CONVEGNO

˵SISMA ED ELEMENTI NON STRUTTURALI Approcci, Stati Limite e Verifiche Prestazionali˝

Bologna – 24 ottobre 2014

PROGETTO DI ELEMENTI NON STRUTTURALI SOGGETTI AD AZIONI SISMICHE

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IUSS Pavia

Nonstructural components can be classified into three main categories:

–Architectural Components

–Building Utility Systems

–Building Contents

- Architectural Components
  - Built-in nonstructural components that form part of the building.
  - Examples: partitions and ceilings, windows, doors, lighting, interior or exterior ornamentation, exterior panels, veneer, and parapets.

- Architectural Components

Exterior Cladding

Veneers

- Architectural Components

Gypsum Wallboard Partitions

Ceiling Systems

- Architectural Components

Window Systems

Doors

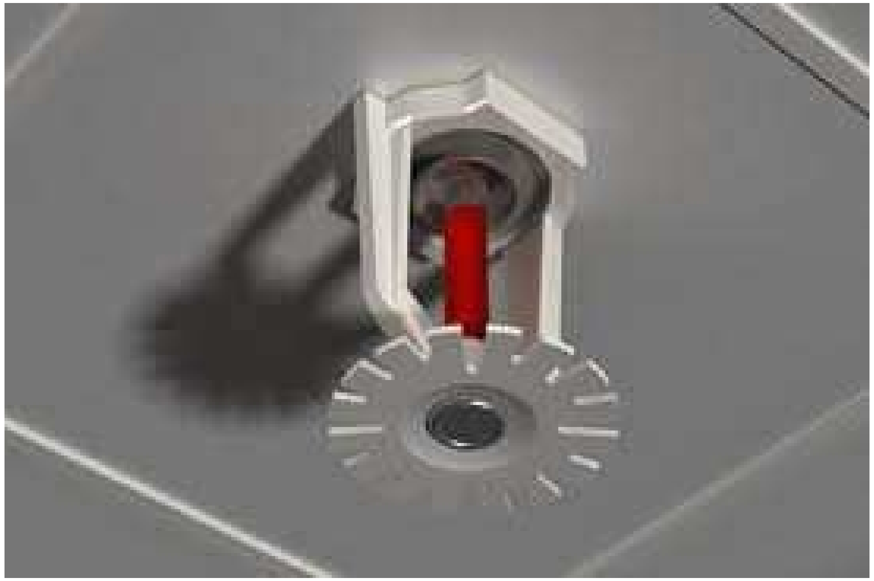
- Architectural Components

Parapets

- Building Utility Systems
  - Built-in nonstructural components that form part of the building.
  - Examples: mechanical and electrical equipment and distribution systems, water, gas, electric, and sewerage piping and conduit, fire suppression systems, elevators or escalators, HVAC systems, and roof-mounted solar panels.

- Building Utility Systems

Piping Systems



Pressurized Fire Sprinkler Systems

- Building Utility Systems

Elevators

Escalators

- Building Utility Systems

HVAC Systems

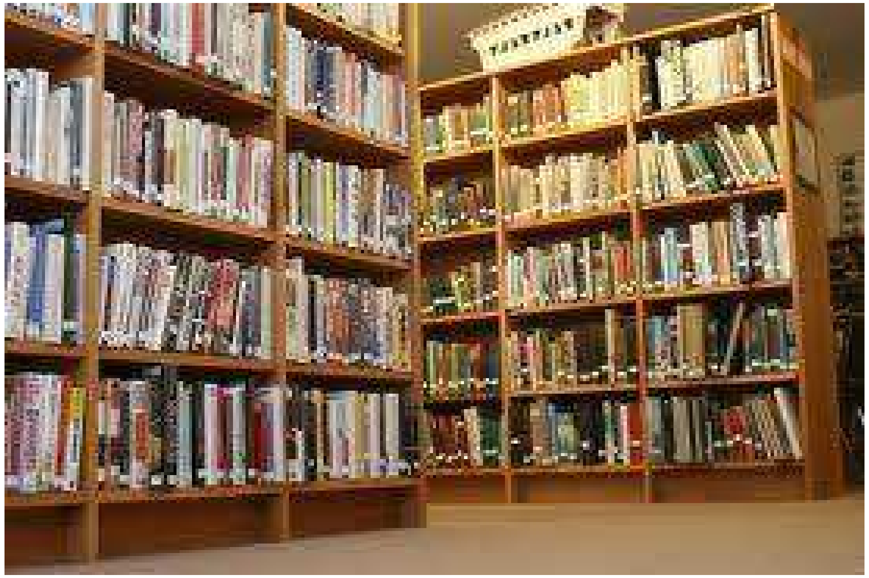
- Building Contents
  - Nonstructural components belonging to tenants or occupants.
  - Examples: computer and communications equipment; cabinets and shelving for record and supply storage; library stacks; kitchen and laundry facilities; furniture; movable partitions; lockers; and vending machines.
  - Judgment needed to identify critical items in a particular building.

- Building Contents

Computer Equipment

Communication Equipment

- Building Contents



Library Stacks

Kitchen Furniture

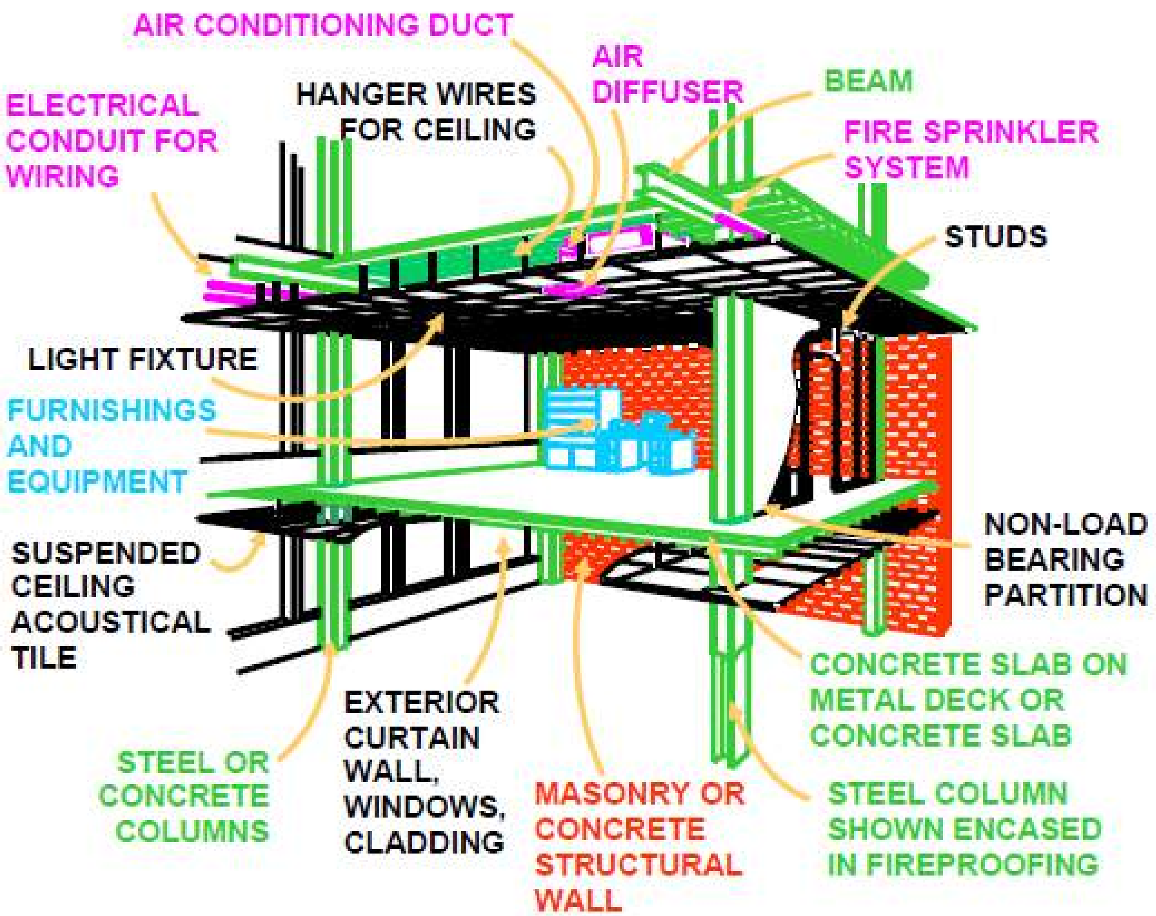
- Building Contents

Vending Machines

- Building Contents

Cabinets and Shelving

Classification of Nonstructural Components

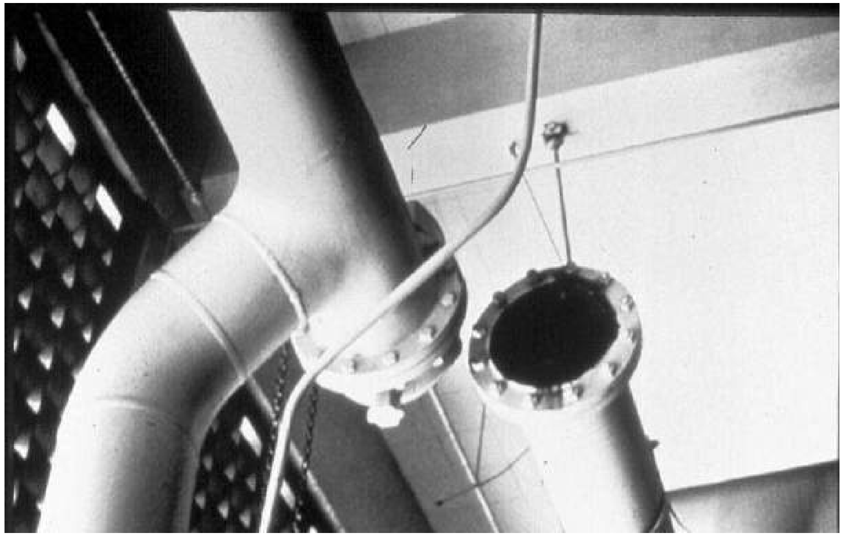


Architectural Components
Building Utility Systems
Building Contents

Structural Components

Source: FEMA 74

HOW DID THEY PERFORM IN PAST EARTHQUAKES?



Emilia Romagna 2012

Seattle 2001

Collapse of an unreinforced masonry parapet

Rupture of water line due to the shifting of a storage tank

Failed bookshelves in a library

Failures of suspended lighting fixtures in an office building

Cracking of heavy masonry partition walls

Boarded shattered windows in control tower of Sea-Tac Airport


EUCENTRE - European Centre for Training and Research in Earthquake Engineering

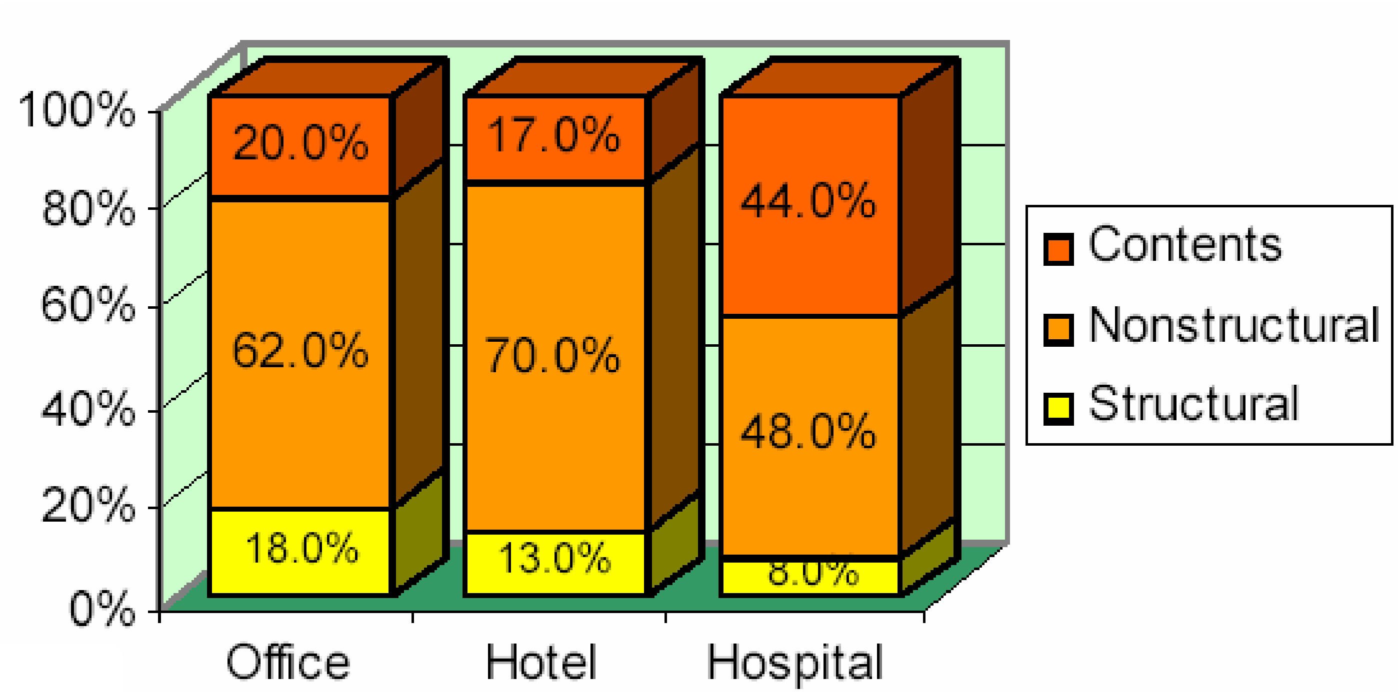

UNIVERSITY AT BUFFALO
CIVIL, STRUCTURAL AND ENVIRONMENTAL ENGINEERING
SEESL
STRUCTURAL ENGINEERING AND EARTHQUAKE SIMULATION LABORATORY
2009 - 2011


So... What's Shaking ?


EUCENTRE - European Centre for Training and Research in Earthquake Engineering

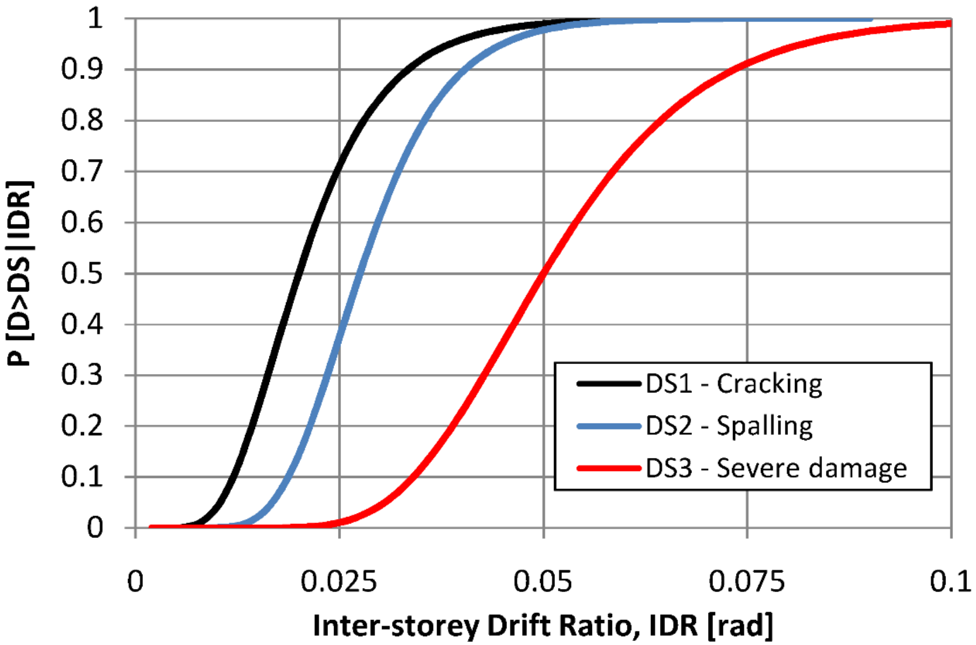
Importance of Considering Nonstructural Components in Seismic Design

- Nonstructural components represent the major portion of the total investment in typical buildings



ıg ı. Investments in building construction (Miranda 2003)

- Damage to nonstructural components occurs at seismic intensities much lower than those required to produce structural damage
  - Steel moment-resisting frames yield at story drifts beyond 1% while gypsum partition walls show significant crack at drifts as low as 0.25%
  - In many past earthquakes, losses from damage to nonstructural building components have exceeded losses from structural damage.



Damage State	Description	θ_{med}^{*} [rad]	β^{**}
DS1	Beams or joints exhibit residual crack widths > 1.5mm. No significant spalling.	0.02	0.4
DS2	Spalling of cover concrete. Transverse reinforcement exposed.	0.0275	0.3
DS3	Exposed longitudinal reinforcement. Core crushing possible. Buckling or rupture of reinforcement may occur.	0.05	0.3

*Median inter-storey drift **Lognormal standard deviation

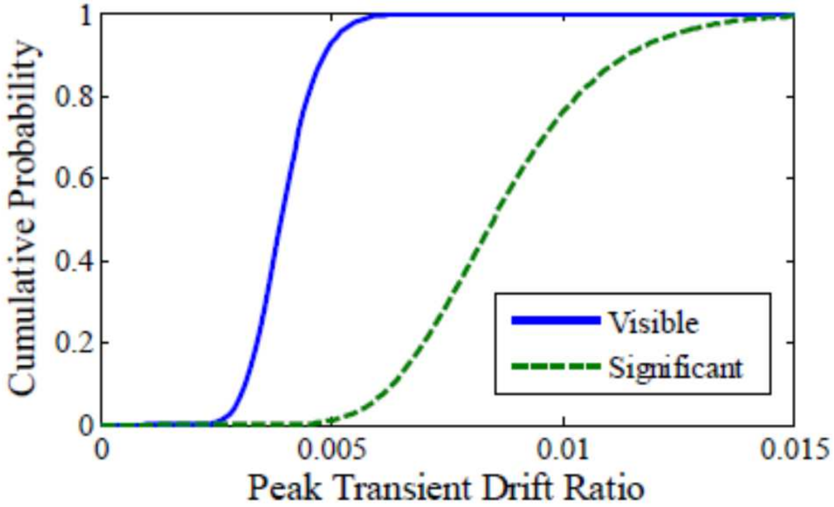
Direct losses

Sample fragility function (left) and damage state parameters (right) for a modern interior RC beam-column joint

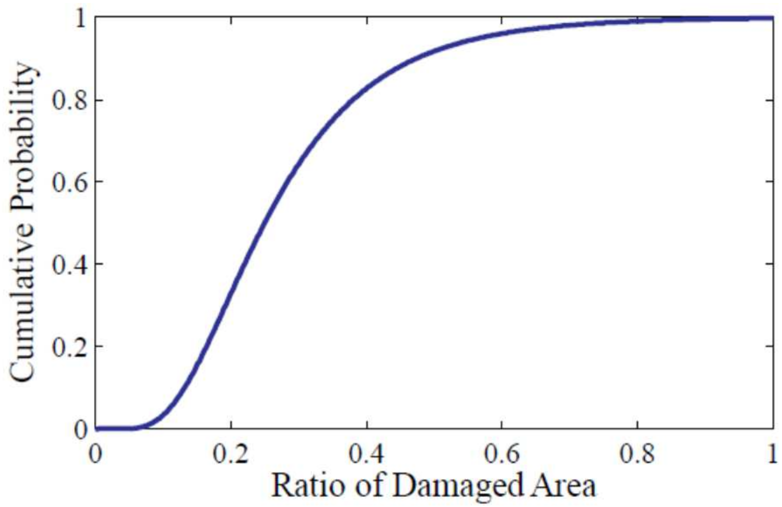
(Values taken from ATC 58)

Causes of Seismic Damage to Nonstructural Components

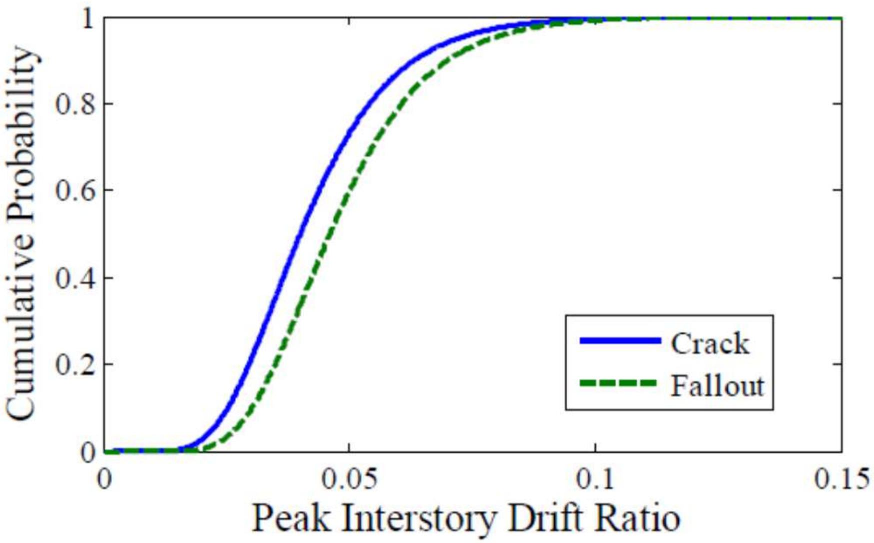
- Earthquake ground shaking has three primary effects on nonstructural elements in buildings:
  - Inertial Effects
  - Distortions imposed on nonstructural components
  - Separation or pounding at the interface between components and structures
  - Nonstructural interaction



gypsum drywall partitions

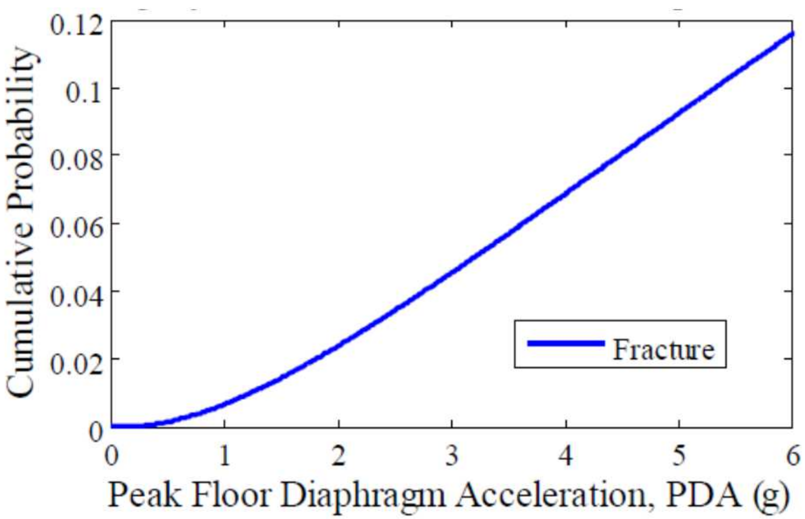


interior paint

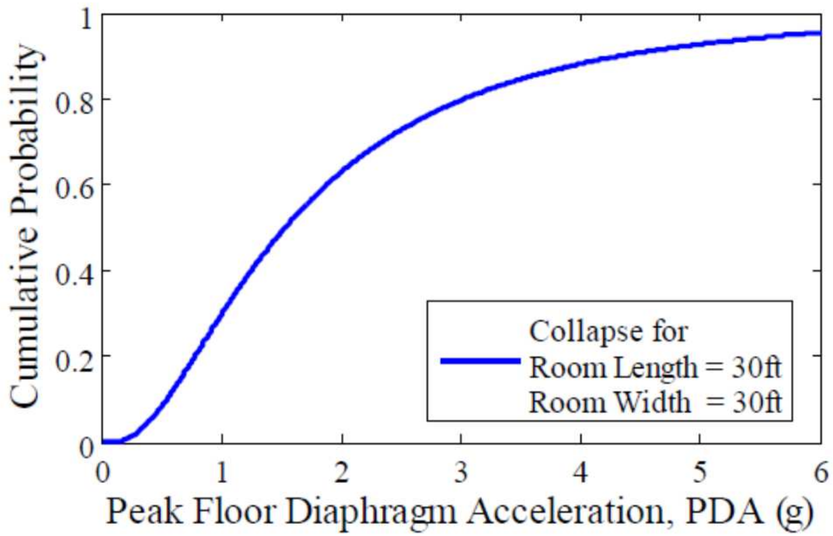


exterior glazing

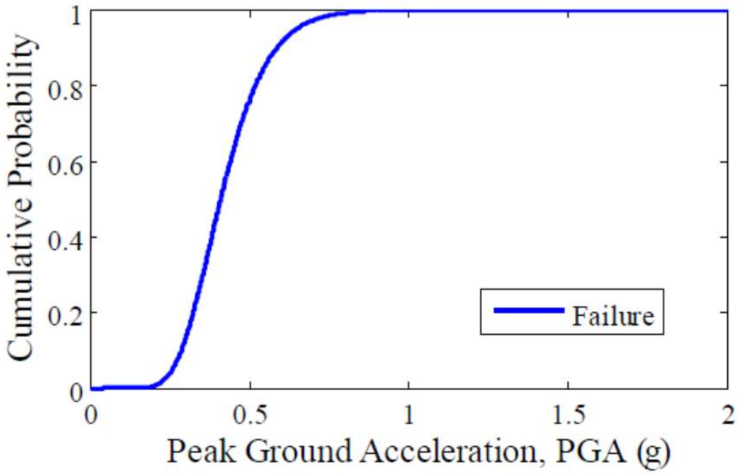
Fragility functions (Mitrani-Reiser 2007)



automatic sprinkler systems

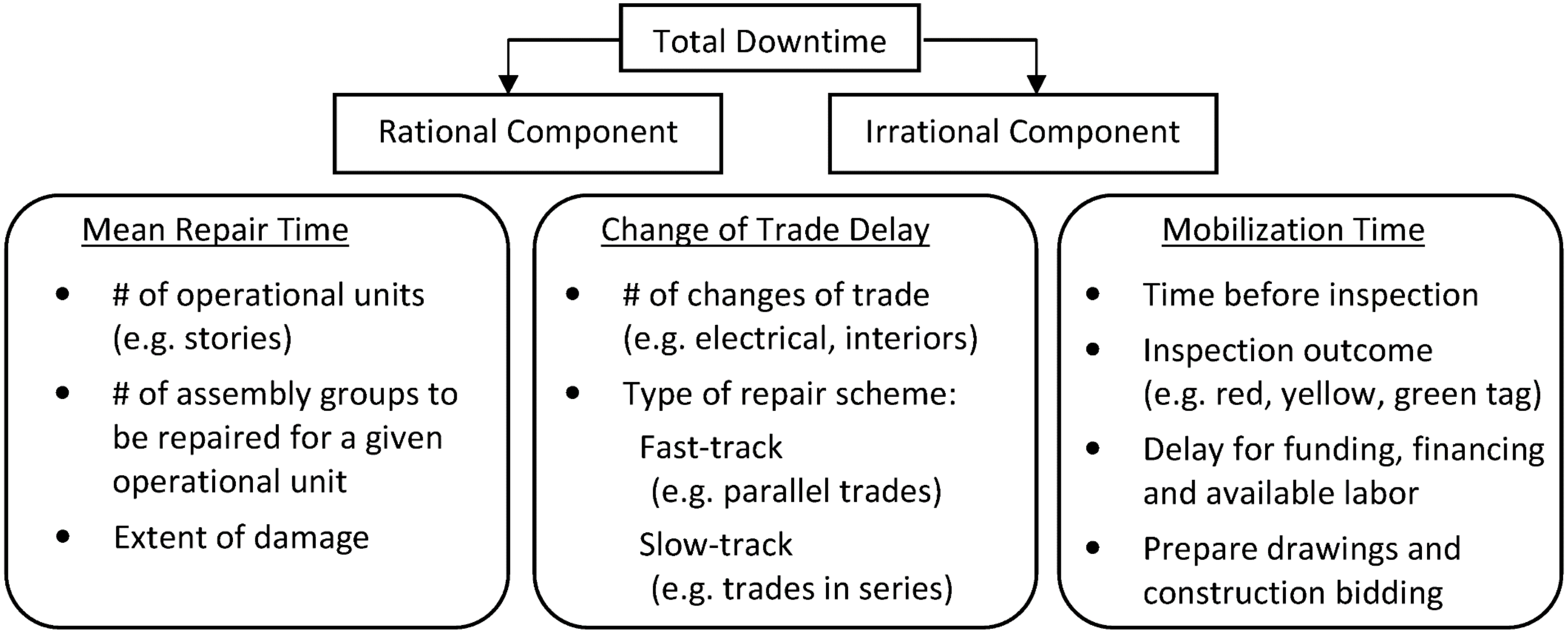


acoustical ceiling



hydraulic elevators

Fragility functions (Mitrani-Reiser 2007)



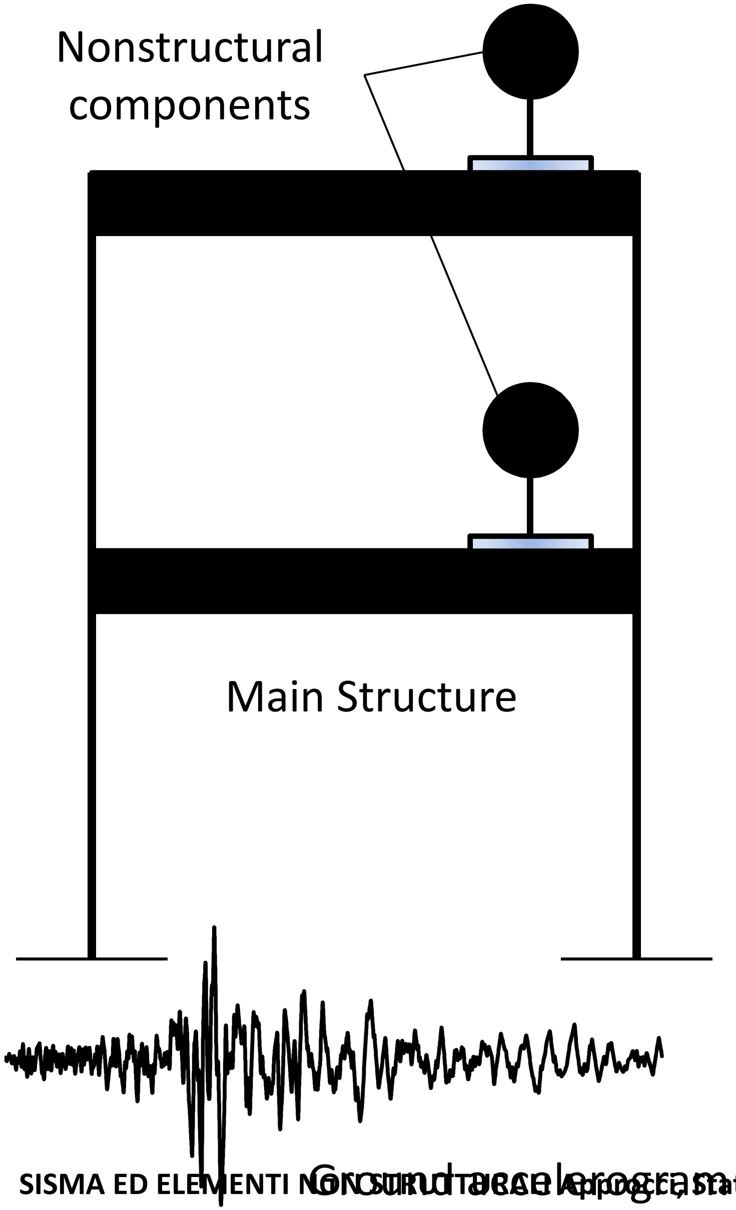
Indirect losses

Various aspects that can contribute to the downtime of a building following a seismic event

Analysis Methods

- Direct Analysis Method
  - Complete modeling of structural and nonstructural components
  - Ground input motions
- Cascading Analysis Method
  - Uncoupled analyses of structural and nonstructural components
  - Dynamic properties and floor responses of the primary structure are first estimated neglecting interaction with the nonstructural components
  - Structural response at the attachment level is then considered as the input motion for the estimation of the response of the nonstructural component.
  - Most popular cascading approach: Floor Response Spectrum (FRS) Method

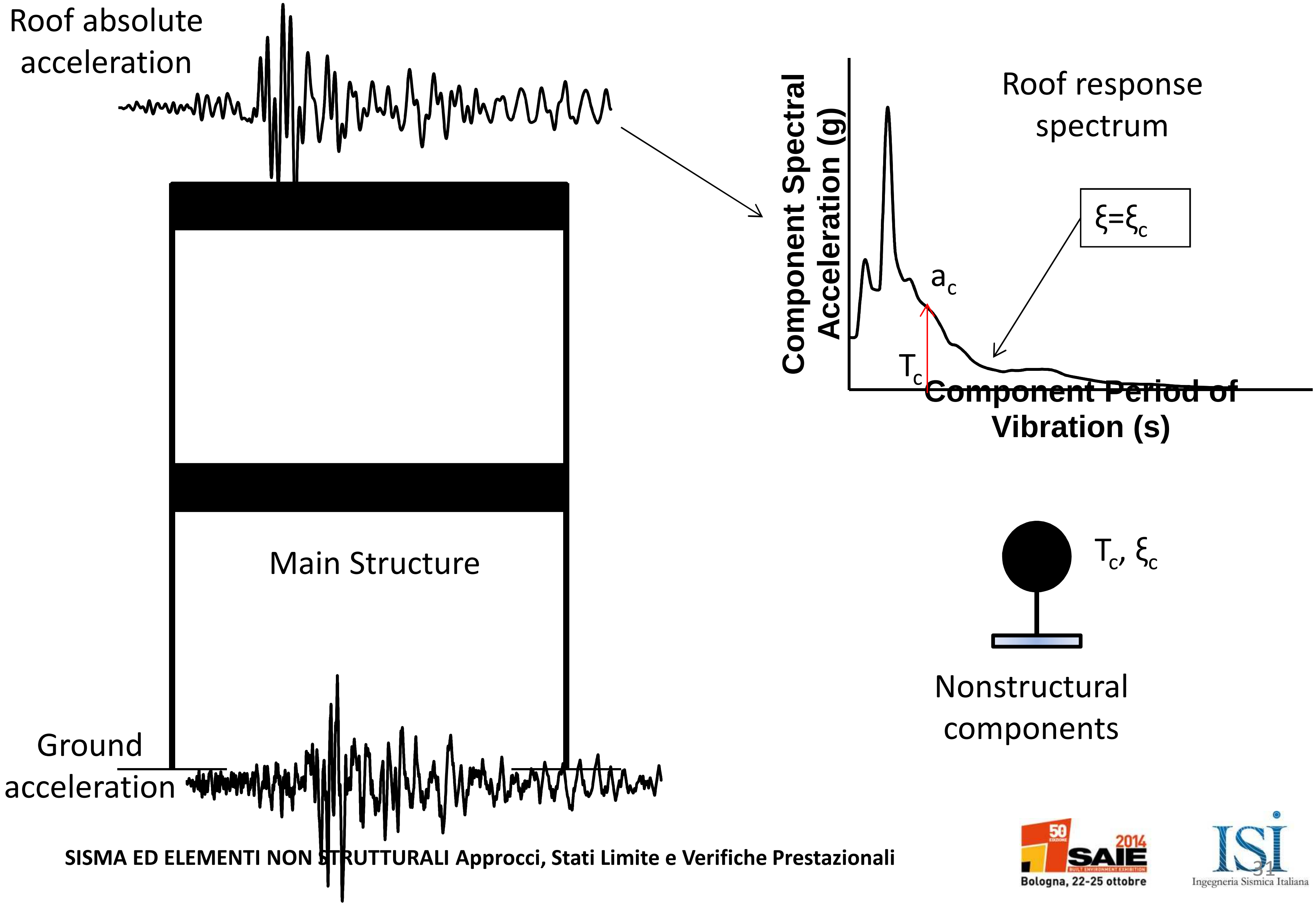
Direct analysis approach



Ground accelerogram

- Cannot be used at preliminary design stages;
- High level of complexity, experience on the side of the designer is required;
- Models characterized by significant number of degrees of freedom;
- Complications connected to the mutual interaction between structure and components;

Floor response spectra approach



Floor response spectra: direct generation

Approximate procedure: direct generation of floor response spectra approach.

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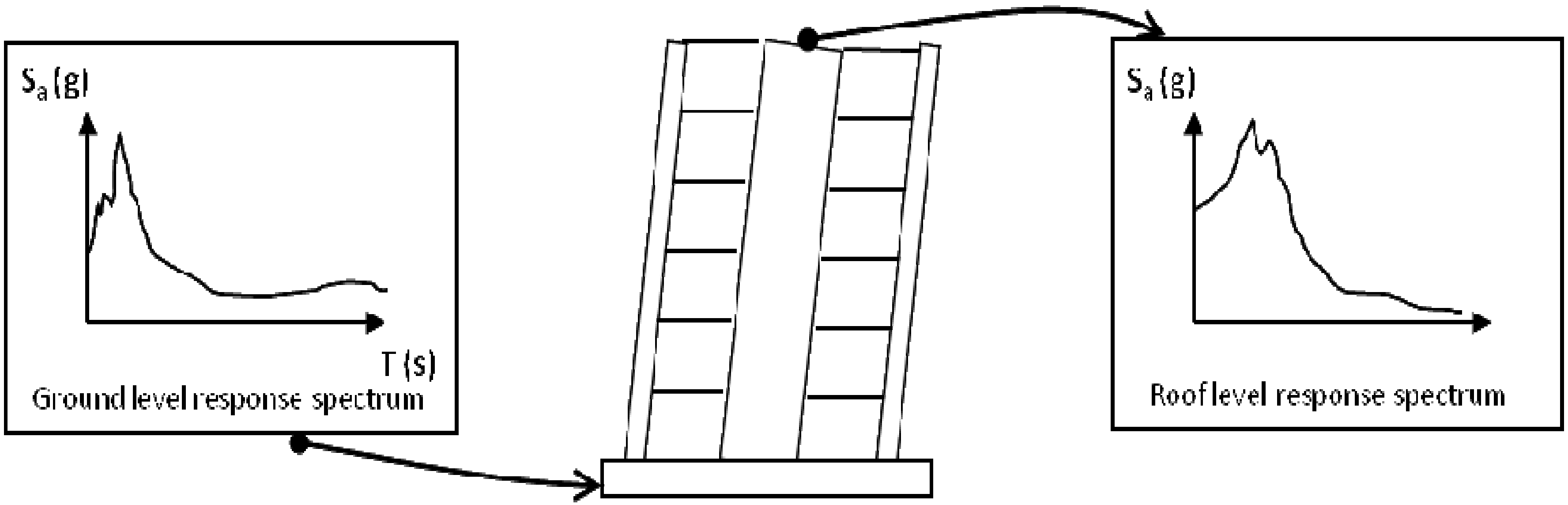
Procedure adopted by current international codes
(mainly empirical expressions are provided)



Objectives of Seismic Design Requirements for Nonstructural Components

- Primary intent: maintain life safety
- Achieved by:
  - limiting large displacements of nonstructural components
  - design of proper anchorage to the main structure
  - minimizing the potential for internal damage suffered by nonstructural components, particularly in critical facilities.

EUROCODE 8 APPROACH

$$F_a = \frac{S_a \gamma_a W_a}{q_a}$$

$$S_a = \alpha S \left[3 \frac{\left(1 + \frac{z}{H}\right)}{\left(1 + \left(1 - \frac{T_a}{T_1}\right)^2\right)} - 0.5 \right]$$

- W_a is the component seismic weight
- γ_a is the importance factor (1 or 1.5)
- q_a is the behaviour factor (1 or 2)
- α is the ratio of the design ground acceleration on type A ground, to the acceleration of gravity;
- S is the soil factor
- T_a is the fundamental vibration period of the non-structural element;
- T_1 is the fundamental vibration period of the building in the relevant direction.
- z is the height of the non-structural element above the level application of the seismic action;
- H is the building height measured from the foundation or from the top of a rigid basement.

U.S. code ASCE7-05 (2005) APPROACH

$$F_{ph} = \frac{0.4 a_p S_{DS}}{\left(\frac{R_p}{I_p}\right)} \left(1 + 2\frac{z}{h}\right) W_p$$

$$S_{DS} = \frac{2}{3} S_{MS}$$

$$S_{MS} = F_a S_s$$

$$F_a = f(S_s, site\ class)$$

$$0.3 S_{DS} I_p W_p \leq F_{ph} \leq 1.6 S_{DS} I_p W_p$$

- S_S determined from the mapped 0.2s spectral response acceleration
- a_p component amplification factor (1 to 2.5)
- S_{DS} design earthquake spectral response acceleration at short period
- R_p component response modification factor (1 to 12)
- I_p component importance factor (1 or 1.5)
- z height of the structure at point of attachment of non structural component
- h average roof height of structure relative to the base elevation
- W_p operating weight of nonstructural component.

Nonstructural components are consided rigid if their period of vibration is < 0.06s

New Zealand standard NZS1170.5 APPROACH

$$C_p(T_p) = C(0)C_{Hi}C_iT_p$$

$$F_{ph} = C_p(T_p)C_{ph}R_pW_p \leq 3.6W_p$$

$$C(T) = C_h(T)ZRN(T,D)$$

$$C_{Hi}\begin{cases} = 1 + \frac{h_i}{6} & h_i < 12m \\ = 1 + 10\frac{h_i}{h_m} & h_i \leq 0.2h_m \\ = 3 & h_i \geq 0.2h_m \end{cases}$$

$$C_i(T_p)\begin{cases} = 2 & T_p \leq 0.75s \\ = 0.5 & T_p \geq 1.5s \\ = 2(1.75 - P_p) & 0.75s < T_p < 1.5s \end{cases}$$

- C_{ph} horizontal seismic coefficient
- R_p part risk factor equal (importance factor)
- W_p weight of the non-structural element
- C_{ph} part horizontal component (equal to 1 if component responds elastically)
- C_p horizontal design coefficient
- C(0) elastic hazard spectrum @ T=0s
- Z hazard factor
- R return period factor
- C_{hi} floor height coefficient
- $C_i(T_p)$ dynamic amplification coefficient

Comparison of equivalent static design forces

$$F_a = \frac{\alpha S}{\left(\frac{q_s}{\gamma_s}\right)}\left[3\frac{\left(1+\frac{z}{H}\right)}{\left(1+\left(1-\frac{T_a}{T_1}\right)^2\right)} - 0.5\right]W_p$$

EROCODE 8

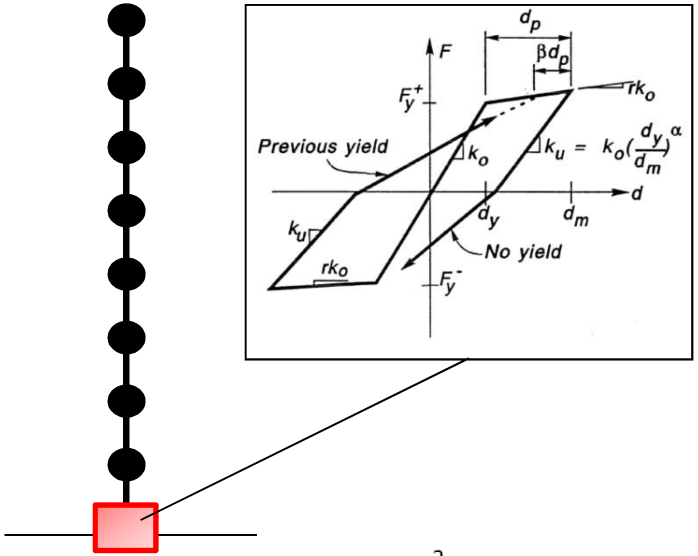
$$F_{ph} = \frac{0.4 a_p S_{DS}}{\left(\frac{R_p}{I_p}\right)}\left(1+2\frac{z}{h}\right)W_p$$

ASCE7-05 (2005)

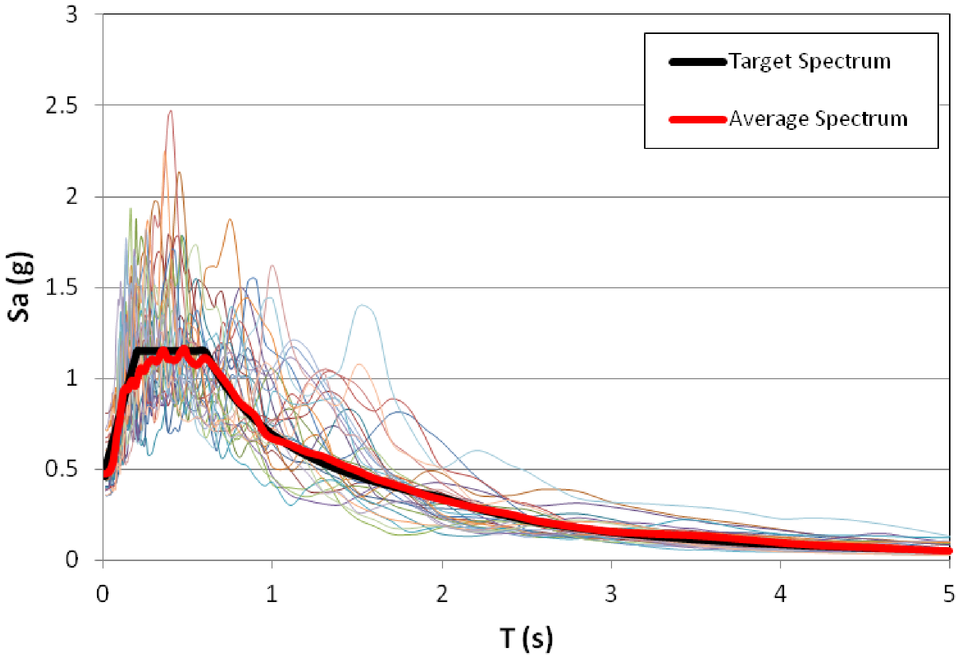
$$F_{ph} = C_p(T_p)C_{ph}R_pW_p$$

NZS1170.5

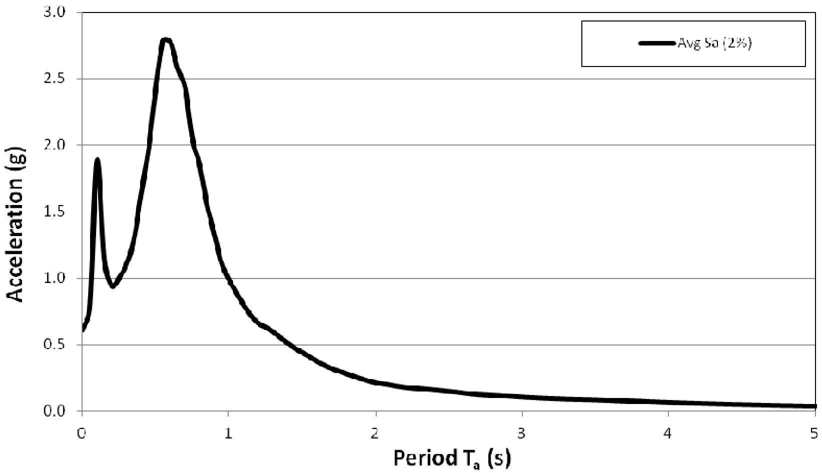
Shortcomings of code approaches

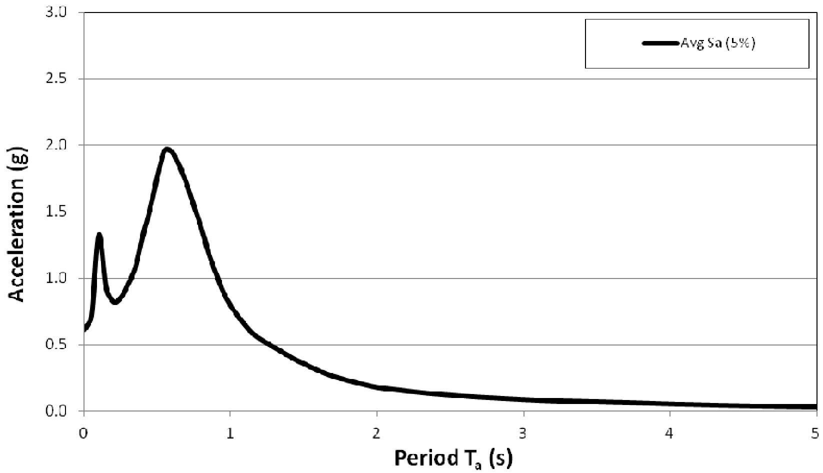


- 2D RUAUMOKO models;
- TAKEDA hysteretic rule for plastic hinges;
- Tangent stiffness proportinal damping;
- Plastic hinges length calculated in line with expressions provided by Paulay and Priestley (1992);

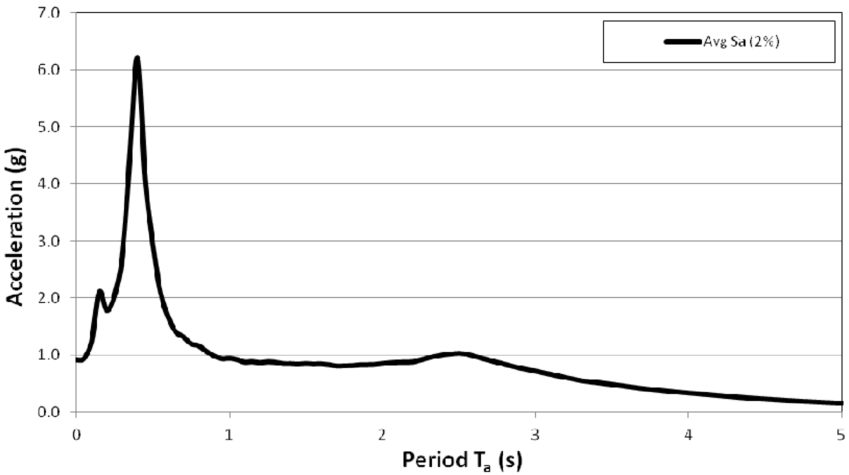


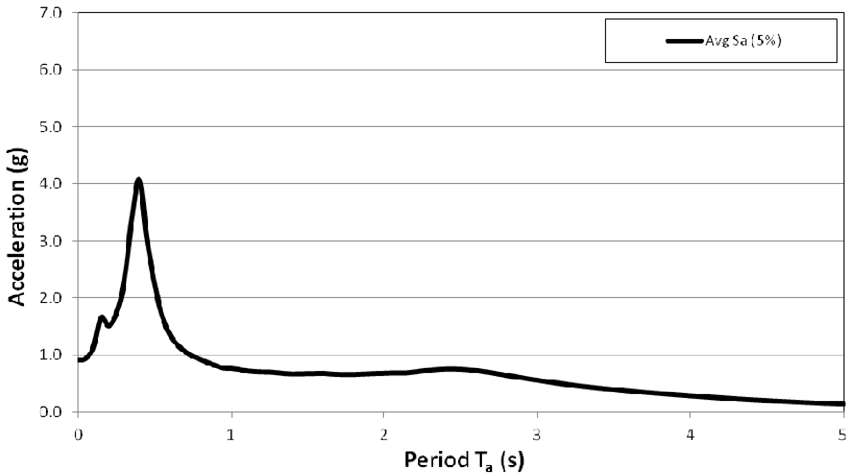
NLTH Analysis vs code recommendations





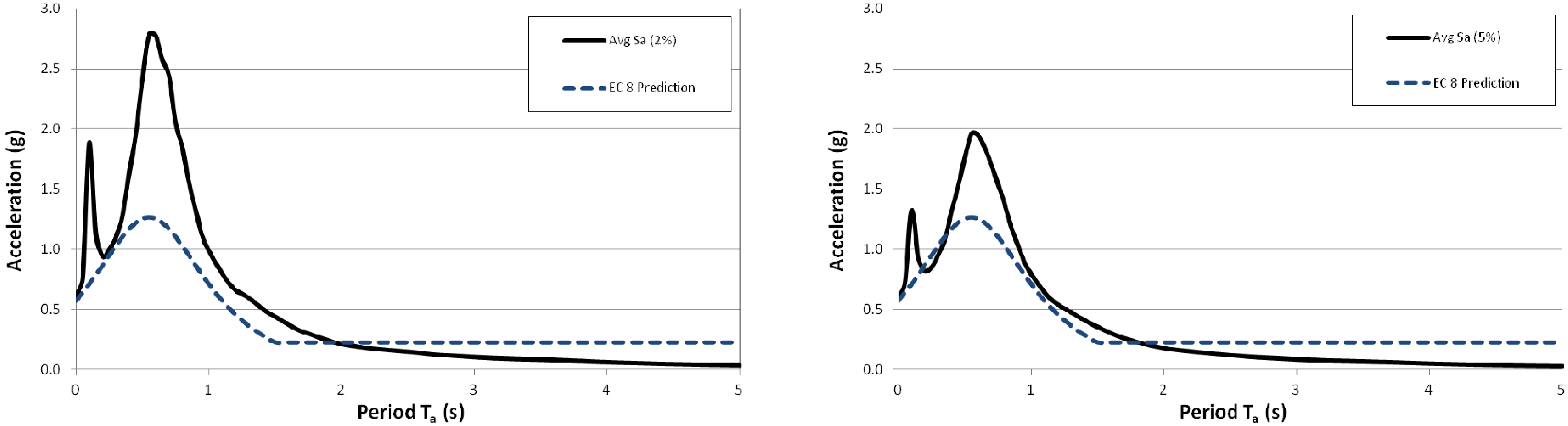
Comparison of roof level response spectra at 2% (left) and 5% (right) damping predicted via NLTH analyses of an 8-storey structure subject to accelerograms compatible with the EC8 spectrum at a PGA = 0.2 g



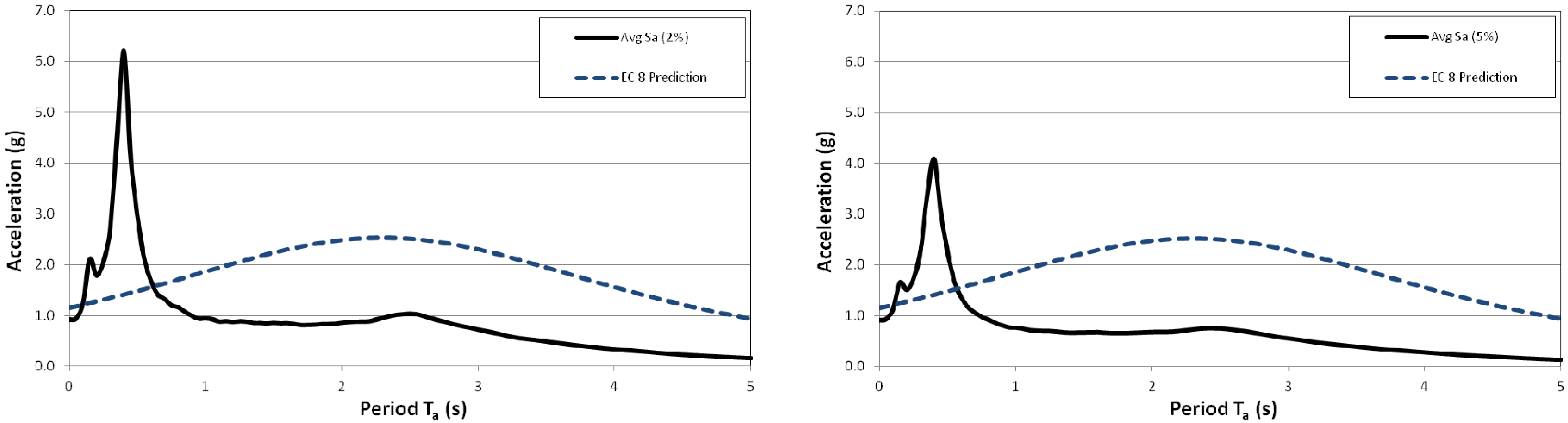


Comparison of roof level response spectra at 2% (left) and 5% (right) damping predicted via NLTH analyses of an 20-storey structure subject to accelerograms compatible with the EC8 spectrum at a PGA = 0.4 g

NLTH Analysis vs code recommendations



Comparison of roof level response spectra at 2% (left) and 5% (right) damping predicted via seismic code approaches and via NLTH analyses of an 8-storey structure subject to accelerograms compatible with the EC8 spectrum at a PGA = 0.2 g



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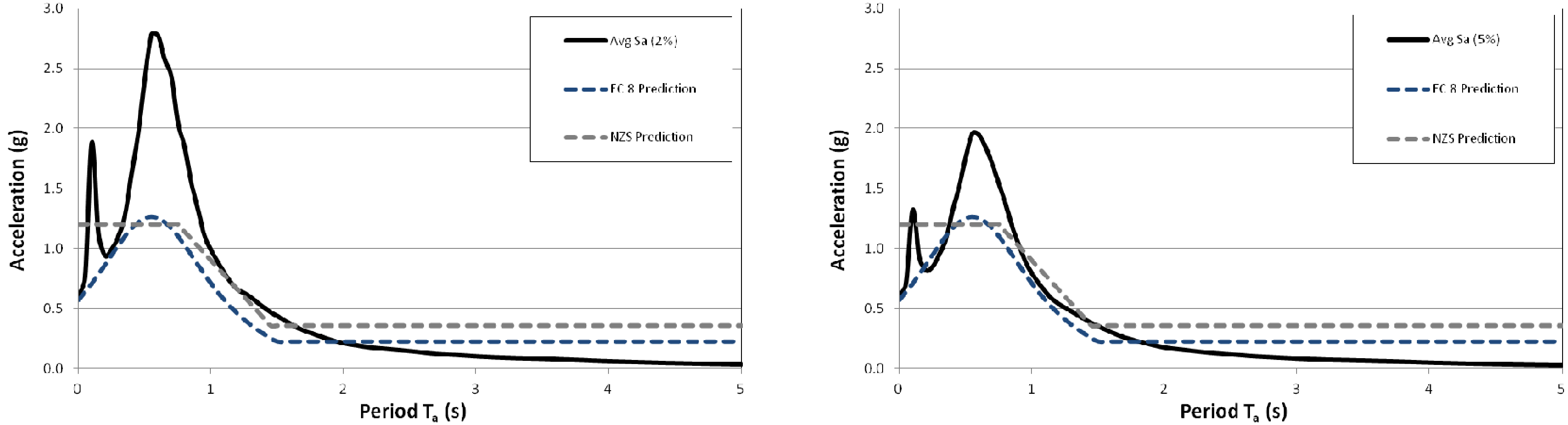
NLTH Analysis vs code recommendations



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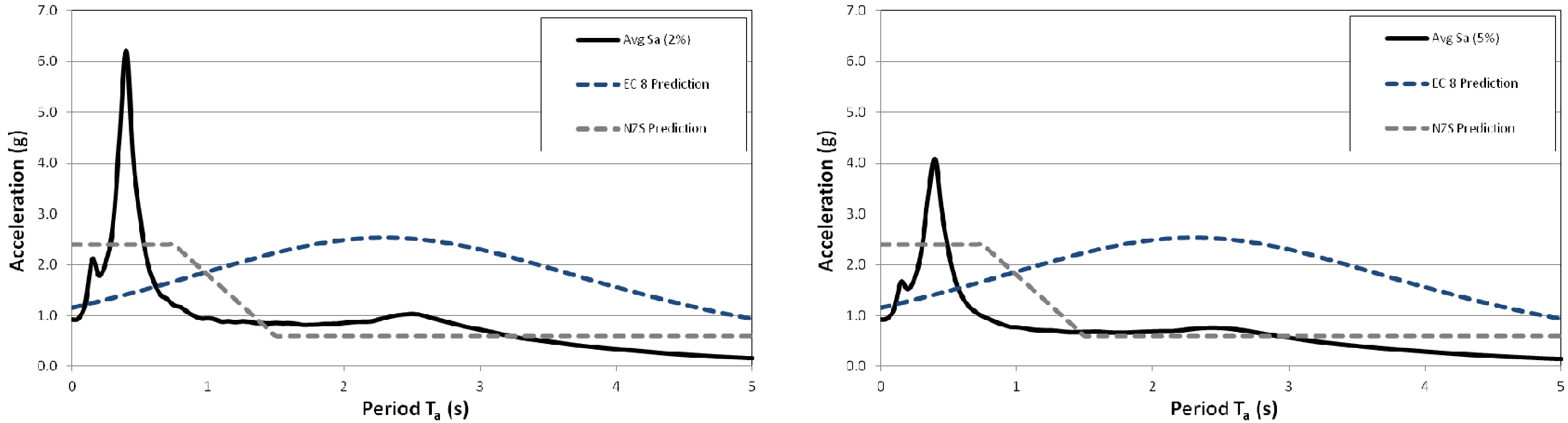

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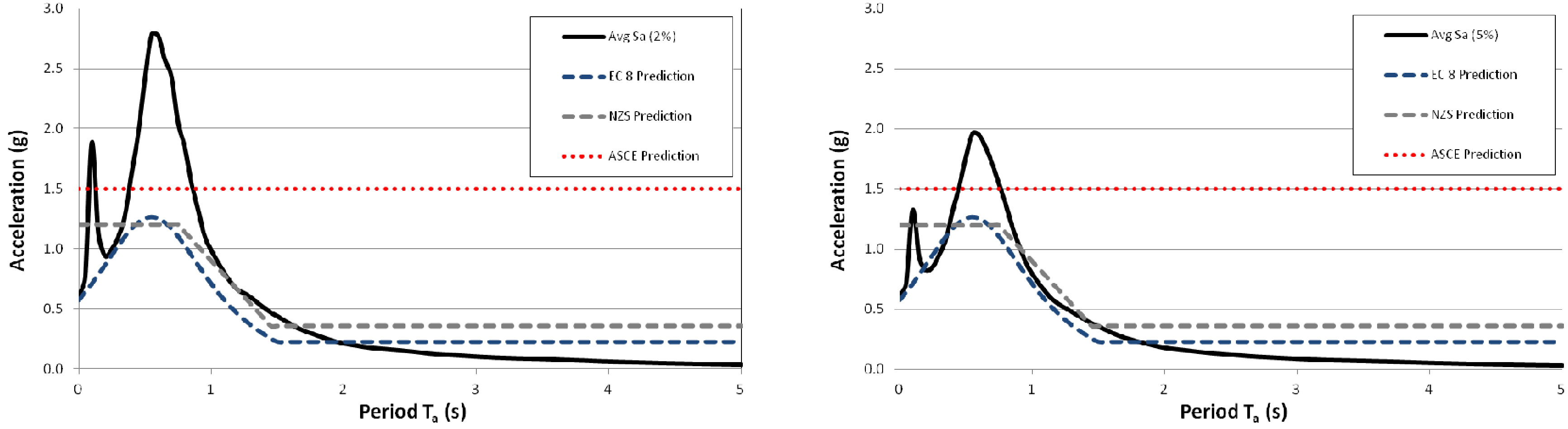

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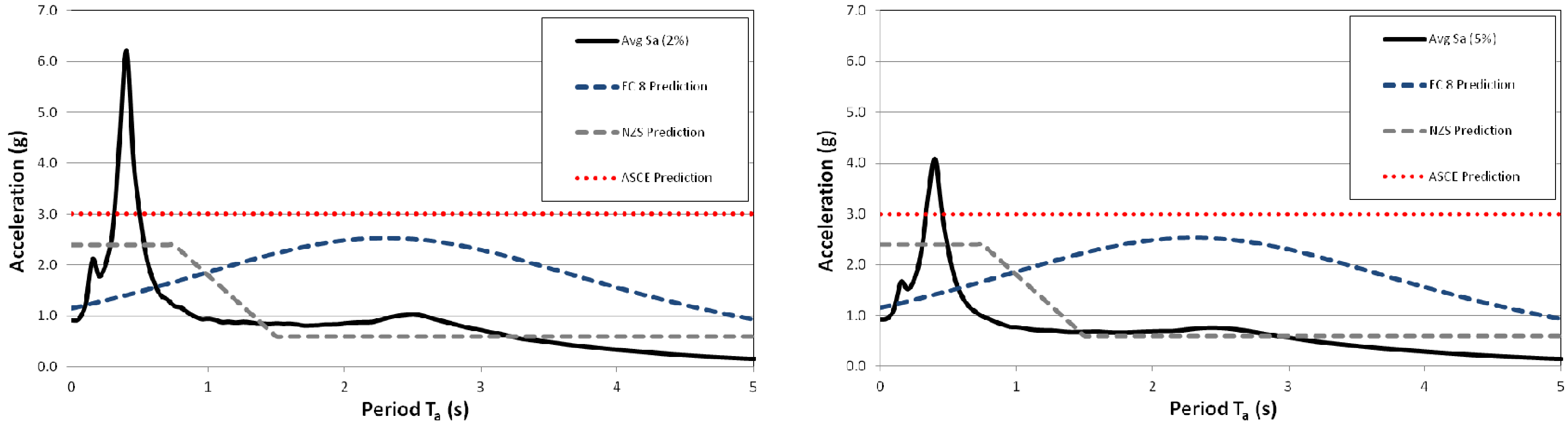


Comparison of roof level response spectra at 2% (left) and 5% (right) damping predicted via seismic code approaches and via NLTH analyses of an 20-storey structure subject to accelerograms compatible with the EC8 spectrum at a PGA = 0.4 g

Main Influence on floor response spectra

- Properties of the main system (period, damping, linear, nonlinear, degrees of freedom …);

- Properties of the ground motion (frequency content, duration, average amplitude …);

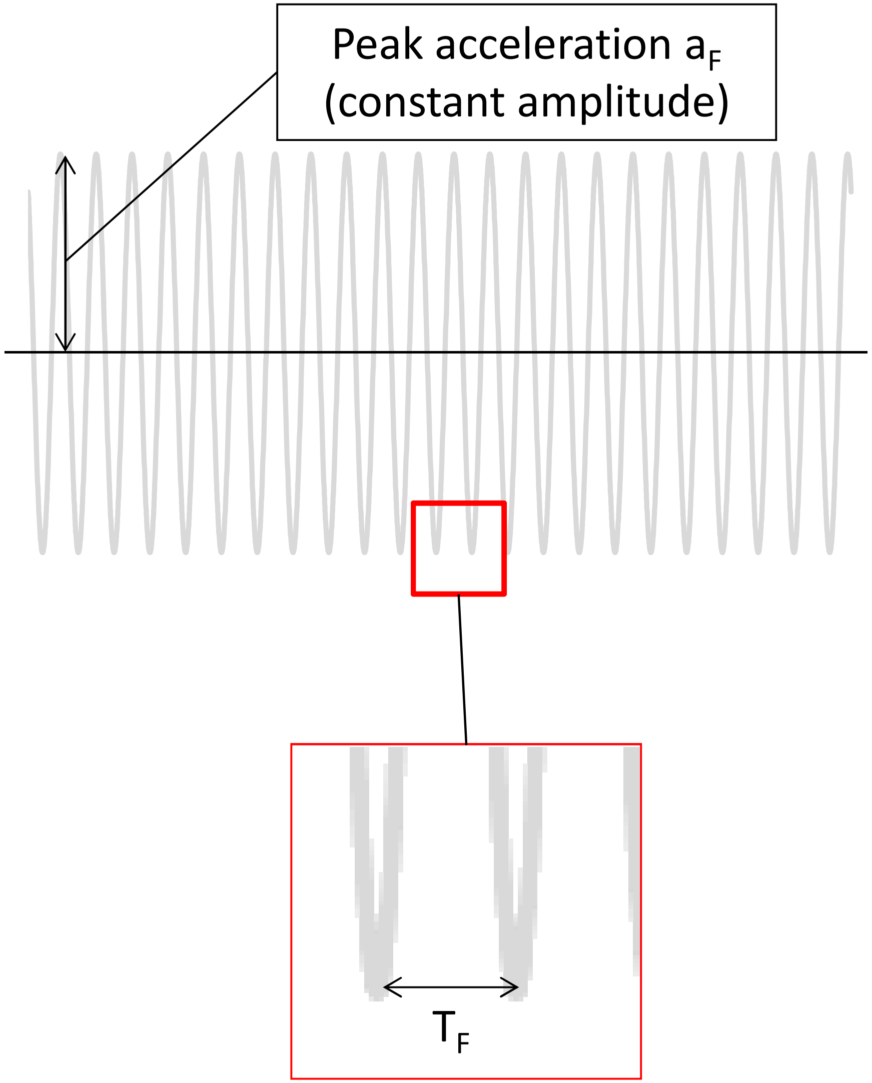
- Properties of the secondary element to be designed;

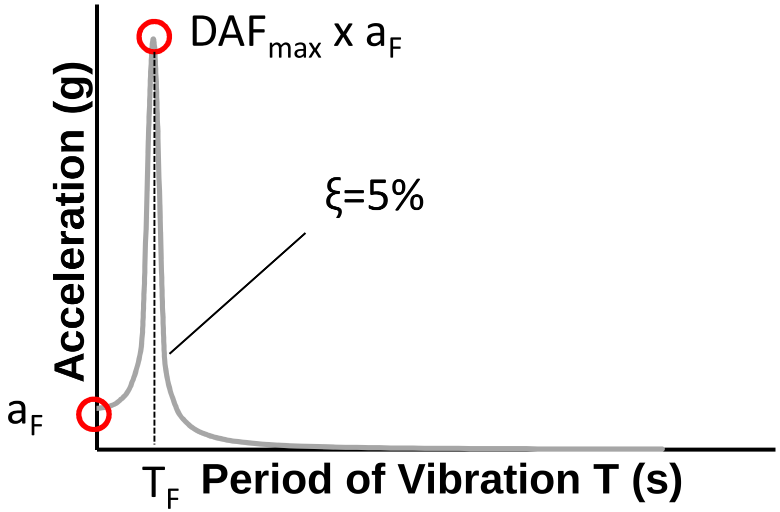
- Location of the component within the building;

TARGET:

Formulate a rational approach to generate floor response spectra directly from ground response spectra, accounting for the aspects listed above.

Reviewing harmonic acceleration case



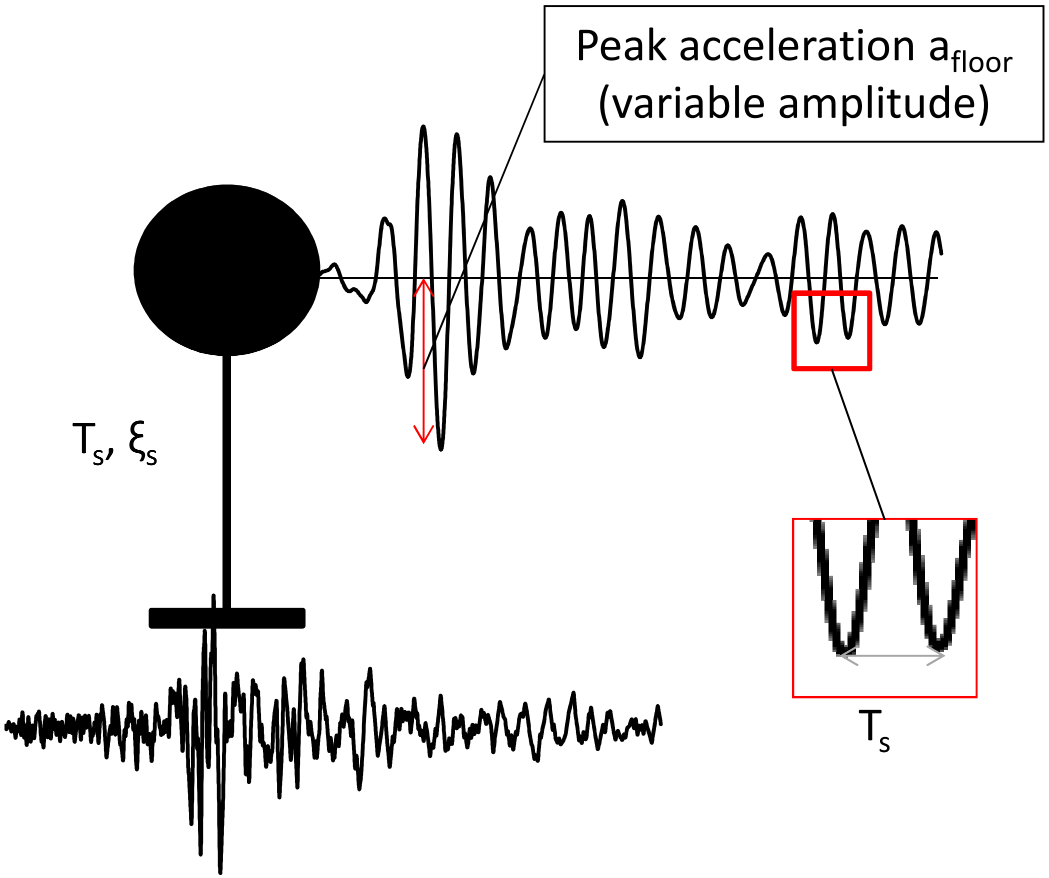


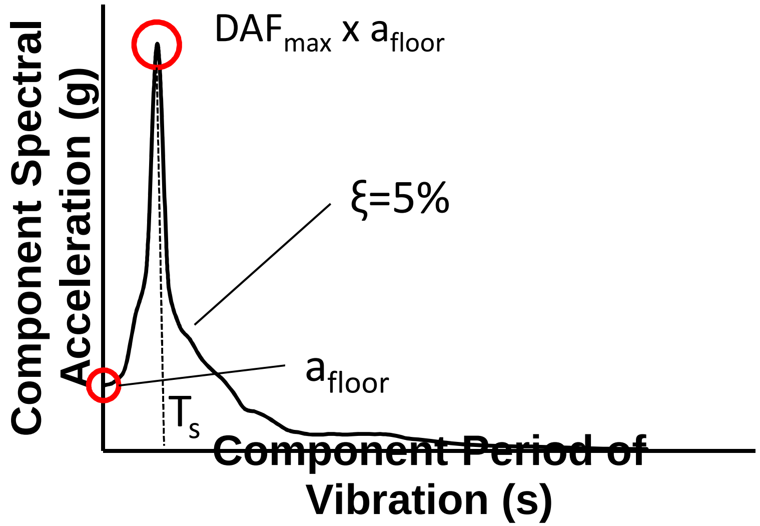
$$a_s = a_F DAF$$

$$DAF = \frac{1}{\sqrt{((1-\beta^2)^2 + (2\xi\beta)^2)}}$$

$$DAF_{max} = \frac{1}{2\xi} \qquad \beta = \frac{T_f}{T}$$

Elastic SDF supporting system

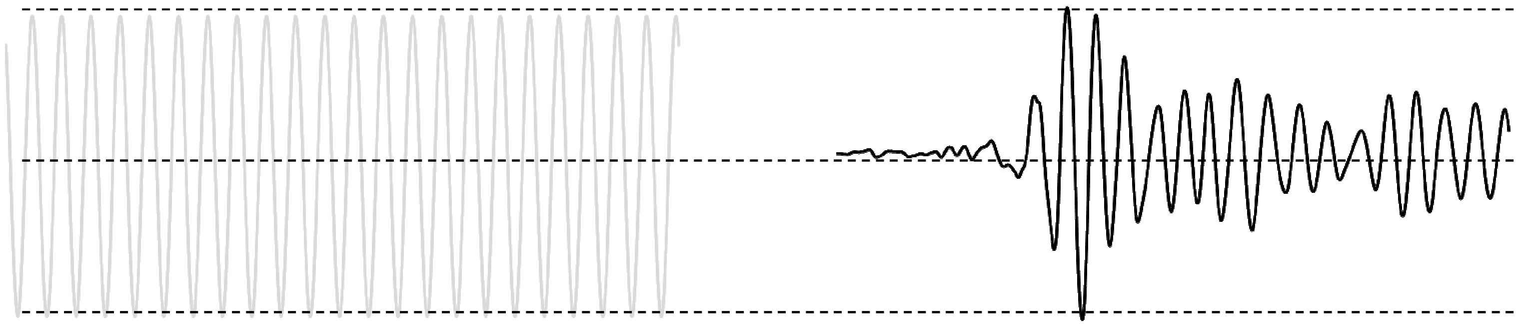




$$DAF_{max} \quad ?$$

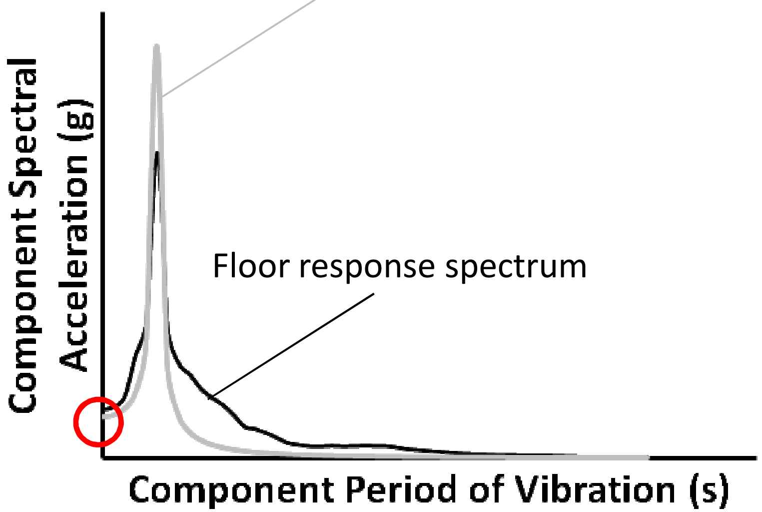
$$a_m = a_{floor} DAF \qquad DAF \quad ?$$

Elastic SDF supporting system



- Infinite duration;
- Constant amplitude;
- <u>Constant forcing frequency;</u>

- Finite duration;
- Variable amplitude;
- <u>Constant forcing frequency;</u>



Harmonic acceleration theory:

- Acceleration at T=0s properly estimated;
- Peak of the spectrum is correctly located on the x-axis but overestimated in terms of intensity;
- Decrasing branch of the spectrum drops too quickly

Elastic SDF supporting system

Achieved objectives:

- Peak of the spectrum can be localized on the x-axis if the period of vibration of the main system is known;
- The peak floor acceleration can be estimated from a ground spectrum if the main system behaves elastically

To be achieved:

- Calibrate a new expression to compute DAF_{max} ;
- Calibrate new expression for the decrasing branch of the spectrum;

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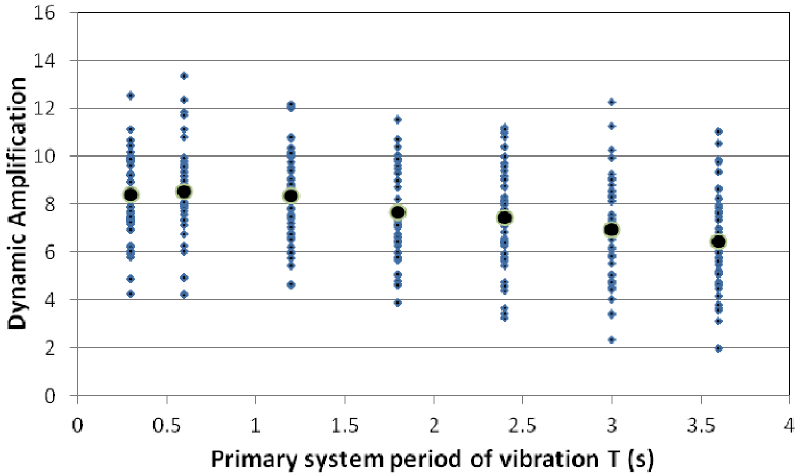
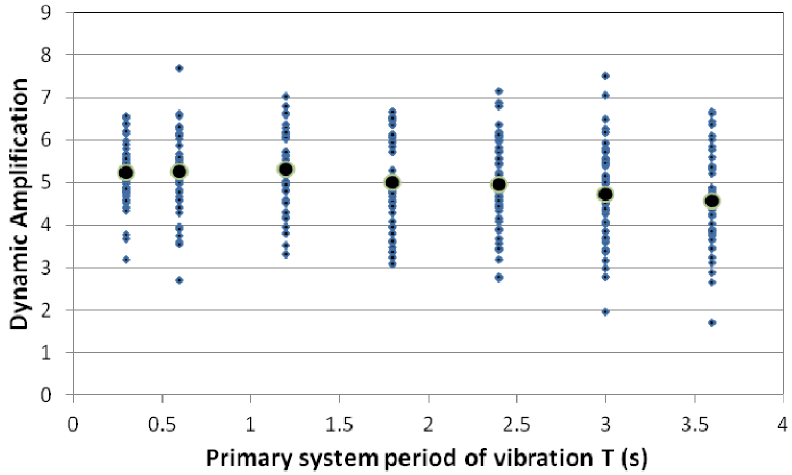
Parametric study via time history analysis:

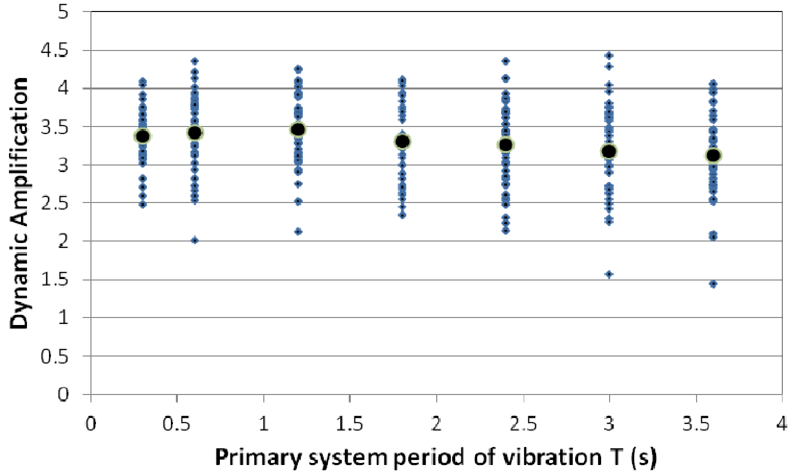
- 7 SDF case study structures with T = 0.3 to 3.6 seconds
- 50 ground motions compatible with the EC8 ground spectrum

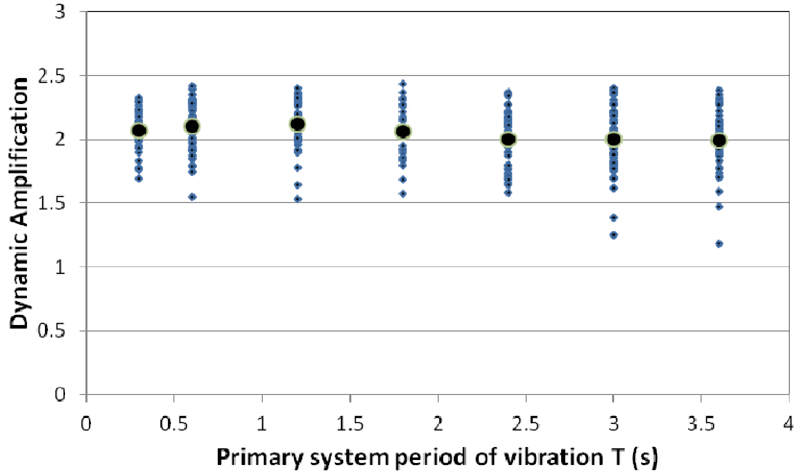
Calibration of a dynamic amplification factor





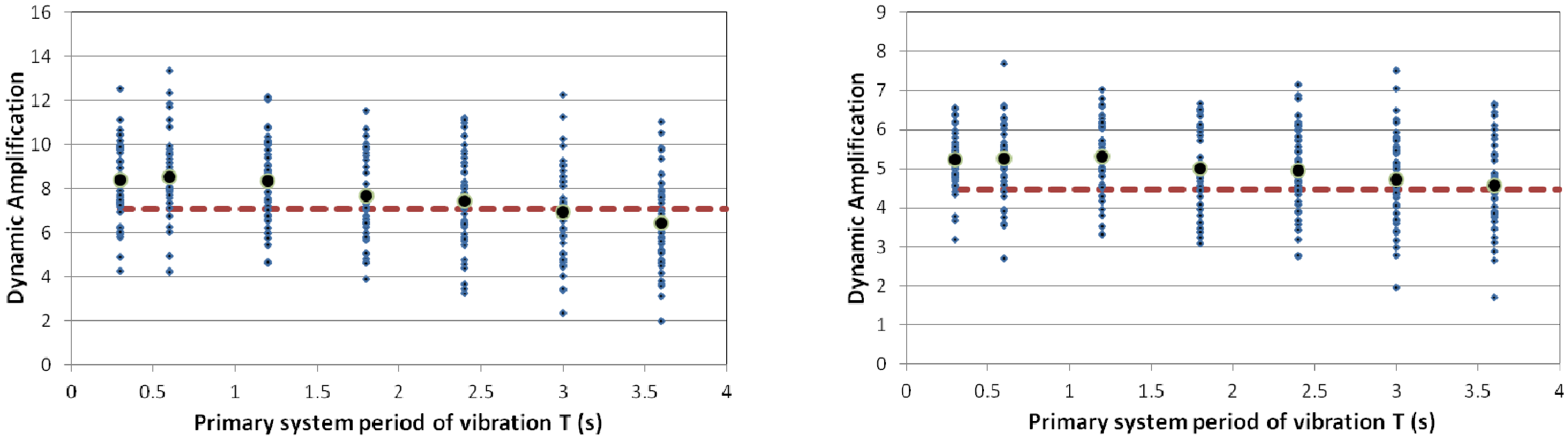
Maximum dynamic amplification factors at 2% (left) and 5% (right) damping calculated via NLTH analyses of 7 elastic SDF case study structures



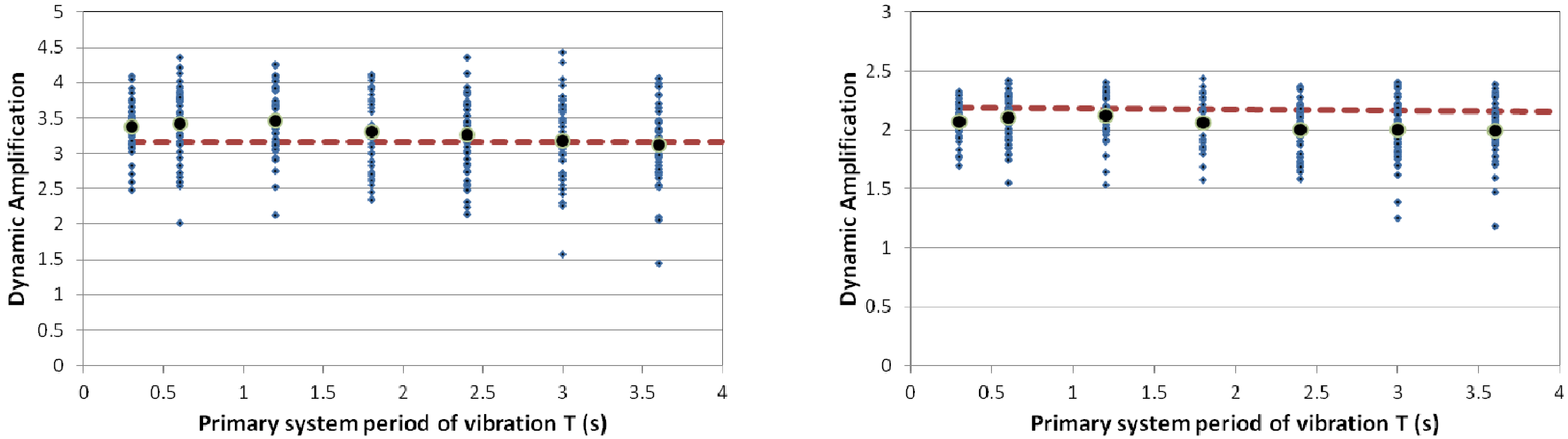


Maximum dynamic amplification factors at 10% (left) and 20% (right) damping calculated via NLTH analyses of 7 elastic SDF case study structures

Calibration of a dynamic amplification factor



Maximum dynamic amplification factors at 2% (left) and 5% (right) damping calculated via NLTH analyses of 7 elastic SDF case study structures compared to the values approximated by the proposed function



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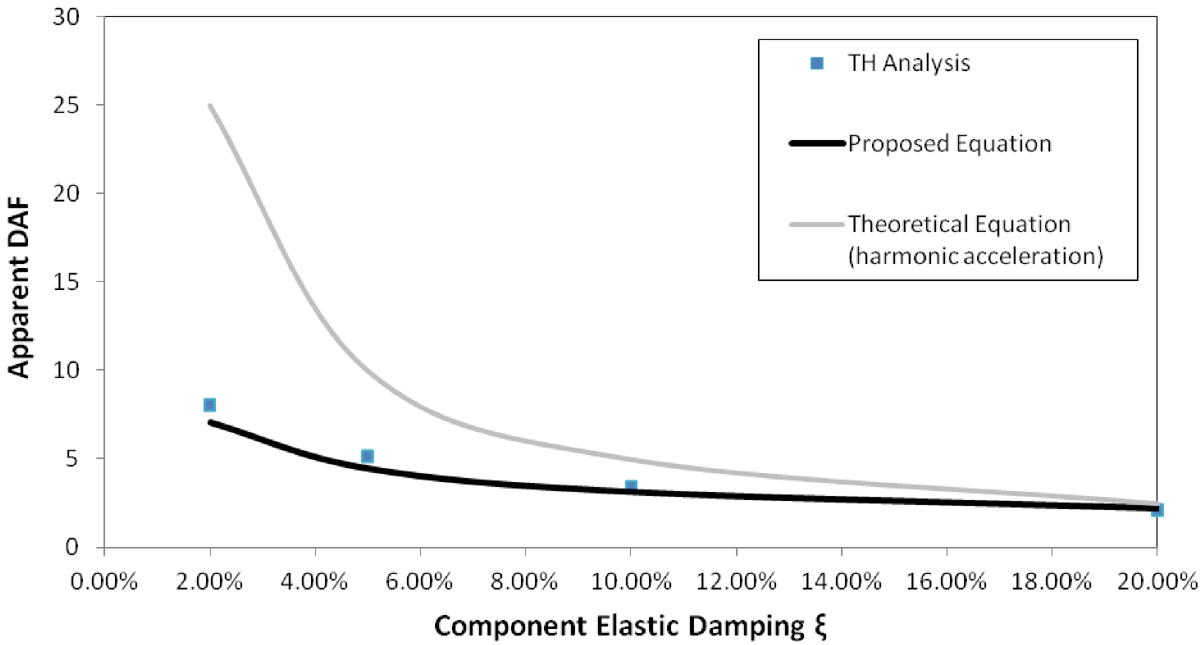
Calibration of a dynamic amplification factor



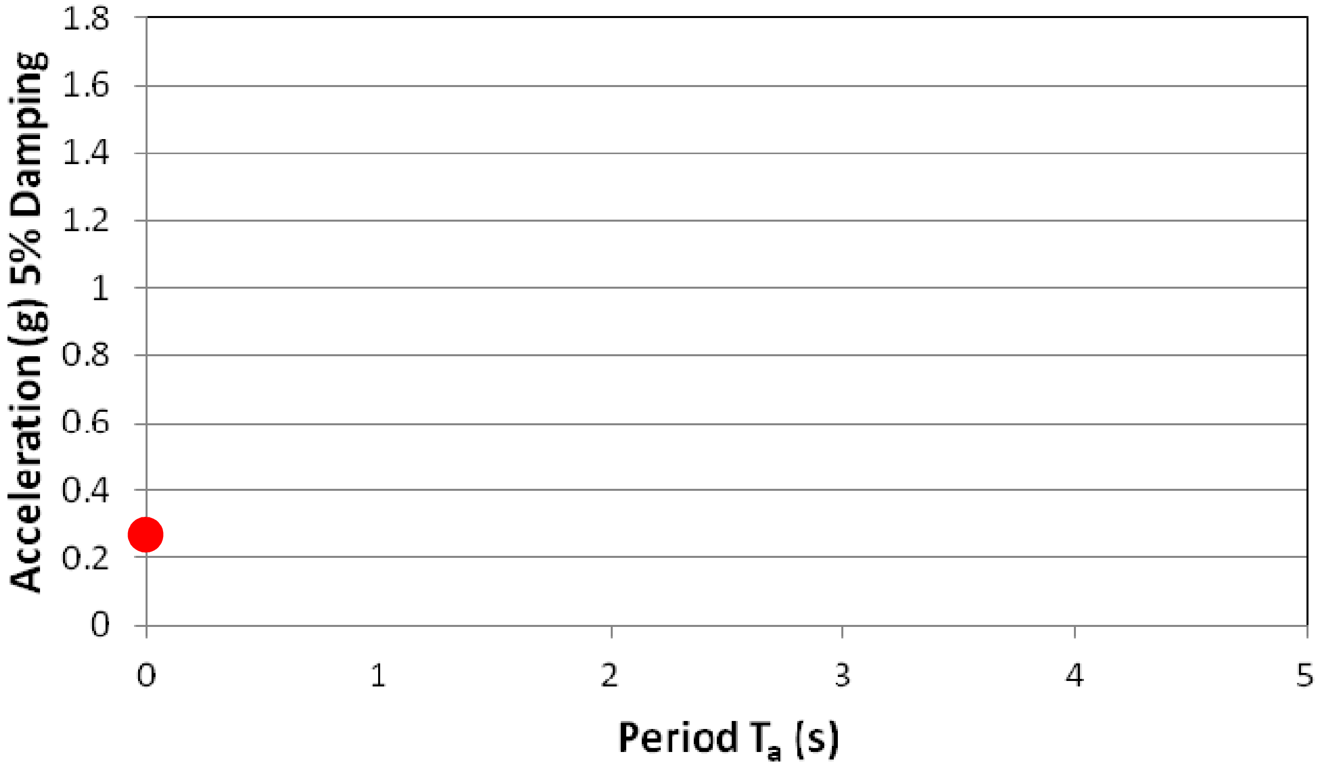
$$DAF_{max} = 1/\xi^{0.5}$$

The proposed equation was succesfully tested for:

- Near source ground motions (16 records);
- Long duration ground motions (12 records);

“Elastic” floor response spectra construction

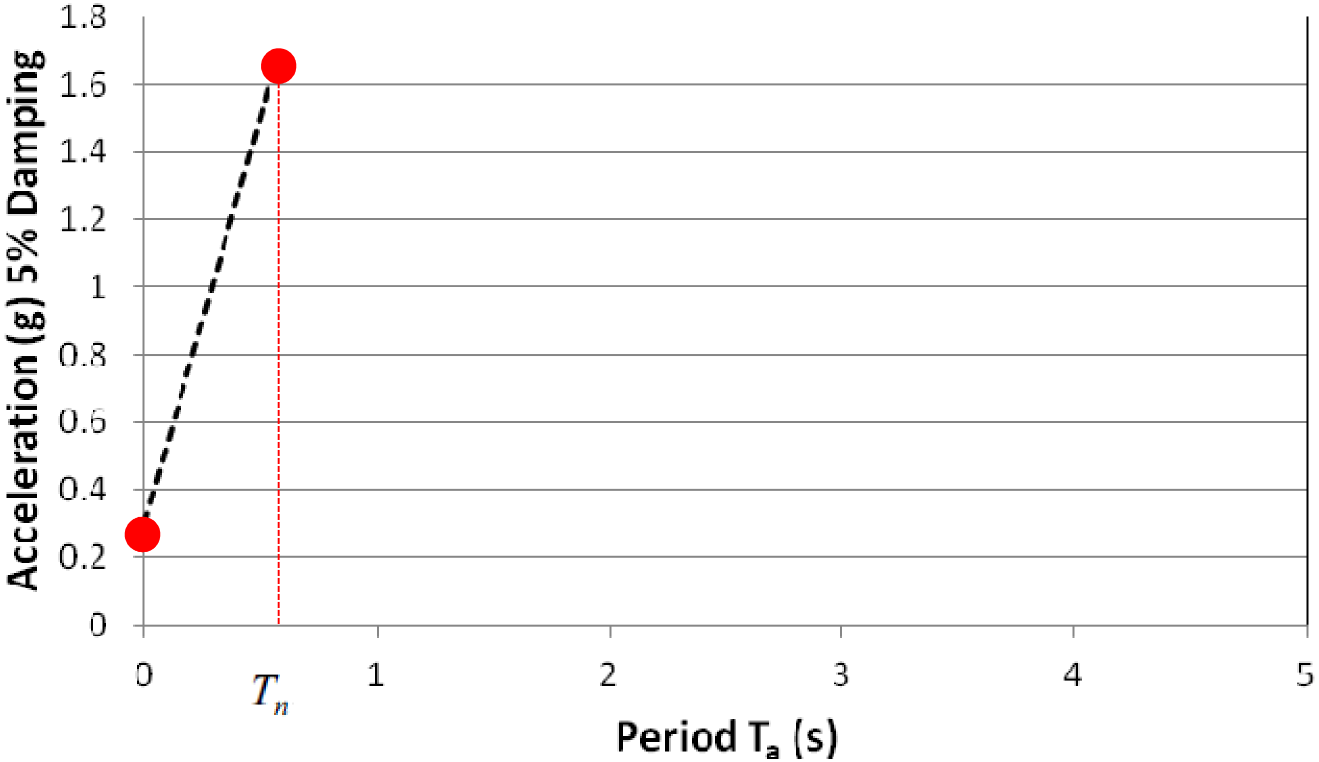
a_{floor}



“Elastic” floor response spectra construction

$$a_{floor}$$

$$a_m = \frac{T}{T_y}\left[a_{floor}(DAF_{max} - 1)\right] + a_{floor} \qquad T < T_n$$



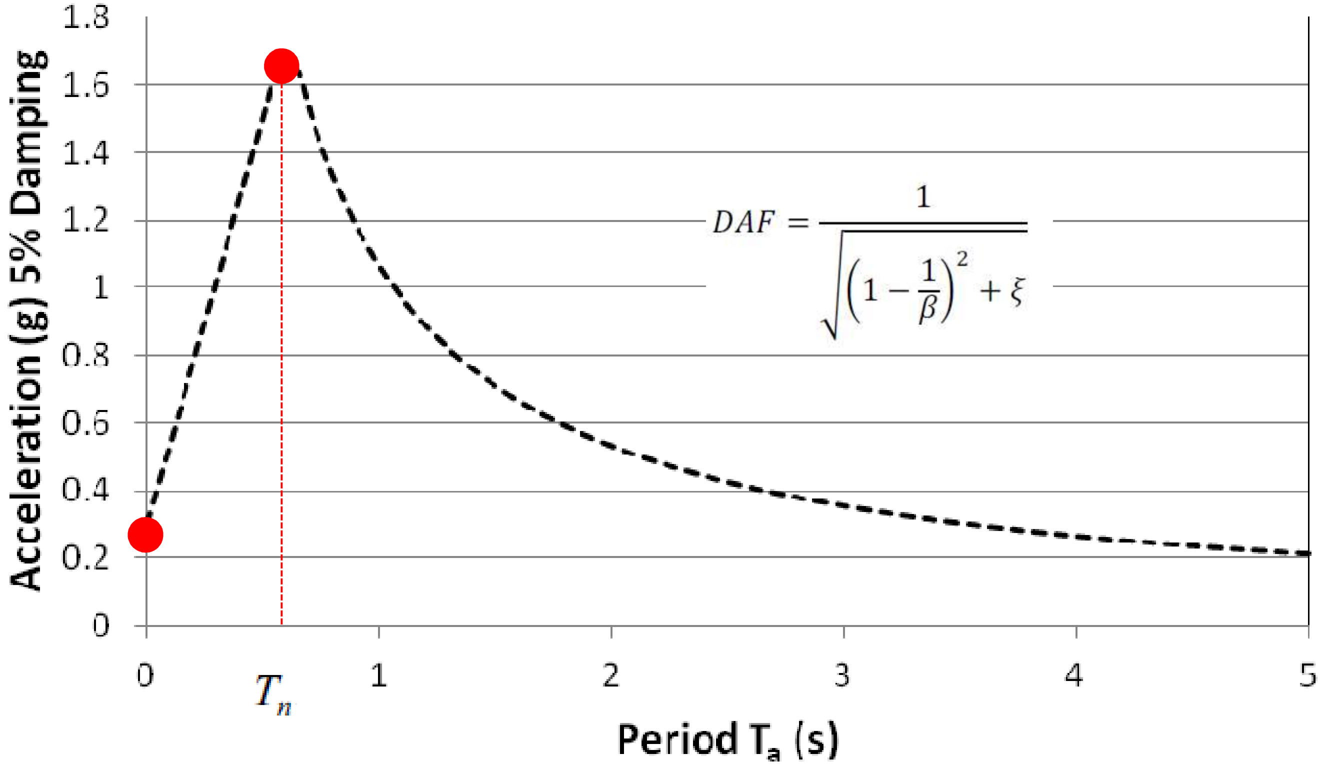
“Elastic” floor response spectra construction

a_{floor}

$$a_m = \frac{T}{T_y}\left[a_{floor}(DAF_{max} - 1)\right] + a_{floor} \qquad T < T_n$$

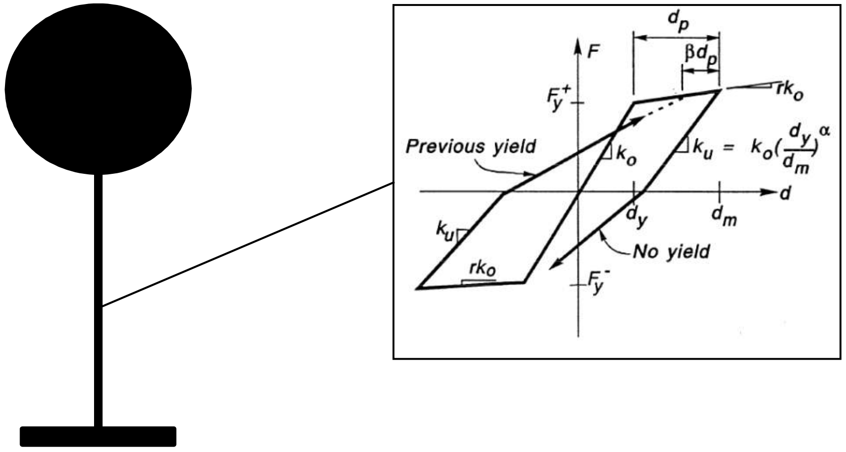
$$a_m = a_{floor} DAF_{max} \qquad T = T_n$$

$$a_m = a_{floor} DAF \qquad T > T_n$$



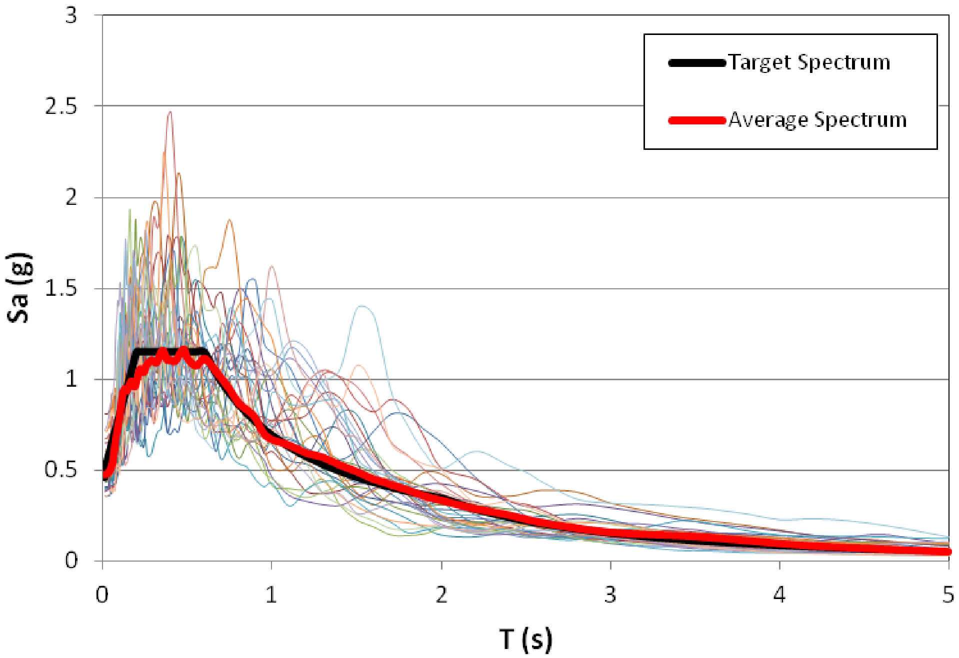
What if the main structure undergoes nonlinear behavior?

Nonlinear SDF supporting system



Case study structures:

- Ts = 0.55, 1.3 and 2 seconds
- TAKEDA hysteretic rule for plastic hinges
- Plastic hinges length calculated in line with expressions provided by Paulay and Priestley (1992);
- ξs = 5% (Tangent stiffness proportional damping)



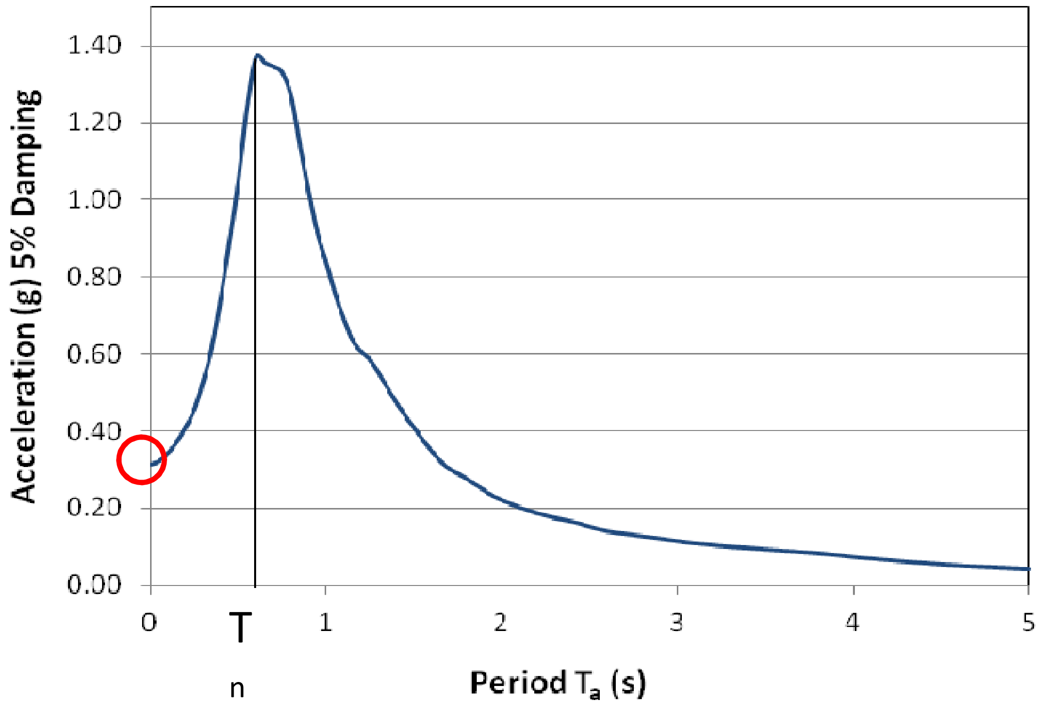
47 Ground motions

- PGA = 0.2g, 0.4g and 0.8g

- ξc = 2,5,10 and 20%

Nonlinear SDF supporting system

Peak ground acceleration = 0.2g

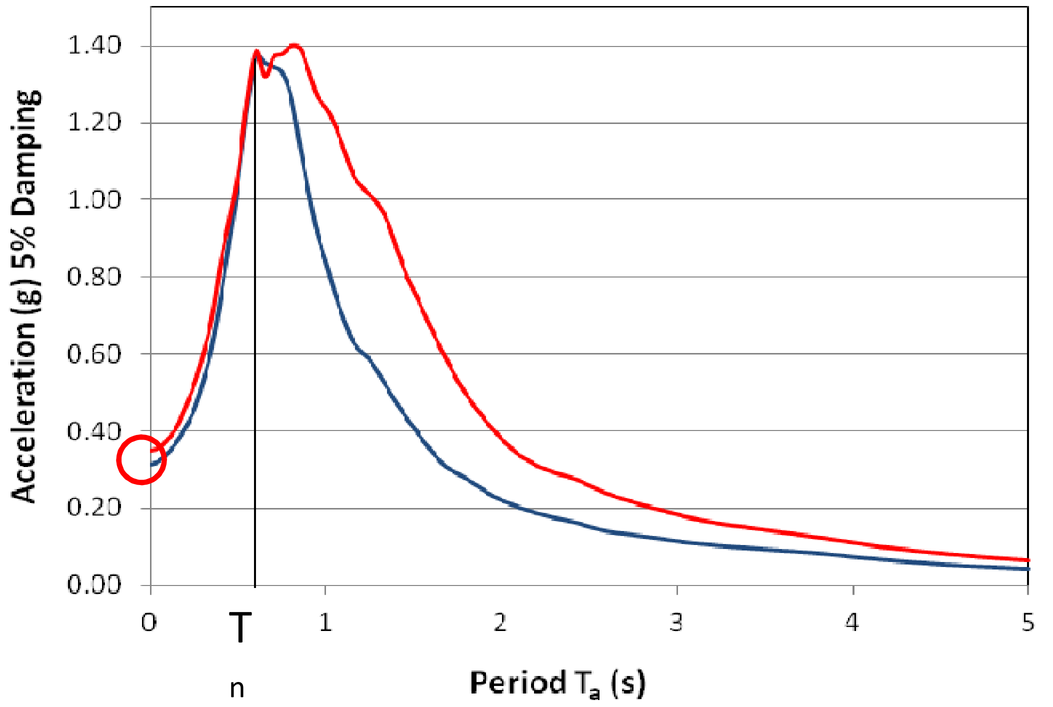


Peak floor acceleration (PFA) = 0.31g
Peak spectral acceleration (PSA) = 1.25 g
Maximum ductility = 1.9

Nonlinear SDF supporting system

Peak ground acceleration = 0.4g

→ Development of a “plateau”

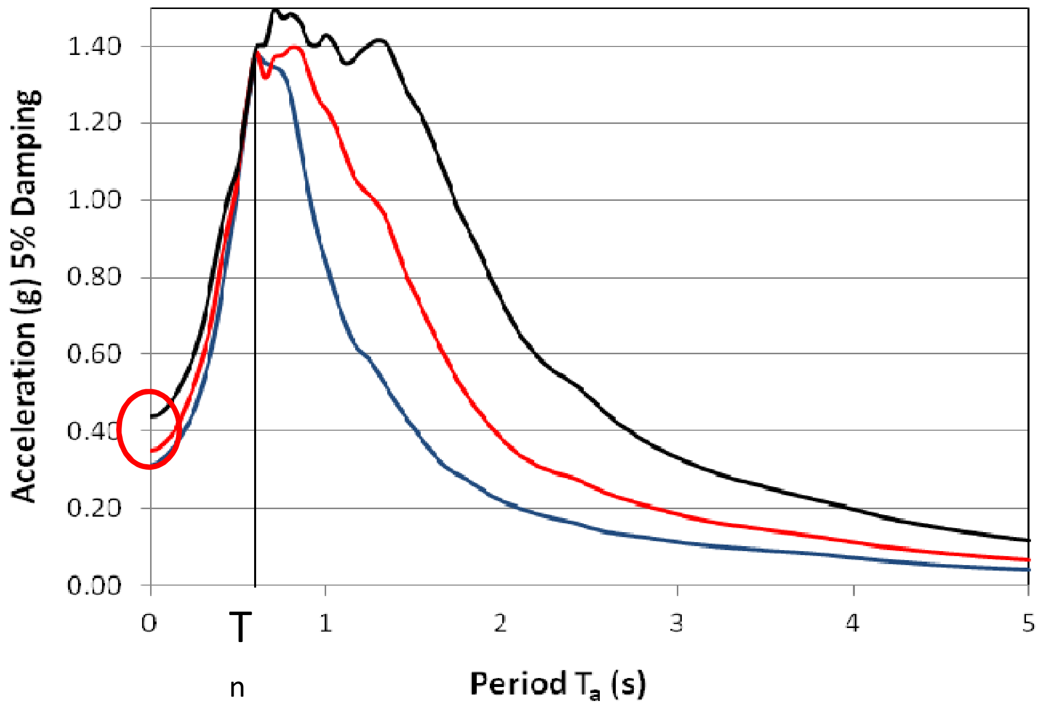


Peak floor acceleration (PFA) = 0.35g
Peak spectral acceleration (PSA) = 1.4 g
Maximum ductility = 4.6

Nonlinear SDF supporting system

Peak ground acceleration = 0.8g

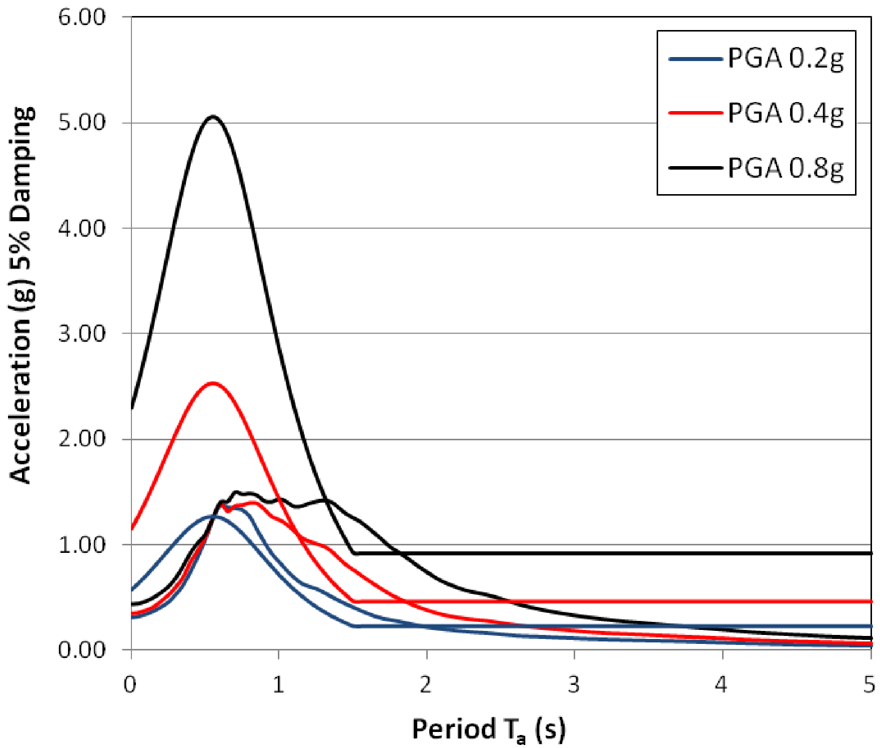
→ Development of a “plateau”



Peak floor acceleration (PFA) = 0.44g
Peak spectral acceleration (PSA) = 1.49 g
Maximum ductility = 9.8

NLTH results vs EC8 predictions

	PGA 0.2 g			PGA 0.4g			PGA 0.8g		
	NLTH	EC8	Error	NLTH	EC8	Error	NLTH	EC8	Error
PFA	0.31	0.57	84%	0.35	1.15	228%	0.44	2.3	422%
PSA	1.25	1.27	1.6%	1.4	2.53	80%	1.49	5.06	239%



Main phenomena developing with nonlinearity of the supporting system

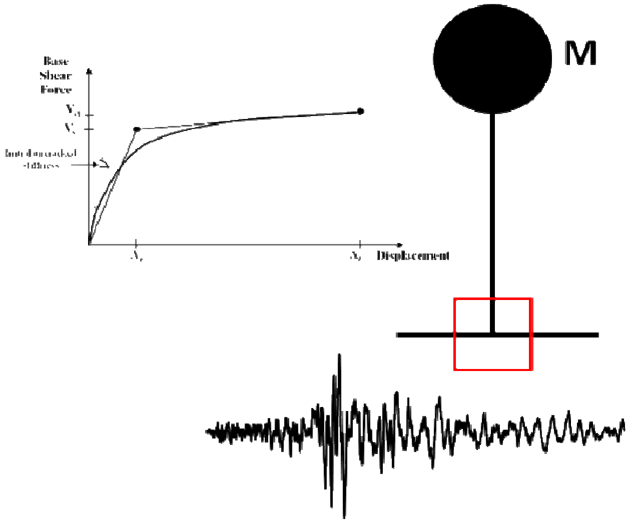
- Peak floor acceleration does not increase along with the earthquake intensity

- Peak spectral accelerations develop into a “plateau” whose extension is a function of the degree of nonlinearity experienced by the main system

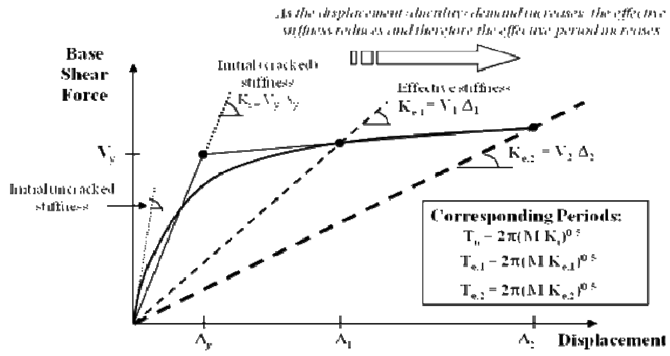
- DAF_{max} slightly decreases as the PGA grows (larger ductility is experienced by the main system)

→ Neglected aspect

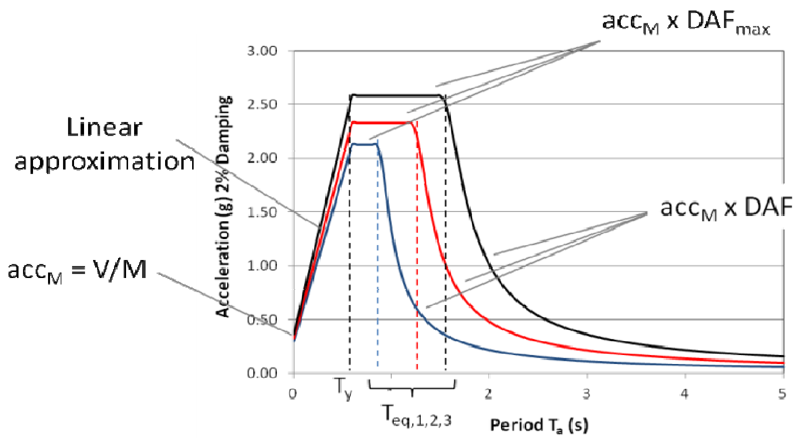
Floor spectra construction: 3 steps procedure



a) Main system is designed in line with any of the allowed code approaches



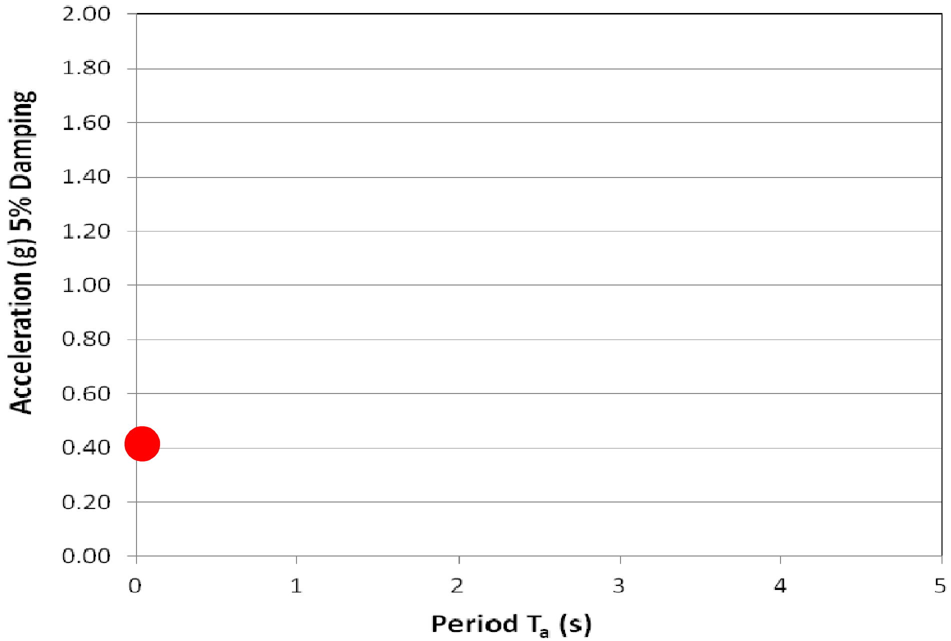
b) Design EQ level for seismic design of components is established. Main system performance is estimated accordingly.



c) Floor response spectra are constructed accordingly to the information obtained at b) and in line with equation 10

“Inelastic” floor spectra construction

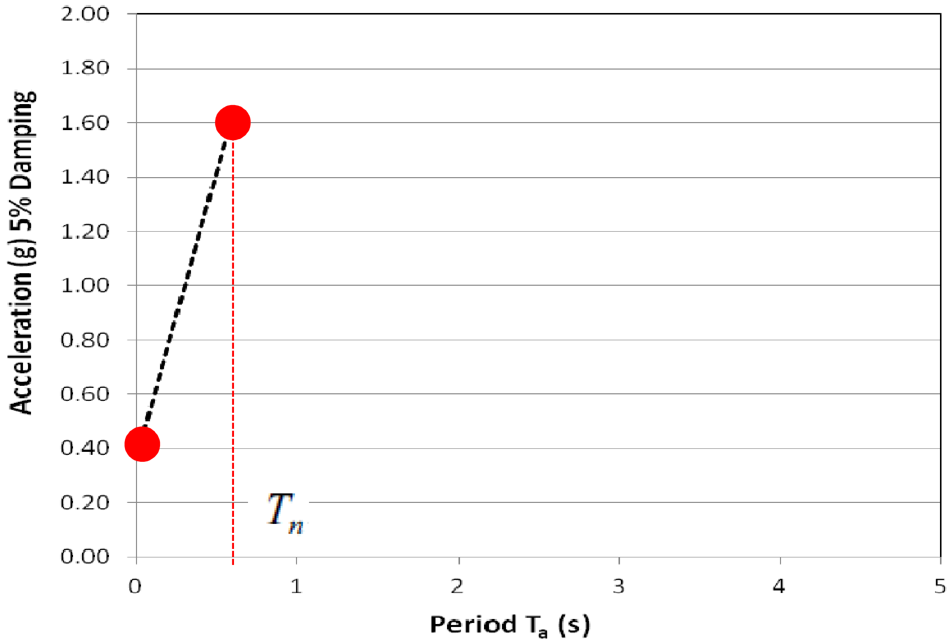
$$a_{\max} = \frac{V_b}{M} = \frac{V_y[1 + r(\mu - 1)]}{M}$$



"Inelastic" floor spectra construction

$$a_{\max} = \frac{V_b}{M} = \frac{V_y[1 + r(\mu - 1)]}{M}$$

$$a_m = \frac{T_a}{T_y} \cdot [a_{\max}(\mathrm{DAF}_{\max} - 1)] + a_{\max} \quad \text{for} \quad T_a < T_n$$



“Inelastic” floor spectra construction

$$a_{\max} = \frac{V_b}{M} = \frac{V_y[1+r(\mu-1)]}{M}$$

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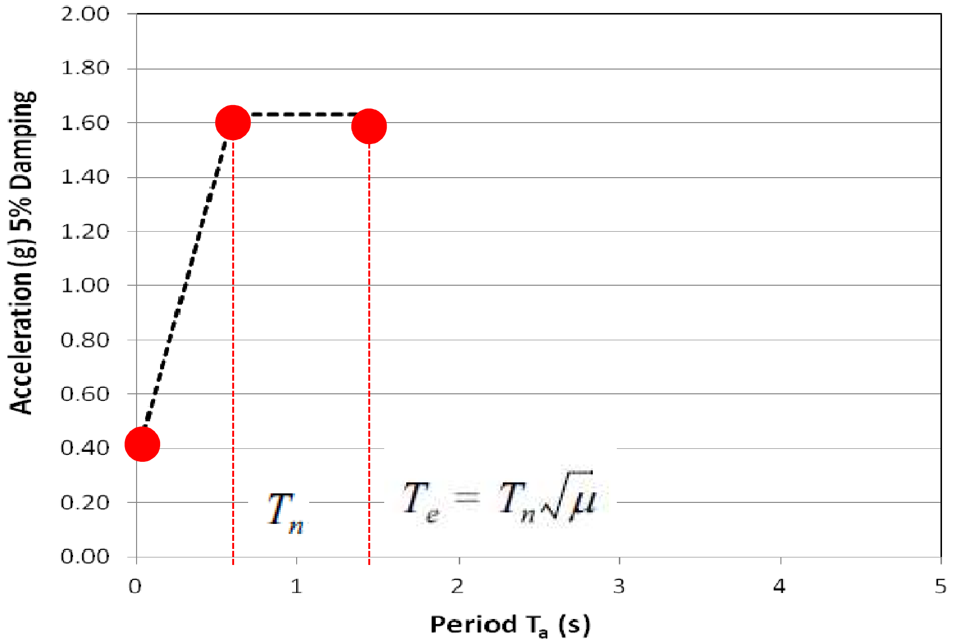
$$a_m = a_{\max}\mathrm{DAF}_{\max} \quad \text{for} \quad T_n \le T_a \le T_e$$



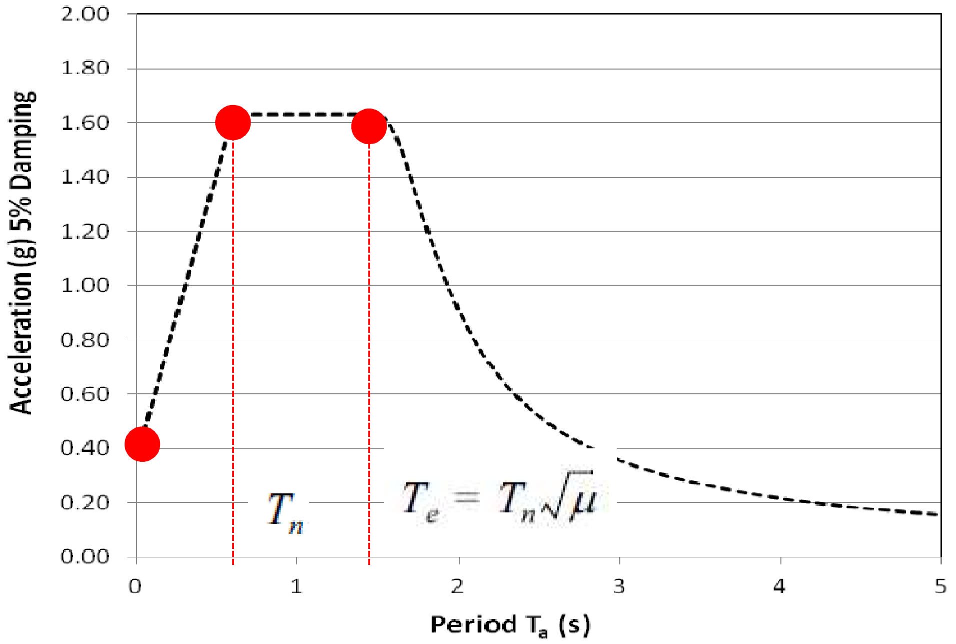
“Inelastic” floor spectra construction

$$a_{\max} = \frac{V_b}{M} = \frac{V_y[1+r(\mu-1)]}{M}$$

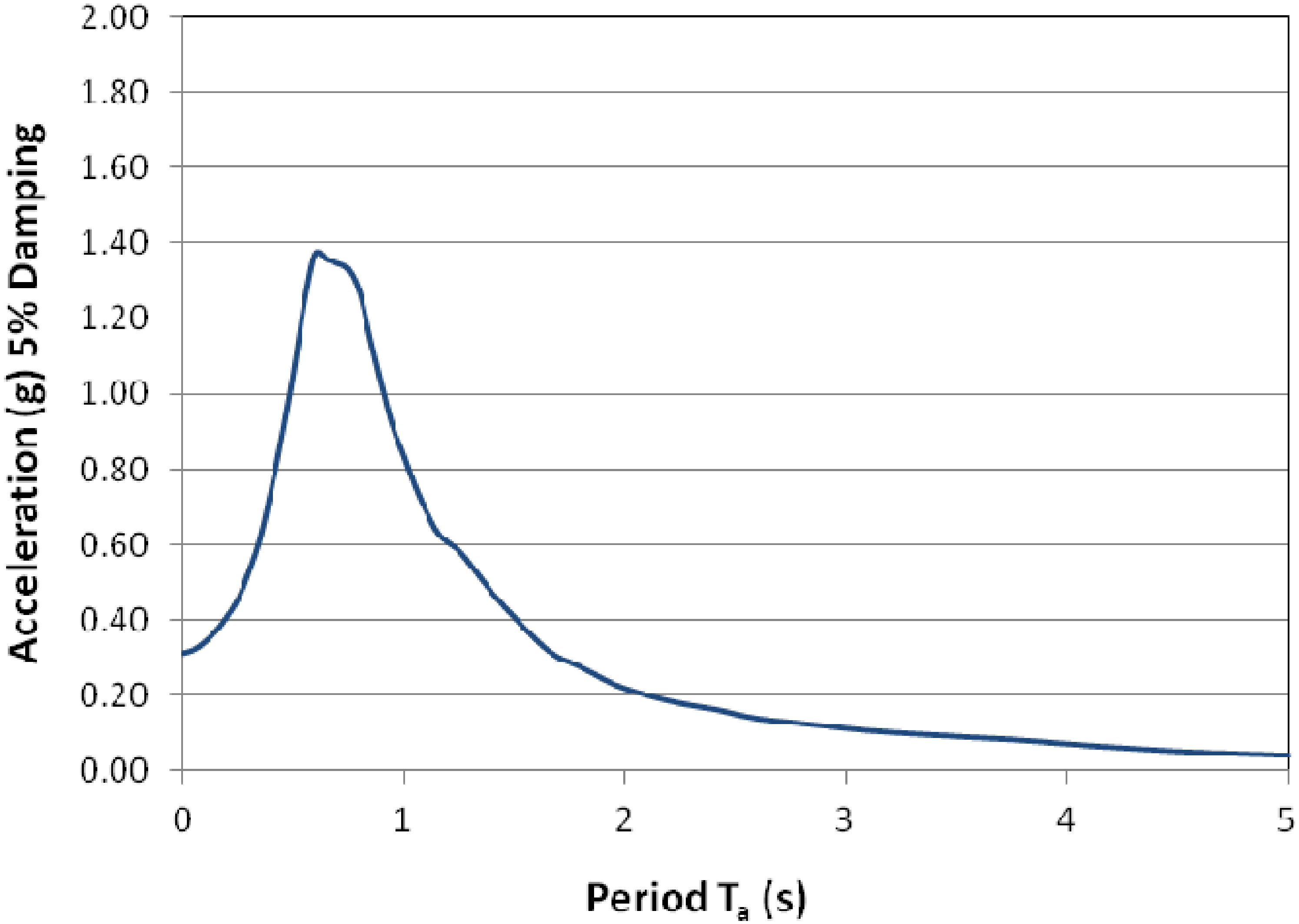
$$a_m = \frac{T_a}{T_y}\cdot[a_{\max}(\mathrm{DAF}_{\max}-1)]+a_{\max} \quad \text{for} \quad T_a < T_n$$

$$a_m = a_{\max}\mathrm{DAF}_{\max} \quad \text{for} \quad T_n \le T_a \le T_e$$

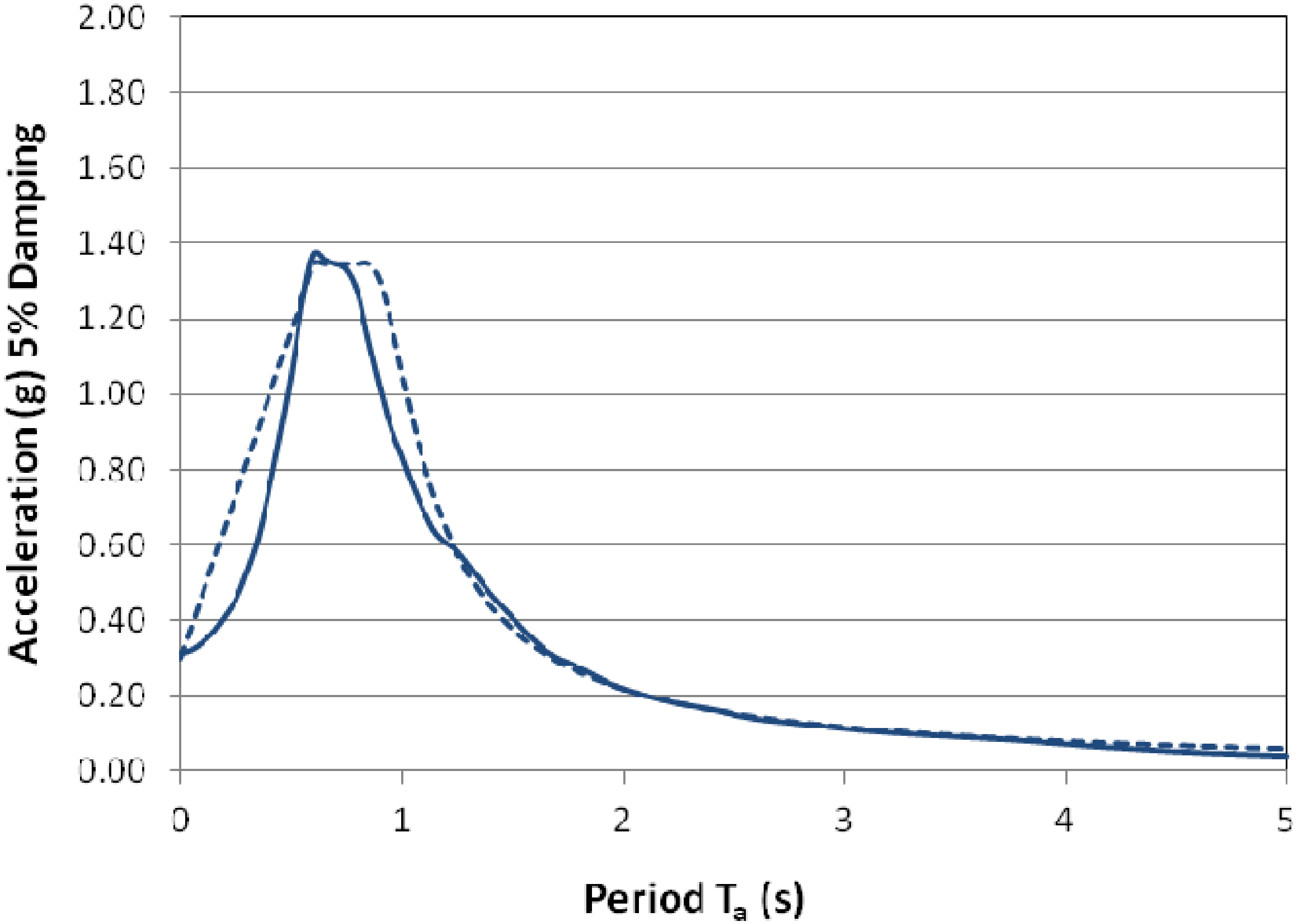
$$a_m = a_{\max}\mathrm{DAF} \quad \text{for} \quad T_a > T_e$$



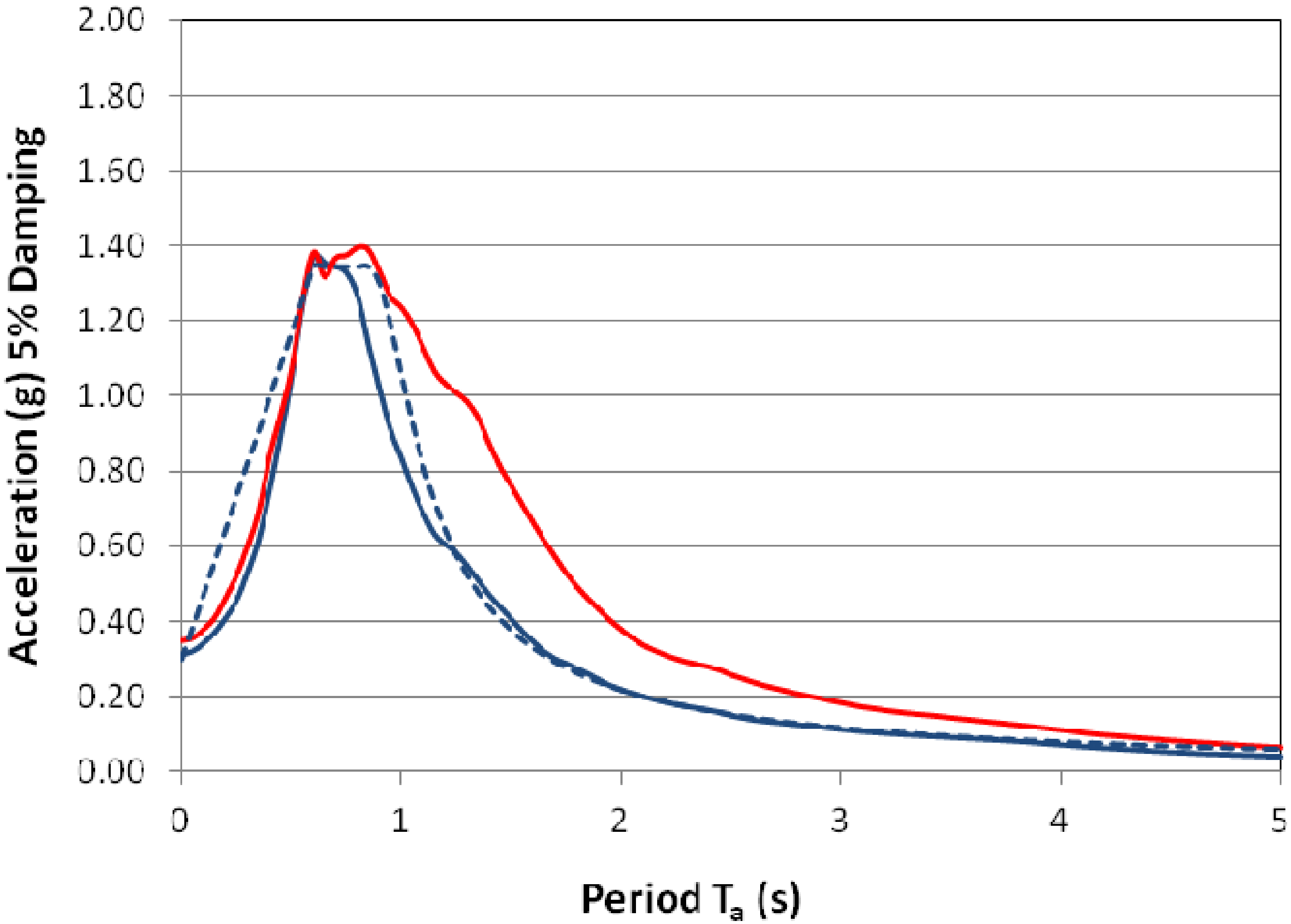
NLTH Analysis vs proposed approach:



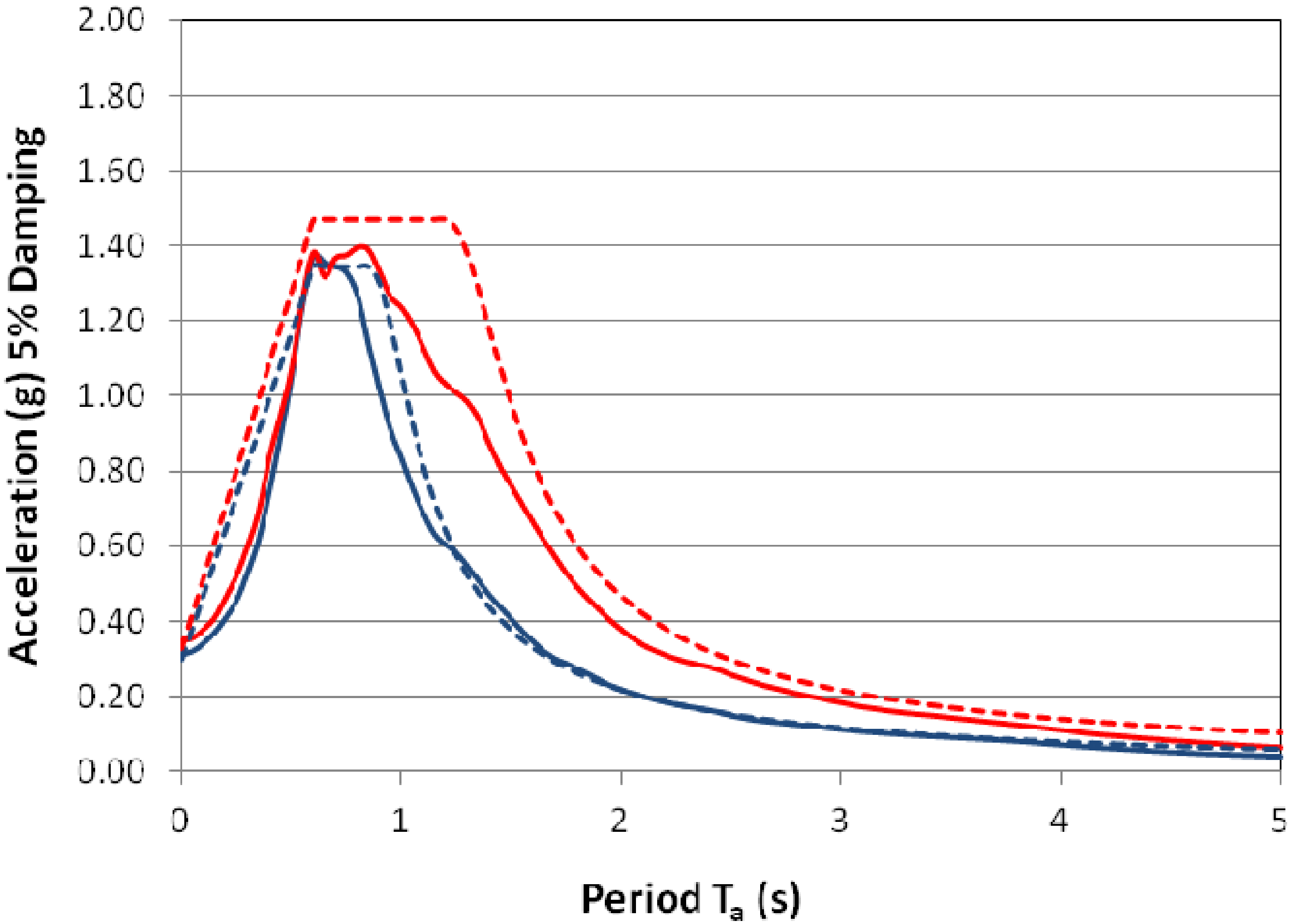
NLTH Analysis vs proposed approach:



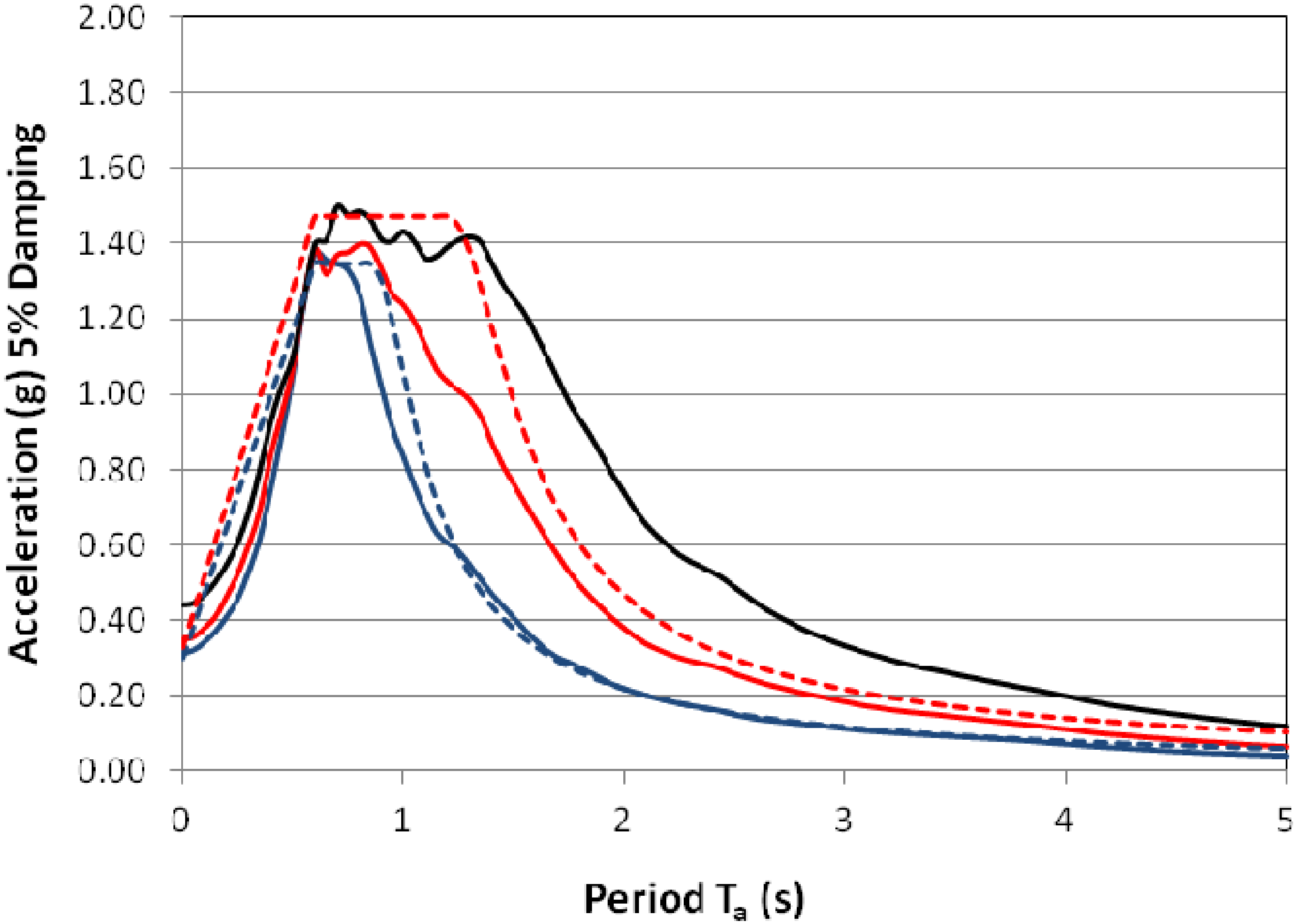
NLTH Analysis vs proposed approach:



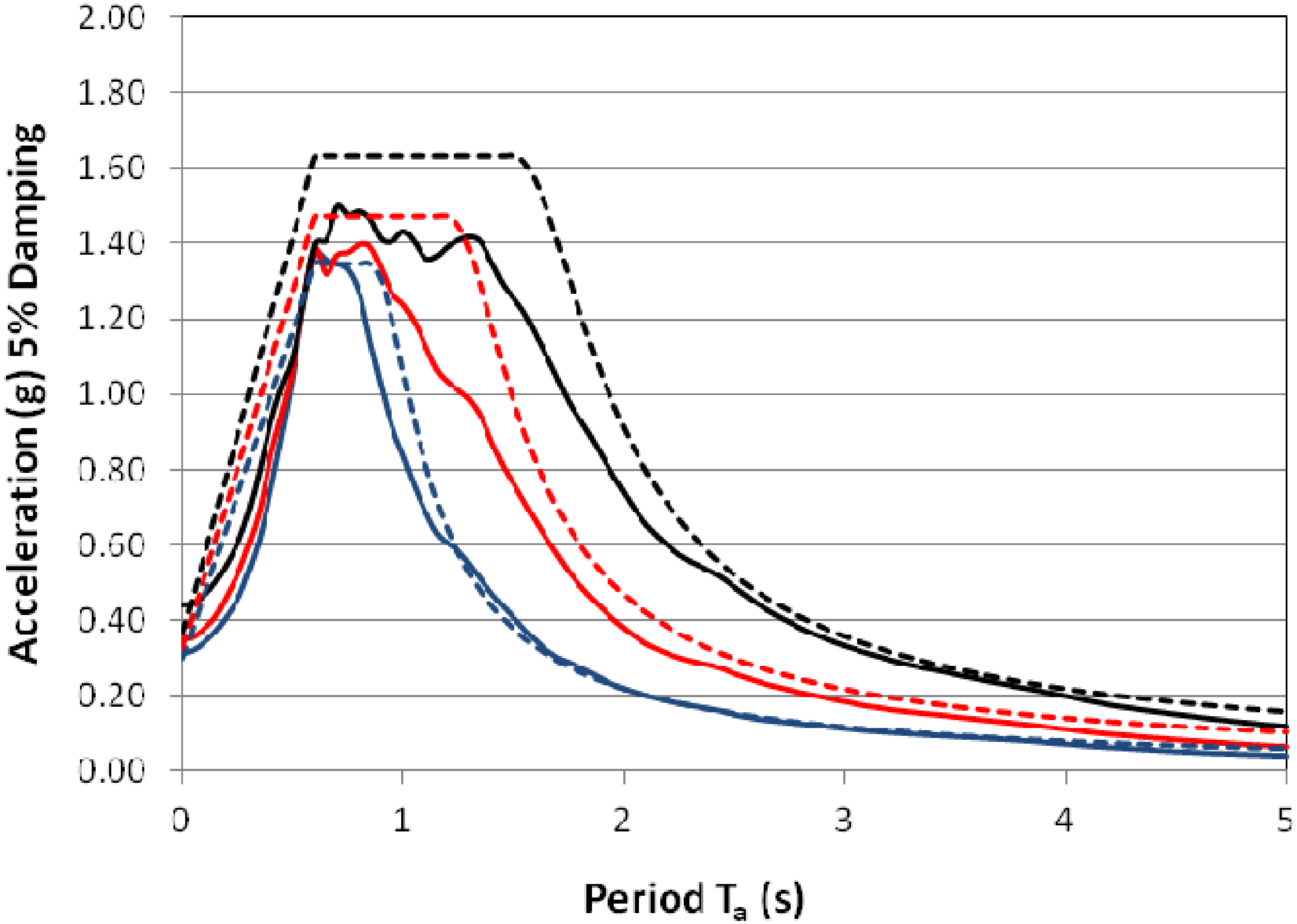
NLTH Analysis vs proposed approach:



NLTH Analysis vs proposed approach:



NLTH Analysis vs proposed approach:



- In 1908, Professor Modesto Panetti from the University of Turin wrote:
  - *…the effects of earthquakes on structures are in fact a structural dynamics problem, which is much too complicated to address…*
- So far, the earthquake engineering community believes that:
  - *…the effects of earthquakes on nonstructural elements are in fact a structural dynamics problem, which is much too complicated to address…*
- Today, we have the tools to address this problem for nonstructural elements. We need the motivation to do it.