EXPERIENCES WITH THE IMPLEMENTATION OF EUROCODE 2

Bert Ziems[1], Nemetschek Frilo[2]

Abstract

In 2007 the draft version of the German national annex was finished. Nemetschek Frilo was a member of the advisory board. After a trial period organised by the leading German engineer organisations, called “Pilot Project”, the final version of the national annex was published in 2010.

In addition to the Original Eurocode and the German national annex, the national annexes of Austria, the Czech Republic, Belgium, the Netherlands, Italy (NTC) and the UK too were implemented in to Nemetschek Frilo programs for reinforced concrete design. Based on experiences during the implementation of different annexes the subject of the present paper is the comparison of the influence due to the national particularities on the results of design. It focuses on the verifications at the cross sectional level in the Ultimate Limit State (Bending, shear capacity) and the Serviceability Limit State (limitation of stress and crack width).

Keywords: Eurocode, concrete, design, comparisons, bending, shear, crack, stress

1 Introduction

Nemetschek Frilo is developing computer software for structural analysis and design for more than 30 years. With more than 80 applications we cover a wide range of demands in practice. As a subsidiary of the Nemetschek AG, currently active in more than 140 countries, our mission changes more and more to the implementation of international design standards. In this context, we have been concerned already with the Eurocodes for several years now, beginning in the end of the last decade.

In 2007 the draft version of the German national annex was finished, with Friedrich + Lochner GmbH being a member of the advisory board. After a trial period organised by the leading German engineer organisations, called “Pilot Project”, the final version of the national annex was published in 2010.

In addition to the original Eurocode and the German national annex [2], the national annexes of Austria [1], the Czech Republic [6], Belgium [5], the Netherlands [4], UK [3] and Italy [12] too were implemented in our programs for reinforced concrete design.
(Note: Instead of national annex is in Italy a document NTC to be taken into account)

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[1] Bert Ziems , Friedrich+Lochner GmbH01067 DresdenSchweriner Str.25Tel. 0351 87610, info@frilo.com
[2] Nemetschek Frilo, Friedrich+Lochner GmbH01067 DresdenSchweriner Str.25Tel. 0351 876140

Therefore we were able to compare the influence of the national particularities of these seven national annexes on the results of design which shall be the subject of the present paper. It focuses on the verifications at the cross sectional level in the Ultimate Limit State (bending and shear capacity) and the Serviceability Limit State (limitation of stress and crack width). Due to the existence of the worked examples of the European Concrete Platform the basis of the comparisons seems to be at a level of sufficient objectivity.

Other interesting and more complex fields of design, e. g. deflection, punching, the 2nd Order design of columns or fire design demand greater efforts and may be the subject of further investigations. With our software we are able to make a contribution to this.

2 Material

The design results are significantly influenced by the materials and their properties. The concrete strengths from EC2 table 3.1 are used consistently in all considered annexes, only few individual strength classes are added.

C28/35	UK/ IT/ NL
C32/40	UK/ IT
C53/65	NL
C100/115	GER

With exception of Italy, the reinforcing steel grades B500A and B500B are used in all other considered countries. More grades are used in some other countries only.

B450A, B450C	IT
B500C	UK
B550A, B550B	CZ, A
B600A	A

3 Durability

The requirements to ensure sufficient durability result by the classification into exposure classes for reinforcement corrosion (XC, XD, XS) and for concrete attack (XF, XA, XM). The only exception is found in the Belgian NA, where a classification in the less differentiated setting classes EI, EE, ES and EA is used. Due to this classification the minimum strength classes for concrete results, with special definitions in most of the National annexes, differ from the original EC2 table E.1.

A	[1] Tab.9
GER	[2] Tab.E1.D
UK	[3] Tab.NA.2
NL	[4] no requirements
B	[5] NBN 15-001
CZ	[6] Tab. E1.CZ
IT	[7] Tab. C4.1.IV

In dependency of classification into exposure classes too results the minimum concrete cover of the reinforcement. For a usage duration of 50 years applies to the original Eurocode values appropriate table 4.4N with requirement class S4. (CZ, NL annexes coincide with table 4.4N, others with more or less diverging own tables).

A	[1]	Tab.1
GER	[2]	Tab. 4.4DE
UK	[3]	Tab. NA.2
IT	[7]	Tab. 4.1.III
B	[8]	Tab. 4.4ANB

Similarly uneven is the procedure for modified conditions as elevated concrete class, quality control, plate-shaped components or modified usage duration, in the original Eurocode appropriate table 4.3N (CZ, NL annexes coincide with table 4.3N, others with more or less diverging own tables).

A	[1]	chapter 4.4
GER	[2]	Tab. 4.3DE
UK	[3]	Tab. NA.2
IT	[7]	Tab. C4.1.IV
B	[8]	Tab. 4.3ANB

The minimum concrete cover must not be less than the value which results from composite conditions (Table 4.2 and 4.4.1.2. (3)) .The national particularities are minimal for reinforcing steel.

Thus the minimum concrete cover has to be increased by the safety margin according to 4.4 1.3. Values differing from the 10 mm appropriate original EC are found in the NA below:

A	5 mm
GER	10 … 15 mm
NL	5 mm

A comparison of the durability requirements is difficult because of the large national differences and shows uneven results, as can be seen in the worked example No. 4 from [10].

Example:

WE 4.1:	WE 4.3
Interior component, D_s=20 mm, D_b= 8mm	Sea environment, D_s=20 mm, D_b= 8mm

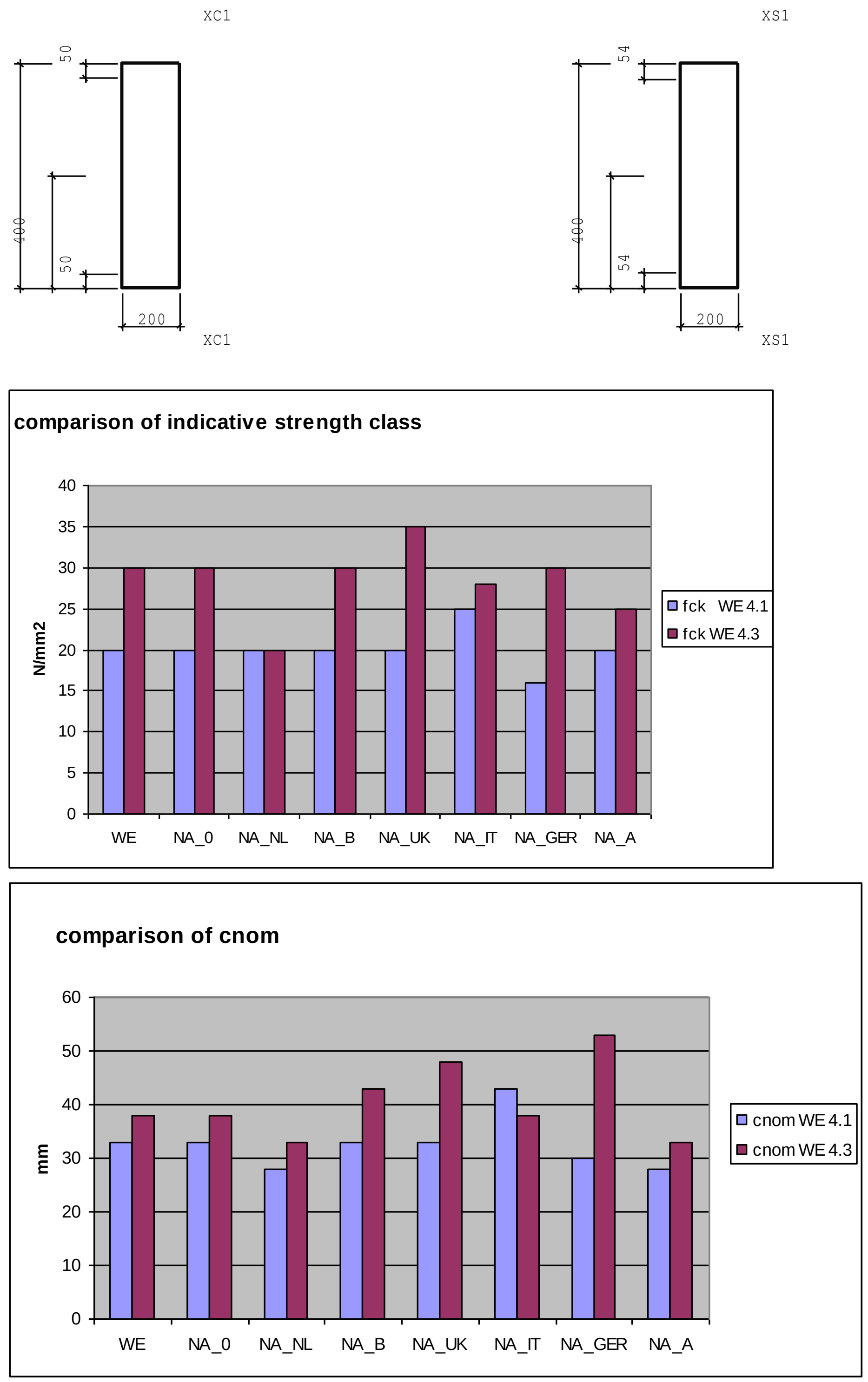


Fig. 1 durability requirements

4 ULS – Design for bending with axial forces

Because of the various durability requirements for concrete cover different lever arms result in the bending design, which can cause differences in the required reinforcement area of 11% in a component with a low height as in the following example.

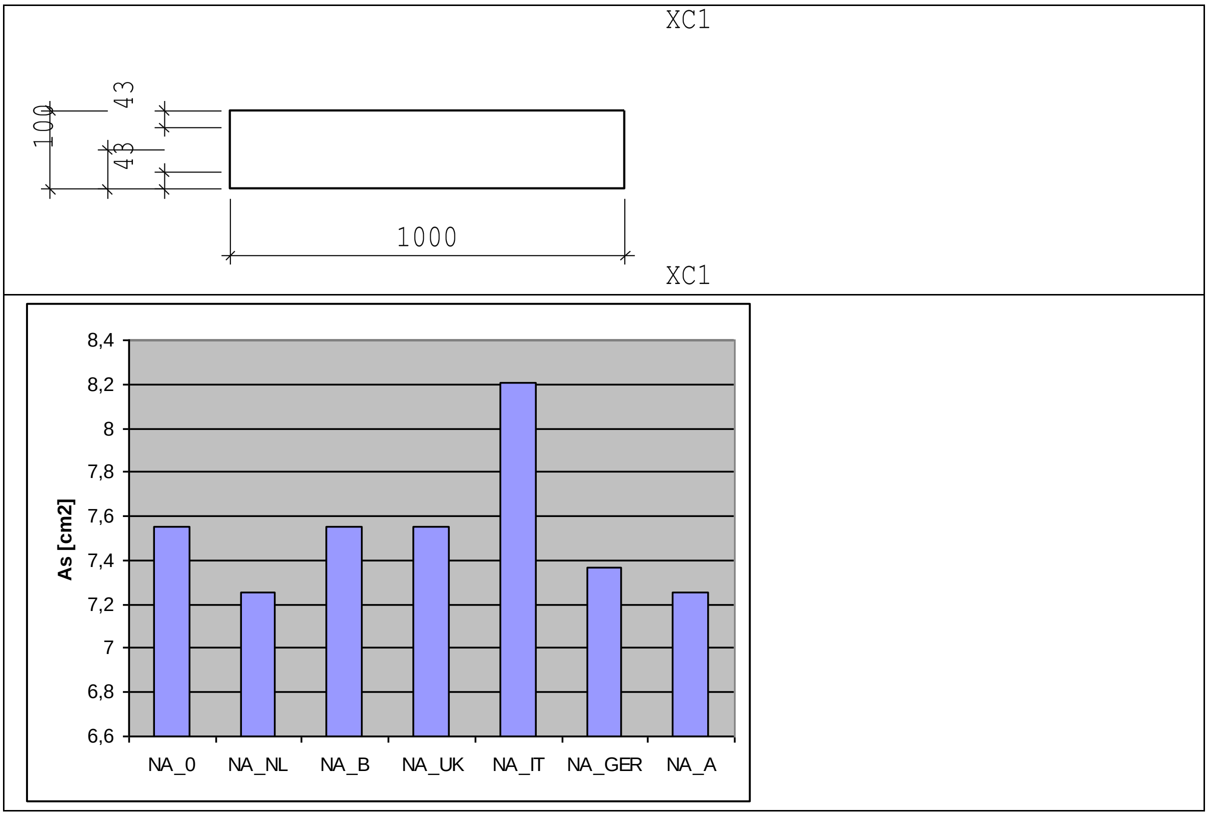


Fig. 2 influence of durability on result of bending design

In the following investigations the differences resulting by durability requirements are not be considered. They also do not address differences in the load combinations, though after our experiences the differences of required reinforcement can reach values up to 15 % under certain unfavorable conditions.

Key parameters for the design work are the stress- strain curves used for concrete and reinforcing steel, which differ in the NA's in several details as shown below.

Stress-strain curve for concrete according Fig. 3

While ε_{c2} and ε_{cu} are treated uniformly according to EC2 table 3.1, there are differences in the peak value $f_{cd} = \alpha_{cc} * f_{ck}/\gamma_c$, because of the nationally defined factor for long time effects α_{cc} and partial safety factor γ_c. The difference in the example below is 15 %. With regard to the different acceptance of reduced safety factors for prefabricated components the deviation increases up to 22 %, comparing for instance the peak values that can be used according to the Italian and the Austrian NA.

α_{cc}	Normal weight concrete 3.1.6	γ_c	Permanent/transient situation 2.4.2.4	Prefabricated component Min. (γ_c,Red4)
EN	1,0	EN	1,5	1,30
GER	0,85	GER	= EN	1,35
UK	0,85	UK	= EN	= EN
A	= EN	A	= EN	= EN
IT	0,85	IT	= EN	1.4
B	0,85	B	=EN	=EN
NL	=EN	NL	=EN	not allowed
CZ	=EN	CZ	=EN	=EN

Tab. 1 Parameters of stress strain curve for concrete

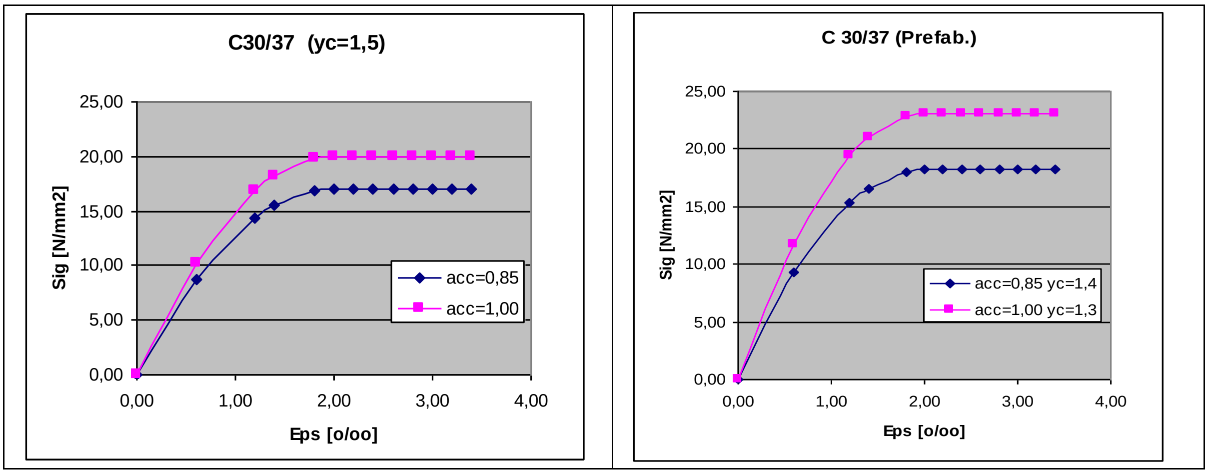


Fig. 3 Stress-strain curves for concrete

<u>Stress-strain curve for reinforcement steel:</u>

γ_s	Permanent/transient situation.	prefabricated component Minimum A2.1/A2.2	ε_{ud}	
EN	1,15	1,05	EN	$0{,}9*\varepsilon_{uk}$
GER	= EN	1,15	GER	25 o/oo
UK	= EN	= EN	UK	=EN
A	= EN	= EN	A	=EN
IT	=EN	1,15	IT	=EN
B	=EN	=EN	B	$0{,}8*\varepsilon_{uk}$
NL	=EN	1,15	NL	=EN
CZ	=EN	=EN	CZ	=EN

Tab.2 Parameters of the stress strain curve for reinforcement steel

Because of the uniform young's modulus E_s the yield strain in the curve is also uniform. The same applies to the partial safety factor. A reduction of this factor for prefabricated components is only applicable with different values. The maximum failure strain is different in some annexes too.

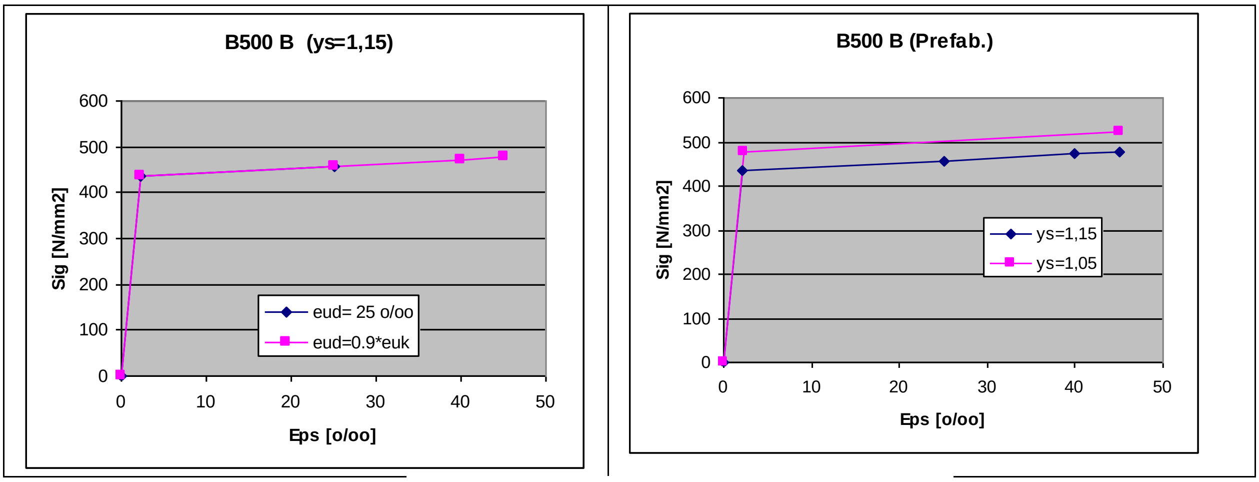
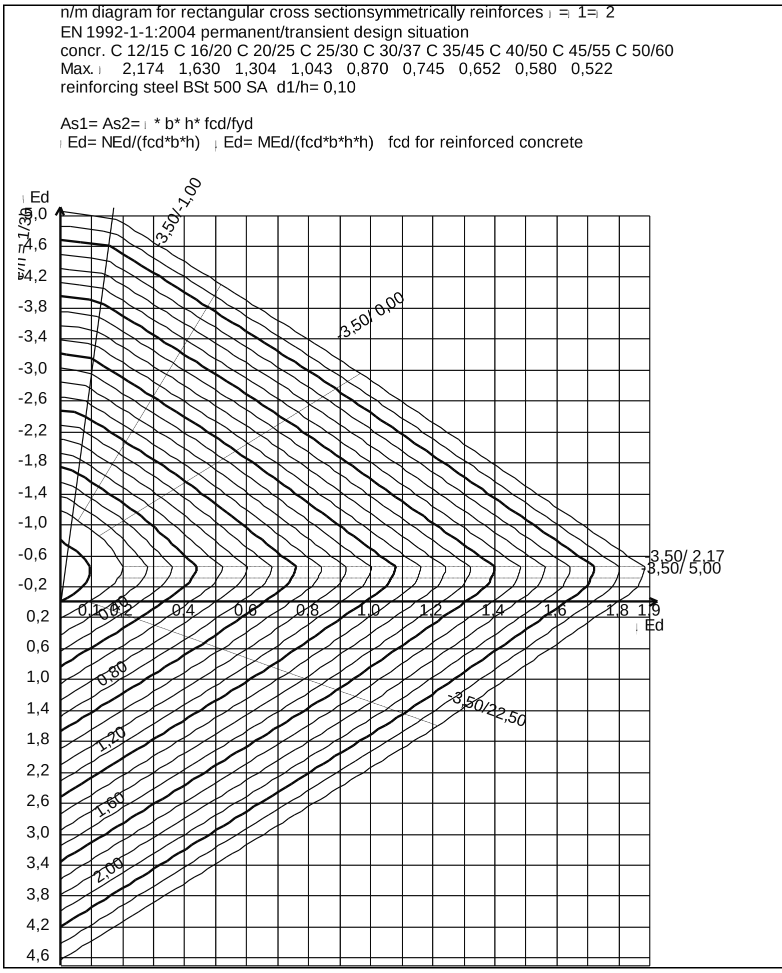


Fig. 4 Stress strain curves for reinforcement steel

As the example shows, in spite of different maximum strains the curve is almost identical. Regarding the acceptance of different safety factors the difference arising in the design value f_{yd} is about 10%.



A descriptive representation showing the differences for the design result arising from the above mentioned may be obtained by comparing design charts. The program "reinforced concrete design-B2" by FRILO provides a function for the production of design charts for any circular and square cross sections, with defined normalized reinforcing distances and curves with the normalized resisting internal forces for different degrees of reinforcement.

Fig. 5 design chart for square cross section

Design charts generated with this function can be found at our homepage www.frilo.com.(concrete: charts for normal strength concrete and for each class of high-strength concrete, cross section: square, circle, circular ring, reinforcement spacing d_1 / h: 0.1, 0.2)

With the help of a special approach in the creation of the diagrams the effect of varying individual parameters can be shown descriptively. By specifying in this case of an non-normalized reinforcement quantity can be compared also variants with different design situations and standards or national annexes with different stress- strain curves. The representation of resisting internal forces in this case is based on the characteristic value f_{ck}.

Example:

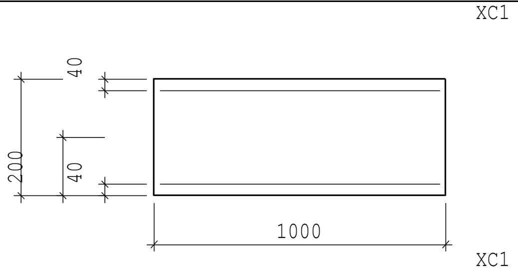
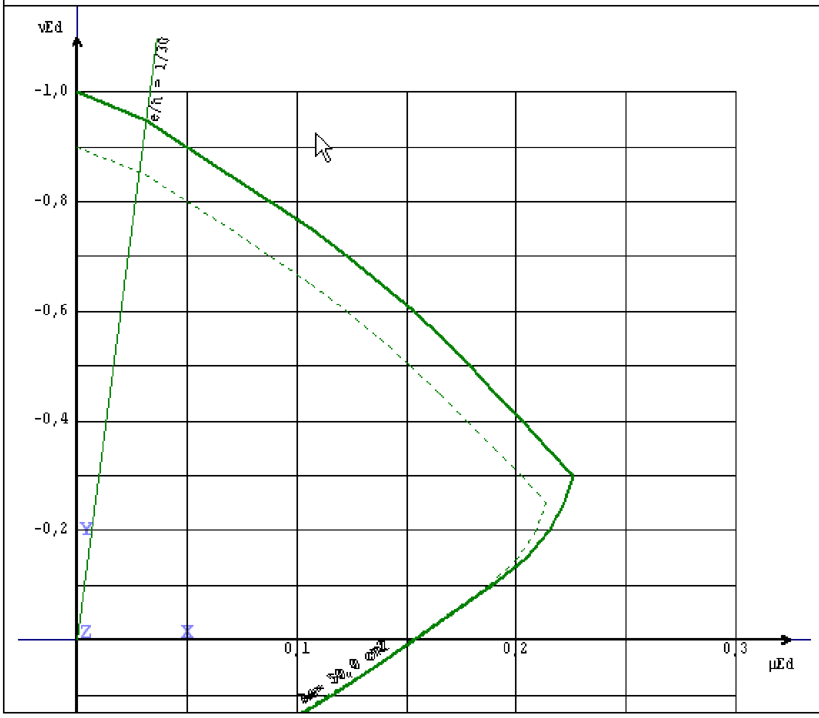
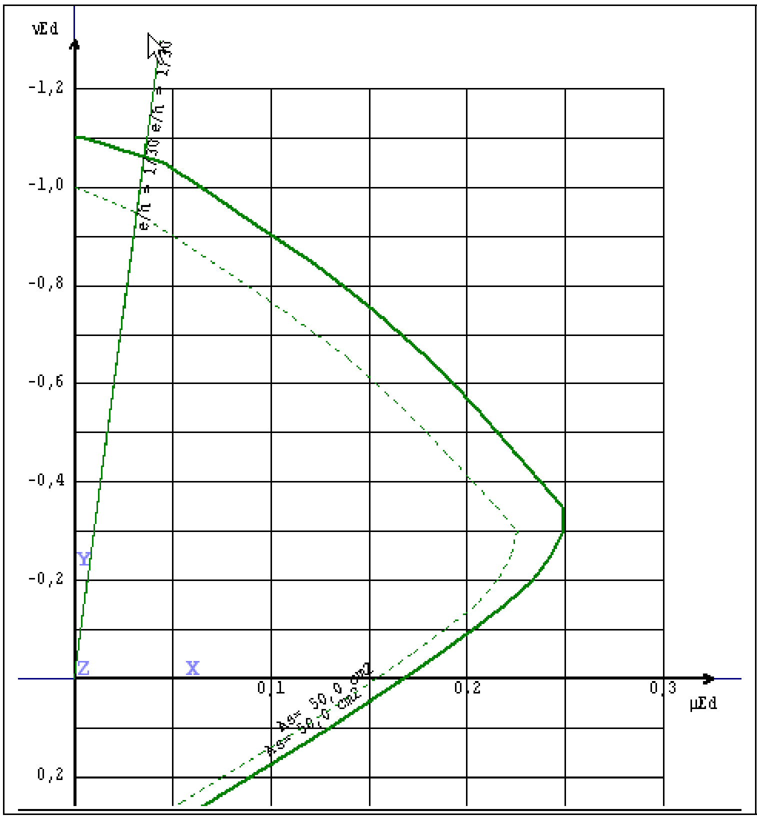
	concrete C30/37 reinforcement B500A A_{s1}= A_{s2}= 50 cm^2
	The required reinforcement with α_{cc}= 1.0 (original code, NA of Austria, the Netherlands, the Czech Republic) is lower than with α_{cc}=0.85 (German, Belgian, Italian and British NA). The largest differences in the bearing capacity occur under high compression stresses

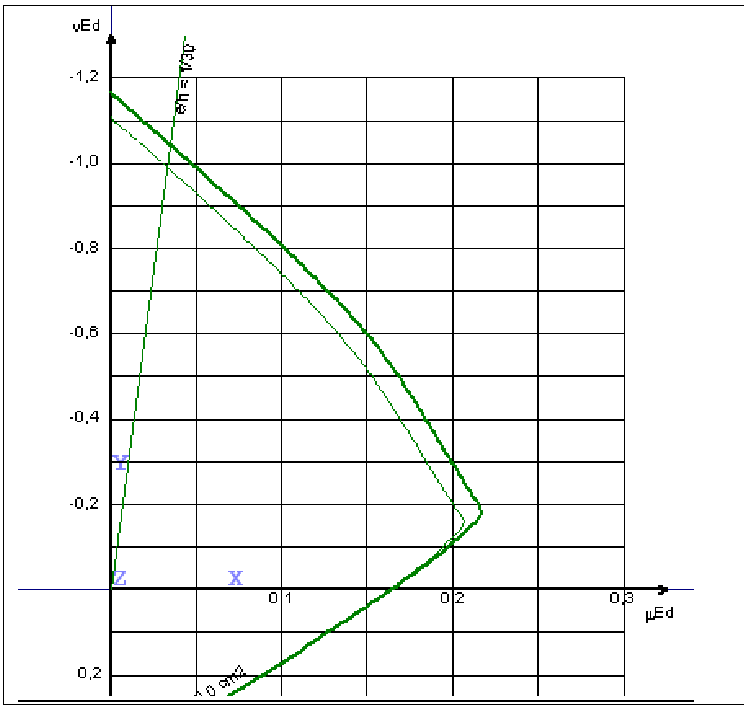
Fig. 6 Deviations caused by different peak values of the used stress strain curves for concrete



The difference is increasing for prefabricated components due to the different national acceptance of reduced material factors for concrete and steel.

In this case a significant difference can also be observed in the range of tension.

Fig. 7 Increasing deviations for prefabricated elements



This comparison for a C90/105 concrete shows, that with high-strength concrete and highly reinforced cross sections the consideration of concrete displaced by the steel component cannot be neglected.

The required reinforcement for the same load bearing capacity with the adoption of this effect is significantly higher!

Fig. 8 High strength concrete and consideration of concrete displaced by steel

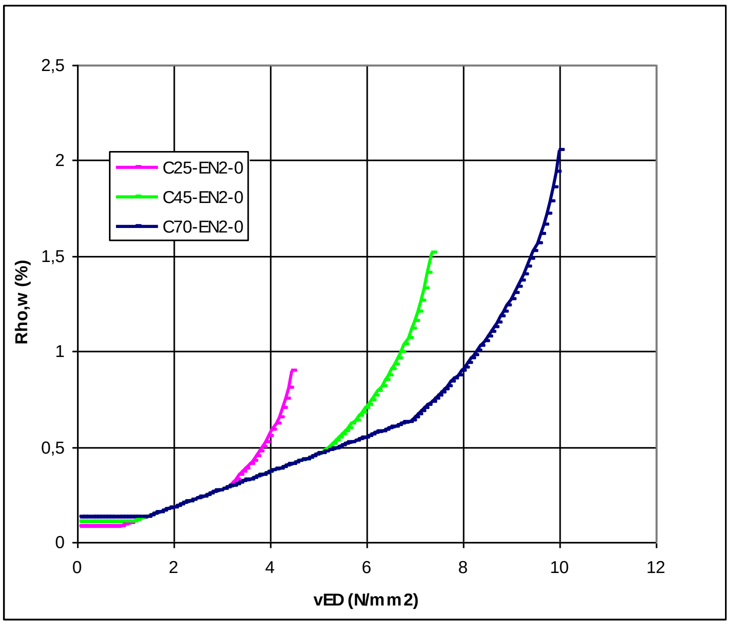
Summary:

In case of a superposition of these effects caused by durability, load combination and material properties may the most favourable result differ from the worst up to 30 %!

5 Shear Resistance

The figure below shows qualitatively the required shear reinforcement with increasing shear force calculated with the recommendations of the original Eurocode. By using the normalized values $v_{Ed} = V_{Ed} / (b_w * z)$ and $\rho_w = A_{sw} / (s * b_w)$ a representation independent of the cross-section is achieved. A consideration of the particularities according the lever arm is not part of this comparison.

Three general sections of the curve can be distinguished, which have different characteristics for the three concrete classes



Section 1 left:	$\rho = \rho$ (Min)= const, while $V_{Ed} <= V_{Rd,s} = A_{sw}/s * z * f_{ywd} * \cot(\Theta_{min})$
Section 2 middle:	ρ linearly increasing, calculated with Θ min while $V_{Ed} <= V_{Rd,max} = \alpha_{cw} * b_w * z * \nu_1 * f_{cd} * (\cot(\Theta_{min}) + \tan(\Theta_{min}))$
Section 3 right:	ρ disproportionately growing, calculated with growing strut inclination, so that $V_{Rd,max}(\Theta) = V_{Ed}$, The maximum shear capacity is achieved when Θ= 45 degrees.

Fig. 9 Required shear reinforcement with increasing shear force

Example:

Now the curves for a concrete C25/30 are compared for different national annexes. A different intensity can be obtained for the three sections. Both, the required reinforcement and the maximum shear resistance result in an uneven picture.

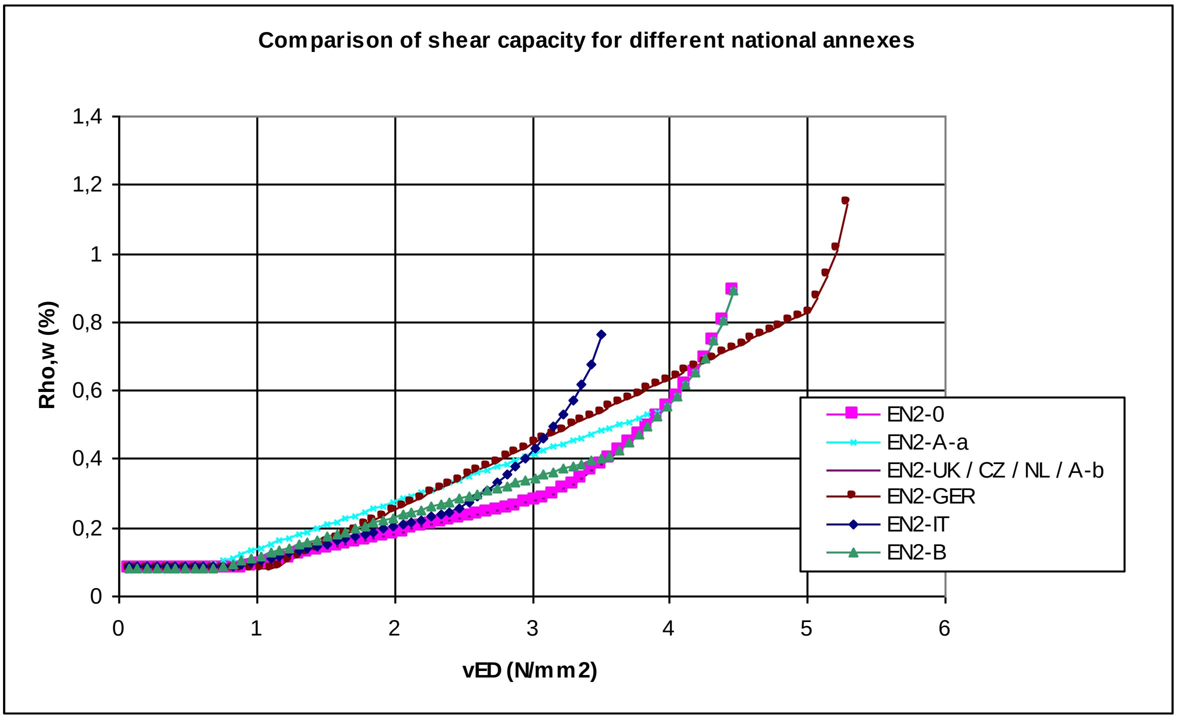


Fig. 10 Comparison of shear capacity between the national annexes

The curves based on the NA of UK, CZ, NL and Austria are identical with the original Eurocode, where the lower limit of strut inclination cot Θ = 2.5, the reduction factor ν_1=0.54 and the design value f_{cd} is calculated with α_{cc}=1.0. Note, that in the UK a design value f_{cd} with α_{cc}=1.0 can be used for shear design [11].

The Austrian national annex states, that the bending reinforcement in the midspan also has to be present at the supports, otherwise the steeper curve EN2-A-a is valid, since the strut inclination is limited to cot Θ = 1.6.

For Belgium the lower limit of strut inclination is higher (cot Θ = 2.0). Similar to the curve EN2-A-a the gradient of curve EN2-B is higher in the middle section.

For Italy (EN2-IT) the required reinforcement is generally slightly higher because of the lower strength of the steel grade B450. The maximum of shear capacity is significantly lower since the use of ν_1=0.5 and α_{cc}=0.85. Note, the factor ν_1 here is independent on concrete strength and for f_{ck} > 40 N/mm2 a higher shear capacity is achieved than with ν_1=0.6* (1- fck/250) appropriate the recommendations in the original code.

For Germany (EN2_GER) the greatest shear capacity is calculated because of ν_1=0.75, but in case of low to medium loading levels higher shear reinforcement is required. This is caused by the German particularity to limit the strut inclination even if $V_{Rd,max}$ is not reached. According the "truss model with crack friction" the inclination angle results from the equation $\cot \Theta = (1.2 + 1.4 * \sigma_{cp}/f_{cd})/(1-VR_{dcc}/V_{Ed})$.

Summary:

The minimum required reinforcement is usually based on original Eurocode with the exception of a narrow section with high stresses, where according the German NA a lower reinforcement is required. For some NA up to 40% higher reinforcements may be required under certain conditions. The maximum of shear capacity based on original Eurocode is 20 % above or below the values of the individual NA's.

6 SLS - Stress Limitation and

The differences between the considered NA's are minimal, for both the permissible values and the methods for stress limitation. Note: still in ENV 1992-1-1 4.4.1.2 (3) a comment recommended to consider long term effects caused by creep with a reduced modulus of elasticity for concrete. Unfortunately this is not to be found in the current version of Eurocode, may be because it is self evident.

7 Limitation of Cracks

	X0, XC1	XC2/XC3/XC4	XS1-3, XD1-3	remarks
EN	0,4mm PC	0,3mm PC	0,3 mm PC	PC = quasi- permanent combination
GER	=EN	=EN	=EN	
UK	0,3 mm PC	=EN	=EN	
A	=EN	=EN	=EN	
I	AO 0,3 mm PC 0,4 mm FC	AA 0,2 mm PC 0,3 mm FC	AM 0,2 mm PC 0,2 mm FC	Assignment is done via classes of aggressiveness AO,AA,AM two verifications with different boundary conditions FC= frequent combination
B	EI 0,4 mm PC	EE1,EE2, EE3 0,3 mm PC	EE4, ES1, 2, 3, 4 0,3 mm PC	assignment is done via environment classes EXX acc. NBN B 15-001
NL	=EN	=EN	=EN	if c > c_{nom} a modified greater permissible crack with can be used
CZ	=EN	=EN	=EN	

Tab. 3 load combination and permissible crack widths

National differences result from the requirements of durability, i.e. the permissible crack width and the specifications of the load combination to be used according EC2 table 7.1. The table above shows the requirements for reinforced members with the special features of Italy, the Netherlands and Belgium.

But this shall not be the subject of further investigation. For that the methodology of the crack with verification will be considered. The formula to determine the crack width is well known.

$$w_k = s_{r,\,max} * (\varepsilon_{sm} - \varepsilon_{cm})$$

While national differences are small in determining the strain difference, different ways to determine the crack spacing exist because of the national parameters k_1, k_2, k_3 and k_4

$$\Delta\varepsilon = (\varepsilon_{sm} - \varepsilon_{cm}) = (\sigma_s - k_t * f_{cteff} / \rho_{p.eff} * (1 + \alpha_e * \rho_{p.eff}))/E_s >= (1 - k_t) * \sigma_s$$

$$s_{r,max} = k_3 * c + k_1 * k_2 * k_4 * \phi / \rho_{eff}$$

Example

To calculate the maximum bar diameter of the reinforcement the formula for crack width has been changed as follows:

$Ds = (w_k/\Delta\varepsilon - s_{r1})/s_{r2}$ *with* $s_{r1} = k_3{*}c$ *and* $.s_{r2} = k_1{*}k_2{*}k_4/\rho_{p.eff}$

Two groups can be obtained. While in one approach the term s_{r1} is assumed to be zero (k_3= 0: Austria, Germany), the others use in practise a value s_{r1} > 3,4* 25 mm = 85 mm depending on to concrete cover (k_3= 3.4: UK, Italy, Netherlands, Belgium, Czech).

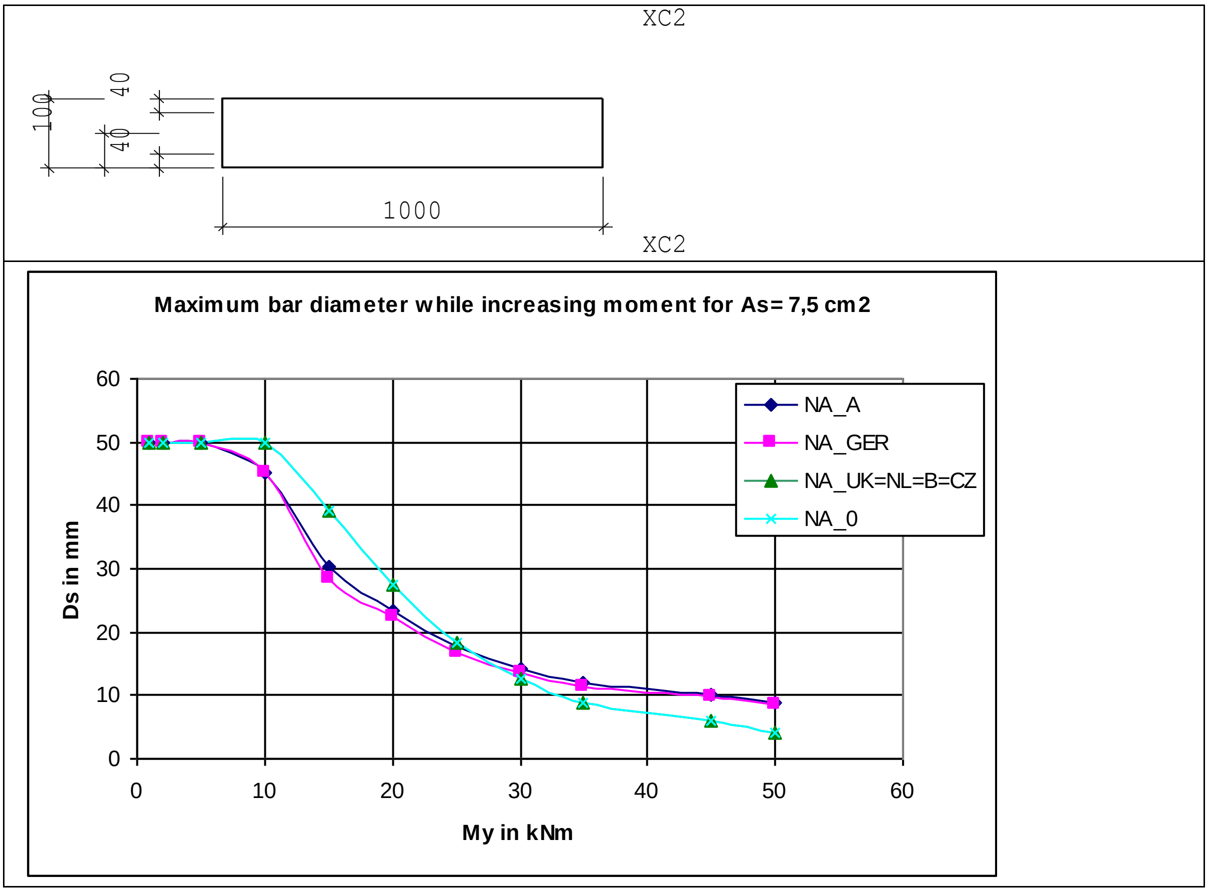


Fig. 11 Maximum bar diameter while increasing moment

Looking at the curve of maximum bar diameter with increasing moment and a defined reinforcement area, results from s_{r1} > 0 compared to calculations with s_{r1}= 0 at low moments a larger permissible diameter, with larger moments, however, a smaller diameter. The reason is the increase of Δε with increasing moments resulting in a greater influence of the term s_{r1}.

For the considered cross-section with a moment My = 50 kNm a hardly realizable boundary diameter of 8 mm is calculated. The following examines how the boundary diameter changes by an increase in reinforcement.

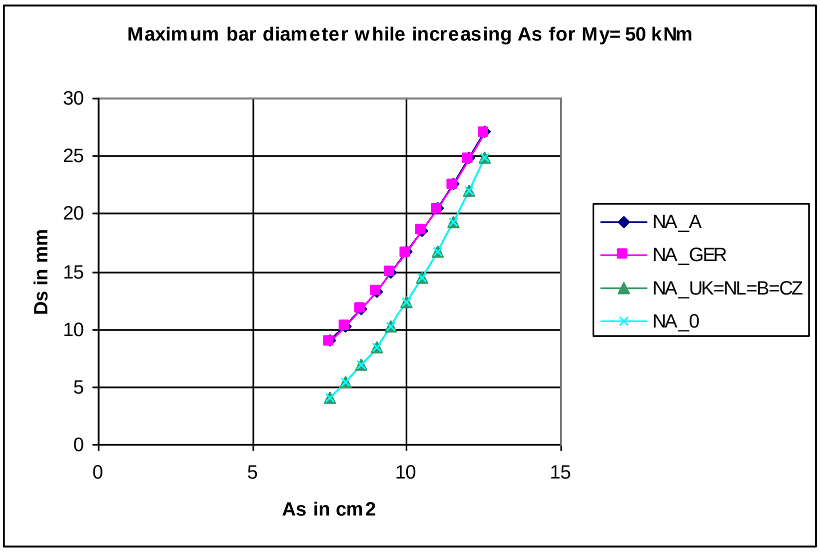


Fig. 12 Maximum bar diameter while increasing As

It can be observed, that with increasing reinforcement the determined diameters according to different NA's converge. This trend can be explained by a decreasing strain difference Δε caused by an increasing reinforcement area and consequently a decreasing influence of the term s_{r1} in the formula for maximum diameter shown above.

Finally the influence of a variation of concrete cover is investigated. Therefore the cross section was changed into a narrower cross section and the cover was varied from 20 mm up to 55 mm, which was assumed as the maximum cover c_{nom} including the allowance Δc_{dev}. That means for the distance of the reinforcement a variation from 30 mm up to 80 mm, assuming a bar diameter of 20 mm.

The result is a strongly increasing difference between that NA's considering the cover in the term s_{r1} and the others with s_{r1}= 0.

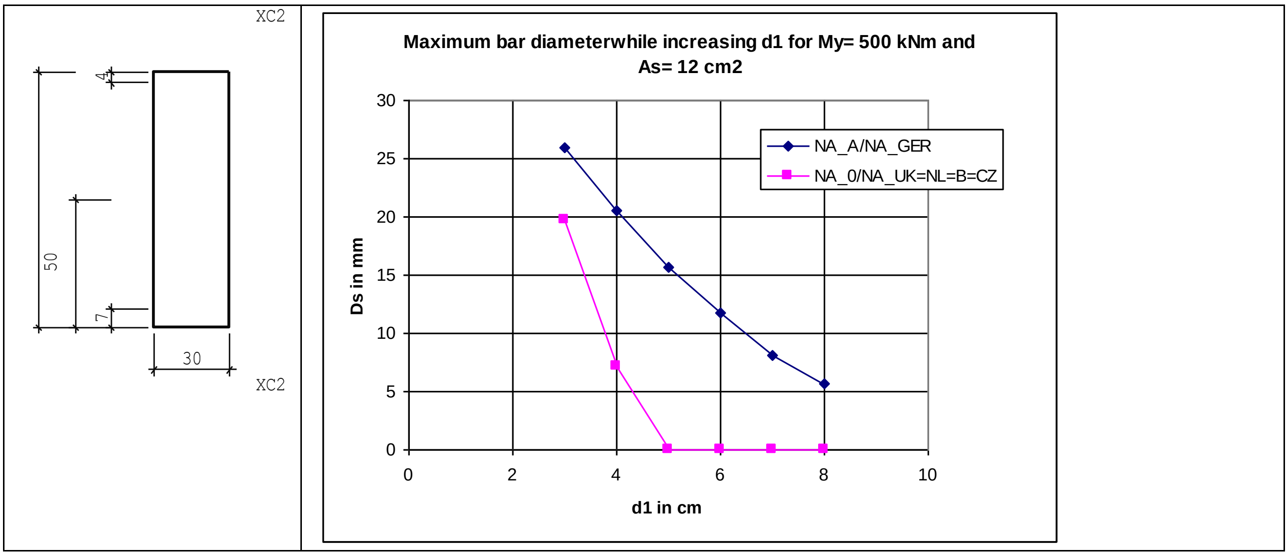


Fig. 13 Maximum bar diameter while increasing distance of reinforcement

Summary:

The differences according crack width limitation can reach under unfavorable conditions (high moment, great concrete cover) up to 50 %.

8 Conclusions

The conducted comparisons at the cross section level show some significant differences, which are caused by quite a few differently defined national parameters. Should this be a reason to be disappointed by more than 20 years Eurocode development? I do not think that we should see it like this. The greatest progress and success brought by the current Eurocode is a common language, a common understanding between engineers and researchers all over Europe. Only this common language allows such solid comparisons between design codes, which will be the basis for a new Eurocode generation with a higher degree of consensus. We have to bear in mind, that some of the differences will remain because they are made of different national security needs and different national experiences in reinforced concrete practice. It is the task of the next years to find out, at what parameters it is possible to dispense with national differences. In my opinion software companies are able to support this process and they should be included in the efforts to prepare the new generation of Eurocodes.

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Organizers of the current workshop in Prague, vicarious Prof. J. Prochazka
My colleagues from SCIA, vicarious A. Bastiaens
My colleagues from FRILO, vicarious J. Hoffmann

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