Seismic design of concrete bridges: Some key issues to be addressed during the evolution of Eurocode 8 - Part 2

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Extended abstract

As of 2015, Eurocodes have entered the next stage of their development aiming at the publication of the second generation of the relevant EN Standards. The evolution of the Eurocodes has been scheduled by CEN in four phases, each involving different standards, and its main objectives are: (a) revision of the existing codes with a view to improving the 'ease of use', increasing harmonisation through the reduction of National Determined Parameters (NDPs), covering aspects of the assessment, re-use and retrofitting of existing structures, strengthening the requirements for robustness, and developing new Eurocodes; (b) essential maintenance of the existing Eurocodes and publication of relevant amendments to the existing standards in case emergent safety issues arise, and (c) promotion of the use of Structural Eurocodes in states outside the European Community. CEN/TC 250 has recently launched phase 3 of the systematic review of existing Eurocodes which includes among others the review of Eurocode 8 Part 2 (EN998-2) (CEN 2005). In this context, the present study, coinciding with the ongoing CEN public enquiry and feedback process on EN1998-2, attempts to identify some issues associated with the seismic design of concrete bridges designed for ductile behaviour of the piers that need, in the authors' view, some revision, namely: the criteria for regular/irregular bridge response and the associated behaviour factor (EN1998-2: §4.1.8), and the capacity design verification of the deck (EN1998-2: §5.6.3.6). The relevant clauses of EN1998-2 are critically presented and examples of the code-prescribed procedures by implementation in an actual bridge designed according to 'standard' European practice are used to point out associated pitfalls and some possible remediation.

Seismic design of bridges for ductile behaviour according to EN1998-2 intends to ensure a relatively uniform distribution of inelastic deformation demand among the selected dissipating zones (i.e. the pier ends), and hence increase the reliability of response spectrum analysis (Bardakis & Fardis 2011) performed using design spectra reduced by a global q-factor. As a means to this end, a 'regularity' index ($r_{i,}$) is defined at each intended plastic hinge location of a ductile member (i) and principal direction of the bridge as the ratio of the elastic bending moment demand at the hinge of the considered member under the 5% damped elastic response spectrum (i.e. disregarding q-factors) to the design flexural resistance of the same section with its actual reinforcement, both terms accounting for the concurrent action of the non-seismic action effects in the seismic design situation. Representing an approximate measure of local ductility demand, the extreme values of r_i among the intended plastic hinges are used

to quantify the 'regularity' of inelastic deformation demand distribution per principal direction of the bridge. A bridge is considered to have 'regular' seismic behaviour in the considered direction when r_{max}/r_{min} is lower than a limiting value ρ_0 set as an NDP with a recommended value of 2.0, and irregular behaviour otherwise. In view of the previous considerations, 'irregularity' of structural response is associated with widely varying overstrength among piers or more precisely with allocation of strength (to at least one pier) in excess of the seismic demand, and is treated in EN1998-2 either by decreasing the allowable q-factor, thus penalising the 'irregular' bridge, or by requiring nonlinear response history analysis (NLRHA). Irrespective of the efficiency of the adopted measure compared to others (e.g. AASHTO 2011, Guirguis & Mehanny 2013, Ayala & Escamilla 2013) in harmonising inelastic demands among piers, implications from the implementation of r_i indices resulting in overdesigning members, may arise from detailing constraints. For example, adoption of the same longitudinal reinforcement ratio in the top and bottom of pier columns monolithically connected to the deck will normally result in 'irregular' behaviour in the transverse direction of a straight bridge, even in the case of piers with similar height, if r_i are calculated in all locations of intended plastic hinges (i.e. in both pier ends). Similarly, adoption of relatively high minimum longitudinal reinforcement ratios promotes in general 'irregularity' among piers of unequal height. The previous implications are identified through a simple application in an actual 3-span bridge with single-column piers monolithically connected to the deck, and a possible treatment of the issue is subsequently proposed.

Modern codes (e.g. AASHTO 2011, Caltrans 2013, CEN 2005) require an explicit verification of the elastic response of 'capacity protected' members (e.g. the deck) when the components of the bridge energy dissipation system (e.g. the pier ends) reach their overstrength. EN1998-2 in particular, requires that non-significant yielding will occur in the deck under the 'capacity design effects' (EN1998-2: §5.3) determined from equilibrium conditions at the intended plastic mechanism, when all intended flexural hinges develop an upper fractile of their flexural resistance (i.e. overstrength). A general procedure for the estimation of the 'capacity design effects' in each principal direction of the considered bridge is provided in Annex G of the code (CEN 2005); it involves (a) the calculation of pier overstrength, (b) calculation of the change of the flexural moment of the deck corresponding to the increase of the moments of the pier plastic hinges, and (c) calculation of the deck 'capacity design effect' by superimposing the calculated response from (b) to the permanent action effects accounted in the seismic design situation. Implementation of the procedure in the previously considered 3-span bridge and subsequent verification using a more refined approach based on NLRHA and moment-curvature analysis of pier and deck sections (denoted as NLRHA in Fig. 1) indicate that the target performance prescribed by the code (i.e. no significant yielding of the deck) is achieved, since the 'capacity design effects' are found lower than the relevant yield moments of the deck (see Fig. 1 with regard to the longitudinal response). However, a couple of certain pitfalls are also identified. More specifically, the definition of the intended plastic mechanism in the transverse direction of the bridge may not be obvious especially as the complexity of the system increases (e.g. non-simultaneous yielding among pier columns of different geometry/detailing or between column ends), while the estimation of deck flexural moment demands corresponding well within the cracked state of deck sections (case of negative moments in Fig. 1) renders questionable the validity of analysis when the flexural stiffness of the deck is calculated on the basis of uncracked sections (a common assumption also adopted in US codes) and indicates the need for further consideration on this important issue.

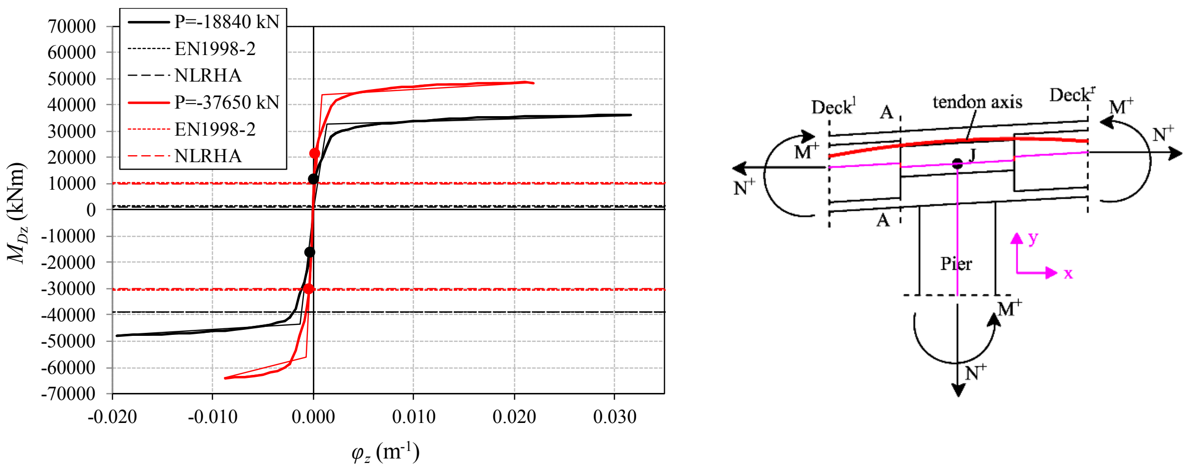

Fig. 1 Moment-curvature curves (flexure about z axis) of deck section A-A under different levels of prestressing force compared with the deck 'capacity design' moments (dashed lines) obtained from EN1998-2 and the NLRHA-based approach

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