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SEISMIC ANALYSIS AND DESIGN OF BRIDGES ACCORDING TO EC8-2: COMPARISON OF DIFFERENT ANALYSIS METHODS ON A THEORETICAL CASE-STUDY

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ABSTRACT

This paper aims to compare the different structural analysis methods described in Eurocode 8-2 and specialised literature dedicated to bridge seismic design, such as force-based and displacement-based modal spectral analysis, push-over analysis (different alternative approaches) and non-linear dynamic time-history analysis. For this purpose, those different methods are applied on a theoretical case-study consisting in a 306 meters long prestressed concrete deck, reinforced concrete piers bridge, located in in high seismic zone of the French seismic zoning and designed for ductile behavior alternatively according to Eurocode 8-2 and the former French seismic rules "AFPS92".

Besides the theoretical and practical comparison of the different methods of analysis and associated results, the study also highlights the main differences and changes between Eurocode 8-2 and former French seismic rules "AFPS92". In the end, the paper addresses some upgrading propositions to Eurocode 8-2 text and content.

INTRODUCTION

In most of European countries, the "new" European Standards for structural design (Eurocodes) has deeply modified engineers practices. In France, since January 2012, with the entry into effect of the new national seismic legislation (MEDDTL, 2010-2011), the owners of transportation infrastructures are enforced to apply the new national and European seismic standards for bridges: Eurocode 8-2 and its French National Annex (CEN/TC250, 2005) for the design of any new bridge structure and adjacent retaining wall located in newly defined seismic regions.

In comparison with the former rules established in the AFPS 92 Guide for Earthquake-Resistant Protection of Bridges (AFPS, 1995), this new regulation framework enables to take advantage of latest scientific and technological advances in seismic design and analysis of structures, such as probabilistic seismic hazard evaluation, non-linear structural analysis and anti-seismic devices use.

However, it also raises many theoretical and practical questions: What are the main differences between the different EC8-2 more or less sophisticated analysis methods? Do they present a satisfying level of convergence? What is their applicability domain and level of reliability? What level of expertise do they require to be well understood and apply? What main changes and consequences in terms of seismic performance and construction costs of structures do they lead to, in comparison with former French seismic rules AFPS92?

In order to answer those questions, the different analysis methods where applied and compared on a theoretical case-study bridge, alternatively designed according to Eurocode 8-2 and the former French seismic rules "AFPS92", for different seismic contexts (moderate and high seismic zones) and

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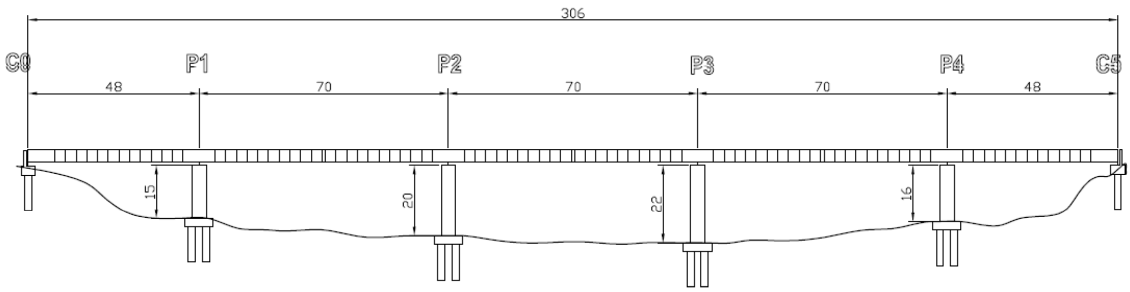
choices of conception (ductile, limited-ductility or using anti-seismic devices). However, only the case of the bridge designed using ductility concept and located high seismic zone is presented in this paper, since it has shown to be the most critical and interesting for addressed issue in terms of illustration and testing of the different theoretical phenomena covered by the different analysis methods, such as plastic hinge formation, ductility demand, hysteretic damping…

The study enables to highlight main theoretical and practical differences between the various tested analysis methods in terms of analysis assumptions and computing tools, scientific reliabilty, entrance needed data and level of expertise required. It also allows to address some possible upgrading propositions to Eurocode 8-2 text and content. Even though it should be noted that the study focuses only on structural aspects with no consideration for soil-structure interaction aspects (rigid foundations assumption), the seismic performance of the bridge predicted from the different methods can be compared with the design expected behavior, and the impact of the French seismic standards evolution on this type of bridges structures can be evaluated in terms of seismic performance, level of conservatism and associated costs.

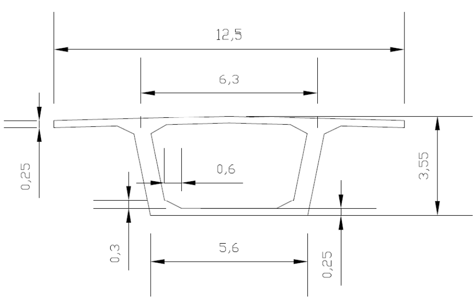
CASE-STUDY BRIDGE CONFIGURATION – DESIGN CONCEPT AND MODELLING

The theoretical case-study considered for this paper consists in a five spans, 306 meters long bridge (48m + 3x70m + 48m) with post-tension prestressed concrete caisson continuous deck and reinforced concrete rectangular hollow piers (respective heights: 15m, 20m, 22m and 16m). This case-study was inspired from a real existing bridge, which has been transformed to fit geometrical regularity EC8-2 conditions in terms of span distribution, piers height and sections, as well as skew and curvature (straight bridge, no skew), as illustrated on Fig.1.

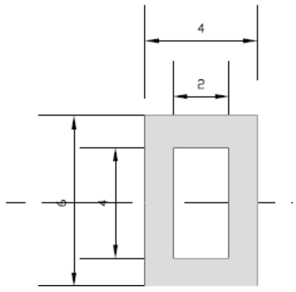


Bridge elevation

Quasi-similar existing bridge under construction



Deck caisson cross section



Piers cross section

Figure 1. Case-study geometrical description

The total weight of the structure is evaluated to 112.2MN. The deck, including non-structural elements and equipments, accounts for 83MN.

Piers and abutments foundations consist in 2m x 0.8m rectangular section piles. They are founded on deposits of very dense gravel, at several tens of meters in thickness, characterised by a gradual increase of mechanical properties with depth identified as groung type B according to Eurocode 8-1 (CEN/TC250, 2004). Because of the rigidity of the foundation system, soil-structure interaction is not considered in the study and foundations are assumed to be perfectly rigid.

The bridge is simply supported on the abutments and piers P1 and P4 through a pair of pot-bearings allowing free sliding and rotation in longitudinal axis, whereas central piers P2 and P3 are rigidly connected to the deck. Analysis have been performed with the same simplified frame elements model described by Fig.2 for both modal spectral analysis, pushover analysis and dynamic time history analysis, using SAP2000$^{@}$ software (CSI, 1998).

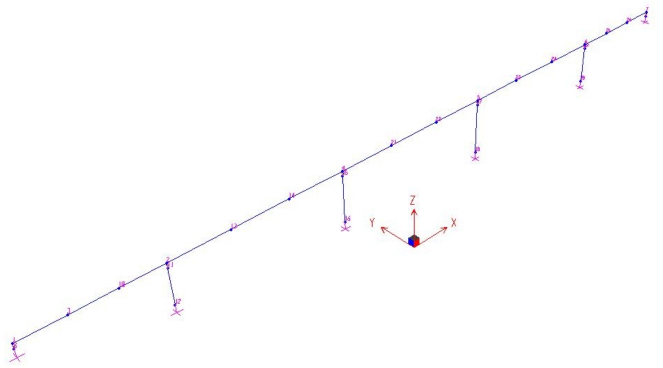


Figure 2. Simplified frame elements SAP2000$^{@}$ model

SHORT DESCRIPTION OF TESTED ANALYSIS METHODS

Analysis methods for bridge seismic design and assessment that are tested in this paper are mainly extracted from Eurocode 8-2 (CEN/TC250, 2005) (Kolias et al., 2012). However, in order to get a wider overview of the addressed issue, alternative approaches from specialised literature dedicated to seismic bridge analysis (Priestley et al., 1996, 2008) are also presented and used.

Force-based modal spectral analysis associated with behavior factor q

This first set of analysis methods is nowadays probably still the most commonly used approach for seismic design of structures in most regions of the word subjected to earthquake hazard.

The first step consists in proceeding to elastic modal analysis in order to obtain eigenvalues (natural periods of vibration) and eigenvectors (natural mode shapes) according to well known structural dynamic theories (Clough and Penzien, 1993). Elastic forces are then evaluated from elastic estimates of structure natural periods together with code design acceleration spectrum for five percent damping. Most significant modes responses are finaly combined together using a quadratic combination in order to get the global dynamic elastic response of the structure. In this approach, the effective cracked stiffness of the piers is evaluated from design ultimate moment M_{Rd} using EC8-2 informative annex C method 2, whereas following EC8-2 requirements, the uncracked bending stiffness and 50% of the uncracked torsional stiffness are considered for the prestressed concrete deck.

When the bridge geometry fits some regularity consideration in terms of piers height, mass distribution, limited skew and curvature, simplified method based on fundamental mode only can be alternatively used. Depending on the particular characteristics of the bridge, this method may be applied using different approaches for the model, namely Rigid Deck Model, Flexible Deck Model (Rayleigh Method) or Individual Pier Model as described in Eurocode 8-2.

In order to account for favorable plastic energy dissipation and hysteretic damping, force demands in the structure are uniformly reduced from the elastic level by dividing by the code-specified force-reduction factor, usually called behavior factor q, the value of which depends on the the assumed ductility capacity of the structure.

When derived from the pre-divided by q code design acceleration spectrum, displacement levels need to be re-multiplied by the displacement ductility factor μ_d, the value of which depends of the fundamental period range (equal-displacement, equal-energy or equal-force) of the structure in the considered horizontal direction, according to EC8-2 requirements and Newmark general principles (Newmark and Veletsos, 1960). In most cases of typical bridges, equal-displacement rule can be applied and μ_d=q.

Displacement-based modal spectral analysis

Inspired from Newmark's equal-displacement rule presented above, many research efforts have been made in recent years into the development of direct displacement based seismic analysis methods. Those methods are based on the observation that displacements (and related material strains) are better indicators of damage potential than are forces.

Starting from the same general modal spectral analysis has described above, the displacement-based modal spectral analysis used in this study uses displacements derived from elastic response spectrum as the starting point demand parameter. Force demands are then derived from those displacements by adjusting their values on the effective performance curves of the resisting piers. This alternative spectral analysis thus requires a preliminary step that consists in deriving the performance Force-displacement curves of the piers from materials stress-strain relationships (including concrete transverse confinement effect) and sections bi-linearized moment-curvature curves, as illustrated on Fig.3.

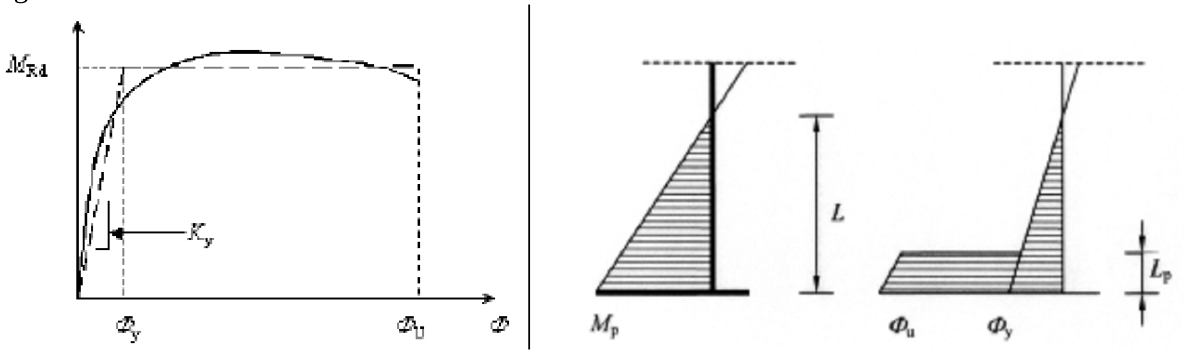


Figure 3. Moment-curvature analysis and distribution over the piers height, from EC8-2 (CEN/TC250, 2005)

For this analysis that results in the evaluation of piers yield, plastic and ultimate displacements (respectively noted d_y, $d_{p,u}$ and d_u), simplified empirical relations and equations extracted from EC8-2 informative annex E were used (Eq.1 and 2).

$$d_u = d_y + d_{p,u} = \frac{\phi_y L^2}{3} + (\phi_u - \phi_y) L_p (L - \frac{L_p}{2}) \qquad (1)$$

where:

- ϕ_y and ϕ_u are respectively the yield and ultimate curvatures resulting from moment-curvature analysis
- L is the distance from the end section of the plastic hinge to the point of zero moment in the pier
- L_p is the evaluated plastic hinge length where pier deformation concentrates:

$$L_p = 0.10L + 0.015 f_{yk} d_{bL} \qquad (2)$$

with longitudinal reinforcement of characteristic yield stress f_{yk} (in MPa) and bar diameter d_{bL}.

Pushover analysis (EC8-2 approach)

Pushover analysis consists in a static non-linear analysis of the structure under monotonically increased horizontal loads, representing the effect of a horizontal seismic component. The main objectives of the analysis are the estimation of the sequence and the final pattern of plastic hinge formation, the estimation of the redistribution of internal forces following the formation of plastic hinges, and the assessment of the force-displacement curve of the structure ("capacity curve") and of the deformation demands of the plastic hinges up to the ultimate constitutive materials strain limits.

In the basic approach described in EC8-2 informative annex H, horizontal forces are distributes according to the initial elastic fundamental mode shape, and the displacement demand evaluation of the reference point (chosen at the centre of mass of the deck) is based on the code elastic response spectrum for five percent damping.

Main criticisms that can be addressed on this basic pushover analysis approach consist in the facts that it does not take unto account some dynamic or non-linear behavior aspects of prime importance such as higher modes effects, structural softening, modification of the vibration modes and damping increase with post-yield plastic deformations and damage.

Alternative pushover analysis (performance point approach)

As recognized to be a very powerful tool for seismic performance evaluation of structures, the static non-linear pushover analysis has become a new trend due to its simplicity compared with the conventional dynamic time-history analysis procedure (see below). In recent years, considerable research effort has therefore been put to develop some extensions and improvements of pushover analysis methods (Faella et al., 2004). Most of them are based on the performance point concept which consists in intersecting the performance curve by the demand acceleration-displacement (or force-displacement) spectrum, by considering the equivalent secant effective stiffness instead of the initial, as represented on Fig.4.

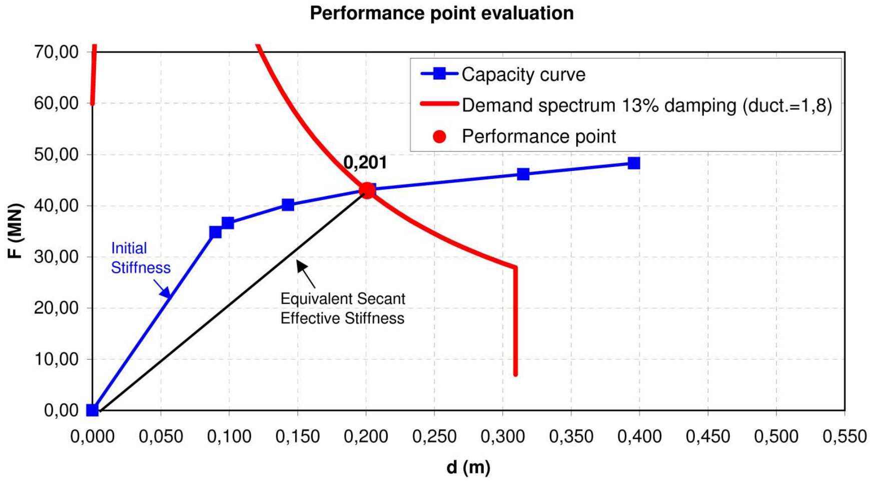


Figure 4. Equivalent secant effective stiffness and performance point definitions

In the present study, main differences between used alternative pushover analysis compared with basic EC8-2 appraoch pushover analysis are the followings:

- Equivalent multimodal shape based on spectral deformation response in order to account for higher modes contribution;
- Re-evaluation of equivalent mode shape at each load increment;
- Performance point approach accounting for structural softening with post-yield plastic response (euivalent effective secant stiffness);
- Equivalent displacement derived for general dynamic analysis theory ($d_{eq}=\Sigma(m_i d_i^2)/\Sigma(m_i d_i)$) instead of centre of mass of the deck reference point displacement;
- Equivalent damping ξ_{eq} evaluated from Takeda model as described by Otani (1981) and Kowalsky and Ayers (2002) and expressed by Eq.3 from reached ductility demand μ_d.

$$\xi_{eq} = 0.05 + \frac{1}{\pi}\left(1 - \frac{1-0.03}{\sqrt{\mu_d}} - 0.03\sqrt{\mu_d}\right) \geq 0.05 \qquad (3)$$

Non-linear dynamic time-history analysis

Dynamic response of structures can also be obtained through direct numerical integration of non-linear differential equations of motion using specialized structural analysis programs. The seismic input then consists of ground motion time-histories (accelerograms). It has to be noted that for new bridges design, Eurocode 8-2 requires that at least three pairs of accelerograms shall be used, selected from recorded events with magnitudes, source distances, and mechanisms consistent with those that define the design seismic action at the location of the bridge.

In the present study, non-linear time-history analysis is performed using SAP2000@ program that enables to obtain directly and easily the time dependent response of the structure elements such as plastic hinges and damping devices (Fig.5). In order to reduce calculation time, only those specific regions were modelled as non-linear elements. The rest of the structure, that was assumed to remain elastic, was considered with uncracked stiffness except for the torsionnal stiffness of the deck, in accordance with EC8-2 recommandations.

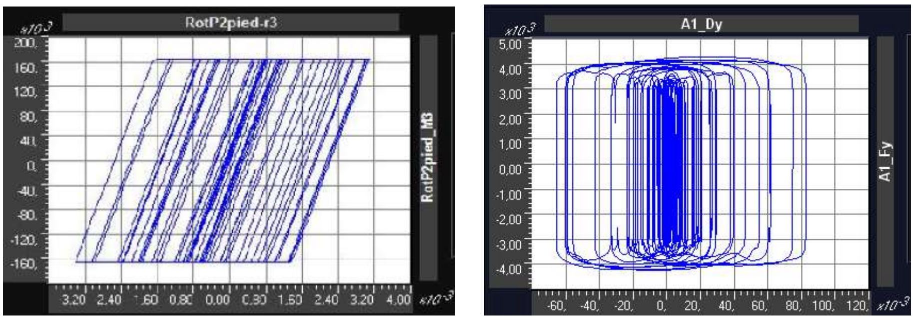


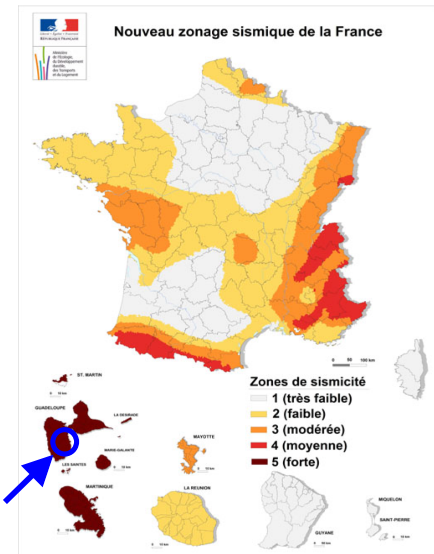
Figure 5. Time dependent response of plastic hinges (left) and damping devices (right) from SAP2000@

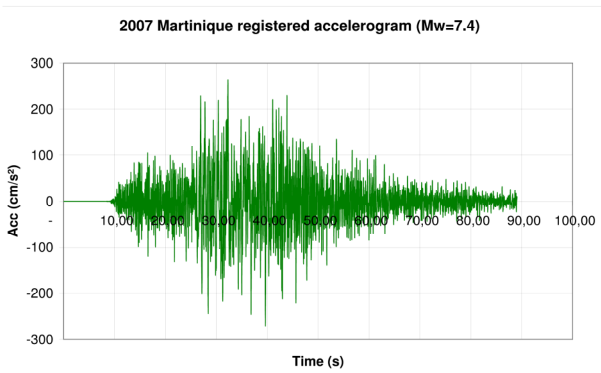
MAIN RESULTS, LEARNINGS AND DISCUSSION

Design seismic action

The bridge is supposed to be located on Guadeloupe Caribbean Island (Fig.6) at seismic zone Z5 according to the French seismic zoning (MEDDTL, 2010), with an associated reference peak ground acceleration $a_{gR} = 3.0 m/s^2$. The design seismic action is calculated by a response spectrum of type 1. The ground type is B, so the characteristic periods are $T_B = 0.15s$, $T_C = 0.50s$ and $T_D = 2.00s$, while the soil factor is $S = 1.20$ and the topographic amplification factor is $\tau = 1.3$. The importance factor is $\gamma_I = 1.4$ (importance class IV associated to bridges of critical importance), leading to a seismic action in horizontal directions of $a_g = \gamma_I . a_{gR} = 1.4 \times 3.0 m/s^2 = 4.2 m/s^2$ (for comparison, former French seismic rules “AFPS92” would have lead to $4.5 m/s^2$).

Ground motion time-history used for non-linear dynamic time-history analysis was extracted from nearby 2007 Martinique Island Earthquake ($M_w = 7.4$) registered accelerogram, which was artificially adjusted to design seismic action a_g, as illustrated by Fig.6.





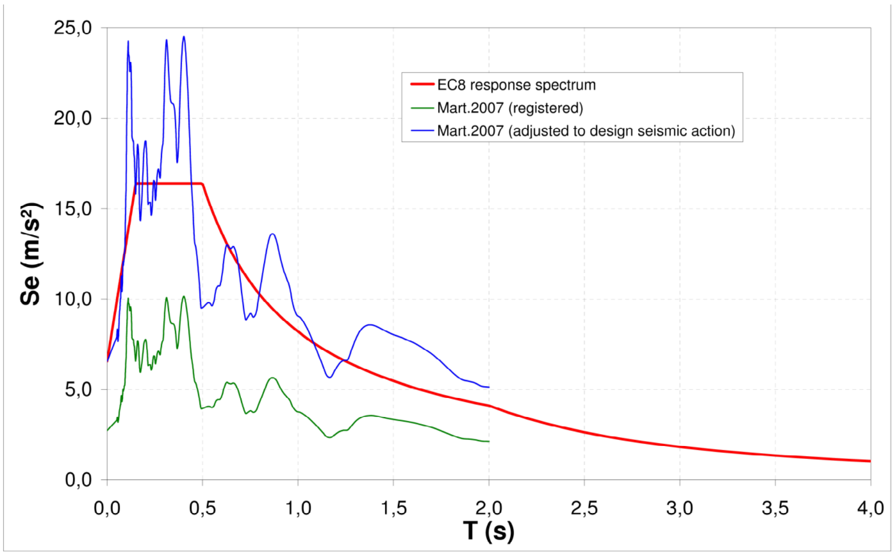


Figure 6. Bridge location on the French zoning and corresponding design seismic action representations

Seismic structural system, ductility class, and reinforcement design of concrete piers

Lateral stoppers (shear-keys) are disposed on each pier and abutment. Therefore, the main elements resisting seismic forces are the central piers P2 and P3 in the longitudinal direction, which are assumed to be fully fixed to the foundations and to the deck, and all piers and abutments in the transversal direction, which are assumed to be fully fixed to the foundations and pinned-connected to the deck (Fig.7). A ductile seismic behaviour is selected for these elements. The value of the behavior factor resulting from the design process is q = 2.7, for both longitudinal and transversal direction.

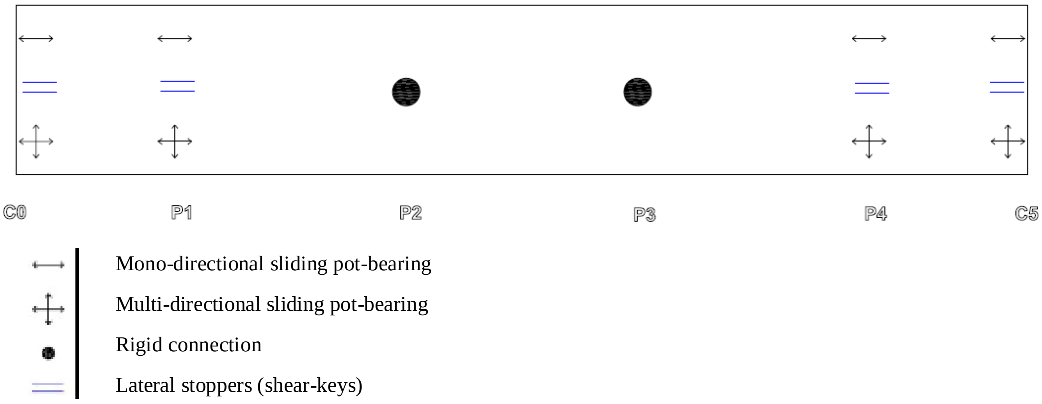


Figure 7. Seismic structural system

Reinforcement design of the concrete piers, based on effective cracked stiffnesses, concluded to ϕ32mm longitudinal rebars with 0.20m spacing on both external and internal faces of the hollow rectangular section, leading to resisting moment $M_{Rd,long}$ = 160MNm in the longitudinal axis and $M_{Rd,trans}$ = 230MNm in the transversal axis. Transerval reinforcement is imposed by deatiling EC8-2 requirements. It consists in 4 ϕ25mm (resp. ϕ20mm) hoops, 0.16m vertcally spaced in the longitudinal (resp. transversal) direction, for resisting shear forces evaluated to $V_{Rd,long}$ = 47.9MN and $V_{Rd,ltrans}$ = 46.3MN.

For comparison, reinforcement design based on former French AFPS92 seimic rules concluded to dispose 2ϕ40mm longitudinal rebars with 0.20m spacing on both external and internal faces ($M_{Rd,long}$ = 360MNm ; $M_{Rd,trans}$ = 510MNm) and transerval reinforcement consisting in 4 ϕ32mm hoops in both longitudinal and transversal axis, 0.15m vertcally spaced ($V_{Rd,long}$ = 83.6MN ; $V_{Rd,ltrans}$ = 126.5MN). This significant increase essentially comes from the fact that uncracked stiffnesses derived from gross-section inertia was used instead of effective cracked stiffnesses.

Results obtained from the different analysis methods and comparison

Results obtained from the different analysis methods converge to a global ductility demand μ_d of about 2.3 in the longitudinal direction and 1.6 in the transversal direction. More detailed demand parameters such as seismic induced shear forces and bending moments, displacement demand, local ductility demand in piers, number of formed plastic hinges or force-reduction factors are given in Table 1 for the longitudinal direction and compared with associated structural capacities (at Significant Damage and Near-Collapse Ultimate Limit-States).

Table 1. Results obtained from the different analysis methods in the longitudinal direction

Longitudinal direction	**Pre-design**	**Meth.1:** Force-based modal spectral analysis	**Meth.2:** Displ.-based modal spectral analysis	**Meth.3:** Pushover analysis (EC8-2)	**Meth.4:** Pushover analysis (perf. point)	**Meth.5:** Non-linear dynamic time-history analysis
Considered behavior factor: **q**	**2.7** *(P2, P3)*	**2.44** *(P2, P3)*	X			
Maxi authorized q factor from EC8-2 for the piers type: $q_{max,i}$	***(3.20)*** *(P2)*					
Global ductility demand: μ_d	**2.7**	**2.44**	X	**2.24**	**2.38**	**1.45**
Global duct. capacity at significant damage LS: $\mu_{Rd,SD\text{-}LS}$	X			***(2.62)***		
Global duct. capacity at near-collapse LS: $\mu_{Rd,NC\text{-}LS}$	X			***(3.23)***		
Total force demand: **F** (MN)	**24.71**	**27.34**	**29.33**	**29.27**	**29.30**	**29.33**
Deck displacement demand: **d** (m)	**0.235**				**0.250**	**0.152**
Seismic displ. capacity of expansion joints: $d_{lim,i}$ (m)	***(0.235)***					
Global displ. capacity at yield: d_y (m)	X			***(0.105)***		
Global displ. capacity at significant damage LS: $d_{SD\text{-}LS}$ (m)	X			***(0.275)***		
Global displ. capacity at near-collapse LS: $d_{SD\text{-}NC}$ (m)	X			***(0.339)***		
Number of formed plastic hinges	**4** *($P2^B$, $P3^B$, $P2^T$, $P3^T$)*					
Local maxi ductility demands: $\mu_{d,i}$	**2.70** *(P2)*		**2.42** *(P2)*	**2.42** *(P2)*	**2.58** *(P2)*	**1.57** *(P2)*
Local duct. capacities at significant damage LS: $\mu_{Rd,SD\text{-}LS,i}$	X		***(2.67)*** *(P2)*	***(2.84)*** *(P2)*		***(2.67)*** *(P2)*
Local duct. capacities at near-collapse LS: $\mu_{Rd,NC\text{-}LS,i}$	X		***(3.29)*** *(P2)*	***(3.49)*** *(P2)*		***(3.29)*** *(P2)*
Induced shear forces: $V_{Ed,i}$ (MN)	**13.71** *(P2)*	**15.17** *(P2)*	**15.33** *(P2)*	**14.88** *(P2)*	**14.85** *(P2)*	**15.33** *(P2)*
Piers shear resistances: $V_{Rd,i}$ (MN)	***(47.9)***					
Induced bending moments: $M_{Ed,i}$ (MNm)	**154.3** *($P2^B$)*	**170.7** *($P2^B$)*	**161.5** *($P3^B$)*	**165.8** *($P3^B$)*	**166.5** *($P3^B$)*	**161.5** *($P3^B$)*
Piers resisting moments: $M_{Rd,i}$ (MNm)	***(160.0)*** *($P2^B$)*		***(161.5)*** *($P3^B$)*	***(165.8)*** *($P3^B$)*	***(166.5)*** *($P3^B$)*	***(161.5)*** *($P3^B$)*
Reduction-force factor: $r_i = q.M_{Ed,i}/M_{Rd,i}$	**2.60** *($P2^B$)*		X			

Legend:				
	Behavior	**Elastic ($\mu \leq 1$)**	**Limited ductility ($\mu \leq 1.5$)**	**Ductile ($\mu > 1.5$)**
	Demands	**≤ Standards capacities**		**> Standards capacities**

Those results can also be presented in a more synthetic way using performance point representation that consists in intersecting the performance curve by the demand force-displacement) spectrum, as illustrated on Fig.8 for the transversal direction.

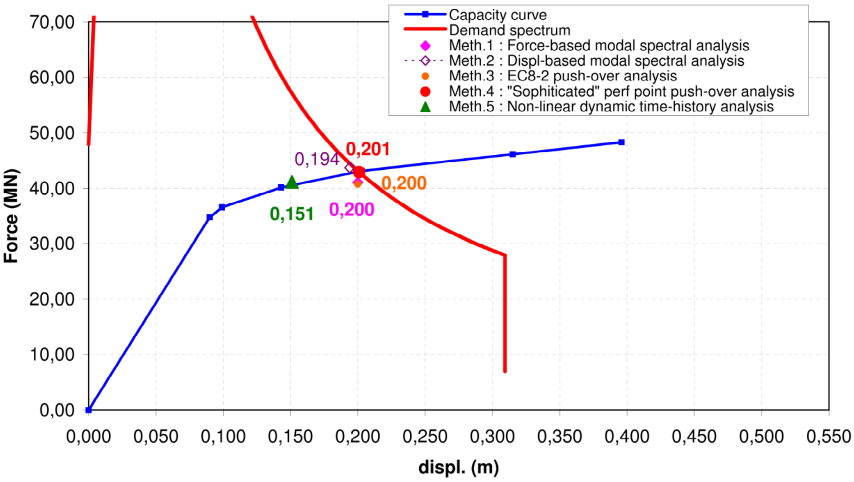


Figure 8. Comparison of the different analysis methods (transversal direction)

It appears from those differents results that a fairly good convergence is obtained from the different approaches. Non-linear dynamic time-history analysis presents the largest shift compared to other methods, due to both influences of representativity of the chosen accelerogram and different assumptions in the cracked effective length.

The relative influence of each individual assumption or empirical simplified equation have been quantified in Table 2 with indication of methods under concern. Those different parameters effects range from less than 4% up to ±30%. Most influent parameters correspond to yield curvature ϕ_y estimation and effective cracked stiffness value and extent (length of taking into account). Hovewer, favorable compensations can be observed when combining those different approximations together, leading to a quite satisfying level of precision on global results (within a range of about ±25% precision on global seism demands, ±10% if not considering non-linear dynamic time-history analysis). Despite of the favourable geometrical regularity and simplicity of the structure, this results are believed to be within an acceptable accuracy range with regards to the complexity of the theoretical phenomena that are taken into account in the analysis and the different simplified empirical assumption and equation that are used. In particular, it has been shown that influent parameters such as plastic hinges length and hysteretic damping (Blandon, 2004) can vary significantly from one case to another and are quite complex to determine analytically, as illustrated on Fig.9.

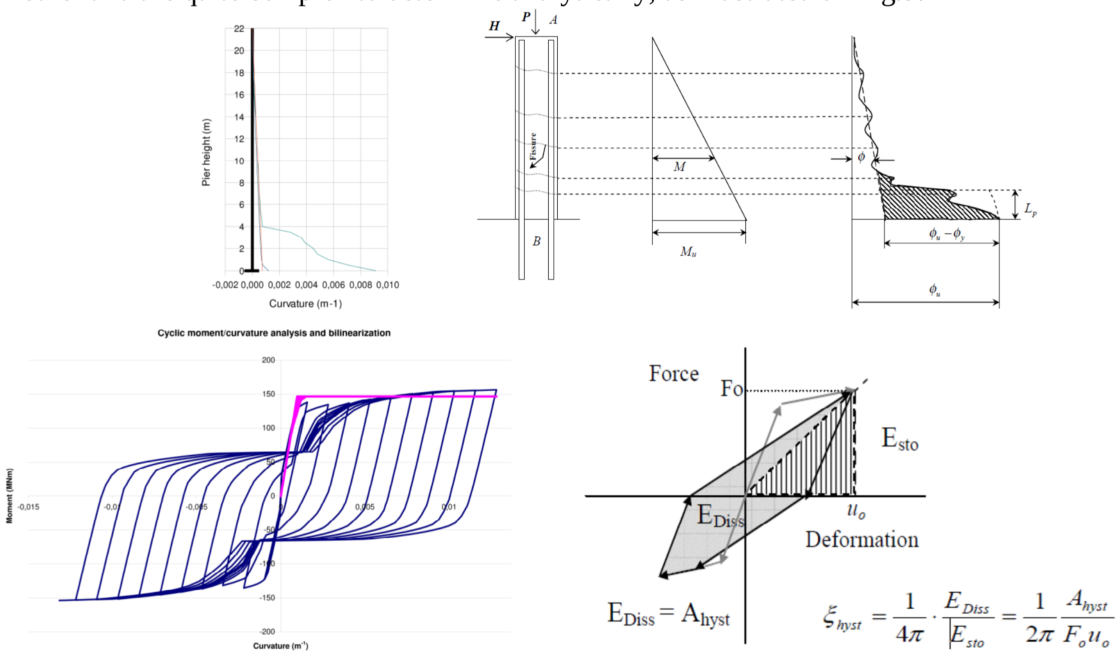


Figure 9. Analytical tentatives for plastic hinge length and equivalent hysteretic damping determinations

Table 2. Relative influence of individual assumptions and empirical simplified equations on results obtained from the different methods

Assumptions and empirical simplified equations	**Relative approximation range**	**Meth.1:** Force-based modal spectral analysis	**Meth.2:** Displ.-based modal spectral analysis	**Meth.3:** Pushover analysis (EC8-2)	**Meth.4:** Pushover analysis (perf. point)	**Meth.5:** Non-linear dynamic time-history analysis
1. Empirical evaluation of yied curvature: $\phi_y = 2.1\varepsilon_y/d$	-27% to +23% (on ductility demand)	**X**				
2. Value and extent (full height or localised in plastic hinge) of effective cracked stiffness: $J_{eff} = 1.2\ M_{Rd}/E_c$	-30% to +12% (displ. and force demands)	**X**				**X**
1 + 2	-27% to +36% (displ. and ductility demands)	**X**				
3. Uniform division of force demands by behavior factor q	-21% to +16% (local force demands)	**X**				
4. Reference point chosen at the centre of mass of the deck	< 4% (global transversal displ.)	**X**		**X**		
5. Equal-displacement rule	-30% to -20% (displ. when equal-energy rule should apply)		**X**	**X**		
6. Superior modes effects neglected	< 2% (longitudinal) < 10% (transversal)		**X**			
7. Internal forces redistribution neglected	-8% to –6% (displ. capa) < ±3% (force demand)	**X**	**X**			**X** (N var. effect on M-ϕ curves
8. Modal changes with plasticity neglected	< 2% (longitudinal) < 10% (transversal)	**X**	**X**	**X**		
9. Representativeness of single accelerogram compared to seismic spectrum demand	-20% on displacements for the case-study					**X**

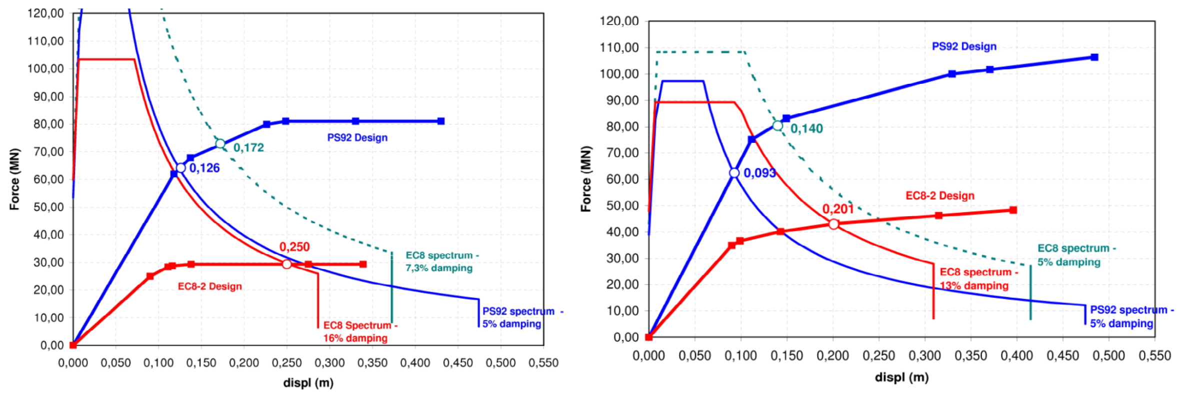
Despite the fact that non-linear dynamic time-history analysis obviously represents the most realistic and sophisticated way to assess the response of structures to earthquake type sollicitations, it appears that obtained results can very dependant on modelling choices and chosen ground motion time history input. Moreover, it can become very time consumming when considering complex complete numerical models and several accelerogramm traces as required by Eurocode 8 Standards. In comparison, pushover analysis methods, and more specifcally the performance point based approach, seem to be the most adapted to clearly represent and explain in suitable and understandable way the seismic behavior of this type of regular structure with deck connected piers, since they enable, using relatively simple numerical tools, to take into account many important aspects such as structural plastic redundancy, internal force redistribution and columns post-elastic effective stiffness and equivalent damping.

Considering these general statements and results, some propositions can be addressed in order to upgrade the text and content of Eurocode 8-2. First, the evaluation of the effective cracked stiffness and the regions of the structure where it should be considered or not (elements cracked length extent) could advantageously be clarified and made more consistant within the differents parts of the code. Then, alternative pushover analysis method based on performance point approach and effective secant equivalent stiffness as illustrated on Fig.4 could be proposed instead of the approach described in informative annex H, which is based on equal-displacement general theory and initial stiffness.

It has also been demonstrated that simplified monomodal methods based on estimated fundamental natural vibration mode, like rigid deck model, individual pier model or Rayleigh method, lead to approximation ranges from 30% up to 90%. This confirms that the field of application of those approaches should be strictly limited to smaller simple regular bridge configurations with piers of limited mass, in accordance with EC8-2 requirements.

Predicted seismic performance of the bridge and comparison with expected behavior

Using performance point pushover analysis representation, it appears that the seismic performance within the longitudinal direction (that can expressed in terms of ductility demand μ_d of about 2.3) is quite consistant with the assumed behavior factor q = 2.7 (Fig.10a). Standard resistance exceedances are very small and they only concern ductile (bending moments) or non-structural (expansion joints) elements, which indicates a good seismic performance of the bridge under design seismic action as well a good consistancy of this prediction with expected design behavior. In the transversal direction, as illustrated on Fig.10b, the ductility demand μ_d is about 1.6, which corresponds to a confortable conservative margin, due to the fact that seismic design of piers reinforcement was governed by longitudinal direction. When considering former French AFPS92 seismic rules bridge design and seismic demand, it clearly raises that those previous recommandations were excessively conservative for seismic design of bridge structures (by a factor of about two). This high conservatism was mainly due to unappropriate assumptions on effective sections stiffness for ductile design, that was evaluated using gross-section inertia considerations instead of effective cracked stiffness. A confortable level of conservatism remains when the bridge is supposed to be submitted to EC8-2 seismic action, despite the fact that this action leads to a demand increase of 37 to 50%.



a) Longitudinal direction   b) Transversal direction

Figure 10. Comparison between Eurocode 8-2 and previous French AFPS92 seismic rules using performance point representation

The same observation can be addressed from the case-study of the bridge located in moderate seismic zone (French Pyrenean region) and designed for limited-ductility, because of unappropriate value of the behavior factor (q=1) for limited-ductility in the former French rules. On this case illustrated by Fig.11 (transversal direction), it can be noted that the accurracy range is a little narrower, since the structure does not exhibit any post-yield plastic incursion, because design process was conservatively performed using gross uncracked section inertia assumption according to both codes recommandations.

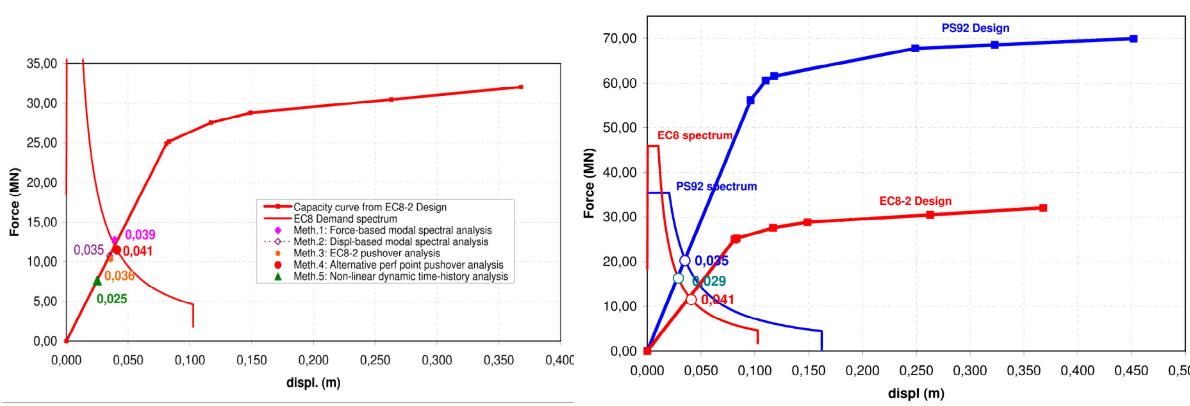


Figure 11. Case-study of the bridge located in in moderate seismic zone (French Pyrenean region) and designed for limited-ductility (transversal direction)

CONCLUSIONS

Different structural analysis methods for bridge seismic design and assessment from Eurocode 8-2 and specialised literature were compared on a theoretical case-study bridge designed for ductile behavior, alternatively according to EC8-2 and the former French seismic rules "AFPS92". A fairly good convergence within a range of about 25% precision between the different approaches has been observed, which is believed to be within an acceptable accuracy range with regards to the complexity of the theoretical phenomena that govern non-linear dynamic response od structures. The study also clearly shoved that former French AFPS92 seismic rules were excessively conservative, due to unappropriate assumptions on effective sections stiffness. Among the different tested methods, performance point pushover analysis approach has appeared to be the most adapted to assess and illustrate the seimic response of the structure. Based on those results, some modification propositions can be addressed to Eurocode 8-2, which consist in claryfying and upgrading the consistancy of the effective cracked stiffness taking into account within the differents parts of the code and in proposing an alternative pushover analysis method to informative annex H. More tests and investigations should however be performed to extrapolate those conclusions to other type of bridges or other consigurations including irregular geometries, other types of bearings, or critical soil-structure interaction.

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