Proceedings of the 3rd Croatian Conference on Earthquake Engineering - 3CroCEE
19–22 March 2025, Split, Croatia

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.120

NEW PROVISIONS FOR SEISMIC DESIGN OF BRIDGES

Ana Mandić Ivanković (1), Mladen Srbić (2), Gordana Hrelja Kovačević (3)

(1) *Prof. PhD, Faculty of Civil Engineering, Department of Structural Engineering, ana.mandic.ivankovic@grad.unizg.hr*
(2) *Assis. Prof. PhD, Faculty of Civil Engineering, Department of Structural Engineering, mladen.srbic@grad.unizg.hr*
(3) *Faculty of Civil Engineering, Department of Structural Engineering, gordana.hrelja.kovacevic@grad.unizg.hr*

Abstract

The paper presents an overview of the new provisions of the second generation of the EN 1998 standards dedicated to design of bridges for earthquake resistance based on available phase of development of standards. Main document Eurocode 8 – Design of structures for earthquake resistance – Part 2: Bridges (FprEN 1998-2:2024) is analysed in relation to the introductory Part 1-1: General rules and seismic action (FprEN 1998-1-1:2024) and basic document on Basis of structural and geotechnical design (FprEN 1990:2022) together with Part 1 on New structures (prEN 1990-1:2024). Authors aim to reveal the changes in the determination of seismic actions and in the approach to design of bridges for earthquake resistance and to emphasise the most important novelties.

Keywords: Eurocode 8, bridges, seismic action, consequence classes, limit states, ductility classes, modelling, analysis, detailing

1. Introduction

The first-generation of Eurocode 8 deals with seismic design and assessment of bridges through Part 1: General rules, seismic actions and rules for buildings (EN1998-1 or in Croatia HRN EN 1998-1:2008) [1], Part 2: Bridges (EN 1998-2 or in Croatia HRN EN 1998-2:2008) [2] and Part 3: Assessment and retrofitting of buildings (EN1998-3 or in Croatia HRN EN 1998-3:2008) [3]. Part 1 presents a traditional approach to seismic design, based on linear analysis and force-reduction, through the "behaviour factor" q, accounting for ductility, redundancy, and overstrength. Capacity design is enforced to ensure that the structure possesses the necessary global ductility [1, 4]. Part 2 follows this approach and provides provisions for seismic action, analysis, strength verification and detailing of bridges. It also includes a special section on seismic isolation with provisions covering the application of this method of seismic protection to bridges [2]. In addition, it offers 11 Annexes among which is the informative one on static non-linear (pushover) analysis. Part 3, at the time of release, among its main novelties, included nonlinear static analysis and displacement-based assessment. One further novelty of Part 3 was a framework to deal with the specific uncertainty of existing structures [3, 4]. Although the European standard EN 1998-3 [3] covers the assessment and retrofitting of buildings, and its Part 2 [2] focuses primarily on the seismic design of new bridges, a combination of these normative prescriptions may be used in the assessment of the existing bridges, but with additional improvement of certain aspects, such has been shown on the example of seismic assessment of existing reinforced-concrete arch bridges [5].

The second generation of Eurocode 8 is more clearly reconstructed in three levels covering: (i) seismic action & general rules in EN 1998-1-1, (ii) design of new structures, buildings in EN 1998-1-2 and bridges in EN 1998-2 and (iii) assessment of existing structures, both buildings and bridges, in EN 1998-3 [6]. Main document for bridge design *Eurocode 8 – Design of structures for earthquake resistance – Part 2: Bridges* (FprEN 1998-2:2024) [7] on one hand introduces new parts such are specific rules for cable stayed and extradosed bridges and integral abutment bridges and some additional information on timber bridges but on the other hand is very much reduced in size as seismic action is mainly covered in FprEN 1998-1-1 [8] and majority of previous annexes are removed to either FprEN 1998-1-1 [8] or FprEN 1990 [9] or to standards dedicated to seismic isolators [10].

In this paper the main provisions for earthquake resistant bridges based on current phase of development of standards will be overviewed aiming to reveal the changes in the determination of seismic actions

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.120

and in the approach to design of new bridges and to emphasise the most important novelties. Specific rules mentioned above, and annexes will not be discussed here as they are still to be further analysed and are beyond the scope of the basic approaches elaborated in this paper.

2. Basic requirements

2.1. Bridges in different Consequence Classes

Importance classes (I less than average, II average and III bridges of critical importance for maintaining communications) that were used to reliability differentiation in the first-generation of Eurocode 8 are being replaced by Consequence Classes (CC) which are given in Annex A2 of FprEN 1990 [6,9] on bridges, table A.2.1 as a Nationally Defined Parameters (NDP) so each country may adapt these definitions to its own need. Most bridges will fall within CC2 which covers normal consequence or CC3a which is the lower class of the high consequence class which covers railway bridges on main railway lines, bridges over main railway lines and bridges over and under major roads. Major roads are not just those roads considered as major in terms of volume of traffic or velocity such are highways or motorways. Those are also roads that have a major role for civil protection like i.e. the only road that connects an industrial plant or a small town to the main road network [10].

Table 1. Bridges in different consequence classes as in [9] and values of coefficient δ provided as NDP in [7]

Consequence class	Description of consequence	Bridge examples	Coefficient δ
CC4	Highest	-	-
CC3b	High (upper class)	Where an increased level of reliability is required, when specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties	1,60
CC3a	High (lower class)	Railway bridges on main railway lines, bridges over main railway lines, bridges over and under major roads	1,25
CC2	Normal	Bridges not in other consequence classes	1,00
CC1	Low	Short span bridges on local roads with little traffic (provided they do not span over main railway lines or major roads)	0,60
CC0	Lowest	Elements other than structural	-

Consequence class will influence the choice of coefficient δ which is one of the parameters necessary to define the seismic action index $S_\delta = \delta\ F_\alpha\ F_T\ S_{\alpha,475}$. In this formula δ is a coefficient that depends on the consequence class of the considered structure, F_α is the site amplification factor, F_T is the topography amplification factor and $S_{\alpha,475}$ is the reference maximum spectral acceleration for the return period of 475 years. The values of δ applicable to bridges (Table 1) are provided in FprEN 1998-2 [7] as Nationally Defined Parameters (NDP) so each country may adapt these definitions to its own need. Depending on the range of S_δ values, seismic action classes are defined in FprEN 1998-1-1 [8] as *very low* seismic action class with a range of seismic action index $S_\delta < 1{,}30$ m/s^2, *low* with a range 1,30 m/s^2 $\leq S_\delta < 3{,}25$ m/s^2, *moderate* with a range 3,25 m/s^2 $\leq S_\delta < 6{,}50$ m/s^2 and *high* with a range $S_\delta \geq 6{,}50$ m/s^2.

2.2. Limit states

In the first generation of Eurocodes limit states (No Collapse and Damage Limitation) defined in the parts 1 [1] and 2 [2] for new structures were not consistent with those defined in part 3 [3] for existing structures (Near Collapse, Significant Damage, Damage Limitation). Therefore, second generation of Eurocode 8 provides a homogenisation of limit states definition through all its parts with better consistency with definition of Ultimate Limit States and Serviceability Limit States in Eurocode 0, introducing, in addition, a new Fully Operational limit state [6,7,8,9]. At least one Ultimate Limit State verification is mandatory to reach the safety of the structure and the choice of Serviceability Limit States

Proceedings of the 3rd Croatian Conference on Earthquake Engineering - 3CroCEE
19–22 March 2025, Split, Croatia

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.120

to be verified is up to National Annex or the contract [6]. To satisfy the seismic performance requirements for new structures the non-exceedance of the SD limit state shall be verified and the choice of other limit states to be verified could be defined in National Annex.

Table 2. Details of ultimate and serviceability limit states for seismic design situations

Limit states		Quantitative definition [8]
Ultimate Limit States ULS	Near Collapse NC	LS of Near Collapse (NC) shall be defined as one in which the structure is heavily damaged, with large permanent drifts, but retains its vertical load bearing capacity; most ancillary components, where present, have collapsed.
	Significant Damage SD	LS of Significant Damage (SD) shall be defined as one in which the structure is significantly damaged, possibly with moderate permanent drifts, but retains its vertical load bearing capacity; ancillary components, where present, are moderately damaged. The structure is expected to be repairable but, in some cases, it may be uneconomic to repair.
Serviceability Limit States SLS	Damage Limitation DL	LS of Damage Limitation (DL) shall be defined as one in which the structure is only slightly damaged and economic to repair, with negligible permanent drifts, undiminished ability to withstand future earthquakes and structural members retaining their full strength with a limited decrease in stiffness; ancillary components exhibit only minor damage that can be economically repaired.
	Operability OP	Fully Operational LS (OP) shall be defined as one in which the structure is only slightly damaged and economic to repair, allowing continuous operation of systems hosted by the structure.

3. Seismic action

Seismic action is defined in FprEN 1998-1-1 [8] and design seismic action is specified either in terms of return period $T_{LS,CC}$ or in terms of the performance factor $\gamma_{LS,CC}$ both given in FprEN 1998-2 [7], tables 4.2 (NDP) and 4.3 (NDP) respectively, as a function of the limit state and the consequence class. Return periods and performance factors for bridges are the same as for buildings for the same limit state and consequence class. Return periods $T_{LS,CC}$ and performance factors $\gamma_{LS,CC}$ depend on the target reliability indices $\beta_{t,LS,CC}$ which is suggested in Annex F.3 of FprEN 1998-1-1 [8] and are both Nationally Defined Parameters. Target reliability indices $\beta_{t,LS,CC}$ is related to the probability of failure of the bridge or of exceedance of the limit state. In the first generation of Eurocode 8 importance factors were used so, for comparison, for the significant damage for a less than normal important bridge (CC1) this factor previously was 0,85 now it is 0,80 (* Table 4) and for more important bridges (CC3) it was 1,30 now it is 1,1 or 1,25 (** Table 4) [10].

Table 3. Return period $T_{LS,CC}$ for bridges provided as NDP values in [7] (in years)

Limit state	Consequence class			
	CC1	**CC2**	**CC3a**	**CC3b**
NC	600	1600	2500	5000
SD	275	475	600	900
DL	100	115	125	140

Table 4. Performance factor $\gamma_{LS,CC}$ for bridges provided as NDP values in [7]

Limit state	Consequence class			
	CC1	**CC2**	**CC3a**	**CC3b**
NC	1,10	1,50	1,75	2,20
SD	0,80*	1,00	1,10**	1,25**
DL	0,60	0,60	0,65	0,65

Proceedings of the 3rd Croatian Conference on Earthquake Engineering - 3CroCEE
19–22 March 2025, Split, Croatia

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.120

An important aspect of seismic action for bridges is how to deal with the topographic amplification because these are not single point facilities but extended structure. Guidance for the evaluation of the topographic amplification factor F_T for bridges is given in FprEN 1998-2 [7]. Since the bridge has different points of contact of the structure at the supports with the ground it would be necessary to come up with a set of values of topographic amplification factor F_T, one per each support. Unless spatial variability is considered in the analysis, an average value of the topographic amplification factor will have to be used to amplify the uniform excitation applied to the bridge. This is applicable when the bridge model does not include the soil domain with its geometry which is in majority of all cases when regular fixed based model or a simple model with soil structure interaction is used [10].

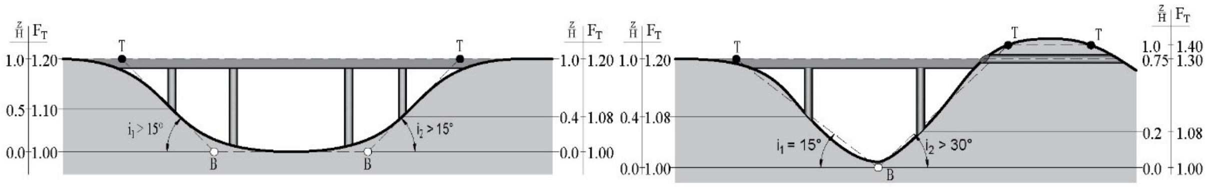


Figure 1. Calculation of topographic amplification factors F_T at supports in the case of wide valleys (left) and narrow valleys (right) where H is a slope height, z is height of support with respect to slope base point B and i is inclination angle of slope, reproduced based on [7]

3.1.Site consideration

When designing the bridge that will be built in close vicinity to active and shallow faults, permanent displacements arising from ground failure or fault rupture can result in imposed deformations with severe consequences for bridges. In this case the code FprEN 1998-2 [7] gives a recommendation that ground permanent displacements should be evaluated through specific studies as this problem is not manageable with a simplified treatment [10]. Some information on the seismic part of the differential displacement at the fault crossing can be found in prEN 1998-4:2023 [11], Annex E as pipelines share the same vulnerability with bridges due to their extended nature [10]. For other considerations regarding the site, as stability of slopes or liquefaction susceptibility, reference is made to prEN 1998-5:2024 [12].

4. Characteristics of earthquake resistant bridges

Strategies recommended for satisfying the performance requirements of non-exceedance of the limit states for the given seismic action can be achieved with either energy dissipation in members or anti-seismic devices or both as combining both strategies is common situation in bridges. Bridge foundations should not intentionally be used as a source of energy dissipation mainly due to limited asses for repairs [7,10].

Further important issue, which was already introduced in the first generation of Eurocode 8, is that the global torsional resistance of a bridge around the vertical axis should not rely on the torsional rigidity of a single pier. For example, a highway overpass with a middle pier, transversally fixed at abutments is fine in this sense, but if it has sliding bearings at the abutments or if the bridge is a single span one, this phenomenon has to be considered [7,10].

4.1.Primary and secondary members

Distinction between primary and secondary members is introduced in the general part FprEN 1998-1-1 [8] for buildings and other structures and the part for bridges FprEN 1998-2 [7] reads that supporting members (piers and abutments) resisting the seismic force in the given direction (longitudinal or transverse) should be designated as primary emphasising that the number of primary members may be less than a total number of supporting members.

Example is a continuous bridge over four spans with two abutments, one short and two taller piers (Figure 2). Unidirectional longitudinally sliding bearings on the abutments and short pier will result those supporting members to be considered as secondary members in longitudinal direction. Taller piers which are connected to the deck, because the deck is used to restrain the top displacement of relatively

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.120

slender piers, are considered as primary members in the longitudinal direction. In addition, if, in moderate and high seismic action class, deck is connected to one abutment through dampers or anti seismic devices or rigid connection, engaging its vibration to reduce displacement, this abutment will also be considered as primary seismic member in the longitudinal direction. In the above-described case, as sliding bearings are unidirectional longitudinally, all supports are considered as primary in the transverse direction [7,10]. In addition, FprEN 1998-2 [7] defines a sacrificial element, typically abutment's back walls, which can be designed to fail in the event of an impact with a deck as long as they are easily repairable and are protecting the remaining parts, i.e. deck joint [10]. Guidelines for design of the abutment's back walls should be provided so that damage is minimal and controlled.

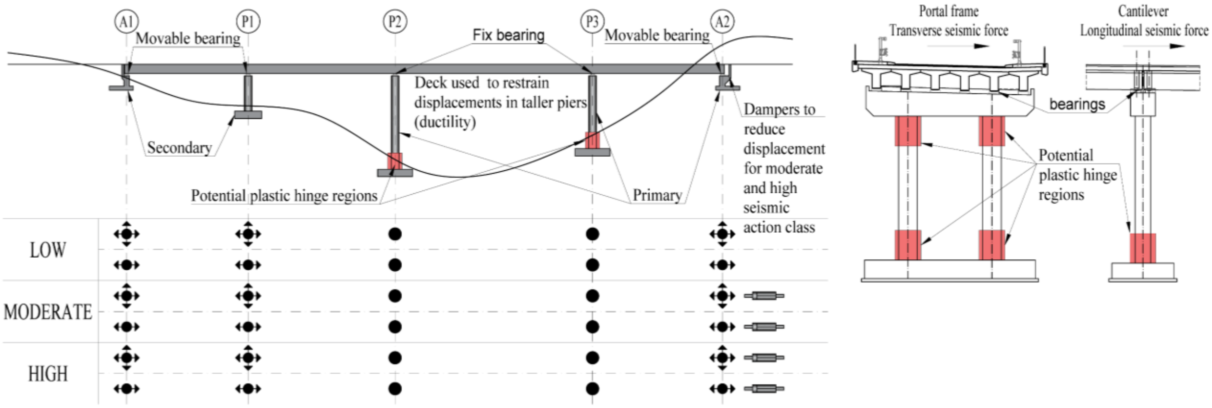


Figure 2. Primary and secondary members of the bridge example and the location of potential plastic hinges, reproduced based on [10]

4.2. Choice of Ductility Class

For bridges that are exploiting ductility to satisfy performance requirements (and not those who rely on seismic isolation) the following provisions are given. The primary structure should be assigned a ductility class according to FprEN 1998-1-1 [8]. Ductility classes are described based on the structure deformation capacity and cumulative energy dissipation capacity in the table 5.

Table 5. Provisions for choosing ductility class depending on the seismic action class and consequence class of the bridge

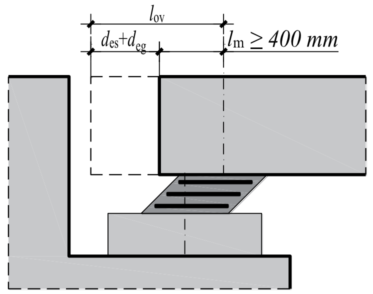
Ductility		**Seismic action class**		
Ductility class categories	Description of the structure deformation capacity and cumulative energy dissipation capacity	**Low**	**Moderate**	**High**
DC1	The overstrength capacity is taken into account, while the inelastic deformation capacity and energy dissipation capacity are disregarded.	+ (all CC)	-	-
DC2	The local overstrength capacity, the local deformation capacity and the local energy dissipation capacity are taken into account. Global plastic mechanisms are controlled by limited verifications.	+ (all CC)	+ (all CC)	+ (CC1)
DC3	The ability of the structure to form a global plastic mechanism at SD limit state is verified and its local overstrength capacity, local deformation capacity and local energy dissipation capacity are taken into account.	+ (all CC)	+ (all CC)	+ (CC2, CC3)

This ductility class should be unique for the bridge - the same for all members and in all directions. Assigning the unique ductility does not mean the same as that the behaviour factor q will be unique, unique ductility means that approach to detailing will be the same for all primary structures as ductility class determines detailing and not just the value of behaviour factor q [10]. Not all ductility classes can be used irrespective of seismic action class or seismic intensity. In high seismic action class, bridges of CC2 and higher should be designed for DC3 but bridge of CC1 could be designed for DC2. In addition, seismic design for DC1, which exploit overstrength capacity only, should not be adopted in moderate and high seismic action classes which altogether leads to the provisions for choosing ductility class for bridges as presented in the table 5 [7,10].

The locations of critical zones should be chosen to ensure accessibility for inspection and repair. Such locations should be clearly indicated in the appropriate design documents. For DC2 and DC3 structures, a dependably stable partial or full mechanism should develop in the structure through the formation of flexural plastic hinges [7]. Potential plastic hinges may not be foreseen wherever, they are usually in the piers and not necessary in all piers, i.e. in the longitudinal direction they will form in the primary members only as indicated in Figure 2 [10]. The bridge deck should remain within the elastic range except in the case of adjacent simply supported spans made of precast concrete girders beams connected with continuity slabs [7]. Plastic hinges are allowed in the continuity slabs, but they cannot be relied upon for energy dissipation [10].

4.3. Unseating and pounding

Important principle when designing connections between supporting and supported members is to avoid unseating which is provided with appropriate overlap lengths (Figure 3). In addition, pounding between adjacent parts which can oscillate and get in contact should be avoided. An exception is the sacrificial back wall, but it cannot be provided in all cases as the deck should be tolerant to this pounding as well. For example, large shock forces on sensitive components such as prestressing anchorages at the prestressed concrete girder head should be prevented by ductile/resilient members or special buffers [7].



$\boldsymbol{l_{ov}} = l_m + d_{eg} + d_{es}$	**overlap length**
$\boldsymbol{l_m}$	**minimum support length** (>400mm)
$\boldsymbol{d_{eg}} = (2d_g / l_g) l_{eff} \leq 2d_g$	**relative displacement** (spatial variation)
d_g	expected ground displacement
l_g	distance parameter
l_{eff}	effective length of the deck
$\boldsymbol{d_{es}}$ (= d_{Ed} or $d_{Ed}+s$)	**effective seismic displacement** (s, the link slack)

Figure 3. Minimum overlap length at connections, reproduced based on [13,14 & 7]

In order to size the gaps to avoid unseating and pounding the evaluation of displacements is necessary [10] which is done through the formula which is the same as in the first generation of Eurocode 8: $d_{Ed} = d_G$ "+" d_E "+" $\psi_2 d_T$; where d_G is the governing value between the short-term (opening) and long-term (end of design service life) values of the relative displacement due to the permanent and quasi-permanent actions (e.g. post-tensioning, shrinkage and creep for concrete decks), d_E is the design relative displacement due to seismic action (displacement of adjacent portions), d_T is the design relative displacement due to thermal movements and ψ_2 is the combination factor for the quasi-permanent value of thermal action, given in FprEN 1990, Table A.2.7(NDP) [9]. For timber bridges the above equation should also include d_ω which is the displacement due to average moisture content variation Δ_ω [10]. It should be noted that even when the bearings are dimensioned for sufficient displacement, dimensioning the expansion joint devices for smaller displacements may result in the superstructure pounding the abutment wall, therefore its appropriate design is necessary as mentioned above.

5. Modelling & Analysis

5.1. Mass, stiffness and damping

In terms of modelling the first issue is to add non-structural masses to structural ones. These are the masses due to traffic and masses due to water. The proportion of the mass corresponding to traffic needs

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.120

to be added only for road and railway bridge with severe traffic conditions and this is by employing coefficient $\psi_{E,I}$ with the value 0,2 for road and 0,3 for traffic bridges. It has to be noted that load models for traffic are different and only uniformly distributed load of the corresponding load model (LM1 for loads and LM71 for railways) should be considered for the evaluation of additional masses [10]. Calculation of the added mass of entrained water for immersed piers is the same as it was in the first generation of Eurocode 8 and covered with the only Annex left (Annex B) from this previous Eurocode version [10].

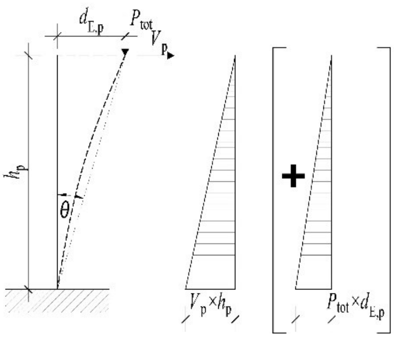
When designing bridges for ductility classes DC2 and DC3, in critical regions of primary seismic members, where energy dissipation is considered as a part of the seismic design strategy, the elastic stiffness of each member should correspond to its secant effective stiffness at the elastic limit. For reinforced concrete piers this is estimated as secant stiffness of the cracked section at the initiation of the yield of the reinforcement which is approximated as 50% of the corresponding stiffness of the uncracked members for the force-based approach. In the bridge deck where no inelastic action happens, the gross stiffness of 100 % should be considered with the reduction to 25% for the continuity slabs where plastic hinges are allowed but they cannot be relied upon for energy dissipation [7,10].

When cracking occurs, torsional stiffness of the deck decreases in a much more relevant manner which is the reason why for open sections or slabs torsional stiffness is taken with 0% of the uncracked torsional stiffness which means it is neglected, and for box sections it is proportional to 50% or 30 % depending on whether the box is made of prestressed or reinforced concrete respectively. Particular care should be dedicated to curved bridges where torsion contributes to the static equilibrium. Curved bridge is the bridge with an angle between the initial and final tangents to the curved longitudinal axis larger than 25° [7,10].

When elastic response spectrum analysis is used weighting factors for damping are 0,02 for welded steel, 0,04 for bolted steel, 0,05 for reinforced concrete and 0,02 for prestressed concrete based on the material of the members where larger part of the deformation energy is dissipated during the seismic response, which is the same as in the first generation of Eurocode with an added value 0,03 for timber. For response history analysis individual damping should be assigned to each component and when the q-factor approach is used, there is no correction of damping in the reduced spectrum [7,10].

5.2. Second order effects

Second order effects (P-Δ effects) may be neglected when pier top displacement sensitivity coefficient θ is less than 0,1. This sensitivity coefficient presenting the ratio of second order moments over first order moments (or $P_{tot}d_{E,p}$ over $V_p h_p$, where P_{tot} is vertical force; $d_{E,p}$ pier top displacement, V_p shear force and h_p pier hight) is computed differently depending on whether linear force-based approach (L-FBA) or non-linear displacement-based approach (NL-DBA) is used [7,8,10].



L-FBA:
$\theta = (P_{tot}d_{E,p}) / (q_R q_S V_p h_p)$
$d_{E,p} = q_{disp} d_r$

NL-DBA:
$\theta = (P_{tot}d_{E,p}) / (V_p h_p)$
$d_{E,p} = d_t^*$

d_r displacement calculated under the reduced spectrum associated to the design seismic action

q_{disp} $= q$; if $T_1 \geq T_B$,
$= [1+(q-1)\,T_B/T_1]; \leq 3q$; if $T_1 < T_B$

d_t^* target displacement of the equivalent single degree of freedom oscillator

Figure 4. P-Δ effects and corresponding terms

5.3. Vertical component of seismic action

Vertical component of the seismic force should be considered when verifying: (i) structural members of prestressed concrete decks because prestressed members are sensitive to vertical loads, (ii) structural

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.120

members in cable stayed bridges which have inclined forces and axial forces in pylon, (iii) anti-seismic devices which may be very sensitive to axial forces or (iv) inclined piers where both vertical and horizontal component of seismic force lead to axial force variations in the members. On the other hand, effects of vertical component may be omitted for piers in low or moderate seismic intensities [7,10].

5.4. Behaviour factors for the force-based approach (linear analysis)

In the force-based approach, the seismic action should take the form of a reduced spectrum, derived from the elastic response spectrum by introducing the behaviour factor q, which accounts for overstrength, deformation capacity and energy dissipation capacity [8].

The values of behaviour factors for the force-based approach have basically not changed with respect to those in the first generation of Eurocode 8 but they have been split so their contribution is clearly separated into contribution of overstrength (which is taken with q_s=1.5, which means that for DC1 the final q = 1,5) and in the contribution coming from redundancy q_R and ductility q_D which are specified in the table 6 depending on the type of ductile member and for ductility classes DC2 and DC3 for which all three components are to be added to the final value q [7,10].

Table 6. Example of maximum values of behaviour factors for reinforced concrete piers and abutments, extracted from [7]

Type of Ductile Members	q_R	q_D		$q = q_S\, q_R\, q_D$	
		DC2	DC3	DC2	DC3
Multiple double-bending vertical RC piers (i.e. more than one monolithically connected pier in longitudinal direction or multicolumn piers in transverse direction)	1,2	1,3λ(a_s)	2,0λ(a_s)	2,3λ(a_s)	3,6λ(a_s)
Multiple single-bending vertical RC piers (i.e. more than one pin-connected pier in longitudinal direction or single-column piers in transverse direction)	1,0	1,3λ(a_s)	2,0λ(a_s)	2,0λ(a_s)	3,0λ(a_s)
Inclined RC struts in bending	1,1	1,0λ(a_s)	1,3λ(a_s)	1,6λ(a_s)	2,1λ(a_s)
Abutments rigidly connected to the deck	1,1	1,0	1,1	1,6	1,8
Abutments of integral abutment bridges	1,0	1,0	1,0	1,5	1,5
λ(a_s) = $\sqrt{1/3}$ if a_s<1; λ(a_s) = $\sqrt{a_s/3}$ if 3 > a_s ≥ 1; λ(a_s) =1 if a_s ≥ 3; where $a_s = L_V/h$ is the shear span ratio, ratio between the distance from the plastic hinge to the point of zero moment and the cross-section height in the plane of deformation					

Nevertheless, second generation of Eurocode 8 introduces accounting for the foundation flexibility when selecting the behaviour factor [7]. If soil-structure interaction is considered in the bridge model, the ductility should be reduced because the deformation of the soil-foundation system absorbs a portion of overall deformation. Ductility will need to be reduced also for (i) high axial force, (ii) non-accessibility of critical zones for inspection and repair and for (iii) irregular inelastic demand. The latter applies as for the application of the force-based approach, bridges should be considered to have regular seismic behaviour. If bridges have irregular seismic behaviour, they should be designed either by using a reduced q value or by employing the displacement-based approach [10].

It needs to be emphasised, while the ductility class for the bridge is unique as explained in the previous chapter 4.2, different values of the behaviour factor q may be used in each of the two horizontal directions. When the bridge is curved or skew and different ductility is available in different directions for each supporting pier, the lower q of the primary members govern the ductility class. When the bridge is neither curved nor skew, the higher value of q may be used in the direction of higher available ductility, while using lower value of q in the orthogonal, less ductile direction [7]. This is a typical case

Proceedings of the 3rd Croatian Conference on Earthquake Engineering - 3CroCEE
19–22 March 2025, Split, Croatia

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.120

of a straight bridge with wall type piers where ductility is very different in the longitudinal and transversal direction.

5.5. Displacement-based approach (nonlinear analysis)

Methods used to implement the displacement-based approach are: (i) non-linear static analysis and (ii) response history analysis. Single mode non-linear static (pushover) method is an invariant of N2 version of the method. N2 was first proposed and applied to buildings in 1987, and in 1997 it was applied to the example of a bridge with appropriate adaptations. The N2 method includes two types of calculation and two structural models: 1) non-linear static calculation on the real structural model - model with multiple degrees of freedom and 2) non-linear dynamic calculation on a simplified model - model with one degree of freedom. The static calculation determines the properties of the structure (e.g. stiffness) that are required for the dynamic calculation. Compared to the non-linear calculation of the dynamic time response, the calculation according to the N2 method is significantly simplified, as it is performed on a simplified model. Moreover, in the N2 method, the seismic load is usually defined by nonlinear response spectra determined on the basis of standard elastic spectra, which avoids difficulties and doubts in the selection of accelerograms required for the nonlinear calculation of the time response. Due to the simplification of the method, it also has certain limitations. It is suitable for the calculation of structures where there is no major influence of higher vibration modes and where the dominant vibrations do not change significantly with different intensities of seismic loading [15, 16].

Single mode pushover method is applicable only to straight bridges (less than 25°) with two modes having modal mass larger than 60% in the two plan directions [10]. Assuming the influence of the pier masses to be minor, the above condition is always met in the longitudinal direction of approximately straight bridges. The condition is also met in the transverse direction when the distribution of the stiffness of piers along the bridge provides an approximately uniform lateral support to a relatively stiff deck. This is the most common case for bridges where the height of the piers decreases towards the abutments or does not present intense variations. When, however, the bridge has one stiffer and unyielding pier, located between groups of regular piers, the system cannot be approximated in the transverse direction by a single degree of freedom and pushover analysis can lead to unrealistic results. A similar exception holds for long bridges, when very stiff piers are located between groups of regular ones, or in bridges in which the mass of some piers has a significant effect on the dynamic behaviour, in either of the two directions. When possible and expedient, such irregular arrangements can be avoided, e.g. by providing sliding connection between the deck and the pier(s) that cause the irregularity [7]. Single pushover method, N2 method is a simple and it should stay simple as indicated with the note that verification are carried out independently in the longitudinal and in the transversal direction [7].

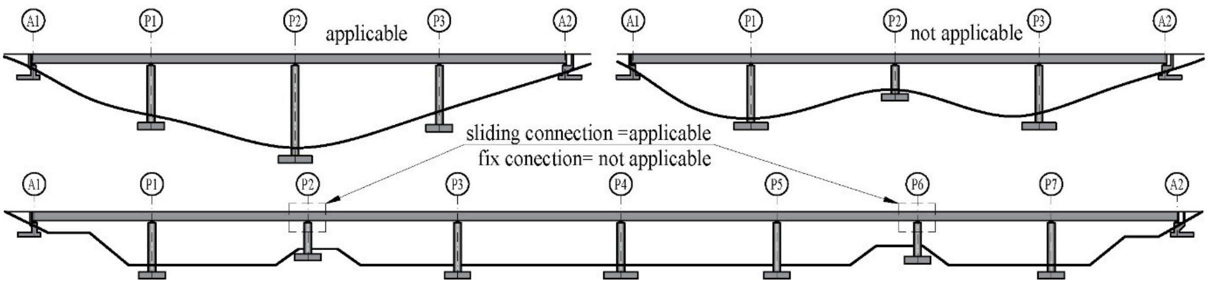


Figure 5. Applicability of single mode pushover method

When single pushover method (N2) method is not possible, Eurocode [7,8] allows to use alternative methods. Non-linear static analysis methods accounting for the response of higher modes such are modal pushover methods are permitted with the standard, but they are not further explained, and their limitations are noted, emphasising they are not of general applicability. This means that when N2 method is not possible, modal pushover method may be applied but better way is to use non-linear response analyses [10]. The choice of number and type of input motions, as well as calculation of seismic action effects, should in this case comply with FprEN 1998-1-1 [8].

Proceedings of the 3rd Croatian Conference on Earthquake Engineering - 3CroCEE
19–22 March 2025, Split, Croatia

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.120

5.6. Combination of the seismic action with other actions

The design value E_d of the effects of actions in the seismic design situation should be determined in accordance with FprEN 1990:2022 [9] with following provisions of FprEN 1998-2:2024 [7]. Action effects due to imposed deformations may be neglected except in the case of bridges in which the seismic action is resisted by elastomeric laminated bearings. In this case, the action effects due to imposed deformations (caused by temperature, shrinkage, settlements of supports, residual ground movements due to seismic faulting) should be accounted for. Displacement due to creep does not normally induce additional stresses to the system and can therefore be neglected. Creep also reduces the effective stresses induced in the structure by long-term imposed deformations (e.g. by shrinkage) [7].

6. Verification of structural members to limit states

Clause 6 of FprEN 1998-2:2024 [7] should be applied to the earthquake resisting system of bridges designed for DC1, DC2 or DC3. This clause provides provisions for the design of structural members and for the detailing of the critical regions of each member type. Outside the critical regions, the detailing of structural members should satisfy relevant provisions in standards dedicated to concrete, steel or composite structures.

Verification of Significant Damage (SD) limit state according to the force-based approach should be carried out in terms of local resistances while demand on non-ductile members should be obtained as capacity design effects. Brittle and other undesired failure mechanisms should be avoided by deriving design action effects from capacity design if not exceeding those obtained with q=1. Formula for overstrength moment of a flexural plastic hinge M_o, associated variables, graphical representation of capacity design bending moment within the length of a member developing plastic hinges are provided in relevant subclauses of FprEN 1998-2:2024 [7]. It has to be noted that values of the overstrength partial factor γ_{Rd} (recommended as 1,1 when M_o is used to calculate seismic action effects on the shear mechanism, and 1,0 otherwise) may be adapted as a Nationally Defined Parameters (NDP) [13]. Flexural resistance of sections in critical regions of concrete members should be satisfied with $M_{Ed} \leq M_{Rd}$ where M_{Rd} is the design flexural resistance of the section in accordance with standard for concrete structures. For structures of DC1 M_{Ed} is derived from analysis for the seismic design situation, including second-order effects if needed; and for shear resistance verification, seismic action effect A_{Ed} should be multiplied by the behaviour factor q. For structures of DC2 and DC3 M_{Ed} is the design moment accounting for capacity design effects and for shear resistance verification, design action effect should account for capacity design effect. In addition, for structures of DC2 and DC3 any joint between a vertical ductile pier and the deck or a foundation member adjacent to a plastic hinge in the pier should be designed in shear to resist the capacity design effects of the plastic hinge in the relevant direction. Definition of variables, stress condition and formulas are provided in relevant subclauses of FprEN 1998-2:2024 [7]. It needs to be mentioned that no significant yielding should occur in the deck and elastic response of foundation members is preferred for bridges [13].

In case the Near Collapse (NC) limit state is used, verifications should be carried out with the displacement-based approach, via non-linear static or response-history analysis since the seismic action for this limit state can drive the structure into the non-linear range to an extent where results of a linear analysis are less reliable than they are at the SD limit state [7]. In case the DL or OP limit state is required, verification may be carried out with the force-based or the displacement-based approach [7].

7. Detailing for ductility

Detailing for ductility should be applied to primary seismic members (piers and abutments) of bridges designed for DC2 and DC3 through plastic hinging and aims to ensure a minimum level of curvature/rotation ductility at the plastic hinges. General requirement is that in concrete piers longitudinal reinforcement ratio ρ_L should be higher than 0,5% (in the previous version it was 1,0%) and the diameter of the bar d_{bL} should be equal or higher than 16 mm. Further on several requirements for critical regions in concrete piers will be elaborated.

Proceedings of the 3rd Croatian Conference on Earthquake Engineering - 3CroCEE
19–22 March 2025, Split, Croatia

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.120

Length of critical regions l_{cr} is estimated as the largest of the: (i) the depth of the pier section within the plane of bending perpendicular to the axis of rotation; (ii) distance between M_{max} and 0,8 M_{max} locations but not larger than 1,5xdepth of the pier section from (i). This is for the cases when ratio $\eta_k = N_{Ed}/A_c f_{ck} \leq 0,3$, while for $0,3 < \eta_k \leq 0,6$ critical length should be increased by 50%. It has to be noted that this length of critical regions is only used for detailing of the reinforcement and not for estimating the plastic hinge rotation [7, 13]. The longitudinal reinforcement should remain constant and fully effective over the length of the critical region l_{cr} without splicing by lapping or welding.

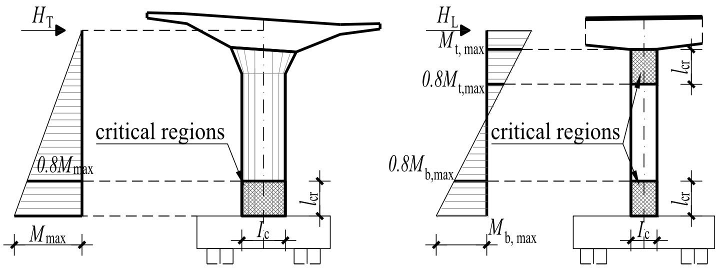


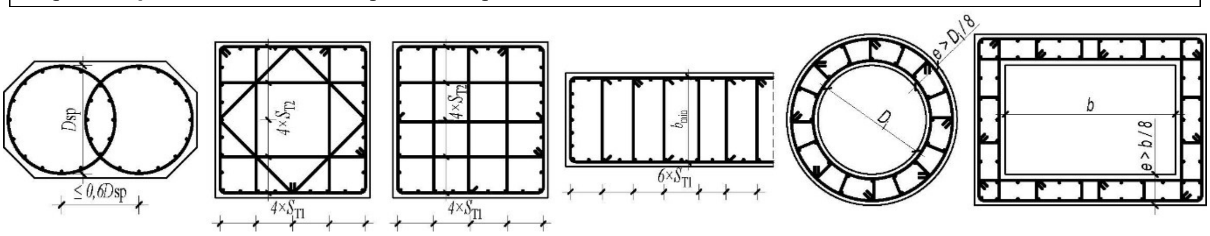
Figure 6. Location of critical regions in concrete piers: for the transversal (left), for longitudinal direction (right), reproduced based on [13]

Confinement should be ensured within the critical regions of the primary seismic members, through rectangular or circular hoops and/or cross ties and spirals, with dimeter $d_{bT} \geq 10$ mm. The quantity of confining reinforcement should be defined through the mechanical reinforcement ratio $\omega_{wd} = \rho_w (f_{yd}/f_{cd})$ where ρ_w is transverse reinforcement volumetric ratio, presented in the table 7 together with the minimum amount of confining reinforcement $\omega_{wd,min}$. This minimum amount is to be provided over the entire length of the critical region, it may be reduced gradually outside the critical region but not less than 50% over an additional adjacent length l_{cr}. In addition, outward buckling of longitudinal reinforcement shall be avoided along potential hinge areas by restraining it by transverse reinforcement (hoops or cross-ties) perpendicular to the longitudinal bars at a spacing $s_L \leq 5d_{bL}$ and $s_T \leq 200$mm.

Table 7. Provisions for confining reinforcement extracted from [7]

Section type	**Type of reinforcement**	ρ_w	$\omega_{wd,min}$		s_L **and/or** s_T
			DC2	**DC3**	
Rectangular	Hoops or ties	$\rho_w = A_{sw}/(s_L b)$	0,08	0,12	$s_L \leq 6d_{bL}$; $s_L \leq 1/5b_{min}$ $s_T \leq 1/3b_{min}$ or 200mm for $b_{min} \leq 1{,}0$m or 300mm for $b_{min} > 1{,}5$m
Circular	Spiral or hoop bars	$\rho_w = 4A_{sp}/(D_{sp}s_L)$	0,12	0,18	$s_L \leq 6d_{bL}$; $s_L \leq 1/5b_{min}$

A_{sw} is the total area of a layer of hoops or ties in the direction of confinement under consideration; s_L is the spacing of confining reinforcement in the longitudinal direction; b is the dimension of the concrete core perpendicular to the direction of the confinement under consideration, measured to the outside of the perimeter hoop; s_T is the transverse distance between hoop legs; b_{min} smallest dimension of confined concrete core; d_{bL} longitudinal bar diameter; A_{sp} is the area of the spiral or hoop bar; D_{sp} is the diameter of the spiral or hoop bar



a) b) c) d) e)

Figure 7. Confinement details in concrete piers using (a) interlocking spirals/hoops in polygonal cross section; (b) 4 closed overlapping hoops; (c) 3 closed overlapping hoops plus cross-ties; (d) closed overlapping hoops plus cross ties in wall type rectangular cross section; (e) closed overlapping hoops plus cross ties in hollow piers, reproduced based on [7]

Proceedings of the 3rd Croatian Conference on Earthquake Engineering - 3CroCEE
19–22 March 2025, Split, Croatia

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.120

8. Conclusions

In this paper authors overviewed the main provisions for earthquake resistant bridges introduced with the 2nd generation of Eurocodes. The most important novelties are (i) the determination of seismic action as a function of the clearly defined limit state and the consequence classes defined for bridges, (ii) choosing ductility class of the bridge depending on the seismic action class and consequence class of the bridge and further on (iii) clear separation of behaviour factor into contribution of overstrength, redundancy and ductility. The work on Croatian National Annex of EN 1998-2, contemplating allowable national choice through the value of Nationally Determined Parameters, is under progress and will be the subject of a further publication. It should be emphasized that a complete insight into the benefits of the 2nd generation of Eurocode will only be possible by reviewing and developing of all standards and national annexes related to bridges, and only the experience with actual bridge projects will reveal whether one of the main goal, simpler application in practice, is reached.

References

[1] EN 1998-1:2004. Eurocode 8: Design of structures for earthquake resistance Part 1: General rules, seismic actions and rules for buildings, CEN- European Committee for Standardization, Bruxelles, 2004.

[2] EN 1998-2:2005. Eurocode 8: Design of structures for earthquake resistance, Part 2: Bridges, CEN- European Committee for Standardization, Bruxelles, 2005.

[3] EN 1998-3:2005. Eurocode 8: Design of structures for earthquake resistance, Part 3: Assessment and retrofitting of buildings, CEN- European Committee for Standardization, Bruxelles, 2005.

[4] Franchin, P., Noto, F.(2023): Reliability-based partial factors for seismic design and assessment consistent with second-generation Eurocode 8, *Earthquake Engineering & Structural Dynamics*: Volume 52, Issue 13, https://doi.org/10.1002/eqe.3840

[5] Franetović, M., Mandić Ivanković, A., Radić, J. (2014): Seismic assessment of existing reinforced-concrete arch bridges, *Građevinar* 66, 8, 691-703

[6] Bisch, P. (2022): EC8 Webinars Second Generation of Eurocode, Webinar 1-1.1 *Organisation and concepts of EN 1998*, 30th March 2022, https://ec8webinars.org/, (accessed September 21, 2024).

[7] FprEN 1998-2:2024. Eurocode 8: Design of structures for earthquake resistance Part 2: Bridges, CEN- European Committee for Standardization, Bruxelles, 2024.

[8] FprEN 1998-1-1:2024. Eurocode 8: Design of structures for earthquake resistance Part 1-1: General rules and seismic action, CEN- European Committee for Standardization, Bruxelles, 2023.

[9] FprEN 1990:2022. Eurocode - Basis of structural and geotechnical design & prEN 1990-1:2024 Part 1: New structures, CEN- European Committee for Standardization, Bruxelles, 2023.

[10] Franchin, P. (2024): EC8 Webinars Second Generation of Eurocode, Webinar 2.1 *Bridge classification and structural analysis*, 5th April 2024, https://ec8webinars.org/, (accessed September 21, 2024).

[11] prEN 1998-4:2023. Eurocode 8: Design of structures for earthquake resistance Part 4: Silos, tanks, pipelines, towers masts and chimneys, CEN- European Committee for Standardization, Bruxelles, 2023.

[12] FprEN 1998-5:2024. Eurocode 8: Design of structures for earthquake resistance Part 5: Geotechnical aspects, foundations, retaining and underground structures, CEN, Bruxelles, 2023.

[13] Davi, D. (2024): EC8 Webinars Second Generation of Eurocode, Webinar 2.3: Verifications of structural members and detailing for ductility, 5th April 2024, https://ec8webinars.org/, (accessed September 21, 2024).

[14] Panagiotakos, T. (2024): EC8 Webinars Second Generation of Eurocode, Webinar 2.4: Bridges equipped with antiseismic devices, 5th April 2024, https://ec8webinars.org/, (accessed September 21, 2024).

[15] Mandić Ivanković A., Srbić M., Skokandić D., Friedl P. (2024): Application of Pushover Analysis in Seismic Assessment of 50 years old Bridge. Report of the IABSE Congress San Jose 2024: Beyond Structural Engineering in Changing World. ISBN: 978-3-85748-205-2. Pg 201-208

[16] Isaković T., Fischinger M. (2009): Simplified nonlinear method for the analysis of concrete bridges, *Građevinar,* 61 (7)