Identified key issues in document D5

CEN-TC250-SC2_N1706_CDG_WG_1_prEN_1992-1-1_D5_Working_file_2020-05-29_Rev_11

Contents

1. Shear Calculation (eq. 8.13)

The new formula for the shear capacity seems even more restrictive than the previous one

D4: $$\tau_{Rd,c} = \frac{0{,}66}{\gamma_c}\left(100\rho_l \cdot f_{ck} \cdot \frac{d_{dg}}{d}\right)^{1/3} \geq \tau_{Rdc,min}$$

D5: $$\tau_{Rd,c} = C_{Rd,c2}\left(100\rho_l \cdot f_{ck} \cdot \frac{d_{dg}}{d}\right)^{\frac{1}{3}} \geq \tau_{Rdc,min}$$

With $C_{Rd,c2}$:

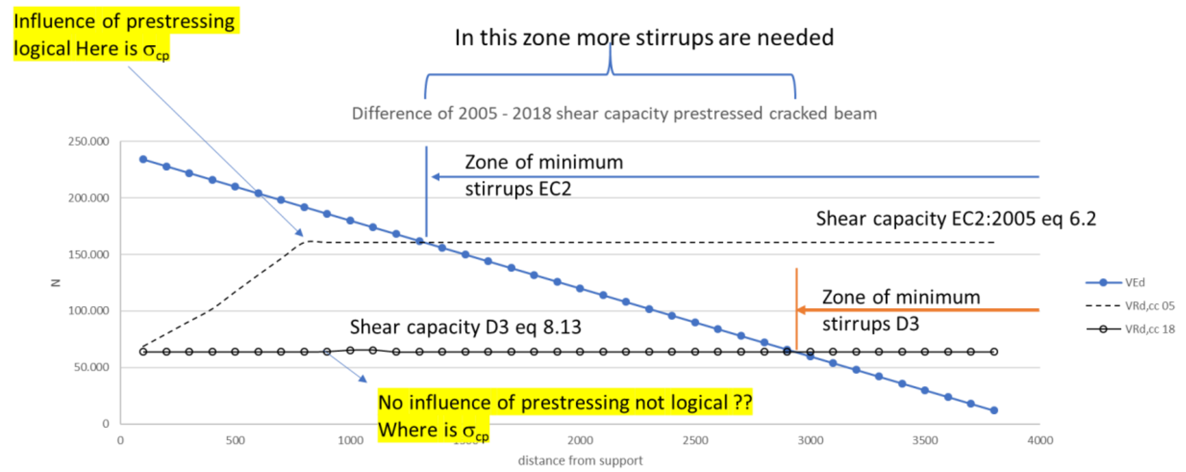
NOTE: The values of Table 8.2(NDP) for $C_{Rd,c2}$ apply unless the National Annex gives different values.

Table 8.2(NDP) – Values for $C_{Rd,c2}$ for Formula (8.13)

Design situation	In case τ_{Ed} is calculated according to (3) on the basis of d_{nom} [a]	In case τ_{Ed} is calculated according to (3) on the basis of d_d [b]
persistent and transient	0,46	0,51
accidental design situations	0,60	0,64

[a] nominal value of effective depth
[b] design value of effective depth according to A(7).

The formula still does not take into account the influence of prestressing (no σ_{cp} is considered). As practical result, a bigger zone requires the addition of stirrups as indicated in the graph below (NEW GRAPH NEEDED).



CRITICAL ISSUE – Negative vote

Belgian comments on D4 - rephrasing necessary

$k_1.\sigma_{cp}$ has disappeared from formula (8.13). The only way to take into account prestress is through clauses (4) and (5). But this impact is only very limited, resulting in many more stirrups (see graph in annex 1).	Investigate the influence of prestress and propose a new formula. If research is not feasible, the formula of the current EC2 should be adopted.

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Formula (8.13) is based on the Critical Shear Crack Theory. For the explanation of the formula, reference is made to the background document of D4.
The background document says that tests show that in non-slender beams the critical shear crack develops near the load introduction region (figure C8.4(b)).The shear strength in non-slender beams is almost 4 times bigger than in slender beams (figure C8.4(a)). Slender one-way members without transverse reinforcement have a similar behavior to the one described in figure C8.4(a). One-way members with limited shear slenderness (a/d < 2,5) and prestressed members behave like figure C8.4(b) due to the arching action.
For slender one-way members without transverse reinforcement, a mechanical model is given (formula C8.4 transformed to formula C8.13, included in D4). This model has been compared to 669 test results of slender reinforced concrete beams without transverse reinforcement.
The influence of a normal force applied in the centroid of the section can be taken into account by introducing an effective shear span a_{cs} (formula C8.15). This was compared to 158 test results of simply supported beams under point load and with an axial force applied in the centroid of the section.
Prestressing has three potential influences on the shear strengh. The influence of a prestressing force with an eccentricity e_p can be taken into account by introducing another effective shear span a_{cs} (formula C8.16).
According to the background document, the influence of prestressing (centric and eccentric) is not compared to test results. Formula C8.16 is also not implemented in D4, only formula C8.15 is.
According to 8.2.2(5) of D4, the prestressing effects should be considered in the values of M_{Ed}, V_{Ed} and N_{Ed} to be used in expression (8.17). Is it possible that the influence of prestressing is way too conservative based on the Critical Shear Crack Theory?
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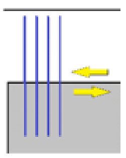
2. Shear at interfaces (eq. 8.55)

This issue may affect the use of lattice girders in floor plates.

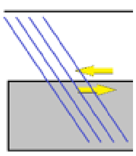
The general case is:



1) The reinforcement is perpendicular to the surface of the interface, α = 90° → the capacity of the reinforcement is given by the following equation:

$$V_{Rds} = A_s \cdot f_{yd} \cdot (\mu \cdot \sin(\alpha) + \cos(\alpha))$$

If α=90° then cos(α) = 0 and sin(α) = 1



2) All reinforcement is placed under an angle with the surface of the interface 45° < α < 90° → the capacity of the reinforcement is given by the equation:

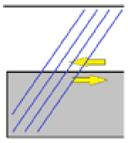
$$V_{Rds} = A_s \cdot f_{yd} \cdot (\mu \cdot \sin(\alpha) + \cos(\alpha))$$

If α=45° then cos(α) = 0.7 and sin(α) = 0.7

$$V_{Rds} = A_s \cdot f_{yd} \cdot (\mu \cdot 1/\sqrt{2} + 1/\sqrt{2})$$

The specific case of lattice girders should be added. If lattice girders are used, you may count on a contribution of every diagonal provided that 35° ≤ α ≤ 145° as long as both the positive as well as the negative contributions of cos(a) are taken into account.

Can we agree with this statement?	Agree on the need for an exception of lattice girders in floor plates Proposal to add in accordance with EN 1992-1-1:2004, 6.2.5 (3): “Where the connection between the two different concretes is ensured by reinforcement (beams with lattice girders), the steel contribution to v_{Rdi} may be taken as the resultant of the forces taken from each of the diagonals provided that 45° ≤ α ≤ 135°.”



3) All reinforcement is placed under an angle with the surface of the interface
90° < α < 135° → the capacity of the reinforcement is given by the equation:

$$V_{Rds} = A_s \cdot f_{yd} \cdot (\mu \cdot \sin(\alpha) + \cos(\alpha))$$

In this case there is no contribution of reinforcement allowed cos(α) will be negative. $V_{Rds} = 0$

3. Design anchorage length (equations 11.2 and 11.3)

Two new formulas are given for the design anchorage length

1. Ribbed and indented bars

(2) For ribbed and indented bars $\phi \leq 20$ mm, with $f_{ck} \geq 25$ MPa, $c_d \geq 1{,}5\phi$ (c_d according to Figure 11.3c) and $\gamma_C = 1{,}5$, the design anchorage length in tension l_{bd} may be calculated as:

$$l_{bd} = k_{lbs} \cdot \phi \qquad (11.2)$$

where coefficient k_{lbs} may be taken from Table 11.1 or calculated using Formula (11.4) as a function of the reinforcement stress σ_{sd}.

Table 11.1 – Coefficient k_{lbs} as a function of the design stress σ_{sd} for γ_C = 1,5

D4:

σ_{sd} [MPa]	≤ 200	> 200 ≤ 250	> 250 ≤ 300	> 300 ≤ 350	> 350 ≤ 390	> 390 **≤ 435**	> 435 ≤ 480	> 480 ≤ 520	> 520 ≤ 610
k_{lbs}	16	22	29	36	43	**50**	58	65	83

(2) For ribbed bars with $\phi \leq 32$ mm and indented bars with $\phi \leq 14$ mm in common cases the design anchorage length l_{bd} divided by diameter in tension and compression in persistent and transient design situations may be taken from Table 11.1(NDP).

Table 11.1 (NDP) – Anchorage length of straight bars divided by diameter l_{bd} / ϕ

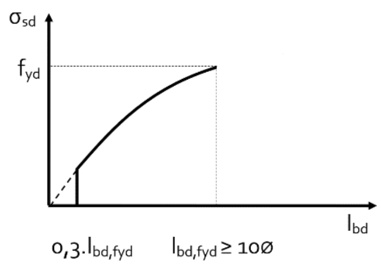
ϕ [mm]	Anchorage length l_{bd} / ϕ							
	f_{ck}							
	20	**25**	**30**	**35**	**40**	**45**	**50**	**60**
≤ 12	47	42	38	36	33	31	30	27
14	50	44	41	38	35	33	31	29
16	52	46	42	39	37	35	33	30
20	56	50	46	42	40	37	35	32
25	60	54	49	46	43	40	38	35
28	63	56	51	47	44	42	40	36
32	65	58	53	49	46	44	41	38

NOTE: The values of table 11.1(NDP) are derived from Formula (11.2). This table is valid for $c_d \geq 1,5\phi$, σ_{sd} = 435 MPa and for bars in good bond conditions. For bars in poor bond conditions in concrete members the values should be multiplied by 1,2. For σ_{sd} < 435 MPa the values may be multiplied by (σ_{sd} / 435), but consider l_{bd} / $\phi \geq 10$.

D5:

NEED TO BE ASSESSED	What are the impacts?

Belgian comment to D4 - rephrasing necessary

According to Model Code 90, a minimum anchorage length is necessary to ensure a minimum active anchorage length and to take into account tolerances. $l_{b,min}$ was defined as max{0,3.Ø.f_{yd}/(4.f_{bd}), 10Ø, 100 mm}.The first one relates to ductility. In the event of an accidental damage, the construction might require to perform in a ductile manner. This can be achieved if the anchorage length is sufficient to enable the reinforcement to reach a higher stress than the design stress. The second one ensures a minimum value in case the reinforcement is designed for yielding with high concrete strengths and large concrete covers. The third one has to compensate for deviations in the placement of the reinforcement. The same approach is used in Model Code 2010 and the current EC2. In D4, only one value (15Ø) needs to be calculated for the minimum anchorage length. No information can be found why this decision was taken. It is obvious why the requirement of 100 mm has been deleted. Defining a minimum anchorage length to take into account tolerances is probably not the correct way. The maximum permitted deviation on the longitudinal placement of the reinforcement should be specified in other standards (e.g. EN 13670 in case of cast in situ concrete or EN 13369 in case of precast concrete). The minimum anchorage length in D2 was set to 12Ø because the current value of 10Ø seemed to be too short, taking into account the tolerances and uncertainties in defining the position of the cross section where the reinforcement force is fully transferred. In D4 the minimum anchorage length was changed to 15Ø because Great Britain's suggested value of 15Ø is more 'reasonable' according to the PT. What is the technical reason for setting this value? A minimum value such as 10Ø, 12Ø or 15Ø is independent of the stress in the reinforcement (σ_{sd}). This can lead to large values, even if σ_{sd} is low. Furthermore, this seems to be in contraction with figure 9.2 of the current EC2, where the resisting tensile force progress linear from the beginning of the reinforcement to a	Define a minimum anchorage length to obtain sufficient ductility in cases with low stresses in the reinforcement, e.g. 0,3.$l_{bd,fyd}$. Consider making a distinction between isolated and non-isolated members. Without technical arguments, change the minimum anchorage length in case of yielding from 15Ø to 10Ø. Provide a graph, such as the one below, to clarify the minimum anchorage length. 

distance equal to l_{bd}. Anchorages at bearings are rarely subjected to high values of σ_{sd}, unless the reinforcement is curtailed, which is not possible in some precast products, e.g. hollow core slabs. The value of 15Ø for example leads to bearing details of hollow core slabs that are difficult or impossible to execute (for example 2 hollow core slabs with a nominal bearing length of 70 mm placed on a 140 mm wide wall). Therefore, a distinction must be made between reinforcement designed for yielding and reinforcement with low design stresses. In case of low stresses, a minimum anchoring length should be defined to obtain ductility. This minimum length cannot be expressed only in relation to the diameter of the bar, unless a distinction is made between precast concrete and in-situ concrete. In case of yielding stresses, a minimum anchorage length should be defined to avoid very low values in case of high concrete strengths and large concrete covers. Apart from that, it is recommended to make a distinction between isolated (e.g. beams) and non-isolated members (e.g. floor elements). Non-isolated elements behave in a different way in case of accidental damage, due to the ability to spread loads, resulting in a more ductile construction. In other words, the ductility requirement for floors is different from that for beams.	

2. For other elements

D4:
$$\ell_{bd} = k_{lbs} \cdot \phi \left(\frac{25\text{ MPa}}{f_{ck}}\right)^{\frac{1}{2}} \left(\frac{\phi}{20\text{ mm}}\right)^{\frac{1}{3}} \left(\frac{1.5\phi}{c_d}\right)^{\frac{1}{2}} \geq 15\phi$$

Where
$$k_{lbs} = 50\left(\frac{\sigma_{sd}}{435\text{ MPa}} \cdot \frac{\gamma_C}{1.5}\right)^{\frac{3}{2}}$$

D5:
$$l_{bd} = k_{lb} \cdot k_{cp} \cdot \phi \cdot \left(\frac{\sigma_{sd}}{435}\right)^{n_\sigma} \cdot \left(\frac{25}{f_{ck}}\right)^{\frac{1}{2}} \cdot \left(\frac{\phi}{20}\right)^{\frac{1}{3}} \cdot \left(\frac{1{,}5\phi}{c_d}\right)^{\frac{1}{2}} \geq 10\phi \qquad (11.2)$$

where

$n_\sigma = 1{,}0$ for $\sigma_{sd} \leq 435$ MPa,

$n_\sigma = 1{,}5$ for $\sigma_{sd} > 435$ MPa;

k_{cp} coefficient accounting for casting effects on bond conditions:

- $k_{cp} = 1{,}0$ for bars with good bond conditions according to Figure 11.4;
- $k_{cp} = 1{,}2$ for poor bond conditions and for all bars used in slipform construction unless it is shown that the vertical bars cannot move during casting;
- $k_{cp} = 1{,}4$ for all bars executed under bentonite or similar slurries unless data is available for the specific slurry to be used.

NOTE: The following values for k_{lb} apply, unless the National Annex gives different values:
$k_{lb} = 50$ for persistent and transient design situations, and
$k_{lb} = 36$ for accidental design situations.

Ratios in Formula (11.2) with bar diameter shall be limited as $(\phi / 20\text{ mm}) \geq 0{,}6$ and as $(1{,}5\phi / c_d) \geq 0{,}4$.

NEED TO BE ASSESSED	What are the impacts?

Belgian comment to D4 - rephrasing necessary

Ø/20 in formula (11.3) shall not be taken smaller than 0,6. According to the background document, referring to fib Bulletin 72, the limit Ø ≥ 12,5 mm reflects experimental evidence (probably reflecting a low relative rib area for small bars), but based on figure 3-12 in fib Bulletin 72, this limit can be removed. According to the background document, Ø/20 is therefore fixed to ≥ 0,5 in prEN1992-1-1:2018. However, this change has not been made.	Change Ø/20 ≥ 0,6 to Ø/20 ≥ 0,5
According to 11.4.2(4), the design anchorage length l_{bd} may be reduced by 20% for a reinforcement located in favourable positions during concreting. The comments in D4 say that the reduction factor, as well as the definition of good bond conditions, needs to be verified on the basis of ongoing researches. This is also written in the background document to D4. As long as the results of these researches are not known, it is not justified to change the reduction from 30% (current EC2; 1/(1/0,7) = 0,7) to 20%.	Change the reduction coefficient for k_{lbs},to the current value of 30%

4. Concrete cover for bonds (table 13.1)

The new concrete cover for bond for pre-tensioned tendons is defined in table 13.1 (chapter 13.5.1):

Minimum spacing *s*	Minimum cover: $c_{min,b} / \phi$	
	Strand	Indented wire
s = 2 ϕ	3.0	4.5
s ≥ 2.5 ϕ	2.5	4.0

If in most of the cases the concrete cover is defined by durability issues, in some others (e.g. inside the holes of hollowcores), the new proposed values may create problems.

It is noted that Fib model code 2010 § 7.13.2.2 suggests values of 1.5ϕ for strands or plain wires and 2.5ϕ for indented wires:

In order to ensure the transfer of the bond forces between concrete and reinforcing bars c_{min} shall be larger than:

For pretensioned strands or wires c_{min} shall be larger than:

- 1,5 the diameter of the strand or plain wire
- 2,5 the diameter of an indented wire

Additionally, DiBt study showing a higher value is not publicly available and cannot be challenged and Industry experience with present values shows no problems.

Actions	Advocate for the reduction of the cover for bond to the present values

Belgian comments to D4 - can we agree with it?

The minimum concrete covers in table 13.1 are based on the papers of the PhD-researches at EMB-RWTH of Stephan Geßner in 2017 and Andreas Nitsch in 2001. These minimum values are valid for the maximum stress (0,8.f_{pk}) and without transverse reinforcement to prevent splitting, which is certainly not the general situation.	Remove table 13.1, indicate that transverse reinforcement is the best way to prevent splitting and refer to product standards for those elements which cannot be produced with transverse reinforcement.

However, since there are only a few precast elements that do not have any splitting reinforcement, it is better to generalize the case with splitting reinforcement and to refer to specific product standards and their factory production control for elements without transverse reinforcement. On top of that, splitting cracks are always detected during the factory production control, before the elements are shipped to the construction site. These elements are therefore rejected so that structural safety is not compromised. For example, the minimum concrete covers in the current version of EC2 and in the current version of EN 1168 have been used for many years in the production of hollow core slabs, without problems (more than 1 million m² per year in Europe). The same values are used for many years in the production of prestressed beams for beam-and-block floor systems, according to EN 15037-1. These years of practical experience cannot be ignored.	

5. Transmission length of prestressing (equation 13.4)

The new formula for the transmission length of prestress (13.4, chapter 13.5.3):

$$l_{pt} = \frac{\alpha_1 \cdot \alpha_2 \cdot \sigma_{pm0}}{\eta_1 \cdot \sqrt{f_{ck}(t)}} \cdot \phi$$

With the coefficients:

α_1 = 1,0 for gradual release
= 1,25 for sudden release

α_2 = 0,47 for indented wires
= 0,30 for 3- and 7-wire strands

Gives higher values compared to the present code.

It seems that the intention of the project team is not to increase the values, but to give them in a new formulation. It would therefore be possible to "recalibrate" the parameter α_2 to achieve similar results as today.

Considering an α_{ct} = 1, the equivalence with the present situation is achieved by defining α_2

- Indented wires

  $\boldsymbol{\alpha_2 = 0,4}$ instead of 0,47 (draft 3) and 0,25 (EC2)

- 3- and 7- wire strands

  $\boldsymbol{\alpha_2 = 0{,}26}$ instead of 0,3 (draft 3) and 0,19 (EC2)

It is noted that the same considerations are valid for the anchorage length l_{bpd}.

Actions	Agree on and propose the coefficients for α_2

6. Dependence of the shear formula from the design tensile strength (equation 13.9)

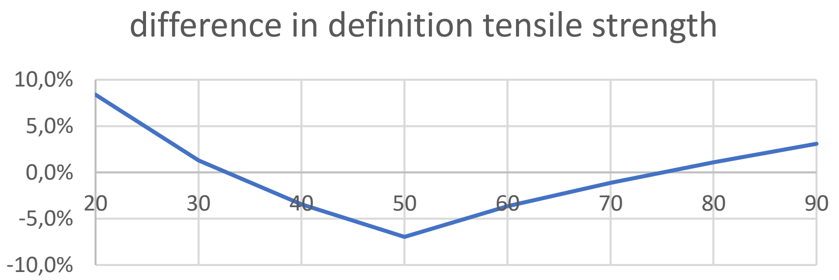
The new definition of the design tensile strength (formula 5.5 in chapter 5.1.6):

$$f_{ctd} = k_{tt} \frac{f_{ctk,0,05}}{\gamma_C}$$

Is not an issue per se, but it creates problems when inserted in the shear formula 13.9

$$\tau_{Rd,c} = \frac{I}{d \cdot S} \sqrt{(f_{ctd})^2 - \alpha_l \sigma_{cp} f_{ctd}}$$

Considering the new formula with the dependence on $\sqrt{\frac{f_{ck}}{\gamma_C}}$, there is a loss of capacity for concretes between 32MPa and 75 MPa (up to - 8% for 50 MPa) as shown below:



This is particularly severe for hollowcores (made of C50), when in practice or during tests the previous shear capacity is confirmed.

To be decided	BIBM comment on it?

7. Limitation to 500mm for the validity of the shear resistance of precast members without shear reinforcement (chapter 13.5.5)

The shear formula 13.9 in chapter 13.5.5:

$$\tau_{Rd,c} = \frac{I}{d \cdot S} \sqrt{(f_{ctd})^2 - \alpha_l \sigma_{cp} f_{ctd}}$$

Is limited in validity for members which effective depth is not larger than 500 mm. The reason for the limitation is that tests have been carried out on elements which depth is lower or equal to 500 mm and no size effect is provided in the formula.

Effects have been quantified and an increase of stirrups of 15% is expected.

Tests with elements with a height > 500 mm have been performed and confirm the validity of the equation even for elements higher than 500mm:

- Shear capacity of prestressed ad reinforced concrete members – modelling and experimental validation (ISBN 978-94-6018-901-2) – beams of h = 700 mm are tested
- Reduction of conventional shear reinforcement in pretensioned concrete beams by using steel fibre concrete – Nele van den Buverie (PhD thesis) – beams of h = 900 mm are tested

Actions	Provide arguments to delete the limitation to 500 mm: • Shear resistance is a physical effect (see Circle of Mohr) and is independent from the size • Clarify that the region is uncracked in bending, and therefore presents a linear behaviour • Show the results of tests with elements 700 mm deep to show that this is still valid • The present version of the code allows its use for such members; it is currently used by the industry with no issues reported

Belgian comments to D4 - can we agree with it?

The use of f_{ctd} in formula (13.9) is unacceptable because of the loss in capacity of 30%, due to the factor k_{tt} = 0,7. Many years of initial type testing of prestressed hollow core slabs, according to annex J of EN 1168, proves that the current formula is a safe approach of the actual shear capacity. But this formula is also used for many years to calculate the shear capacity in uncracked zones of prestressed beams. In formula (8.55) for the shear resistance at the interface, c.f_{ctd} is replaced by $c_{v1}.\sqrt{f_{ck}}/\gamma_c$. The difference between f_{ctd} and $\sqrt{f_{ck}}/\gamma_c$ is recalculated in c_{v1}. This calibration is clearly based on the current EC2, using only the f_{ck}-values between 25 and 40 MPa, as these are the most common values for a cast in situ concrete. Since formula (13.9) is the only one with f_{ctd} , f_{ctd} should be replaced by $0{,}40.\sqrt{f_{ck}}/\gamma_c$, calibrated with the current EC2, using only the f_{ck}-values between 45 and 60 MPa, as these are the most common values for precast concrete.	Replace f_{ctd} by $0{,}40.\sqrt{f_{ck}}/\gamma_c$.
According to the PT-comments, the limitation of the effective depth to 500 mm has been decided in a meeting of Ad Hoc Group TC250/SC2-TC229. The reason would be that this formula has been calibrated on the basis of tests on hollow core slabs with d < 500 mm. And since the formula has no size effect, it would be unsafe to use it for larger elements. Formula (13.9) is derived from the Circle of Mohr, which can be used for the calculation of any material strength. For concrete this can be used in uncracked zones. Formula (13.9) was not calibrated on the basis of tests. There is no size effect in the formula, because an uncracked section behaves independently of its height. The height of hollow core slabs in EN 1168 is limited to 500 mm. The reason for this is of a technical/manufacturing nature. Because of this limitation, only a few tests on hollow core slabs thicker than 500 mm have been carried out. Besides that, the tests on hollow core slabs, according to annex J of EN 1168, are necessary because the concrete properties for the input of the calculation of the shear resistance depends on the proper functioning of the production machine. But formula (13.9) is also used for many years to calculate the shear capacity in uncracked zones of prestressed beams higher than 500 mm. For example, Nele Van den Buverie performed a full scale test, in 2007 in the Magnel Laboratory at the University of Ghent, on a prestressed I-beam with a height of 900 mm and without stirrups in the I-shaped part of the beam. The calculated shear capacity was 560 kN according formula 6.4 of the current EC2. Compared to the experimental shear capacity of 597 kN, formula 6.4 of the current EC2 gives a safe approach. Annex 2 shows the principle of the calculation according to the current EC2 of a simply supported prestressed roof beam. Formula (13.9) in combination with formula (8.13) results in 16% more stirrups!	Remove the limitation to an effective depth of 500 mm.

8. Partial factors (chapter 4.3.3 and Annex A)

The proposal to modify the partial factors in the core of the text (chapters 4.3.3 and 4.3.4) has been dropped. The partial factors of table 4.3 **may be** modified following annex A (inclusion of Muttoni's proposal as an alternative way)

Can we live with this?

9. Exposure Resistance Classes approach (chapter 6)

The new concept of Exposure Resistance Classes (6.4) was introduced in chapter 6 "durability". It replaces the "structural classes", but with a different approach.

At first reading, these possible issues were spotted:

- New definition of c_{min}:

$c_{min} = \max \{c_{min,dur} - D_{cdur,red} + D_{cdur,abr}; c_{min,b}; 10 \text{ mm}\}$

  The reduction of concrete cover linked to the "Special quality control" (as in table 6.2) have disappeared

- 6.5.2.2 (4) For prestressing tendons, pre- or post-tensioned, the cover values in Table 6.3(NDP) and Table 6.4(NDP) should be increased by +10 mm, except where the internal bonded posttensioning systems are provided with protection level 2 or 3 according to 5.4.1, and internal unbonded prestressing tendons are encased in corrosion resistant sheaths.

Need for assessing the new proposal

10. Punching Shear (chapter 8.4.3)

The formula for Punching Shear (8.70) changed in a similar way as the one of shear resistance and again seems more penalising

(1) The design punching shear stress resistance ~~in [MPa]~~ shall be calculated as follows:

$$\tau_{Rd,c} = \frac{0,6}{\gamma_C} k_{pb} \left(100\rho_l \cdot f_{ck} \cdot \frac{d_{dg}}{d_v}\right)^{1/3} \leq \frac{0,6}{\gamma_C} \sqrt{f_{ck}} \quad (8.70)$$

where

~~f_{ck} is in [MPa]~~

$$\rho_l = \sqrt{\rho_{l,x} \cdot \rho_{l,y}} \quad (8.71)$$

$\rho_{l,x}$, $\rho_{l,y}$ are the bonded flexural reinforcement ratios in the x- and y-directions respectively. The values of $\rho_{l,x}$ and $\rho_{l,y}$ should be calculated as mean values over the width b_s defined in Figure 8.22.

d_{dg} is defined in 8.2.1(4);

k_{pb} is the punching shear gradient enhancement coefficient that may be calculated as:

$$k_{pb} = \sqrt{5\mu_p \frac{d_v}{b_0}} \leq 2,5 \quad (8.72)$$

μ_p is a coefficient accounting for the shear force gradient and bending moments in the region of the control perimeter. Its value may be ~~assumed~~ taken as follows:

- $\mu_p = 8$ for internal columns;
- $\mu_p = 4$ for edge columns and ends of walls;
- $\mu_p = 2$ for corner columns and corners of walls;
- for other cases, or for slabs with openings and inserts affecting the curved sectors of the control perimeter, ~~the~~ coefficient μ_p may be calculated as $\mu_p = \alpha_p / 45°$ where α_p is the sum of the angles of the curved sectors of the control perimeter as defined in Figure 8.23.

D4:

8.4.3 Punching shear resistance of slabs without shear reinforcement

(1) The design punching shear stress resistance shall be calculated as follows:

$$\tau_{Rd,c} = C_{Rd,c3} \cdot k_{pb} \left(100\rho_l \cdot f_{ck} \cdot \frac{d_{dg}}{d_v}\right)^{\frac{1}{3}} \leq C_{Rd,c3} \sqrt{f_{ck}} \quad (8.70)$$

where

$$\rho_l = \sqrt{\rho_{l,x} \cdot \rho_{l,y}} \quad (8.71)$$

$\rho_{l,x}$, $\rho_{l,y}$ are the bonded flexural reinforcement ratios in the x- and y-directions respectively. The values of $\rho_{l,x}$ and $\rho_{l,y}$ should be calculated as mean values over the width b_s defined in Figure 8.22.

d_{dg} is defined in 8.2.1(4);

k_{pb} is the punching shear gradient enhancement coefficient that may be calculated as:

$$k_{pb} = \sqrt{5\mu_p \frac{d_v}{b_0}} \leq 2,5 \quad (8.72)$$

μ_p is a coefficient accounting for the shear force gradient and bending moments in the region of the control perimeter. Its value may be taken as follows:

- $\mu_p = 8$ for internal columns,
- $\mu_p = 4$ for edge columns and ends of walls,
- $\mu_p = 2$ for corner columns and corners of walls,
- for other cases, or for slabs with openings and inserts affecting the curved sectors of the control perimeter, coefficient μ_p may be calculated as $\mu_p = \alpha_p / 45°$ where α_p is the sum of the angles of the curved sectors of the control perimeter as defined in Figure 8.23,

$C_{Rd,c3}$ is a coefficient which depends on the design situation and the assumption for determing the effective depth.

D5:

NOTE: The values of Table 8.4(NDP) for $C_{Rd,c3}$ apply unless the National Annex gives different values.

Table 8.4(NDP) – Values for $C_{Rd,c3}$ for Formula (8.70)

Design situation	In case τ_{Ed} is calculated according to (3) on the basis of d_{nom} [a)]	of d_d [b)]
persistent and transient	0,41	0,45
accidental design situations	0,54	0,58
[a)] nominal value of effective depth [b)] design value of effective depth according to A(7).		

Need to assess the impact of the new formula

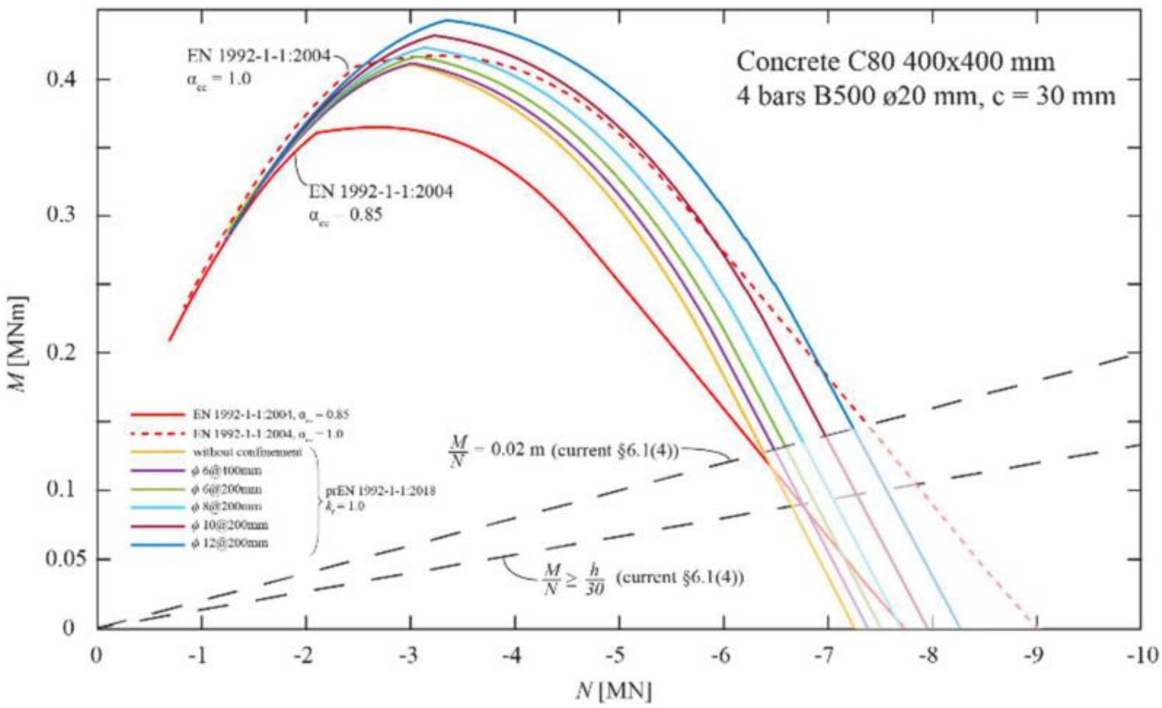
11. Design Compressive strength (equation 5.3)

The formula 5.3 in chapter 5.1.6:

$$f_{cd} = \eta_{cc} \cdot k_{tc} \frac{f_{ck}}{\gamma_C} \qquad \text{with} \qquad \eta_{cc} = \left(\frac{40}{f_{ck}}\right)^{\frac{1}{3}} \leq 1$$

can still be an issue for high strength concretes.

Comparisons made by the project team shows however that the “new” values are better than with the previous code (see graph below), especially if confinement is considered



Question	Shall we support French proposal to use coefficient 55 instead of 40?

12. k_{tc} (equation 5.3)

With the latest draft, a new proposal for the value of k_{tc} is given:

5.1.6 Design assumptions

(1) The value of the design compressive strength shall be taken as

$$f_{cd} = \eta_{cc} \cdot k_{tc} \frac{f_{ck}}{\gamma_C} \qquad (5.3)$$

where

η_{cc} is a factor to account for the difference between the undisturbed compressive strength of a cylinder and the effective compressive strength that can be developed in the structural member ~~component~~. It shall be taken as:

$$\eta_{cc} = \left(\frac{40}{f_{ck}}\right)^{\frac{1}{3}} \leq 1{,}0 \qquad (5.4)$$

k_{tc} is a factor considering the effect of high sustained loads and of time of loading on concrete compressive strength.

NOTE: The value is k_{tc} = 1,00 for $t_{ref} \leq 28$ days for concretes with classes CR and CN and $t_{ref} \leq 56$ days for concretes with class CS, and k_{tc} = 0,85 for other cases including when $f_{ck}(t)$ is determined in accordance with 5.1.3(4), unless the National Annex gives different values.

For a C80/95, the present value of f_{cd} varies between 48,57 and 57,14 MPa (depending on the national value of k_{tc}). With the new proposal, f_{cd} = 38,55 MPa, that corresponds to one or two classes less in the present system.

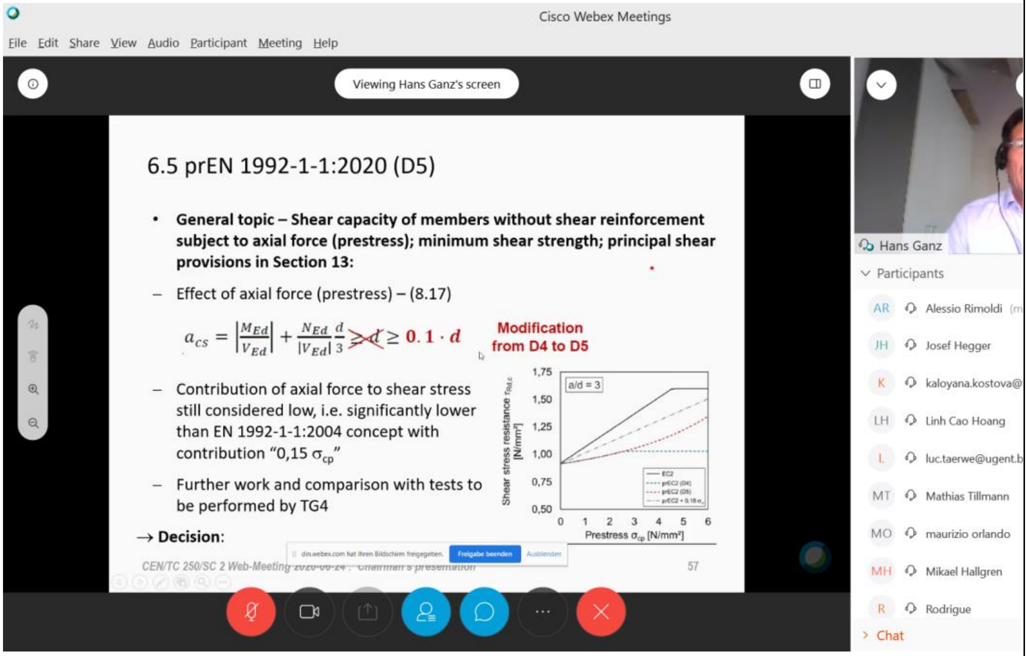
	A new penalising issue for high strength concrete used in precast

13. Summary of actions by 17 July 2020

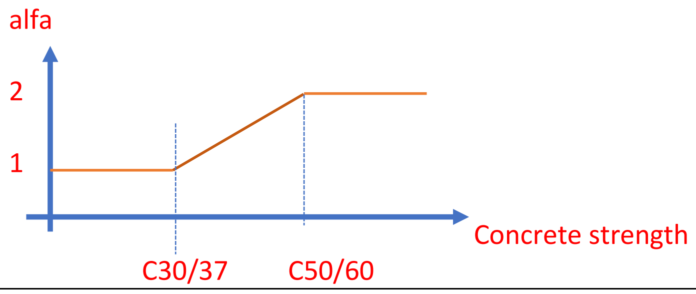
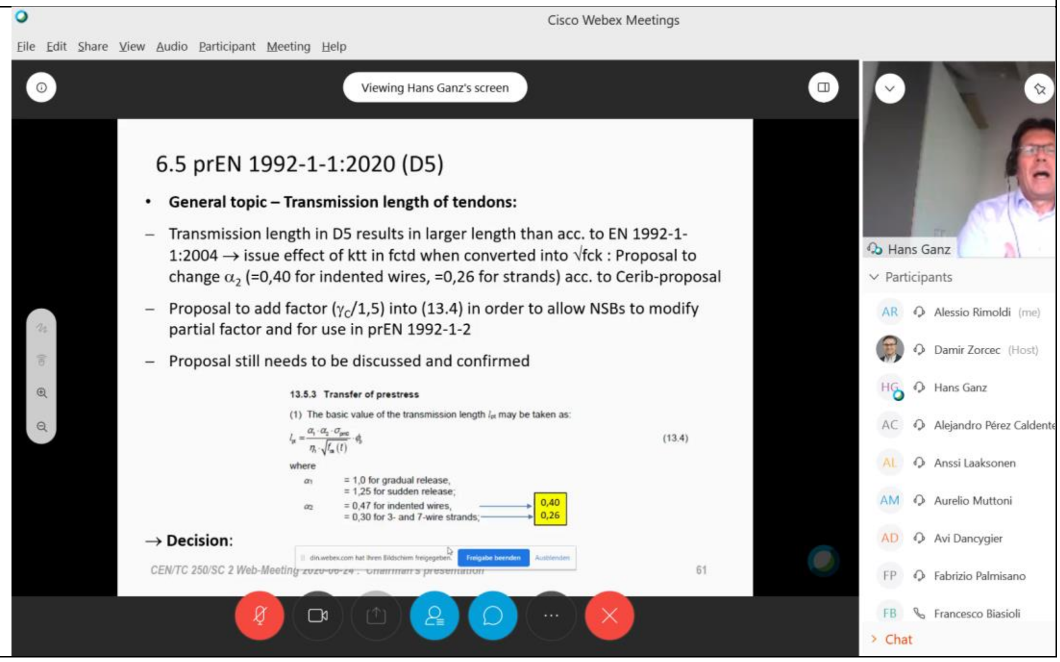
In italic, notes from WG1 and SC2 meetings of 22-24 June 2020

<table>
<tr><th>Topic</th><th>People</th><th>Action</th></tr>
<tr><td>1. Shear calculation</td><td>Pieter</td><td>Make a proposal on how to change clause (2) of 8.2.2
Delete clause (5)</td></tr>
<tr><td></td><td></td><td>The topic is identified as requiring further work and possibly better results. SC2 is calling for reasons why include or not gamma$_c$ parameter. Muttoni: using gamma$_c$ would reproduce the problems of current Eurocodes.
Several options on the adaptation of the formula were put on the table:

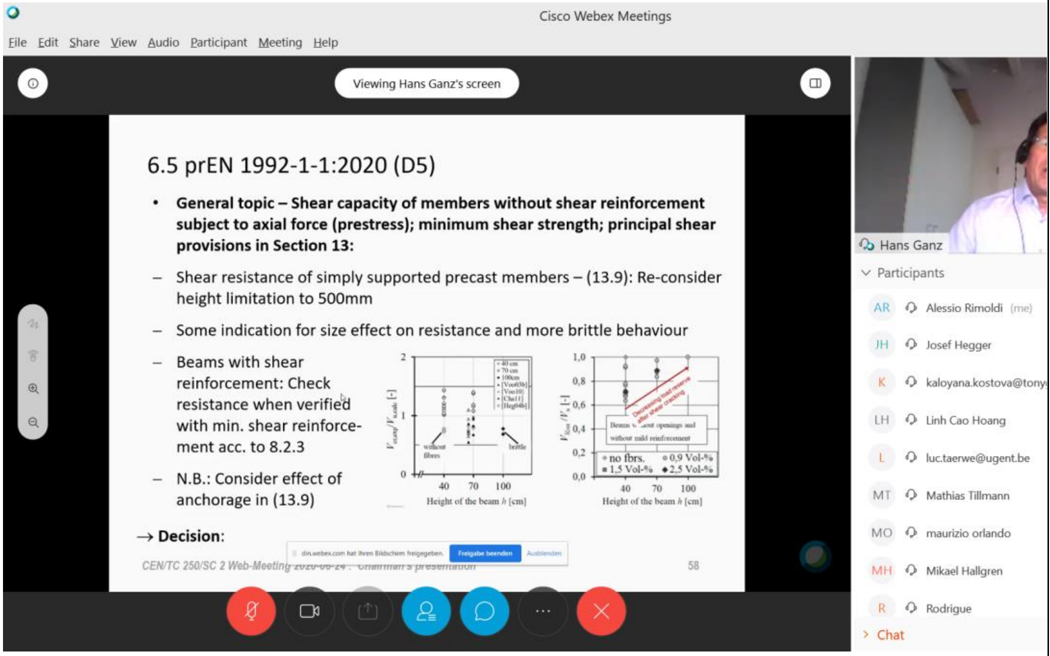
1. *C_{rdc2} could have 2 different values, one for normal reinforcement, one for prestressing.*
2. *FR: Gamma$_c$ should be introduced to take into account the quality control of the concrete; Muttoni answer: using Annex A for reducing the coefficient could be a possibility. Hegger: formula is quite open, because C_{rdc} is a NDP and Annex A will explain how to change C_{rdc} in function of beta.*
3. *Another possibility would be to have a different safety factor (e.g. 1,35) where the resistance does not depend linearly with the concrete strength (e.g. for shear).*
4. *Adjust the "d" value of 8.13 by taking into account shear slenderness*



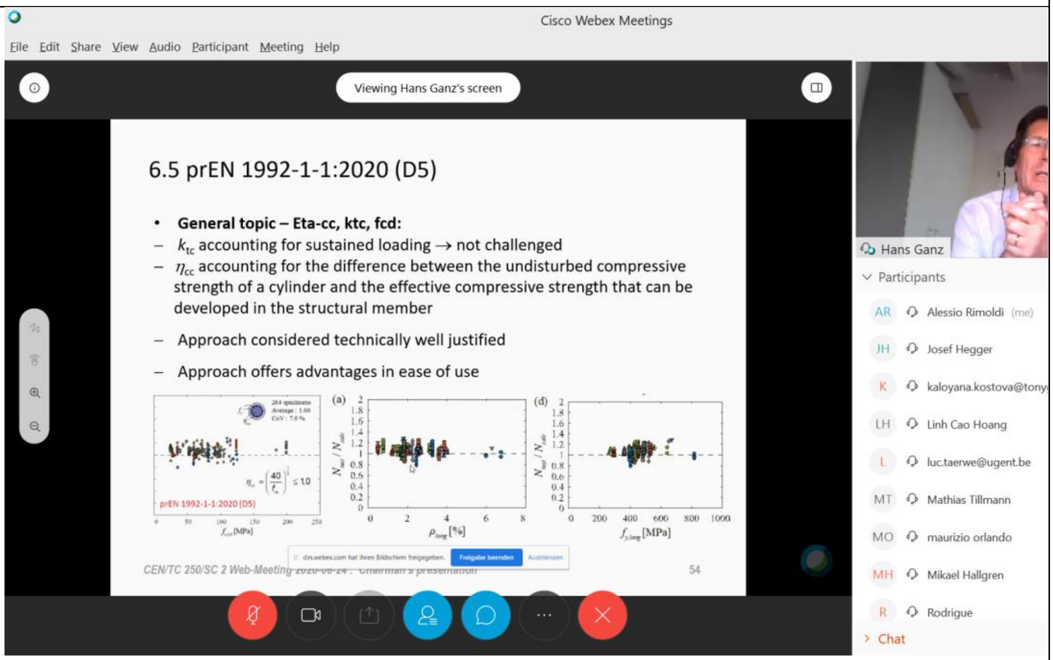
D5 gives better results than D4, but still below the present value of EC2.
TG4 will make a comparison with test results.
Spain: they have developed a new formula with critical crack model, which would give better values (M. Arrieta). Ganz is pushing for having a proper solution satisfying the precast sector demands.
Another possibility: instead of 0,15 gamma$_c$, it could be 0,22 gamma$_c$ divided by the safety factor</td></tr>
<tr><td>2. Shear at interfaces</td><td>Alessio</td><td>Modify 8.2.6 (10) as following:
When reinforcement is required across the interface to satisfy Formulae (8.55) or (8.56) it may be averaged over a length not exceeding 2z; <u>the steel contribution to t_{Rdi}</u></td></tr>
</table>

		<u>may be taken as the resultant of the forces taken from each of the diagonals provided that 45° ≤ a ≤ 135°</u>
3. Design anchorage length and lap length	Jean-Baptiste	Compare the values given in the present EC2 with D5; Propose a value for k_{lb} which would give similar values as today of the anchorage length as today (preliminary estimation 30-40) Propose a different value for k_{lb} for lap length (preliminary estimation 70-80, factor 2 to the anchorage length) Visualise the impact of D5 on the present construction practices (if possible) reintroduce the relation with gamma,c
		Pieter's comment after WG1 meeting: k_{lb} for anchorage we could use 35 and for laps we can multiply with 1.5 then it is more in line with present EC should be done as a NDP. In D5 anchorage and lap length are the same
4. Concrete cover for bonds	Pieter	Make a proposal that would be more in line with present situation A suggestion could be to have a dependence of the minimum cover to the concrete strength like 3/alfa 
	Ronald	Provide data from real life
5. Transmission length of prestressing	Nathalia	Propose new values of $alfa_2$ for achieving similar results as today (see values in the text above)
		

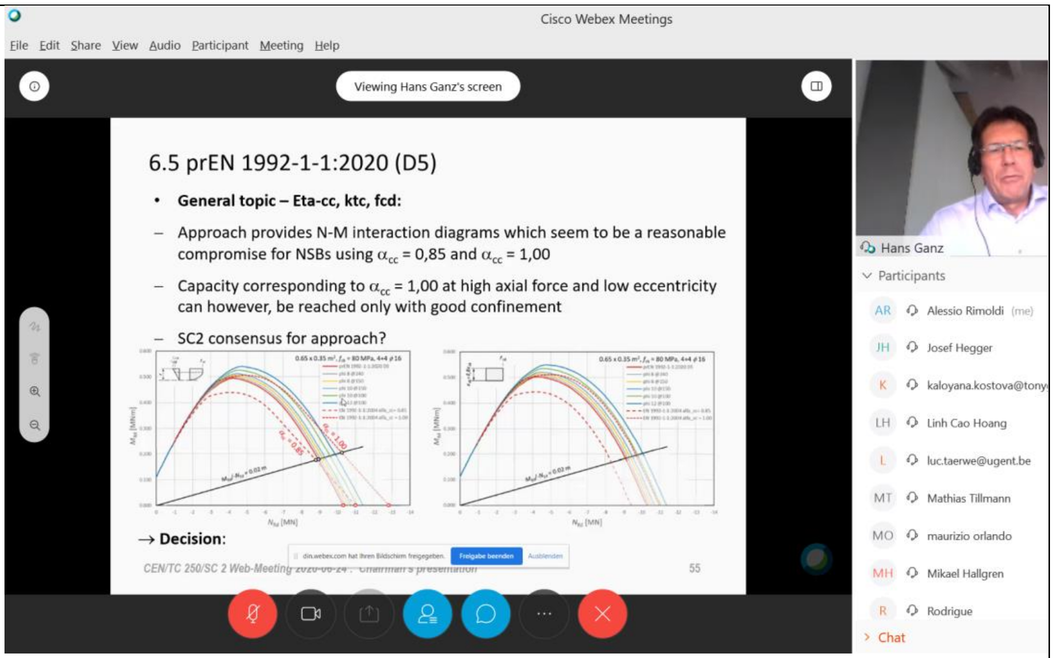
<table>
<tr><td></td><td></td><td>Ganz promoted the idea to calibrate the new formulation such that it could give the same value as in the current Eurocode. He proposed to accept CERIB proposal for the new values. Ganz will discuss with the different groups.
Tony: there is no link to gamma$_c$, which could give issues in fire design. A factor gamma$_c$ divided by 1,5 could solve the issue of fire design</td></tr>
<tr><td>6. & 7. Shear formula and 500 mm limitation</td><td>Bart</td><td>Make a proposal to reintroduce the old definition of fctd (0,4 * square root of f_{ck} over gamma$_c$) and to extend the limitation value of 500 mm.</td></tr>
<tr><td></td><td></td><td>Following the WG1 meeting of 22/23 June 2020, the old definition received a very negative feedback; also the limitation of 500 mm seems quite challenging; the current proposal seems to go towards : “limit 500 mm unless minimum reinforcement is used.”
During the SC2 meeting, the topic was also discussed



HEGGER: Test results of steel fibre reinforced beams shows that the beams with smaller depths have an increased capacity, whereas for higher beams cracking is critical. Not enough robustness in the structures with high elements without shear reinforcement. He also mentioned temperature effects through the height of the beam would reduce the capacity because of higher stresses (Wjite disagrees).
On this issue, Ganz defended the idea of trying to get closer to the previous situation. Mentioned that Pieter and Luc Tarwe would bring additional data.
The effect of anchorage should be taken into account (like EN 1168), maybe also the effect of flexible supports. Laaksonen found that for thick slabs there is risk of longitudinal crack in the web, close to the top surface that gives sudden crack. Tony Jones: what is the effect of permanent loads? Do we need to use the k_{tt}=0,7 coefficient here? Even 0,8 would be penalising. HEGGER: Sustained loads are more critical for tensile. However, k_{tt} is a NDP, SC2 should only give an advice. Maybe it would be worth advocating for a k_{tt}=1 as proposal from SC2?</td></tr>
<tr><td>8. Partial factors</td><td>Mathias</td><td>Propose to re-introduce the old annex A (as in D4) in parallel with the new annex A of D5
Work on making some calculations with new annex A to assess the impacts</td></tr>
</table>

<table>
<tr><td>9. Exposure resistance classes</td><td>Alessio</td><td>Follow the discussions between the different involved TCs (SC2, TC 104, TC 229 and TC 51)
Advocate for parallel systems (the present and the new one) to be both applicable in the future EC2
Link with developments in EN-206 is critical</td></tr>
<tr><td>10. Punching shear</td><td>Alessio</td><td>Try and find support in the BIBM Technical Commission to study this issue and propose a similar solution than for</td></tr>
<tr><td>11. & 12. Design compressive strength</td><td>Mathias</td><td>Study the influence of the new definition of k_{tc}
Propose a different definition in the note if necessary</td></tr>
<tr><td></td><td></td><td>


Ganz: k_{tc} is not challenged as today
Eta$_{cc}$ is defined as mentioned above. No countries issued any comment on the approach, nor on the values. There is a proposal from France to use 55 instead of 40; but Ganz believe that 40 is a better compromise between those countries using alfa$_{cc}$ 0,85 and 1. High strength concrete is favoured in other parts. Test shows that the issue is mainly for columns, but not if you consider confinement. To recover the difference, one can play on gamma$_c$ as well. Proposal from UK (joke?): define K_{tc} = 1/eta$_{cc}$
</td></tr>
</table>



For columns, we cannot reach the same capacities than the current Eurocode (M-N graph) even with a value of 40. High strength concrete with > 80 MPa would be penalised (> 5% compared to an alfa$_{cc}$= 1 in the current EC2).
With 0,35 % of strain in concrete, steel will work better.
Wjite: calculating with confinement will have an effect on the effective length of elements; he hopes that confinement will not be used in practice!! Muttoni: confinement will be used more in countries where seismic is an issue. Jones: using confinement is not easy in UK; as they have already 0,85, the new code is giving higher capacities.

14. Personal considerations after June 2020 meeting

I felt a positive attitude of SC2 experts towards the BIBM requests, in particular from the convenor Hans Rudolph Ganz. Most of our critical points have been mentioned in the second part of the meeting as “need to be solved”. Besides the issue of high strength columns and shear resistance of high elements, I did not feel any particular request for reducing the present capacities.

A good “calibrated” proposal would therefore be acceptable for all topics but these two.

- For high strength columns, I see little chances to have the factor 55 included instead of 40 (French proposal). It seems indeed that taking confinement into account, all calculations would lead (for a concrete with 80 MPa strength) to a range between the old alfa$_{cc}$ 0,85 and 1;
- For shear capacity (especially for high elements) **it is crucial to be able to provide as many data as possible** to demonstrate that the current shear level is safe. If it is the case, several means to correct the formula were proposed (see table above)

On the other hand, there was a feeling that most of the “blocking” points are coming from the precast sector. Although this is probably due to the good advocacy we have been done during the last years (and more recently in particular), I would try to avoid to concentrate too much the focus on our sector:

- On one side, this could geopardise our success (it is only an issue for a small part of the industry; if the rest is happy, why shall we change?)
- On the other, I still have the general feeling that the new EC2
    - Is more complex than before;
    - Is based on completely new concepts, mainly calibrated on relatively small test campaigns;
    - Is finally leading to structures that are less competitive and less sustainable than before.

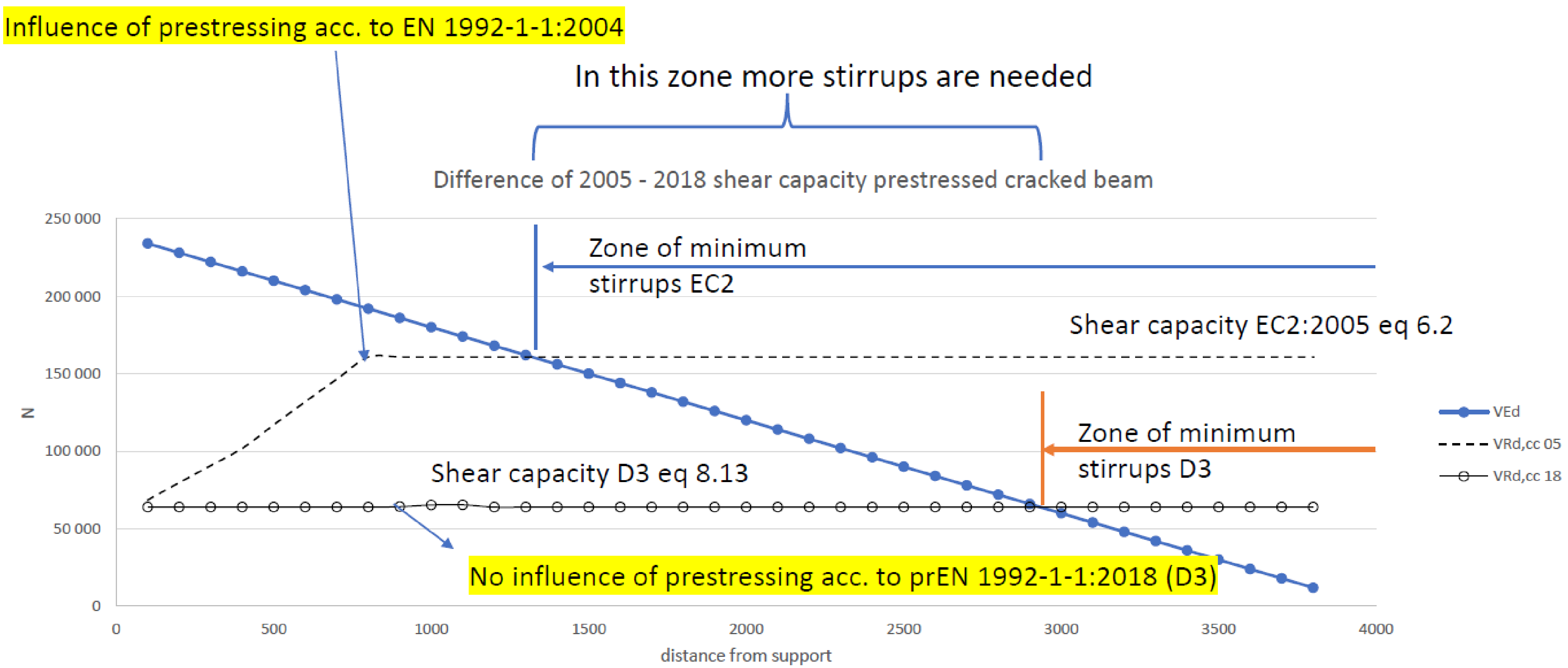
As a general conclusion, I think that there is still space for improvement within the present framework, but we need to act fast following the plan reported above.

Please find below my personal list of key players in the discussions:

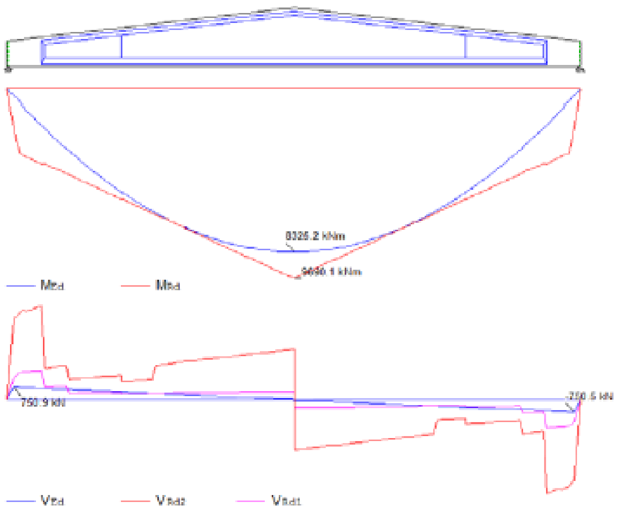
Hans Rudolph Ganz – convenor of SC2 (CH)	Mikael Hallgren – Convenor of WG 1 (SE)
Aurelio Muttoni – Convenor of PT1 (CH)	Simon Wijte – Dutch expert
Tony Jones – UK representative, working for MPA (Mineral products Association)	Josef Hegger – German expert (and convenor of TG4)

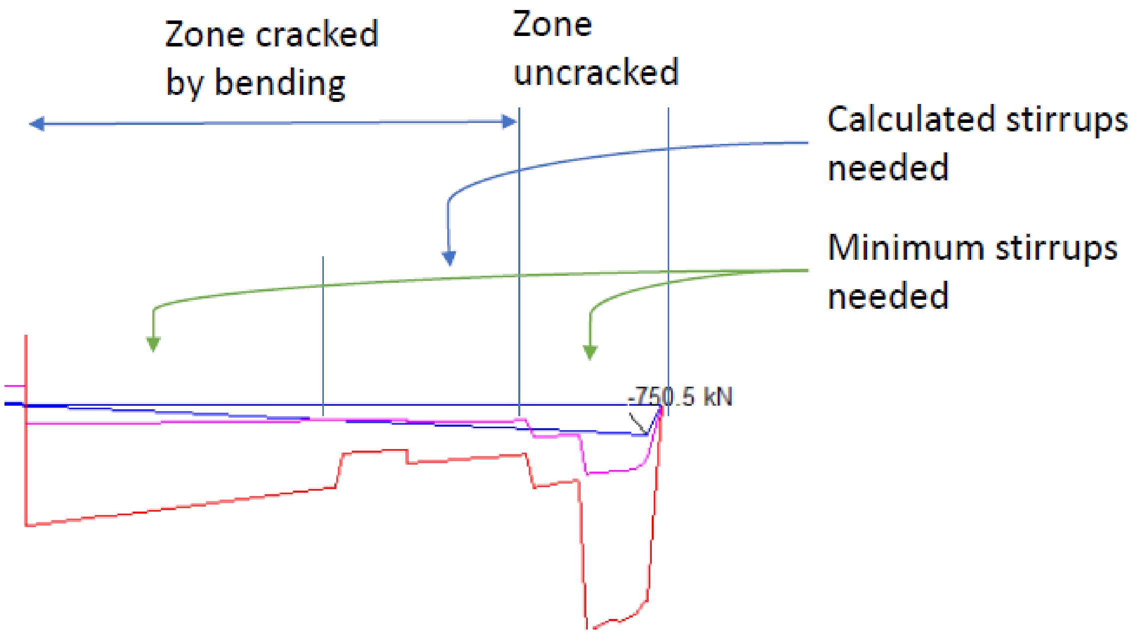
Should any of you have a privileged contact with one of these people, a discussion with them based on our final proposal and prior to the September meeting would be helpful.

Annex 1


Influence of prestressing acc. to EN 1992-1-1:2004
In this zone more stirrups are needed
Difference of 2005 - 2018 shear capacity prestressed cracked beam
Zone of minimum
stirrups EC2
Shear capacity EC2:2005 eq 6.2
Zone of minimum
stirrups D3
Shear capacity D3 eq 8.13
No influence of prestressing acc. to prEN 1992-1-1:2018 (D3)
250 000
200 000
150 000
100 000
50 000
0
N
0
500
1000
1500
2000
2500
3000
3500
4000
distance from support
VEd
VRd,cc 05
VRd,cc 18

Annex 2


750.9 kN
MEd
MRd
VEd
VRd2
VRd1


Zone cracked by bending
Zone uncracked
Calculated stirrups needed
Minimum stirrups needed
-750.5 kN