Einführung des EC2 in Belgien
The introduction of EC2 in Belgium

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The draft European Standard ENV 1992-1-1 has been published as Belgian Standard NBN B15-002 already in 1999 together with the National Application Document (NAD). Hence Belgian designers have acquainted an almost 10 year experience with the basic concepts and design guidelines which also appear in EN 1992-1-1 (Part 1 of Eurocode 2). In the national annex of NBN EN 1992-1-1 most of the recommended values of the NDP's have been adopted. In the following, a survey is given of the main clauses where specific choices have been made which deviate from the recommended values. Also some personal reflections on particular design approaches in EC2 are presented.

1 Material characteristics

1.1 Concrete

The design value of the compressive concrete strength is given by

$$f_{cd} = \alpha_{cc} \frac{f_{ck}}{\gamma_c} \tag{1}$$

whereas before (ENV 1992-1-1) f_{cd} was defined as f_{ck}/γ_c. The coefficient α_{cc} takes into account long term effects on the compressive strength and unfavourable effects resulting from the way the load is applied. The value of α_{cc} should be between 0.8 and 1.0 with 1.0 as recommended value. In the well-know parabola-rectangular diagram, which is used for the design of cross-sections in the ULS, the maximum design stress is f_{cd} whereas in ENV 1992-1-1 it is equal to 0.85 f_{cd}. When the recommended value α_{cc} = 1.0 is used, the maximum design stress is increased from 0.85 f_{ck}/γ_c to 1.00 f_{ck}/γ_c or an increase by 17.6 %. In Belgium, it was deemed that there was no fundamental reason to increase the design compressive strength in the ULS and hence in the National Annex (NA) α_{cc} = 0.85, which yields the same results as a design according ENV 1992-1-1. The coefficient 0.85 is based on tests by H. Rüsch [1]. In fig. 1 it can be seen that, for these particular tests, the so-called "failure limit" and "creep limit" tend to a value which is about 80 % of the short term strength f_c. Hence setting the coefficient α_{cc} equal to 0.85, is not really conservative. One could argue that part of the ultimate load is live load and that considering α_{cc} = 0.85 is too conservative. However, in many cases where structural failures occurred due to crushing of concrete, the nature of the direct and indirect actions was mainly sustained.

The design value of the tensile strength is given by

$$f_{ctd} = \alpha_{ct} \frac{f_{ctk\,0,05}}{\gamma_c} \tag{2}$$

with α_{ct} accounting for the same effects as α_{cc}. The recommended value is 1. As there is no fundamental reason to change the design values with respect to ENV 1992-1-1, α_{cc} = 1 in the Belgian NA.

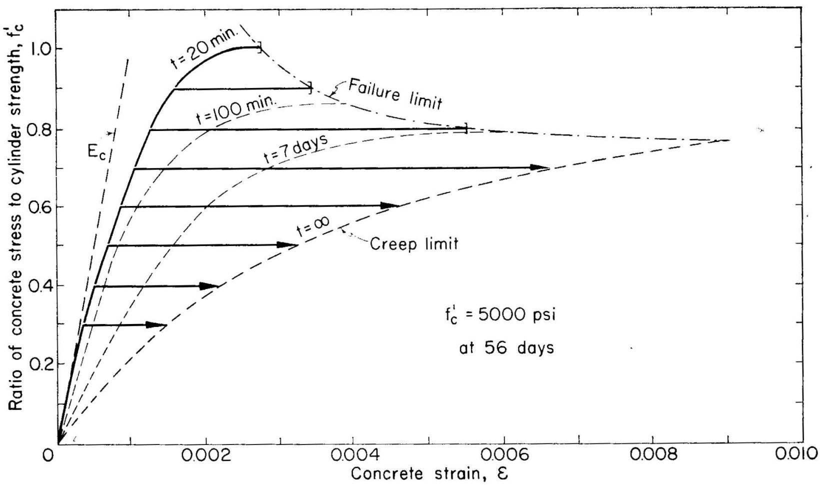


Fig. 1. Influence of sustained loading on the compressive strength of concrete according to Rüsch [1] (age at loading : 28 days)

1.2 Reinforcing steel

The application rules for design and detailing in Eurocode 2 are valid for a specified yield strength f_{yk} ranging between 400 and 600 MPa. In the Belgian NA, the upper value is reduced to 500 MPa as no experience is available for steels with higher yield strengths.

In the design stress-strain diagram (fig. 3.8 in EN 1992-1-1), the steel strain is limited to $\varepsilon_{ud} = 0.9\ \varepsilon_{uk}$ as recommended value. In the Belgian NA, this limit is considered both for the diagram with an inclined second branch as for the diagram with a horizontal second branch. In EN 1992-1-1, no ultimate tensile strain is considered in the latter case. When the steel class is not known, the value $\varepsilon_{uk} = 2.5$ % is used which corresponds to class A according to annex C of EN 1992-1-1. In order to unify design charts and software applications a value of $\varepsilon_{ud} = 10$ ‰ is suggested.

Also for prestressing steel (fig. 3.10 in EN 1992-1-1), the steel strain is limited to $0.9\ \varepsilon_{uk}$ for both types of second branches. In case more accurate values are not known, the recommended values in the Belgian NA are $f_{p0.1k}/f_{pk} = 0.9$ and $\varepsilon_{uk} = 3.5$ % for bonded prestressing tendons. For unbonded tendons and at deviators, couplers and anchorages, the reduced value $\varepsilon_{uk} = 2.0$ % is recommended. It is suggested to limit the ultimate strain in ULS design to $\varepsilon_{p,t}(x) + 0.01$ whereby $\varepsilon_{p,t}(x)$ corresponds to $P_{m,t}(x)$.

2 Linear elastic analysis with limited redistribution

In the Belgian NA, attention is drawn to the fact that linear elastic analysis (LEA) is a special case of LEA with limited redistribution as it corresponds to $\delta = 1$. The coefficient δ is the ratio of the redistributed moment to the elastic bending moment. In this case, equations (5.10a) and (5.10b) of EN 1992-1-1 become

$$\frac{x}{d} \leq \boldsymbol{0.45} \quad for \quad f_{ck} \leq \boldsymbol{50}\ N\ /\ mm^{\boldsymbol{2}} \tag{3}$$

$$\frac{x}{d} \leq \frac{\boldsymbol{0.37}}{\boldsymbol{0.6} + \boldsymbol{0.0014} / \varepsilon_{cu\,\boldsymbol{2}}} \quad for \quad f_{ck} > \boldsymbol{50}\ N\ /\ mm^{\boldsymbol{2}} \tag{4}$$

The $f_{ck} \leq 50$ N/mm^2 the ultimate strain ε_{cu2} in the parabola-rectangular diagram is equal to 3.5 ‰ whereas for $f_{ck} \leq 50$ N/mm^2 it decreases with increasing f_{ck}.

3 Ultimate limit states

For slabs on a beam grid (continuous edge support), the value of $V_{Rd,c}$ is multiplied by 1.25 according to the Belgian NA. This increase in shear strength was already considered in the Belgian version of ENV 1992-1-1. A similar increase for beam supported slab edges can also be found in former versions of the Belgian Standard for the design of concrete structures.

It was considered that the formulas in wich f_{cd} appears should give the same results as the corresponding formulas in ENV 1992-1-1. Taking into account the difference in definition of f_{cd} as explained above, the value of ν is defined as

$$\nu = \frac{\boldsymbol{0.6}}{\alpha_{cc}} \left[\boldsymbol{1} - \frac{f_{ck}}{\boldsymbol{250}} \right] \tag{5}$$

in which α_{cc} has been introduced. Also the expressions for ν_1 are multiplied by $1/\alpha_{cc}$.

In the Belgian NA, the upper limit of cotθ for compression struts in shear and torsion design is taken equal to 2.0 instead of 2.5, which is in agreement with the Belgian NAD of ENV 1992-1-1. It is deemed that compression struts with a slope corresponding to cotθ = 2.5 are very rarely observed in tests. Moreover, when the design of vertical stirrups is based on cotθ = 2.5, the value of shear reinforcement per unit length would be reduced by a factor of 2.5 corresponding to the classical approach with cotθ = 1. In many cases, minimum values for shear reinforcement will govern the design.

In tests on beams without stirrups, crack inclinations at mid-depth of about 45° are frequently observed as shown in figs. 2 and 3. Also in the classical shear tests performed by Leonhardt [2,3], inclinations of the main shear cracks at mid-depth of beams are not significiantly smaller than 45° as shown in fig. 4.

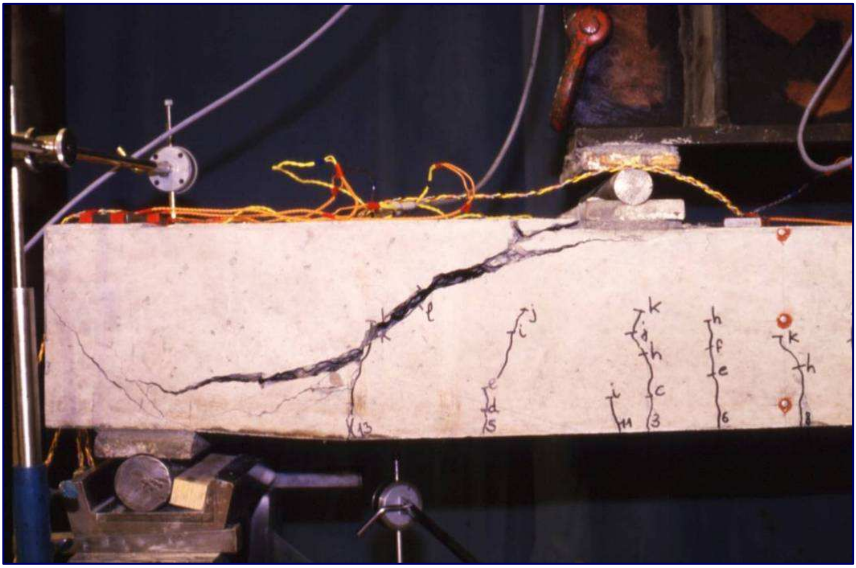


Fig. 2. Shear failure of a concrete beam, without stirrups with rectangular cross-section

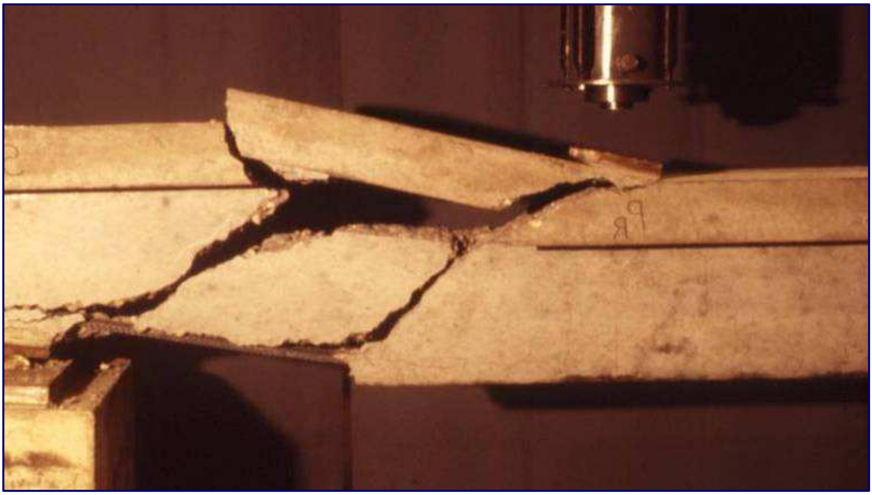


Fig. 3. Shear failure of a concrete beam, without stirrups with T-shaped cross-section

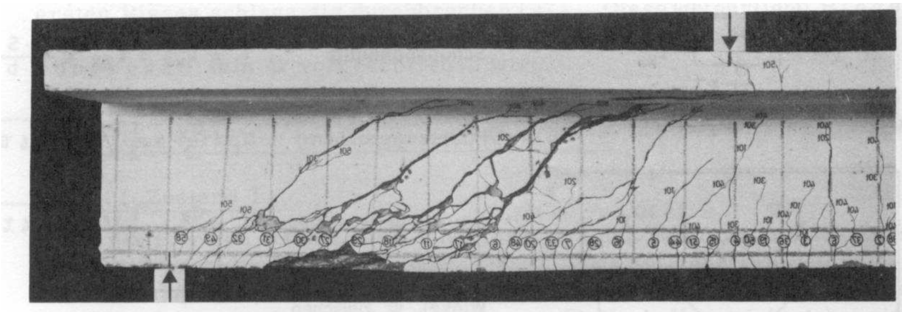


Fig. 4. Shear failure of concrete beam as reported by Leonhardt [2,3]

In EN 1991-1-1 the so-called "standard method" for beams with stirrups is not included anymore. This means that V_{Rdc}, which is the equivalent of V_{Rd1} in ENV 1992-1-1, is not considered for the calculation of shear reinforcement. This should be compensated by chosing a θ value smaller than 45°. However, looking again to the shear tests performed by Leonhardt (fig. 5), it can be seen that the curves with the experimental values of the stirrup stress σ_{sw} are approximately parallel to the dashed line based on the Ritter-Mörsch truss model with θ = 45°. The observed shift which depends on the geometry of the cross-section, corresponds to $V_{Rd,c}$. In the EC2 shear model, the neglect of this contribution is compensated by the choice of a strut inclination θ < 45°. This means that in fig. 5, the dashed line corresponding to θ = 45°, is rotated clockwise with the origin of the diagram as fixed point. Although all the experimental points can be "reached" in this way, this procedure does not correspond to the mechanical behaviour of the beams considered.

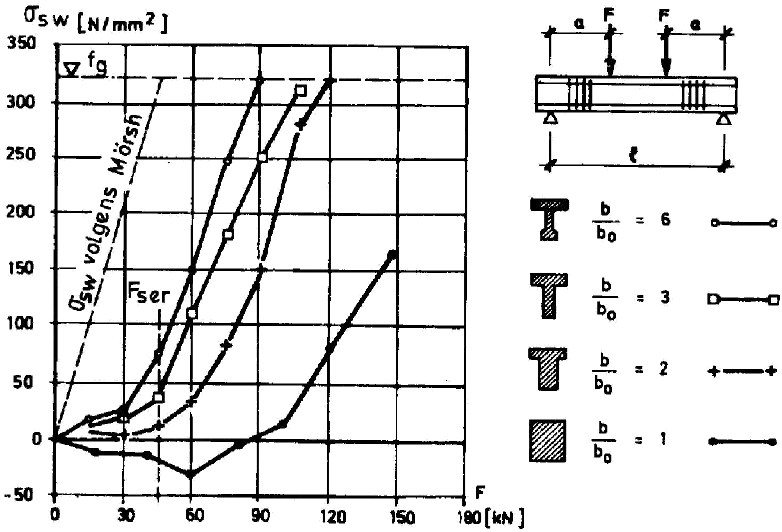


Fig. 5. Measured stress in stirrups according Leonhardt [2]

In a recent paper by A. Cladera and A.R. Mari [4], the new Eurocode 2 shear procedure is discussed. On the one hand the method is very simple to apply for practising engineers but on the other hand it neglects variables which may be of primary importance for some beams. The following Table 1 is taken from [4]. The authors compare the expermimentally observed ultimate shear forces V_{fail} in a total of 122 beams with the shear resistances V_{pred} predicted by different design models. The models considered are:

- the actual EC2 (EC-2, 2003)
- ENV 1992-1- (EC-2, 1991)
- ACI 318-02 Code
- the Canadian draft CSA A23.3-94
- the semi-analytical method proposed by Cladera and Mari [5]

The last procedure is based on a truss model with a variable angle of inclination of the struts plus a concrete contribution. The value of θ depends on the longitudinal strain in the web and the non-dimensional shear.

The results given in Table 1 indicate that for reinforced concrete members with web reinforcement, the mean value of the ratio V_{fail}/V_{pred} is equal to 1.64 for the "new" EC2-model and 1.19 for the "old" EC2-model. The coefficients of variation are 32.34 % and 17.95 % respectively. For the tests considered, the model proposed in [5] performed best.

Table 1. Verification of different shear procedures for reinforced concrete beams with stirrups

Beam specimens	Number	Average V_{fail}/V_{pred}					CoV V_{fail}/V_{pred}				
		EC-2 2003	EC-2 1991	ACI 11-3	CSA 2003	Cladera 2004	EC-2 2003	EC-2 1991	ACI 2003	CSA 2003	Cladera 2004
All	122	1.64	1.19	1.38	1.13	1.06	32.24	17.95	22.25	17.27	15.44
d≥750 mm	9	1.20	1.04	0.97	0.93	1.07	10.04	17.41	18.83	14.15	12.08
$\rho_w f_y \leq 1$ MPa	92	1.80	1.20	1.41	1.15	1.09	27.01	18.48	23.04	17.94	15.37
$\rho_w f_y > 1$ MPa, $\rho_w f_y \leq 2$ MPa	22	1.23	1.18	1.34	1.12	1.02	17.79	14.32	15.78	11.55	10.82
$\rho_w f_y > 2$ MPa	8	0.86	1.06	1.13	0.95	0.88	12.54	18.92	17.28	13.41	13.11
$f_c \leq 70$ MPa	73	1.64	1.14	1.35	1.09	1.04	30.14	14.47	14.16	19.61	12.11
$F_c > 70$ MPa	49	1.63	1.26	1.42	1.18	1.09	35.47	20.31	25.26	19.93	18.93
$\rho_l \leq 2$ %	25	1.50	1.03	1.12	1.01	1.09	30.17	13.80	20.23	12.74	11.94

4 Serviceability limit states

In those cases (e.g. crack width and stress limitation) where reference is made to the exposure classes as defined in EN 206-1, also the "environmental classes" as defined in the Belgian NA to EN 206-1 are mentioned. These environmental classes are linked to environments which are relevant and clearly identifiable for practical applications. The correspondance between both types of classes is shown in Table 2.

In the Belgian NAD of ENV 1992-1-1, it was required that under the characteristic combination of loads, compressive stresses in concrete should not exceed 0.5 f_{ck} for all exposure classes. In the Belgian NA to EN1992-1-1 this requirement is somewhat relaxed. The stress limit is increased up to 0.6

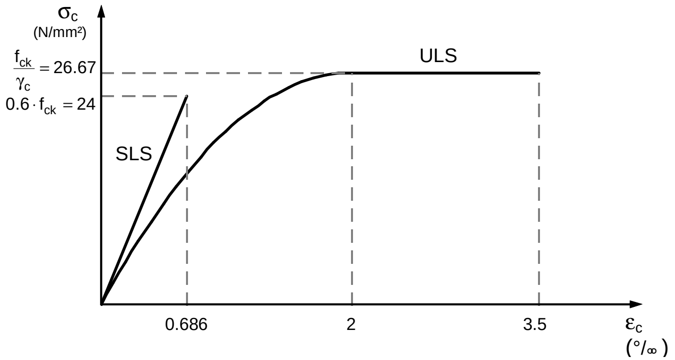
f_{ck} except for exposure classes XD, XF and XS where the limit is kept at 0.5 f_{ck}.

It is instructive to compare the design stress-strain curves for verification of ULS and SLS. In fig. 6 a comparison is made of design stress-strain diagrams for the ultimate limit states (ULS) and the serviceability limit states (SLS) for a concrete

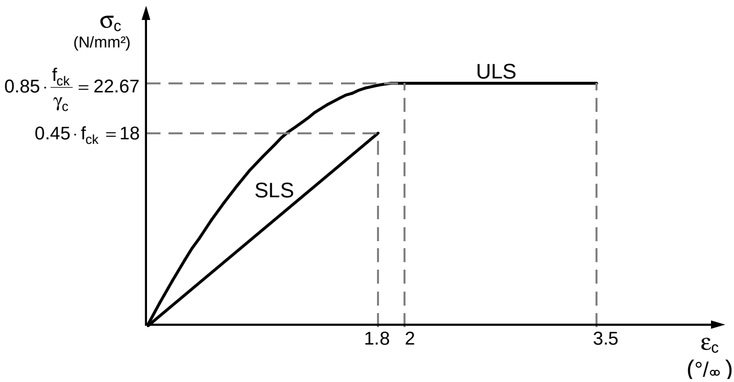
Table 2. Correspondance between exposure classes and environmental classes according to Belgian national annex to EN 206-1

Environmental classes		Exposure classes	
Designation	Description of environment	Plain concrete	RC and PC
E0	**non aggressive**	X0	n.a.
EI	**interior environment**	X0	XC1
EE	**exterior environment**		
EE1	no frost	X0	XC2
EE2	frost, no contact with rain	XF1	XC3, XF1
EE3	frost, contact with rain	XF1	XC4, XF1
EE4	frost and deicing salts (presence of water containing deicing salts)	XF4	XC4, XD3, XF4
ES	**Marine environment**		
ES1	no frost and no direct contact with sea water	XA1	XC2, XS2, XA1
ES2	frost and no direct contact with sea water	XF1	XC4, XS1, XF1
ES3	submerged in sea water	XA1	XC1, XS2, XA1
ES4	tidal and splash zone	XF4, XA1	XC4, XS3, SF4, XA1
EA	**aggressive environment**		
EA1	weak aggressive environment	XA1	XA1
EA2	moderate aggressive environment	XA2	XA2
EA3	severe aggressive environment	XA3	XA3

with f_{ck} = 40 N/mm^2. Fig. 6a is based on α_{cc} = 1 for the ULS and E_c = 35000 N/mm^2 and $\sigma_c \leq 0.6\ f_{ck}$ for the SLS. Fig. 6b is based on α_{cc} = 0.85 and $\sigma_c \leq$ 0.45 f_{ck} for the SLS whereby in the latter case the strains ε_c are multiplied by (1+φ) with the creep coefficient φ = 2.5. One can see that in the first case, there is only a small difference of 2.67 N/mm^2 between the maximum stress levels, although the load levels are quite different. Combining α_{cc} = 0.85 in the ULS with the limitation 0.6 f_{ck} in the SLS, it is found that the maximum allowed stress level in the SLS (24.0 N/mm^2) is higher than the maximum stress in the ULS (22.67 N/mm^2) which is quite a strange situation.



(a)



(b)

Fig. 6. Comparison of design stress-strain diagrams for SLS and ULS under short-term loading (a) and long-term loading (b)

5 Conclusions

In Belgium, ENV 1992-1-1 was already adopted as standard since 1999, together with a National Application Document (NAD).

The main deviations with respect to the recommended values in EN 1992-1-1 which appear in the Belgian National Annex (NA) are related to

- the design values of compressive and tensile strength of concrete where $\alpha_{cc} = 0.85$ and $\alpha_{ct} = 1.00$
- the limitation of the ultime strain in the stress-strain design diagram for reinforcing and prestressing steel
- the values of f_{cd} and cotgθ in ULS design for shear and the value of V_{Rdc} for slabs on a beam grid
- the limitation of the compressive concrete stress under the characteristic combination of actions.

6 References

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