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Project TP15-071
Rebend Connection Pyraplex® (serrated protective casing)
Static Calculations according to DIN EN 1992-1-1 with NA(D) and DBV Data Sheet “Rückbiegen von Betonstahl und Anforderungen an Verwahrkästen nach Eurocode 2“ (Rebending of reinforcement steel - requirements on protective casings according to Eurocode 2), January 2011

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This calculation comprises 60 pages and the following annexes:

Contents

PART I EXPLANATIONS OF THE STATIC CALCULATIONS

1 Preliminary remark

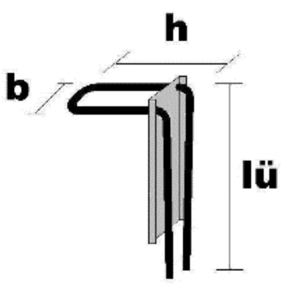
1.1 General

Within the static calculation presented here, the design of the rebend connection Pyraplex® of the company Nevoga is explained according to DIN EN 1992-1-1 with NA(D) and DBV Data Sheet "Rückbiegen von Betonstahl und Anforderungen an Verwahrkästen nach Eurocode 2" (Rebending of reinforcement steel - requirements on protective casings according to Eurocode 2) (Version January 2011) and the design values of the load bearing capacity under the boundary conditions specified here for cases a, b, c, d and e are stated.

Basically, the Pyraplex® types differ in the number of reinforcement layers within one casing (single layer: Type A and double layer: Type B) and in the design of the bar ends, which are offered straight or curved. The surface of the Pyraplex® rebend casing is designed with chequered pyramidal frustums which feature a biaxial shear force transfer both parallel and transverse to the construction joint. According to Figure 6.9 of DIN EN 1992-1-1, this surface condition complies with a serrated joint (cf. also /4/). The application range of the respective type depends largely on the classification into one of the five design cases a, b, c, d and e according to Figure 8 of DBV Data Sheet.

In Part I, after the general explanations for the structural design of rebend connections, the verifications for all six cases are described according to Figure 8 of DBV Data Sheet and the input parameters for the determination of the working load limits tables are given. Part II comprises the WLL tables of those cases which serve as annexes to the type test report.

Pyraplex® reinforcement connections – serrated elements:



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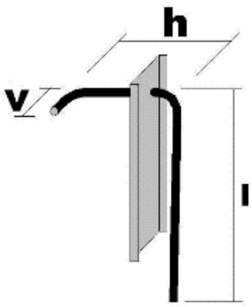

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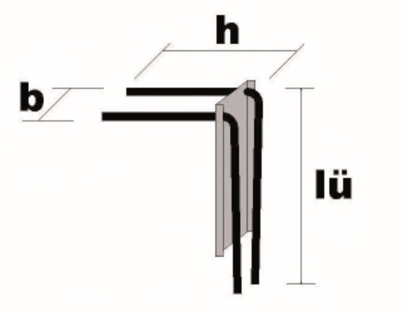


TYPE "DD"

1.2 Overview Pyraplex® protective casings

In the following, the steel plate geometry of the serrated Pyraplex® casings is illustrated.

In the case of the serrated Pyraplex® casings, the arrangement and size of the pyramidal frustums to produce the serration in longitudinal and transverse direction according to Figure 6.9 /1/ is determined by the sizing of the bar spacings s. The steel plate widths are manufactured in the versions 112, 142, 172, 202, 222.

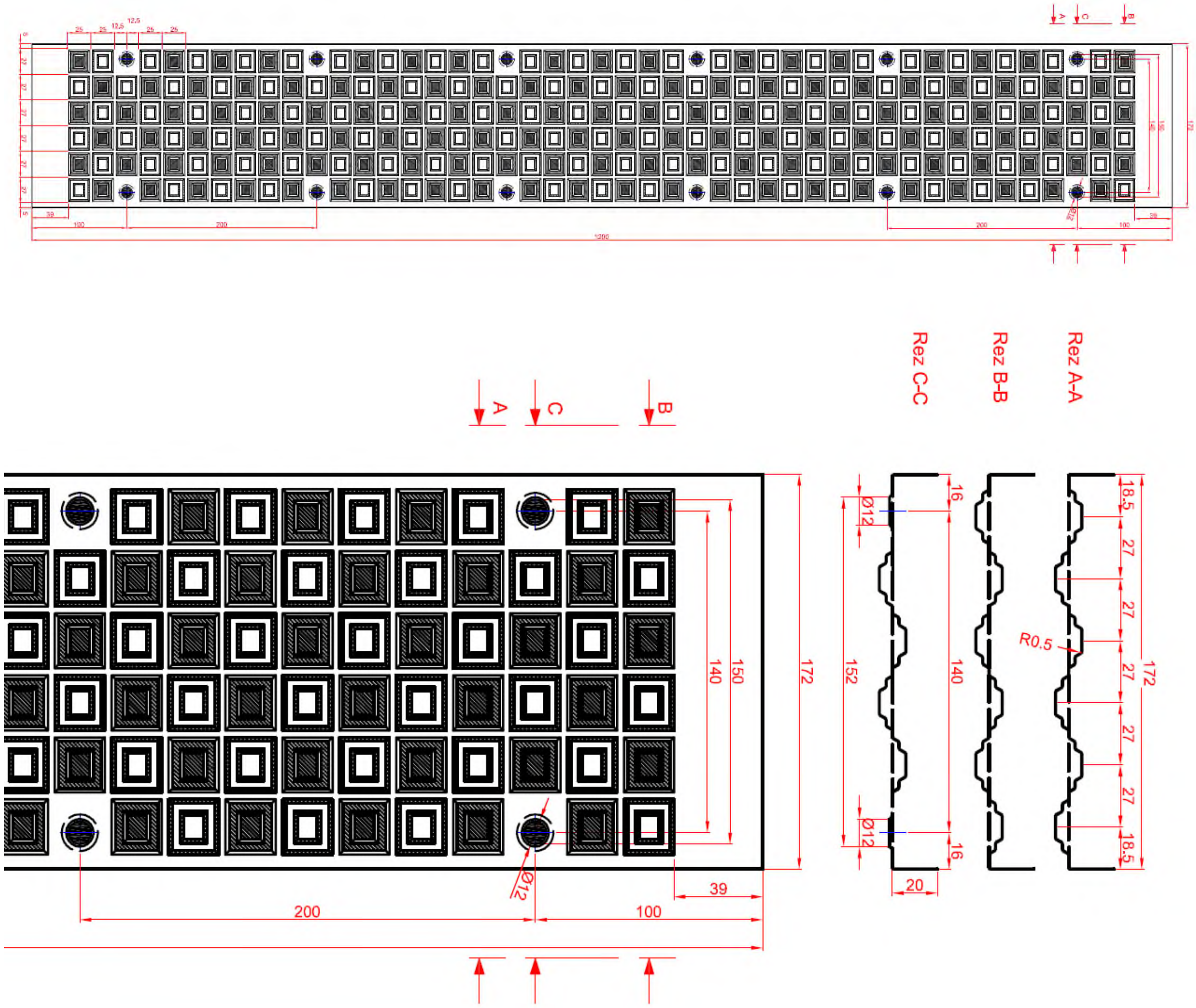


Figure 1 Geometry of Pyraplex® protective casing (b = 172mm) for bar spacing s = 20cm

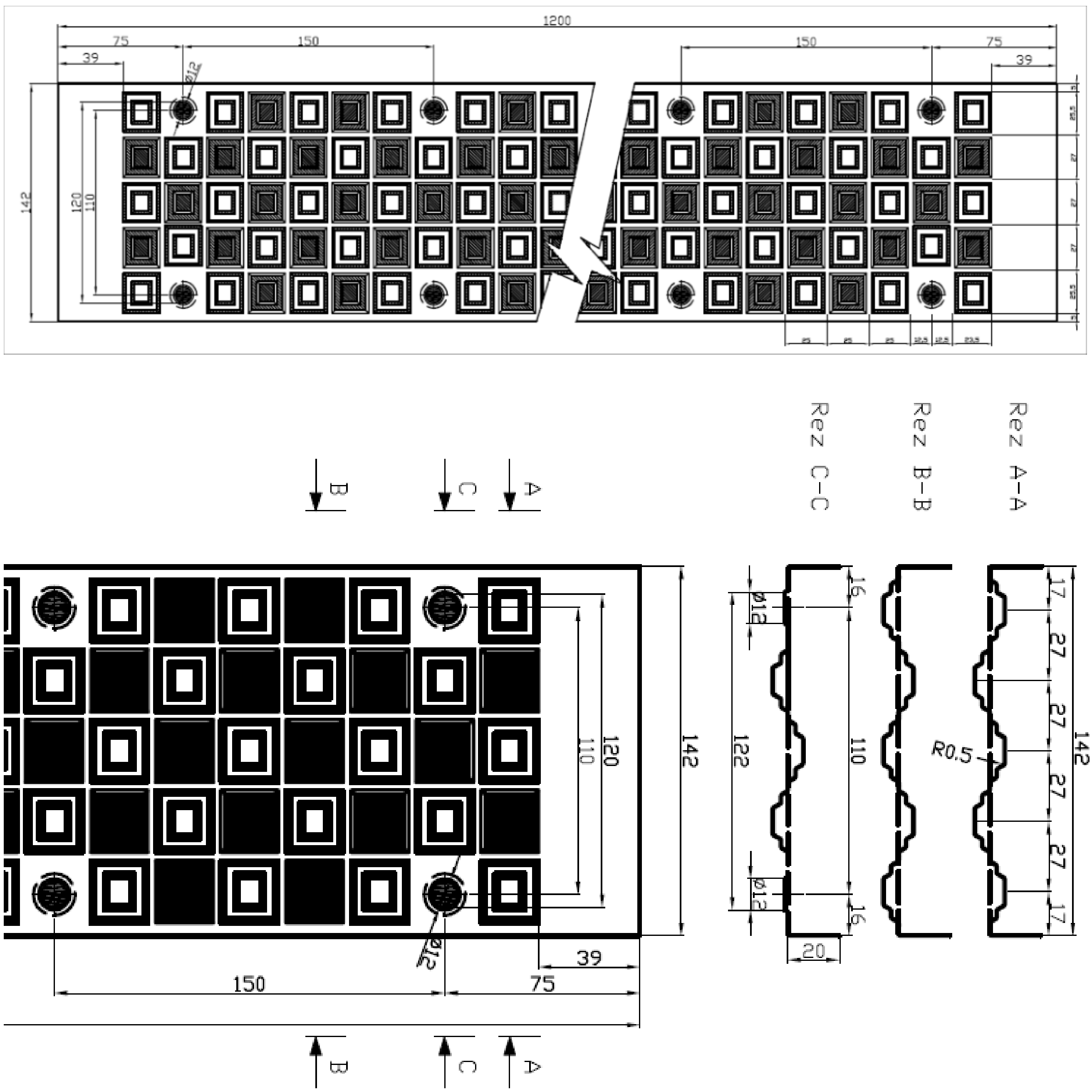


Figure 2 Geometry of Pyraplex® protective casing (b = 142mm) for bar spacing s = 15cm

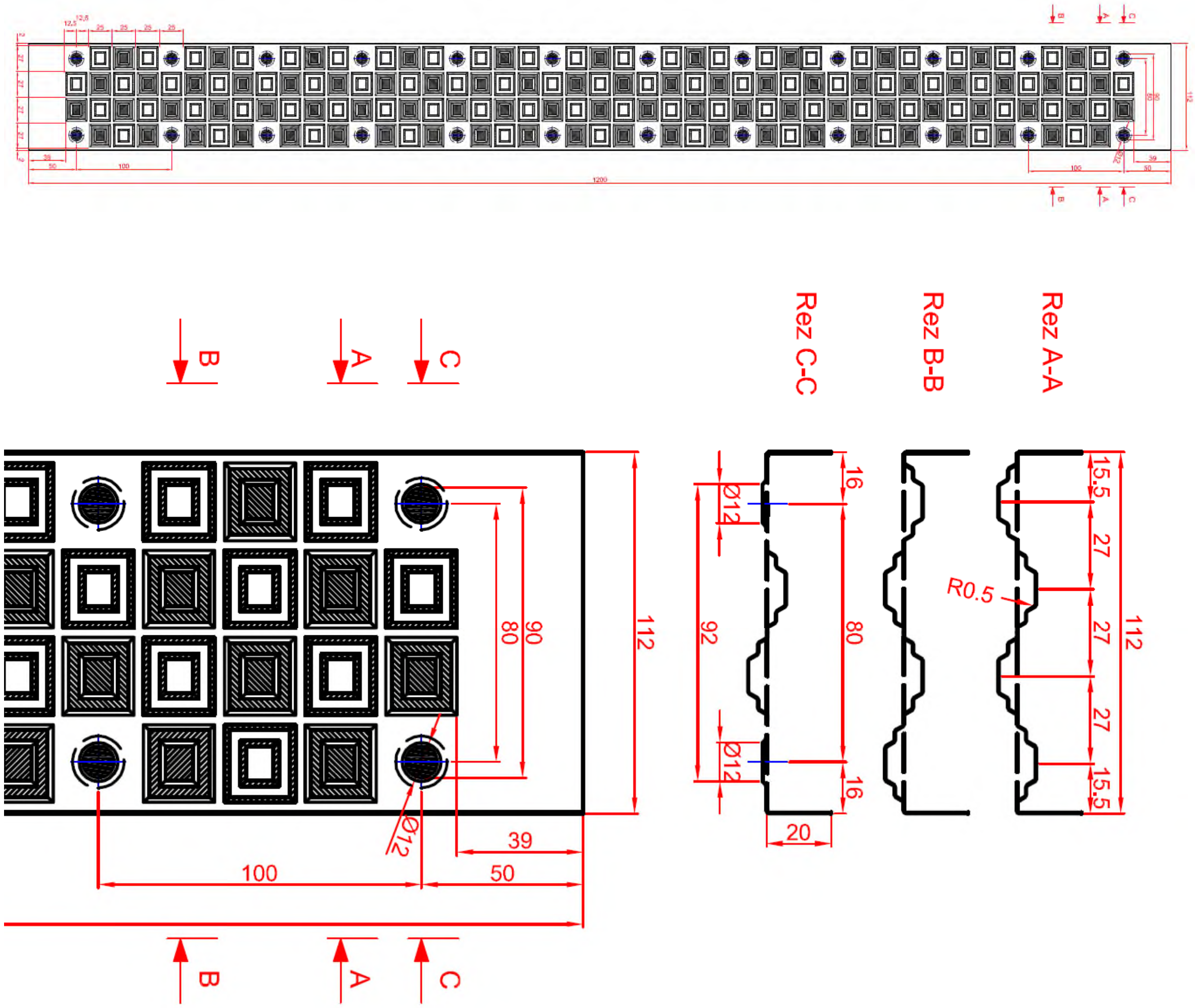


Figure 3 Geometry of Pyraplex® protective casing (b = 112mm) for bar spacing s = 10cm

All other steel plate types are similarly produced from the geometry shown in Figure Bild 6.9 /1/.

2 References and compilation of the documents

2.1 References

The design of the rebend connection Pyraplex® is made on the basis of the following references:

/1/ EC2, DIN EN 1992-1-1, Fassung 2011-01 mit nationalem Anhang EC2, DIN EN 1992-1-1 NA:2011-01 [EC 2]

/2/ Nationaler Anhang EC2, DIN EN 1992-1-1 NA:2013-04 [EC NA(D)] und NA A1 2015-12

/3/ DBV-Merkblatt „Rückbiegen von Betonstahl und Anforderungen an Verwahrkästen nach Eurocode 2“ (Fassung Januar 2011)

/4/ “Stellungnahme zur Einstufung der Oberflächenbeschaffenheit der Pyraplex®-Bewehrungsanschlüsse“ vom 24.02.2015 vom Deutscher Beton- und Bautechnik-Verein E.V.

/5/ „Produktprogramm PYRAPLEX® des Herstellers NEVOGA

2.2 Documents

The present static calculation for the rebend connection PYRAPLEX® according to DIN EN 1992-1-1 and NA(D) comprises the following documents.

- Part I: Explanations of the static calculations
- Part II: Tabular compilation of the shear load resistances for PYRAPLEX® rebend connections according to cases a, b, c, d and e of DBV Data Sheet “Rebending” Figure 8

3 Formula symbols

Forces

F_{Va}	Force that can be transmitted per bar via the anchorage length or overlap length [kN] ($F_{Va,alt}$ – anchorage existing concrete BA I; $F_{Va,neu}$ – overlap new concrete BA II)
F_{sd}	Design force of the rebend reinforcement per bar: $A_{s,PYRAPLEX}$® · 0.8 f_{yd} [kN]

Forces for cases a and b according to /3/, Figure 8

v_{Edi}	Design value of the acting shear stresses in the bond joint [kN/m]
v_{Rdi}	Longitudinal shear strength of a bond joint; additive equation consisting of the shear force percentage resulting from the bond reinforcement and the percentage resulting from friction and adhesion [kN/m]
$v_{Rdi,c}$	Shear strength of the bond joint resulting from friction and adhesion [kN/m]
$v_{Rdi,s}$	Shear strength of the bond joint resulting from bond / shear reinforcement [kN/m]
$v_{Rdi,max}$	Maximum shear resistance of the joint [kN/m]

Forces for cases c, d, e and f according to /3/, Figure 8

V_{Ed}	Design value of the acting shear force [kN]
V_{Rd}	Decisive shear load resistance [kN]
v_{Rd}	Decisive shear load resistance per running meter [kN/m]
$V_{Rd,c}$	Design value of the shear force that can be transmitted without shear reinforcement according to Eq. 6.2a /1/ [kN/m]
$V_{Rd,c}$	Minimum value of the shear load bearing capacity of flexural reinforced building component without shear reinforcement Eq. 6.2b /1/ [kN/m]
$V_{Rd,s}$	Design value of the absorbable shear force limited by the load bearing capacity of the shear reinforcement [kN/m]
$V_{Rd,max}$	Design value of the absorbable shear force limited by the compressive strut strength [kN/m]

Stresses

f_{yd}	Design value of the yield stress of the reinforcing steel [N/mm²]
f_{cd}	Design value of the concrete compressive strength [N/mm²]
f_{ctd}	Design value of the axial tensile force of the concrete [N/mm²]
σ_{cp}	Normal stress in the axis of the building component. For cases a and b according to /3/, Figure 8 (shear force transfer parallel to the joint) the longitudinal normal stress in the total cross section parallel to the joint (negative pressure) [N/mm²]

σ_n	Normal stress perpendicular to the joint in cases a and b according to /3/, Figure 8 (negative pressure) [N/mm²]

Geometry

α	Inclination of the shear reinforcement to the component axis, here always $\alpha = 90°$
θ	Inclination of the compressive strut to the component axis
a_s	PYRAPLEX® rebend reinforcement [cm²/m]
a_{sw}	Shear reinforcement in the connected component in cases c to f according to /3/, Figure 8 [cm²/m²]
b	Effective component width for the transfer of shear force. In cases c to f according to /3/, Figure 8 always 1.0m
d	Effective depth of the component [mm]
$l_{b,rqd}$	Basic value of the anchorage length of the concrete reinforcing steel [mm]
$l_{b,eq}$	Replacement anchorage length of hooks, angled loops and loops [mm]
l_{bd}	Design value of the anchorage length of the concrete reinforcing steel [mm]
l_0	Design value of the overlap length [mm]

Coefficients

c	Roughness coefficient in bond joints (according to /3/ Table 1) [-]
μ	Roughness coefficient in the joint (according to /3/ Table 1) [-]
ν	Reduction coefficient for the concrete compressive strength as a function of the surface condition (according to /3/ Table 1) [-]
η_1	Correction factor for lightweight concrete; here always 1.0 for the exclusive use of general purpose concrete
ρ_l	Longitudinal reinforcement ratio for the determination of $V_{Rd,ct}$ considering the geometric existing reinforcement and the actually anchored force for cases c to f according to /3/ Figure 8.

4 Design bases

4.1 Preconditions

(a) The application range of PYRAPLEX® rebend connections is defined by the application range of EC2 with NA(D) and DBV Data Sheet "Rebending".

(b) In case of different concrete strength classes of the building components assembled by the rebend connection, the lower concrete strength is always applied.

(c) The connected components are predominantly loaded at static load.

(d) The influence of constant tensile stresses resulting from load on or restraint in the concrete joint is not considered within these calculations. Tensile stresses arising from restraint which are not taken into account here must not cause the formation of separating cracks.

(e) The design of the shear force is always based on an angle α of the shear reinforcement of 90°.

(f) Hot bending or hot rebending may become necessary for bar diameter $d_s \geq 16$mm. The PYRAPLEX® programme features a maximum bar diameter $d_s = 12$mm. Therefore, the case of hot rebending is excluded.

(g) The percentage of lapped bars is always > 30%.

(h) The minimum bending roll diameters according to EC NA(D) Table 8.1DE are complied with.

(i) Vertically to the curvature plane of the end anchorage, the concrete cover c of the longitudinal reinforcement as per /1/ Figure 8.3 or $c_d > 3\varnothing$ as per /1/ Table 8.2 is adhered to. Otherwise, $\alpha_1 = 1.0$ must be used as specified in Table 8.2.

(j) Good bond conditions according to EC NA(D) chapter 8.4.2 are always present.

(k) It is assumed that the overlap lengths feature the same bar diameters. The connecting reinforcement may have smaller bar diameters than the rebend reinforcement, provided that this is statically and structurally sufficient.

(l) The planning and execution of the overlap joints corresponds to Figure 8.7 according to /1/. In this case, the clear spacing must not be smaller than 8∅. The only exceptions are Pyraplex® connections with ∅12mm / 100mm.

(m) The design is made for case a according to /2/ Figure 8 for a joint length of 1.0m and for cases c and f for a joint width of 1.0m.

4.2 Material parameters

Concrete reinforcing steel:

BSt 500 B (hot rolled)	f_{yk}	$= 500 N/mm^2$
Safety factor according to /1/	γ_s	$= 1.15$
Design value for cold bending of the reinforcing steel:	f_{yd}	$= 0.8 \cdot f_{yk} / \gamma_s$

Concrete:

C20/25	f_{ck}	$= 20 N/mm^2$
C25/30	f_{ck}	$= 25 N/mm^2$
C30/37	f_{ck}	$= 30 N/mm^2$
Safety factor according to /1/ for concrete:	γ_c	$= 1.5$
Design value concrete	f_{cd}	$= 0.85 \cdot f_{ck} / \gamma_c$
Bond stress f_{bd} according to /1/, Equation (8.2)	f_{bd}	$= 2.25 \cdot \eta_1 \cdot \eta_2 \, \alpha_{ct} \cdot f_{ctk;0.05} / \gamma_c$
Coefficient to consider the long-term effect:		
In general	α_{ct}	$= 0.85$
For determination of bond stresses	α_{ct}	$= 1.0$

5 Compilation of the failure modes and design resistances

5.1 Failure modes

The following failure modes are considered for the individual design cases:

1. Anchorage failure
2. Shear force failure or longitudinal shear failure of the joint ($v_{Rdi,ct}$ with $v_{Rdi,s}$ and $v_{Rdi,max}$)
3. Shear force failure of the component to be connected with or without shear reinforcement ($V_{Rd,c}$ or $V_{Rd,s}$ with $V_{Rd,max}$)

5.2 Anchorage failure

5.2.1. Preconditions and calculation assumptions

Failure modes

Here, anchorage failure means both, the failure of the end anchorage of the reinforcement as well as the failure of the overlap lengths.

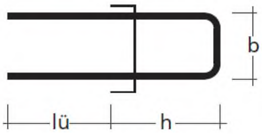
Bond conditions

In the verification, it is assumed for the entire area of the rebend connections that there is a good bond between reinforcement and concrete in accordance with EC NA(D) 8.4.2 (2). Good bond conditions in the upper reinforcement layer also apply for cases d and e according to /3/ Figure 8. The maximum width of the installed PYRAPLEX® connection is 225mm (PYRAPLEX® BW225). Therefore, it can be excluded that, in building practice, the spacing between the upper reinforcement and the lower slab edge is > 300mm.

Design of the bar ends

With regard to the determination of the anchorage or the overlap length, the static calculation follows the below assumption:

- **Straight bar ends** in the building component to be connected are verified with regard to over-lapping using a bar of the same diameter. In this case, the overlap length l_0 is taken as the length of the bar outside the protective casing (see Figure 4):

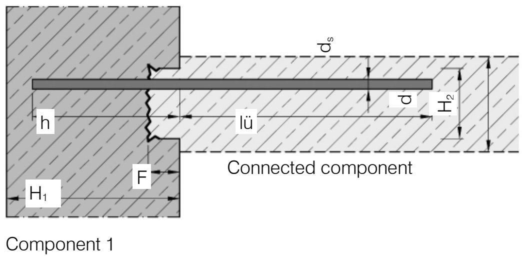


stirrup height (h)

∅ 8 mm h = 17 cm
∅ 10 mm h = 17 cm
∅ 12 mm h = 17 cm

$l_{b,eq}$ or $l_{b,rqd} = h - F$

$l_{0,vorh} = l_ü$



$F = 30$mm for $d_s = 8$mm,
$F = 36$, 40 and 50mm for $d_s = 10$mm,
$F = 36$, 40 and 50mm for $d_s = 12$mm

Figure 4: Definition of the anchored bar lengths of the PYRAPLEX® connection with the designations of the PYRAPLEX® catalogue

The coefficients α_1, α_2, α_3, α_5, α_6 for the overlap length are determined according to EC NA(D) Table 8.2 and 8.3. In principle, a complete tensile lap is carried out, i. e. the percentage of the lapped bars is always >33% and the execution on the building site corresponds to Figure 8.7 as per /1/ in such a way that an α_6 value of 1.0 is sufficient except for connections with ∅12mm / 100mm ($\alpha_6 = 1.4$).

- **Bent bar ends** (Type L) or loops (Type U) in building component 1 are verified as corresponding end anchorage according to EC2 with NA(D) chapter 8.4. It is assumed that the arranged bending roll diameters according to EC NA(D) Table 8.1DE as well as the stirrup width b meet the requirements for angular hooks according to EC NA(D) Figure 8.5DE.

Furthermore, for the verification of the end anchorage, it is also assumed that $\alpha_1 = 0.7$ is always valid for α_1 according to EC 2, Table 8.2, i. e. at least one concrete cover of c or $c_d \geq 3∅$ is given perpendicular to the plane of curvature.

If, according to NA(D) to Table 8.2, transverse tension is given at a right angle to the reinforcement plane and no limitation of the crack width to $w_k = 0.2$ mm has been proven (close position of the stirrups), then $\alpha_1 = 1.5$ shall be applied.

5.2.2. Verification of the anchorage / overlap lengths according to EC 2 with NA(D)

The verification of the anchorage and overlap length of concrete reinforcing steel is made according to EC 2 with NA(D). The required anchorage length is regulated according to chapter 8.4.4 and the required overlap length according to chapter 8.7.3. Both verifications are based on the basic value of the anchorage length $l_{b,rqd}$ or $l_{b,eq}$ which is used to determine the required anchorage length l_{bd} and the required anchorage length l_0 according to EC 2.

The basic value of the anchorage length $l_{b,rqd}$ is calculated according to EC 2 NA(D) Eq. (8.3) as follows:

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd}}$$

where
- Ø diameter of the reinforcing bar
- σ_{sd} existing steel stress in the ultimate limit state of the bar at the beginning of the anchorage length
- f_{bd} $= 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd}$ according to EC 2, Eq. (8.2)

and thus corresponds to the length required to transfer the force from the rebar $F_{sd} = A_s \cdot f_{yd}$ into the concrete, assuming a constant bond stress f_{bd}.

The required anchorage length l_{bd} results according to EC 2, Eq. (8.4) and NA(D), (NCI) to 8.4.4 (1) as follows

$$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} \geq l_{b,\min}$$

where
- α_1 = coefficient for the anchorage type of the bars assuming sufficient concrete cover
- α_2 = coefficient for the minimum concrete cover according to EC NA(D) Figure (8.3)
- α_3 = coefficient for a transverse reinforcement
- α_4 = coefficient for one or more welded cross bars
- α_5 = coefficient for a pressure transverse to the splitting-tensile-crack plane within the required anchorage length
- $l_{b,rqd}$ = basic value from EC NA(D) Eq. (8.3)

representing a reduced value of $l_{b,rqd}$ depending on utilisation ratio and effectiveness of the anchorage.

Furthermore, the length used for anchorage must at least correspond to a structural minimum dimension of:

$l_{b,min} \geq \max\{0.3 \cdot \alpha_1 \cdot \alpha_2 \cdot l_{b,rqd};\ 10 \cdot Ø,\ 100mm\}$ for anchorages under tension

$l_{b,min} \geq \max\{0.6 \cdot l_{b,rqd};\ 10 \cdot Ø,\ 100mm\}$ for anchorages under compression

According to NA(D) (NCI) to 8.4.4 (1), the minimum value 100mm may be fallen below for anchorages under compression and the minimum value 10Ø may be reduced to 6.7Ø at direct support.

The required overlap length l_0 is calculated according to EC 2 with NA(D) Eq. (8.10) as follows:

$$l_0 = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b,rqd} \geq l_{0,\min}$$

where α_6 = coefficient for the percentage of lapped bars according to EC NA(D) Table 8.3DE

This value may not fall below the minimum value of the overlap length $l_{0,min}$ under consideration of (NCI) to 8.7.3 (1):

$l_{0,min} = \max \{0.3 \cdot \alpha_1 \cdot \alpha_4 \cdot \alpha_6 \cdot l_{b,rqd};\ 15 \cdot \varnothing;\ 200mm\}$

Table 1 Coefficient α_6 according to Table NA 8.3 from /2/

Lap	Bar - Ø	Lap percentage of a reinforcement layer	
		≤ 33 %	> 33 %
Tension	< 16 mm	1.2 [a]	1.4 [a]
	≥ 16 mm	1.4 [a]	2.0 [b]
Compression	all	1.0	1.0
If the clear bar spacings a ≥ 8 Ø (Figure 8.7) and the edge distance in the lap plane c_1 ≥ 4 Ø (Figure 8.3) are adhered to, the coefficient α_6 may be reduced to:			
[a] $\alpha_6 = 1.0$			
[b] $\alpha_6 = 1.4$			

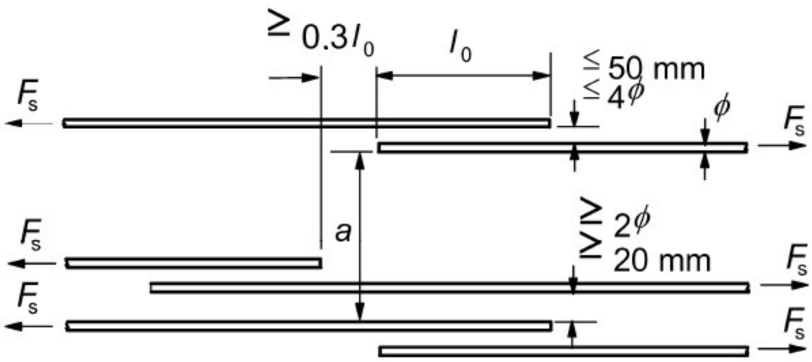


Figure 5 Arrangement of the lapped bars according to EC 2, Figure 8.7

In this type statics it is assumed that the majority of the reinforcement ratios fulfil the condition a ≥ 8∅ and that $\alpha_6 = 1.0$ may be applied when calculating the Pyraplex® rebend connections. Exceptions are all connections with ∅ 12mm / 10mm. This must be taken into account both during planning as well as execution.

5.2.3. Verification for PYRAPLEX® rebend connections

The verification of the anchorage or overlap lengths of PYRAPLEX® rebend connections is made according to EC 2 with NA(D) as described above.

As standard, PYRAPLEX® rebend connections are manufactured with a length h of 170mm (h according to Figure 4). In most cases, this length is not sufficient to transfer the bar force F_{sd} to the concrete with f_{yd} = 435 Mpa. In this case, according to DIN EN 1992-1-1 chapter 8.4.3 (2) Equation 8.3, the actual steel stress σ_{sd} may be considered. From Equation 8.3 of EC 2 and $F_{va} = A_s \cdot \sigma_{sd}$ it follows for the anchored force F_{va} per bar spacing s:

$$\frac{F_{va}}{s} = f_{bd} \cdot \frac{\pi \cdot \phi}{s} \cdot l_{bd} \qquad \text{in [kN/m]} \qquad \textit{(Eq. 1)}$$

where s = bar spacing

$$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd}$$

where $l_{b,rqd}$ length of the bar according to Figure 4.

Analogously, the maximum transferable force for overlaps is determined by omitting the coefficient α_4 and additionally taking into account the coefficient α_6.

Within the calculation, the minimum values of the anchorage lengths are verified:

Direct support:

$l_{b,min} = 6.7 \cdot Ø$ or $l_{b,min} = 100$mm

Others:

$l_{b,min} = 10 \cdot Ø$

and $l_{b,min} = 0.3 \cdot \alpha_1 \cdot \alpha_4 \cdot l_{b,rqd}$

To determine $l_{b,min}$ using $l_{b,rqd}$, $\sigma_{sd} = f_{yd}$ is conservatively set according to page 12.

Analogously, the minimum values of the overlap length are determined by considering the coefficient α_6 as well as the extended requirements on $l_{s,min} = 15 \cdot Ø$ and $l_{s,min} = 200$mm.

5.2.4. Anchorage at a shear force transverse to the joint and direct support with shear reinforcement in the connected component:

The anchorage at the end bearing according to DIN EN 1992-1-1, chapter 9.2.1 assumes a monolithic design of the component connection. Equation (9.3) /1/ takes into account the difference between the tension flange forces according to the truss analogy and the beam theory and must be fully anchored to the support according to standard.

$$\Delta F_{Ed} = |V_{Ed}| \cdot a_l/z + N_{Ed} \geq V_{Ed} / 2$$

where $a_l = z/2 \cdot (\cot\theta - \cot\alpha) \geq 0$

z = internal lever arm

The force to be anchored is decisively determined by the truss model of a net truss which serves as a basis. The protective casings with a smooth joint surface do not fulfil the requirements according to /1/ chapter 6.2.5 (NA.6), therefore a simple truss model is taken as a basis for the verification of the anchorage. This results in the requirement to anchor the entire force F_{Ed} from the truss analogy.
For protective casings with a serrated joint, however, a net truss can form at the end bearing, since the concrete compressive struts can fan out at the “serrations” of the joint surface.

In case c according to DBV Data Sheet, this truss network is used as a basis for a serrated casing. For the longitudinal and transverse spacings, the regulations according to Tables NA.9.1 and NA.9.2 of DIN EN 1992-1-1, line 2 shall be adhered to.

without shear reinforcement in the connected building component:

The anchorage at the end bearing without shear reinforcement is made in compliance with DIN EN 1992-1-1, chapter 9.2.1.3 (2), according to which the offset dimension $a_l = d$ is to be used for Equation (9.3). It applies:

$$\Delta F_{Ed} = |V_{Ed}| \cdot a_l/z + N_{Ed} = V_{Ed} \cdot 1.0\ d/z + N_{Ed} = V_{Ed} \cdot 1.11 + N_{Ed} \geq V_{Ed} / 2$$

5.3 Shear load resistances

5.3.1. General distinction of cases

The shear load resistance of a cross section in the area of rebend connections is determined according to EC2 with NA(D) chapter 6.2 or according to DBV Data Sheet “Rebending”, chapter 5.3. Based on Figure 8 in DBV Data Sheet, the design is divided into six cases. These can be classified into two groups of shear force transfer:

(1.) A shear force is transferred parallel to the joint.

(2.) A shear force is transferred perpendicular to the joint.

On the one hand, the two groups differ with regard to the verification – the standard specifies different formulae for the shear forces that can be transferred – and on the other hand with regard to the function of the rebend reinforcement. In group 1, the rebend reinforcement forms the bond reinforcement of the joint, in group 2, the rebend reinforcement serves as the longitudinal reinforcement of the connected building component. The design concepts of both groups are explained in the following. Specific features of the cases shown in Figure 8 of DBV Data Sheet are furthermore described in chapter 6.

5.3.2. Shear force transfer parallel to the concrete joint

A significant change as compared to the previous concept is the introduction of an additive equation for the design resistance v_{Rdi}. The percentage contact area of the bond joint is formed by the percentage contact area of the concrete resulting from friction and adhesion as well as the load bearing mechanisms of the bond reinforcement.

In compliance with the previous nomenclature, the percentage contact area of the concrete resulting from friction and adhesion is referred to as $v_{Rdi,c}$.

The shear capacity in longitudinal direction of grout joints between floor slabs and wall elements (in-plane effect) may be determined accordingly. When the joints are predominantly cracked, however, c = 0 must generally be applied for smooth and rough joints and c = 0.5 for serrated joints.

This case is not examined for the type statics.

Transferable longitudinal shear force with disregard of the bond reinforcement:

If the bond reinforcement is not taken into calculation, the design value of the shear force that can be transferred must be determined according to Eq. (6.25) of DIN EN 1992-1-1 without the percentage contact area $v_{Rdi,s}$:

$$v_{Rdi,c} = c \cdot f_{ctd} + \mu \cdot \sigma_n$$

where c roughness coefficient as a function of the surface condition of the building component that transfers the shear force according to Figure 8 /3/ (see chapter 5.4.1)

f_{ctd} design value of the concrete tensile strength according to 3.1.6 (2)P /1/

μ friction coefficient as a function of the surface condition of the building component that transfers the shear force according to Figure 8 /3/ (see chapter 5.4.1)

σ_n stress resulting from the minimum normal force orthogonal to the joint which can act simultaneously with the shear force (positive for compression where $\sigma_n < 0.6\ f_{cd}$ and negative for tension). If σ_n is a tensile stress, generally $c \cdot f_{ctd}$ should be set to 0

If the grout joint features a smoother surface condition than the protective casing, the transferable shear force can be determined in two different ways:

1. b = width of the casing + applicable width of the concrete joint with the surface coefficients c and μ of the concreting surface
2. b = width of the casing with c and μ of the casing surface

Further details can be found in chapter 6.

Transferable longitudinal shear force with applicable bond reinforcement

When applying bond reinforcement, the transferable longitudinal shear force according to Equation (6.25) of DIN EN 1992-1-1 is supplemented by the additive percentage contact area of the bond reinforcement. This results from a truss model consisting of a concrete compressive strut and a tension strut made of reinforcing steel (in this case rebend reinforcement). Both components must be verified:

<u>Design value of the percentage contact area of the bond reinforcement $v_{Rdj,sy}$ according to EC Eq. 6.25:</u>

$$v_{Rdi,s} = \rho \cdot f_{yd} \cdot (1.2 \cdot \mu \cdot \sin\alpha + \cos\alpha)$$

where ρ $= A_s / A_i$
cross-sectional area of the PYRAPLEX® rebend connection per unit length to be applied with the anchorage ratio. The anchorage ratio must be determined analogously to chapter 5.2.3.

f_{yd} design value of the yield stress of the reinforcing steel according to chapter 4.2

α angle between joint and rebend reinforcement, here always 90°

μ friction coefficient as a function of the surface condition of the component transferring the shear force according to Figure 8 /3/ (see chapter 5.4.1)

<u>Design value of the maximum shear resistance of the joint $v_{Rdi,max}$:</u>

$$v_{Rdi,max} = 0.5 \cdot \nu \cdot f_{cd}$$

where ν reduction coefficient for the concrete compressive strength as a function the surface condition.

It applies:

$\nu = 0.70$ for serrated joints;

$\nu = 0.50$ for rough joints;

$\nu = 0.20$ for smooth joints;

$\nu = 0$ for very smooth joints (the friction percentage $\mu \cdot \sigma_n$ in Eq. 6.25 of DIN EN 1992-1-1 may be used; however, v_{Rdi} must not exceed the value $v_{Rdi,max}$ for smooth joints).

f_{cd} = design value of the concrete compressive strength

5.3.3. Shear force transfer perpendicular to the joint

If the shear force is transferred perpendicular to the joint, the design of this area is carried out according to DIN EN 1992-1-1 chapter 6.2.2 (no shear reinforcement in the connected component) or according to chapter 6.2.3 (shear reinforcement in the connected component). In this case, the rebend reinforcement forms the longitudinal reinforcement of the connected component.

Transferable shear force without shear reinforcement in the connected component

According to DIN EN 1992-1-1, the design value of the shear force is determined without mathematically required shear reinforcement according to Eq. (6.2a). This standard forms the basis for the design of monolithic building components. As a result of the shear force transfer via a defined joint, the value according to Eq. (6.2a) must be reduced in the ratio of the roughness coefficient c / 0.5 due to the less favourable surface condition of the joint (cf. also /2/, chapter 6.2.5 (NA.6)):

$$V_{Rd,c} = [\, C_{Rd,c} \cdot \kappa \cdot (\, 100\, \rho_l \cdot f_{ck}\,)^{1/3} + \kappa_1 \cdot \sigma_{cp}\,] \cdot b_w \cdot d$$

where $C_{Rd,c}$ $= 0.15 / \gamma_c$ according to DIN EN 1992-1-1, 6.2.2 (1) (NDP)

κ $= 1 + \sqrt{200 / d} \leq 2.0$

ρ_l = longitudinal reinforcement ratio anchored according to Figure 6.3, DIN EN 1992-1-1 (see below)

$= A_{sl} / (b_w \cdot d) \leq 0.02$

A_{sl} = Area of the tensile reinforcement (rebend reinforcement), which is placed at least (l_{bd} + d) beyond the considered cross section.

σ_{cp} longitudinal normal stress in the axis of the total cross section (negative concrete compressive stress)

$= N_{Ed} / A_c < 0.2 \cdot f_{cd}$

where A_c = contact surface of the connected component (casing plus concrete joint)

b_w $= 1.0$m

d = effective depth of the connected component

According to DIN EN 1992-1-1, a minimum value of the shear load bearing capacity $V_{Rd,c}$ of components with flexural reinforcement and without shear reinforcement may be taken into account as specified in Eq. (6.2b). This value forms the minimum shear load bearing capacity for the design of monolithic components. Due to the shear force transfer via a defined joint, the value according to Eq. (6.2b) must be reduced in the ratio of the roughness coefficient c because of the more unfavourable surface condition of the joint:

$$V_{Rd,c} = (c_j / 0.5) \, [v_{min} + \kappa_1 \cdot \sigma_{cp}] \cdot b_w \cdot d$$

where v_{min} $= (0.0525/ \gamma_c) \cdot k^{3/2} \cdot f_{ck}^{1/2}$ for $d \leq 600$mm

$= (0.0375/ \gamma_c) \cdot k^{3/2} \cdot f_{ck}^{1/2}$ for $d > 800$mm

(intermediate values may be interpolated)

γ_c = partial safety factor for reinforced concrete according to 2.4.2.4 (1), Table 2.1DE of DIN EN 1992-1-1 /1/

κ scale factor where $\kappa = 1+\sqrt{(200/d)}$ 1 ≤ 2.0

b_w = smallest cross-sectional width within the tension zone of the cross section

d = effective depth of the flexural reinforcement in the considered cross section

f_{ck} = characteristic value of the concrete compressive strength

σ_{cp} = design value of the longitudinal concrete stress at the centre of gravity of the cross section where $\sigma_{cd} = N_{Ed} / A_c < 0.2 \cdot f_{cd}$

N_{Ed} = design value of the longitudinal force in the cross section resulting from external effects or prestressing ($N_{Ed} > 0$ as longitudinal compressive force)

κ_1 = 0.12

For the design, the minimum shear load bearing capacity $V_{Rd,c}$ is compared to the shear load bearing capacity $V_{Rd,c}$. In accordance with the standards, the higher value may be applied for the calculated load bearing capacity.

Transferable shear force with shear reinforcement in the connected component

In principle, the determination of the transferable shear force follows DIN EN 1992-1-1 chapter 6.2.3. This design is based on a truss model illustrated in Figure 6.5 of the standard. The rebend reinforcement represents the horizontal tension chord of this truss.

According to DIN EN 1992-1-1 Eq. (6.7aDE), the inclination of the compressive strut as well as the restriction of the maximum inclination to $\cot \theta = 1.0$ as per /1/ may be freely selected within the following limitations. In the connected components, the inclination must be ensured at a distance d/2 from the joint:

$$1.0 \leq \cot\theta \leq \frac{1.2-1.4\sigma_{cd}/f_{cd}}{1-V_{Rd,cc}/V_{Ed}} \leq 3.0$$ for general purpose concrete

where σ_{cd} longitudinal normal stress in the axis of the total cross section (negative concrete compressive stress)

$= N_{Ed} / A_c$ where A_c = contact surface of the connected component (protective casing plus concrete joint)

f_{cd} design value of the concrete compressive strength

$V_{Rd,cc}$ $= 0.48 \cdot c \cdot f_{ck}^{1/3} (1 - 1.2 \cdot \sigma_{cd} / f_{cd}) \cdot b_w \cdot z$

c roughness coefficient of the casing or the concrete joint as specified in 6.2.5 (2) of DIN EN 1992-1-1

z internal lever arm of the flexural design of the connected component $= 0.9$ d. The value for z may however not exceed $d - 2c_{v,l} \geq d - c_{v,l} - 30$mm according to DIN EN 1992-1-1 chapter 6.2.3 (1)(NCI) ($c_{v,l}$ = actual concrete cover of the longitudinal reinforcement in the concrete compression zone).

b_w smallest cross-sectional width within the tension zone of the cross section

V_{Ed} design value of the acting shear force

A calculated value cot $\theta < 1$ is inadmissible!

Design value of the transferable shear force $V_{Rd,s}$ as per Eq. (6.8) (load bearing capacity of the bond reinforcement):

$$V_{Rd,s} = (A_{sw} / s) \cdot z \cdot f_{ywd} \cdot \cot \theta$$

where A_{sw} cross-sectional area of the shear reinforcement in the connected component

s distance of the shear reinforcement measured in the direction of the component axis

f_{ywd} design value of the yield stress of the reinforcing steel

z internal lever arm of the flexural design $= 0.9\,d \leq d - 2c_{v,l} \geq d - c_{v,l} - 30$mm as per DIN EN 1992-1-1 chapter 6.2.3 (1)(NCI)

Design value of the maximum shear force $V_{Rd,max}$ analogous to Eq.(6.9) (load bearing capacity of the compressive strut):

$$V_{Ed} \leq 0.3 \cdot V_{Rd,max} = 0.3 \cdot b_w \cdot z \cdot \nu_1 \cdot f_{cd} / (\cot \theta + \tan \theta)$$

where 0.3 reduction of the maximum compressive strut bearing capacity according to /3/

b_w smallest cross-sectional width within the tension zone of the cross section

ν_1 reduction coefficient for the concrete strength at shear cracks
$\nu_1 = 0.75 \cdot \nu_2$ $\nu_2 = (1.1 - f_{ck}/ 500) \leq 1.0$

α angle between component axis and shear reinforcement; here always 90°

f_{cd} design value of the concrete compressive strength

Load bearing capacity of the tension chord in the truss

PYRAPLEX® rebend connections must be able to transfer the horizontal tensile forces of the truss. It must be verified if the horizontal forces can be anchored according to the truss analogy or transferred by a full lap joint of the reinforcement. The transferable bar force for a given bar length is determined as specified in chapter 5.2.3. The force to be transferred can be calculated from the selected inclination of the compressive strut as follows:

$$F_{sd} / s = V_{Ed} \cdot \cot \theta + \sigma_{cd} \cdot h \leq F_{va} / s \qquad \text{(as per chapter 5.2.3)}$$

5.4 Explanations to determine the shear load resistances

5.4.1. Surface condition

The shear load resistance of the joint depends on the roughness of the surface. Four different surface conditions are specified in DIN EN 1992-1-1 /1/ chapter 6.2.5 (2):

- very smooth: the surface was cast against steel, plastic or wooden formwork. Untreated joint surfaces shall be classified as very smooth joints when using concrete with a flowable or highly flowable consistency in the first concreting step (spread class ≥ F5).

- smooth: the surface was trowelled or produced by slipform or extruding method or it remained without any further treatment after compaction.

- rough: a surface with a roughness of at least 3mm produced by raking with a spacing of at least 40mm or produced by suitable exposure of the aggregates or by other methods which lead to an appropriate load bearing behaviour; as an alternative, the surface may feature a defined roughness[10)].

- serrated: when the geometry of the serration corresponds to the specifications in Figure 6.9 /1/. When an aggregate with $d_g \geq 16$mm is used and the grain structure is exposed for at least 6mm, the joint may be classified as serrated[10)].

[10)] With regards to the definition of the surface roughness, see also DAfStb Heft 525. The roughness parameter for the classification into the category “rough” should be $R_t \geq 1.5$mm as mean roughness depth according to the sand patch method by Kaufmann or $R_p \geq 11$mm as maximum profile peak height. The roughness parameter for the classification into the category “serrated” should be $R_t \geq 3.0$mm as mean roughness or $R_p \geq 2.2$mm as maximum profile peak height. The values should be verified as mean values of at least three measurements.

The coefficients c, μ and ν resulting from the surface condition are specified in Figure 8, Table 1 of DBV Data Sheet as follows:

Joint surface	c [1)]	μ	ν [3)]
serrated	0.50	0.9	0.70
rough	0.40 [2)]	0.7	0.50
smooth	0.20 [2)]	0.6	0.20
very smooth	0	0.5	0 [4)]

1) At dynamic or fatigue load, the concrete bond (adhesion) must not be taken into account ($c = 0$).

2) If tension arises resulting from actions at right angles to the joint, the value c = 0 must be set.

3) For concrete strength classes ≥ C55/67, the values must be multiplied with the factor ($1.1 - f_{ck}/500$) with f_{ck} in [N/mm]².

4) For very smooth joints, the friction percentage in Eq. 6.25 may be used up to the limit $\mu \cdot \delta_n \leq 0.1 f_{cd}$.

The condition of the surface is considered in all calculations of the absorbable load bearing capacity of joints reinforced with PYRAPLEX®rebend connections (all cases a to f according to Figure 8, DBV Data Sheet).

The following surface conditions were assumed for the calculation:

Protective casing	Concrete joint
serrated	smooth

The surface condition of the concrete surface must be indicated in the drawings. If it is not possible to ensure a smooth surface according to Heft 525 of DAfStb, the load bearing capacity for a very smooth surface must be determined; the table values in Part II are no longer valid.

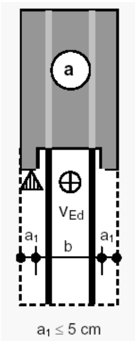
5.4.2. Normal stress on the joint

The shear load resistance is influenced by normal stress on the joint. These normal stresses can be caused by restraint (temperature, shrinkage, etc.) as well as by external loads (prestressing, normal forces, etc.). In the tables given in Part II, normal stresses are always excluded. However, it is basically possible to make a design for this. It must be considered that the normal stresses do not cause the formation of a separating crack in the joint. Therefore, in the following explanation of the shear load resistance of the joint, normal stresses are taken into account in the formulae.

6 Compilation of the design resistances analogous to DBV Data Sheet “Rebending”

6.1 Case a Shear force parallel to the concrete joint

6.1.1. Description of the case as specified in DBV Data Sheet



The shear force is transmitted parallel to the concrete joint. Only two-layer PYRAPLEX® types are used. Due to the design of the concrete joint as illustrated in the adjacent figure, the compression strut is mainly supported by the protective casing of the PYRAPLEX® connection. According to DBV Data Sheet, chapter 5.3(3), the concrete joints at the side of the casing are only regarded as load bearing from a width $a_1 \geq$ 5cm.The surface conditions of the casing and the concrete joint determine the roughness coefficient which is included in the design of the load bearing capacity of the joint.

6.1.2. Mechanical model

The truss model as illustrated below forms the basis of the design concept:

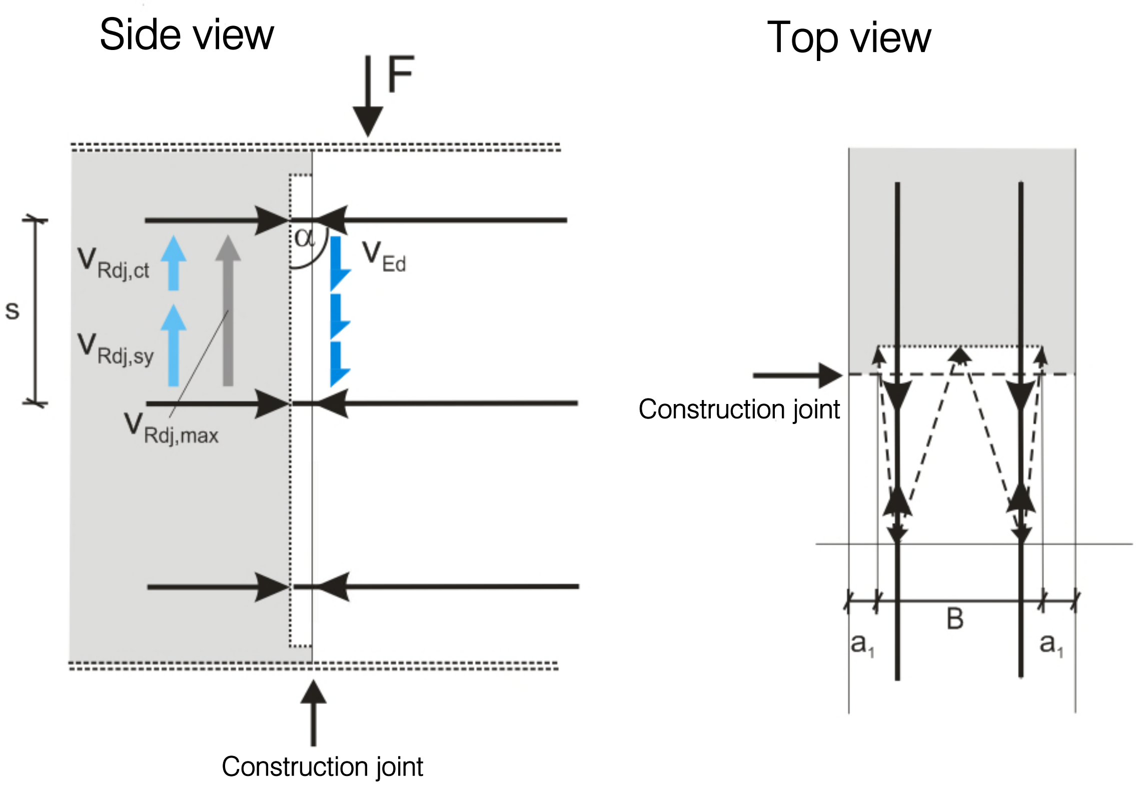


Figure 6: Truss model for the shear force transfer in concrete joints for case a of DBV Data Sheet /2/ for a1 < 5cm

If $a_1 \geq 5$ cm, the transferable shear force can be determined according to one of the following concepts when the surface conditions of the concrete joint and the protective casing are different.:

1. b = width of the casing + allowable width of the concrete joint with the surface coefficients c and μ of the smoother of the two surfaces
2. b = allowable width of the concrete joint with c and μ of the concrete joint
3. b = width of the protective casing with c and μ of the casing

The shear force that can be absorbed is mainly determined by the joint reinforcement of the rebend connection and the roughness of the concrete joint (casing, lateral concrete surface).

Comparison of the surface condition of the protective casing (VK) to that of the concrete joint (BF):

	equivalent		smoother *)		rougher	
	b	c, μ	b	c, μ	b	c, μ
$a_1 \geq 5$cm	B+2a_1	VK = BF	B+2a_1	VK	B+2a_1	BF
or			2a_1	BF	B	VK

*) Example: Protective casing smooth, concrete joint rough

In the connected component, the bar ends are calculated for an overlap with bars of the same diameter.

6.1.3. Design according to DIN EN 1992-1-1 /1/ and DBV Data Sheet /3/

The transfer of shear forces parallel to the concrete joint is described in DIN EN 1992-1-1, chapter 6.2.5, "Shear at the interface between concrete cast at different times".

Effect: The shear force v_{Ed} to be transferred per m of unit length can be determined according to DIN EN 1992-1-1 Eq. (6.25).

The design resistance for the shear force transfer of the joint is calculated as specified in DIN EN 1992-1-1 Eq. (6.25). The additive design equation is composed of the percentage contact area of the concrete resulting from adhesion and friction as well as of the percentage contact area of the bond reinforcement.

Absorbable longitudinal shear force of a bond joint:

$$V_{Rdi} = V_{Rdi,c} + V_{Rdi,s} \leq V_{Rdi,max}$$

where $V_{Rdi,c}$ percentage contact area of the concrete resulting from adhesion and friction

$V_{Rdi,s}$ percentage contact area resulting from the bond reinforcement

$V_{Rdi,max}$ maximum permissible longitudinal shear force of the joint

To determine the maximum possible longitudinal shear forces that can be absorbed by Nevoga PYRAPLEX® rebend connections in the bond joint, it must be ensured that the anchorage length of the rebars is appropriate. In the new design concept, the explicit determination of the compressive strut angle is no longer required. From Equation (6.25) of DIN EN 1992-1-1 /1/, the inclination of the truss model is determined using the friction coefficient μ. The factor is $1.2 \cdot \mu$.

Percentage contact area of the concrete resulting from adhesion and friction:

$$v_{Rdi,c} = c \cdot f_{ctd} + \mu \cdot \sigma_n$$

where c roughness coefficient as a function of the surface condition of the building component transferring the shear force according to 6.2.5 (2) /1/ (see chapter 5.4.1)

f_{ctd} design value of the concrete tensile strength of the 1st or 2nd casting section (the smaller value is decisive) where $f_{ctd} = \alpha_{ct} \cdot f_{ctk;0.05} / \gamma_c$ according to DIN EN 1992-1-1 3.1.6 (2)P; $\alpha_{ct} = 0.85$ and $\gamma_c = 1.5$ according to 3.1.6 (2)P (NDP)

μ friction coefficient as a function of the surface condition of the building component transferring the shear force according to 6.2.5 (2) /1/ (see chapter 5.4.1)

σ_n $= n_{Ed} / b < 0.6\, f_{cd}$
normal stress perpendicular to the joint (positive concrete compressive stress), which can be caused by restraint or external loads

Depending on the surface conditions of the concrete joint and the protective casing, different approaches to the calculated joint width b as well as the coefficients c and μ are recommended. Therefore, in the type statics, the joint width b is not taken into account on the action side but on the resistance side (see also chapter 6.2.2).

Percentage contact area of the joint resulting from the bond reinforcement:

Design value of the shear force $v_{Rdi,s}$ limited by the load bearing capacity of the shear reinforcement as per Eq. (6.25):

$$v_{Rdi,s} = \rho \cdot f_{yd} \cdot (1.2 \cdot \mu \cdot \sin \alpha + \cos \alpha)$$

where ρ $= A_s / A_i$

f_{yd} design value of the yield stress of the reinforcing steel
$= 0.8 \cdot f_{yk} / \gamma_s$

α angle between joint and rebend reinforcement; normally 90°

μ friction coefficient as a function of the surface condition of the joint according to 6.2.5 (2), DIN EN 1992-1-1

Design value of the maximum shear force $v_{Rd,max}$ analogous to Eq. (6.25) (load bearing capacity of the compressive strut):

$$v_{Rdi,max} = 0.5 \cdot \nu \cdot f_{cd}$$

where ν reduction factor for the concrete compressive strength as a function of the surface condition (cf. chapter 5.4.1)

f_{cd} design value of the concrete compressive strength

Restriction:

The shear capacity in the longitudinal direction of grout joints between slabs and shear walls (in-plane effect) may be determined accordingly. However, when the joints a mostly cracked, as a rule c = 0 must be applied for smooth and rough joints and c = 0.5 for serrated joints.

6.1.4. Input parameters to determine the shear load bearing capacity in PART II:

In the design formulae indicated in chapter 6.1.3, the following input parameters were chosen for the determination of the values in Part II:

Material parameters

f_{yd}	$0.8 \cdot 500 / 1.15 = 347.8$N/mm²
f_{cd}	$0.85 \cdot f_{ck} / 1.5$ where f_{ck} according to concrete strength class
c	= 0.5 (serrated surface of the protective casing)
μ	= 0.9 (serrated surface of the protective casing)
ν	= 0.7 (serrated surface of the protective casing)

Geometry

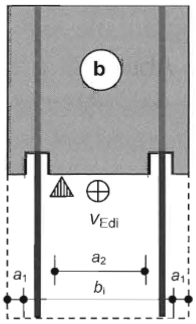
a_1	< 5cm
b	width of the serration of the applied PYRAPLEX® casing

Other

σ_n	= 0N/mm²
a_s	cross-sectional area of the chosen rebend connection [cm²/m]

6.2 Case b Shear force parallel to the concrete joint considering a concrete joint

6.2.1. Description of the case as specified in DBV Data Sheet



$a_1 < 50$ mm
$a_2 \geq 50$ mm with surface condition according to DIN EN 1992-1-1, 6.2.5

The shear force is transmitted parallel to the concrete joint. Due to the design of the concrete joint as illustrated in the adjacent figure, the compression strut is supported by the protective casing of the PYRAPLEX® connection as well as by the concreting surface. The surface conditions of the casing and the concrete joint determine the roughness coefficient. If the concrete at the lateral edge (area “a_1”) is used to determine the load bearing capacity, a uniform concrete surface condition is assumed in the areas a_1 and a_2.

The assumption is always a two-layer rebend reinforcement per unit length

6.2.2. Mechanical model

The truss model as illustrated below forms the basis of the design concept:

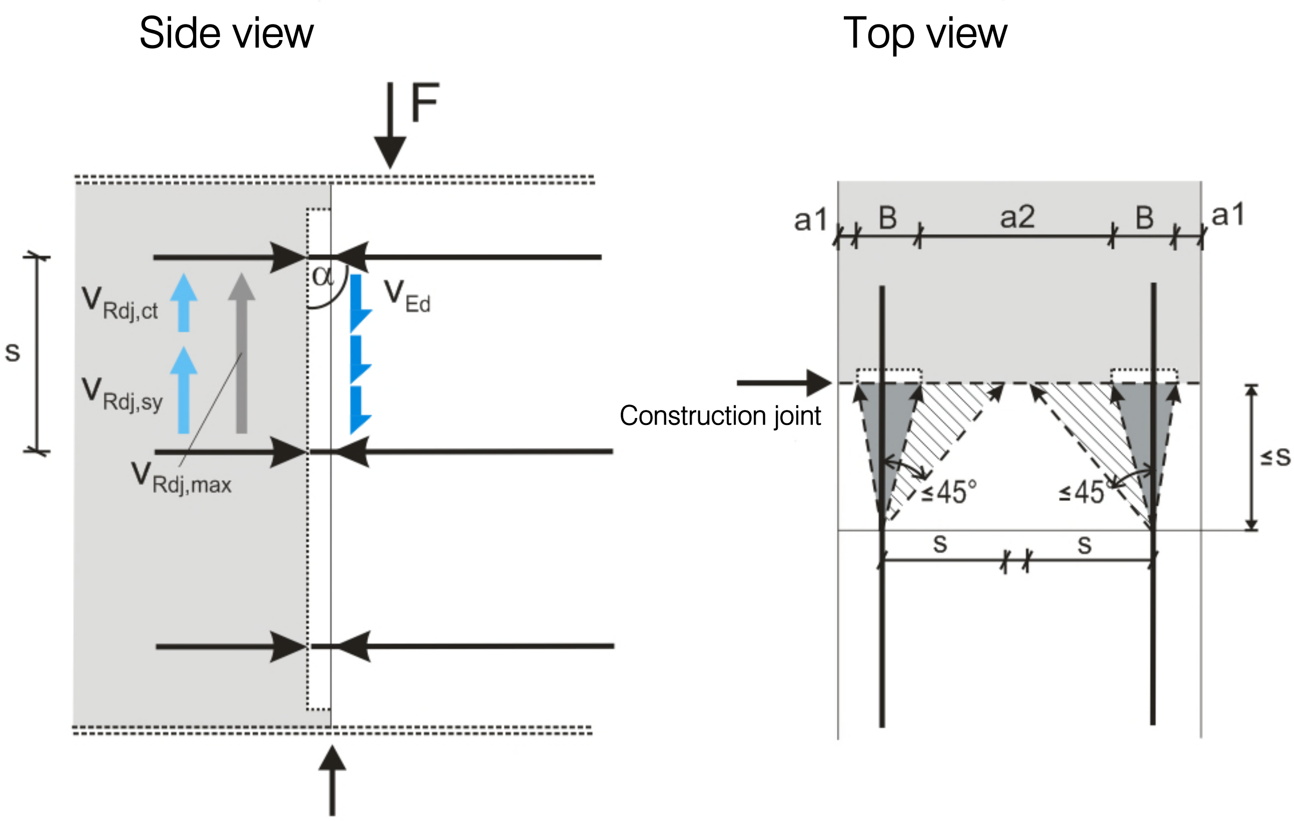


Figure 7: Truss model for the shear force transfer in concrete joints for case b according to DBV Data Sheet /3/ for $a_1 < 5$ cm and $a_2 \geq 5$cm

The transferable shear force can be determined according to one of the following concepts if the surface conditions of the concrete joint and the protective casing are different:

1. b = width of the casing + allowable width of the concrete joint with the surface coefficients c and μ of the smoother of the two surfaces
2. b = allowable width of the concrete joint with c and μ of the concrete joint
3. b = width of the protective casing with c and μ of the casing

Comparison of the surface condition of the protective casing (VK) to the concrete joint (BF):

	equivalent		smoother *)		rougher	
	b	c, μ	b	c, μ	b	c, μ
$a_1 < 5$cm	$2B + a_2$	VK = BF	$2B + a_2$	VK	$2B + a_2$	BF
or			a_2	BF	2B	VK
$a_1 \geq 5$cm	$2B+a_2+2a_1$	VK = BF	$2B+a_2+2a_1$	VK	$2B+a_2+2a_1$	BF
or			a_2+2a_1	BF	2B	VK

*) Example: Protective casing smooth, concrete joint rough

In the connected component, the bar ends are calculated for an overlap with bars of the same diameter.

Remark on transverse tensile stresses

If the concrete joint features a rougher surface condition than the protective casing, the transverse tension resulting from the inclined compressive strut must be verified. For transverse tensile stresses exceeding
$f_{ctd} = \alpha_{ct} \cdot f_{ctk;0.05} / \gamma_c$ ($\alpha_{ct} = 0.85$, $\gamma_c = 1.5$), a transverse tensile reinforcement must be placed on site.

6.2.3. Design according to DIN EN 1992-1-1 and DBV Data Sheet

The transfer of shear forces parallel to the concrete joint is specified in DIN EN 1992-1-1, chapter 8.4.4 (NCI) "Bemessungswert der Verankerungslänge" (Design value of anchorage length). Accordingly, the surface condition of the joint is decisive for the absorbable shear force – apart from existing shear reinforcement.

Consideration of the concrete joint

Area a_1: The lateral concrete strips are only used to determine the load bearing capacity if they have a width of $a_1 \geq 5$cm.

Area a_2: The area between the protective casings with its defined roughness is applied for the calculation of the load bearing capacity of the joint. The respective width is limited by the formation of the truss. In the direction of the protective casing, the resulting compressive strut is supported by the reinforcement at an inclination angle of 45° at a distance s from the joint (see Figure 7, right drawing). The inclination of the compressive strut at right angles to the casing is limited to 45°. This results in the value of 2 · s for the maximum applicable width between the rebars.

The further design is made analogous to case a

6.2.4. Input parameters to determine the shear load bearing capacity:

Within the framework of this type statics, no values of the load bearing capacity for case b are given. The following input parameters can however be chosen for the calculation of the design specified in chapter 6.2.3:

Material parameters

f_{yd}	$0.8 \cdot 500 / 1.15 = 347.8$ N/mm²
f_{cd}	$0.85 \cdot f_{ck} / 1.5$ where f_{ck} according to concrete strength class
c	= 0.5 (serrated surface of the protective casing)
c	= 0.2 (smooth surface of the concrete)
μ	= 0.9 (serrated surface of the protective casing)
μ	= 0.6 (smooth surface of the concrete)
ν	= 0.7 (serrated surface of the protective casing)
ν	= 0.2 (smooth surface of the concrete)

Geometry

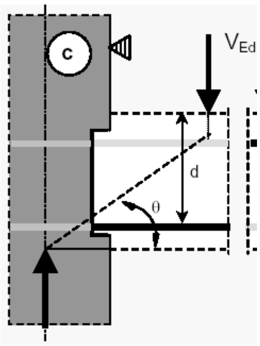
a_1	< 5cm
a_2	< 5cm Due to the modular composition of the rebend connections, the existing steel plate widths are applied for serrated joints within the framework of this type statics.

Other

σ_n	= 0N/mm²
a_s	cross-sectional area of the chosen rebend connection [cm²/m]

6.3 Case c Shear force perpendicular to concrete joint – connection wall to floor slab

6.3.1. Description of the case as specified in DBV Data Sheet



The PYRAPLEX® rebend connection serves to form a horizontal line support for an articulated floor slab to wall connection.

The PYRAPLEX® rebend elements form a reinforcement connection of the upper, structural reinforcement and the lower flexural reinforcement of the floor slab to the wall elements. Thus, the slab is directly supported. If the upper reinforcement layer is considered as load bearing, the connection must be verified according to case e (restrained).

6.3.2. Mechanical model

The truss model as illustrated below forms the basis of the design concept:

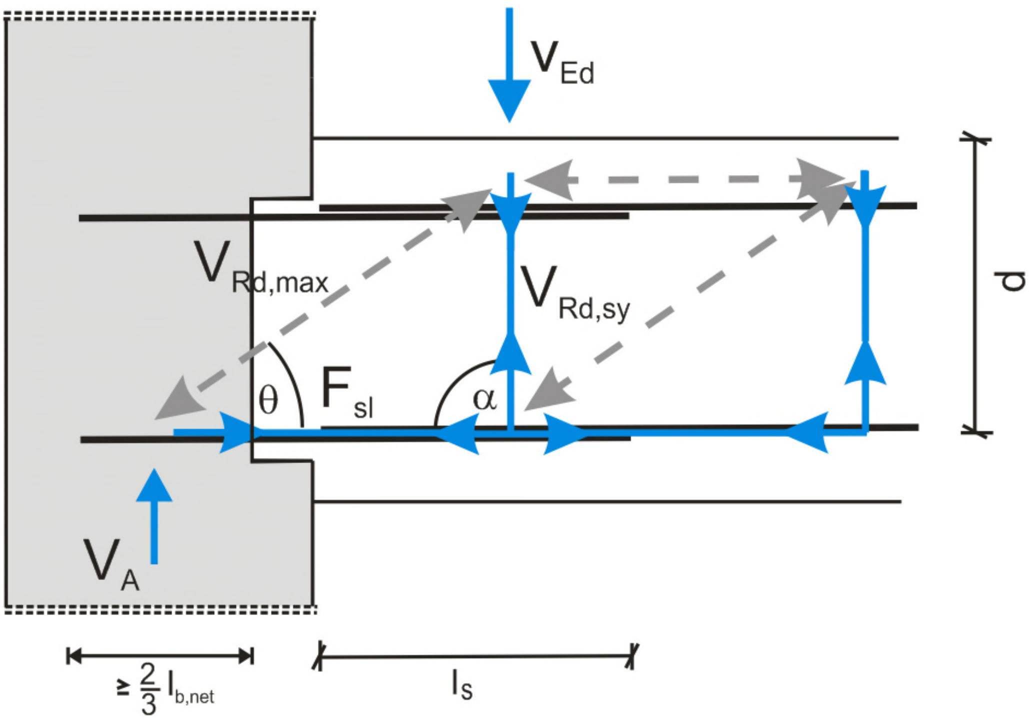


Figure 8: Truss model for the shear force transfer in concrete joints for case c according to DBV Data Sheet “Rebending” /3/

If the shear force is transferred perpendicular to the joint, this area is designed analogous to DIN EN 1992-1-1, chapter 6.2.2 (no shear reinforcement in the connected component) or according to chapter 6.2.3 (shear reinforcement provided in the connected component). In this case, the rebend reinforcement forms the longitudinal reinforcement of the connected component.

In this case, the maximum load bearing capacity of the rebend connection is limited either by the minimum anchorage forces $F_{Va,Alt}$ (BA I) or $F_{Va,neu}$ (BA II), the yield stress f_{yd} of the rebars of the rebend connection or the maximum absorbable compressive strut strength $V_{Rdi,max}$. In the calculation conducted here, the concept is pursued to determine the maximum load bearing capacity of the rebend connection according to the generally accepted rules of technology.

The maximum load bearing capacity for $V_{Rdi,s}$ at a compressive strut angle of $\cot \theta = 1.0$ results from the truss model according to Figure 8. The required shear reinforcement must then also be determined using the compressive strut angle $\theta = 1.0$. The effect is that the flatter the angle of the compressive strut ($\cot \theta \geq 1.0$), not only the maximum load bearing capacity of the rebend connections is reduced, but also the required quantity of shear reinforcement in the connected building component.

6.3.3. Design according to DIN EN 1992-1-1 and DBV Data Sheet “Rebending of reinforcement steel - requirements on protective casings according to Eurocode 2” (January 2011)

The design of the transferable shear force in the connection wall to floor slab is carried out as specified in chapters 6.2.2 and 6.2.3 of DIN EN 1992-1-1 or the modified formulae given in DBV Data Sheet. Two cases are distinguished for the transferable shear force:

1. No shear reinforcement is placed in the floor slab (connected component).
2. Shear reinforcement is placed in the floor slab.

Load bearing capacity of the joint without shear reinforcement in the floor slab:

$$\boxed{V_{Rd,c} = (c / 0.5) \cdot [\, 0.15 / \gamma_c \cdot k \cdot (100\, \rho_l \cdot f_{ck})^{1/3} + 0.12 \cdot \sigma_{cp} \,] \cdot b_w \cdot d}$$

where c = roughness coefficient of the protective casing or the concrete joint

$k = 1 + \sqrt{200 / d} \leq 2.0$

ρ_l = longitudinal reinforcement ratio anchored according to Figure 6.3, /1/ (see below)

$= A_{sl} / (b_w \cdot d)$ where $A_{sl} = A_{s,PYRAPLEX^{®},Biegezug} \cdot l_{b,net,vorh} / l_{b,net,erf}$

$\leq 1.0 \cdot A_{s,PYRAPLEX^{®},Biegezug}$ (DBV Data Sheet)

σ_{cp} longitudinal normal stress in the axis of the total cross section (negative concrete compressive stress)

$= n_{Ed} / A_c$ where A_c = contact area of the connected component (protective casing plus concrete joint)

b_w smallest cross-sectional width within the tension zone of the cross section

d = effective depth of the connected component

As an alternative, the shear load bearing capacity in the floor slab can also be determined by the minimum design value $V_{Rd,c}$ according to Eq. 6.2b of DIN EN 1992-1-1. In this case, the equation is extended by the pre-factor to take into account the joint load bearing capacity c / 0.5 analogous to Eq. 6.2 of DBV

Data Sheet /3/. The higher value of both equations is applied for the shear load bearing capacity of PYRAPLEX® rebend connections without shear reinforcement in the connected component.

$$V_{Rd,c} = (c_j / 0.5) \cdot (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d$$

where v_{min} $= (0.0525/\gamma_c) \cdot k^{3/2} \cdot f_{ck}^{1/2}$ for $d \leq 600$mm

$= (0.0375/\gamma_c) \cdot k^{3/2} \cdot f_{ck}^{1/2}$ for $d > 800$mm

(intermediate values may be interpolated)

γ_c = partial safety factor for reinforced concrete as per 2.4.2.4 (1), Table 2.1DE of DIN EN 1992-1-1

k scale factor where $\kappa = 1 + \sqrt{(200/d)} \leq 2.0$

d = effective depth of the flexural reinforcement in the considered cross section

f_{ck} = characteristic value of the concrete compressive strength

σ_{cp} = design value of the longitudinal concrete stress at the centre of gravity of the cross section where $\sigma_{cp} = N_{Ed} / A_c < 0.2 \cdot f_{cd}$

N_{Ed} = design value of the longitudinal force in the cross section resulting from external loads or pre-stressing ($N_{Ed} > 0$ as longitudinal compressive strength)

k_1 = 0.12

Load bearing capacity of the joint with shear reinforcement in the floor slab:

According to DIN EN 1992-1-1 Eq. (6.7aDE), the inclination of the compressive strut may be freely selected within the following limits. In the connected building components, the inclination must be ensured at a distance of cot θ · d/2 from the joint:

$1.0 \leq \cot\theta \leq \frac{1.2 - 1.4\sigma_{cd}/f_{cd}}{1 - V_{Rd,cc}/V_{Ed}} \leq 3.0$ for general purpose concrete, here chosen as $\cot\theta = 1.0$;

The lower limitation of $\cot\theta = 1.0$ is made according to DBV Data Sheet.

where f_{cd} design value of the concrete compressive strength

$V_{Rd,cc}$ $= 0.48 \cdot c \cdot f_{ck}^{1/3} (1 - 1.2 \cdot \sigma_{cd} / f_{cd}) \cdot b_w \cdot z$

z internal lever arm of the flexural design

$= 0.9\,d \leq d - 2c_{v,l} \geq d - c_{v,l} - 30$mm according to DIN EN 1992-1-1 chapter 6.2.3(1)

V_{Ed} design value of the acting shear force

c roughness coefficient according to Table 1, Figure 8 /3/

Design value of the transferable shear force VRd,s as per Eq. (6.8) (load bearing capacity of the shear reinforcement):

$$V_{Rd,s} = (A_{sw} / s) \cdot f_{ywd} \cdot z \cdot \cot \theta$$

where A_{sw} cross-sectional area of the shear reinforcement in the connected component

s distance of the shear reinforcement in the direction of the component axis

f_{ywd} design value of the yield stress of the reinforcing steel

z internal lever arm of the flexural design
$= 0.9\,d \leq d - 2c_{v,l} \geq d - c_{v,l} - 30mm$ according to DIN EN 1992-1-1 chapter 6.2.3(1)

$\cot \theta$ $= 1.0$

Design value of the maximum shear force $v_{Rd,max}$ analogous to Eq. (78) (load bearing capacity of the compressive strut):

$$V_{Ed} \leq 0.3 \cdot V_{Rd,max} = 0.3 \cdot b_w \cdot z \cdot \nu_1 \cdot f_{cd} / (\cot \theta + \tan \theta)$$

where 0.3 reduction for the maximum load bearing capacity of the compressive strut as per /3/

b_w smallest cross-sectional width within the tension zone of the cross section

ν_1 reduction coefficient for the concrete strength at shear cracks
$\nu_1 = 0.75 \cdot \nu_2$ $\nu_2 = (1.1 - f_{ck}/ 500) \leq 1.0$

α angle between component axis and shear reinforcement; here always 90°

f_{cd} design value of the concrete compressive strength

$\cot \theta$ $= 1.0$

Load bearing capacity of the tension chord in the truss

PYRAPLEX® rebend connections must be able to transfer the horizontal tensile forces of the truss. It must be verified if the horizontal forces occurring in the developing truss can be transferred by the connection by means of anchoring or overlapping. The transferable bar force for a given bar length is determined as specified in chapter 5.2.3. The force to be transmitted can be calculated from the selected inclination of the compressive strut as follows:

$$F_{sd} / s = V_{Ed} \cdot \cot \theta + \sigma_{cp} \cdot h \qquad \leq F_{va} / s \qquad \text{(as per chapter 5.2.3)}$$

Anchorage:

The required anchorage length $l_{b,d}$ results according to DIN EN 1992-1-1 Eq. (8.4)

$$l_{b,d} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} \geq l_{b,min}$$

where
α_1 coefficient to consider the anchorage types as per /1/, Figure 8.1 and Table 8.2

α_2 coefficient to consider the minimum concrete cover as per /1/, Figure 8.3 and Table 8.2

α_3 coefficient to consider transverse reinforcement as per /1/, Table 8.2

α_4 coefficient to consider one or several welded cross bar(s) within the required anchorage length l_{bd} as per /1/, Table 8.2

α_5 coefficient to consider compression transverse to the splitting tensile crack plane within the required anchorage length as per /1/, Table 8.2

$l_{b,rqd}$ required basic value of the anchorage length for the anchorage of the force $A_s \cdot \sigma_{sd}$

where: $l_{b,rqd} = (\varnothing / 4) \cdot (\sigma_{sd} / f_{yd})$

and represents a value of $l_{b,rqd}$ reduced by the degree of utilisation and effectiveness of the anchorage. Moreover, the length used for the anchorage must at least correspond to a minimum structural value of $l_{b,min}$:

$l_{b,min} = 10 \cdot \varnothing$ or $l_{b,min} = 100$mm (or $6.7 \cdot \varnothing$ at direct support)

and $l_{b,min} = 0.3 \cdot l_{b,rqd}$ for anchorage of tension bars

$= 0.6 \cdot l_{b,rqd}$ for anchorage of compression bars

The anchorage length of the flexural tensile reinforcement may be calculated with $\alpha_5 = 2/3$ at direct support according to /1/ (NCI) to Table 8.2.

In the design, l_{bd} is verified. The length of the bar from the backside of the protective casing is taken into account. If the length falls below the minimum $l_{b,min}$, the anchorage is not load bearing. If the bar length ranges between $l_{bd,max}$ ($\sigma_{sd} = f_{yd}$) and $l_{b,min}$, the maximum force that can be anchored is determined with the existing bar length and design (hook, straight). This results in:

$F_s / s = f_{bd} \cdot (\pi\, d_s / s) \cdot l_{bd}$ *analogous to calculation chapter 5*

It is also verified if the overlap length of the straight bar ends with the lower longitudinal reinforcement of the floor slab is sufficient. It is assumed that the free end of the PYRAPLEX® rebend connection is effective as overlap length.

When determining l_{bd} , the actually existing steel stress of the longitudinal reinforcement can be taken into account.

6.3.4. Input parameters to determine the shear load bearing capacity in PART II:

The tables in Part II show shear load bearing capacities for building components with and without shear reinforcement in the floor slab. For the design formulae given in chapter 6.3.3, the following input parameters were chosen to determine the values in Part II:

Material parameters

f_{yd}	$0.8 \cdot 500 / 1.15 = 347.8 N/mm^2$
f_{cd}	$0.85 \cdot f_{ck} / 1.5$ where f_{ck} according to concrete strength class
c	0.5 (serrated surface of the protective casing)

Geometry

b_w	1.0m
θ	45° (as maximum inclination of the compressive strut)
α	90° (perpendicular shear reinforcement)
t	casing depth $\geq$ 20mm
a_1	casing cover $\geq$ 20mm
h_A	support height $h_A \geq 10 \cdot t$
l_A	support length $l_A \geq 5 \cdot h$

Other

σ_{cp}	$= 0 N/mm^2$

No cracks in the support area parallel to the joint (additional condition for the percentage contact area of the bracket = “Konsolentraganteil”).

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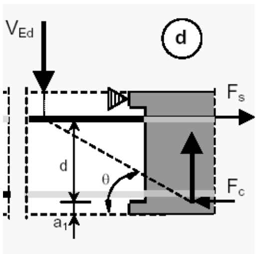
6.4 Case d Shear force perpendicular to the joint

6.4.1. Description of the case as specified in DBV Data Sheet



Two building components are connected using two-layer PYRAPLEX® rebend connections. In contrast to cases c and f, a truss is formed for the transfer of the shear force with the horizontal tension component in the upper reinforcement layer.

The effective depth of the truss corresponds to the spacing between the upper reinforcement layer and the lower edge of the protective casing. The shear force is transferred exclusively via the back side of the casing.

This case of shear force transfer is assumed when two floor slab elements are connected using a two-layer PYRAPLEX® rebend connection, which extends over the entire effective depth as illustrated in the drawing and if the floor slab is exposed to a negative moment (tension above).

6.4.2. Mechanical model

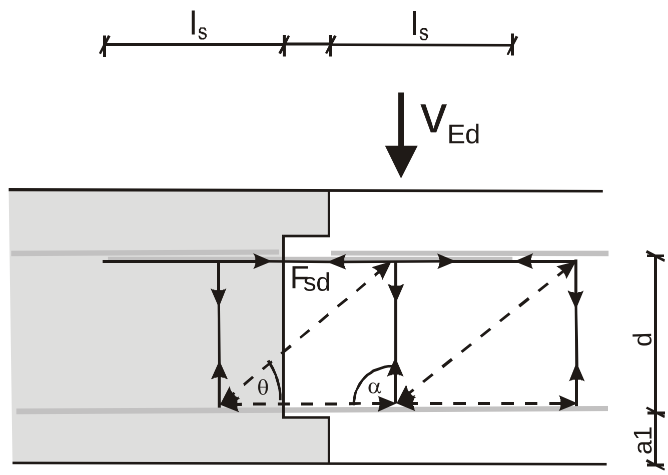
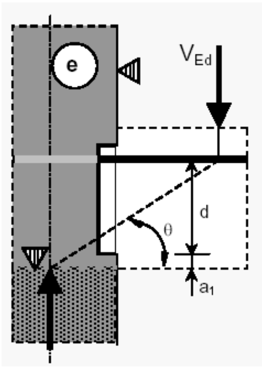


Figure 9: Truss model for the shear force transfer in concrete joints for case d

The verifications are made as for case e (see next chapter) with the difference that the overlap must be verified twice in case d.

6.5 Case e Shear force perpendicular to the concrete joint – horizontal tension component above

6.5.1. Description of the case as specified in DBV Data Sheet



Two building components are connected using two-layer PYRAPLEX® rebend connections. In contrast to case c, a truss is formed for the transfer of shear force, with the horizontal tension component in the upper reinforcement layer. Thus, a clamped connection is formed.

The effective depth of the truss corresponds to the spacing between the upper reinforcement layer and the lower edge of the casing. The shear force is transferred exclusively via the back side of the casing.

This case of shear force transfer is assumed if a floor slab is connected to a wall element which has a concrete joint at the level of the lower edge of the slab and the formation of the grain structure in this area of the wall could thus prevent the compressive strut from being safely supported on the reinforcement layer. The floor slab is clamped in the wall.

6.5.2. Mechanical model

Cases d and e do not differ regarding their load bearing behaviour. The only difference is the anchorage in building component 1. Straight bar ends in component 1 correspond to an overlap and are found in connection case d. Bent bar ends in component 1 are verified as to end anchorage and are considered for case e.

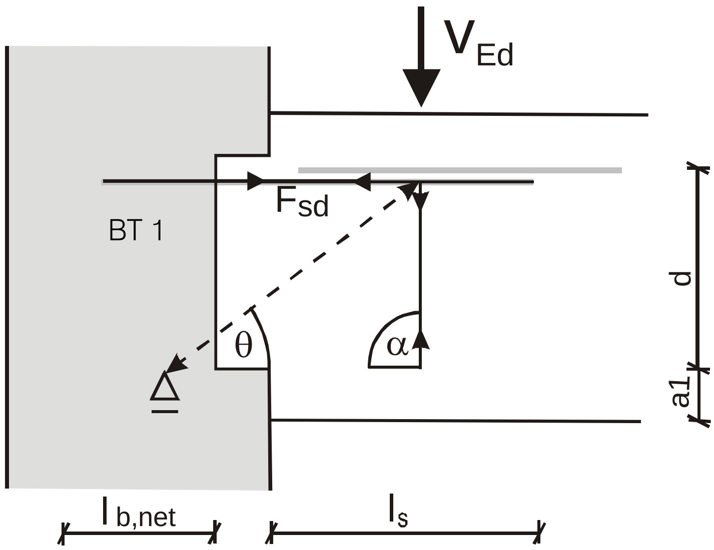


Figure 10: Truss model of the shear force transfer in concrete joints for case e

6.5.3. Design according to DIN EN 1992-1-1 and DBV Data Sheet “Rebending of reinforcement steel - requirements on protective casings according to Eurocode 2” (January 2011)

The design of the transferable shear force is carried out according to chapters 6.2.2 and 6.2.3 of DIN EN 1992-1-1 or the modified formulae specified in DBV Data Sheet. Two cases are distinguished for the transferable shear force:

1. No shear reinforcement is placed in the floor slab.
2. Shear reinforcement is placed in the floor slab.

Load bearing capacity of the joint without shear reinforcement in the floor slab:

$$\boxed{V_{Rd,c} = (c / 0.5) \cdot [\,0.15 / \gamma_c \cdot k \cdot (100\, \rho_l \cdot f_{ck})^{1/3} + 0.12 \cdot \sigma_{cp}\,] \cdot b_w \cdot d}$$

where c = roughness coefficient of the protective casing or the concrete joint

$\kappa = 1 + \sqrt{200/d} \leq 2.0$

ρ_l = longitudinal reinforcement ratio anchored as per Figure 6.3, /1/ (see below)

$= A_{sl} / (b_w \cdot d)$ where $A_{sl} = A_{s,PYRAPLEX^{®},Biegezug} \cdot l_{b,net,vorh} / l_{b,net,erf} \leq 1.0 \cdot A_{s,PYRAPLEX^{®},Biegezug}$

σ_{cp} longitudinal normal stress in the axis of the total cross section (negative concrete compressive stress)

$= n_{Ed} / A_c$ where A_c = contact area of the connected component

b_w smallest cross-sectional width within the tension zone of the cross section

d = effective depth of the connected component

As an alternative, the shear load bearing capacity in the floor slab can also be determined by the minimum design value $V_{Rd,c}$ according to Eq. 6.2b of DIN EN 1992-1-1. In this case, the equation is extended by the pre-factor to take into account the joint load bearing capacity c / 0.5 analogous to Eq. 6.2 of DBV Data Sheet /3/. The higher value of both equations is applied for the shear load bearing capacity of PYRAPLEX® rebend connections without shear reinforcement in the connected component.

$$\boxed{V_{Rd,c} = (c / 0.5) \cdot (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d}$$

where $v_{min} = (0.0525/\gamma_c) \cdot \kappa^{3/2} \cdot f_{ck}^{1/2}$ for $d \leq 600$mm

$= (0.0375/\gamma_c) \cdot \kappa^{3/2} \cdot f_{ck}^{1/2}$ for $d > 800$mm

(intermediate values may be interpolated)

γ_c = partial safety factor for reinforced concrete as per 2.4.2.4 (1), Table 2.1DE of DIN EN 1992-1-1

k scale factor where $\kappa = 1 + \sqrt{(200/d)} \leq 2.0$

d = effective depth of the flexural reinforcement in the considered cross section

f_{ck} = characteristic value of the concrete compressive strength

σ_{cp} = design value of the longitudinal concrete stress at the centre of gravity of the cross section where $\sigma_{cp} = N_{Ed} / A_c < 0.2 \cdot f_{cd}$

N_{Ed} = design value of the longitudinal force in the cross section resulting from external loads or pre-stressing ($N_{Ed} > 0$ as longitudinal compressive strength)

k_1 = 0.12

Load bearing capacity of the joint with shear reinforcement in the floor slab:

According to DIN EN 1992-1-1 Eq. (6.7aDE), the inclination of the compressive strut may be freely selected within the following limits. In the connected components, the inclination must be ensured at a distance of $\cot\theta \cdot d/2$ from the joint:

$1.0 \leq \cot\theta \leq \frac{1.2-1.4\sigma_{cd}/f_{cd}}{1-V_{Rd,cc}/V_{Ed}} \leq 3.0$ for general purpose concrete, here chosen as $\cot\theta = 1.0$

The lower limitation of $\cot\theta = 1.0$ follows DBV Data Sheet

where f_{cd} design value of the concrete compressive strength

$V_{Rd,cc}$ $= 0.48 \cdot c \cdot f_{ck}^{1/3} (1 - 1.2 \cdot \sigma_{cd} / f_{cd}) \cdot b_w \cdot z$

z internal lever arm of the flexural design
$= 0.9\, d \leq d + a_1 - 2c_{v,l}$ according to /1/ chapter 6.2.3 (1)

V_{Ed} design value for the acting shear force

Design value of the transferable shear force $V_{Rd,s}$ as per Eq. (6.8) /1/ (load bearing capacity of the shear reinforcement):

$$\boxed{V_{Rd,s} = A_{sw} / s \cdot f_{ywd} \cdot z \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha}$$

where A_{sw} cross-sectional area of the shear reinforcement in the connected component

s distance of the shear reinforcement in the direction of the component axis

f_{ywd} design value of the yield stress of the reinforcing steel

α inclination of the shear reinforcement to the longitudinal component axis

z internal lever arm of the flexural design according to DIN EN 1992-1-1 chapter 6.2.3 (1)

$= 0.9\, d \leq d + a_1 - 2c_{v,l}$

Design value of the maximum shear force VRd,max analogous to Eq. (6.9) /1/ (load bearing capacity of the compressive strut):

$$V_{Ed} \leq 0.3 \cdot V_{Rd,max} = 0.3 \cdot b_w \cdot z \cdot \nu_1 \cdot f_{cd} / (\cot\theta + \tan\theta)$$

where 0.3 reduction for the maximum load bearing capacity of the compressive strut as per /3/

b_w smallest cross-sectional width within the tension zone of the cross section

ν_1 reduction coefficient for the concrete strength at shear cracks
$\nu_1 = 0.75 \cdot \nu_2 \qquad \nu_2 = (1.1 - f_{ck}/500) \leq 1.0$

α angle between component axis and shear reinforcement; here always 90°

f_{cd} design value of the concrete compressive strength

Load bearing capacity of the tension chord in the truss

PYRAPLEX® rebend connections must be able to transfer the horizontal tensile forces of the truss. It must be verified if the horizontal forces occurring in the developing truss can be transferred by the connection by means of anchoring or overlapping. The transferable bar force for a given bar length is determined as specified in chapter 5.2.3. The force to be transmitted can be calculated from the selected inclination of the compressive strut as follows:

$$F_{sd} / s = V_{Ed} \cdot \cot\theta + \sigma_{cp} \cdot h \qquad \leq F_{va} / s \quad \text{(as per chapter 5.2.3)}$$

Anchorage:

The overlap of straight bar ends with the connecting reinforcement as well as the end anchorage of bent bar ends are verified as explained in chapter 5.2.3. The degree of utilisation of the longitudinal reinforcement is taken into account when determining l_{bd}.

Flexural design:

Due to the above shear force verification, a bending moment at the place of clamping of

$$M_{Ed} \leq V_{Ed} \cdot z \cdot \cot\theta$$

is covered. At higher clamping moments, the connection must be verified by a separate design for bending and shear force.

6.5.4. Input parameters to determine the shear load bearing capacity in PART II:

The tables in Part II show the shear load bearing capacities for building components with and without shear reinforcement in the floor slab. For the design formulae given in chapter 6.5.3, the following input parameters were chosen to determine the values in Part II:

Material parameters

f_{yd}	$0.8 \cdot 500 / 1.15 = 347.8$N/mm²
f_{cd}	$0.85 \cdot f_{ck} / 1.5$ where f_{ck} according to concrete strength class
c	0.5 (serrated surface of the protective casing)

Geometry

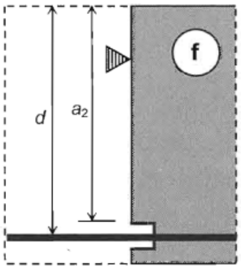
b_w	1.0m
$c_{v,l}$	28mm
a_1	$\geq$ 10mm
θ	= 45° (as maximum inclination of the compressive strut)
α	90° (perpendicular shear reinforcement)
z	For the calculation of the lever arm z, the value d is increased by a_1 according to Figure 8 /3/, because the standard equation ensures a sufficient safety of the concrete cover in the concrete compression zone.

Other

σ_{cp}	= 0N/mm²

6.6 Case f Shear force perpendicular to the concrete joint – connection floor slab to floor slab with tensile reinforcement below

6.6.1. Description of the case as specified in DBV Data Sheet



$a_2 \geq 50$mm with surface roughness as per DIN EN 1992-1-1, 6.2.5 (see Table 1)

Two floor slab sections are connected using single-layer PYRAPLEX® rebend connections. The connected area is under positive moment load (tension zone below).

In principle, also two-layer PYRAPLEX® types can be used in the given connection. In the design, it should however be assumed that the compressive strut is not supported on the lower part of the protective casing but runs analogous to Figure 11.

The effective depth of the truss corresponds to the spacing between the lower reinforcement layer and the upper edge of the floor slab irrespective of the height of the protective casing.

6.6.2. Mechanical model

The truss model as illustrated below forms the basis of the design concept:

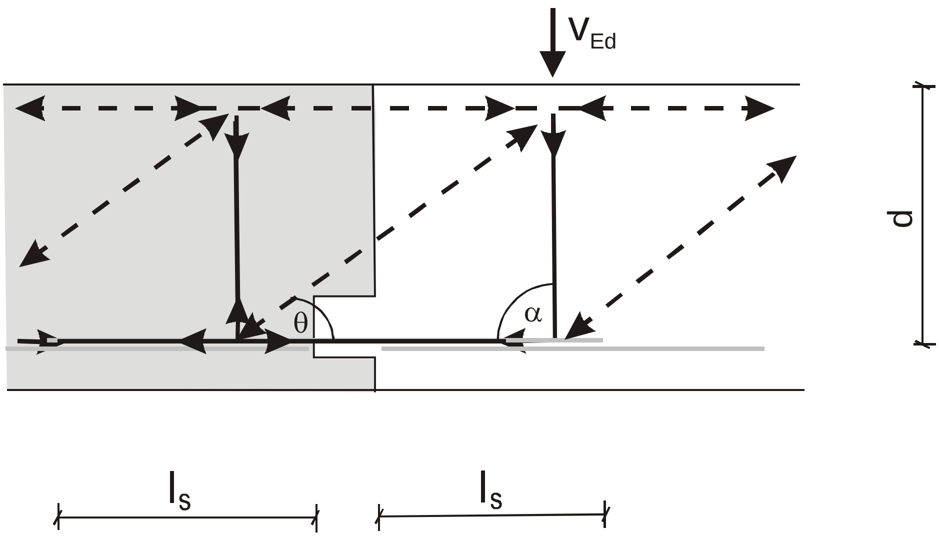


Figure 11: Truss model for the shear force transfer in concrete joints for case f of DBV Data Sheet

6.6.3. Design according to DIN EN 1992-1-1 and DBV Data Sheet

The design of the transferable shear force is carried out according to chapters 6.2.2 and 6.2.3 of DIN EN 1992-1-1 or the modified formulae specified in DBV Data Sheet. Two cases are distinguished for the transferable shear force:

1. No shear reinforcement is placed in the floor slab.
2. Shear reinforcement is placed in the floor slab.

Load bearing capacity of the joint without shear reinforcement in the floor slab:

$$V_{Rd,c} = (c / 0.5) \cdot [\, 0.15 / \gamma_c \cdot k \cdot (100\, \rho_l \cdot f_{ck})^{1/3} + 0.12 \cdot \sigma_{cp}\,] \cdot b_w \cdot d$$

where c = roughness coefficient of the protective casing or the concrete joint

κ = $1 + \sqrt{200/d} \leq 2.0$

ρ_l = longitudinal reinforcement ratio anchored according to Figure 6.2, /1/ (see below)

= $A_{sl} / (b_w \cdot d)$ where $A_{sl} = A_{s,PYRAPLEX^®,Biegezug} \cdot l_{b,net,vorh} / l_{b,net,erf}$
$\leq 1.0 \cdot A_{s,PYRAPLEX^®,Biegezug}$

σ_{cp} longitudinal normal stress in the axis of the total cross section (negative concrete compressive stress)

= n_{Ed} / A_c where A_c = contact area of the connected component

b_w smallest cross-sectional width within the tension zone of the cross section

d = effective depth of the connected component

As an alternative, the shear load bearing capacity in the floor slab can also be determined by the minimum design value $V_{Rd,c}$ according to Eq. 6.2b of DIN EN 1992-1-1. In this case, the equation is extended by the pre-factor to take into account the joint load bearing capacity c / 0.5 analogous to Eq. 6.2 of DBV Data Sheet /3/. The higher value of both equations is applied for the shear load bearing capacity of PYRAPLEX® rebend connections without shear reinforcement in the connected component.

$$V_{Rd,c} = (c / 0.5) \cdot (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d$$

where v_{min} = $(0.0525 / \gamma_c) \cdot k^{3/2} \cdot f_{ck}^{1/2}$ for $d \leq 600$mm
= $(0.0375 / \gamma_c) \cdot k^{3/2} \cdot f_{ck}^{1/2}$ for $d > 800$mm
(intermediate values may be interpolated)

γ_c = partial safety factor for reinforced concrete as per 2.4.2.4 (1), Table 2.1DE of DIN EN 1992-1-1

k scale factor where = $1 + \sqrt{(200/d)} \leq 2.0$

d = effective depth of the flexural reinforcement in the considered cross section

f_{ck} = characteristic value of the concrete compressive strength

σ_{cp} = design value of the longitudinal concrete at the centre of gravity of the cross section where $\sigma_{cp} = N_{Ed} / A_c < 0.2 \cdot f_{cd}$

N_{Ed} = design value of the longitudinal force in the cross section resulting from external loads or pre-stressing ($N_{Ed} > 0$ as longitudinal compressive force)

k_1 = 0.12

Load bearing capacity of the joint with shear reinforcement in the floor slab:

According to DIN EN 1992-1-1 Eq. (6.7aDE), the inclination of the compressive strut may be freely selected within the following limits. In the connected building components, the inclination must be ensured at a distance of cot θ · d/2 from the joint:

$1.0 \leq \cot\theta \leq \frac{1.2-1.4\sigma_{cd}/f_{cd}}{1-V_{Rd,cc}/V_{Ed}} \leq 3.0$ for general purpose concrete, here chosen as cot θ = 1.0;

The lower limitation of cot θ = 1.0 is made according to DBV Data Sheet

where f_{cd} design value of the concrete compressive strength

$V_{Rd,cc}$ $= 0.48 \cdot c_j \cdot \eta_1 \cdot f_{ck}^{1/3} (1 + 1.2 \cdot \sigma_{cd} / f_{cd}) \cdot b_w \cdot z$

z internal lever arm of the flexural reinforcement
$= 0.9\, d \leq d - 2c_{v,l} \geq d - c_{v,l} - 30mm$ according to DIN EN 1992-1-1 chapter 6.2.3 (1)

V_{Ed} design value of the acting shear force

c roughness coefficient according to 6.2.5 (2) of DIN EN 1992-1-1

Design value of the transferable shear force $V_{Rd,s}$ as per Eq. (6.8) /1/ (load bearing capacity of the shear reinforcement):

$$V_{Rd,s} = A_{sw} / s \cdot f_{ywd} \cdot z \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha$$

where A_{sw} cross-sectional area of the shear reinforcement in the connected component

s distance of the shear reinforcement in the direction of the component axis

f_{ywd} design value of the yield stress of the reinforcing steel

α inclination of the shear reinforcement to the longitudinal component axis

z internal lever arm of the flexural design
$= 0.9\, d \leq d - 2c_{v,l} \geq d - c_{v,l} - 30mm$ according to DIN EN 1992-1-1 chapter 6.2.3 (1)

Design value of the maximum shear force $v_{Rd,max}$ analogous to Eq. (6.9) /1/ (Load bearing capacity of the compressive strut):

$$V_{Ed} \leq 0.3 \cdot V_{Rd,max} = 0.3 \cdot b_w \cdot z \cdot \nu_1 \cdot f_{cd} / (\cot\theta + \tan\theta)$$

where 0.3 reduction for the maximum load bearing capacity of the compressive strut as per /3/

b_w smallest cross-sectional width within the tension zone of the cross section

ν_1 reduction coefficient for the concrete strength at shear cracks
$\nu_1 = 0.75 \cdot \nu_2$ $\nu_2 = (1.1 - f_{ck}/ 500) \leq 1.0$

α angle between component axis and shear reinforcement; here always 90°

f_{cd} design value of the compressive strength of the concrete

Load bearing capacity of the tension chord in the truss

PYRAPLEX® rebend connections must be able to transfer the horizontal tensile forces of the truss. It must be verified if the horizontal forces occurring in the developing truss can be transferred by the connection by means of anchoring or overlapping. The transferable bar force for a given bar length is determined as specified in chapter 5.2.3. The force to be transmitted can be calculated from the selected inclination of the compressive strut as follows:

$$F_{sd} / s = v_{Ed} \cdot \cot \theta + \sigma_{cp} \cdot h \qquad \leq F_{va} / s \quad \text{(as per chapter 5.2.3)}$$

Anchorage:

The overlap of straight bar ends with the connecting reinforcement as well as the end anchorage of bent bar ends are verified as explained in chapter 5.2.3. Independent of the chosen PYRAPLEX® type, the overlap is verified in both building components. Moreover, the degree of utilisation of the longitudinal reinforcement is taken into account when determining l_{bd}.

6.6.4. Input parameters to determine the shear load bearing capacity:

The tables in Part II do not show separate shear load bearing capacities for serrated casings since the concrete surface condition a_2 is chosen to be smooth analogous to the smooth casings. No separate values are displayed:

Material parameters

f_{ck}	According to concrete strength class
c	0.2 (smooth surface of the concrete joint)

Geometry

b_w	1.0 m
θ	= 45° (as maximum inclination of the compressive strut)
α	90° (vertical shear reinforcement)

Other

σ_{cd}	= 0N/mm²
Application of a single-layer Nevoga rebend connection (e. g. PYRAPLEX®-Type A)	

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PART II LOAD TABLES OF THE PYRAPLEX® REBEND CONNECTIONS

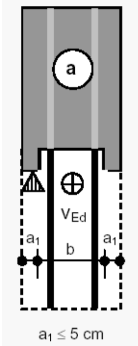
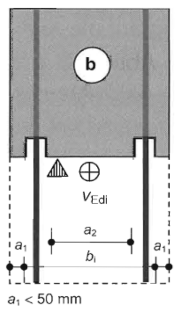
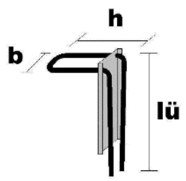
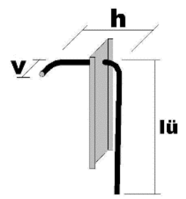
Overview of the load cases according to DBV Data Sheet "Rebending of reinforcement steel – requirements on protective casings", Figure 8 and boundary conditions of the design and calculation bases

It applies for all Pyraplex® (cf. also page II-3):

casing height Ø8 F = 3.0 cm
Ø10 F = 3.6 cm, 4.0 cm and 5.0 cm
Ø12 F = 3.6 cm, 4.0 cm and 5.0 cm

Shear force parallel to the joint

Case a / b

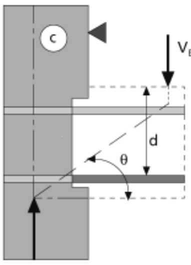
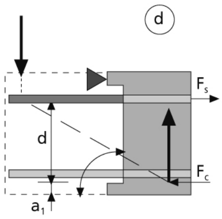
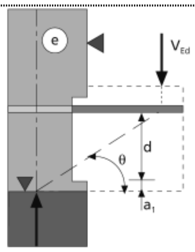
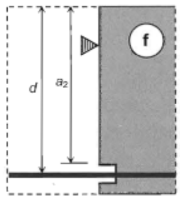
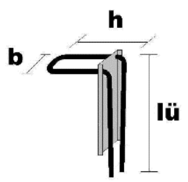
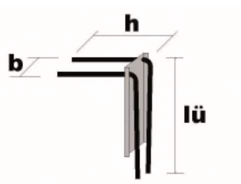
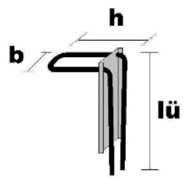
Case	a	b
Static system according to DBV Data Sheet "Rebending of reinforcement steel – requirements on protective casings" Figure 8		
Pyraplex®- / stirrup types	Pyraplex® Type B  **TYPE "B"** with two bar layers Stirrup height (h) Ø8 h = 170mm Ø10 h = 170mm Ø12 h = 170mm Stirrup length ($l_ü$) Ø8 $l_ü$ = 320mm Ø10 $l_ü$ = 390mm Ø12 $l_ü$ = 460mm	Pyraplex® Type A  **TYPE "A"** with one bar layer Bar length (h) Ø8 h = 170mm Ø10 h = 170mm Ø12 h = 170mm Bar length ($l_ü$) Ø8 $l_ü$ = 320mm Ø10 $l_ü$ = 390mm Ø12 $l_ü$ = 460mm
Table of load bearing capacity	Page II-4	Page II-5

Overview of the load cases according to DBV Data Sheet "Rebending of reinforcement steel – requirements on protective casings", Figure 8 and boundary conditions

It applies for all Pyraplex® (cf. also page II-3):

casing height Ø8 F = 3.0 cm
Ø10 F = 3.6 cm, 4.0 cm and 5.0 cm
Ø12 F = 3.6 cm, 4.0 cm and 5.0 cm

Shear force perpendicular to the joint
Cases c / d / e / f

Case	c	d	e	f
Static system according to DBV Data Sheet "Rebending of reinforcement steel – requirements on protective casings" Figure 8				 $a_2 \geq$ 50mm with surface roughness as per DIN EN 1992-1-1, 6.2.5 (see Table 1)
Pyraplex®- / stirrup types	Pyraplex® Type B  **TYPE "B"** with two bar layers Stirrup height (h) Ø8 h = 170mm Ø10 h = 170mm Ø12 h = 170mm Stirrup length ($l_ü$) Ø8 $l_ü$ = 320mm Ø10 $l_ü$ = 390mm Ø12 $l_ü$ = 460mm	Pyraplex® Type DD  **TYPE "DD"** with two bar layers Bar length (h) Ø8 h = 600mm Ø10 h = 600mm Ø12 h = 600mm Bar length ($l_ü$) Ø8 $l_ü$ = 320mm Ø10 $l_ü$ = 390mm Ø12 $l_ü$ = 460mm	Pyraplex® Type B  **TYPE "B"** with two bar layers Stirrup height (h) Ø8 h = 170mm Ø10 h = 170mm Ø12 h = 170mm Stirrup length ($l_ü$) Ø8 $l_ü$ = 320mm Ø10 $l_ü$ = 390mm Ø12 $l_ü$ = 460mm	on request!
Table load bearing capacity	Page II-6 to II-7	Page II-8 to II-9	Page II-10 to II-11	

Geometries of the protective casing on which the type statics are based:

Overview reinforcement connection Pyraplex									
Article number	stirrup type	bar diameter	bar spacing	stirrup width	casing width	casing height	stirrup height	overlap length	element length
		ø	s	b_b	b_k	F	h	$l_ü$	
		[mm]	[cm]	[cm]	[cm]	[cm]	[cm]	[cm]	[cm]
BKV081009E	B	8	10	9	11.2	3.0	17	32	125
BKV081509E	B	8	15	9	11.2	3.0	17	32	125
BKV082009E	B	8	20	9	11.2	3.0	17	32	125
BKV081012E	B	8	10	12	14.2	3.0	17	32	125
BKV081512E	B	8	15	12	14.2	3.0	17	32	125
BKV082012E	B	8	20	12	14.2	3.0	17	32	125
BKV081015E	B	8	10	15	17.2	3.0	17	32	125
BKV081515E	B	8	15	15	17.2	3.0	17	32	125
BKV082015E	B	8	20	15	17.2	3.0	17	32	125
BKV081018E	B	8	10	18	20.2	3.0	17	32	125
BKV081518E	B	8	15	18	20.2	3.0	17	32	125
BKV082018E	B	8	20	18	20.2	3.0	17	32	125
BKV081020E	B	8	10	20	22.2	3.0	17	32	125
BKV081520E	B	8	15	20	22.2	3.0	17	32	125
BKV082020E	B	8	20	20	22.2	3.0	17	32	125
BKV101009E	B	10	10	9	11.2	5.0	17	39	125
BKV101509E	B	10	15	9	11.2	3.6	17	39	125
BKV102009E	B	10	20	9	11.2	3.6	17	39	125
BKV101012E	B	10	10	12	14.2	4.0	17	39	125
BKV101512E	B	10	15	12	14.2	3.6	17	39	125
BKV102012E	B	10	20	12	14.2	3.6	17	39	125
BKV101015E	B	10	10	15	17.2	3.6	17	39	125
BKV101515E	B	10	15	15	17.2	3.6	17	39	125
BKV102015E	B	10	20	15	17.2	3.6	17	39	125
BKV101018E	B	10	10	18	20.2	3.6	17	39	125
BKV101518E	B	10	15	18	20.2	3.6	17	39	125
BKV102018E	B	10	20	18	20.2	3.6	17	39	125
BKV101020E	B	10	10	20	22.2	3.6	17	39	125
BKV101520E	B	10	15	20	22.2	3.6	17	39	125
BKV102020E	B	10	20	20	22.2	3.6	17	39	125
BKV121009E	B	12	10	9	11.2	5.0	17	46	125
BKV121509E	B	12	15	9	11.2	5.0	17	46	125
BKV122009E	B	12	20	9	11.2	4.0	17	46	125
BKV121012E	B	12	10	12	14.2	5.0	17	46	125
BKV121512E	B	12	15	12	14.2	3.6	17	46	125
BKV122012E	B	12	20	12	14.2	3.6	17	46	125
BKV121015E	B	12	10	15	17.2	5.0	17	46	125
BKV121515E	B	12	15	15	17.2	3.6	17	46	125
BKV122015E	B	12	20	15	17.2	3.6	17	46	125
BKV121018E	B	12	10	18	20.2	4.0	17	46	125
BKV121518E	B	12	15	18	20.2	3.6	17	46	125
BKV122018E	B	12	20	18	20.2	3.6	17	46	125
BKV121020E	B	12	10	20	22.2	3.6	17	46	125
BKV121520E	B	12	15	20	22.2	3.6	17	46	125
BKV122020E	B	12	20	20	22.2	3.6	17	46	125

The geometries were transferred correspondingly to types “A” and “DD”.

Case a – Pyraplex® Type B

Case a - Pyraplex® Type B

Assumptions: Shear load bearing capacity parallel to the joint where $v_{Ed,i} = \beta \cdot V_{Ed/z}$ (here $\beta = 1.0$)
(according to DBV Data Sheet "Rebending", Figure 8 and EC2 with NAD)
classification SERRATED according to EC2 with NAD
$a_1 \leq$ 5cm, $\sigma_{cp} = \sigma_n = 0$

$(v_{Rdi,c} + v_{Rdi,s}) \cdot b_i \leq v_{Rdi,max} \cdot b_i$ [kN/m]

	Type B / 112			Type B / 142			Type B / 172			Type B / 202			Type B / 222		
	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37
Ø8/20	165.5	192.1	216.9	178.4	207.1	233.8	191.3	222.0	250.7	204.2	237.0	267.6	212.8	247.0	278.9
Ø8/15	207.5	240.8	272.0	220.4	255.8	288.8	233.3	270.7	305.7	246.2	285.7	322.6	254.8	295.7	333.9
Ø8/10	291.5	338.3	382.0	304.4	353.3	398.9	317.3	368.2	415.8	330.2	383.2	432.7	338.8	393.2	444.0
Ø10/20	190.3	220.8	249.3	203.2	235.8	266.2	216.1	250.7	283.1	229.0	265.7	300.0	237.6	275.7	311.3
Ø10/15	240.5	279.1	315.2	253.4	294.1	332.1	266.3	309.0	349.0	279.2	324.0	365.9	287.8	334.0	377.1
Ø10/10	309.5	359.2	405.6	344.9	400.3	452.0	366.8	425.7	480.7	379.7	440.6	497.6	388.3	450.6	508.8
Ø12/20	215.0	249.5	281.8	233.3	270.8	305.7	246.2	285.7	322.6	259.1	300.7	339.5	267.7	310.7	350.8
Ø12/15	255.5	296.5	334.8	293.6	340.7	384.8	306.5	355.7	401.7	319.4	370.7	418.6	328.0	380.6	429.8
Ø12/10	363.5	421.8	476.4	376.4	436.8	493.3	389.3	451.8	510.2	429.2	498.1	562.4	448.6	520.6	587.8

Remark: Contrary to the standard regulations of EC2, the joint width b_i is taken into account on the resistance side

Case b – Pyraplex® Type A

Case b - Pyraplex® Type A

Assumptions: Shear load bearing capacity parallel to the joint where $v_{Ed,i} = \beta \cdot V_{Ed}/z$ (here $\beta = 1.0$)
(according to DBV Data Sheet "Rebending", Figure 8 and EC2 with NAD)
classification SERRATED according to EC2 with NAD
$a_1 \leq$ 5cm, $\sigma_{cp} = \sigma_n = 0$, joint width = existing casing widths
b = design width of the wall = wall thickness – $2a_1$

	$(v_{Rdi,c} + v_{Rdi,s}) \cdot b_i \leq v_{Rdi,max} \cdot b_i$ [kN/m]																	
	2 x Type A / 112			2 x Type A / 112 u. 142			2 x Type A / 142			2 x Type A / 142 u. 172			2 x Type A / 172			2 x Type A / 172 u. 202		
	b = 250			b = 280			b = 300			b = 340			b = 370			b = 400		
	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37
Ø8/20	204.3	237.0	267.7	226.2	262.4	296.4	242.7	281.6	318.0	255.6	296.5	334.9	268.4	311.5	351.8	281.3	326.5	368.7
Ø8/15	240.3	278.8	314.8	253.2	293.8	331.7	282.9	328.2	370.7	295.8	343.2	387.5	308.6	358.2	404.4	321.5	373.1	421.3
Ø8/10	312.3	362.4	409.2	325.2	377.3	426.1	338.1	392.3	443.0	351.0	407.2	459.9	363.8	422.2	476.8	376.7	437.2	493.7
Ø10/20	231.3	268.4	303.1	255.4	296.4	334.7	272.8	316.6	357.5	285.7	331.5	374.4	298.6	346.5	391.3	311.5	361.5	408.2
Ø10/15	276.3	320.6	362.0	289.2	335.5	378.9	323.1	374.9	423.3	336.0	389.8	440.2	348.8	404.8	457.1	361.7	419.8	474.0
Ø10/10	366.3	425.0	479.9	379.2	440.0	496.8	392.1	454.9	513.7	405.0	469.9	530.6	417.8	484.9	547.5	430.7	499.8	564.4
Ø12/20	258.3	299.7	338.4	284.7	330.3	373.0	303.0	351.6	397.0	315.9	366.5	413.9	328.7	381.5	430.8	341.6	396.4	447.7
Ø12/15	312.3	362.4	409.2	325.2	377.3	426.1	363.3	421.5	476.0	376.2	436.5	492.9	389.0	451.4	509.8	401.9	466.4	526.7
Ø12/10	420.3	487.7	550.7	433.2	502.6	567.6	446.1	517.6	584.5	458.9	532.6	601.4	471.8	547.5	618.3	484.7	562.5	635.2

Remark: Contrary to the standard regulations of EC2, the joint width b_i is taken into account on the resistance side

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Case c, without shear reinforcement in the floor slab – Pyraplex® Type B

Case c - Pyraplex® Type B

Assumptions: Shear load bearing capacity transverse to the joint
(according to DBV Data Sheet “Rebending”, Figure 8 and DIN EN 1992-1-1 with NA),
surface of the protective casing SERRATED; $\sigma_{cp} = 0$;
c_{nom} = 28 mm

maximum shear load bearing capacity of the connection without shear reinforcement in the floor slab:
max $v_{Rd,c}$ [kN/m]

	Type B / 112			Type B / 142			Type B / 172			Type B / 202			Type B / 222		
	d=120 mm			d=150 mm			d=180 mm			d=210 mm			d=230 mm		
	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37
Ø8/20	53.1	59.4	65.1	66.4	74.2	78.7	78.7	78.7	78.7	78.7	78.7	78.7	78.7	78.7	78.7
Ø8/15	53.1	59.4	65.1	66.4	74.2	81.3	79.7	89.1	97.6	91.3	102.1	104.9	96.7	104.9	104.9
Ø8/10	53.1	59.4	65.1	66.4	74.2	81.3	79.7	89.1	97.6	91.3	102.1	111.8	96.7	108.1	118.5
Ø10/20	53.1	59.4	65.1	66.4	74.2	81.3	79.7	89.1	97.6	91.3	102.1	111.8	96.7	108.1	118.5
Ø10/15	53.1	59.4	65.1	66.4	74.2	81.3	79.7	89.1	97.6	91.3	102.1	111.8	96.7	108.1	118.5
Ø10/10	56.6	60.9	65.1	66.4	74.2	81.3	79.7	89.1	97.6	91.3	102.1	111.8	96.7	108.1	118.5
Ø12/20	53.1	59.4	65.1	66.4	74.2	81.3	79.7	89.1	97.6	91.3	102.1	111.8	96.7	108.1	118.5
Ø12/15	55.8	60.1	65.1	66.4	74.2	81.3	79.7	89.1	97.6	91.3	102.1	111.8	96.7	108.1	118.5
Ø12/10	63.9	68.8	73.1	74.1	79.8	84.8	83.7	90.2	97.6	91.6	102.1	111.8	96.7	108.1	118.5

According to DIN EN 1992-1-1, chapter 9.3.1.2, para. (1), for solid slabs, at least half of the required span reinforcement must be led to the support and anchored there under consideration of DIN EN 1992-1-1, para. (3), NA.(5)P.

The tensile force coverage must be verified separately in the overlap area of the rebend connection.

Case c, with shear reinforcement in the floor slab – Pyraplex® Type B

Case c - Pyraplex® Type B

Assumptions: Shear load bearing capacity transverse to the joint
(according to DBV Data Sheet “Rebending”, Figure 8 and DIN EN 1992-1-1 with NA),
surface of the protective casing SERRATED; $\sigma_{cp} = 0$;
c_{nom} = 28 mm

maximum shear load bearing capacity of the connection with shear reinforcement in the floor slab:
max v_{Ed} [kN/m]

	Type B / 112			**Type B / 142**			**Type B / 172**			**Type B / 202**			**Type B / 222**		
	d=120 mm			d=150 mm			d=180 mm			d=210 mm			d=230 mm		
	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37
Ø8/20	81.6	87.4	87.4	87.4	87.4	87.4	87.4	87.4	87.4	87.4	87.4	87.4	87.4	87.4	87.4
Ø8/15	81.6	102.0	116.6	116.6	116.6	116.6	116.6	116.6	116.6	116.6	116.6	116.6	116.6	116.6	116.6
Ø8/10	81.6	102.0	122.4	119.9	149.8	174.8	158.1	174.8	174.8	174.8	174.8	174.8	174.8	174.8	174.8
Ø10/20	81.6	102.0	122.4	119.9	136.6	136.6	136.6	136.6	136.6	136.6	136.6	136.6	136.6	136.6	136.6
Ø10/15	81.6	102.0	122.4	119.9	149.8	179.8	158.1	182.1	182.1	182.1	182.1	182.1	182.1	182.1	182.1
Ø10/10	81.6	102.0	122.4	119.9	149.8	179.8	158.1	197.6	237.2	196.4	245.4	273.2	221.9	273.2	273.2
Ø12/20	81.6	102.0	122.4	119.9	149.8	179.8	158.1	196.7	196.7	196.4	196.7	196.7	196.7	196.7	196.7
Ø12/15	81.6	102.0	122.4	119.9	149.8	179.8	158.1	197.6	237.2	196.4	245.4	262.3	221.9	262.3	262.3
Ø12/10	81.6	102.0	122.4	119.9	149.8	179.8	158.1	197.6	237.2	196.4	245.4	294.5	221.9	277.3	332.8

According to DIN EN 1992-1-1, chapter 9.3.1.2, para. (1), for solid slabs, at least half of the required span reinforcement must be led to the support and anchored there under consideration of DIN EN 1992-1-1, para. (3), NA.(5)P.

The tensile force coverage must be verified separately in the overlap area of the rebend connection.

Case d, without shear reinforcement in the floor slab – Pyraplex® Type DD

Case d - Pyraplex® Type DD

Nevoga®

Assumptions: Shear load bearing capacity transverse to the joint
(according to DBV Data Sheet “Rebending”, Figure 8 and DIN EN 1992-1-1 with NA),
surface of the protective casing SERRATED; $\sigma_{cp} = 0$;
values of the load bearing capacity of Type DD for an anchorage in the existing concrete where h = 0.6m

	maximum shear load bearing capacity of the connection without shear reinforcement in the floor slab: $v_{Rd,c}$ [kN/m]														
	Type DD / 112			Type DD / 142			Type DD / 172			Type DD / 202			Type DD / 222		
	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37
Ø8/20	42.5	47.5	52.1	55.8	62.4	68.3	69.1	77.2	78.7	78.7	78.7	78.7	78.7	78.7	78.7
Ø8/15	42.5	47.5	52.1	55.8	62.4	68.3	69.1	77.2	84.6	82.3	92.1	100.9	90.2	100.8	104.9
Ø8/10	42.5	47.5	52.1	55.8	62.4	68.3	69.1	77.2	84.6	82.3	92.1	100.9	90.2	100.8	110.5
Ø10/20	42.5	47.5	52.1	55.8	62.4	68.3	69.1	77.2	84.6	82.3	92.1	100.9	90.2	100.8	110.5
Ø10/15	42.6	47.5	52.1	55.8	62.4	68.3	69.1	77.2	84.6	82.3	92.1	100.9	90.2	100.8	110.5
Ø10/10	48.7	52.5	55.8	58.4	62.9	68.3	69.1	77.2	84.6	82.3	92.1	100.9	90.2	100.8	110.5
Ø12/20	43.7	47.5	52.1	55.8	62.4	68.3	69.1	77.2	84.6	82.3	92.1	[1] 100.9	[1] 90.2	100.8	110.5
Ø12/15	48.1	51.8	55.0	57.6	62.4	68.3	69.1	77.2	84.6	82.3	92.1	[1] 100.9	[1] 90.2	100.8	110.5
Ø12/10	55.0	59.3	63.0	66.0	71.1	75.5	76.1	82.0	87.1	85.5	92.2	[1] 100.9	[1] 90.9	100.8	110.5

A clamping moment of $m_{Ed} = v_{Ed} \cdot z$ is covered by the existing rebend reinforcement. For larger moments, the tensile force coverage must be verified separately.
The concrete cover was set at c_{nom} = 28mm.

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Case d, with shear reinforcement in the floor slab – Pyraplex® Type DD

Case d - Pyraplex® Type DD

Nevoga®

Assumptions: Shear load bearing capacity transverse to the joint
(according to DBV Data Sheet "Rebending", Figure 8 and DIN EN 1992-1-1 with NA),
surface of the protective casing SERRATED; $\sigma_{cp} = 0$;
values of the load bearing capacity of Type DD for an anchorage in the existing concrete where h = 0.6m

maximum shear load bearing capacity of the connection with shear reinforcement in the floor slab: max v_{Ed} [kN/m]

	Type DD / 112			Type DD / 142			Type DD / 172			Type DD / 202			Type DD / 222		
	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37
Ø8/20	71.4	87.4	87.4	87.4	87.4	87.4	87.4	87.4	87.4	87.4	87.4	87.4	87.4	87.4	87.4
Ø8/15	71.4	89.3	107.1	109.7	116.6	116.6	116.6	116.6	116.6	116.6	116.6	116.6	116.6	116.6	116.6
Ø8/10	71.4	89.3	107.1	109.7	137.1	164.5	147.9	174.8	174.8	174.8	174.8	174.8	174.8	174.8	174.8
Ø10/20	72.7	90.8	109.0	110.9	136.6	136.6	136.6	136.6	136.6	136.6	136.6	136.6	136.6	136.6	136.6
Ø10/15	72.7	90.8	109.0	110.9	138.7	166.4	149.2	182.1	182.1	182.1	182.1	182.1	182.1	182.1	182.1
Ø10/10	72.7	90.8	109.0	110.9	138.7	166.4	149.2	186.5	223.8	187.4	234.3	273.2	212.9	266.2	273.2
Ø12/20	74.0	92.4	110.9	112.2	140.3	168.3	150.5	188.1	196.7	188.7	196.7	[1] 196.7	[1] 196.7	196.7	196.7
Ø12/15	74.0	92.4	110.9	112.2	140.3	168.3	150.5	188.1	225.7	188.7	235.9	[1] 262.3	[1] 214.2	262.3	262.3
Ø12/10	74.0	92.4	110.9	112.2	140.3	168.3	150.5	188.1	225.7	188.7	235.9	[1] 283.1	[1] 214.2	267.8	321.3

A clamping moment of $m_{Ed} = v_{Ed} \cdot z$ is covered by the existing rebend reinforcement. For larger moments, the tensile force coverage must be verified separately.
The concrete cover was set at c_{nom} = 28mm.

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Case e, without shear reinforcement in the floor slab – Pyraplex® Type B

Case e - Pyraplex® Type B

Assumptions: Shear load bearing capacity transverse to the joint
(according to DBV Data Sheet "Rebending", Figure 8 and DIN EN 1992-1-1 with NA),
surface of the protective casing SERRATED; $\sigma_{cp} = 0$;

maximum shear load bearing capacity of the connection without shear reinforcement in the floor slab: max $v_{Rd,c}$ [kN/m]

	Type B / 112			Type B / 142			Type B / 172			Type B / 202			Type B / 222		
	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37
Ø8/20	42.5	47.5	52.1	52.5	60.9	68.3	52.5	60.9	68.8	52.5	60.9	68.8	52.5	60.9	68.8
Ø8/15	42.5	47.5	52.1	55.8	62.4	68.3	69.1	77.2	84.6	70.0	81.2	91.7	70.0	81.2	91.7
Ø8/10	42.5	47.5	52.1	55.8	62.4	68.3	69.1	77.2	84.6	82.3	92.1	100.9	90.2	100.8	110.5
Ø10/20	42.5	47.5	52.1	55.8	62.4	68.3	62.8	72.9	82.3	62.8	72.9	82.3	62.8	72.9	82.3
Ø10/15	42.6	47.5	52.1	55.8	62.4	68.3	69.1	77.2	84.6	82.3	92.1	100.9	83.8	97.2	109.8
Ø10/10	48.7	52.5	55.8	58.4	62.9	68.3	69.1	77.2	84.6	82.3	92.1	100.9	90.2	100.8	110.5
Ø12/20	43.7	47.5	52.1	55.8	62.4	68.3	69.1	77.2	84.6	75.4	87.5	98.8	75.4	87.5	98.8
Ø12/15	48.1	51.8	55.0	57.6	62.4	68.3	69.1	77.2	84.6	82.3	92.1	100.9	90.2	100.8	110.5
Ø12/10	55.0	59.3	63.0	66.0	71.1	75.5	76.1	82.0	87.1	85.5	92.2	100.9	90.9	100.8	110.5

A clamping moment of $m_{Ed} = v_{Ed} \cdot z$ is covered by the existing rebend reinforcement. For larger moments, the tensile force coverage must be verified separately.
The concrete cover was set at $c_{nom} = 28mm$.

Case e, with shear reinforcement in the floor slab – Pyraplex® Type B

Case e - Pyraplex® Type B

Assumptions: Shear load bearing capacity transverse to the joint
(according to DBV Data Sheet "Rebending", Figure 8 and DIN EN 1992-1-1 with NA),
surface of the protective casing SERRATED; $\sigma_{cp} = 0$;

maximum shear load bearing capacity of the connection with shear reinforcement in the floor slab: max v_{Ed} [kN/m]

	Type B / 112			Type B / 142			Type B / 172			Type B / 202			Type B / 222		
	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37	C20/25	C25/30	C30/37
Ø8/20	58.3	67.7	76.4	58.3	67.7	76.4	58.3	67.7	76.4	58.3	67.7	76.4	58.3	67.7	76.4
Ø8/15	71.4	89.3	101.9	77.8	90.3	101.9	77.8	90.3	101.9	77.8	90.3	101.9	77.8	90.3	101.9
Ø8/10	71.4	89.3	107.1	109.7	135.4	152.9	116.7	135.4	152.9	116.7	135.4	152.9	116.7	135.4	152.9
Ø10/20	69.8	81.0	91.5	69.8	81.0	91.5	69.8	81.0	91.5	69.8	81.0	91.5	69.8	81.0	91.5
Ø10/15	72.7	90.8	109.0	93.1	108.0	121.9	93.1	108.0	121.9	93.1	108.0	121.9	93.1	108.0	121.9
Ø10/10	72.7	90.8	109.0	110.9	138.7	166.4	139.6	162.0	182.9	139.6	162.0	182.9	139.6	162.0	182.9
Ø12/20	74.0	92.4	106.5	83.7	97.2	109.7	83.7	97.2	109.7	83.7	97.2	109.7	83.7	97.2	109.7
Ø12/15	74.0	92.4	110.9	111.7	129.6	146.3	111.7	129.6	146.3	111.7	129.6	146.3	111.7	129.6	146.3
Ø12/10	74.0	92.4	110.9	112.2	140.3	168.3	150.0	174.1	196.5	162.5	188.6	212.9	167.5	194.4	219.5

A clamping moment of $m_{Ed} = v_{Ed} \cdot z$ is covered by the existing rebend reinforcement. For larger moments, the tensile force coverage must be verified separately.
The concrete cover was set at c_{nom} = 28mm.

Set up in Aachen in March 2016, translates into english January 2020,

Dr.-Ing. W. Roeser

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