Schöck Combar®

Design guideline

for Combar® reinforced concrete acc. to Eurocode 2

(BS EN 1992-1-1:2004 and NA to BS EN 1992-1-1)

April 2016

Information

Source

- Technical Information Schöck Combar®
- General Building Authorithy Certification (AbZ) Schöck Combar® Z-1.6-238 (from 04. June 2014)
- Design concept for GFRP shear reinforcement (Kurth/Hegger, RWTH Aachen) published in „Bauingenieur“ Band 88, Oktober 2013 with the titel „Zur Querkrafttragfähigkeit von Betonbauteilen mit Faserverbundkunststoff-Bewehrung – Ableitung eines Bemessungsansatzes“.

Note

- When reffering to EC2 in this guide line, this is to be understood at any time as BS EN 1992-1-1:2004 and NA to BS EN 1992-1-1.

Layout in grey

- Gray layouts for any text, illustrations, formulas or values are not covered by the german approval Z-1.6-238 and to be understood as manufacturers recommendations.

Software

- The design software for Schöck Combar® can be downloaded on www.schoeck.co.uk

Content

1. Introduction

This design guideline is a summary of our research and knowledge of the GFRP reinforcement Combar®.
All standards of the German approval Combar® (Z-1.6-238) are considered. Aspects which are not yet governed by the DIbt approval are also covered in this guideline, based on our own design recommendations being reviewed by our partners (University of Aachen, Prof. Hegger & partner). For further help please contact our service engineers.
The purpose of this guideline is to provide assistance to engineers who are in charge of the design of GFRP reinforced concrete. It does not discharge the engineer from his obligation to exercise due care and the generally acknowledged rules of technology should be still carefully attended and not be overruled.

Combar® types

	⌀8	⌀12	⌀16	⌀20	⌀25	⌀32
straight bar	german approval	german approval	german approval	german approval	german approval	no german appr.
headed bar	-	no german appr.	no german appr.	no german appr.	no german appr.	no german appr.
bent bar	-	no german appr.	no german appr.	no german appr.	-	-

1.1 Significant difference to the design of steel reinforced concrete

When designing Combar® reinforced concrete members attention needs to be taken to the following:
The E-modulus of straight Combar® bars is 60.000N/mm². Combar® bars are linearly elastic up to failure. Yielding is not observed. Hince there is no effect of strain hardening. In Combar® reinforced concrete members like structural frames plastic hinges are not featured. Therefore the advantage of a redistribution of loads (and moments) must not be taken. Generally not only strength limit states are to be considered but particularly serviceability limit state (SLS). Especially deflections and crack control must be examined.

- The partial factor for Combar® is $\gamma_F = 1.3$
- Combar bars do not corrode and therefore concrete cover is solely required for the transmission of forces between the bars and the surrounding concrete (for all exposure conditions).
- Due to the low E-modulus compared to steel GFRP bars must not be taken as compression reinforcement. Nevertheless Combar® bars can be positioned in the compression zone (anchorage, nominal reinforcement).
- The German approval for Combar (Allgemeine bauaufsichtliche Zulassung, AbZ Combar) given by the DIBt (Deutsches Institut für Bautechnik in Berlin) does not cover laps of reinforcing bars yet, since up to now not all conceivable geometric combinations in any building environment could be tested and proved.
- Due to the low E-modulus the shear resistance of a member without shear reinforcement (Combar for flexural reinforcement) VRd,c is lower than it is for members with flexural reinforcement of steel. The German approval „ AbZ Combar" does not cover yet the use of Combar® bars as shear reinforcement.

1.2 Durability concept

All rated Combar® values are valid for 100 years design life and can be used without any decrease. They are proved for 100 years lifetime and a constant environmental temperature of 40° C. It is not necessary to increase the minimum concrete cover in order to achieve longer design life.

1.3 Bond behaviour

Bond properties are close to those of steel Due to the way of verification the bond values are slightly lower than those of steel.

1.4 E-Modulus

It is key to consider the E-modulus of Combar in all calculations. The E-modulus is determined by the percentage of fibers and the type of fibers. For Combar it is 60000 N/mm², which is the highest value achievable for GFRP. Therefore Combar is a „high grade" GFRP bar.

1.5 Bending shapes

Our production process of bent bars ⌀12, ⌀16 and ⌀20 mm allows all shapes like L-shapes, S- or Z-shapes or stirrups (similar to steel shapes). E-Modulus for bent bars is 50000 N/mm². They can be used as shear reinforcement in a concrete member. As mentioned before this is not yet covered by the german approval, but the design concept is being provided by Prof. Hegger and has been published in Germany in October 2013. Prof. Hegger's design concept has been developed at RWTH (university) of Aachen and is based on extensive series of tests using different GFRP reinforcing bars.

1.6 Headed bars

Headed bars are available for ø12mm and larger diameters. They can be used in order to decrease anchorage lengths. The high-strength polymer concrete which is sprayed on the bar end enables transmission of larger forces since a larger area is being activated to transfer the load. Headed bars are not covered by the German approval but the design procedure has been proved and tested by our partners (university of Aachen) similar to the procedure of bent bars.

1.7 Fire behaviour

Combar® is flame resistant and it contains only a little percentage of flammable resin. The bond between the bar and the concrete is guaranteed by the ribs. The resin gets weaker at high temperatures and the bar loses bond to the surrounding concrete due to deformation of the ribs. Therefore secure bond between ths bars and the surrounding concrete cannot be guaranteed in the case of fire. Concrete members with higher requirements than 30min fire resistance are to be designed with larger concrete cover than those with steel reinforcement or fire protection layers are to be installed.

2. Basis of design

2.1 Ultimate limit state (ULS)

- $E_d \leq R_d$
- E_d – Design value of the effect of actions.
- R_d – Design value of the resistance to the action.

General combination of actions

Design situations	Combination of actions
Persistent / transient design situations E_d	$E_d = \sum_{j\geq 1} \gamma_{G,j} \cdot E_{Gk,j} \oplus \gamma_{Q,1} \cdot E_{Qk,1} \oplus \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot E_{Qk,i}$
Accidental design situations E_{dA}	$E_{dA} = \sum_{j\geq 1} \cdot \gamma_{GA,j} \cdot E_{Gk,j} \oplus E_{Ad} \oplus \gamma_{QA,1} \cdot \psi_{1,1} \cdot E_{Qk,1} \oplus \sum_{i>1} \gamma_{QA,i} \cdot \psi_{2,i} \cdot E_{Qk,i}$

⊕: in combination with | 1: decisive variable action

2.2 Serviceability limit state (SLS)

- $E_d \leq C_d$
- E_d – Design value of the effect of action (e. g. deformation)
- C_d – Design value of the serviceability limit state criteria (e. g. permitted deflection)

Load combinations for SLS design

Design situation	Combination of actions
infrequent combination of actions $E_{d,char}$	$E_{d,char} = \sum_{j\geq 1} E_{Gk,j} \oplus E_{Qk,1} \oplus \sum_{i>1} \psi_{0,i} \cdot E_{Qk,i}$
frequent combination of actions $E_{d,frequ}$	$E_{d,frequ} = \sum_{j\geq 1} E_{Gk,j} \oplus \psi_{1,1} \cdot E_{Qk,1} \oplus \sum_{i>1} \psi_{2,i} \cdot E_{Qk,i}$
quasi-permanent combination of actions $E_{d,perm}$	$E_{d,perm} = \sum_{j\geq 1} E_{Gk,j} \oplus \sum_{i\geq 1} \psi_{2,i} \cdot E_{Qk,i}$

⊕: in combination with | 1: decisive variable action

2.3 values of Psi factors

Imposed loads in buildings	Ψ_0	Ψ_1	Ψ_2
Imposed loads $Q_{k,N}$:			
Category A: domestic, residential areas	0.7	0.5	0.3
Category B: office areas	0.7	0.5	0.3
Category C: congregation	0.7	0.7	0.6
Category D: shopping areas	0.7	0.7	0.6
Category E: storage areas	1.0	0.9	0.8
Imposed loads $Q_{k,V}$:			
Category F: traffic area, vehicle weight ≤ 30 KN	0.7	0.7	0.6
Category G: traffic area, 30 KN< vehicle weight ≤ 160 KN	0.7	0.5	0.3
Category H: roofs	0	0	0
Snow loads $Q_{k,S}$:			
altitude below and at NN + 1000 m	0.5	0.2	0
altitude above NN + 1000 m	0.7	0.5	0.2
Wind loads $Q_{k,W}$	0.6	0.2	0
Soil settlement $Q_{k,A}$	1.0	1.0	1.0
Other actions	0.8	0.7	0.5

2.4 Partial factors γ_F for actions on building members

	exterior actions				indirect actions	
	permanent actions (G_k) γ_G		variable actions (Q_k) γ_Q		accidental actions, constraint actions γ_Q	
Design situation	favourable	unfavourable	favourable	unfavourable	favourable	unfavourable
Persistent / transient design situations	1.00	1.35	0	1.50	0	1.00
Accidental design situations	1.00	1.00	0	1.00	0	1.00

2.5 Partial factors γ_M for materials

Design situation	γ_c concrete	γ_f Combar®
ULS - persistent and transient	1.5	1.3
Accidental	1.3	1.1

2.6 Important properties for concrete

[N/mm²]	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
f_{ck}	20	25	30	35	40	45	50
$f_{ck,\, cube}$	25	30	37	45	50	55	60
f_{cd} [with $\gamma_C = 1.5$]	11.3	14.2	17.0	19.8	22.7	25.5	28.3
E_{cm}	30,000	31,000	33,000	34,000	35,000	36,000	37,000

Note: $E_{cm} = 22.000 * (f_{cm} / 10)^{0.3}$

2.7 Important properties for Combar® [DIBt AbZ Z-1.6-238] german approval, for concrete ≥ C20/25

		determined static system	indetermined static system
Char. value long term tensile strength	f_{fk}	580 N/mm²	480 N/mm²
Design value long term tensile strength [with γ_f = 1.3]	f_{fd}	445 N/mm²	370 N/mm²
Tension modulus of elasticity	E_f	60,000 N/mm²	60,000 N/mm²
Strain at ULS	ε_{fd}	7.4 ‰	6.1 ‰

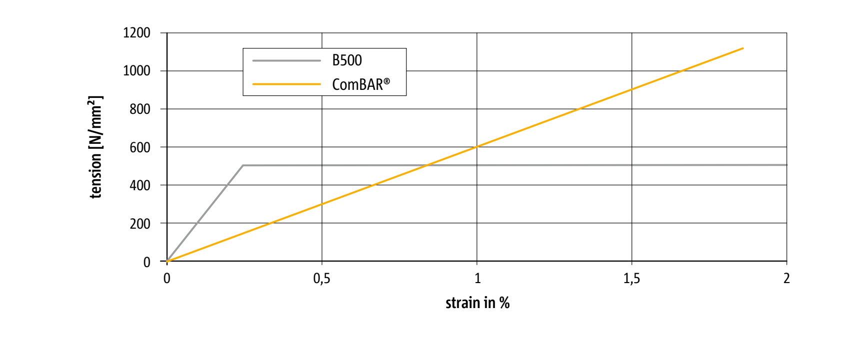
Adjustment factor between determined and indetermined static systems to be considered with $\eta_{rot} \approx 0.83$.

2.8 Important properties for Combar® bent bars for concrete ≥ C20/25

		flexural reinforcement determined static systems	flexural reinforcement indetermined static systems and shear reinforcement
Char. value long term tensile strength	f_{fk}	250 N/mm²	208 N/mm²
Design value long term tensile strength [with γ_f = 1.3]	f_{fd}	192 N/mm²	160 N/mm²
Tension modulus of elasticity	E_f	50.000 N/mm²	50.000 N/mm²
Strain at ULS	ε_{fd}	3.8 ‰	3.2 ‰

Adjustment factor between determined and indetermined static systems to be considered with $\eta_{rot} \approx 0.83$.

Comparison Schöck Combar® and steel B500



2.9 Cross section of Combar® [DIBt AbZ Z-1.6-238]

⌀ [mm]	Outer diameter [mm]	Design cross section [mm²]	weight [kg/m]	german approval
Straight bar 8	9	50	0.13	yes
Straight bar 12	13	113	0.29	yes
Straight bar 16	18	201	0.52	yes
Straight bar 20	22	314	0.79	yes
Straight bar 25	27	491	1.21	yes
Straight bar 32	34	804	1.94	no
Bent bar 12	15.5	106	0.30	no
Bent bar 16	19.8	191	0.48	no
Bent bar 20	23.8	287	0.69	no

3. Durability and concrete cover

Combar® bars do not corrode and therefore concrete cover is solely required for the transmission of forces between the bars and the surrounding concrete (for all exposure conditions):

$c_{nom} = \varnothing_f + \Delta c$ mit Δc = 10 mm Ortbeton (bei Fertigteilen Δc = 5mm)

For concrete exposion classes acc. to EC2 to be considered.

Concrete cover c_{nom} [mm] for Combar® bars						
⌀	8	12	16	20	25	32
in situ concrete	18	22	26	30	35	42
precast concrete	13	17	21	25	30	37
Minimum concrete cover for fire resistance						
R30	30 mm for all diameters					
R60	50 mm for all diameters					
R90	65 mm for all diameters					
R120	85 mm for all diameters					

4. Bending design with / without axial force

The design is performed by iteration of the strain plain under the same assumptions as those used in the design of steel reinforced concrete members. Material properties for Schöck Combar® are to be considered Due to the low E-modulus Combar® bars must not to be taken as compression reinforcement. Nevertheless Combar® bars may be postioned in the compression zone (anchorage, nominal reinforcement, etc.)
The design value for tensile strength in the design table has ben taken as f_{yd} = 435 N/mm².

4.1 Internal forces

Schöck Combar® bars behave linearly elastic up to failure far above 1000 N/mm². Yielding is not observed Plastic hinges do not form. As a result, the loads on GFRP reinforced concrete elements cannot be determined using plastic limit analysis. As cracked sections transfer increasingly larger loads, moment redistribution is observed only to a very limited extent in Combar® reinforced concrete members.
Moment redistribution should therefore not be considered in the design. For safety reasons non-linear material properties should not be considered in the design. They may be considered in the analysis of members and in the determination of deflections.

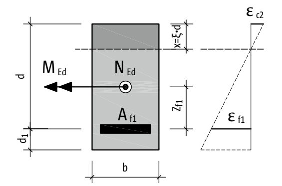
4.2 ω-Table Combar® (f_{fd} = 435 N/mm² for determined static systems)

concrete ≥ C20/25

$M_{Ed1} = M_{Ed} - N_{Ed} \cdot z_{s1}$ (N_{ed} positive as tension force)

$\mu_{Edf} = M_{Ed1} / (b \cdot d^2 \cdot f_{cd})$

req. $A_f = (\omega_1 \cdot b \cdot d \cdot f_{cd} + N_{Ed}) / f_{fd}$



					determined static systems		indetermined static systems	
μ_{Edf}	ω_f	ξ	ς	ε_c [‰]	ε_f [‰]	f_{fd} [N/mm²]	$\eta_{rot} \cdot \varepsilon_f$ [‰]	$\eta_{rot} \cdot f_{fd}$ [N/mm²]
0.001	0.0010	0.017	0.994	-0.123	7.250	435	6.000	360
0.006	0.0061	0.041	0.986	-0.311	7.250	435	6.000	360
0.011	0.0112	0.056	0.981	-0.431	7.250	435	6.000	360
0.016	0.0164	0.068	0.977	-0.529	7.250	435	6.000	360
0.025	0.0258	0.086	0.971	-0.679	7.250	435	6.000	360
0.050	0.0523	0.123	0.957	-1.021	7.250	435	6.000	360
0.075	0.0794	0.154	0.945	-1.321	7.250	435	6.000	360
0.100	0.1070	0.182	0.934	-1.610	7.250	435	6.000	360
0.125	0.1360	0.208	0.922	-1.908	7.250	435	6.000	360
0.150	0.1650	0.235	0.910	-2.229	7.250	435	6.000	360
0.200	0.2270	0.292	0.882	-2.989	7.250	435	6.000	360
0.240	0.2800	0.346	0.856	-3.500	6.605	396	5.466	328
0.250	0.2950	0.364	0.849	-3.500	6.118	367	5.063	304
0.300	0.3710	0.458	0.810	-3.500	4.146	249	3.431	206
0.350	0.4580	0.565	0.765	-3.500	2.692	162	2.228	134
0.360	0.4770	0.589	0.755	-3.500	2.442	147	2.021	121
0.370	0.4970	0.614	0.745	-3.500	2.203	132	1.823	109
0.380	0.5180	0.640	0.734	-3.500	1.973	118	1.633	98
0.390	0.5400	0.667	0.723	-3.500	1.751	105	1.449	87
0.400	0.5630	0.695	0.711	-3.500	1.535	92	1.270	76

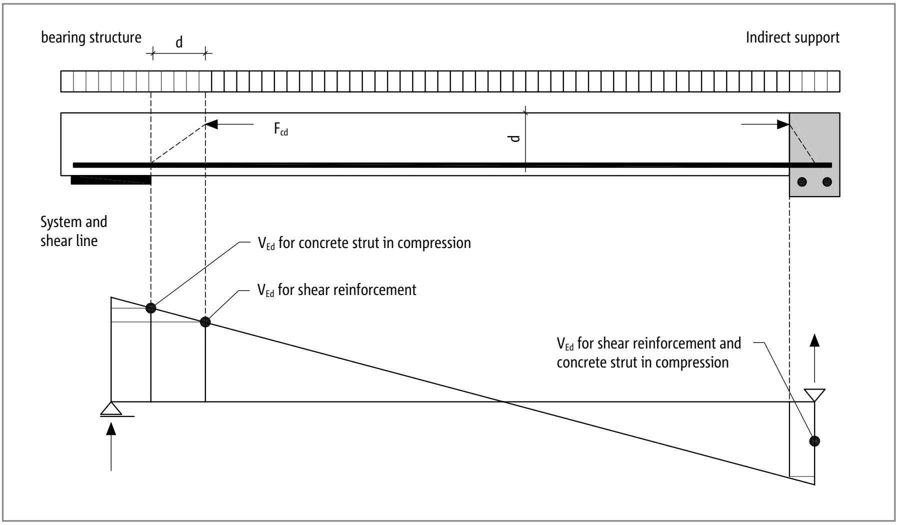
Complete table available on www.schoeck.de

5. Shear design

5.1 Decisive shear force

Most unfavourable positions for shear design

For uniformly distributed load and direct support VEd may be taken at the distance d from support for the design of shear reinforcement. For the case of indirect support VEd must be taken at the axis of support.



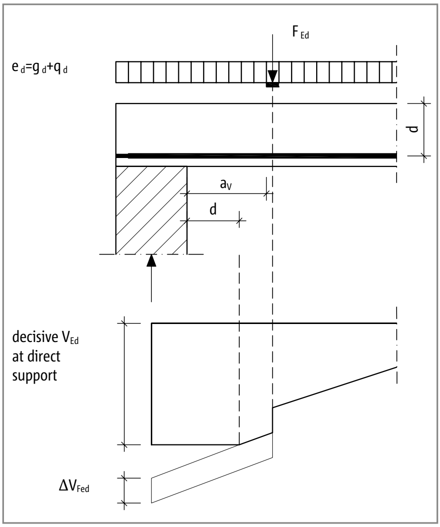
Single loads near direct support ($0.5d \le a_v < 2d$) may be decreased by the factor β_E for shear design.

$$\beta_E = \frac{a_v}{2 \cdot d}$$

Generally it must be fullfilled that (without considera-tion of β_E):

$$V_{Ed} \le 0{,}3375 \cdot b_w \cdot d \cdot f_{cd}$$

When calculating $V_{Rd,max}$ this reduction is not allowed.



5.2 Design for members without shear reinforcement

a) Design according to the german approval (AbZ): $\beta_E \cdot V_{Ed} \leq V_{Rdc}$

Equation 6.2a of BS EN 1992-1-1/NA is replaced for Schöck Combar® by:

$$V_{Rd,c} = \frac{0{,}138}{\gamma_c} \cdot \kappa \cdot (100 \cdot \rho_l \cdot \frac{E_f}{E_s} \cdot f_{ck})^{1/3} \cdot b_w \cdot d$$

where:

γ_c = Partial factor for concrete

$\kappa = 1 + \sqrt{200 / d} < 2{,}0$; (d in mm)

ρ_l = reinforcement ratio for longitudinal reinforcement; $\rho_l = A_{fl} / (b_w\, d) \leq 0.02$

$E_f / E_s = E_{Combar®} / E_{steel}$

f_{ck} = char. compressive cylinder strength of concrete

b_w = width of the web on T, I or L beams

d = effective depth of tension reinforcement

b) Design acc. to Hegger/Kurth (deviant from german approval): $V_{Ed} \leq V_{Rdc}$

$$V_{Rd,c} = \beta_R \cdot \frac{1}{425 \cdot \gamma_c} \cdot \kappa \cdot (100 \cdot \rho_l \cdot E_{fl} \cdot f_{ck})^{1/3} \cdot b_w \cdot d$$

where

$ß_R = \frac{3}{a_V/d} \geq 1$ increasing factor to cover single point loads close to the support

When considering the increasing factor β_R on the resistance side, the decrease factor $ß_E$ on the load side must not be considered.

$\kappa = 1 + \sqrt{200 / d} < 2{,}0$; (d in mm)

ρ_l = reinforcement ratio for longitudinal reinforcement; $\rho_l = A_{fl} / (b_w \cdot d) \leq 0.02$

E_{fl} = E-Modulus of longitudinal reinforcement

f_{ck} = char. compressive cylinder strength of concrete

b_w = width of the web on T, I or L beams

d = effective depth of tension reinforcement

5.3 Design for members with shear reinforcement

The use of Combar® bars as shear reinforcement is not covered by the german approval.
To be on the safe side we recommend (deviant from EC2):

5.3.1 By approximation (Mörsch)

a) Deviant from the german approval for Combar® we recommend the following conservative procedure, which can be done manual. a) The strain in the Combar® shear reinforcement is limited to the same value as that in steel reinforcement: :

$$\varepsilon_f = \varepsilon_s = \frac{435 \text{ N/mm}^2}{200.000 \text{ N/mm}^2} = 0{,}2175 \text{ ‰}$$

This insures that the truss analogy acc. to BS EN 1992-1-1 and BS EN 1992-1-1/NA clause 6.2.3 respectively picture 6.5. is applicable to Combar®. For reinforcing elements Combar® headed bolts or Combar® bent bars can be used. Headed bolts are developped for anchorage, considering a design tensile strength in the bar of f_{fwd} = 130 N/mm². Shear reinforcement (heads) are to be anchored in the compression zone of the concrete. A minimum concrete cover of the shear reinforcement has to be considered. A concrete cover at the end heads (in the axis of the bars) is not required, as they can be placed directly onto the formwork.

b) When calculating $V_{Rd,f}$ and using Schöck Combar® double headed bolts (rectangular to the member axis) equation 6.8 is replaced by:

$$V_{Rd,f} = \frac{A_{fw}}{S_w} \cdot f_{fwd} \cdot z \cdot \cot \Theta$$

5.3.2 Exact procedure according to Prof. Hegger

In the final report „Querkrafttragfähigkeit von Betonbauteilen mit Faserverbundkunststoff-Bewehrung – Ableitung eines Bemessungsansatzes" [Bauingenieur Band 88, Oktober 2013] by M. Kurth und J. Hegger, a more exact, less conservative procedure for the determination of the shear strength of concrete members reinforced with Combar® bars is developed.

In this procedure the load bearing capacity of the concrete and that of the shear reinforcement are added to obtain the overall load bearing capacity of the section.

$V_{Rd} = V_{Rd,c} + V_{Rd,f}$

The load bearing capacity of the shear reinforcement is:

$V_{Rd,f} = a_{fw} \cdot f_{fwd} \cdot z \cdot \cot \Theta$

Where

a_{fw} Querschnittsfläche der FVK-Querkraftbewehrung
f_{fwd} Design value of tensile strength of the FVK-shear reinforcement ($f_{fwd} \leq E_{fw} \cdot \varepsilon_{fwd}$)
z inner lever arm
Θ compression angle (= β_f)

and

$$\varepsilon_{fwd} = 2{,}3 + \frac{2 \cdot EI^* \text{ [MNm}^2\text{]}}{30} \leq 7{,}0 \text{ [‰]}$$

$EI^* = E_{fl} \cdot A_{fl}(0{,}8 \cdot d)^2$

$$\Theta = \arctan \left[\sqrt[3]{\frac{M / V \cdot a_{fw} \cdot E_{fw}}{A_{fl} \cdot E_{fl}}} \right] \left\{ {\geq 20° \atop \leq 50°} \right. \text{; for M use the related value to V}$$

The design value of the shear strength may not exceed the maximum value $V_{Rd,max}$.

$$V_{Rd,max} = V_{Rd,c} + \frac{1{,}1 \cdot b_w \cdot z \cdot f_{cm}^{2/3}}{\gamma_c \cdot (\cot(\Theta) + \tan(\Theta))}$$

The correct moduli of elasticity are to be used for each type of reinforcement depending on whether straight Combar® bars with or without heads (60.000 N/mm²) or bent bars (50.000 N/mm²) are used. The procedure according to Hegger can be used for mixed reinforcement (e. g. longitudinal bars Combar®, stirrups steel).

6. Crack control and minimum reinforcement

6.1 Crack control

Combar® Stäbe don't corrode. Crack control in order to protect the reinforcement is not necessary. In the german approval AbZ Combar® cracks are limited to $w_{k,\perp} \leq 0.4$ mm transverse to the bar axis and to $w_{k,II} \leq 0.2$ mm parallel to the bar axis (only in anchorage zones).

reinforcement purely for crack control based on steel properties can be converted into Combar® reinforcement. Basic term:

$$\frac{w_{k\ ComBAR}}{w_{k\ B500}} = \frac{200.000\ N/mm^2}{60.000\ N/mm^2} \left(\frac{\varnothing_{ComBAR}}{\varnothing_{B500}}\right)\left(\frac{f_{ComBAR}}{f_{B500}}\right)^2 = 1{,}0$$

Applied formula for same bar diameters: req. $A_{ComBAR} = \sqrt{\frac{200}{60}} \cdot A_{B500} = 1{,}83 \cdot A_{B500}$

6.2 Minimum reinforcement for ductile behaviour

Clause 9.2.1.1. of EC2 not valid for Combar®.
A minimum reinforcement for ductile behaviour has to be installed.
Failure of the member at the appearance of first cracks without omen must be prevented. Immediatly after the first crack has appeared the tension in the bar is equal to the tension strength of the concrete $f_{fr} = f_{ctm}$.

Applied formula for flexure only $f_{ctm} = \frac{M_{cr}}{W_c}$ respectively $M_{cr} = f_{ctm} \cdot \frac{b \cdot h^2}{6}$ (rectangle section).

$$A_{f,min} = \frac{M_{cr}}{\sigma_f \cdot z} = \frac{f_{ctm} \cdot W_c}{\sigma_f \cdot z}$$

where $\sigma_f = 0.83 \cdot f_{fk} = 481$ N/mm² (acc. to german approval AbZ, deviant from EC2)

6.3 Maximum reinforcement

The cross section allowed to be considered for bending must not exceed the maximum value of $A_{f,max} = 0.035\ A_c$

7. Limitation of deflection

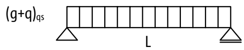
Due to the rather low E-Modulus of Combar® deflection should be carefully examined The German approval gives detailed instructions for the calculation of deflections. The following table below provides a first estimation. (also see: www.schoeck.co.uk)

7.1 Required reinforcement and utilisation at ULS for a deflection of ≤ L/250

One way single span with uniformly distributed load (UDL)

example: For L = 4.5 m and h = 200 mm 130 mm² are required in order not to exceed the maximum allowed deflection. At ULS only 43 mm² are required. Utilisation at ULS is therefore only 33%.

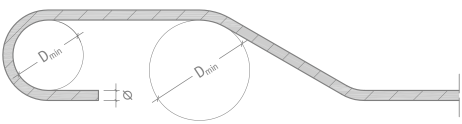
b = 1.0 m
C25/30
q = 3.5 kN/m² and g + $q_{quasi\text{-}permanent}$ = g + 0.3q



slab thickness h	160 mm	180 mm	200 mm	250 mm	300 mm
L = 3.5m L/250 = 14 mm	630 mm² 46%	210 mm²	230 mm²	280 mm²	330 mm²
L = 4.0m L/250 = 16 mm	1630 mm² 24%	1150 mm² 31%	660 mm² 51%	280 mm²	330 mm²
L = 4.5m L/250 = 18 mm	2320 mm² 20%	1700 mm² 27%	1300 mm² 33%	620 mm² 61%	330 mm²
L = 5.0m L/250 = 20 mm		2810 mm² 20%	2140 mm² 25%	1230 mm² 38%	670 mm² 64%
L = 5.5m L/250 = 22 mm			3260 mm² 20%	2150 mm² 27%	1400 mm² 38%
L = 6.0m L/250 = 24 mm				3520 mm² 19%	2320 mm² 27%

= Minimum reinforcement acc. to german approval AbZ Combar® for c_v = 15 mm, d_f = 12 mm; = ρ ≥ 2% (unthrifty)

8. Minimum bending diameter

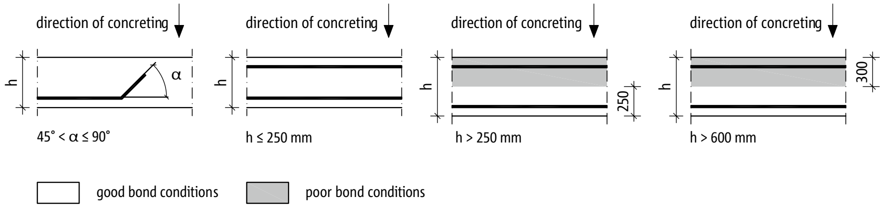


Hooks, loops		
For all ⌀ take D_{min} = 7 • ⌀	⌀ 12 mm	84 mm
	⌀ 16 mm	112 mm
	⌀ 20 mm	140 mm

9. Anchorage

9.1 Bond conditions

Bond conditions acc. to EC2



9.2 design anchorage length

basic anchorage length: $l_{b,rqd} = (\varnothing/4) \cdot (\sigma_f / f_{bd})$ where $\sigma_f = f_{fd} = 445\ N/mm^2$

9.2.1 bond values acc. to german approval (AbZ Z-1.6-238), f_{bd} [N/mm²]

	C12/15	C16/20	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
good bond conditions	1.45	1.77	2.03	2.26	2.33	2.39	2.45	2.51	2.58
poor bond conditions	1.09	1.32	1.53	1.78	2.01	2.23	2.34	2.46	2.58

9.2.2 basic anchorage lengths $l_{b,rqd}$ in mm for straight Combar® bars (acc. to german approval)

Concrete grade	Bond conditions	Bar diameter d_f in mm					
		8	12	16	20	25	32
C 20/25	good	440	660	880	1100	1370	1750
	poor	580	870	1160	1450	1820	2330
C 25/30	good	400	590	790	990	1230	1580
	poor	500	750	1000	1250	1560	2000
C 30/37	good	380	570	760	960	1190	1530
	poor	440	660	890	1110	1380	1770
C 35/45	good	370	560	750	930	1170	1490
	poor	400	600	800	1000	1250	1600
C 40/50	good	360	550	730	910	1140	1450
	poor	380	570	760	950	1190	1520
C 45/55	good	360	530	710	890	1110	1420
	poor	360	540	720	910	1130	1450
C 50/60	good	350	520	690	860	1080	1380
	poor	350	520	690	860	1080	1380

Values for bond stress acc. to table 9.2.1

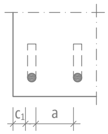
9.3 Anchorage lengths (only tension bars)

$l_{bd} = \alpha_1 \cdot \alpha_5 \cdot l_{b,rqd} \cdot (A_{f,req} / A_{f,prov}) \geq l_{b,min}$

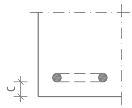
factors α_1 and α_5 (only tension bars)

straight bar end		$\alpha_1 = 1.0$
hooks, loops		$\alpha_1 = 0.7$
loops with $D_{min} \geq 15\ \varnothing$		$\alpha_1 = 0.5$
transverse pressure	c_d [1] $\geq 3\ \varnothing$, others $\alpha_1 = 1.0$	$\alpha_5 = 1 - 0.04 \cdot \rho$ transverse pressure at ULS along l_{bd}

[1] $c_d = \min\{a/2; c_1\}$ for bent or hooked bars



[1] $c_d = c$ for looped bars



9.4 Shift rule

Shift distance for curtailment a_1

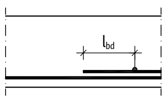
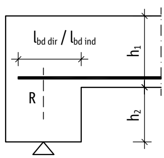
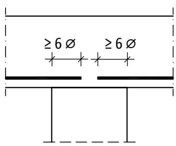
$$a_1 = \frac{z}{2} \cdot \cot\theta \geq 0$$

θ = angle of compression strut

z = inner lever arm; generally it may be taken as $z = 0.9 \cdot d$

For slabs without shear reinforcement $a_1 = 1.0 \cdot d$

Required anchorage lengths

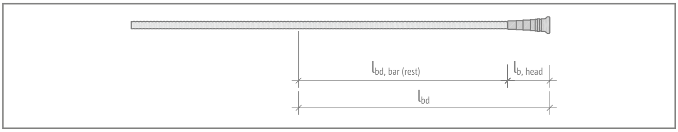
Application		Anchorage length	Annotation
Span		l_{bd}	Anchorage of tension bars $l_{b,min} = 0.3 \cdot \alpha_1 \cdot l_{b,rqd} \geq 10 \cdot \varnothing$
Endsupport direct ($h_1 \leq h_2$) Endsupport indirect ($h_1 > h_2$)		$l_{bd,dir} = l_{bd,ind} = l_{bd}$ $l_{bd} \geq 10 \cdot \varnothing$	At least 25% of the bottom reinforcement of the span must be anchored at the support. l_{bd} is required from the line of contact of the support
support between spans		$l \geq 6 \cdot \varnothing$	l is required from the line of contact of the support

9.5 Reduction of anchorage length by using headed bolts

The required anchorage length of straight Combar® bars can be reduced by installing bars with end heads. The long term bearing capacity of the head (design value) has been identified in creep rupture tests. If the required anchorage load is higher than the bearing capacity of the head, the remaining load has to be transfered by the straight portion of the bar. This means l_{bd} is the sum of the length of the head ($l_{b,head}$) and the length of the straight portion of the bar ($l_{bd, bar}$) which might be required in addition to the head.

$F_d = F_{head, d} + F_{bar, d}$

$l_{bd} = l_{b,head} + l_{bd,bar(Rest)}$



Geometry and design values for Combar® end heads

⌀ bar [mm]	$l_{b, head}$ [mm]	⌀ $head_{end}$ [mm]	anchorage force $F_{head, d}$ [kN]
12	60	30	31
16	100	40	68
20	100	45	85
25	100	50	106
32	100	64	136

9.6 Required reinforcement in end supports

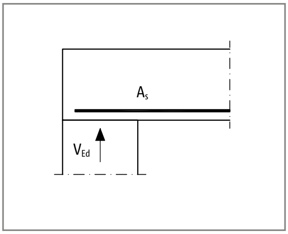
$F_{Ed} = |V_{Ed}|$ [kN] $\cdot (a_1 / z) + N_{Ed}$ [kN] where $z \approx 0.9 \cdot d$

$F_{Ed} \geq 0{,}5 \cdot V_{Ed}$

req. A_f [cm²]$= F_{Ed} / (f_{fd} / 10)$

$a_1 = \frac{z}{2} \cdot \cot\theta \geq 0$ members with shear reinforcement

$a_1 = 1.0 \cdot d \geq 0$ members without shear reinforcement



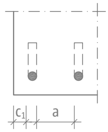
10. Laps of bars (tension bars only)

The german approval does not cover laps of reinforcing bars yet, since up to now not all conceivable geometric combinations in any building environment could be tested and proved. .

10.1 Laps (tension bars only)

$$l_0 = \alpha_1 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b,rqd} \cdot \left(\frac{A_{f,req.}}{A_{f,prov.}}\right) \geq l_{0,min} \quad \geq 20\text{ cm}$$

$$l_{0,min} = 0.3 \cdot \alpha_1 \cdot \alpha_6 \cdot l_{b,rqd} \quad \geq 15\, d_f$$



% lapped bars relative to total cross-section area	< 25 %	33%	50%	>50%
α_6	1.0	1.15	1.4	1.5

11. Provided bars (cross section)

cross section of bars a_f [mm²/m]

bar spacing [mm]	bar diameter d_f [mm]						bars per m run
	8	12	16	20	25	32	
50	1006	2262	4022	6283			20.0
60	838	1885	3352	5236			16.7
70	719	1616	2873	4488	7013	11489	14.3
75	671	1508	2681	4189	6545	10723	13.3
80	629	1414	2514	3927	6136	10053	12.5
90	559	1257	2234	3491	5454	8936	11.0
100	503	1131	2011	3142	4909	8042	10.0
125	402	905	1609	2513	3927	6434	8.0
150	335	754	1341	2094	3273	5361	6.7
200	252	566	1006	1571	2455	4021	5.0
250	201	452	804	1257	1964	3217	4.0

= unthrifty

Cross section of Combar® reinforcement in a beam A_f [mm²]

⌀ bar [mm]	numbers of bars									
	1	2	3	4	5	6	7	8	9	10
8	50	101	151	201	252	302	352	402	453	503
12	113	226	339	452	566	679	792	905	1018	1131
16	201	402	603	804	1006	1207	1408	1609	1810	2011
20	314	628	942	1256	1570	1884	2198	2512	2826	3140
25	491	982	1473	1964	2455	2945	3436	3927	4418	4909
32	804	1608	2413	3217	4021	4825	5629	6434	7238	8042

Max. number of Combar® bars in one layer

The nominal diameter of a Combar® bar corresponds to the core diameter, which has to be taken in calculations. The outer diameter of a Combar® bar exceeds the core diameter by 1 to 2 mm (see table 2.9).

width of beam b [mm]	Nominal diameter [mm]					
	8	12	16	20	25	32
10	2	2	1	1	(1)	-
15	4	3	3	2	(2)	1
20	6	5	4	3	2	2
25	8	6	5	(5)	3	2
30	9	8	6	(6)	4	3
35	11	9	8	7	5	4
40	13	11	9	8	6	5
45	(15)	12	(11)	9	7	5
50	16	14	12	10	8	6
55	18	15	13	11	9	7
60	20	17	14	12	10	8

Values without consideration of stirrups.
bar spacing $a \geq d_f$ respectively ≥ 20 mm (acc. to EC2). Values in () are underrunning the required spacing slightly.

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