EAN 2017
55th conference on experimental stress analysis

Assessment of serviceability criteria in Eurocodes

Markova Jana[1,a] and Holicky Milan [1,b]

[1]Klokner Institute, Czech Technical University in Prague, Czech Republic

[a]jana.markova@cvut.cz, [b]milan.holicky@cvut.cz

Keywords: serviceability constraints; reliability assessment; target reliability.

Abstract. The load-bearing capacity, serviceability or durability of a structure designed in accordance with different standards or nationally implemented Eurocodes might be expected within a broad range. The actual structural resistance depends not only on used theoretical models and selected reliability elements for calculation of structural resistance but also on various prescriptive rules recommended in applied current standards including structural detailing. Moreover, in some cases the theoretical models given in various standards for determining structural resistance or serviceability provide considerably different probability of over-crossing the specified design value

Introduction

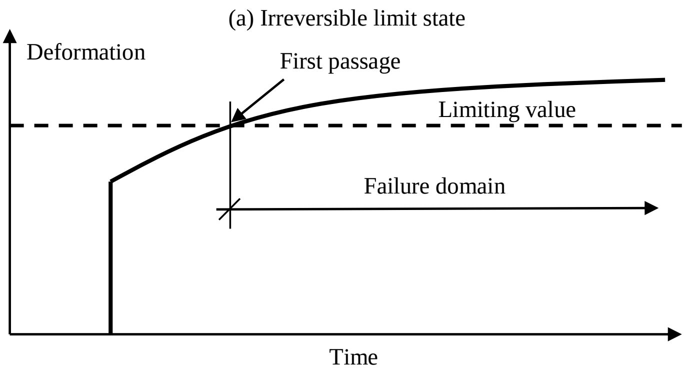
Construction works are designed using methods provided in national, European or international standards. The Eurocodes for structural design allow the national selection of more than 1500 Nationally Determined Parameters (NDPs) including alternative design approaches, combinations of actions, values of partial factors and other reliability elements and also serviceability constraints. The actual reliability of a designed structure depends on applied national standards or selected NDPs in the National annexes to Eurocodes of the CEN Member countries.

The reliability of structural members designed for the serviceability limit states according to current National Annexes to Eurocodes may have a considerable scatter and during the current evolution of Eurocodes should be further harmonised.

Serviceability limit states

The serviceability limit states concern the functioning of a structure or structural members, comfort of people and appearance of the construction works. Taking into account the time dependency of load effects, two types of serviceability limit states should be distinguished, as illustrated in Figure 1.

- Irreversible serviceability limit states (see Figure 1(a)), which are those limit states that remain permanently exceeded even when the actions that caused the infringement are removed; the failure domain is the total time following the first passage of the limiting value.
- Reversible serviceability limit states (see Figure 1(b)), which will not be exceeded when the actions which caused the infringement are removed; the failure domain consists of all parts where the response is above the limiting value.



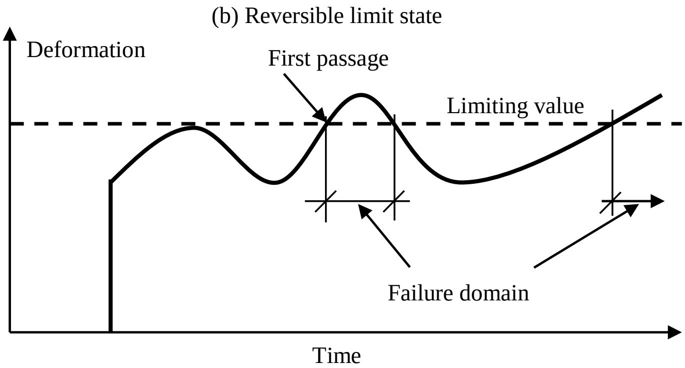


Figure 1: Irreversible and reversible limit states

For irreversible limit states the design criteria are similar to those of ultimate limit states, but with reduced reliability. The first passage of the limit state is decisive. For reversible limit states the first infringement (first passage) does not necessarily lead to the loss of serviceability.

Various serviceability requirements can be formulated taking into account the acceptance of infringements, their frequency and their duration. Generally, three types of serviceability limit states are applicable as follows:

- no infringement is accepted,
- specified duration and frequency of infringements are accepted,
- specified long-term infringement is accepted.

The serviceability criteria are then associated as appropriate with the characteristic, frequent and quasi-permanent values of variable actions.

What must be verified is that the design values of the actions' effects, E_d, specified via the criterion in question and calculated on the basis of suitable combinations, remain lower than the corresponding design limit values C_d of the relevant serviceability criterion

$$E_d \leq C_d \quad (1)$$

Conceptually, all such serviceability limit state checks conform to this general relation, though, they may concern aspects that are quite distinct one from the other, such as limitations to deflections, or crack widths in reinforced concrete structures.

Presently the recommended serviceability constrains are not provided in EN 1990 [5] for basis of structural design. It is expected that selected recommendations will be given in new, revised EN 1990. Limiting design values of vertical deflections based on several National Annexes to Eurocodes of the CEN Member countries, and also currently recommended in the latest draft of new EN 1990 by the Subcommittee CEN/TC 250/SC10 are given in Table 1, for natural frequencies in Table 2 and for crack widths limits in Table 3.

Table 1. Recommended limiting values of deflections for selected structures

Structures	National Annexes	EN 1990
Roofs non-accessible	$w_{max} \leq L/200$ up to $\leq L/300$	rigid roofing: $w_2+w_3 \leq L/250$ resilient roofing: $w_2+w_3 \leq L/125$
Roofs accessible	$w_{max} \leq L/250$ up to $\leq L/300$	partition walls: - with openings: $w_2+w_3 \leq L/1000$ - no opening: $w_2+w_3 \leq L/500$ reinforced walls: $w_2+w_3 \leq L/350$ removable walls: $w_2+w_3 \leq L/250$
Floors	$w_{max} \leq 15$ up to 28 mm; $L/400$; $L/250$ $L/400$ main girders; supporting plasters $L/250$; brittle materials 15 mm	
Structures supporting crane runways	$w_{max} \leq L/600$ up to 700; $w_{max} \leq 25$ mm	$w_{max} \leq L/600$; $w_{max} \leq 25$ mm
Appearance	$w_{max} \leq L/250$	$w_{max} \leq L/250$

For the serviceability limit state of a structure not to be exceeded when subjected to vibrations, the natural frequency of vibrations of the structure should be kept above appropriate values which depend upon the function of the building and the source of the vibration, and agreed with the client or the relevant authority.

Table 2. Appropriate values of natural frequencies for selected structures

Structures	National Annexes	MC 2010	EN 1990
Gymnasia and sports halls	> 3 up to > 6,0 Hz	8	> 8,0 Hz
Concert halls without permanent seats	> 8,0 Hz	7	> 7,0 Hz
Concert halls with permanent seats	> 8,0 Hz	3,4	> 5 Hz
Floors and staircases of public buildings	> 5 up to > 6 Hz		> 5 Hz
Floors of residential or office buildings	> 3 up to > 5 Hz	1,4-4; 4	> 3 Hz

Recommended crack width limits provided in EN 1992-1-1 are given in Table 3.

Table 3. Crack width limits in EN 1992-1-1 [5]

Exposure class	Quasi-permanent combination	Frequent load combination
X0, XC1	0,4 mm	0,2 mm
XC2, XC3, XC4	0,3 mm	0,2 mm
XD1, XD2, XS1, XS2, XS3		Decompression

MC 2010 [9] recommends for reinforced concrete members in exposure classes XC, XD, XF, XS the crack width limit w_{lim} = 0,3 mm under the quasi-permanent combination of actions.

Probabilistic verification of structural reliability

The knowledge of the reliability level of the structure designed according to the national standards or nationally implemented Eurocodes and also the reliability (credibility) of prescriptive analytical models or serviceability constrains in standards can be used for optimisation of design procedures or for further harmonisation of standards. The structural member or theoretical model may be considered as reliable, if the condition $p_F < p_t$ (or $\beta > \beta_t$) is satisfied where the probability of failure p_F is given as

$$P_f = P\{g(\boldsymbol{X}) < 0\} = \int_{g(\boldsymbol{X})<0} \varphi_X(\boldsymbol{x})\,d\boldsymbol{x} . \qquad (2)$$

The failure probability p_F may be expressed by reliability index $\beta = -\Phi^{-1}(p_F)$, where Φ is the distribution function of standardised normal variable. The probability of failure p_t and reliability index β_t are the specified (target) values that should not be exceeded during the intended reference period.

The reliability differentiation of structures in EN 1990 [4] is based on three different levels of failure consequences with respect to the ultimate limit states (consequence classes CC1 to CC3). However, for the serviceability limit states similar differentiation is not provided yet. In some cases this differentiation of structures in serviceability limit states might also be useful for distinguishing the severity of potential consequences. Presently EN 1990 [4] recommends for the reversible limit states the target reliability index $\beta_t = 0$ and for the irreversible serviceability limit states $\beta_t = 1{,}5$ (for the fifty years reference period).

Some further recommendations for the target values of reliability indices in the serviceability limit states are given in the JCSS Probabilistic Model Code [6] where the target indices β_t are recommended in a range from 1,3 to 1,7 for CC1 to CC3.

The reliability analysis of structural members for the ultimate or serviceability limit states can be determined through the probability p_{F1} of the action effects $E(\boldsymbol{X})$ randomly exceeding the structural resistance $R(\boldsymbol{X})$ according to the following relationship

$$p_{F1} = P\{(\xi_R R(\boldsymbol{X}) - \xi_E E(\boldsymbol{X})) < 0\}. \qquad (3)$$

where $\boldsymbol{X}$ is a vector of basic variables and ξ_R and ξ_E are the model uncertainties of a resistance and action effects.

The reliability (credibility) of theoretical models given in current standards can be examined by means of the credibility of specified design value $v_d(\boldsymbol{x}_d)$ (e.g. design stress, design deflection, design crack width) based on the vector of design variables. The probability p_{F2} of exceeding the design value $v_d(\boldsymbol{x}_d)$, which was determined according to relevant theoretical formulae and recommendations of prescriptive design procedure, might be analysed as

$$p_{F2} = P\{(v_d(\boldsymbol{x}_d) - \xi_E v(\boldsymbol{X})) < 0\}. \qquad (4)$$

where $\boldsymbol{X}$ is a vector of random basic variables and ξ_E represents the uncertainties of the effects of basic variables and model uncertainties in the considered limit state.

The serviceability requirements in the limit states of crack width are analysed for an example of a reinforced concrete member as follows.

Verification of structural reliability with respect to crack width

The time-independent reliability analysis of a reinforced concrete slab for the limit state of crack width is dealing with the probability p_{F1} of the random crack width $w(X)$ over-crossing the required constraint w_{lim} expressed by

$$p_{F1} = P\{(v_{lim}(\boldsymbol{x}_d) - \xi_E v(\boldsymbol{X})) < 0\}. \qquad (5)$$

where $\boldsymbol{X}$ is a vector of basic variables and ξ_E a model uncertainty for action effects.

The probabilistic models of basic variables entering the equation (5) are based on recommendations of JCSS and reliability analyses developed in the Klokner Institute CTU, and they are listed in contributions [7,8].

The theoretical models assume different probabilities of exceeding the characteristic value of crack width, or maximum crack spacing. The probability of over-crossing the characteristic crack width w_k is 5 % according to Eurocodes, MC 2010 [9] and CSN 73 1201 [3], 10 % in CEP FIP Model Code 1990, 20 % in BS 8110 [2].

The results of reliability analysis of the reinforced concrete slab (height from 0,19 to 0,29 m) determined using the software Comrel are illustrated in Figure 2.

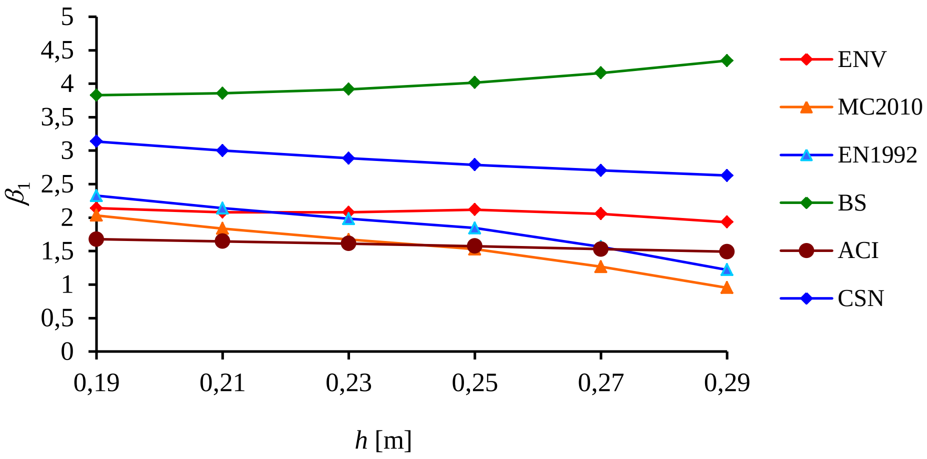


Figure 2: Reliability of a reinforced concrete slab for the limit states of crack width

The results of reliability analyses show that the reliability of structural member depends on used theoretical models for crack width. The reliability index β_1 is in a broad range from 1 to 4,5. The reliability of the slab seems to be rather high according to British (BS) and also Czech (CSN) national standards.

The credibility (reliability) of the specified characteristic value of crack width w_k is also verified. The probability p_{F2} of over-crossing the crack width w_k according to relationship (4) for the slab expressed here by reliability index β_2 is illustrated in Figure 3.

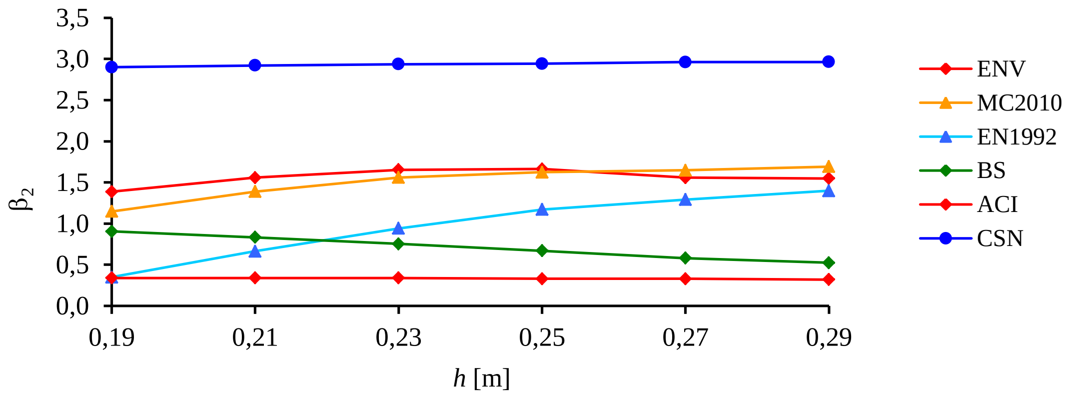


Figure 3: Credibility of specified characteristic value of crack width w_k

Analysis of the credibility of specified crack width w_k indicates that the reliability index β_2 appears to be low for the theoretical models introduced in national American and British standards and rather high in Czech standard (about 2,9). The credibility of theoretical models seems to be sufficient in Eurocodes and Model Code 2010 [9], satisfying the target reliability index β_t = 1,5.

Concluding remarks

Presently the basic Eurocode EN 1990 introduces general recommendations for the verification of the serviceability limit states only. The development of supplementary provisions is needed including classification of structures based on the consequences of failure. An example of verification of a reinforced concrete slab with respect to the limit state of crack width indicates that the same limiting serviceability constraint (crack width limit) is compared with characteristic values of crack width obtained on the basis of a broad range of normative recommendations and also having different statistical meaning.

Reliability indices β assessed in the analysis of the credibility of the analytical crack width formulae and the reliability of reinforced concrete slab with respect to limit crack width have a significant scatter and in some cases seem to be rather low.

It appears that the probabilistic methods can be effectively used for the calibration of serviceability constrains used in design or verification of structures. They might be applied for further harmonisation of national standards or parameters NDPs in the second generation of Eurocodes.

Acknowledgments

This is an outcome of the research projects 16-04132S supported by the Czech Science Foundation and LD15037 supported by the Ministry of Education.

References

[1] ACI 318-89. 1989. Manual of Concrete Practice, reported by ACI Committee 301. American Concrete Institute. Detroit

[2] BS 8110. 1989. Structural Use of Concrete. Part 2: Code of Practice for Design and Construction, British Standards Institution.

[3] ČSN 73 1201. 1986. Design of Concrete Structures and Amendments.

[4] EN 1990. 2002. Basis of Structural Design, CEN

[5] EN 1992-1-1. 2004. Design of Concrete Structures - Part 1: General Rules and Rules for Buildings, CEN

[6] JCSS Probabilistic Model Code. 2002.

[7] Marková J., Holický M. 2000. *Probabilistic Analysis of Crack Width*. Acta Polytechnica. Vol. 40. No. 2/2000, Prague, Czech Republic. pp. 56-60

[8] Marková J., Holický M. 2009. *Probabilistic Assessment of Crack Width for Existing Bridges*, in Esrel 2009, Prague, Czech Republic

[9] Model Code 2010, Design Code, fib, 2010