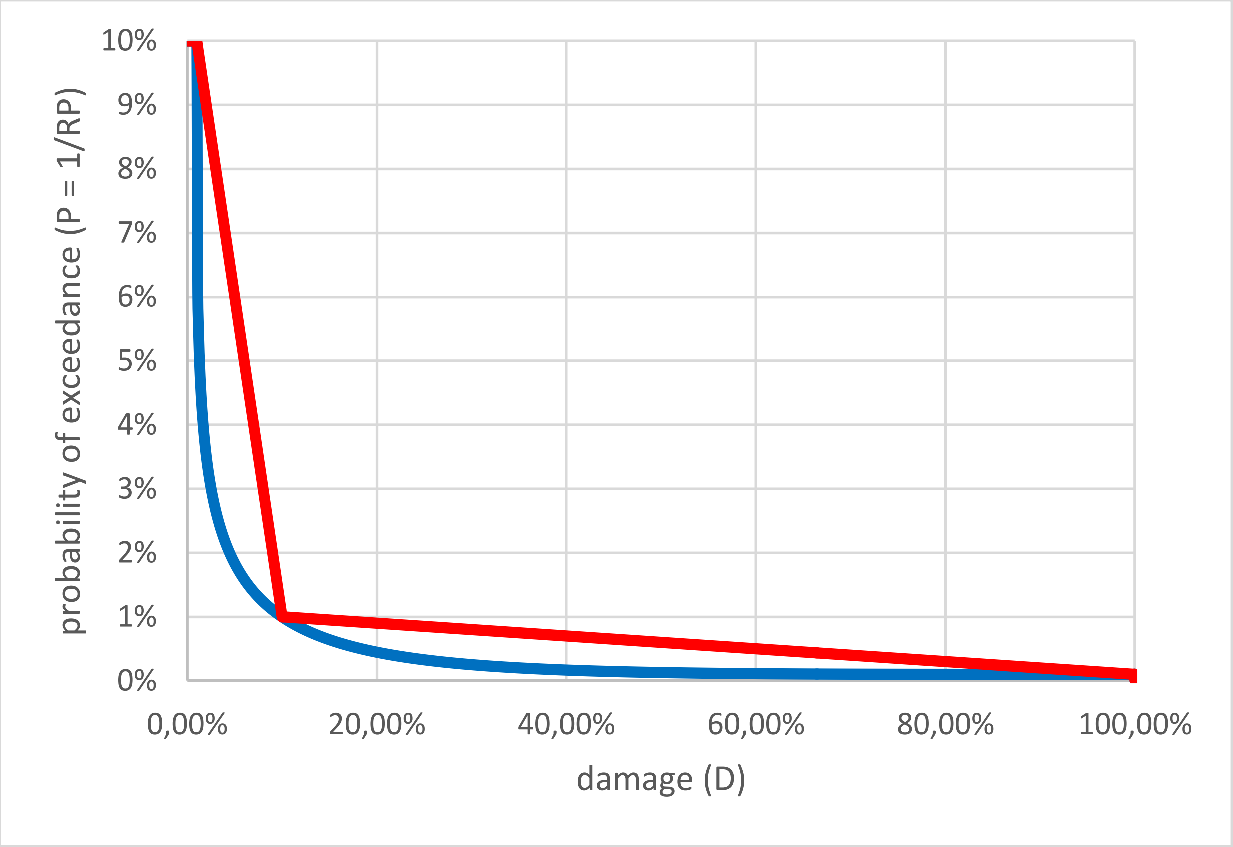
Use the expected annual loss
as a design tool?

EAL (expected annual loss) = $\int (po \times D)\,dD$

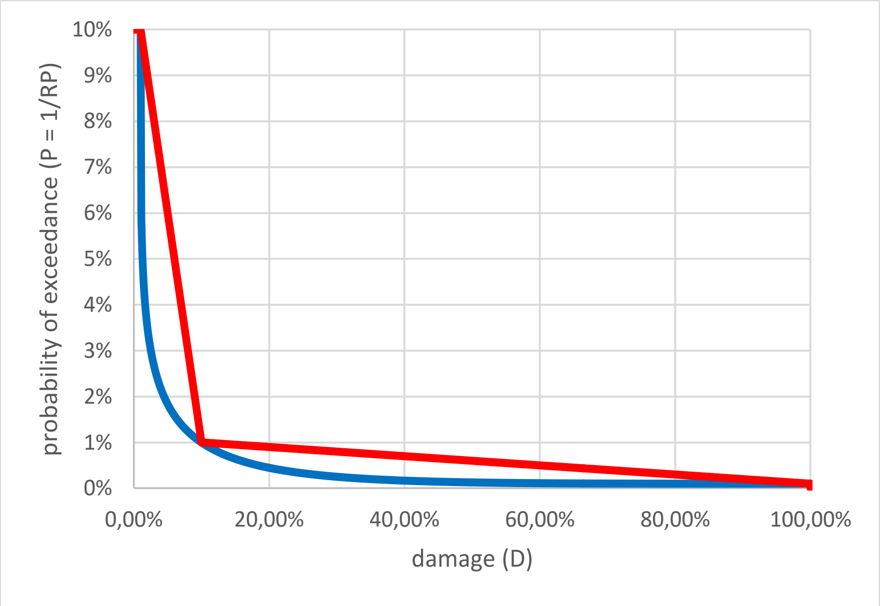
as a tool to design

po = yearly probability of occurrence
D = level of damage
RP = return period (1/po)





EAL as a tool to design



Derive an equation for the blue curve

Polynomial $$P = k_1 + k_2 D^{k_3}$$

With: $$P_{collapse} = k_1 + k_2$$

Or (simpler and better):

$$P = P_{collapse} + \left(P_{zerodamage} - P_{collpase}\right) \cdot \sin^{\alpha}\left(\cos^{-1}\left(\frac{D - D_{zero}}{D_{collapse} - D_{zero}}\right)^{\frac{1}{\alpha}}\right)$$

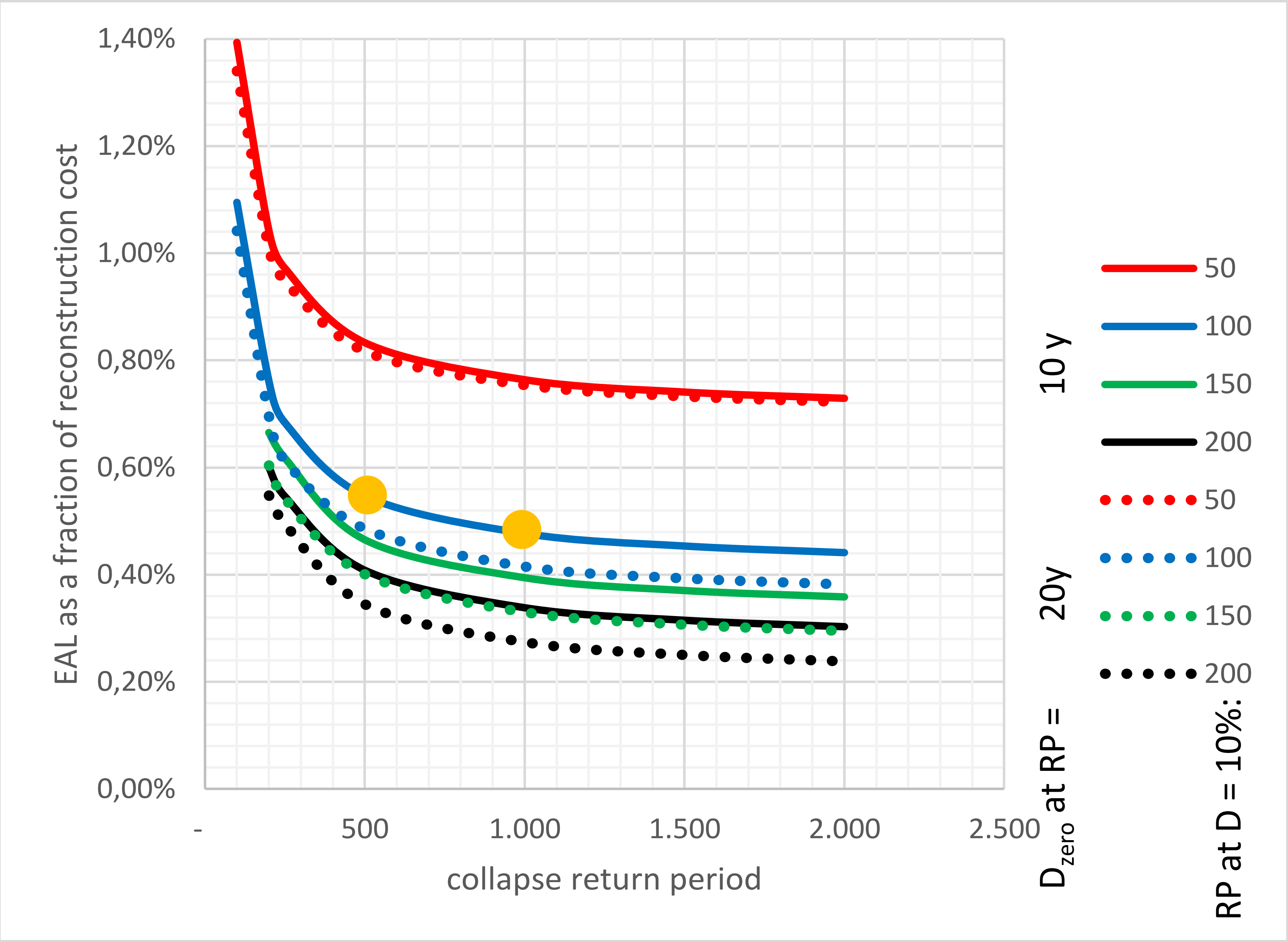
forced to pass through the two extreme points and governed by the single parameter α to pass through the f point.

E.G.:

$D_{collapse}$ = 100%
D_{zero} = 1%
D_f = 10%

$P_{collapse}$ = 1/1000
$P_{zerodamage}$ = 1/10
$P_{zerodamage}$ = 1/100

EAL as a tool to design

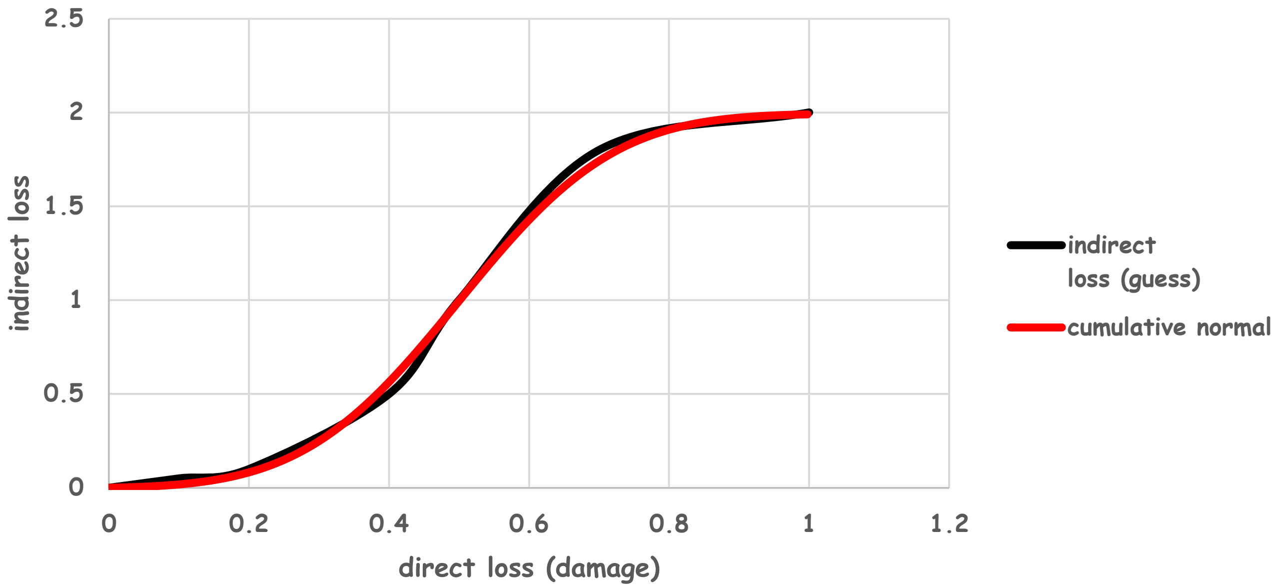


EAL as a tool to design

Indirect cost for bridges

		Bridge over major river	Highway interchange
Detour	Ld (km)	30	15
N vehicles	Nv	10,000	10,000
Cost ave per km	Ckm (€)	0.50	0.50
EALi day (LdxNvxCkm)	EALid (€)	150,000	75,000
Length	L (m)	1,000	150
Cost reconstr	Cr (€)	50,000,000	10,000,000
EALid % Rc	EALid/Cr	0.30%	0.75%
Closure days to Rc	Dcr (days)	333	133
Time reconstr (d)	Tr (days)	670	670
Indirect cost reconstr	Tr/Dcr	2,01	5,03

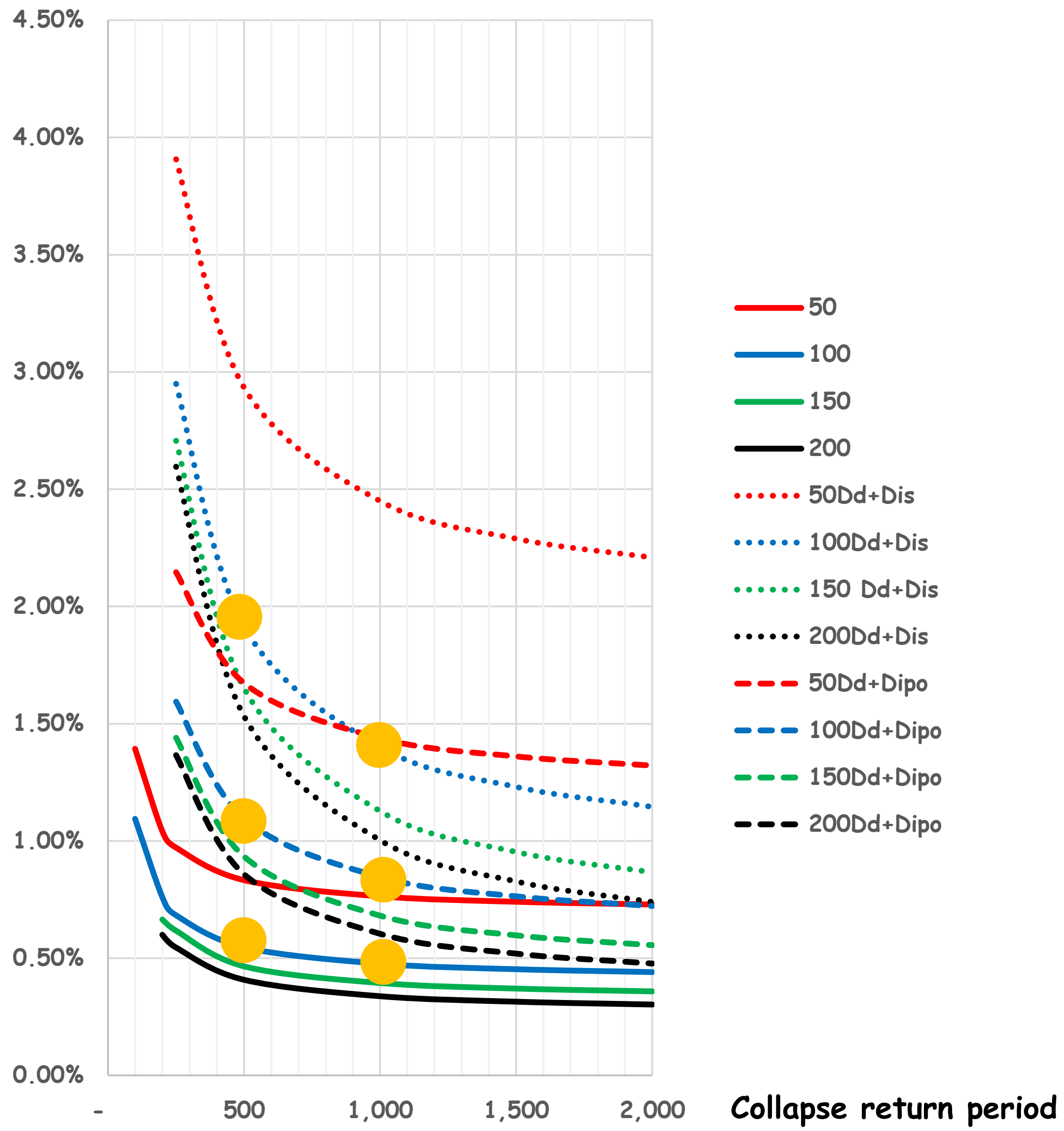
Relation between direct loss (damage) and indirect loss (cost of traffic interruption)



EAL as a tool to design

Total EAL as a function of:

- Assumed ground motion
- Collapse return period
- Damage limitation



Bridge response

- Event sequence ⇨
- Complete push-over to collapse
- Cost of event prevention
- Longitudinal vs. transversal

1) Barrier damage
2) Joint damage
3) Bearing damage
4) Bearing failure + unseating
5) Pier shear damage/collapse
6) Pier flexural d/c
   a) Concrete spalling
   b) Bar yielding
   c) Bar buckling
   d) Permanent tilting
7) Foundation rotation
8) Foundation displacement
9) Liquefaction
10) Isolation system damage
11) Isolation system collapse
12) …

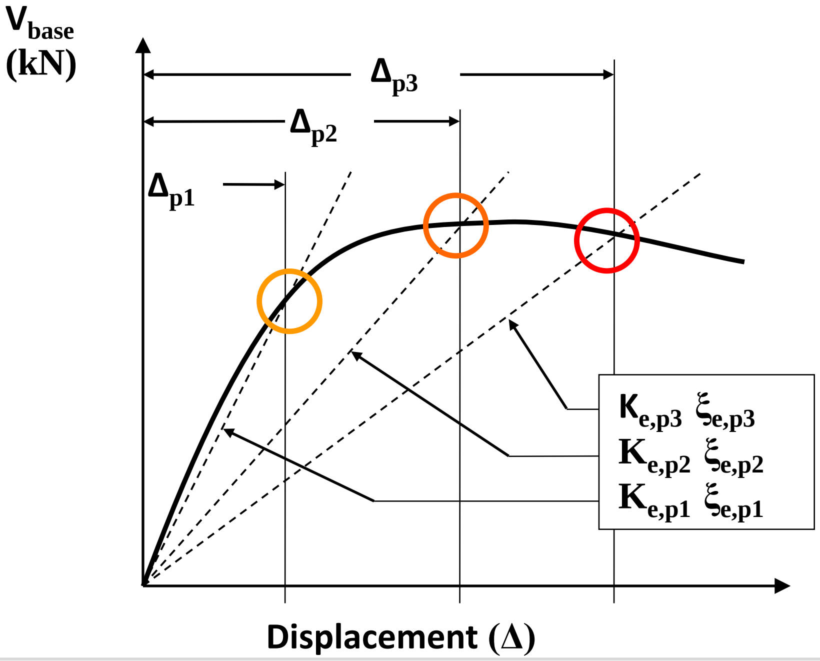
Cost of prevention

Cost of repair

Duration of repair

Bridge response

Complete push-over to collapse (not point verification capacity > demand)



Does not matter how low the probability

Even when capacity design is applied

What happens next?

e.g.:

Joint damage?
Bar yielding?
Bar buckling?
Permanent tilting?
Foundation rotation?

Bridge response

Cost of event prevention and repair, time for repairing ⇨

NOTE:

- A three-span 150 m bridge may rest on 8 I.D.
- Cost range 50-100 k€
- Indirect loss for 90 days closure for replacement 6-7 M€
- Probability of occurrence?

E.G. 1:

What is the cost of increasing the displacement capacity of a joint?
What is the cost of a new joint?
How much time is required to obtain a new joint (reduced speed)?
How much time is required to substitue the joint (bridge closed)

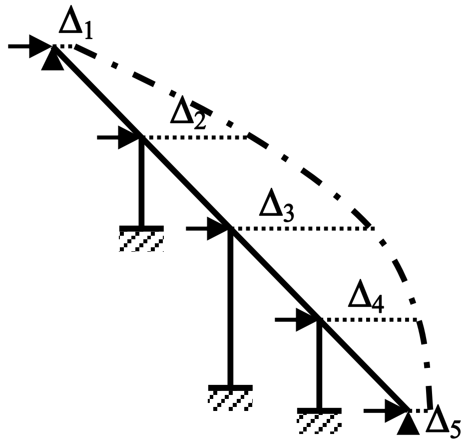
E.G. 2:

What is the cost of increasing the displacement capacity of an isolation device?
What is the cost of a new I.D.?
How much time is required to obtain and install new I.D.s (bridge possibly closed)?

Bridge response

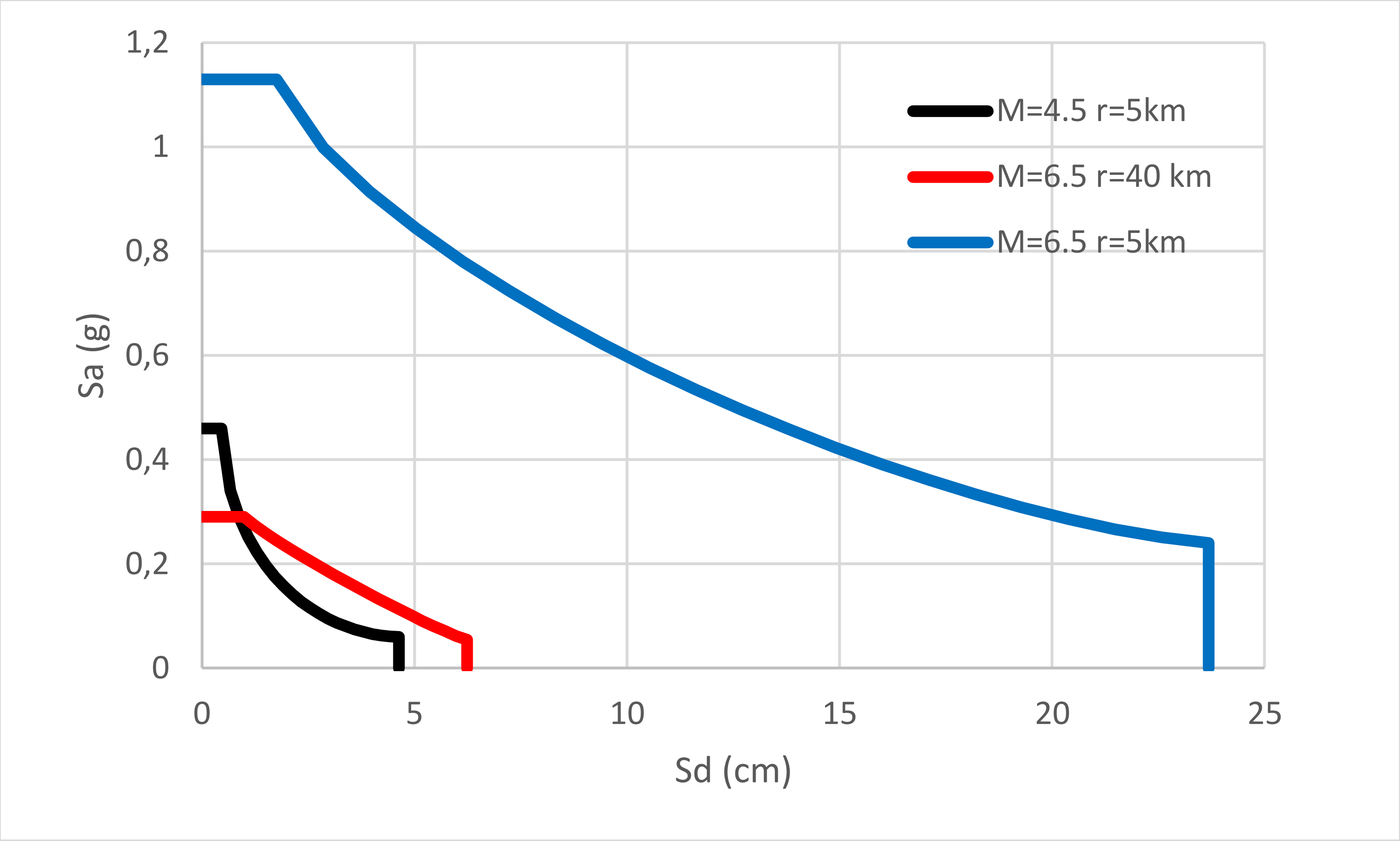
Longitudinal vs. transversal ⇨ Longitudinal response is «always» governing design

Transversal:
- Displacement governed by deformed shape
- No temperature problems
- All piers (and abutments) reacting



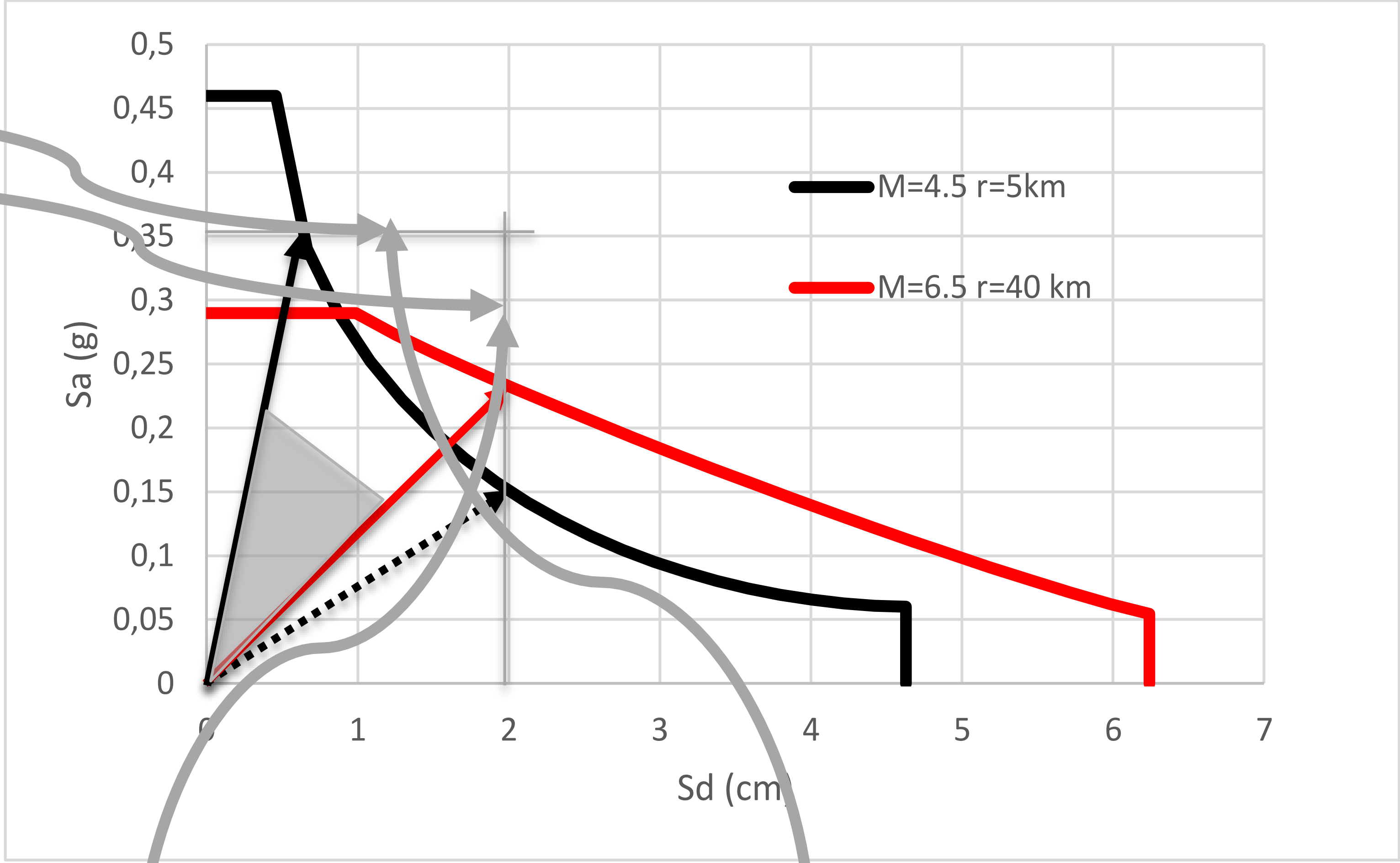
Longitudinal:
- Same displacement demand at all piers
- Need to accomodate temperature induced displacements
- Few piers reacting unless shock transmitters are used

Effects of magnitude and distance
Possible different return period



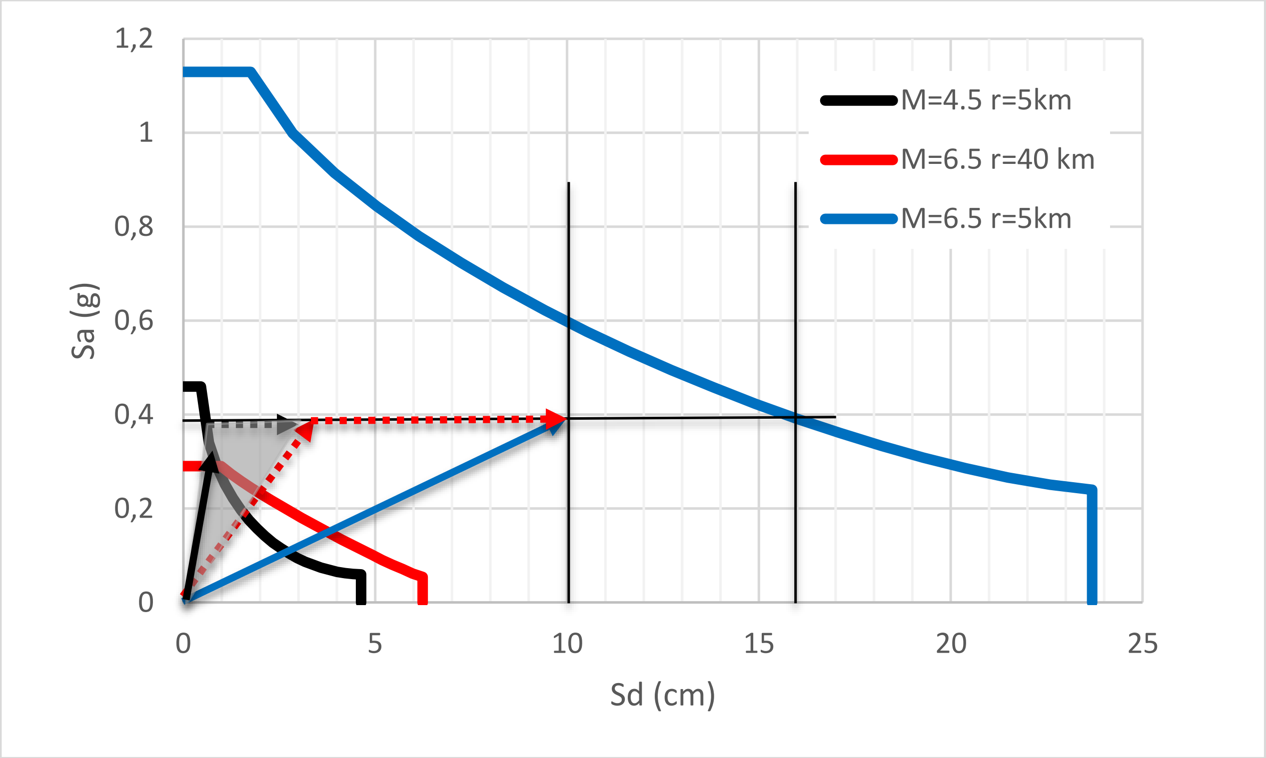
Design for frequent ground motion

Deck acceleration and displacement limits imposed by accepted level of damage



Design for elastic response, $\Delta_d < 20$ mm; $S_{ad} = 0.35$ g

Design for rare ground motion



Design for Δ_d =100 mm.
Elastic response impossible and not compatible with design for frequent event.
Consider correction factor η = 0.6.

Re-visiting Earthquake Resistant Design of Bridges

G. Michele Calvi

IUSS Pavia and Eucentre Foundation