4. Post-installed Reinforcement

4.1. General

The following chapter explains the design theory behind post-installed reinforcement bar (rebar) applications and is intended to provide the reader with a basic understanding on the design requirements related to the Eurocode EN 1992-1-1 and the European Assessment Document (EAD) 330087-00-0601.

Table 1 provides an overview of the specifications of our WIT-Rebar systems:

Table 1: Specification overview of WIT-Rebar systems

	WIT-PE 1000	WIT-UH 300	WIT-PE 510	WIT-VM 250
				
European Technical Assessment	ETA-19/0543	ETA-17/0036	ETA-20/1037	ETA-12/0166
Material	Two-component reactive resin mortar, pure epoxy	Urethane vinyl ester hybrid mortar	Two-component reactive resin mortar, pure epoxy	Two-component reaction resin mortar, vinyl ester
REBAR diameter	8 – 40 mm	8 – 32 mm	8 – 40 mm	8 – 32 mm
Drill hole cleaning with hollow drill-bit system	✔	✔	✔	X
Gelling- / working time at 20°C	30 min	3 min	30 min	6 min
Minimum curing time in dry concrete at 20°C	12 h	30 min	12 h	45 min
Minimum curing time in wet concrete at 20°C	24 h	60 min	24 h	90 min
Maximum embedment depth $l_{v,max}$	2000 mm	2000 mm	2000 mm	2000 mm
Temperature of base material in-service	-40°C – +80°C	-40°C – +80°C	-40°C – +80°C	-40°C – +80°C
Temperature of base material at installation	+5°C – +40°C	-5°C – +40°C	+5°C – +40°C	-10°C – +40°C
Fire resistance / Seismic / 100 years	✔ / ✔ / ✔	✔ / ✔ / ✔	✔ / X / X	✔ / X / X
Software / Eurocode	✔ / ✔	✔ / ✔	✔ / ✔	✔ / ✔

4.2. Anchor Theory vs. Rebar Theory

Table 2 shows a comparison of potential failure modes for rebar used as anchor and post-installed rebar connection.

Table 2: Comparison of potential failure modes

Rebar used as Anchor (EN 1992-4)		Post-installed Rebar Connection (EN 1992-1-1)	
Failure modes in Tension	**Failure modes in Shear**	**Failure modes in Tension**	**Failure modes in Shear**
Steel failure of fastener	Steel failure of fastener without lever arm	Steel failure of reinforcing bar	
	Steel failure of fastener with lever arm	Bond failure	
Pull-out failure of fastener	Concrete pry-out failure	Splitting failure	
Combined pull-out and concrete failure	Concrete edge failure		
Concrete cone failure			
Splitting failure			

4.2.1. Rebar used as Anchor

Situations where the concrete needs to take up tension loads from the anchorage or where reinforcing bars are designed to carry shear loads should be considered as "rebar used as anchors" and designed according to anchor design method such as given e.g. in the guidelines of EN 1992-4 or simplified in this Design Manual. Those guidelines verify all possible failure loads in tension and shear.

4.2.2. Post-installed Rebar Connection

The design of the rebar anchorage is performed according to structural concrete design codes, e.g. EN 1992-1-1. With a given test regime and the assessment criteria EAD 330087, it is proven that the load transfer for post-installed reinforcing bars is similar to cast in bars if the stiffness of the overall load transfer mechanism is similar to the cast-in system. The efficiency depends on the strength of the adhesive mortar against the concentrated load close to the ribs and on the capacity of load transfer at the interface of the drilled hole.

In many cases the bond values of post-installed bars are higher compared to cast in bars due to better performance of the adhesive mortar. But for small edge distance and/or narrow spacing, splitting or spalling forces become decisive due to the low tensile capacity of the concrete.

4.3. Post-installed rebar anchorage - The assessment criteria of EOTA-EAD 330087

The guideline specifies a number of tests in order to qualify products for post-installed rebar applications. These are the performance areas checked by the tests:

1. Bond strength in different strengths of concrete
2. Substandard hole cleaning in dry and wet concrete
3. Influence of temperature
4. Correct injection
5. Installation direction
6. Influence of sustained loads
7. Freeze-thaw conditions
8. High alkalinity and sulphurous atmosphere
9. Corrosion resistance
10. Resistance to fire

If an adhesive meets all assessment criteria, rebar connections carried out with this adhesive can be designed with the bond strength and minimum anchorage length according to EN 1992-1-1 as given in the tables below for different Würth injection mortars.

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Adhesives (or in conjunction with a certain drilling procedure) which do not fully comply with all assessment criteria can still obtain an approval:

- If the bond strength obtained in tests does not fulfil the specified requirements, then bond strengths lower than those given by EN 1992-1-1 shall be applied. These values are given in the respective approval.
- If it cannot be shown that the bond strength of reinforcing bars post-installed with a selected product and cast-in reinforcing bars in cracked concrete (w = 0.3 mm) is similar, then the minimum anchorage length $l_{b,min}$ and the minimum overlap length $l_{0,min}$ shall be increased by a factor 1.5.

4.4. Rebar Applications

Products tested according to above guideline can be used for applications in non-carbonated concrete C12/15 to C50/60 (EN 206) only, which are also allowed with straight deformed cast-in bars according to (EN 1992), e.g. those in the following applications:

Note to the following figures: In the figures below, no transverse reinforcement is plotted, the transverse reinforcement as required by EN 1992 shall be present. The shear transfer between old and new concrete shall be designed according to EN 1992.

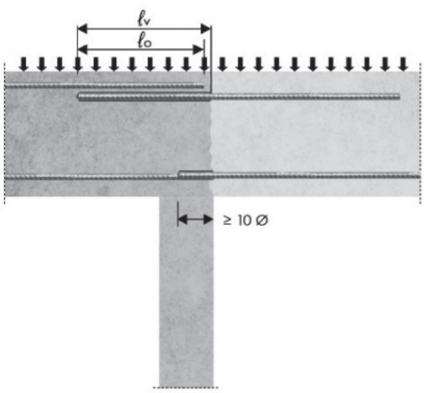

Figure 1: Overlap joint in slabs and beams

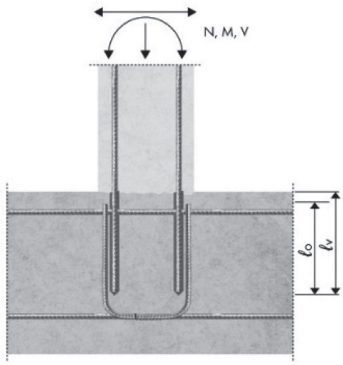

Figure 2: Overlap joint in a foundation of a column or wall where the rebars are stressed in tension

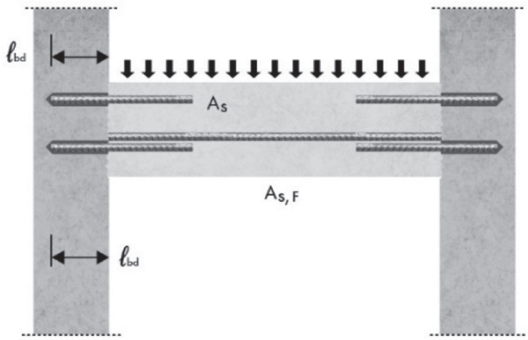

Figure 3: End anchoring of slabs or beams, designed as simply supported

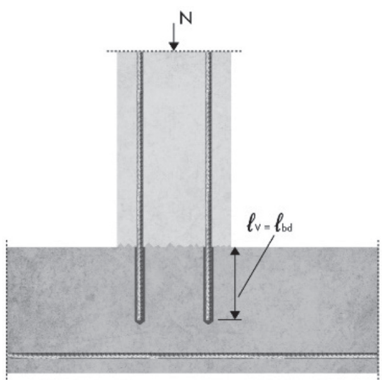

Figure 4: Rebar connection of components stressed primarily in compression. The rebars are stressed in compression

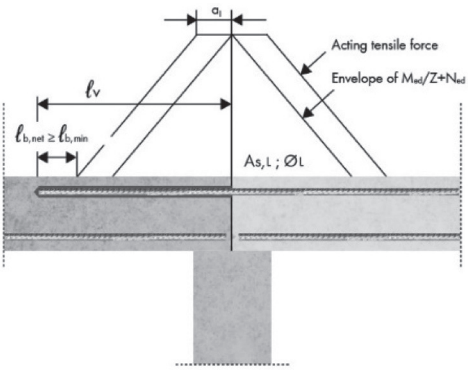

Figure 5: Anchoring of reinforcement to cover the line of acting tensile force

4.5. Design of Anchorage of longitudinal reinforcement with EN 1992-1-1

a) Reinforcing bars shall be so anchored that the bond forces are safely transmitted to the concrete avoiding longitudinal cracking or spalling. Transverse reinforcement shall be provided if necessary.

b) The ultimate bond strength shall be sufficient to prevent bond failure.

4.5.1. The design value of the ultimate bond stress

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot \frac{f_{ctk,0.05}}{1.5}$$

where:

- $f_{ck,\,0.05}$ ... is the 5% fractile characteristic tensile strength of concrete according to Table 3
- η_1 ... is a coefficient related to the quality of the bond condition and the position of the bar during concreting (details see EN 1992-1-1):
  - o η_1 = 1.0 when good conditions are obtained and
  - o η_1 = 0.7 for all other cases and for bars in structural elements built with slip-forms, unless it can be shown that good bond conditions exist
- η_2 ... is related to the bar diameter:
  - o η_2 = 1.0 for Ø ≤ 32 mm
  - o η_2 = (132 - Ø)/100 for Ø > 32 mm

Table 3: Strength characteristics for concrete

Compressive strength class		**C12/15**	**C16/20**	**C20/25**	**C25/30**	**C30/37**	**C35/45**	**C40/50**	**C45/55**	**C50/60**
f_{ck}	[N/mm²]	12	16	20	25	30	35	40	45	50
$f_{ck,cube}$	[N/mm²]	15	20	25	30	37	45	50	55	60
f_{cm}	[N/mm²]	20	24	28	33	38	43	48	53	58
f_{ctm}	[N/mm²]	1.60	1.90	2.20	2.60	2.90	3.20	3.50	3.80	4.10
$f_{ctk,\,0.05}$	[N/mm²]	1.10	1.30	1.50	1.80	2.00	2.20	2.50	2.70	2.90
$f_{ctk,\,0.95}$	[N/mm²]	2.00	2.50	2.90	3.30	3.80	4.20	4.60	4.90	5.30
f_{bd} Ø ≤ 32	[N/mm²]	1.65	1.95	2.25	2.70	3.00	3.30	3.75	4.05	4.35
f_{bd} Ø ≤ 34	[N/mm²]	1.62	1.91	2.21	2.65	2.94	3.23	3.68	3.97	4.26
f_{bd} Ø ≤ 36	[N/mm²]	1.58	1.87	2.16	2.59	2.88	3.17	3.60	3.89	4.18
f_{bd} Ø ≤ 40	[N/mm²]	1.52	1.79	2.07	2.48	2.76	3.04	3.45	3.73	4.00

4.5.2. Development length

Development length is the shortest length needed for a reinforcing bar so that the yield strength can be induced in the bar.

$$N_{Rd,b} = N_{Rd,s}$$

$$f_{bd} \cdot l_{b,develop} \cdot \emptyset \cdot \pi = {f_{yk}}/{\gamma_s} \cdot \frac{\pi \cdot \emptyset^2}{4}$$

$$l_{b,develop} = \frac{\emptyset}{4} \cdot \frac{{f_{yk}}/{\gamma_s}}{f_{bd}}$$

Reinforced concrete members are often designed using strut and tie models. The forces are represented by trusses and the nodes of these trusses have to connect the forces in such a way that they are in balance: The sum of the concrete compression force, the support force and the steel tensile force equals zero. The node can maintain its function only when the bond between the reinforcing bar and the surrounding concrete is activated and in balance with the horizontal component of the concrete compression strength. The node has to physically provide a certain length over which the rebar can develop stress on its left side. This extension on the left side is called "development length" or "anchorage length". The length or the space on the left side depends on the method of anchorage: bend, hook or straight.

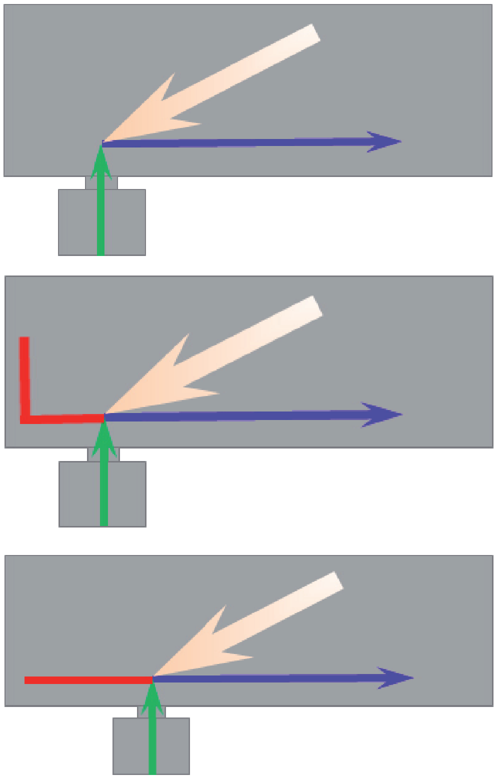

Figure 6: Node of trusses

Table 4: Design loads for good bond conditions

Bar size	Cross sectional area of reinforcement	Characteristic yield strength	Partial factor for reinforcing steel	Design resistance of rein-forcement bar	Design Bond stress	Development length	Minimum anchorage length	Design load for good bond condition, C20/25																									
Ø	A_s	f_{yk}	γ_s	$N_{Rd,s}$	f_{bd}	l_{bd}	$l_{b,min}$	$N_{b,d}$																									
[mm]	[mm²]	[N/mm²]		[kN]	[N/mm²]	[mm]		[kN]																									
8	50.3	500	1.15	21.9	2.25	386	116	7	7	8	8	11	14	17	20	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
10	78.5	500	1.15	34.1	2.25	483	145	-	-	-	11	14	18	21	25	28	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
12	113.1	500	1.15	49.2	2.25	580	174	-	-	-	-	17	21	25	30	34	42	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
14	153.9	500	1.15	66.9	2.25	676	203	-	-	-	-	-	25	30	35	40	49	59	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
16	201.1	500	1.15	87.4	2.25	773	232	-	-	-	-	-	28	34	40	45	57	68	79	-	-	-	-	-	-	-	-	-	-	-	-	-	-
20	314.2	500	1.15	136.6	2.25	966	290	-	-	-	-	-	-	42	49	57	71	85	99	113	127	-	-	-	-	-	-	-	-	-	-	-	-
25	490.9	500	1.15	213.4	2.25	1208	362	-	-	-	-	-	-	-	-	71	88	106	124	141	159	177	194	212	-	-	-	-	-	-	-	-	-
28	615.8	500	1.15	267.7	2.25	1353	406	-	-	-	-	-	-	-	-	-	99	119	139	158	178	198	218	238	257	-	-	-	-	-	-	-	-
32	804.2	500	1.15	349.7	2.25	1546	464	-	-	-	-	-	-	-	-	-	113	136	158	181	204	226	249	271	294	317	339	-	-	-	-	-	-
34	907.9	500	1.15	394.7	2.21	1676	503	-	-	-	-	-	-	-	-	-	-	141	165	188	212	236	259	283	306	330	353	377	-	-	-	-	-
36	1017.9	500	1.15	442.6	2.16	1812	543	-	-	-	-	-	-	-	-	-	-	147	171	195	220	244	269	293	318	342	366	391	415	440	-	-	-
40	1256.6	500	1.15	546.4	2.07	2100	630	-	-	-	-	-	-	-		-	-	-	182	208	234	260	286	312	338	364	390	416	442	468	494	520	546
Anchorage length [mm]								120	130	140	150	200	250	300	350	400	500	600	700	800	900	1000	1100	1200	1300	1400	1500	1600	1700	1800	1900	2000	2100

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4.5.3. Basic anchorage length

The calculation of the required anchorage length shall take into consideration the type of steel and bond properties of the bars. The basic required anchorage length $l_{b,rqd}$ for anchoring the force $A_s \sigma_{sd}$ in a bar assuming constant bond stress equal to f_{bd} follows from:

$$l_{b,rqd} = \frac{\emptyset}{4} \cdot \frac{\sigma_{Sd}}{f_{bd}}$$

4.5.4. Design anchorage length

According to EN 1992-1-1, the design anchorage length, l_{bd} is

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

The factors a_1 to a_5 subscripts take into account the form of the bars, concrete cover, confinement by transverse reinforcement, the influence of welded transverse bars along the design anchorage length and the effect of the pressure transverse to the plane of splitting along the design anchorage length.

In case of a post-installed rebar application, only straight bars are possible.

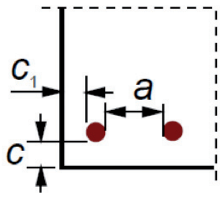

Fig. 1: Values for straight bars in beams and slabs (EN 1992-1-1) Note: c_d = min ($a/2$, c_1, c)

- a_1 = 1.0 for anchorage of straight bars
- a_2: $0.7 \leq 1 - 0.15\,(c_d - \emptyset)/\emptyset \leq 1.0$ for reinforcement bar in tension or a_2 = 1.0 for reinforcement bar in compression
- a_3 = 1.0 no transverse reinforcement
- a_4 = 1.0 no welded transverse reinforcement
- a_5: $0.7 \leq 1 - 0.04\rho \leq 1.0$ for confinement by transverse pressure ρ [MPa] at ultimate limit state along l_{bd}

➢ The product of ($a_2 a_3 a_5$) should be ≥ 0.7.

$l_{b,min}$ is the minimum anchorage length if no other limitation is applied:

- $l_{b,min} \geq \max\,(0.3 \cdot l_{b,rqd};\ 10\emptyset;100\ \text{mm})$ for anchorages in tension
- $l_{b,min} \geq \max\,(0.6 \cdot l_{b,rqd};\ 10\emptyset;100\ \text{mm})$ for anchorages in compression

The minimum anchorage length shall be multiplied by the amplification factor a_{lb} according Table 5 below:

Table 5: Amplification factor a_{lb} related to drilling method for concrete class C12/15 to C50/60

Injection mortar	Drilling method	Bar size	Amplification factor a_{lb}
WIT-PE 1000	All drilling methods	8 mm to 40 mm	1.0
WIT-UH 300	Hammer drilling (HD) Hollowing drill bit system (HDB) Compressed air drilling (CD)	8 mm to 32 mm	1.0
WIT-PE 510	Hammer drilling (HD) Hollowing drill bit system (HDB) Compressed air drilling (CD)	8 mm to 40 mm	1.0
WIT-VM 250	Diamond coring (DD)	8 mm to 40 mm	1.5
	Hammer drilling (HD) Hollowing drill bit system (HDB) Compressed air drilling (CD)	8 mm to 32 mm	1.0

4.5.5. Lap or splice length

According to EN 1992-1-1, the design lap length is
$l_0 = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b,rqd} \geq l_{0,min}$

- α_1 = 1.0 for anchorage of straight bars
- α_2 = 1.0 for reinforcement bar in compression
- α_2: $0.7 \leq 1 - 0.15\ (c_d - \varnothing)/\varnothing \leq 1.0$ for reinforcement bar in tension
- α_3 = 1.0 no transverse reinforcement
- α_5: $0.7 \leq 1 - 0.04\rho \leq 1.0$ for confinement by transverse pressure ρ [MPa] along l_{bd}
- α_6: $1.0 \leq \alpha_6 \leq 1.5$ for influence of percentage of lapped bars relative to the total cross-section area according to the following table:

Table 6: Values of the coefficient

Percentage of lapped bars relative to the total cross-section area	< 25%	33%	50%	>50%
α_6	1.00	1.15	1.40	1.50

Note: Intermediate values may be determined by interpolation

➢ The product of $(\alpha_2 \alpha_3 \alpha_5)$ should be ≥ 0.7.

$l_{0,min}$ is the minimum lap length:

$$l_{0,min} \geq \max\ (0.3 \cdot \alpha_6 \cdot l_{b,rqd};\ 15\varnothing;\ 200\ \text{mm})$$

The minimum lap length shall be multiplied by the amplification factor α_{lb} according Table 5.

4.5.6. Concrete cover

Concrete cover is defined as the minimum distance between the outer surface of the concrete element and the surface of the embedded reinforcement. The nominal concrete cover is defined as a minimum cover plus a deviation allowance Δc_{dev}. The recommended value for Δc_{dev} = 10 mm.
$c_{nom} = c_{min} + \Delta c_{dev}$

The minimum concrete cover c_{min} is to ensure safe transmission of bond forces and protection against steel and fire is defined according to the following equation:

$$c_{min} = \max\ (c_{min,b}; c_{min,dur}; 10\ \text{mm})$$

where

- $c_{min,b}$ = minimum cover due to bond requirement
- $c_{min,dur}$ = minimum cover due to environmental conditions

a) $c_{min,b}$ is equivalent to the diameter of the reinforcing bar.

b) $c_{min,dur}$ can be obtained from Table 8:

Table 8: Values of minimum cover $c_{min,dur}$ requirements with regard to durability for reinforcement steel

		Exposure Class						
		X0	**XC1**	**XC2/XC3**	**XC4**	**XD1/XS1**	**XD2/XS2**	**XD3/XS3**
Structural Class	**S1**	10	10	10	15	20	25	30
	S2	10	10	15	20	25	30	35
	S3	10	10	20	25	30	35	40
	S4	10	15	25	30	35	40	45
	S5	15	20	30	35	40	45	50
	S6	20	25	35	40	45	50	55

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According to the exposure class in a given situation, the table below from EN 1992-1-1 further provides the engineer with the indicative minimum concrete strength class for each exposure class:

Table 9: Indicative minimum strength class

Corrosion										
	Carbonation-induced corrosion				**Chloride-induced corrosion**			**Chloride-induced corrosion from sea-water**		
	XC1	XC2	XC3	XC4	XD1	XD2	XD3	XS1	XS2	XS3
Indicative minimum strength class	C20/25	C25/30	C30/37		C30/37		C35/45	C30/37	C35/45	
Damage to Concrete										
	No risk	**Freeze/Thaw Attack**			**Chemical Attack**					
	X0	XF1	XF2	XF3	XA1	XA2			XA3	
Indicative minimum strength class	C12/15	C30/37	C25/30	C30/37	C30/37				C35/45	

For our WIT-Rebar systems, the concrete cover shall be defined as

$$\min c = \max\left(c_{nom}; c_{min,inst}\right)$$

The minimum cover of post-installed reinforcing bars $c_{min,inst}$ depends on the drilling method:

Table 10: Minimum cover related to drilling method

Drilling method	**Rebar diameter (∅)**	**Without drilling aid**	**With drilling aid**
Hammer drilling (HD) Hollow drill bit system (HDB)	< 25 mm	30 mm + 0.06 l_v ≥ 2∅	30 mm + 0.02 l_v ≥ 2 ∅
	≥ 25 mm	40 mm + 0.06 l_v ≥ 2∅	40 mm + 0.02 l_v ≥ 2 ∅
Diamond drilling (DD)	< 25 mm	Drill rig used as drilling aid	30 mm + 0.02 l_v ≥ 2 ∅
	≥ 25 mm		40 mm + 0.02 l_v ≥ 2 ∅
Compressed air drilling	< 25 mm	50 mm + 0.08 l_v	50 mm + 0.02 l_v
	≥ 25 mm	60 mm + 0.08 l_v	60 mm + 0.02 l_v

Comment: The minimum concrete cover acc. EN 1992-1-1:2004+AC:2010 must be observed

4.5.7. Spacing of bars and laps

The spacing of bars shall be such that the concrete can be placed and compacted satisfactorily for the development of adequate bond. The clear distance (horizontal and vertical) between individual parallel bars or horizontal layers of parallel bars should be not less than the max(∅; (d_g + 5 mm) or 20 mm) where d_g is the maximum size of aggregate (8.2; EN 1992-1-1:2011-01).

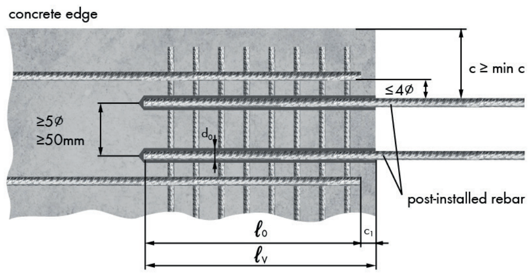

Figure 7: Adjacent laps

The spacing between post-installed reinforcing bars shall be greater max (5∅; 50 mm).

4.5.8. Embedment depth

Embedment depth for overlap joints
For calculation of the effective embedment depth of overlap joints the concrete cover at end-face of bonded-in rebar c_1 shall be considered:

$$l_v \geq l_0 + c_1$$

If the clear distance between the overlapping rebar is greater than 4 Ø the lap length shall be enlarged by the difference between the clear distance and 4 Ø.

4.5.9. Maximum embedment depth

Table 11: Maximum approved embedment depth for WIT-Rebar systems

Bar size, ∅ [mm]		**8**	**10**	**12**	**14**	**16**	**20**	**25**	**28**	**32**	**34**	**36**	**40**
Mortar	Drilling Method *	**Maximum permissible embedment depth, l_{max} [mm]**											
WIT-PE 1000	HD / CD / DD	800	1000	1200	1400	1600	2000	2000	2000	2000	2000	2000	2000
	HDB	800	1000	1000	1000	1000	1000	1000	1000	1000	-	-	-
WIT-UH 300	All methods	1000	1000	1200	1400	1600	2000	2000	2000	2000	-	-	-
WIT-PE 510	HD / CD / DD	800	1000	1200	1400	1600	2000	2000	2000	2000	2000	2000	2000
	HDB	800	1000	1000	1000	1000	1000	1000	1000	1000	-	-	-
WIT-VM 250	All methods	1000	1000	1200	1400	1600	2000	2000	1000	1000	-	-	-

* HD = Hammer drilling, CD = Compressed air drilling, HDB = Hollow drill bit system, DD = Diamond drilling

4.5.10. Transverse reinforcement

The requirements of transverse reinforcement in the area of the post-installed rebar connection shall comply with EN 1992-1-1, Section 8.7.4.

4.5.11. Connection joint

The transfer of shear forces between new concrete and existing structure shall be designed according to EN 1992-1-1, Section 6.2.5 "Shear at the interface between concrete cast at different times". The joints for concreting must be roughened to at least such an extent that aggregate protrude. In case of a carbonated surface of the existing concrete structure the carbonated layer shall be removed in the area of the post-installed rebar connection with a diameter of (Ø + 60 mm) prior to the installation of the new rebar. The depth of concrete to be removed shall correspond to at least the minimum concrete cover for the respective environmental conditions in accordance with EN 1992-1-1. The foregoing may be neglected if building components are new and not carbonated and if building components are in dry conditions.

4.5.12. Failure modes and anchorage length

In most cases the reinforcement bars are placed close to the surface of the concrete member to achieve good crack distribution and economical bending capacity. For splices at wide spacing, the bearing capacity of the concrete depends only on the thickness of the concrete cover. At narrow spacing the bearing capacity depends on the spacing and on the thickness of the cover. In the design codes the reduction of bearing capacity of the cover is taken into account by means of multiplying factors for the splice length. Splitting failure is decisive if the radial cracks propagate through the entire cover. Bond failure is caused by pull-out of the bar if the confinement (concrete cover, transverse reinforcement) is sufficient to prevent splitting of the concrete cover. EN 1992-1-1 controls the failure modes by limiting the a_2 value to $a_2 \geq 0.7$. The spalling of the concrete cover or splitting between bars will be the controlling mode of failure. The value a_2 gives an explicit consideration for splitting and spalling as a function of concrete cover and bar spacing.

If a_2 is less than 0.7, corresponding to cover dimensions of $c_d/\varnothing > 3$ or spacing of $a/\varnothing > 6$, the cover or spacing is large enough so that splitting cannot occur anymore and pull-out will control.

4.6. Fire load case

4.6.1. General information

The load-bearing capacity in case of fire corresponds to the performance characteristic R according to DIN EN 13501-2. A classification of performance characteristics in case of fire according to DIN EN 13501-2 requires a time-dependent fire stress according to the unit temperature time curve (ETK), which is defined in DIN EN 13631. The National Annex to DIN EN 1991-1-2 also requires the application of standard time/temperature curve at any point of the structure for structural elements in building construction. If a sufficient load-bearing capacity under ETK load has been verified, this verification applies irrespective of the later use.

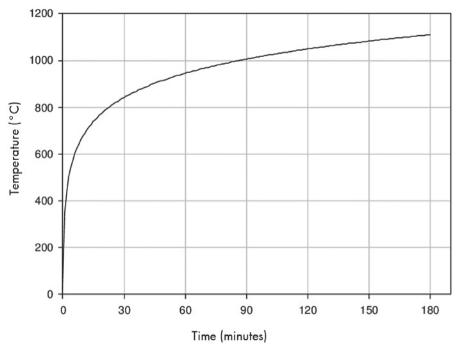

Figure 8: Standard time/temperature curve ISO 834

4.6.2. Application cases

To determine the load-bearing capacity of reinforcement connections in the event of fire, a basic distinction must be made between two applications. In application A, the thermally stressed surface shows the same direction as the reinforcement, which leads to a locally constant but time-varying temperature along the anchorage length l_{bd}.

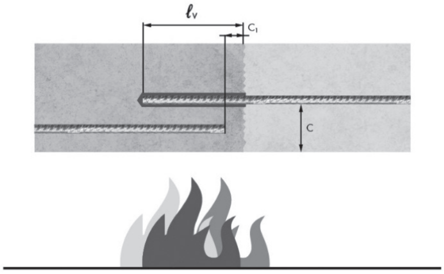

Figure 9: Fire load case - Application A

Alternatively for application B, the post-installed rebar is perpendicular to the thermally stressed surface, which results in a temporally and spatially variable temperature profile along the anchorage length l_{bd}.

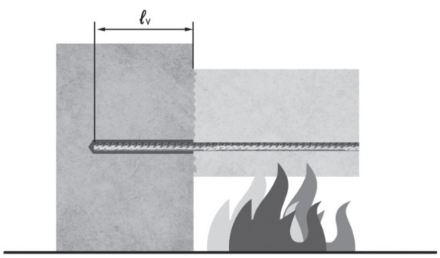

Figure 10: Fire load case - Application B

The distinction between application cases A and B is made exclusively according to the orientation of the flame-exposed surfaces in relation to the direction of the post-installed rebar and is not the same as the distinction between end anchoring and lap joint.

4.6.3. Load-bearing capacity

The load-bearing capacity of post-installed rebar connections in case of fire is significantly affected by the temperature-dependent bond stress $f_{bd,fi}$ (Θ) with

$$f_{bd,fi}(\theta) = k_{fi} \cdot f_{bd,fi}(\theta) \cdot f_{bd,PIR} \cdot \frac{\gamma_c}{\gamma_{M,fi}}$$

which is determined by experimental techniques. The reduction factor k_{fi} (Θ) under fire stress, the design value f_{bd} of the bond stress in cold case according to DIN EN 1992-1-1, which depends on the concrete strength class, and the reduction factor k_b with $f_{bd,PIR} = k_b \cdot f_{bd}$ are specified in the relative ETA. According to DIN EN 1992-1-1, Table 2.1N in accordance with the corresponding national annex for the permanent and temporary design situation, the following applies to the partial safety factor of concrete in cold conditions

$$\gamma_c = 1.5$$

In case of fire, the following applies according to DIN EN 1992-1-2, chapter 2.3 in accordance with the corresponding national appendix for the partial safety factor of concrete

$$\gamma_{M,fi} = 1.0$$

The design values f_{bd} of the composite stress in cold case are shown in Table 3. The values are applicable for all drilling methods, but they depend on the reinforcement bar diameter and are valid for good bond conditions according to DIN EN 1992-1-1, chapter 8.4.2. In case of other bond conditions, the specified values have to be multiplied by a factor of 0.7.

For WIT-PE 1000, the factor k_b can be found in ETA19/0543 in Table C2.

$$k_b = 1.0$$

and thus for all cases

$$f_{bd,PI} = f_{bd}$$

The temperature-dependent reduction factor $k_{b,fi(\Theta)}$ is (depending on the ETA) to be considered. The graph below shows reduction factors $k_{b,fi(\Theta)}$ for all WIT-Rebar systems

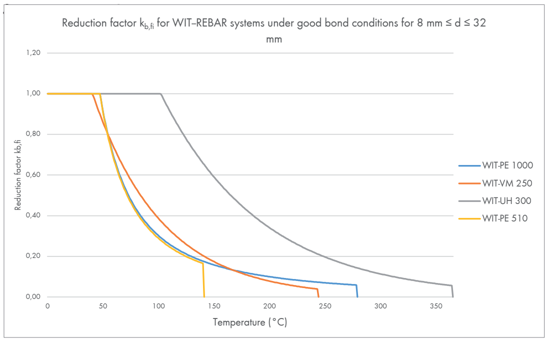

Figure 11: Reduction factor for WIT-Rebar systems (drilling methods HD/HBD/CD) for 8 mm ≤ d ≤ 32 mm

When designing post-installed rebar connections in case of fire, a distinction must be made between pull-out failure and steel failure, in addition to the distinction between application cases A and B.

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4.6.4. Application case A

If the rebar connection in application A is in the same direction as the flamed surface, the function of the unit time-temperature curve results in a temperature along the rebar connection that varies over time but is locally constant. The time-dependent reinforcement temperature in case of fire is only dependent on the geometry of the existing component and the design in case of fire can be carried out using the time-dependent reinforcement temperature $\Theta(t)$ and the time-dependent bond stress $f_{bd,fi}$ ($\Theta(t)$).

4.6.4.1. Pull-out

If the stresses acting on a rebar connection are greater than the bond force that can be absorbed, failure occurs due to pull-out. The proof for the failure mode pull-out is performed in application A by determining the anchorage length $l_{b,rqd,fi}$ (t) required in case of fire. The value $l_{b,rqd,fi}$ (t) describes the basic value of the anchorage length in case of fire and is to be determined according to DIN EN 1992-1-1, equation (8.3) under consideration of the time- and temperature-dependent bond stress.

$$l_{b,rqd,fi}(t) = \frac{\emptyset}{4} \cdot \frac{\sigma_{sd,fi}}{f_{bd,fi}\,(\theta_{(t)})}$$

with

- $\emptyset$ = diameter of the reinforcement bar
- $\sigma_{sd,fi}$ = existing steel stress of the bar at the beginning of the anchorage length in the ultimate limit state under extraordinary design situation according to DIN EN 1990

The design value $l_{bd,fi}$ (t) of the anchorage length in case of fire is obtained analogously to the check under normal temperature according to DIN EN 19921-1, chapter 8.4.4.

4.6.4.2 Steel failure

The temperature-dependent load capacity of the rebar itself is limited by the load capacity of the steel cross-section. According to DIN EN 1992-1-2, Chapter 5.2(4), the reinforcement of statically determinate reinforced concrete structures may be verified in case of fire by means of a temperature criterion. The critical temperature is Θ_{crit} = 500°C. The proof for steel failure is therefore provided if the following applies to the most unfavorable (i.e. warmest) point of the rebar in the post-installed rebar connection.

$$\theta(t) \leq \theta_{crit} = 500°C$$

Alternatively, the verification of the reinforcing bar for steel failure in case of fire can be done by comparing the acting and the bond (tensile) force.

$$N_{fi.\theta(t),Rd} \geq N_{fi,Ed}$$

with $N_{fi,Ed}$: Stress on the bar at the beginning of the anchorage length in the ultimate limit state in case of an extraordinary design situation according to DIN EN 1990. The force that can be sustained in case of fire must be determined taking into account the temperature-dependent decrease of the yield strength according to DIN EN 1992-1-2, Table 3.2a.

$$f_{sy,\theta(t)} = k_{y\,\theta(t)} \cdot f_{yk}$$

You get the bond tensile force in case of fire as:

$$N_{fi,\theta(t),Rd} = k_{y\theta(t)} \cdot f_{yk} \cdot \frac{\pi \,.\, \theta^2}{4} \cdot \frac{1}{\gamma_{M,fi}}$$

In case of fire, the following applies in accordance with DIN EN 1992-1-2, Chapter 2.3 in accordance with the corresponding National Annex for the partial safety factor of reinforcing steel

$$\gamma_{M.fi} = 1.0$$

Table 12: Temperature for different fire durations vs. concrete cover

c [cm]	T [°C] with member thickness = 30 cm					
	Fire duration [min]					
	30	60	90	120	180	240
2	348	516	614	684	783	853
3	242	399	496	566	667	740
4	167	311	403	471	571	644
5	117	241	328	394	491	564
6	88	187	268	330	424	495
7	68	144	218	277	367	435
8	53	114	177	232	318	384
9	42	93	143	193	275	339
10	34	77	118	161	238	299
11	29	64	100	135	205	264
12	26	54	85	115	177	233
13	24	46	73	99	153	205
14	22	39	63	87	132	180
15	21	34	55	76	116	159
16	21	30	48	67	103	140
17	20	27	42	59	92	125
18	20	25	37	52	83	112
19	20	24	33	46	75	102
20	20	23	30	42	67	93
21	20	22	28	38	61	85
22	20	21	26	34	55	79
23	20	21	25	31	51	72
24	20	21	23	29	47	67
25	20	20	23	27	43	63

Table 13: Bond strength for different fire durations

	$f_{bd,fi}$ [N/mm2] for member thickness = 30 cm									
	Good bond conditions, C20/25									
	8 mm ≤ d ≤ 32 mm									
	WIT-PE 1000						WIT-UH 300			
c [cm]	Fire duration [min]									
	30	60	90	120	180	240	30	60	90	120
2	-	-	-	-	-	-	0.23	-	-	-
3	0.25	-	-	-	-	-	0.74	-	-	-
4	0.46	-	-	-	-	-	1.69	0.35	-	-
5	0.80	0.25	-	-	-	-	2.91	0.74	0.29	-
6	1.26	0.38	0.22	-	-	-	3.45	1.36	0.56	0.28
7	1.93	0.58	0.30	0.20	-	-	3.45	2.17	0.96	0.50
8	2.89	0.84	0.42	0.27	-	-	3.45	3.03	1.52	0.83
9	3.45	1.16	0.58	0.36	0.21	-	3.45	3.45	2.19	1.26
10	3.45	1.57	0.80	0.48	0.26	-	3.45	3.45	2.89	1.80
11	3.45	2.11	1.04	0.64	0.33	0.22	3.45	3.45	3.45	2.40
12	3.45	2.80	1.34	0.84	0.42	0.27	3.45	3.45	3.45	3.00
13	3.45	3.45	1.71	1.05	0.53	0.33	3.45	3.45	3.45	3.45
14	3.45	3.45	2.17	1.30	0.66	0.40	3.45	3.45	3.45	3.45
15	3.45	3.45	2.73	1.61	0.82	0.50	3.45	3.45	3.45	3.45
16	3.45	3.45	3.40	1.98	0.99	0.61	3.45	3.45	3.45	3.45
17	3.45	3.45	3.45	2.43	1.18	0.73	3.45	3.45	3.45	3.45
18	3.45	3.45	3.45	2.95	1.40	0.87	3.45	3.45	3.45	3.45
19	3.45	3.45	3.45	3.45	1.66	1.01	3.45	3.45	3.45	3.45
20	3.45	3.45	3.45	3.45	1.95	1.17	3.45	3.45	3.45	3.45
21	3.45	3.45	3.45	3.45	2.29	1.34	3.45	3.45	3.45	3.45
22	3.45	3.45	3.45	3.45	2.66	1.53	3.45	3.45	3.45	3.45
23	3.45	3.45	3.45	3.45	3.07	1.74	3.45	3.45	3.45	3.45
24	3.45	3.45	3.45	3.45	3.45	1.96	3.45	3.45	3.45	3.45
25	3.45	3.45	3.45	3.45	3.45	2.19	3.45	3.45	3.45	3.45

4.6.5. Application case B

If the rebar connection in application B is perpendicular to the direction of the flamed surface, the temperature along the rebar connection changes over time and place - the temperature decreases with increasing distance from the flamed surface.

4.6.5.1. Pull out

A design in case of fire for the failure type pull-out in the form of the determination of a single time-dependent composite stress $f_{bd,fi}$ (Θ(t)) is not sufficient for application B because this stress is variable along the reinforcement connection. A procedure analogous to application A would therefore result in an additional required anchoring length $l_{b,rqd,fi}$ (t) at each point of the rebar.

On the safe side, it is of course conceivable and permissible to determine the required anchorage length $l_{b,rqd,fi}$ (t) analogous to application A, taking into account the most unfavorable (i.e. highest) temperature of the rebar in the existing component. However, the results obtained in this way can be considered as extremely conservative with increasing anchorage length.

A more economical approach, which makes use of the actual load-bearing capacity of the bonded joint, is to prove pull-out failure in application B by comparing the acting and the absorbing forces

$$N_{bd,fi,Rd\,(t)} \geq N_{fi,Ed}$$

The bond force $N_{bd,fi,Rd}$ (t) in the composite joint is obtained by integrating the temperature-dependent composite stress $f_{bd,fi}$ (Θ(t)) via the load-transmitting surface of the rebar

$$N_{bd,fi,Rd}(t) = \pi \cdot \emptyset \cdot \int_0^{l_v} f_{bd,fi}\left(\theta\ (t,x)\right)dx$$

with l_v : Development depth. The bond and acting forces are identical:

$$N_{bd,fi,Rd\,(t)} = N_{fi,Ed}$$

The development depth l_v for a defined time t corresponds to the required anchorage length $l_{b,rqd,fi}$ (t) according to the corresponding ETA and DIN EN 1992-1-1, Equation (8.3). Analogous to the application case A and the cold case, the design value $l_{bd,fi}$ (t) of the anchorage length in case of fire shall be determined according to DIN EN 1992-1-1, chapter 8.4.4.

4.6.5.2. Steel failure

In contrast to the failure due to pull-out, the check for steel failure must be performed at the most unfavorable check section, i.e. taking into account the maximum temperature occurring along the reinforcement bars at a given time t. The verification can be performed analogous to application A by means of the temperature criterion or by comparing the acting and the absorbing force.

Table 14: Tension load for different fire duration

$N_{bd,fi}$ [kN] d = 16 mm										
Good bond conditions, C20/25										
	WIT-PE 1000						WIT-UH 300			
l_v [cm]	Fire duration [min]									
	30	60	90	120	180	240	30	60	90	120
16	20.1	13.6	8.4	4.8	2.2	1.2	24.8	20.0	16.4	13.4
18	23.5	17.1	12.2	7.7	3.5	2.0	28.3	23.5	19.9	16.9
20	27.0	20.5	15.7	11.5	5.3	3.0	31.7	26.9	23.3	20.4
22	30.5	24.0	19.1	15.1	7.9	4.5	35.2	30.4	26.8	23.9
24	33.9	27.5	22.7	18.6	11.4	6.4	38.7	33.9	30.3	27.3
25	35.7	29.3	24.3	20.3	13.4	7.7	40.4	35.6	32.0	29.1
26	37.4	30.9	26.1	22.1	15.2	9.1	42.1	37.4	33.7	30.8
28	40.9	34.4	29.5	25.6	18.8	12.6	45.6	40.8	37.2	34.3
30	44.3	37.9	33.0	29.1	22.4	16.5	49.1	44.3	40.7	37.8
32	47.8	41.4	36.5	32.5	25.8	20.1	52.5	47.8	44.2	41.2
34	51.3	44.8	39.9	36.0	29.3	23.7	56.0	51.2	47.6	44.7
36	54.7	48.3	43.4	39.5	32.8	27.2	59.5	54.7	51.1	48.2
38	58.2	51.8	46.9	42.9	36.2	30.7	62.9	58.2	54.6	51.7
40	61.7	55.3	50.3	46.4	39.7	34.2	66.4	61.6	58.0	55.1
45	70.3	63.9	59.0	55.1	48.4	42.8	75.1	70.3	66.7	63.8
50	79.0	72.5	67.7	63.7	57.1	51.5	83.7	79.0	75.4	72.4
55	87.7	81.2	76.4	72.4	65.7	60.2	92.4	87.6	84.0	81.1
60	96.4	89.9	85.1	81.1	74.4	68.8	101.1	96.3	92.7	89.8
65	105.0	98.6	93.7	89.7	83.0	77.5	109.8	105.0	101.4	98.5
70	113.7	107.3	102.4	98.4	91.7	86.2	118.4	113.7	110.1	107.1
75	122.4	115.9	111.1	107.1	100.4	94.8	127.1	122.3	118.7	115.8
80	131.0	124.6	119.7	115.7	109.1	103.5	135.8	131.0	127.4	124.4
85	139.7	133.3	128.4	124.4	117.7	112.2	144.4	139.7	136.1	133.1
90	148.4	141.9	137.1	133.1	126.4	120.9	153.1	148.3	144.7	141.8
95	157.1	150.6	145.8	141.8	135.1	129.5	161.8	157.0	153.4	150.5
100	165.7	159.3	154.4	150.4	143.8	138.2	170.5	165.7	162.1	159.1
110	183.1	176.6	171.8	167.8	161.1	155.5	187.8	183.0	179.4	176.5
120	200.4	193.9	189.1	185.1	178.5	172.9	205.1	200.4	196.8	193.8
130	217.7	211.3	206.5	202.5	195.8	190.2	222.5	217.7	214.1	211.2
140	235.1	228.6	223.8	219.8	213.1	207.6	239.8	235.0	231.4	228.5
150	252.4	246.0	241.1	237.1	230.5	224.9	257.2	252.4	248.8	245.8
160	269.8	263.3	258.5	254.5	247.8	242.3	274.5	269.7	266.1	263.2

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