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THEME LECTURE

Seismic Design of Bridges: Present and Future

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Acknowledgement

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This lecture:

- Critical overview of Eurocode 8 - 2 and its evolution
- Overview of Performance-Based Design approaches
  - Direct Displacement-Based Design
  - Deformation-based Design
    - Ductile Pier Bridges
    - Seismically isolated bridges
- Comparative Case Study
- Concluding Remarks and Recommendations

Critical overview of Eurocode 8 - 2 and its evolution

Compliance criteria for Performance:

- **The response of the bridge under the design seismic action should be:**
  - **ductile** (plastic deformations develop) → as in many codes

    or
  - **limited ductile** (~ elastic) → a bit peculiar to EC8-2

Limited ductile behaviour ($q=F_{el}/F_d \leq 1.5$)

- where detailing of plastic hinges for ductility is not reliable (convenient)
- where higher modes are significant (e.g. cable-stayed bridges)

- all bridges in regions of low seismicity (?)
- all bridges with seismic isolation
  - but limited ductility is not the same as isolation!...
- active or semi-active control (already used worldwide) not mentioned in EC8-2!

Provisions for ductility

The ultimate displacement d_u is defined as the max displacement that satisfies the following condition:

The structure should be capable of sustaining at least 5 full cycles of deformation to the ultimate displacement:

- ➢ without initiation of failure of the confining reinforcement for reinforced concrete sections,

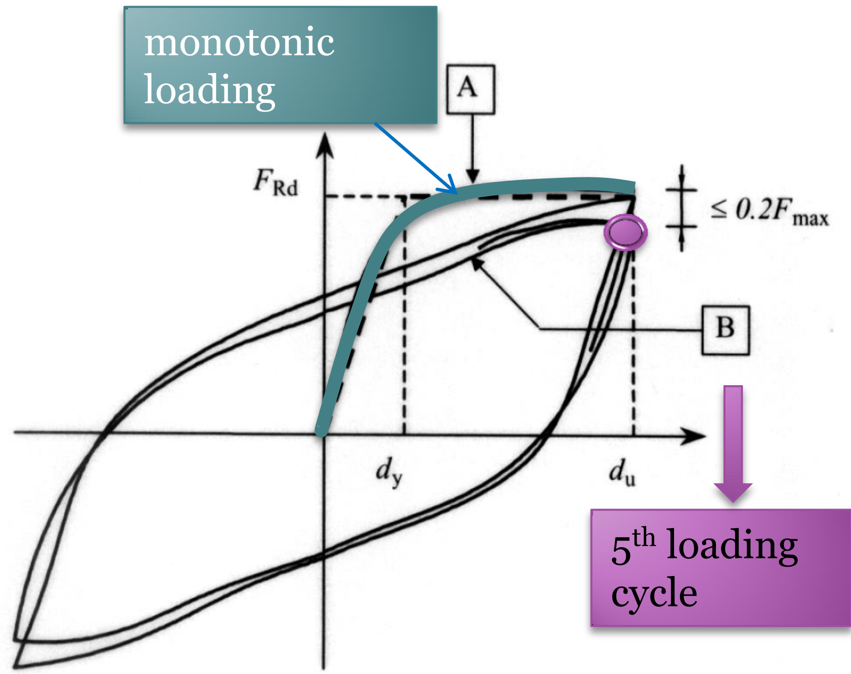
  or

- ➢ local buckling effects for steel sections

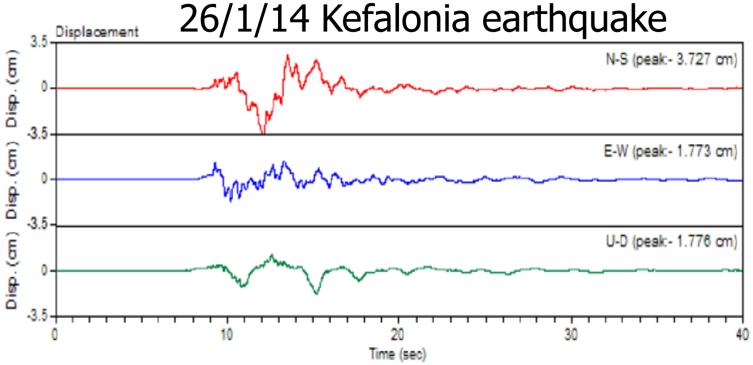
  and

- ➢ without a drop of the resisting force for steel ductile members *or* without a drop exceeding 20% of the ultimate resisting force for R/C ductile members

→ very harsh requirement for structures subjected to typical earthquake ground motions in Europe (~short duration ↔ few 'loading' cycles)



26/1/14 Kefalonia earthquake



Control of displacements

- The bridge should be detailed so as to accommodate the displacements resulting from the design seismic action
- Total design displacement (seismic combination):

$$d_{Ed} = d_E + d_G + \psi_2 d_T \qquad (\psi_2 = 0.5)$$

d_G is the displacement from permanent/quasi-permanent actions, and includes prestressing after losses, shrinkage and creep

d_T is the displacement due to thermal movements, calculated based on EN 1991-1-5:

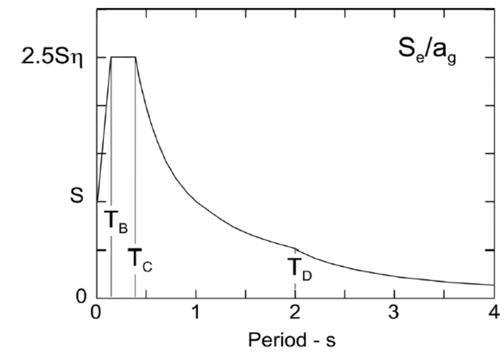
- max contraction due to the difference, $\Delta T_{N,con}$, of the min uniform bridge temperature, $T_{e,min}$, from the initial temperature, T_0,
- vertical temperature difference component, $\Delta T_{M,heat}$, when the top of the deck is hotter than the bottom $\rightarrow$ combine $0.75\Delta T_{M,heat}$ '+' $\Delta T_{N,con}$

➢ design seismic displacement: $d_E = \pm \eta \mu_d d_{Ee}$

$d_E = d_{Ee}$ if d_{Ee} is derived from elastic spectrum (q=1)

$T \geq T_o = 1.25T_c \Rightarrow \mu_d = q$

$T < T_o \Rightarrow \mu_d = (q-1)(T_o/T) + 1 \leq 5q - 4$



➢ d_{Ed} is used for clearances between the deck and critical components (including non-sacrificial backwalls, overlap length at abutments, etc.)

Control of displacements contnd.

- Relative displacement between two independent parts of the bridge:

$$d_E = \sqrt{d_{E1}^2 + d_{E2}^2}$$

- Wherever unpredictable impact between major structural members could occur ductile-resilient members or buffers (energy absorbers) with slack $s \geq d_{Ed}$ shall be provided
- Deck expansion joints and other 'non-critical' elements should cater for a predictable mode of damage, and provide for the possibility of permanent repair

  $\rightarrow$ clearances $p_E \cdot d_E + d_G + p_T \cdot d_T$ (p_E=0.4, p_T=0.5)

  - To be used also for joints between the deck and 'sacrificial' backwalls
- Minimum overlap length at an abutment [§6.6.4]:

  $l_{ov} = l_m + d_{eg} + d_{es}$

  l_m = max {bearing diameter, 400 mm}

  $d_{eg} = \varepsilon_e \cdot L_{eff} \leq 2d_g$ where $\varepsilon_e = 2d_g/L_g$

  L_{eff}: effective length of the deck

  L_g=300÷600m: limit length for uncorrelated ground motion (spatial variability)

  $d_{es} = d_{Ed} + s$ (s: slack of the seismic link, e.g. shear key)

Is EC8-2 a PBD Code?

- It states specific performance objectives (PO):
  - ductile or limited behaviour under 'design' seismic action
  - minimisation of damage under a 'seismic action with a high probability of occurrence'
- It does not specify different earthquake levels for different PO
  - verification carried out for 'design' earthquake only
  - no specific verification of 'minimisation of damage' PO
- It defines a specific plastic mechanism (hinging in piers), when inelastic behaviour is allowed (alternatively: seismic isolation)
  - but does not require verification of local deformation!
- Unlike EC8-1, pier stiffness in EC8-2 is strength-based ↔ iterations!
- What types of bridges result from EC-8 design?
  - in lecturer's experience, clearly overdesigned ones!
  - this is not necessarily bad, but perhaps inefficient/expensive…

Some key issues raised in the evolution phase:

- Comments submitted by 16 national groups during the systematic review phase that ended in February 2017
  - 31-page document distributed among CEN members in March 2017
- Selection of comments by BSI Panel ('Mirror Group') on EC8-2:
  - Resilient designs should be aimed at, for limiting damage and reducing restoration times after earthquakes
  - Bridges with abutments rigidly connected to the deck can be designed considering higher values of q (up to 2.0) if adequate measures are considered to minimise the adverse bridge-backfill interaction effects
  - When a short-span bridge with continuous deck has its abutments embedded in stiff natural soil formations over more than 80% of their height, it can be considered as fully locked-in…
  - When assessing the irregularity of bridges, $\rho = r_{max}/r_{min} \leq \rho_o = 2.0$, the r_i indices should be calculated only for the critical section of each ductile pier (i), i.e. the location of maximum seismic demand (in each direction)

Performance-based Design of Bridges

Key issues in PBD/DBD methodologies

- Type of analysis: Elastic or inelastic analysis, static or dynamic (each of these methods was used in at least one of the existing procedures)
- Definition of seismic input: depends on the type of analysis used, as well as the design approach adopted
  - acceleration spectrum, displacement spectrum, accelerograms
- Stiffness of dissipating zones: paramount in the calculation of bridge displacements (critical parameter in PBD!)
  - depends on reinforcement and the level of induced inelasticity
- Number of directly controlled design parameters: arguably the most critical issue re. future improvements of PB seismic design of bridges
- Number of iterations required: the practicality of design depends on it, especially the number of required analyses with different model

Extensions of the 'standard' DDBD

Pragmatic approach: meet as many criteria as possible with the least number of iterations!

Design Criteria

i. $V_{pier} \geq V(\rho_{req} = \rho_{min})$

ii. $V_{abt} \leq V_u$

iii. $k_{eff} \geq k(T_{eff} = T_D)$

iv. $\Delta_{pier} \leq \Delta_D$ and $\mu_{pier} \leq \mu_u$

v. $\Delta_{abt} \leq \Delta_D$

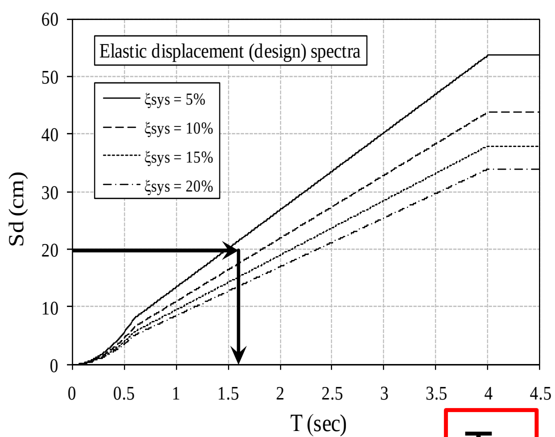
Degree of Fixity at the Top of the Pier

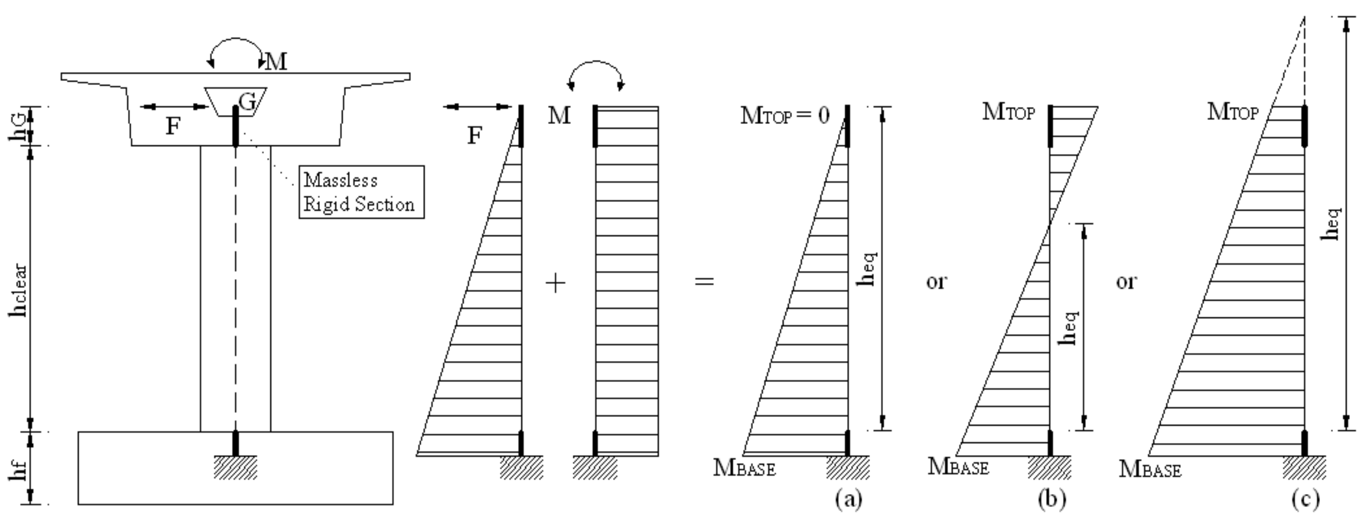
$$k_{eq} = \frac{3EI}{h_{eq}^3} \qquad x_k = \frac{h_{eq}}{h} = \frac{h_{eq}}{h_{clear} + h_G} \qquad k_{pier} = x_k k_{eq}$$

$$\Delta_{y,eq} = \frac{\varphi_y L_{eq}^2}{3}$$

$$x_{\Delta y} = \frac{L_{eq}}{L_{eff}} = \frac{h_{eq} + 0.022 f_y d_{bl}}{h_{clear} + h_G + 0.022 f_y d_{bl}}$$

$$\Delta_{y,pier} = \frac{1}{x_{\Delta y}} \Delta_{y,eq}$$





Kappos, Gidaris, Gkatzogias, BEE 2012

A modal DDBD procedure for bridges

Kappos, Gkatzogias, Gidaris, EESD 2013

Step 0: Definition of initial input parameters

- Geometry (output of ULS and SLS under the pertinent combinations of permanent and transient actions)
- Mass and material properties
- Selection of performance objectives ('damage-based' displacements for selected seismic hazard levels)

Step 1: Selection of the type of displacement pattern

- Relative pier-deck stiffness (RS)

→ flexible displacement pattern always used in modal DDBD

Step 2: Definition of target-displacement profiles (EMS Method)

i. Evaluation of mode shapes *(Φ_j)*:

→ Superstructure: Essentially elastic response (flexural stiffness EI_g)
cracked torsional stiffness (10÷30%)GI_T)

→ Pier columns: Secant stiffness = 10%EI_g (inelastic response)
60%EI_g (elastic response)

Modal DDBD procedure for bridges (contnd)

Step 2-contnd (Definition of target-displacement profiles)

ii. Evaluation of modal participation factors (Γ_j):

$$\Gamma_j = \frac{\boldsymbol{\Phi}_j^T \boldsymbol{m} \boldsymbol{\iota}}{\boldsymbol{\Phi}_j^T \boldsymbol{m} \boldsymbol{\Phi}_j}$$

iii. Evaluation of peak modal displacements ($u_{i,j}$):

$$u_{i,j} = \Gamma_j \Phi_{i,j} S_{dj}$$

iv. Evaluation of expected displacement pattern:

→ Displacement pattern (δ_i) (e.g. SRSS):

$$\delta_i = \sqrt{\sum_j u^2_{i,j}}$$

→ Target-displacement profile (Δ_i):

$$\Delta_i = \delta_i \frac{\Delta_{D,c}}{\delta_c}$$

→ 'Modal' target-displacement profiles ($U_{i,j}$):

$$U_{i,j} = u_{i,j} \frac{\Delta_{D,c}}{\delta_c}$$

at critical point

Hence in EMS Method : $\Delta_i = \sqrt{\sum_j U^2_{i,j}}$

Step 3: Definition of N+1 equivalent SDOF structures

$$U_{sys,j} = \frac{\sum_{i=1}^{n} m_i U_{i,j}^2}{\sum_{i=1}^{n} m_i U_{i,j}}, \quad M_{sys(j)} = \frac{\sum_{i=1}^{n} m_i U_{i,j}}{U_{sys,j}}, \quad x_{sys,j} = \frac{\sum_{i=1}^{n} \left(m_i U_{i,j} x_i\right)}{\sum_{i=1}^{n} \left(m_i U_{i,j}\right)}$$

(N, one for each mode)

Modal DDBD procedure for bridges (contnd)

Step 3-contnd: additional SDOF related to target displacement profile

$$\Delta_{sys}=\frac{\sum_{i=1}^{n} m_i \Delta_i^2}{\sum_{i=1}^{n} m_i \Delta_i},\quad M_{sys}=\frac{\sum_{i=1}^{n} m_i \Delta_i}{\Delta_{sys}},\quad x_{sys}=\frac{\sum_{i=1}^{n}\left(m_i \Delta_i x_i\right)}{\sum_{i=1}^{n}\left(m_i \Delta_i\right)}$$

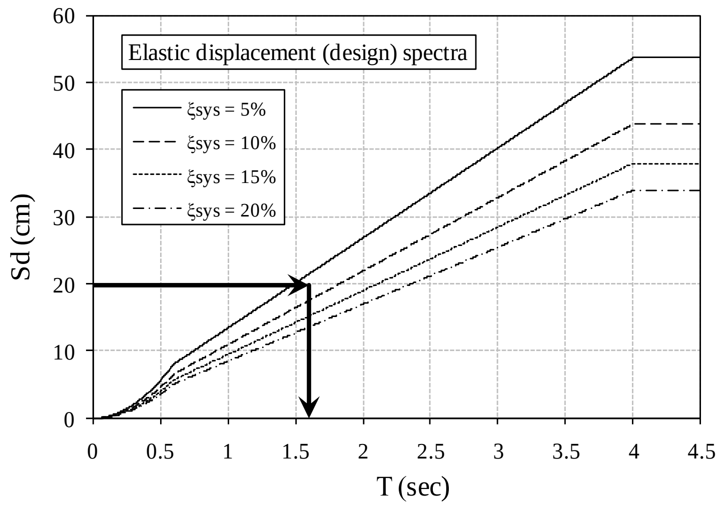
Step 4: Estimation of equivalent viscous damping levels

- h_{eq} : preliminary structural analyses (recommended) or $h_{eq} \approx h_{pier}$
- Displacement ductilities: $\mu_{\Delta_i}=\Delta_i/\Delta_{yi}$, $(or\ \mu_{\Delta_i}=U_{i,j}/\Delta_{yi,j})$ (criterion ***iv***)
- Member damping: $\xi_i=\xi_v+\frac{50}{\pi}\left(\frac{\mu_\Delta - 1}{\mu_\Delta}\right)\%$ (Takeda)
- System damping: $\xi_{sys}=\sum_{i=1}^{n}\left(\frac{W_i}{\sum_{k=1}^{n} W_k}\xi_i\right)$ $\quad \xi_{sys(j)}=\sum_{i=1}^{n}\left(\frac{W_{i,j}}{\sum_{k=1}^{n} W_{k,j}}\xi_{i,j}\right)$
- Work done by each member (assuming F_{Abt}=30% V_B)
  - $W_i=\mu_{\Delta_i}\Delta_i/h_{eq,i}$, $W_{i,j}=\mu_{\Delta_i}U_{i,j}/h_{eq,ij}$ (1st iteration)
  - $W_i=V_i\Delta_i$ $\quad W_{i,j}=V_{i,j}U_{i,j}$ (subsequently)

Modal DDBD procedure for bridges (contnd)

Step 5: Determination of the equivalent SDOF effective periods

- $\left.\begin{array}{l}\Delta_{sys},\ U_{sys,j}\\ \xi_{sys},\ \xi_{sys,j}\end{array}\right\}$ T_{eff}, $T_{eff,j}$ (+criterion ***iii***)



- Effective stiffnesses, base shears:

$$k_{eff} = 4\pi^2 M_{sys} / T_{eff}^{\ 2},\ k_{eff,j} = 4\pi^2 M_{sys,j} / T_{eff,j}^{\ 2}$$

$$V_B = k_{eff}\Delta_{sys},\ V_{B,j} = k_{eff,j} U_{sys,j}$$

Step 6: Verification of design assumptions

- Base shear distribution: empirical equations (1st loop of iterations)
  structural analysis results (subsequent loops)
- Member cracked section stiffnesses: $k_{eff,i} = V_{B,i} / \Delta_i,\ k_{eff,ij} = V_{B,ij} / U_{i,j}$
- *Convergence criterion: $k_{eff,i}$* ($\rightarrow \Delta_i$)
  Non-convergence $\rightarrow$ Step 2 (EMS using updated $k_{eff,i}$)
  Convergence $\rightarrow$ Step 7
- Whenever Δ_i *stabilises,* $U_{i,j}$ also stabilise $\rightarrow$ Δ_i : sole convergence criterion

Modal DDBD procedure for bridges (contnd)

Step 7: Structural analysis

- N structural analyses under inertia loads: $F_{k,j} = V_{B,j}\left(m_k U_{i,j}\right) \Big/ \sum_{i=1}^{n}(m_i U_{i,j})$
- Use of secant stiffnesses ($k_{eff,ij}$) (of Step 6: stabilisation of $U_{i,j}$)
- *Convergence criterion:* $U_{ij} = U_{ij,an}$

  Non-convergence → Updating of $k_{eff,ij}$ values → analyses repeated

  Convergence → $k_{eff,ij}$ (of Step 6) ≈ $k_{eff,ij,an}$ <u>and</u> $F_{abt} \approx F_{abt,an}$

  Convergence → Step 8

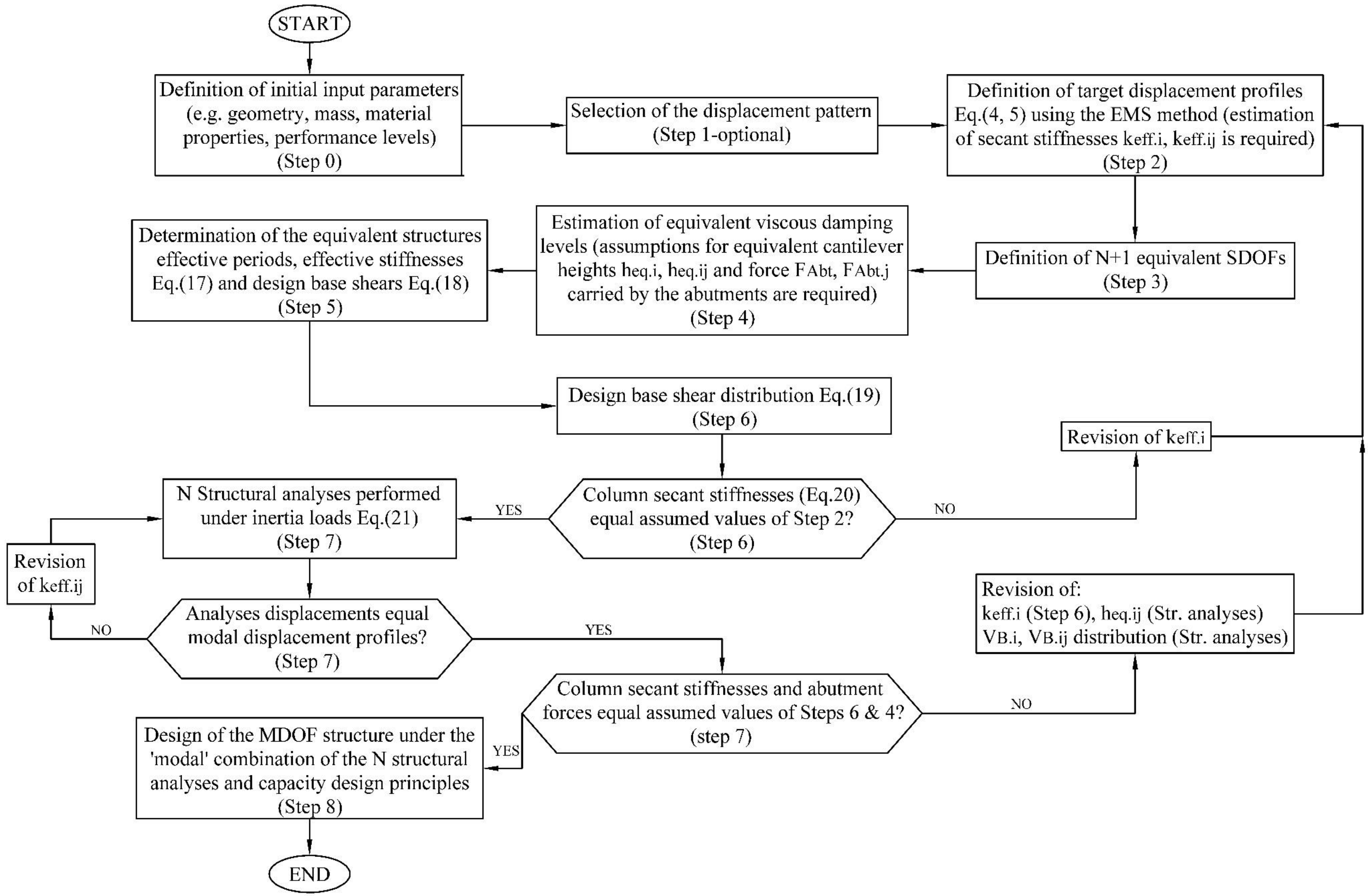
  Non-convergence → Steps 2÷7: Revise $k_{eff,ij}$ (Step 6)

  h_{eq} (N str. an.)

  V_i, V_{ij}, F_{abt} (N str. an.)

Step 8: Design of the MDOF structure

- Combination of peak modal responses (N structural analyses)
- Design according to
  - Capacity design principles
  - Design criterion (i): Revision of Δ_i or D_{col} → Step 1

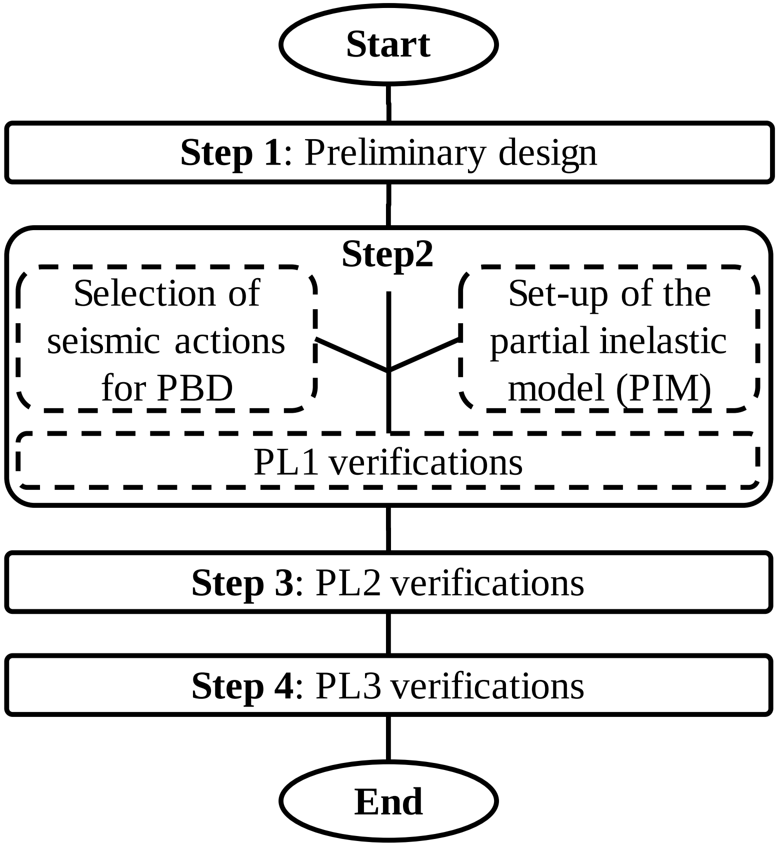


Modal direct displacement-based design of bridges

Deformation-based Design

- Deformations are a (direct) design parameter!
- Initially developed for buildings (final version Kappos & Stefanidou, BEE 2010, includes irregular buildings)
- Then developed for ductile pier bridges (Kappos 2ECEES 2014, Gkatzogias & Kappos Erq & Strs 2015)
- Same concepts (but different procedures) developed for seismically isolated bridges with supplemental damping
  - linear viscous dampers (Gkatzogias & Kappos 16WCEE 2017)
  - nonlinear viscous dampers (Gkatzogias PhD 2018)
  - Forthcoming companion papers in EESD

Def-BD: Bridges with energy dissipation in the piers



Seismic hazard		Structural performance level			
EQ	**T_R (yrs)**	**SP1**	**SP2**	**SP3**	**SP4**
EQI	<50	Ordinary	Non-essential	-	-
EQII	50-100	Essential	Ordinary	Non-essential	-
EQIII	500-1000	Critical I	Essential	Ordinary	Non-essential
EQIV	~2500	Critical II	Critical I	Essential	Ordinary

Service	Full	Operational	Limited	Disrupted
Damage	Negligible	Limited	Significant	Severe
Repair	No/Economic	Economic	Feasible	Non-feasible

Seismic hazard		Analysis type per PL			
EQI	<50	IMP	L	-	-
EQII	50-100	IMP	L+NL	NL	-
EQIII	500-1000	IMP	L+NL	NL	IMP
EQIV	~2500	L+NL	L+NL	NL	IMP

Importance Class

Non-essential

Ordinary

Essential

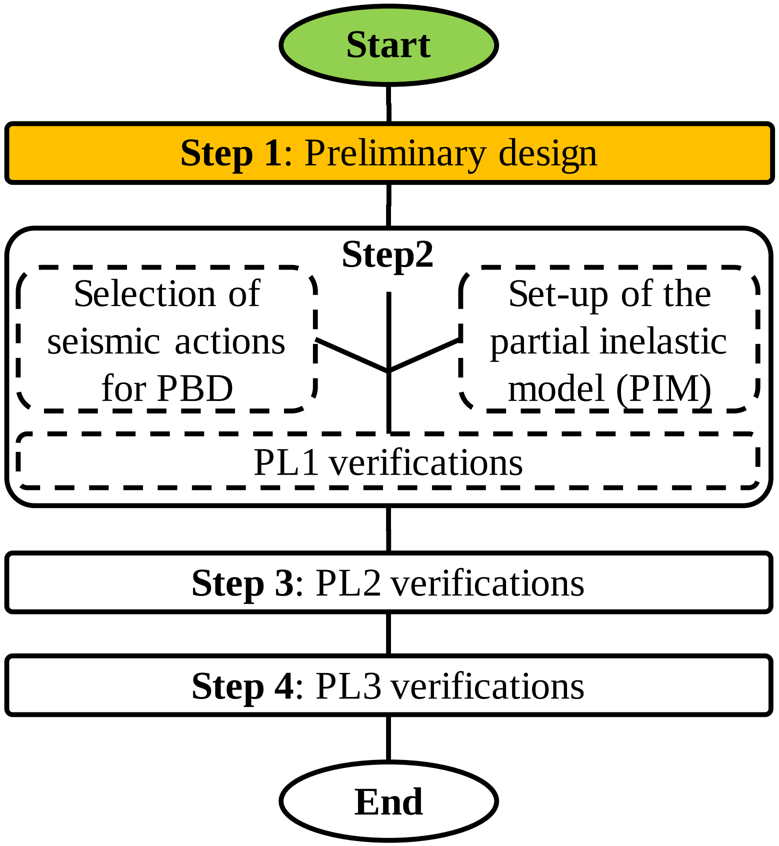
Critical I, Critical II

Type of analysis

IMP: Implicit consideration

L: Linear (static, RSA)

NL: Nonlinear dynamic



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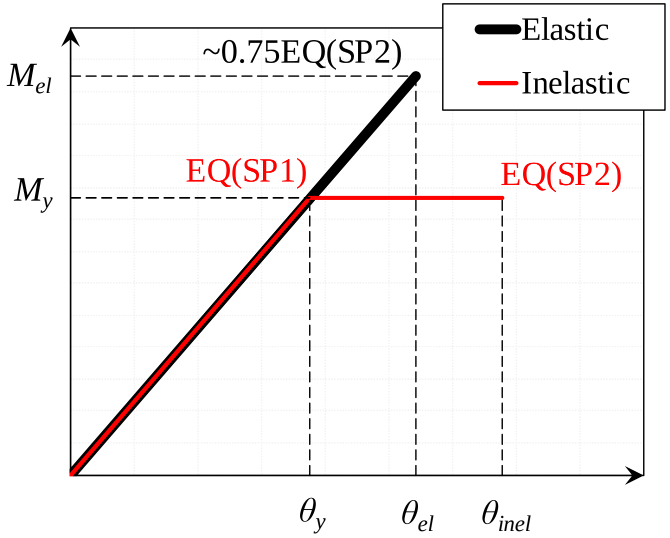
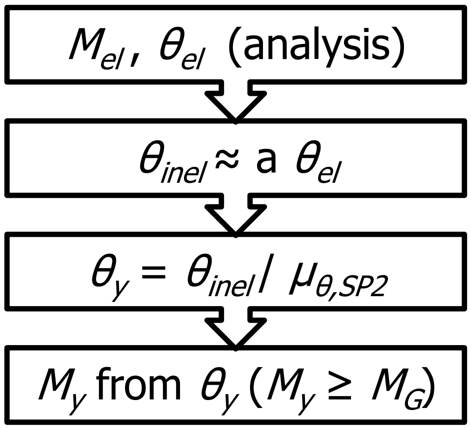
Step 1: Preliminary design

- Establishes basic level of strength under EQ(SP1) for the bridge to remain operational during and after EQ(SP2) (T_R=50~100yrs - Ordinary bridges):
  - elastic analysis run for a fraction (≈0.75) of EQ(SP2): pier strength





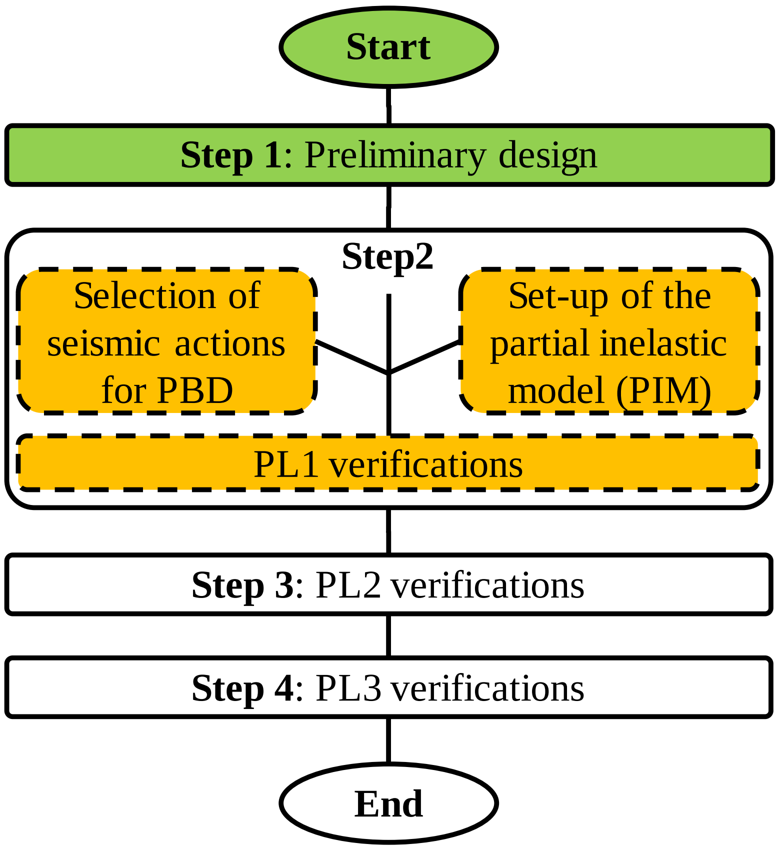
a-values from Bardakis & Fardis (2011)

$$\mu_{\theta,SP2} = 1 + \frac{\theta_{pl,SP2}}{\theta_y} = 1 + \frac{3 \cdot (\varphi_{SP2} - \varphi_y) \cdot L_{pl}}{\varphi_y \cdot h_{eq}}$$

  - elastic analysis run for EQ(SP2): bearing deformability

- the goal is to reach the target μ_θ in the piers and γ_q in the bearings during the operationality earthquake (not to be much lower than it!)

Def-BD: Bridges with energy dissipation in the piers: Step 2



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Step 2: SP2 (operationality) verifications

➢ *Set-up of the partially inelastic model (PIM)*

- Ductile piers modelled as yielding elements (strength from Step 1, stiffness: *M-φ* analysis, e.g RCCOLA.net or AnySection)
- All other parts of the bridge modelled as elastic members (including common bearings; but LRBs should be modelled inelastically)

➢ *Selection of seismic actions*

- Pairs of records are required for 3D analysis (or triplets, if vertical motion is influential)
- Recommended selection criteria: M, R (from deaggregation of hazard analysis), PGA (e.g. ≥0.1g), similarity of spectra, accepted variability in response
- Modern tools (like ISSARS, Katsanos & Sextos 2013) select sets of e.g. 7 records based on such 'multi-criteria', also including the EC8 procedure
- Scaling procedures: EC8-Part 2 (based on considered earthq. components)
- Records are scaled to the level of seismic actions associated with EQ(SP2) (Step 2) and EQ(SP3) (Step 3) (more sophisticated procedures ↔ importance)

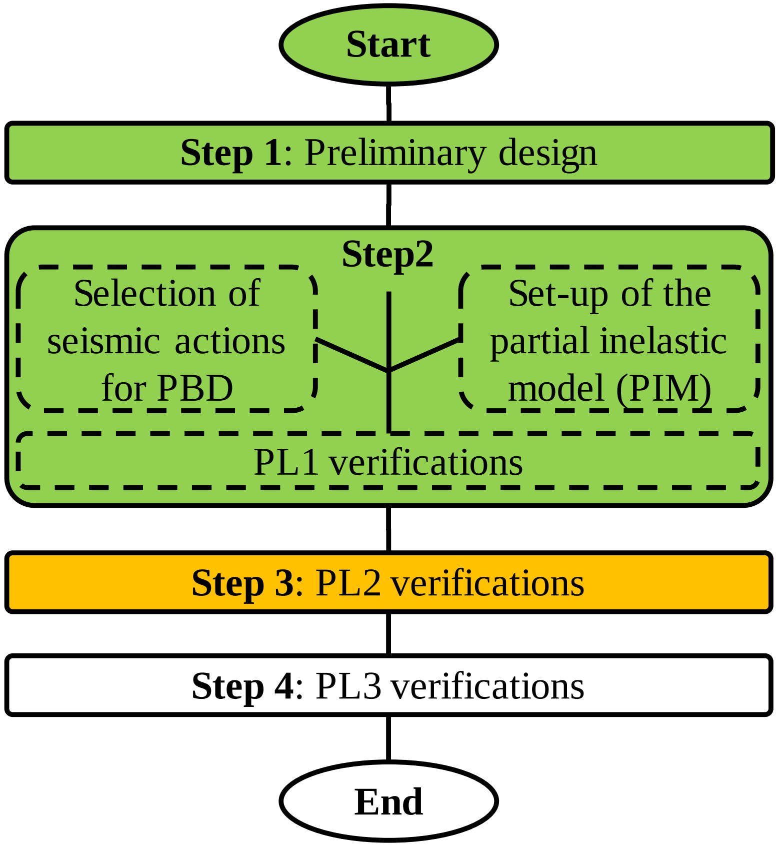
Step 2: SP2 (operationality) verifications

- *Verifications*

- PIM analysed for set of records (≥7) scaled to the seismic action associated with the SP2 operationality requirements
- Verifications include specific limits for pier drifts, ductility factors (μ_θ) and plastic hinge rotations (θ_p); ideally $\mu_{\theta,an} \approx \mu_{\theta,SP2} = f(\varepsilon_c, \varepsilon_s)$

  → Exception: Critical II bridges $\mu_{\theta,an} \approx \mu_{\theta,SP1}$ (elastic response)
- recommended values of $\mu_{\theta,SP2}$ and/or $\theta_{p,SP2}$ vary significantly, e.g. proposals by Eastern (DesRoches *et al*.) and Western (Priestley *et al*.) US teams
- ε_c, ε_y are good basis for estimating damage to R/C piers
- damage to bearings ($\gamma_q \leq 1.0$) should also be checked; might be critical
- joint widths should be such as to prevent damage to backwalls
- Modify ρ_l (deformation control) and/or D (drift control)

Def-BD: Bridges with energy dissipation in the piers: Step 3



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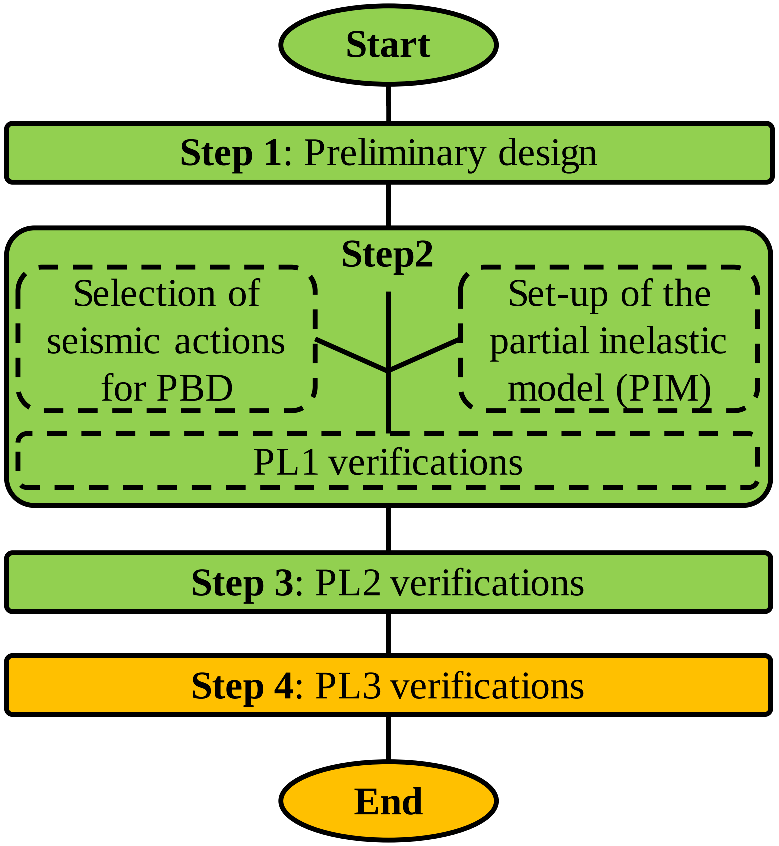
Step 3: SP3 verifications

- The PIM is now analysed for records scaled to the seismic action associated with the SP3 ‘feasibility of repair’ requirements ($T_R \approx 500 \div 1000$yrs in Ordinary bridges)
  - verifications of pier drifts, ductility factors (μ_θ) and plastic hinge rotations (θ_{pl}) based on allowable ε_c , ε_s
  - members assumed elastic are designed in flexure e.g. abutments, deck

    $\rightarrow$ design of the superstructure should aim at the deck being close to cracking, rather than yielding (exception: continuity slabs in beam/girder bridges where yielding is allowed)
  - pier deformation demand is not critical at this PL (except when a hazard level higher than the one corresponding to Ordinary bridges is adopted)
  - elastomeric bearings $\gamma_q \leq 1.5 \div 2.0$

Def-BD: Bridges with energy dissipation in the piers: Step 4



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EQIII	500-1000	Critical I	Essential	Ordinary	Non-essential
EQIV	~2500	Critical II	Critical I	Essential	Ordinary

Service	Full	Operational	Limited	Disrupted
Damage	Negligible	Limited	Significant	Severe
Repair	No/Economic	Economic	Feasible	Non-feasible

Seismic hazard		Analysis type per PL			
EQI	<50	IMP	L	-	-
EQII	50-100	IMP	L+NL	NL	-
EQIII	500-1000	IMP	L+NL	NL	IMP
EQIV	~2500	L+NL	L+NL	NL	IMP

Importance Class

Non-essential

Ordinary

Essential

Critical I, Critical II

Type of analysis

IMP: Implicit consideration

L: Linear (static, RSA)

NL: Nonlinear dynamic

Step 4: SP4 verifications

- ***Design for shear***

- Less ductile failure mode → V_{EQ} should be calculated for higher seismic actions than those considered in Step 3 (apart from Essential, Critical I, II bridges) associated with 'collapse prevention' ($T_R \approx$ 2500yrs – Ordinary bridges)
  - to avoid 3rd set of response-history analyses, V_{EQ} from Step 3 could be empirically scaled; recommended $SF_v \approx 1.15 \div 1.35$
  - no need for code-type conservative capacity design, since inelastic analysis is used!

- ***Detailing of critical members***

- Detailing of R/C piers for confinement, anchorages, lap splices
  - the actual μ_φ values from Step 3 can be used, implicitly associated with 'collapse prevention' (e.g. $SF_\varphi \approx S_a(T)_{EQ(SP4)}/S_a(T)_{EQ(SP3)}$)
- Bearings should be verified based on stability considerations

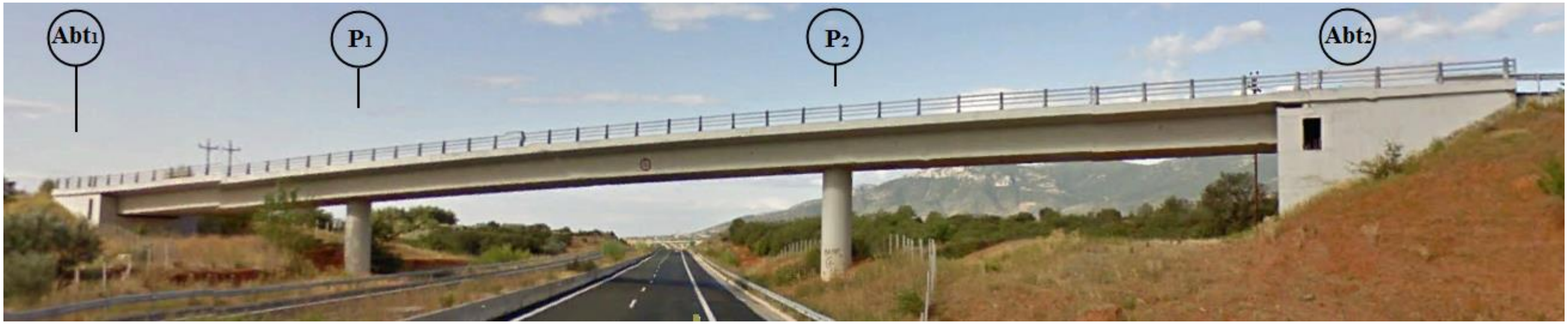
$$N'_{cr} = \frac{\pi \cdot \sqrt{\lambda} \cdot G \cdot S \cdot r'}{t_r} \cdot A_r \rightarrow \gamma_{q,SP4} = \ldots \geq \gamma_{q,EQ(SP4)} = SF_{\gamma q} \cdot \gamma_{q,EQ(SP3)}$$

(e.g. Constantinou *et al*. 2011)

Def-BD: Bridges with energy dissipation in the piers: Case Study

❖ *Description of the studied bridge (T7 Overpass, Egnatia Motorway)*

- 3-span structure (27 - 45 - 27m)
- Prestressed concrete box girder section (variable geometry)
- Deck monolithically connected to the (single-column circular) piers
- Unrestrained transverse displacement at the abutments (elastom. bearings)
- Different pier heights (longitudinal deck slope of 7%)
- Surface foundations



❖ *Design cases*

- Def-BD: design + assessment (multiple PLs)
- MDDBD (Kappos *et al*. 2013): design + assessment (SP3 PL)
- Code-BD: Corresponds to the 'as-built' state, assessment (multiple PLs)

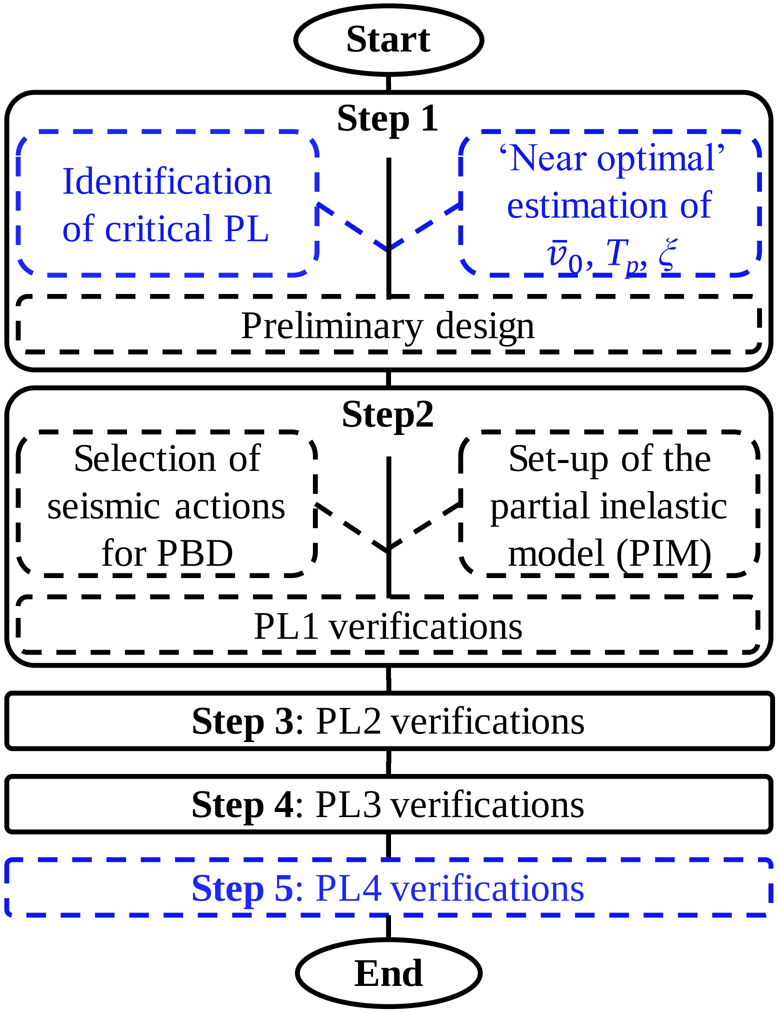
Concluding Remarks on PBD of ductile pier bridges

- *Application of Def-BD:*
  - Operationality PL: governed the design, target deformation actually reached
    Damage-limitation PL: not critical (demand similar to $\rho_{w,min}$ requirements)
    Collapse-prevention PL: critical (↔stability criterion) for bearing deformations
  - Very good prediction of structural response, while resulting in safe design
  - No practical limitations re. structural irregularity, level of analysis sophistication

- *Comparison with MDDBD:*
  - Incorporation of advanced analysis tools (i.e. NLRHA, section analysis) in the case of Def-BD (smaller D) leads to significant cost reduction:
    Zone II: long. steel: 42%, transv. steel: 17%, concrete (piers): 36%
    Zone III: long. steel: 51%, transv. steel: 20%, concrete (piers): 28%
  - Increased computational time and effort

- *Comparison with Code-BD:*
  - Code-BD vs. Def-BD: less damage under the same 'design' earthquake (due to adopted conservatism ↔ increased cost)
  - Def-BD enhanced and controlled structural performance under multiple PLs

Def-BD: Seismically isolated bridges



Seismic hazard		Structural performance			
EQ	**T_R (yrs)**	**SP1**	**SP2**	**SP3**	**SP4**
EQI	<50	Ductile		-	-
EQII	50-100	Isolated	Ductile		-
EQIII	500-1000		Isolated	Ductile	
EQIV	~2500			Isolated	Ductile

Service	Full	Operational	Limited	Disrupted
Damage	Negligible	Limited	Significant	Severe
Repair	No/Economic	Economic	Feasible	Non-feasible

Seismic hazard		Isolated bridges: Analysis type per PL			
EQI	<50	GDE	-	-	-
EQII	50-100	NL	GDE+NL	-	-
EQIII	500-1000	NL	GDE+NL	GDE+NL	-
EQIV	~2500	-	GDE+NL	GDE+NL	NL

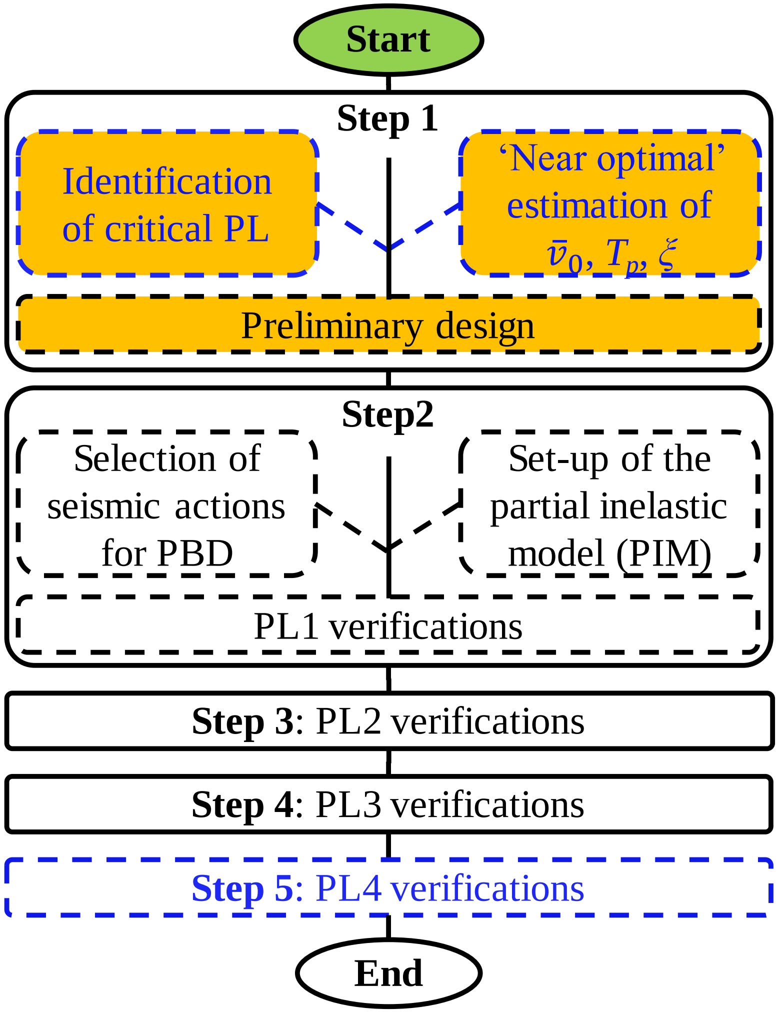
Importance Class

Non-essential, Ordinary, Essential

Type of analysis

GDE: Generalised design equations

NL: Nonlinear dynamic



Seismic hazard		Structural performance			
EQ	T_R (yrs)	SP1	SP2	SP3	SP4
EQI	<50	Ductile		-	-
EQII	50-100	Isolated	Ductile		-
EQIII	500-1000		Isolated	Ductile	
EQIV	~2500			Isolated	Ductile

Service		Full	Operational	Limited	Disrupted
Damage		Negligible	Limited	Significant	Severe
Repair		No/Economic	Economic	Feasible	Non-feasible

Seismic hazard		Isolated bridges: Analysis type per PL			
EQI	<50	GDE	-	-	-
EQII	50-100	NL	GDE+NL	-	-
EQIII	500-1000	NL	GDE+NL	GDE+NL	-
EQIV	~2500	-	GDE+NL	GDE+NL	NL

Importance Class

Non-essential, Ordinary, Essential

Type of analysis

GDE: Generalised design equations

NL: Nonlinear dynamic

Direct estimation of peak response in RDOF passive systems

Extension of Ryan & Chopra (2004) method:

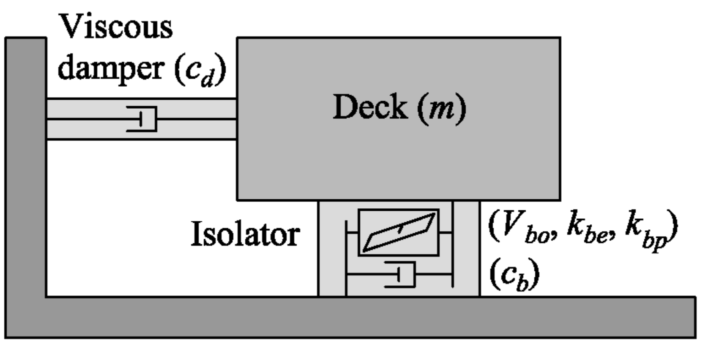
- Linear/bilinear isolators, viscous dampers (linear + nonlinear), combinations
- Generalised design equations (GDEs) for both max u_0, $\ddot{U}_0$
- *Bidirectional excitation + linear viscous damping*
- *Code-based target spectra* (e.g. EN1998-1 'Type 1')

❖ *Isolated SDOF system with linear viscous damping (1 condition)*

$$m\ddot{u}(t)+\left(c_e+c_d\right)\dot{u}(t)+V_0 z\left(t,k_e,u,\dot{u}\right)+k_p u(t)=-m\ddot{u}_g(t)$$

$$\bar{\ddot{u}}(t)=\frac{\ddot{u}(t)}{u_r},\quad \bar{\ddot{u}}_g(t)=\frac{\ddot{u}_g(t)}{\ddot{u}_{g0}},\quad u_r=\frac{V_0}{k_p}$$

$\bar{v}_0=V_0/(mg)$



$$\eta=\frac{\bar{v}_0 g}{\omega_D \dot{u}_{g0}}=\frac{\omega_p^2 u_r}{\omega_D \dot{u}_{g0}}\rightarrow u_r=\frac{\eta\,\omega_D \dot{u}_{g0}}{\omega_p^2}$$

→ strength normalised to seismic intensity

$$\bar{\ddot{u}}(t)+2\omega_p\left(\xi_e+\xi_d\right)\bar{\dot{u}}(t)+\omega_p^2 z\left(t,k_e,u,\dot{u}\right)+\omega_p^2\bar{u}(t)=-\left(\omega_p^2/\eta\omega_D\right)\bar{\ddot{u}}_g(t)$$

$$\ddot{\bar{u}}(t) + 2\omega_p(\xi_e+\xi_d)\dot{\bar{u}}(t) + \omega_p^2 z(t,k_e,u,\dot{u}) + \omega_p^2\bar{u}(t) = -\left(\omega_p^2/\eta\omega_D\right)\ddot{\bar{u}}_g(t)$$

$\xi = 0.05:0.30$

$\eta = 0:1.5$

$T_p = 1:5.0$ s

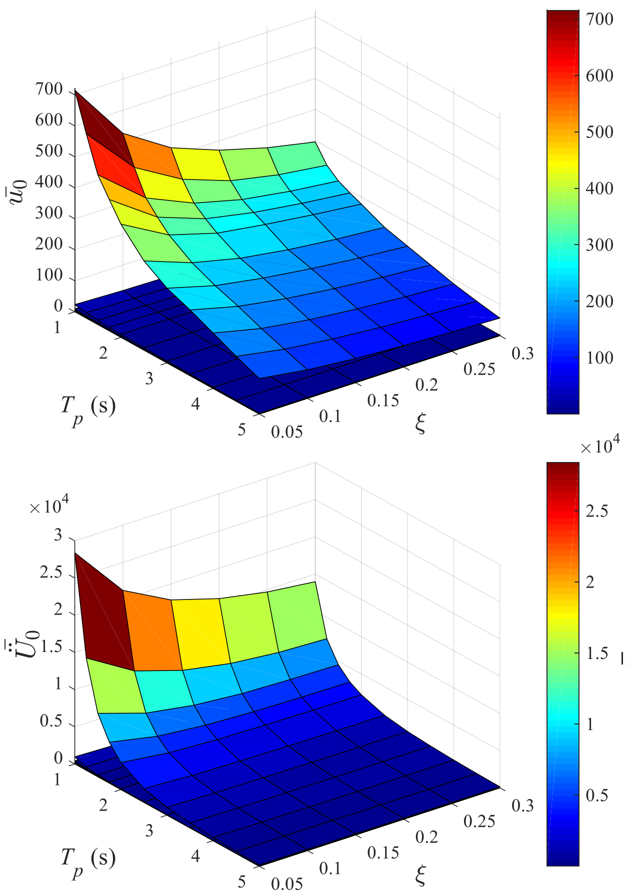
artificial records closely matching EN1998-1

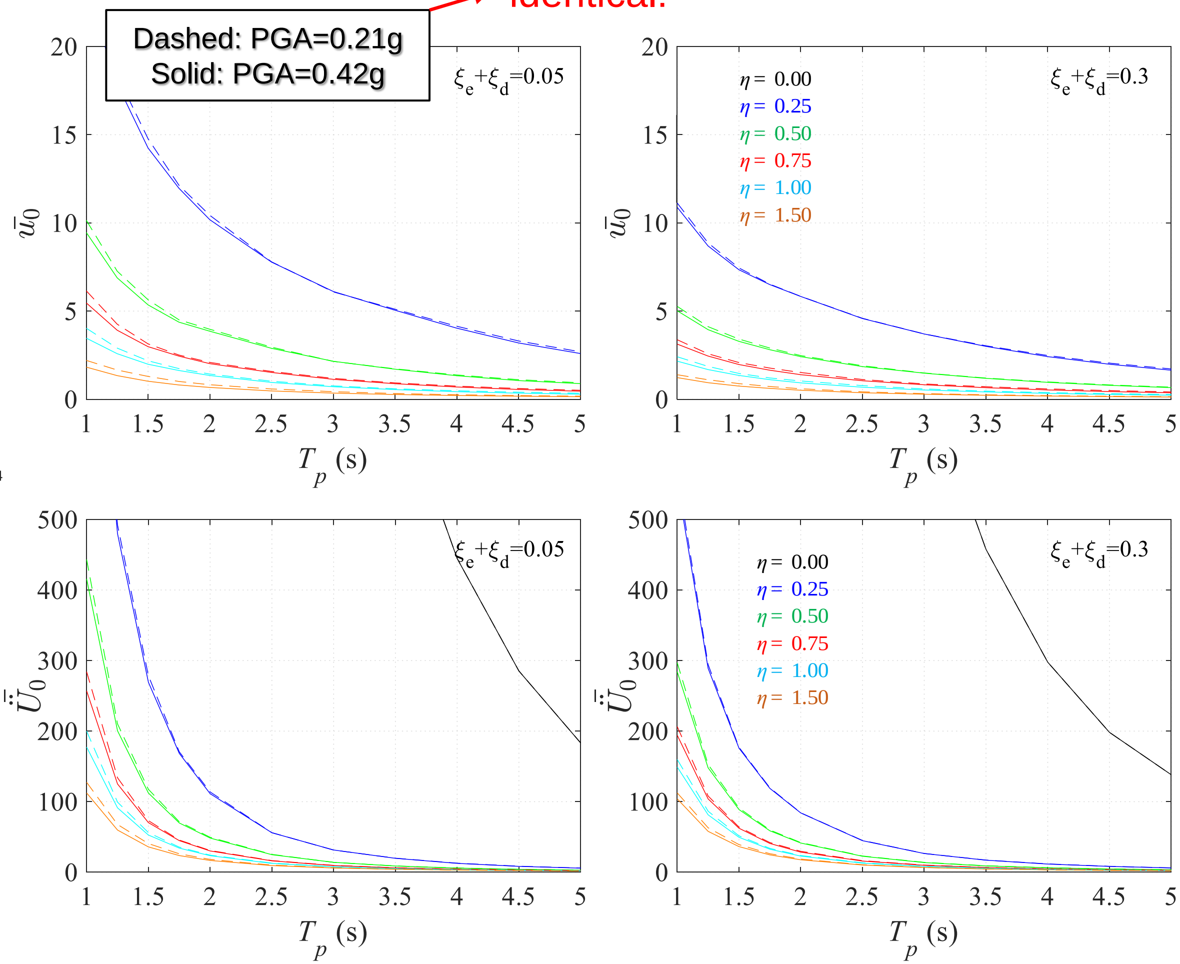
$\bar{u}_0, \ddot{\bar{U}}_0$:

$\perp$ $\ddot{u}_{g0}$

$\not\perp$ freq. content

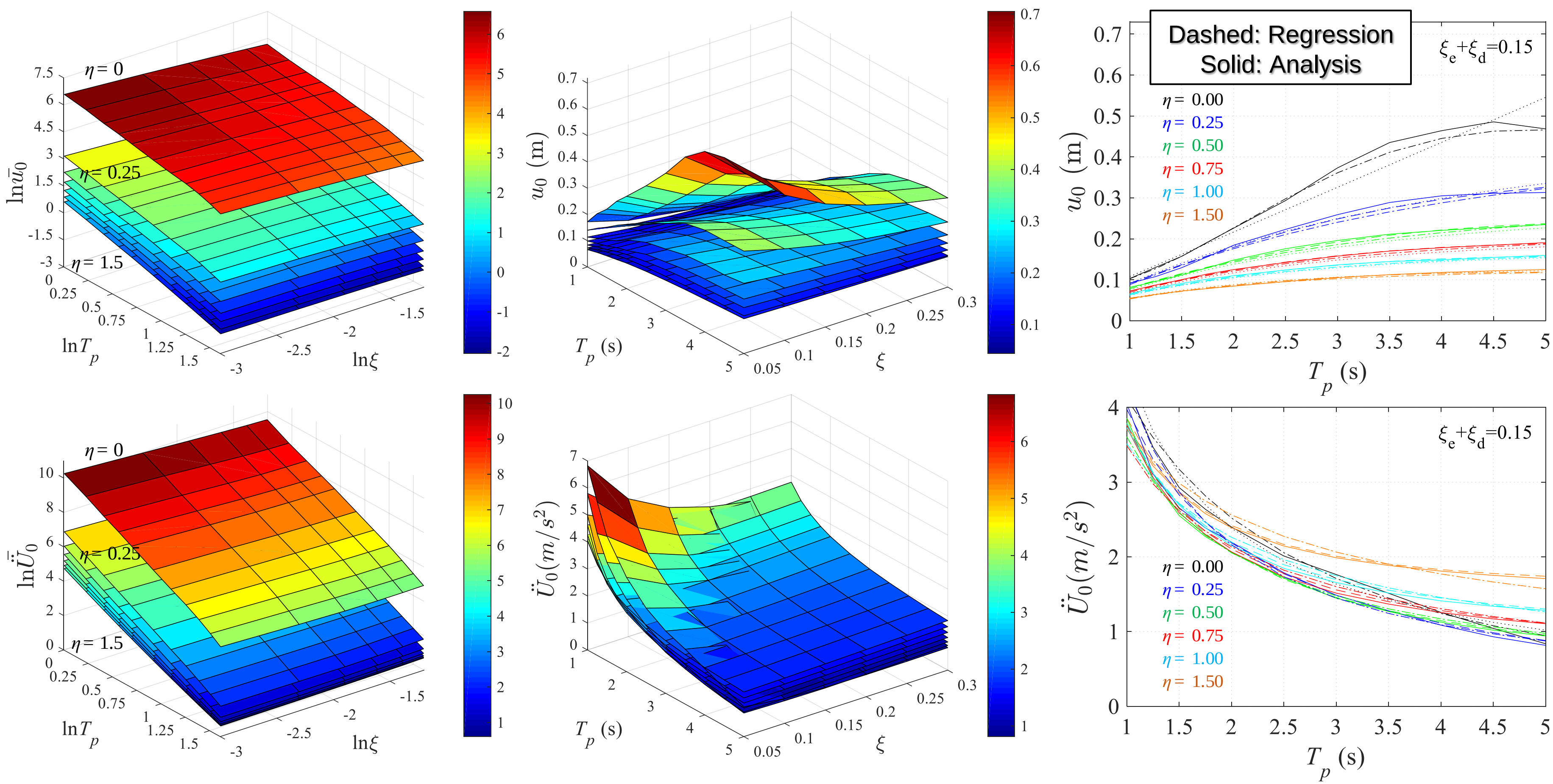
identical!





- Regression analysis → GDEs → Direct estimates of u_0, $\ddot{U}_0$:
  - for $\bar{v}_0$, T_p, ξ
  - for different PLs (i.e. different intensity <u>but</u> common frequency content ↔ spectrum)

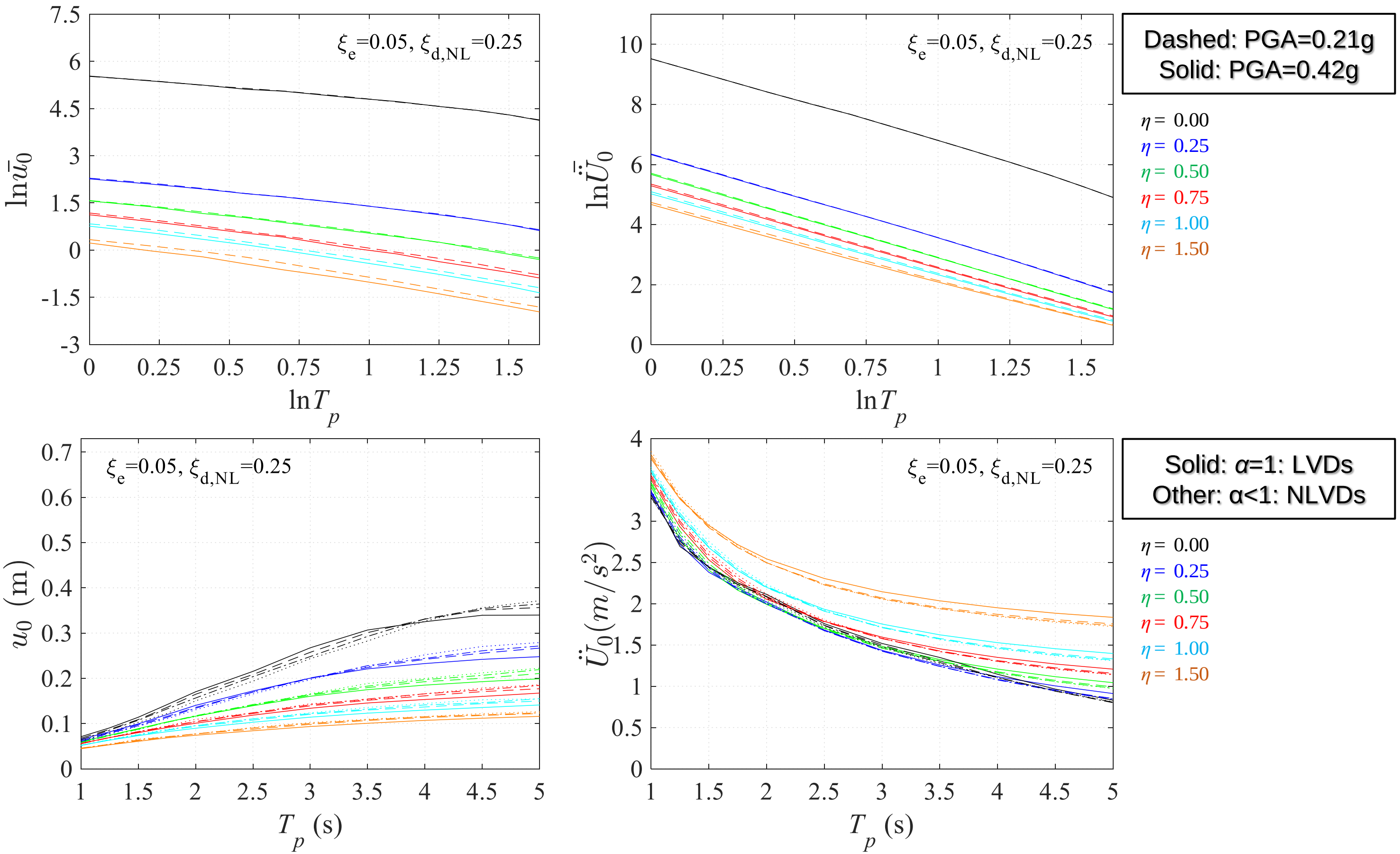
$$\left\{u_0,\ddot{U}_0\right\} = \frac{0.362e^{\text{int}}}{2\pi g}\xi^{(\beta+\gamma\ln\eta+\delta\ln T_p)}\eta^{(1+\varepsilon+\zeta\ln\eta+\kappa\ln T)}T_p^{(2+\lambda+\mu\ln T_p+\nu(\ln T_p)^2)}a_g \qquad \eta = 4.31\frac{\bar{v}_0 g}{a_g}$$



➤ *Isolated SDOF system with nonlinear viscous damping (2 conditions)*

$$\ddot{u}(t)+\left[\left(c_{e,L}\dot{u}(t)+c_{d,NL}\,\mathrm{sgn}\left(\dot{u}(t)\right)\left|\dot{u}(t)\right|^{\alpha}\right)/m\right]+\left[\bar{v}_0 g z\left(t,k_e,u,\dot{u}\right)+\omega_p^2 u(t)\right]=-\ddot{u}_g(t)$$

$$\ddot{\bar{u}}(t)+\left[2\omega_p\xi_{e,L}\dot{\bar{u}}(t)+\frac{2\omega_p\xi_{d,NL}\left(\bar{u}_0\omega_p\right)^{1-\alpha}}{f(\Gamma,\alpha)}\mathrm{sgn}\left(\dot{\bar{u}}(t)\right)\left|\dot{\bar{u}}(t)\right|^{\alpha}\right]+\left[\omega_p^2\left(z\left(t,k_e,u,\dot{u}\right)+\bar{u}(t)\right)\right]=-\left(\omega_p^2/\eta\omega_D\right)\ddot{\bar{u}}_g(t)$$



$\bar{u}_0, \ddot{\bar{U}}_0$:

$\perp$ $\ddot{u}_{g0}$

$\not\perp$ freq. content

$\bar{u}_0, \ddot{\bar{U}}_0$:

$\perp$ nonlinearity *a*

➢ *Isolated 2DOF system with linear viscous damping (bidirectional excitation)*

$$\begin{bmatrix} \ddot{\bar{u}}_x(t) \\ \ddot{\bar{u}}_y(t) \end{bmatrix} + 2\omega_p\left(\xi_{e,L}+\xi_{d,L}\right)\begin{bmatrix} \dot{\bar{u}}_x(t) \\ \dot{\bar{u}}_y(t) \end{bmatrix} + \omega_p^2\left(\begin{bmatrix} z_x\left(t,k_e,\mathbf{u},\dot{\mathbf{u}}\right) \\ z_y\left(t,k_e,\mathbf{u},\dot{\mathbf{u}}\right) \end{bmatrix} + \begin{bmatrix} \bar{u}_x(t) \\ \bar{u}_y(t) \end{bmatrix}\right) = -\left(\omega_p^2/\eta\omega_D\right)\begin{bmatrix} \ddot{\bar{u}}_{gx}(t) \\ \ddot{\bar{u}}_{gy}(t) \end{bmatrix}$$

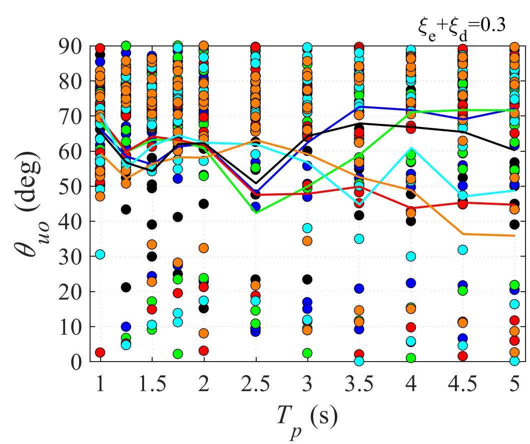
$$\eta = \frac{\bar{v}_0 g}{\omega_D \ddot{u}_{g0,2D}} = \frac{\omega_p^2 u_r}{\omega_D \ddot{u}_{g0,2D}} \qquad \ddot{u}_{g0,2D} = \frac{\sqrt{\ddot{u}_{g0,x}^{\;2}+\ddot{u}_{g0,y}^{\;2}}}{\sqrt{2}}$$

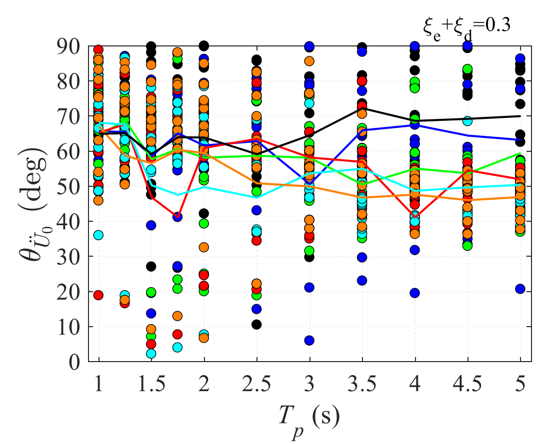
$$\bar{u}_{0,2D} = \max_t \sqrt{\bar{u}_x(t)^2+\bar{u}_y(t)^2}, \quad \ddot{\bar{U}}_{0,2D} = \max_t \sqrt{\ddot{\bar{U}}_x(t)^2+\ddot{\bar{U}}_y(t)^2}$$

$$\theta_{\bar{u}_0} = \tan^{-1}\left(\left|\frac{\bar{u}_y(t_i)}{\bar{u}_x(t_i)}\right|\right) = \theta_{u_0}, \quad \theta_{\ddot{\bar{U}}_0} = \tan^{-1}\left(\left|\frac{\ddot{\bar{U}}_y(t_j)}{\ddot{\bar{U}}_x(t_j)}\right|\right) = \theta_{\ddot{U}_0} \quad \left(0 \le \theta \le 90^o\right)$$

$$\bar{u}_0, \ddot{\bar{U}}_0 : \perp \ \ddot{u}_{g0}$$

- Large dispersion of $\theta \rightarrow$ *$u_{0,2D}$, $\ddot{U}_{0,2D}$ at any random direction*

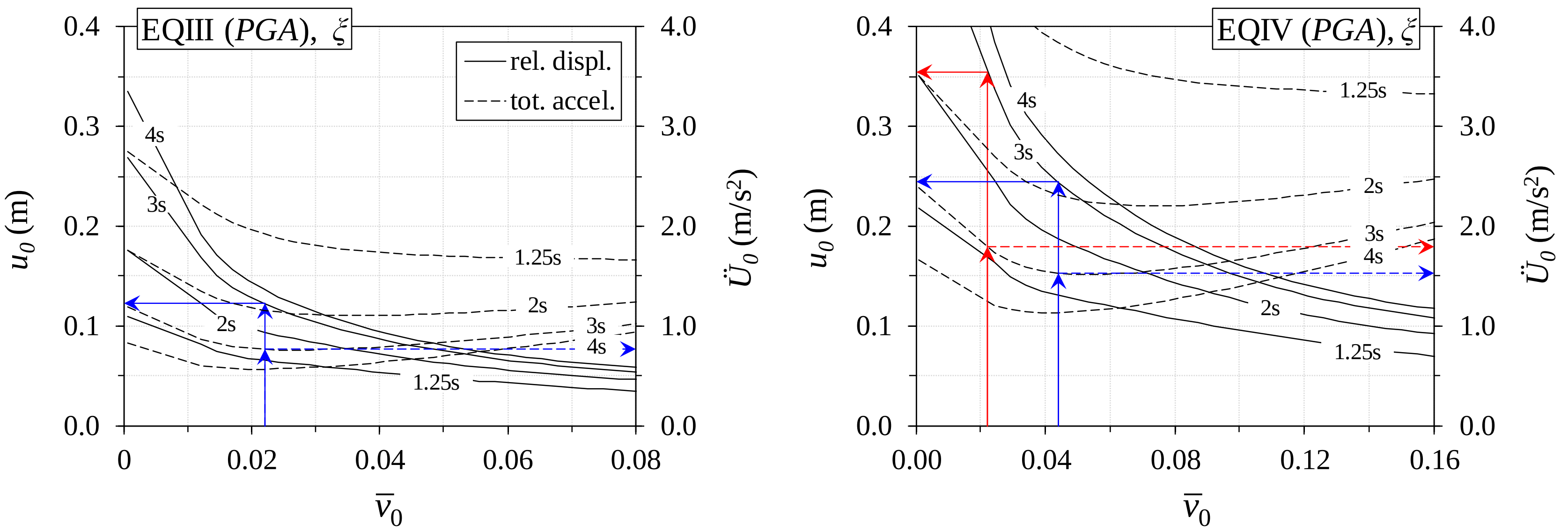




Step 1: Preliminary design

➢ *'Near-optimal' performance under a reference earthquake event*

- 'Near-optimal' → ['Near-minimum' $\ddot{U}_{deck}$] + [controlled u_{isol}, $\bar{v}_0$, T_p, ξ]
- optEQ(SP2) → suboptimal u response under EQ(SP3) → isolators cost (↑)
  optEQ(SP3) → suboptimal $\ddot{U}$ response under EQ(SP2) → ρ_l in piers (↑)



- GDEs: *α*=1.0 (linear viscous damping)
  - Inelastic spectra for the adopted EQIII, EQIV intensities
  - Isolation systems with 'near-optimal' performance under EQIII, EQIV
  - Systems of different isolation + energy dissipation devices

- *(‘Near-optimal’ performance under a reference earthquake event)*

- GDEs: α<1.0 (non-linear viscous damping)

i. optEQ: GDEs assuming α=1.0 → $\{u_0, \ddot{U}_0\}=f(\xi, \bar{v}_0, T_p, \alpha<1)$

ii. ξ_e=0.05 (elastomer)

$\xi_d=\xi-\xi_e$

$$c_e = 2m\omega_p\xi_e, \quad c_d = \frac{2m\omega_p\xi_d\left(u_0\omega_p\right)^{1-\alpha}}{f(\Gamma,\alpha)}$$

$$f(\Gamma,\alpha) = \frac{2^{2+\alpha}\Gamma^2(1+\alpha/2)}{\pi\Gamma(2+\alpha)}$$

u_0 (Step i)

iii. Variability of DPs of devices: Updated c_e, c_d, $\bar{v}_0$, T_p

$DIV(u_0, \ddot{U}_0) > 1-5\%$

iv. $\eta = 4.31\bar{v}_0 g/a_g, \quad \xi_e = c_e/2m\omega_p, \quad \xi_d = c_d f(\Gamma,\alpha)/2m\omega_p\left(u_0\omega_p\right)^{1-\alpha}$

$\alpha = 1.0$ (first iteration)

→ η, $\xi=\xi_e+\xi_d$

v. EQ: ξ, η (Step iv), T_p (Step iii) → GDEs → $\{u_0, \ddot{U}_0\}$

α=1 → Step v
α=1 & η=0 → linearity

→ optimal selection should be made cautiously, except in the case of inherently linear systems (linear viscous dampers + linear isolators) wherein the reference seismic actions have no effect on optimal response

- *'Near-optimal' performance under a reference earthquake event*
  - Select $\bar{v}_0$, T_p, ξ → type of devices [performance + cost + availability]
  - Distribution of $\bar{v}_0$, ξ to a sufficient number of units:
    - Performance: Weight distribution, torsional effects, uniformity in pier stiffness
    - Reliability (no. of devices) + Cost (identical devices, testing) + Availability (*no constraint for maintaining classically damped systems*)
  - Lower bound (LB) and upper bound (UB) design properties

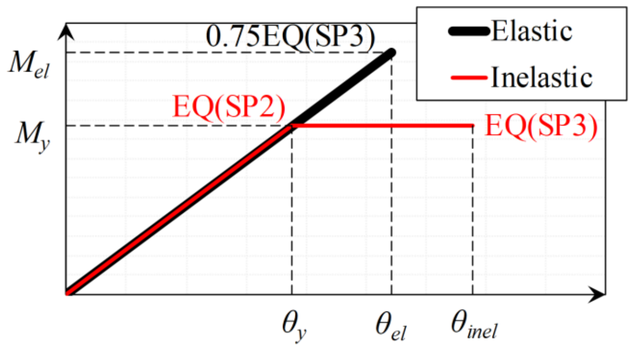
- *Substructure design based on $\ddot{U}_0$*
  - Distribution of $m\ddot{U}$ in piers / abutments (simplified linear analysis)
  - Similarly to Step 1 for bridges with energy dissipation in the piers:

    0.75EQ(SP2) → quasi-elastic pier response

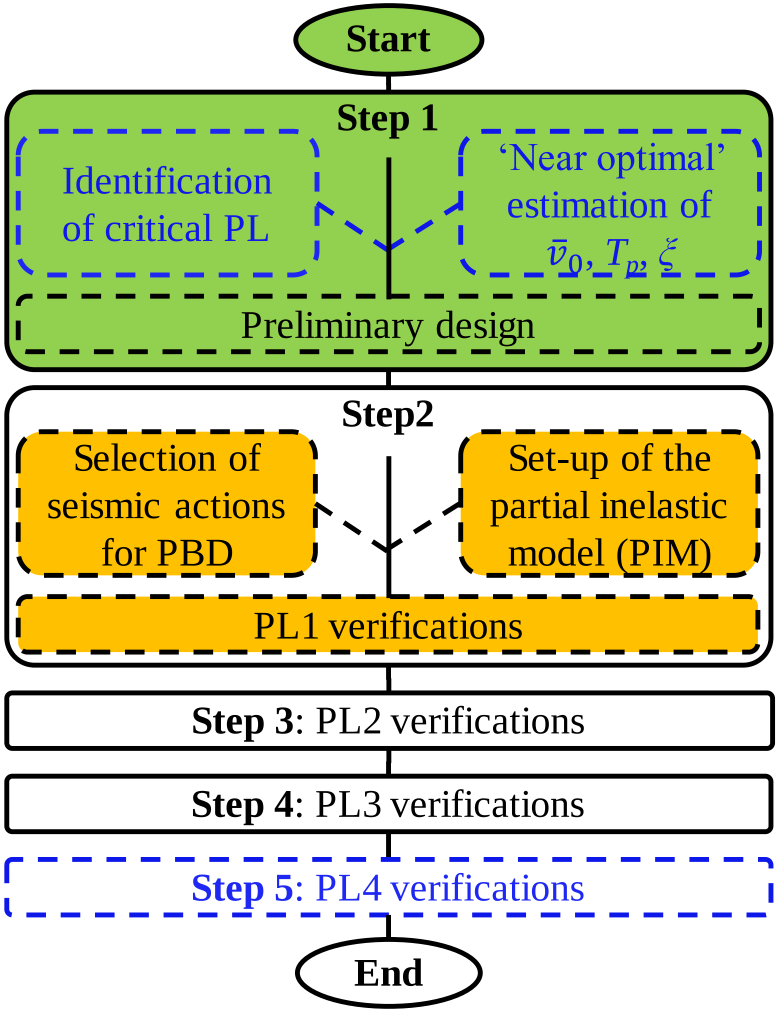
    0.75EQ(SP3) → controlled inelastic pier response (strength ↔ allowable damage)

- *Bidirectional Excitation*
  - $u_{0,2D}$: sustained in any random direction
  - $m\ddot{U}_{0,2D}$: // principal axes of the bridge

    (→ conservatism)



Def-BD: Seismically isolated bridges: Step 2



Seismic hazard		Structural performance			
EQ	**T_R (yrs)**	**SP1**	**SP2**	**SP3**	**SP4**
EQI	<50	Ductile		-	-
EQII	50-100	Isolated	Ductile		-
EQIII	500-1000		Isolated	Ductile	
EQIV	~2500			Isolated	Ductile

Service	Full	Operational	Limited	Disrupted
Damage	Negligible	Limited	Significant	Severe
Repair	No/Economic	Economic	Feasible	Non-feasible

Seismic hazard		Isolated bridges: Analysis type per PL			
EQI	<50	GDE	-	-	-
EQII	50-100	NL	GDE+NL	-	-
EQIII	500-1000	NL	GDE+NL	GDE+NL	-
EQIV	~2500	-	GDE+NL	GDE+NL	NL

Importance Class

Non-essential, Ordinary, Essential

Type of analysis

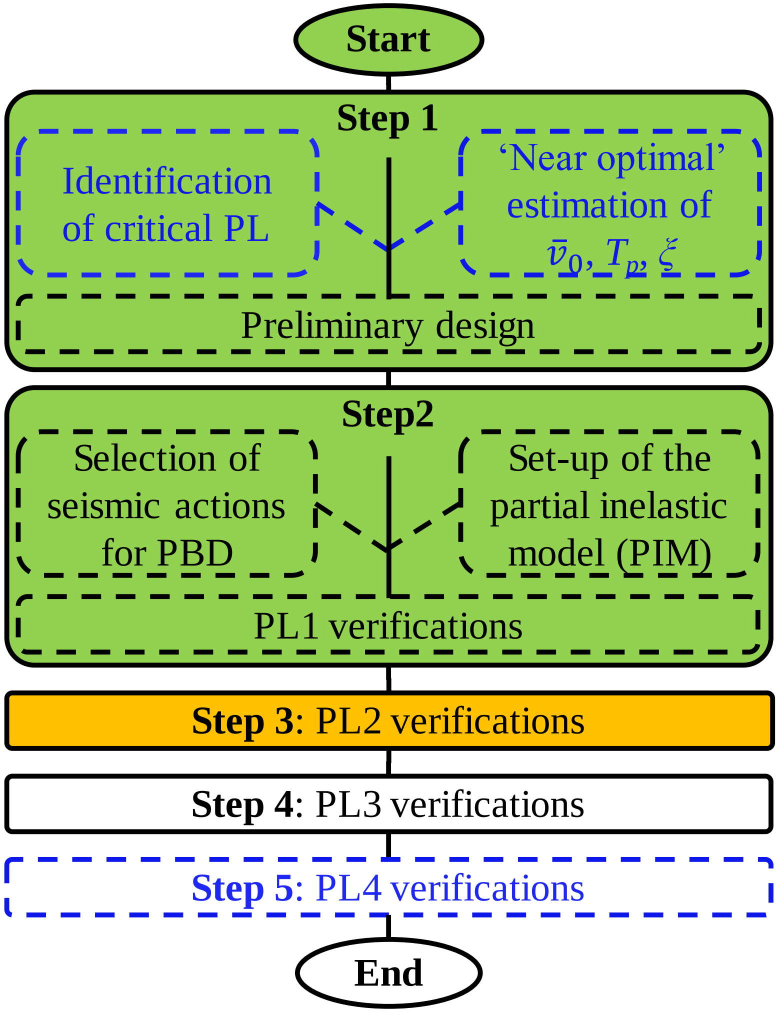
GDE: Generalised design equations

NL: Nonlinear dynamic

Step 2: SP1 verifications

- ***Set-up of the partially inelastic model (PIM) of the bridge***
  - Hysteretic isolators & dampers: nonlinear spring & dashpot elements
  - Remaining parts of the bridge: elastic members
    - Piers: stiffness EI_y (M-φ analysis based on ρ_l from Step 1) or EI_g *(minor effect)*
    - Prestressed concrete deck: EI_g
- ***Selection of seismic actions*** → rotation of pairs of components into their principal axes and consecutive application along the principal axes of the bridge (straight bridges)
- ***Verifications***
  - Operationality → 'full' service (no closure)
    → adequate restoring capability u_{res} / u_0 ($-u_r \le u_{res} \le u_r$)
    → alternatively: $V \le V_y$ (difficult to apply in LRBs)
  - Structural performance → isolators: 'negligible' (or no-) damage ($\gamma_b \le 1.0/SF$)
    → piers: no damage
  - Modifications (if needed) → mechanical properties of isolators
    conformity to Step 1 (use of GDEs, not NLRHA)
    → alternative (less economical) design: e.g. sacrificial devices
  - Non-essential bridges: NLRHA is omitted (verifications using GDEs)

Def-BD: Seismically isolated bridges: Step 3



Seismic hazard		**Structural performance**			
EQ	T_R **(yrs)**	**SP1**	**SP2**	**SP3**	**SP4**
EQI	<50	Ductile		-	-
EQII	50-100	Isolated	Ductile		-
EQIII	500-1000		Isolated	Ductile	
EQIV	~2500			Isolated	Ductile

Service	Full	Operational	Limited	Disrupted
Damage	Negligible	Limited	Significant	Severe
Repair	No/Economic	Economic	Feasible	Non-feasible

Seismic hazard		**Isolated bridges: Analysis type per PL**			
EQI	<50	GDE	-	-	-
EQII	50-100	NL	GDE+NL	-	-
EQIII	500-1000	NL	GDE+NL	GDE+NL	-
EQIV	~2500	-	GDE+NL	GDE+NL	NL

Importance Class

Non-essential, Ordinary, Essential

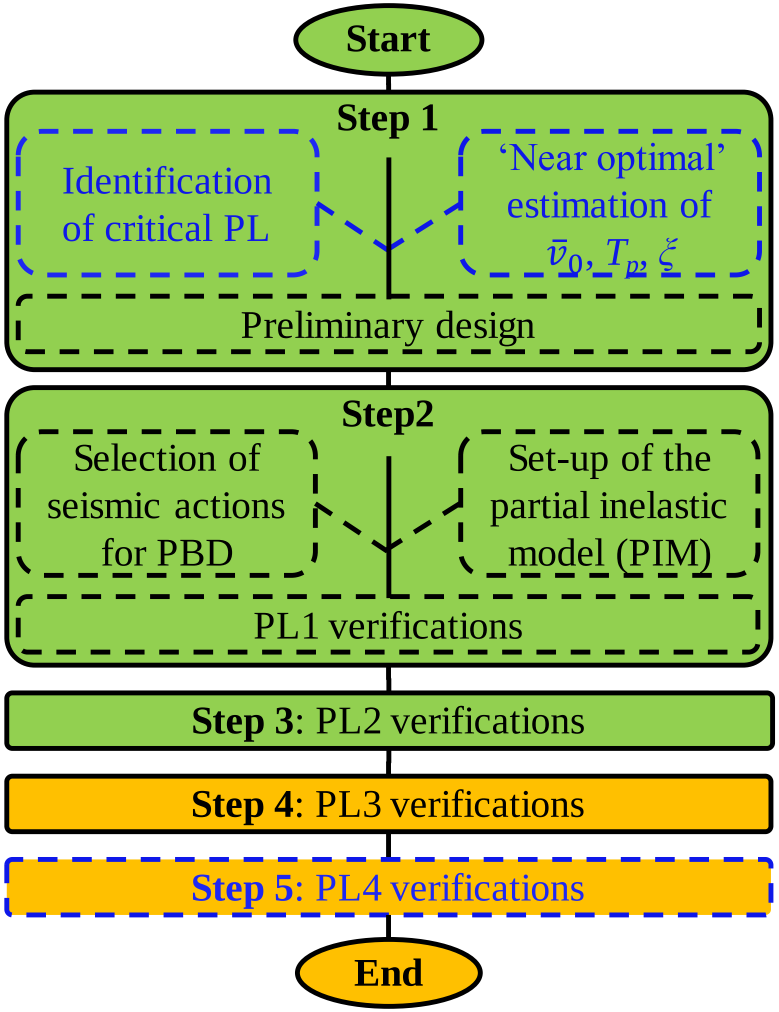
Type of analysis

GDE: Generalised design equations

NL: Nonlinear dynamic

Step 3: SP2 verifications (ensure that the extent of damage is such that the bridge can be repaired after the earthquake without significant disruption of service)

- ***Set-up of the partially inelastic model (PIM) of the bridge***
  - **Hysteretic isolators** & **dampers**: nonlinear spring & dashpot elements (from Step 2)
  - **Piers**: stiffness EI_y (M-φ analysis based on ρ_l from Step 1)
- ***Scaling of seismic actions*** (→ EQ(SP2))
- ***Verifications***
  - **Operationality** → no significant disruption of service
    → **adequate restoring capability** (e.g. **EN1998-2**: $u_0/u_r \geq 0.5$)
  - **Structural performance** → isolators: 'limited' damage ($\gamma_q \leq 1.0$)
    → piers: essentially elastic response ($M<M_y$)
  - **Modifications** → mechanical properties of **isolators** (conformity to Steps 1, 2)
    → ρ_l in **piers**: max requirement from Steps 1, 3



Seismic hazard		Structural performance			
EQ	**T_R (yrs)**	**SP1**	**SP2**	**SP3**	**SP4**
EQI	<50	Ductile		-	-
EQII	50-100	Isolated	Ductile		-
EQIII	500-1000		Isolated	Ductile	
EQIV	~2500			Isolated	Ductile

Service	Full	Operational	Limited	Disrupted
Damage	Negligible	Limited	Significant	Severe
Repair	No/Economic	Economic	Feasible	Non-feasible

Seismic hazard		Isolated bridges: Analysis type per PL			
EQI	<50	GDE	-	-	-
EQII	50-100	NL	GDE+NL	-	-
EQIII	500-1000	NL	GDE+NL	GDE+NL	-
EQIV	~2500	-	GDE+NL	GDE+NL	NL

Importance Class

Non-essential, Ordinary, Essential

Type of analysis

GDE: Generalised design equations

NL: Nonlinear dynamic

Step 4: SP3 verifications ('near-optimal' performance sought in Step 1 and modified in 2, 3)

- *Set-up of the partially inelastic model (PIM) of the bridge*
  - Piers: yielding elements (M-φ analysis based on ρ_l from Step 3)
- *Verifications under EQ(SP3)*
  - Operationality → limited service (*not* explicitly checked)
  - Structural performance

    → isolators: 'ultimate' deformations, stability, uplift

    → dampers: 'ultimate' deformations, forces (F_{max})

    → piers: controlled inelastic flexural response

    → deck: design in flexure, abutments-backfill: activation should better be avoided

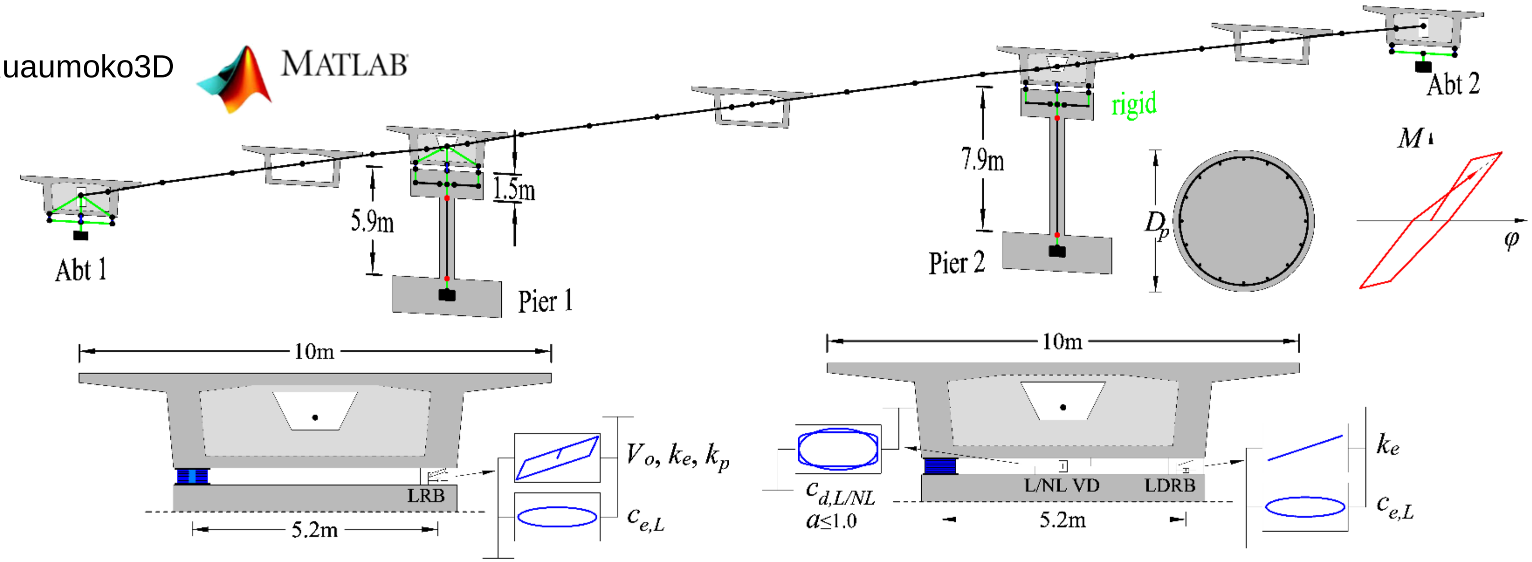
Step 5: SP4 verifications (typically excluded from the PO of Ordinary bridges)

- *Verifications under EQ(SP4)*
  - Non-essential bridges: complex pier-seismic link-deck / abutment-backfill-deck interactions (exhaustion of clearances, pounding effects)

    → simplified treatment: 2 sets of analysis under EQ(SP4) (PIM from Step 4) considering free and constrained movement of the deck at the abutments
  - In all other cases: detailing for confinement, member shear design

- *Studied bridge* (similar to T7 Overpass → Ordinary bridge)
  - Lead rubber bearings (LRBs)
  - Low damping rubber bearings (LDRBs) + Linear viscous dampers (LVDs)
  - Low damping rubber bearings (LDRBs) + NL viscous dampers of α=0.2 (NLVDs)
  - Lead rubber bearings (LRBs) 2D + comparison with EN 1998-2 (CEN 2005)

Ruaumoko3D MATLAB



- *Modelling*
  - Piers: 'standard' point hinge approach (modified Takeda)
  - Isolators: linear/nonlinear springs
  - Damping: stiffness proportional + linear / nonlinear dashpots

Performance criteria

Member	SP1	SP2	SP3	SP4
Isolated pier	-	Yield $\varphi \leq \varphi_y$	Conc. spalling $\varphi \leq \varphi(\varepsilon_c$=3.5-4‰)	Ultimate response $\varphi \leq \min\varphi(\varepsilon_{ccu}$, hoop fracture, long. bar buckling/fracture)
Elastomeric bearing	No damage $\gamma_q \leq 1/SF_{\gamma q}$	Yielding of anchor bolts, cracking of pedestals, lower limit for yielding of shims $\gamma_q \leq 1$	Ultimate response $\gamma_q \leq 2.5$, $\gamma_{tot} \leq 7$, tension, stability	Locked Link activation
Bilinear hysteretic isolator (restoring capability)	Full service $u_{res}, \lambda_{acc} \rightarrow 0$ $V \leq V_0$ (or V_y)	Operational $u/u_r \geq 0.5$	-	-
Viscous damper	-	-	Ultimate response $u \leq u_{stroke}$ $F \leq F_R$	Locked Link activation

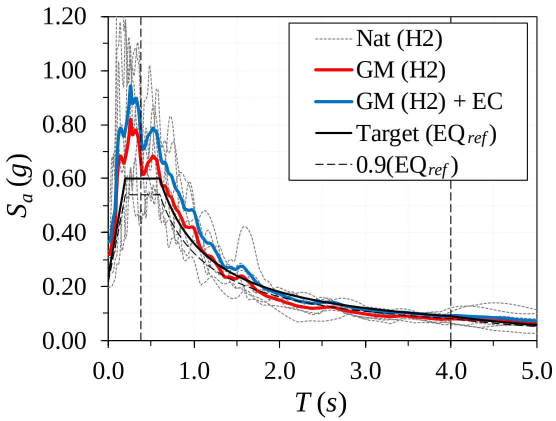
Input motions (Design)

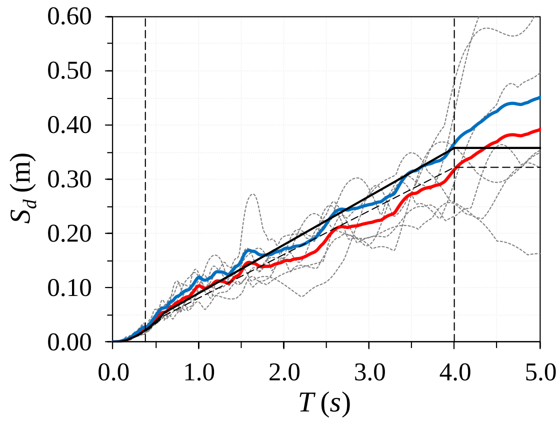
7 records → SF_i (minMSE)
& SF=1.15 (EN1998-2)

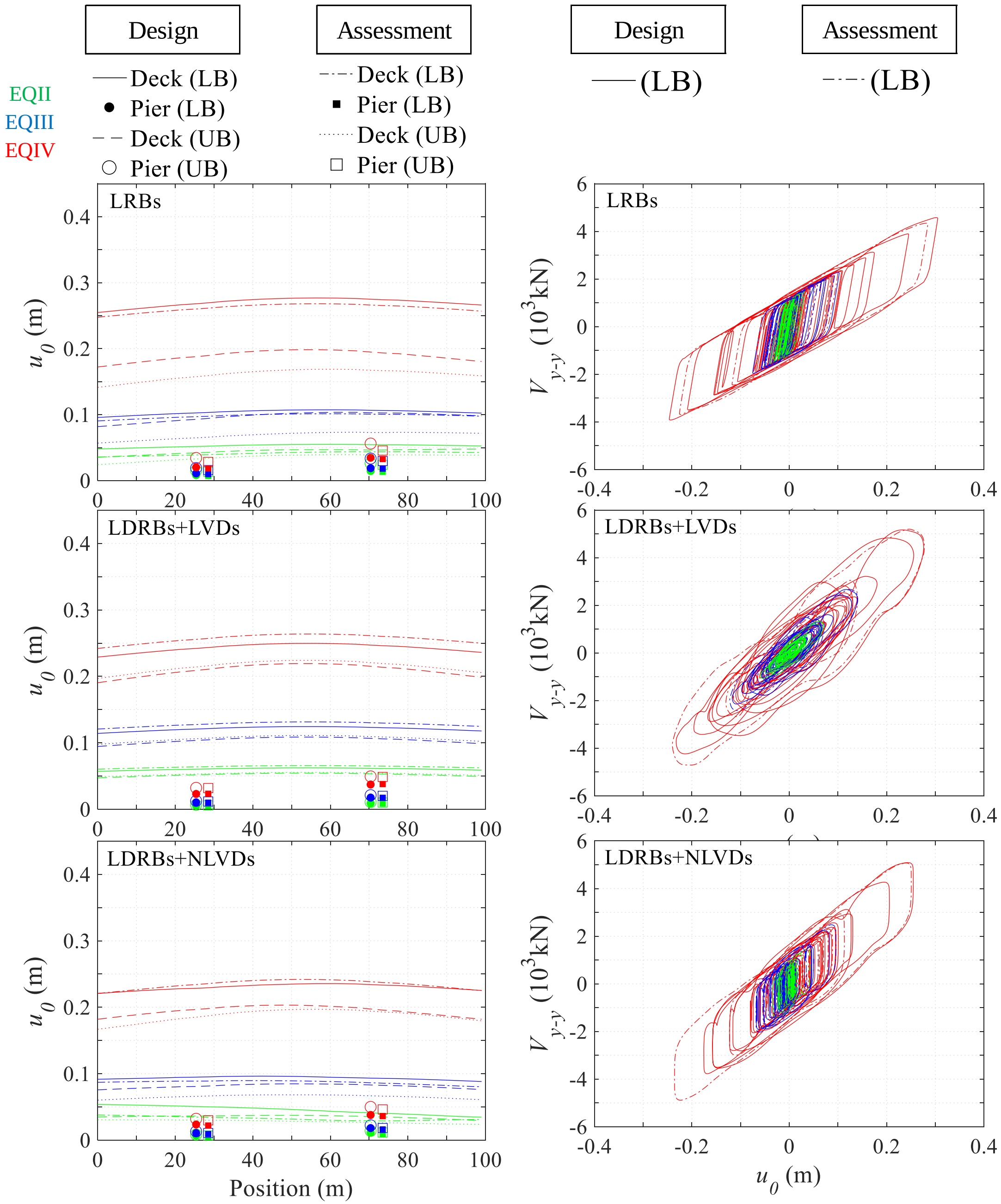
Input motions (Assessment)

10 artificial records

(→ GDEs)

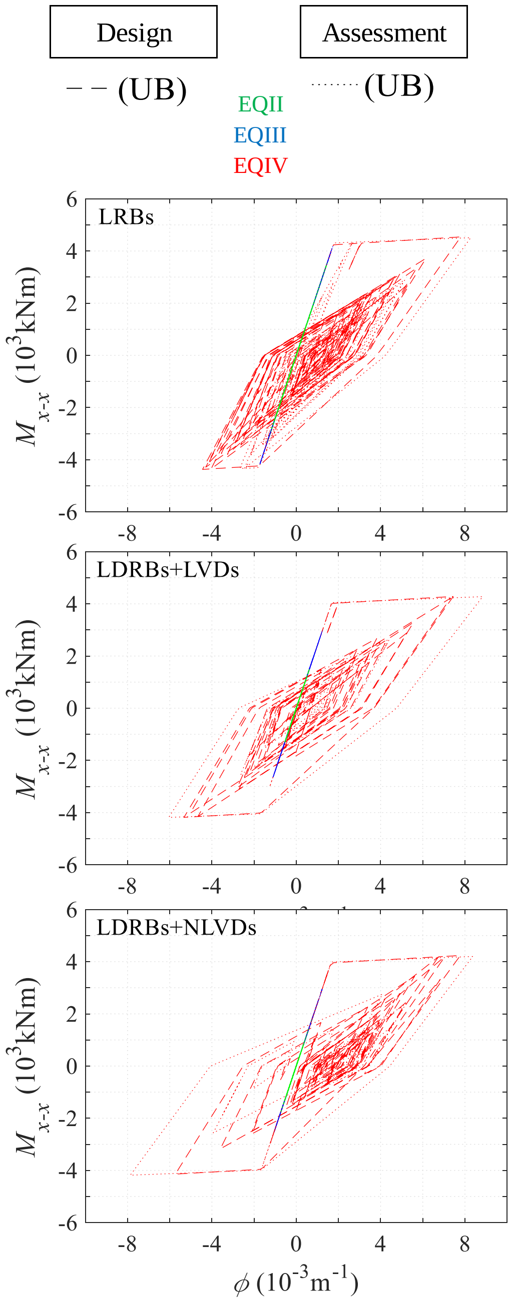






- **Application of Def-BD:**
  - Safe design & close match of Step 1 (GDEs), design, assessment response
    - → Efficient prediction of response through GDEs
    - → Minor effect of substructure

Def-BD: Seismically isolated bridges: Case study



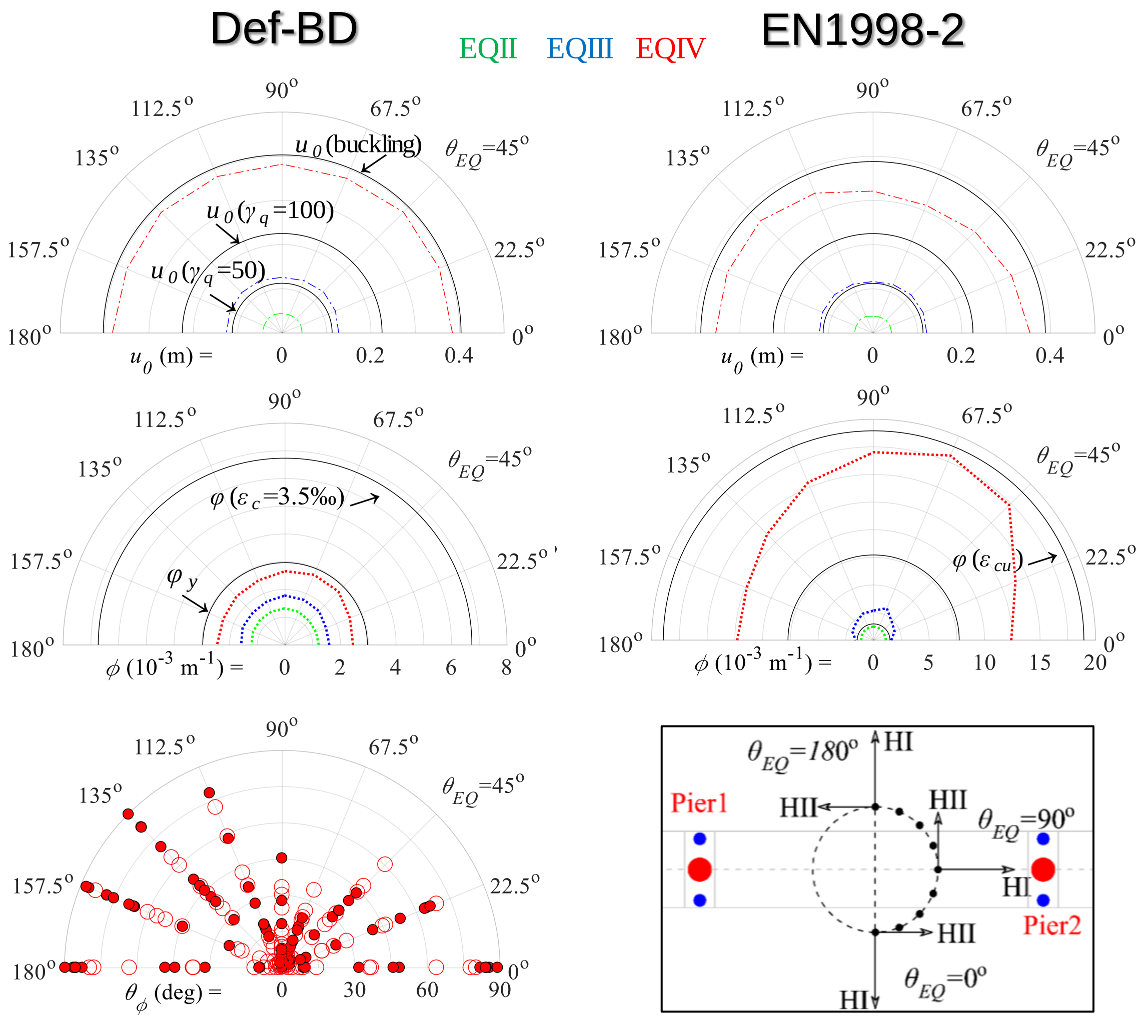
➢ *Application of Def-BD:*

- **SP3-EQIV**: critical for pier (flexure+shear) and isolation system design
  - **Piers**: $M_{y,EQIII(Step3)} < M_{y(Step1)}$ (inelastic pier response critical for design)
  - **Isolators**: allowable vertical stresses, stability, and uplift considerations

- **LRBs, LVDs, NLVDs**: similar performance satisfying all adopted design criteria

LVDs:	↓ ρ_l (13%), ρ_w (9%)	vs. ***LRBs***
NLVDs:	↓ ρ_l (7%), F_{damper} (26%)	vs. ***LVDs***
LRBs:	↓ ρ_l (22%), ρ_w =(38%)	vs. ***ductile-pier***
L(NL)VDs, :	↓ ρ_l (32-37%), ρ_w =(43%)	vs. ***ductile-pier***

- **NLVDs**: reduction **of damper size** demand + **damper force limit** without affecting overall bridge response



Def-BD

- θ_{EQ}=0, 90° adequate for deformation demand estimation
- Elastic response of substructure: $m\ddot{U}_{0,2D}$: // principal axes of the bridge

EN1998-2

- Limitation of the inelastic pier response under the EQ(SP3) rather than EQIV
- Increased number of iterations → underestimation of deformation demand

EN1998-2 → reductions in reinforcing steel demands:

- inconsistent consideration of bidirectional excitation
- limitation of the inelastic pier response under EQIII
- equivalent linearisation approach

Concluding Remarks and Recommendations

Can deformation-based design be useful in the frame of a new generation of codes?

- Simplicity and ease of use: no magical solutions! (i.e. not feasible to have enhanced performance and economy but simpler or easier to use methods…)
  - but for 3-4 decades now, structural design makes full use of software
  - 'hand' methods ('NZ school', DDBD) are suitable for preliminary design only
  - in EC8-2 several iterations are needed since the characteristics of the isolation devices are not known at the beginning of the design procedure
  - more complex, but broader field of application methods like Def-BD emerge as possible candidates for replacing the existing code procedures!

- Performance and economy: enhancement of both was found in cases studied

  For isolated bridges (Def-BD of ductile bridges discussed previously):
  - reliable and stable performance resulting from Def-BD, for *all* PLs
  - in EC8 design (resulting in lower reinforcement demands due to some assumptions made) safety of the isolated structure under EQIV might be an issue, due to large inelastic deformations in substructure elements
  - deficiencies of equivalent linearisation may be more pronounced in systems with velocity-dependent dissipation devices (e.g. fluid viscous dampers)

Thank you for your kind attention!

Websites:

www.city.ac.uk/engineering-maths/staff/professor-andreas-kappos

ajkap.weebly.com/english.html

Design parameters & assumptions

- EC8 ‘Type 1’ elastic spectrum (T_R = 90/475/2500 yrs) (site conditions ‘C’, T_D=4.0s)
- Different seismic hazard zones
- Transverse response of the bridge
- Gap size: Activation of the abutment-backfill system → not considered

Performance criteria

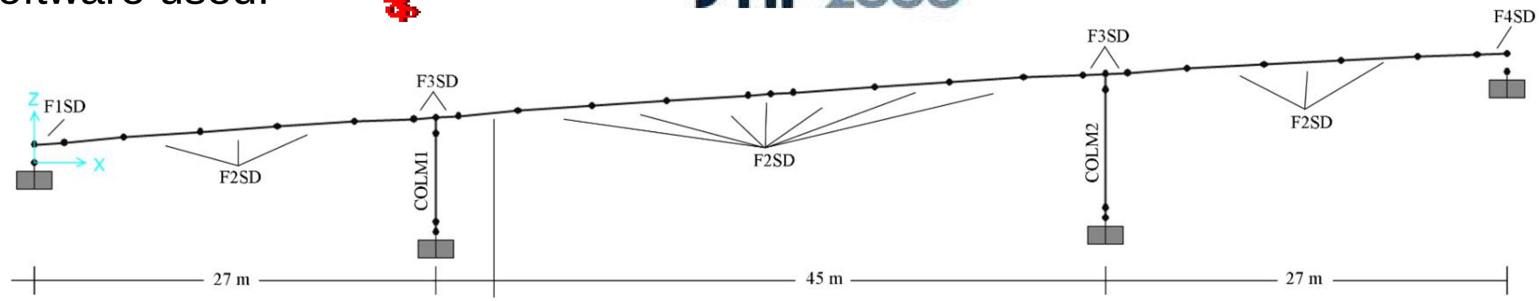
Member	SP1	SP2	SP3	SP4
Ductile pier	Yield	Conc. spalling	Impaired feasibility of repair	Ultimate response
	$\varphi \leq \varphi_y$	$\varphi \leq \varphi(\varepsilon_c$=3.5-4‰)	$\varphi \leq \varphi$(hoop yielding)	$\varphi \leq \min\varphi(\varepsilon_{ccu}$, hoop fracture, long. bar buckling/fracture)
Elastomeric bearing	-	Yielding of anchor bolts, cracking of pedestals, lower limit for yielding of shims	Upper limit for yielding ~ severe bending of shims	Ultimate response
		$\gamma_q \leq 1$	$\gamma_q \leq 1.5$~2	uplift, tension, stability
Seat-type abutment (non-activated)	-	-	Yield	Ultimate response
			$M_{Abt} \leq M_{y,Abt}$	$u \leq u_{slack}$

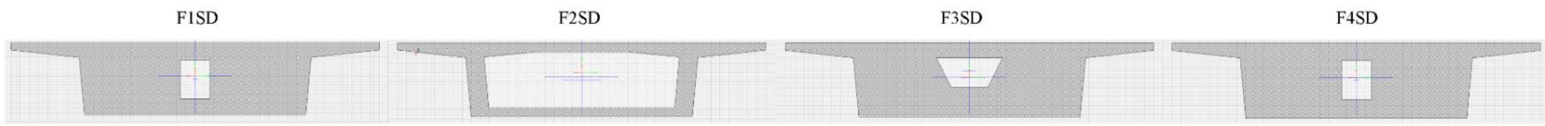
- Pier SP deformations: Based on allowable strains and section analysis

e.g.
$$\mu_{\theta,SP} = 1 + \frac{\theta_{pl,SP}}{\theta_y} = 1 + \frac{3 \cdot \left(\varphi_{SP} - \varphi_y\right) \cdot L_{pl}}{\varphi_y \cdot h_{eq}}$$

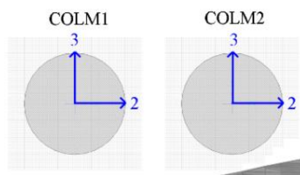
➢ *Analysis of the bridge*

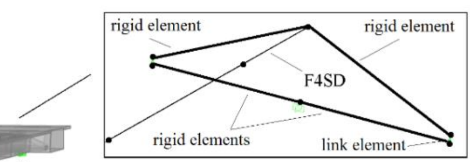
- Software used: Ruaumoko3D SAP2000®

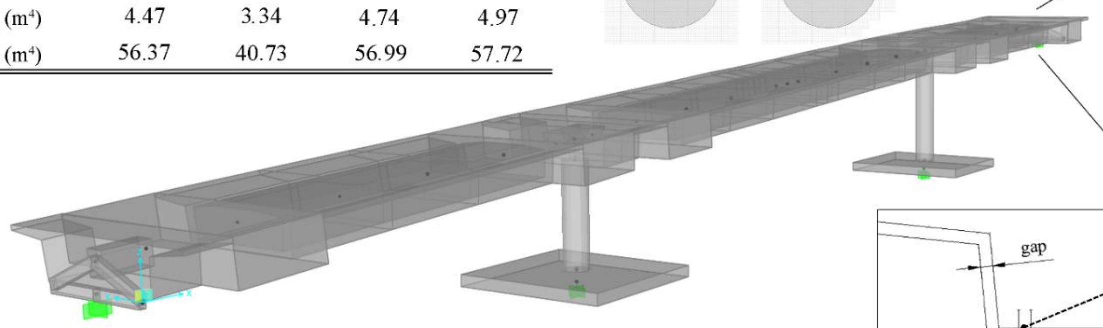


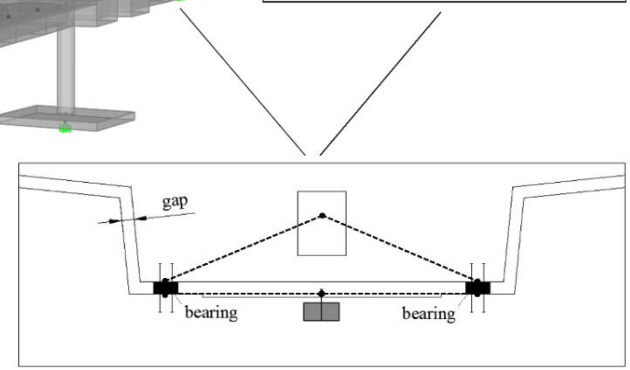


Parameter	F1SD	F2SD	F3SD	F4SD
A (m²)	12.38	5.89	12.35	12.81
I_x (m⁴)	12.28	8.41	12.63	13.54
I_y (m⁴)	4.47	3.34	4.74	4.97
I_z (m⁴)	56.37	40.73	56.99	57.72





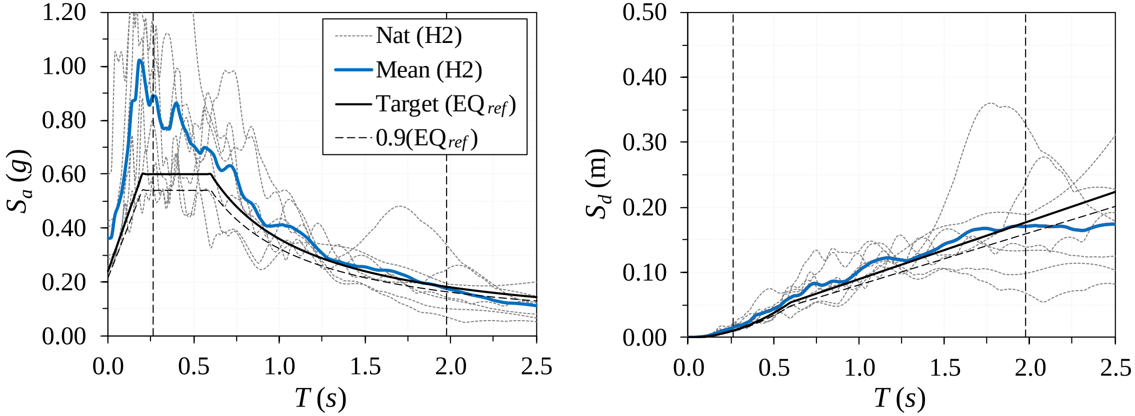




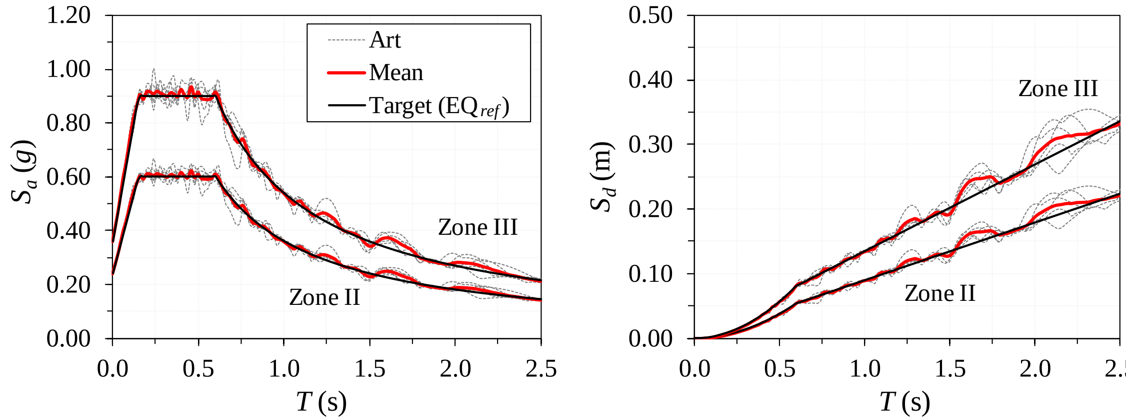
'Standard' point-hinge approach: modified Takeda degrading-stiffness hysteresis rules

Implementation: Selection of input motions (ISSARS)

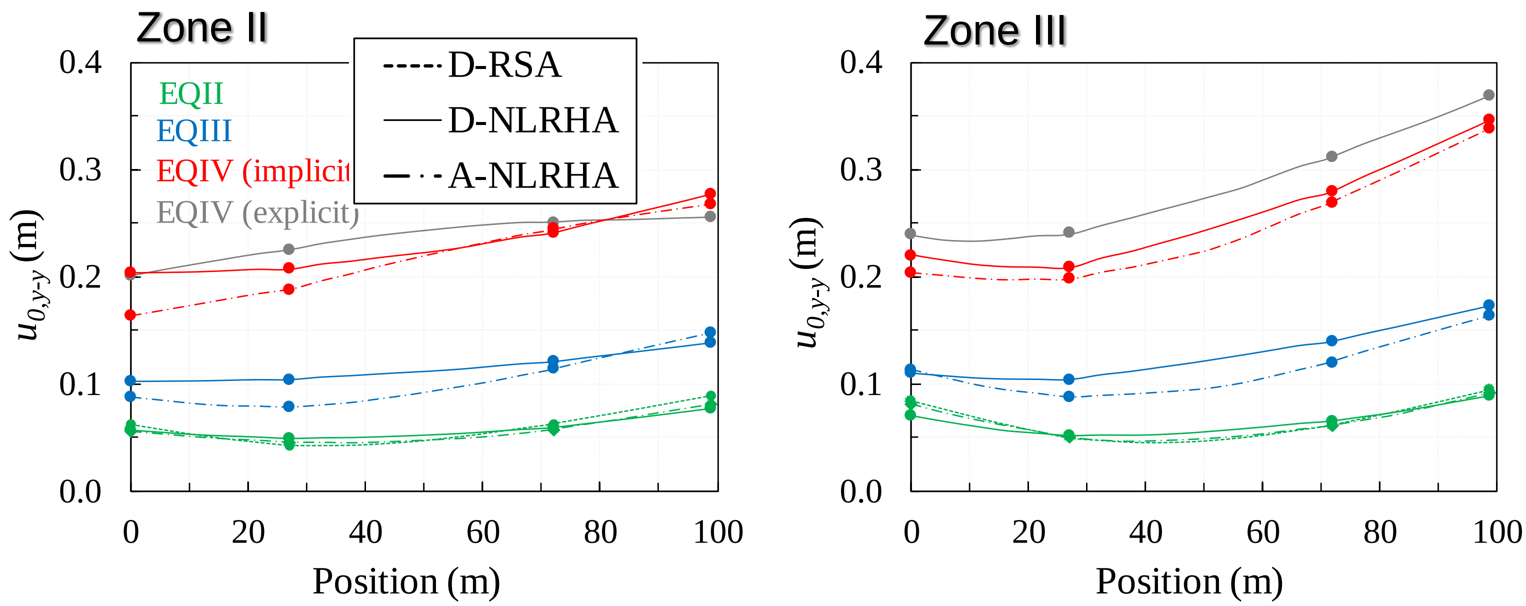
Zone	Suite of records							Scaling factor (SF)	Spectral deviation δ	P_1 SEE (%)	P_2 SEE (%)
II	1	3	5	6	12	13	16	1.18	0.1651	13.17	13.51
III	1	5	6	8	10	13	16	1.81	0.1956	12.33	14.74



Verification



- Are design quantities close to those assumed at the design stage ?
- Is design safe ? (i.e. refined SP ductility factors not exceeded)



➢ *Application of Def-BD:*

- SP2 'operationality' PL: governed pier design, target deformation actually reached

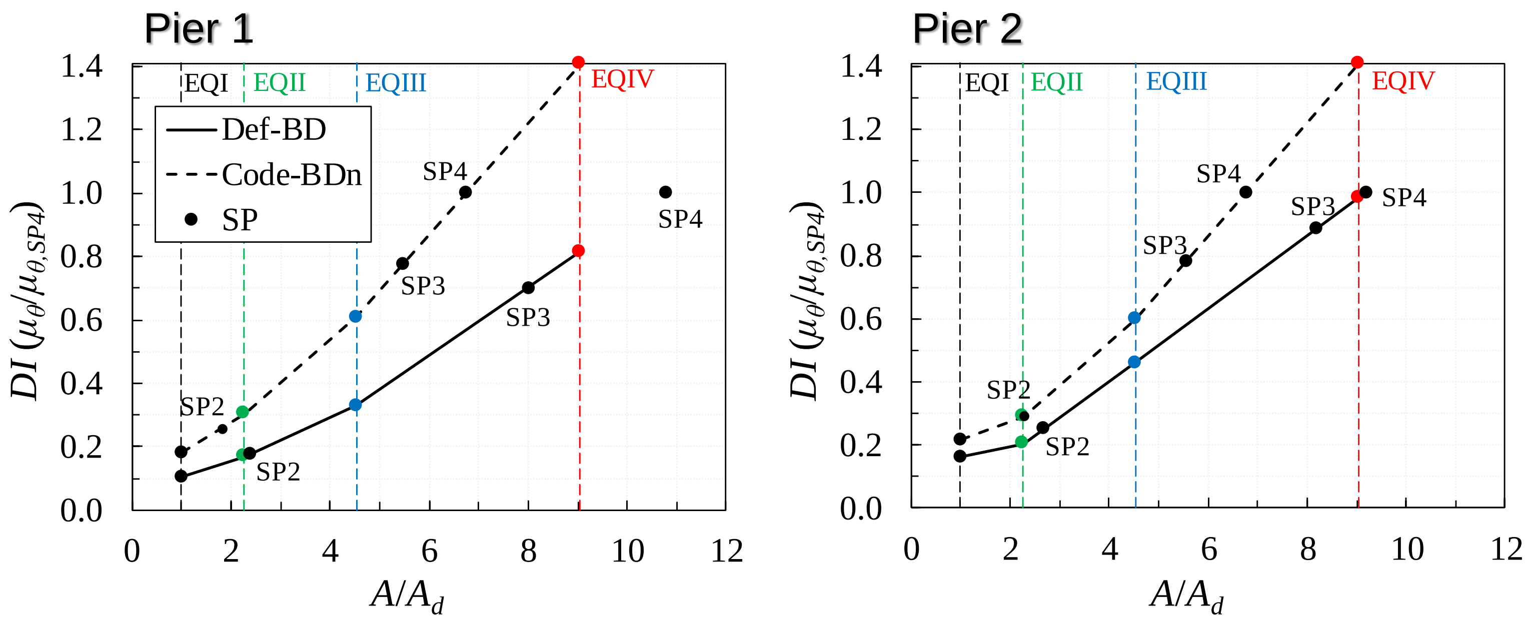
  SP3 PL: not critical (demand similar to $\rho_{w,min}$ requirements)

  SP4 PL: critical (with respect to stability) for bearings deformations
- Very good prediction of structural response while resulting in safe design

➢ *Comparison with MDDBD:*

- Zone II: long. steel: 41%, transv. steel: 17%, concrete (piers): 36%

  Zone III: long. steel: 50%, transv. steel: 20%, concrete (piers): 28%
- Increased computational time and effort due to NLRHA (vs. iterations in MDDBD)



➢ *Comparison with Code-BD*

- Code-BD assessed in terms of a normalised (n) intensity measure, i.e. A/A_d

  Conservatism (overdesign of members, $q<q_{code}$) in small bridges

- PGA=0.05g (Def-BD, Zone II)

  PGA=0.12g (Code-BD, Zone I)

  } A_d (first yielding)

- Pier deformation limits are satisfied only in the case of: Pier 1 (EQII), Piers 1,2 (EQIII)
- Bearing deformation limits are in general violated for all 3 normalised PLs
- Def-BD: enhanced and controlled structural performance under multiple PLs