

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 were 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 choices of conception (ductile, limited-ductility or using anti-seismic devices). However, only the case of the bridge designed using ductility concept and

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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 reliability, 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 modeling

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.

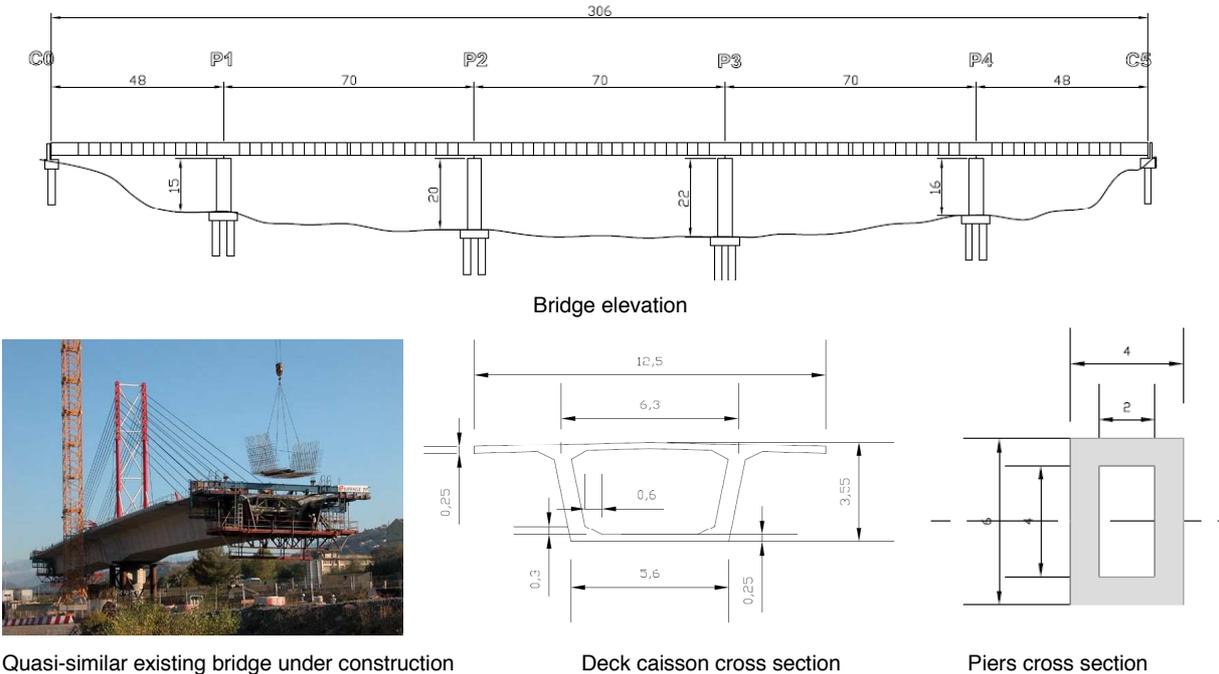


Figure 1. Case-study geometrical description

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 ground 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).

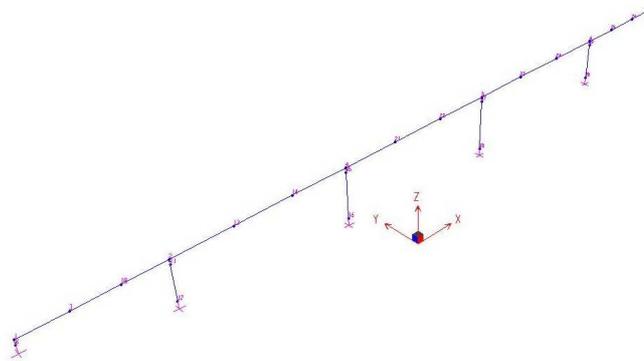


Figure 2. Simplified frame elements SAP2000[®] model

Tested analysis methods

Analysis methods for bridge seismic design and assessment that were tested in this study 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. Tested analysis methods are listed below:

- Force-based modal spectral analysis associated with behavior factor q :

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.

- Displacement-based modal spectral analysis

Starting from the same general modal spectral analysis as from the previous method, this second approach 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 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.

- 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 distributed 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 into 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)

In recent years, considerable research effort has 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.3.

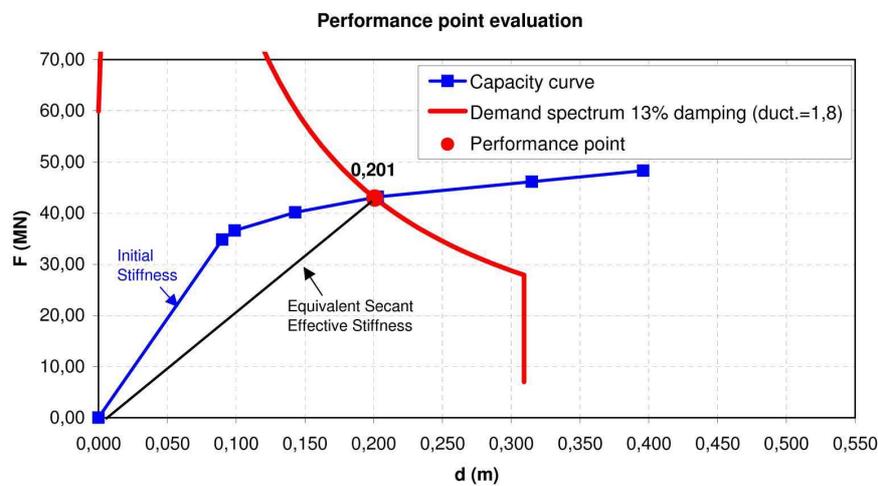


Figure 3. Equivalent secant effective stiffness and performance point definitions

In the present study, main differences between used alternative pushover analysis compared with basic EC8-2 approach pushover analysis consist in accounting for higher modes contribution, re-evaluation of equivalent mode shape at each load increment, accounting for structural softening with post-yield plastic response (equivalent effective secant stiffness), deriving equivalent displacement from general dynamic analysis theory ($d_{eq} = \frac{\sum(m_i d_i^2)}{\sum(m_i d_i)}$) instead of centre of mass of the deck reference point displacement and evaluating equivalent damping ξ_{eq} from Takeda model as described by Otani (1981) and Kowalsky and Ayers (2002) and expressed by Eq.1 from reached ductility demand μ_d .

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

- Non-linear dynamic time-history analysis

In this last method, the dynamic response of structures is obtained through direct numerical integration of non-linear differential equations of motion using specialized structural analysis programs. In the present study, SAP2000® program was used. 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 torsional stiffness of the deck, in accordance with EC8-2 recommendations.

Main results, learning 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 $a_{gR} = 3.0\text{m/s}^2$. The design seismic action is calculated by a response spectrum of type 1. The ground type is B, so the characteristic periods are $T_B = 0.15\text{s}$, $T_C = 0.50\text{s}$ and $T_D = 2.00\text{s}$, 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 $a_g = \gamma_I \cdot a_{gR} = 1.4 \times 3.0\text{m/s}^2 = 4.2\text{m/s}^2$ (for comparison, former French seismic rules “AFPS92” would have lead to 4.5m/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.4.

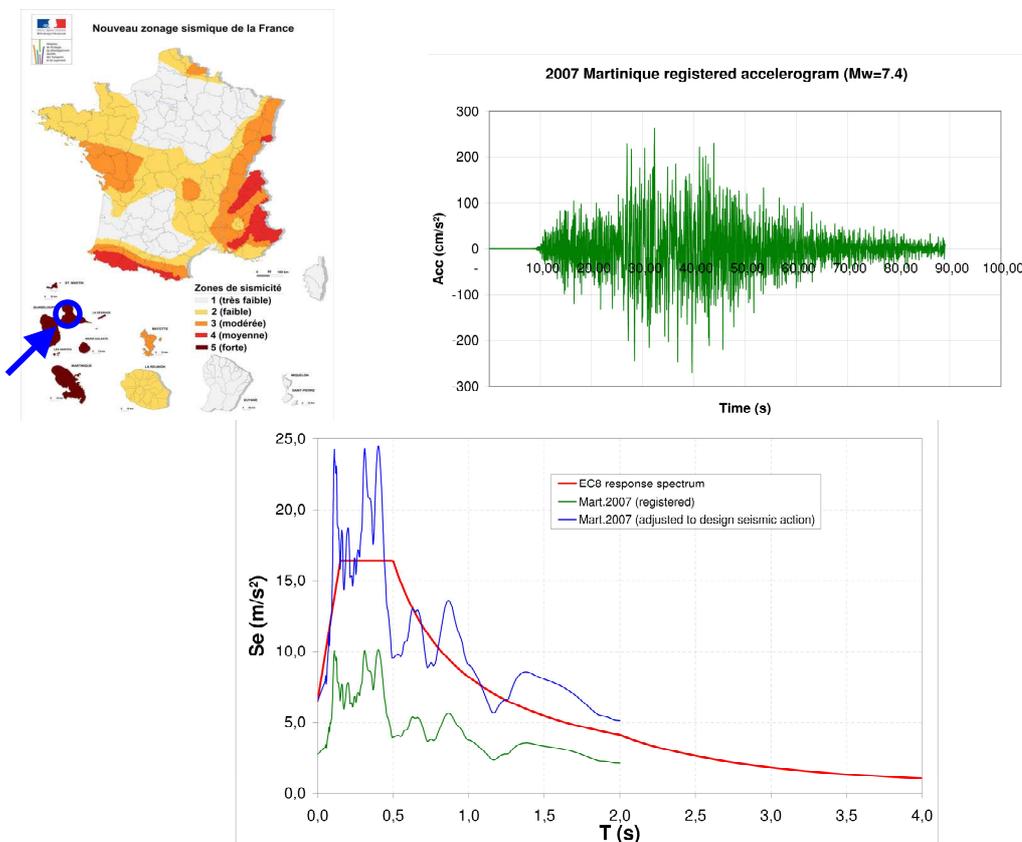


Figure 4. Bridge location on the French zoning and corresponding design seismic action representations

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.5). 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.

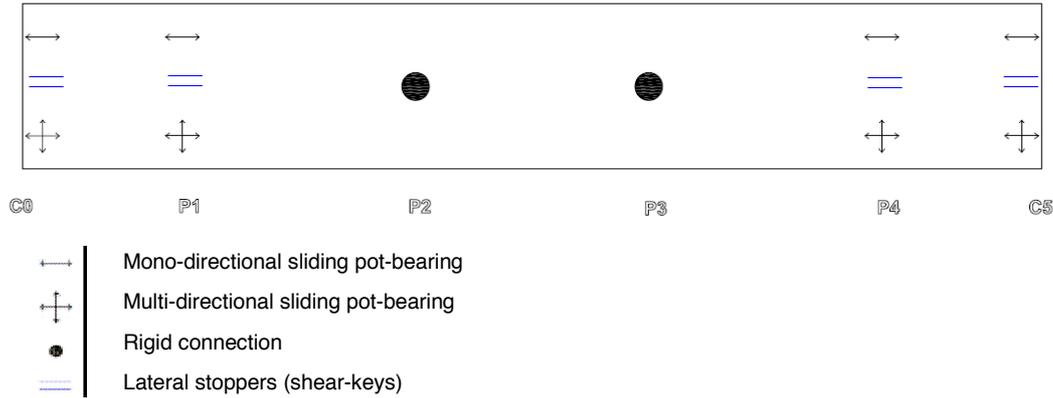


Figure 5. Seismic structural system

Reinforcement design of the concrete piers, based on effective cracked stiffnesses, concluded to $\phi 32\text{mm}$ longitudinal rebars with 0.20m spacing on both external and internal faces of the hollow rectangular section, leading to resisting moment $M_{Rd, \text{long}} = 160\text{MNm}$ in the longitudinal axis and $M_{Rd, \text{trans}} = 230\text{MNm}$ in the transversal axis. Transversal reinforcement is imposed by detailing EC8-2 requirements. It consists in 4 $\phi 25\text{mm}$ (resp. $\phi 20\text{mm}$) hoops, 0.16m vertically spaced in the longitudinal (resp. transversal) direction, for resisting shear forces evaluated to $V_{Rd, \text{long}} = 47.9\text{MN}$ and $V_{Rd, \text{trans}} = 46.3\text{MN}$. For comparison, reinforcement design based on former French AFPS92 seismic rules concluded to dispose $2\phi 40\text{mm}$ longitudinal rebars with 0.20m spacing on both external and internal faces ($M_{Rd, \text{long}} = 360\text{MNm}$; $M_{Rd, \text{trans}} = 510\text{MNm}$) and transversal reinforcement consisting in 4 $\phi 32\text{mm}$ hoops in both longitudinal and transversal axis, 0.15m vertically spaced ($V_{Rd, \text{long}} = 83.6\text{MN}$; $V_{Rd, \text{trans}} = 126.5\text{MN}$). 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. Those results can be presented in a synthetic way using performance point representation that consists in intersecting the performance curve by the demand force-displacement) spectrum, as illustrated on Fig.6 for the transversal direction.

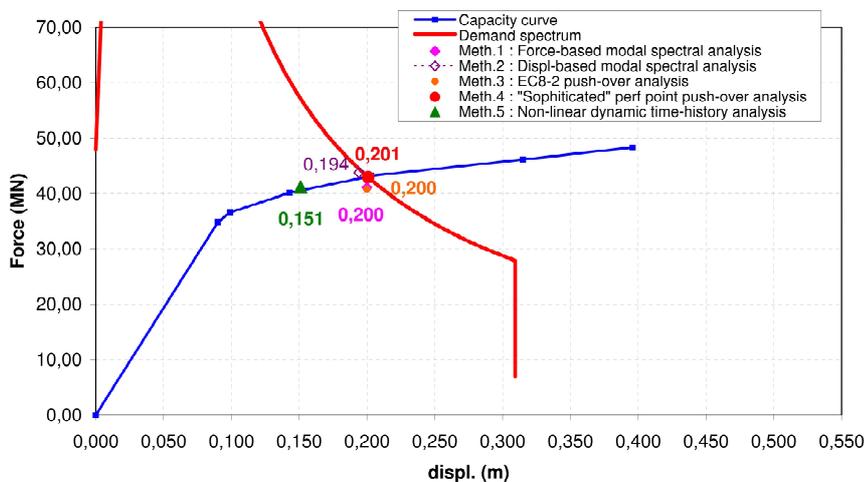


Figure 6. Comparison of the different analysis methods (transversal direction)

It appears from those different 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 1 with indication of methods under concern. Those different parameters effects range from less than 4% up to $\pm 30\%$. Most influent parameters correspond to yield curvature ϕ_y estimation and effective cracked stiffness value and extent (length of taking into account). However, 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 $\pm 25\%$ precision on global seism demands, $\pm 10\%$ 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.

Table 1. 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 yield curvature: $\phi_y = 2.1 \epsilon_v/d$	-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) < $\pm 3\%$ (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 solicitations, it appears that obtained results can be very dependent on modelling choices and chosen ground motion time history input. Moreover, it can become very time consuming when considering complex complete numerical models and several accelerogram traces as required by Eurocode 8 Standards. In comparison, pushover analysis methods, and more specifically the performance point based approach, seem to be the most adapted to clearly represent and explain in a 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 consistent within the different parts of the code. Then, an 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 be expressed in terms of ductility demand μ_d of about 2.3) is quite consistent with the assumed behavior factor $q = 2.7$ (Fig.7a). 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 consistency of this prediction with expected design behavior. In the transversal direction, as illustrated on Fig.7b, the ductility demand μ_d is about 1.6, which corresponds to a comfortable 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 recommendations were excessively conservative for seismic design of bridge structures (by a factor of about two). This high conservatism was mainly due to inappropriate assumptions on effective sections stiffness for ductile design, that was evaluated using gross-section inertia considerations instead of effective cracked stiffness. A comfortable 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%.

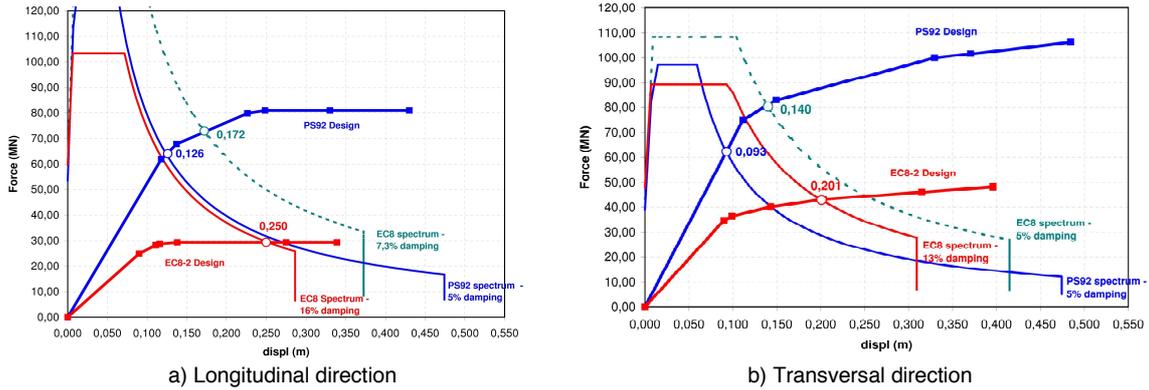


Figure 7. Comparison between Eurocode 8-2 and previous French AFPS92 seismic rules using performance point representation

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.8 (transversal direction), it can be noted that the accuracy 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 recommendations.

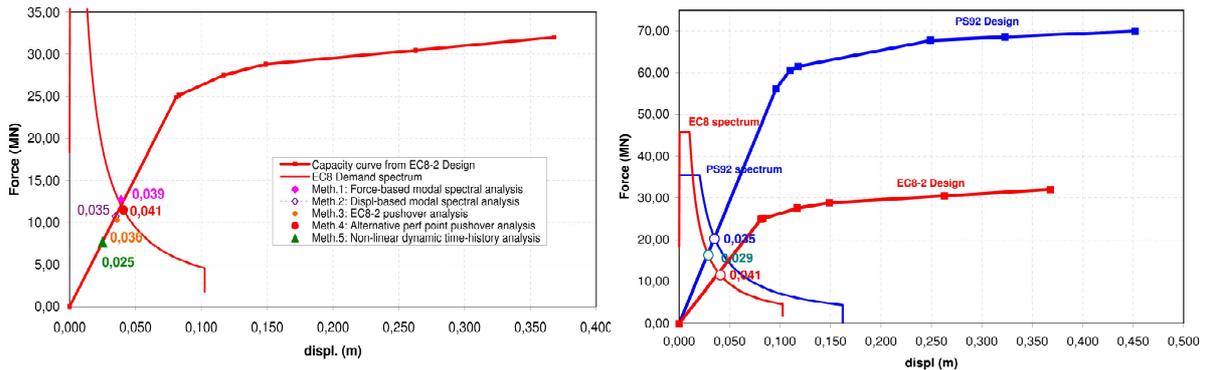


Figure 8. Case-study of the bridge located in moderate seismic zone (French Pyrenean region) and designed for limited-ductility (transversal direction)

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 of structures. The study also clearly showed 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 seismic response of the structure. Based on those results, some modification propositions can be addressed to Eurocode 8-2, which consist in clarifying and upgrading the consistency of the effective cracked stiffness taking into account within the different 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 configurations including irregular geometries, other types of bearings, or critical soil-structure interaction.

REFERENCES

- AFPS (1995) (French Association for Earthquake Engineering), AFPS 92 Guide for Earthquake Resistant Protection of Bridges, Presses ENPC (in French)
- Blandon CA (2004) Equivalent Viscous Damping Equations for Direct Displacement-Based Design, Dissertation Submitted in Partial Fulfilment of the Requirements for the Master Degree in Earthquake Engineering, Rose School, European School of Advanced Studies in Reduction of Seismic Risk, Pavia, Italy
- CEN/TC250 (2005) European Standard: Eurocode 8 – Design of structures for earthquake resistance – Part 2: Bridges, European Committee for Standardization, Brussels
- CEN/TC250 (2004) European Standard: Eurocode 8 – Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings, European Committee for Standardization, Brussels
- Clough RW and Penzien J (1993) Dynamics of Structures, 2nd Ed., McGraw-Hill Book Company, New York
- CSI (1998) SAP2000® - Integrated Finite Element Analysis and Design of Structures – Basic Analysis Reference, Computers and Structures, Inc., Berkeley, California, USA
- Faella G, Giordano A and Mezzi M (2004) “Definition of Suitable Bilinear Pushover Curves in Nonlinear Static Analysis”, *Proceedings of the 13th World Conference on Earthquake Engineering*, Vancouver, B.C., Canada, August 1-6, Paper No. 1626
- Kolias B et al. (2012), Designers’ guide to Eurocode 8: design of bridges for earthquake resistance: EN 1998-2, ICE Publishing, London
- Kowalsky MJ and Ayers JP (2002) “Investigation of equivalent viscous damping for direct displacement-based design” *Pacific Earthquake Engineering Research Center, University of California Berkeley*, PEER-2002/02:173-185
- MEDDTL (2011) (French Ministry of Ecology, Sustainable Development, Transportation and Housing) “Order of the 26 of October 2011 relating to the paraseismic classification and construction rules applicable to bridges in the “normal risk” class”, *Official Journal of the French Republic, 2011 November 10th* (in French)
- MEDDTL (2010) (French Ministry of Ecology, Sustainable Development, Transportation and Housing) “Decree 2010-1255 of the 22 of October 2010 relating to the seismic zoning of the French territory”, *Official Journal of the French Republic, 2010 October 24th* (in French)
- MEDDTL (2010) (French Ministry of Ecology, Sustainable Development, Transportation and Housing) “Decree 2010-1254 of the 22 of October 2010 relating to the prevention of seismic risk”, *Official Journal of the French Republic, 2010 October 24th* (in French)
- Newmark NM and Veletsos AS (1960) “Effect of behaviour of simple systems to earthquake motion”, *Proceedings of the 2nd WCEE (World Conference on Earthquake Engineering)*, Tokyo, Japan, 895–912
- Otani S (1981) “Hysteresis models for Reinforced Concrete for Earthquake Response Analysis” *Journal*, Faculty of Engineering, University of Tokyo, Tokyo, Japan, V. XXXVI, No.2, 125-159
- Priestley MJN, Seible F, Calvi GM and Kowalski MJ (2008) “Displacement-Based Seismic Design of Structures,” *Earthquake Spectra*, Volume 24, Issue 2, pp. 555-557
- Priestley MJN, Seible F, Calvi GM (1996) Seismic Design and Retrofit of Bridges, Wiley, New York