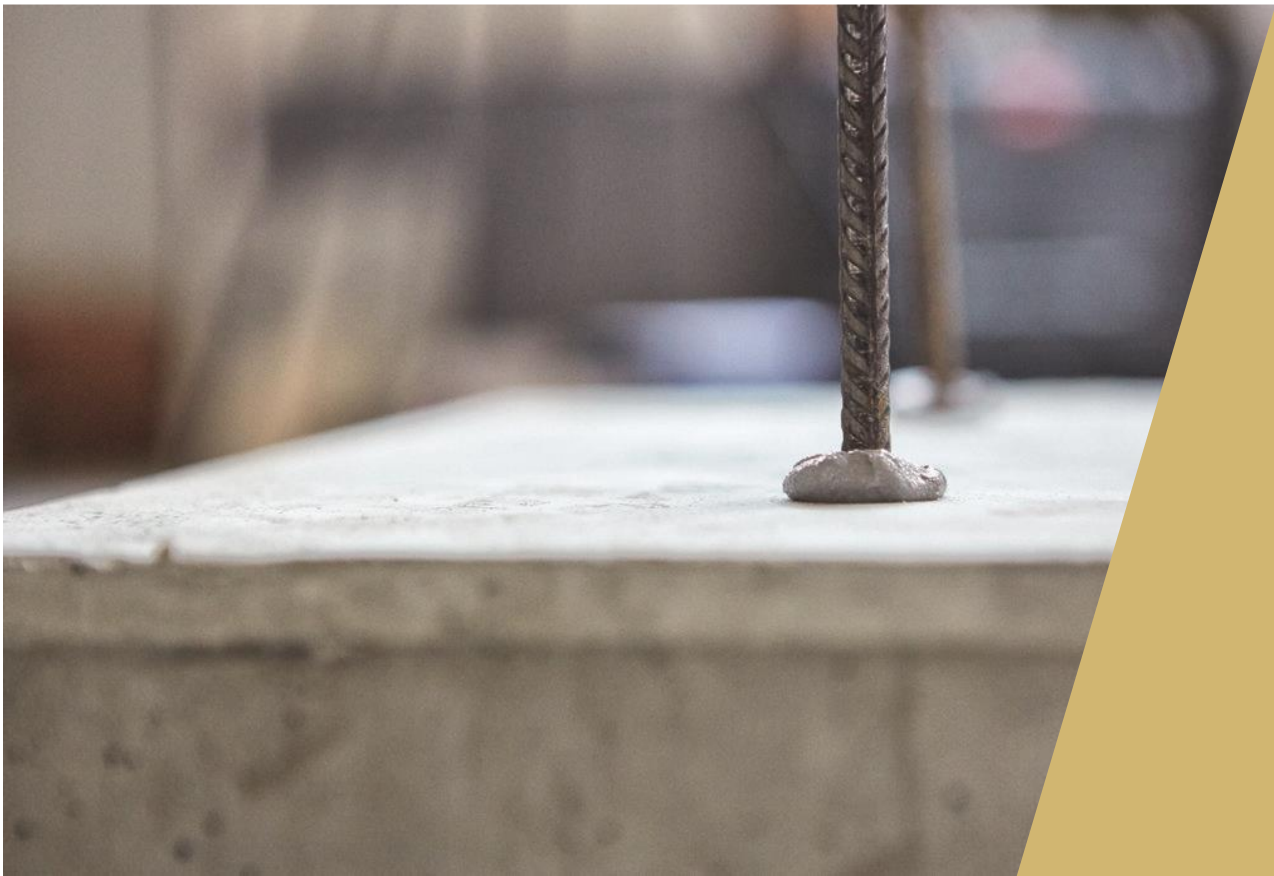
STRENGTHENING AND REHABILITATION WITH CONCRETE OVERLAYS FOR BRIDGES, TUNNELS AND CIVIL STRUCTURES

Structural principles and design
for redundant systems



Foreword

Placing fresh concrete against existing, hardened concrete is a routine task in building construction. In fact, it is a condition which occurs at every joint in concrete construction work. For some time now, the placement of concrete overlays has been gaining in importance as a result of the increasing need for rehabilitation and strengthening of existing structures. For the design of these composite concrete structures, the transfer of internal stresses across the bond interface between new and old concrete is a critical aspect. A design method has been developed based on shear tests specially carried out for this purpose by Hilti Corporate Research for a variety of surface treatments.

The Institute for Concrete Structures of the University of Innsbruck, Austria, provided scientific support during the development of the connectors and the associated design method. At the same time, test results given in the literature were incorporated. Among other things it was found that, contrary to the usual design approach, the full tensile yield strength of the connectors cannot be equated to the tension clamping force across the interface.

Former Professor Dr. techn. Manfred Wicke,
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In contrast to design methods described in the literature, this new design approach considers all three mechanisms: cohesion, friction, and shear resistance (dowel action) of the shear reinforcement positioned across the interface, in determining the effective shear transfer. The compressive stress required at the interface for shear transfer by friction is set up by activating tensile forces in the connectors. The design method is based on a single equation to calculate the resistance of the bond interface for each different surface treatment from the three above components.

With increasing surface roughness, shear resistance and shear stiffness are significantly increased. Furthermore, the distribution of total resistance shared by the three components changes considerably. At the extremes, if the surfaces are very rough, the connectors across the bond interface are primarily stressed in tension, whereas, if the surfaces are smooth, the shear resistance of the connectors themselves (dowel action) predominates. For roughened surfaces, the interlocking effect is sufficient to transfer small shear forces without connectors. It is often adequate for concrete overlays to be anchored only at their perimeter.

The very user-friendly Hilti design method is based on the Eurocode safety concept (prEN 1992-4) and is particularly notable for its transparency. The use of design diagrams makes the method straightforward for designers to apply.

Manfred Wicke

For comments and questions, contact our engineers

Contents

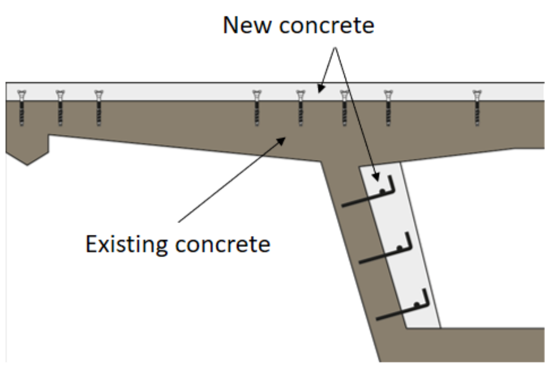
1 Hilti connectors for overlays

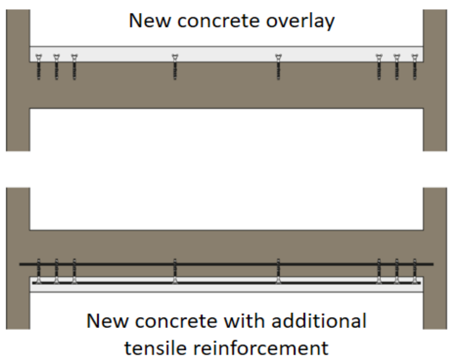
1.1 Range of applications

When a new layer of concrete is applied to existing concrete with the aim of strengthening or repairing a structure, the result is referred to as a composite concrete structure. The overlay is usually cast directly or applied as shotcrete. Its function is to augment the flexural compression or flexural tension zones, depending on the position of placement. Prior to placement of the overlay, the surface of the old concrete member is prepared by suitable means, and pre-wetted.

Shrinkage of the new concrete overlay can be reduced by careful selection of the concrete mix. However, the constraint forces caused by differential shrinkage and, in certain cases, by differential temperature gradients, cannot be avoided. Initially, stresses in the bond interface result from a combination of peripheral loads and internal constraint forces. It must be borne in mind that stresses due to shrinkage and temperature gradients in the new concrete typically reach their maximum at the perimeter (peeling forces). The combination of peripheral and internal stresses often exceeds the capacity of the initial bond, thus requiring the designer to allow for a de-bonded interface. This is particularly true in the case of bridge overlays, which are subject to fatigue stresses resulting from traffic loads.

Furthermore, these stresses vary with time, and bond failure can take place years after installing the overlay. When this happens, the tensile forces set up must be taken up by connectors positioned across the interface. Typical examples are shown schematically in the following table





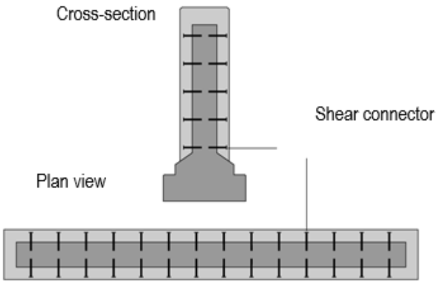
Strengthening of bridges by means of concrete overlays	Strengthening of buildings, decks and slabs

Fig. 1: Examples of overlay applications

1.2 Advantages of the strengthening with concrete overlay

- Simple and reliable application to a variety of cases
- Monolithic structural component behaviour assured
- Shear forces are reliably transferred even if the interface is cracked
- Wide range of applications
- Suitable for use with the most common methods of surface roughening
- Reduced requirements for anchor embedment

1.3 Example of applications

Rehabilitation of a bridge deck	
	• Removal of damaged concrete layer using high-pressure water jetting • Anchoring of additional reinforcement using Hilti injection mortar • Installation of shear connectors using Hilti injection mortar • Placement of new concrete overlay ✓ Monolithic load-bearing behaviour ✓ Reliable shear transfer ✓ Stiff connection ✓ Reduced connector embedment
Strengthening the floor of an industrial building	
	• Removal of covering and any loose overlay • Roughening of surface by shot-blasting • Installation of connectors using Hilti injection mortar according to engineer's instructions • Inspections, if necessary, of concrete surface for roughness and pull-away strength, and of connectors for pull-out strength. • Placement of reinforcement and overlay concrete ✓ Monolithic load-bearing behaviour ✓ Reliable and verifiable shear transfer ✓ Adequate connection stiffness ✓ Small anchorage depth
Strengthening an industrial building foundation	
	• exposure of foundation • Installation of connectors using Hilti injection mortar as per design specifications (smooth surface) • Placement of reinforcement and overlay concrete ✓ Reduced labour ✓ Monolithic load-bearing behaviour ✓ Reduced anchor embedment ✓ Reliable shear transfer ✓ Ductile connection
Repairing and strengthening a pier	
 	• Roughening of concrete surface • Installation of shear connectors using Hilti injection mortar as per design specifications • Placement of reinforcement and overlay concrete ✓ Monolithic load-bearing behaviour ✓ Reliable shear transfer ✓ Stiff connection ✓ Reduced anchor embedment

2 Products overview

The following table summarizes the considered products:

Anchor type		HCC-B + RE500 V3	HUS 3	HIT-V + RE500 V3	HCC-K + RE500 V3	REBAR + RE500 V3
Anchor size		14	8 - 10 - 14	10 - 12 - 16	10 - 16	8… - … 20
Base material	Cracked concrete	V	V	V	V	V
	Non-cracked concrete	V	V	V	V	V
Approvals	European Technical approval (ETA) for concrete overlay static	V	V	X	X	X
	European Technical approval (ETA) for concrete overlay fatigue	V	X	X	X	X
Design method	TR066	V	V	X	X	X
	Hilti method static	X	X	V*	V*	V
	Hilti method seismic	X	X	X	X	V
Setting	High productivity	V	V	X	X	X
	Flexible length	X	X	V	X	V
	Symmetric shape	V	V	V	V	X

* Based on TR066

HIT-RE 500 V3 injection mortar

In the following tables are schematically illustrated the products for shear connection provided by Hilti company and considered in this work.

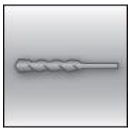
Injection mortar system	Benefits
 Foil pack: HIT-RE 500 V3 (available in 330, 500 and 1400 ml cartridges)	- **SafeSet** technology: Simplified method of borehole preparation using either Hilti hollow drill bit for hammer drilling or Roughening tool for diamond cored applications - Suitable for cracked/non-cracked concrete C 20/25 to C 50/60 - High loading capacity - Suitable for dry and water saturated concrete - Hilti Technical Data for under water application - Long working time at elevated temperatures - Cures down to -5°C - Odourless epoxy

Base material


Concrete (non-cracked)


Concrete (cracked)

Installation conditions

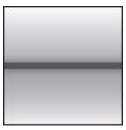

Hammer drilled holes


Diamond drilled holes

SAFESET

Hilti **SafeSet** technology

Load conditions


Static/ quasi-static


Seismic, ETA-C1, C2

Other information


European Technical Assessment


CE conformity


PROFIS Anchor design Software

Approvals / certificates

Description	Authority / Laboratory	No. / date of issue
European Technical Assessment	CSTB	ETA-16/0143 / 2017-07-12
Shockproof fastenings in civil defence installations	Federal Office for Civil Protection, Bern	BZS D 16-601/ 2016-08-31
Fire test report [b)]	MFPA Leipzig	GS 3.2/15-361-4 / 2016-08-04

HUS3 Screw anchor

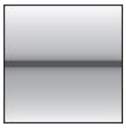
Anchor version	Benefits
 HUS3-H M8, M10, M14	- High productivity - less drilling and fewer operations than with conventional anchors - ETA approval for concrete overlays - Small edge and spacing distance - Three embedment depths for maximum design flexibility - Immediate placement of the reinforcement

Base material

Concrete (non-cracked)

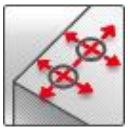
Concrete (cracked)

Load conditions



Static / quasi-static

Installation conditions



Small edge distance and spacing

Other information

European Technical Assessment

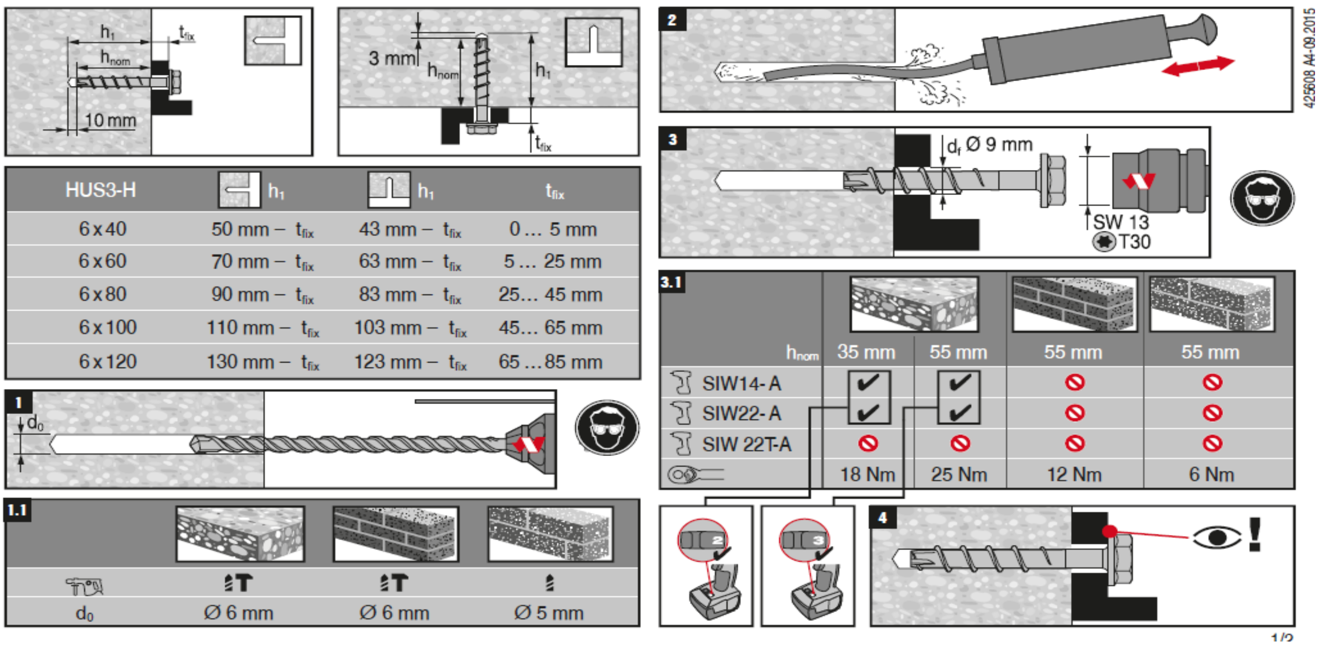
CE conformity

Design Method

EOTA
TR 066

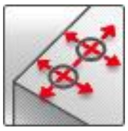
Approvals / certificates

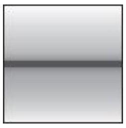
Description	Authority / Laboratory	No. / date of issue
European Technical Assessment	DIBt	ETA-xx/xxxx / 2018-xx-xx



HCC-B Shear connector

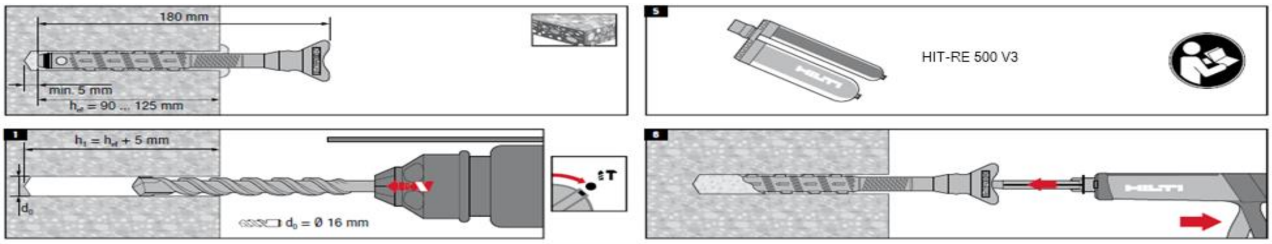
Anchor version	Benefits
 HCC-B Ø 14	- ETA approval for shear connection - Available data according to EOTA TR066 - High shear loads - Small edge and spacing distance - User selection of embedment depths for maximum design flexibility - Anchor head designed to allow the easy placement of the overlay reinforcements - Immediate placement of the reinforcement before the curing time of the resin - fatigue loads - high productivity

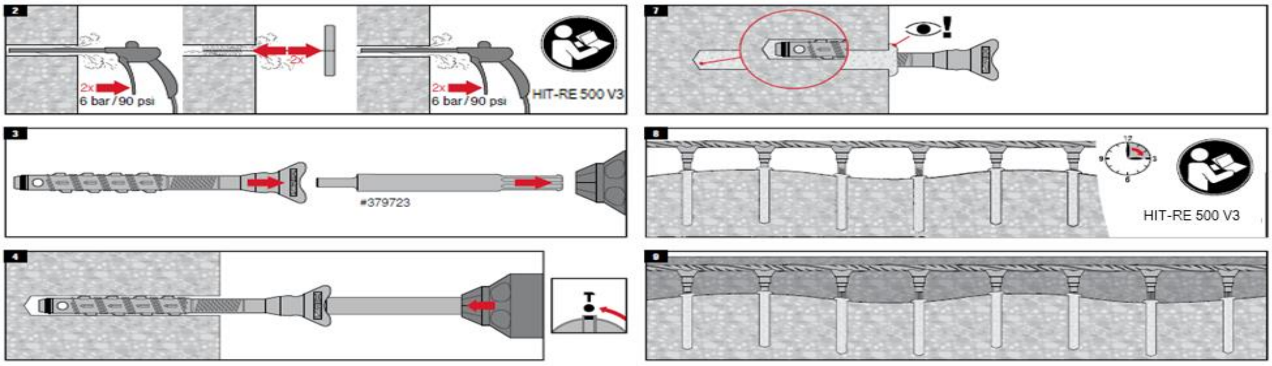
Base material		Installation conditions
		
Concrete (non-cracked)	Concrete (cracked)	Small edge distance and spacing

Load conditions		Other information		Design Method
				EOTA TR 066
Static/ quasi-static	fatigue	European Technical Assessment	CE conformity	

Approvals / certificates

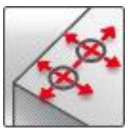
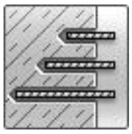
Description	Authority / Laboratory	No. / date of issue
European Technical Assessment	DIBt	ETA-xx/xxxx / 2018-xx-xx

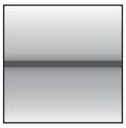




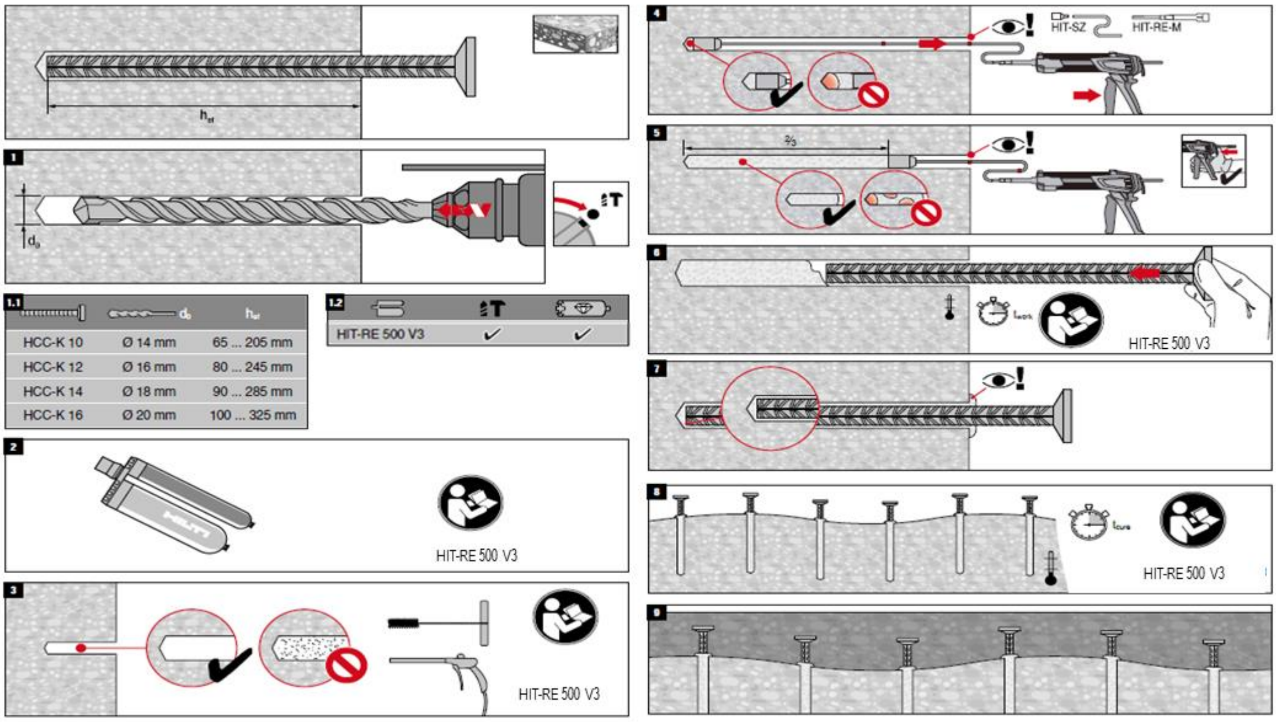
HCC-K Shear connector

Anchor version	Benefits
 HCC-K Ø 10, 12, 14, 16	- Small edge and spacing distance - User selection of embedment depths for maximum design flexibility

Base material		Installation conditions	
 Concrete (non-cracked)	 Concrete (cracked)	 Small edge distance and spacing	 Variable embedment depth

Load conditions	Design Method
 Static/ quasi-static	**Hilti Design Method***

*Design method : EOTA TR066 with parameters based on Hilti's internal testing



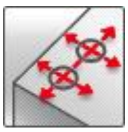
HCC-HIT-V

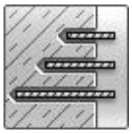
Anchor version	Benefits
 HCC-HIT-V M8, M10, M12, M16	- Small edge and spacing distance - User selection of embedment depths for maximum design flexibility - User selection of embedment depths for maximum design flexibility - flexible length - stainless steel available for highly aggressive environments - head position in new concrete can be adjusted after anchor installation

Base material / Installation conditions

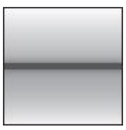

Concrete (non-cracked)


Concrete (cracked)


Small edge distance and spacing


Variable embedment depth

Load conditions / Design Method


Static/ quasi-static

Hilti Design Method*

*Design method : EOTA TR066 with parameters based on Hilti's internal testing

REBAR

Anchor version

REBAR
Ø 8, 10, 12, 14, 16, 20

Benefits

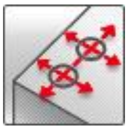
- High shear loads
- Small edge and spacing distance
- User selection of embedment depths for maximum design flexibility
- Seismic loads
- flexible length

Base material

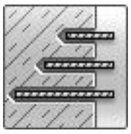
Concrete (non-cracked)

Concrete (cracked)

Installation conditions

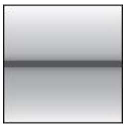


Small edge distance and spacing



Variable embedment depth

Load conditions



Static/ quasi-static

Seismic

Design Method

Hilti Design Method

3 Design of interface

3.1 Basic considerations

Structures made of reinforced concrete or prestressed concrete which have a concrete overlay at least 40 mm in thickness (prEN 1992-4), or at least 60 mm on bridge structures, may be designed as monolithic building components if the shear forces at the interface between the new and the existing concrete are restrained

3.2 Principle and set-up of the analytical model

Forces at the interface between the new and existing concrete are determined from the external forces acting on the building component. In designing the interface, it must normally be assumed that the interface is debonded. The shear connectors crossing the interface must be placed in such a way that shear forces (“shear flow”) at the interface are transmitted at design level.

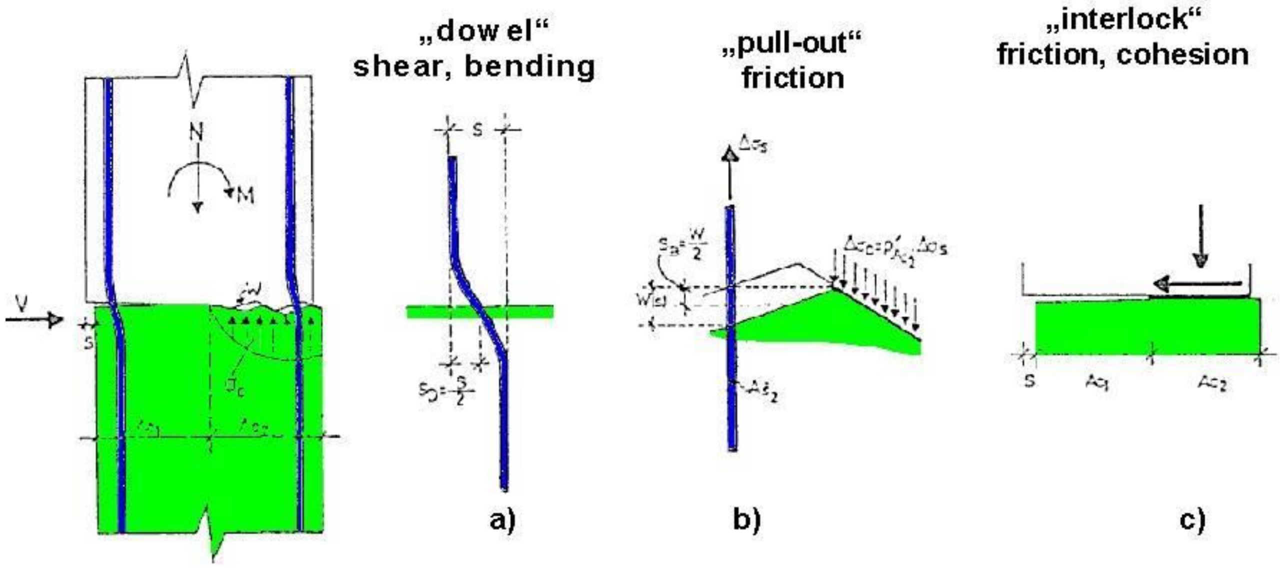


Fig. 8: Load contribution of the different components

Because of separation at the interface, the shear connectors are subject to a tensile force and simultaneously to a bending moment, both of which depend on the roughness of the interface surfaces. If the surfaces are roughened, additional interlocking effects and cohesion can take up part of the shear force at the interface.

3.3 Design shear resistance at the interface, v_{Rd}

The transmission of shear forces at the interface between the new and existing concrete is determined by the roughness and surface finish of the joint as well as of the transverse reinforcement perpendicular to the interface. In general, the following equation applies:

$$v_{Rd} \geq v_{Ed}$$

Where:

v_{Rd}	=	Design resistance of the allowable shear force per meter ("shear flow") in [kN/m] at the interface
v_{Ed}	=	Design value of the shear flow acting at the interface in [kN/m]

Two different models are available for the computation of the longitudinal shear resistance of the interface: the method proposed by the EOTA TR066 and the HILTI method, also indicated as Palieraki method.

3.4 Design shear strength at the interface, EOTA TR066

According to TR066 the following equation define the resistance of the interface:

$$v_{Rd} = \Big\{ \underbrace{c_r \cdot f_{ck}^{\frac{1}{3}}}_{\text{Interlock}} + \underbrace{\mu \cdot \sigma_n}_{\text{pull-out}} + \underbrace{\mu_e \cdot \kappa_{1e} \cdot \alpha_{k1} \cdot \rho \cdot \sigma_A + \kappa_{2e} \cdot \alpha_{k2} \cdot \rho \cdot \sqrt{\frac{f_{yk}}{\gamma_s} \cdot \frac{0{,}85 \cdot f_{ck}}{\gamma_c}}}_{\text{dowel}} \Big\} b_j \leq \underbrace{\left(\beta_c \cdot \nu_e \cdot \frac{0{,}85 \cdot f_{ck}}{\gamma_c}\right) b_j}_{\text{concrete strut}} \quad \text{(TR066,2.31,2.9)}$$

Where:

c_r	=	Coefficient for adhesive bond resistance in a reinforced interface
f_{ck}	=	minimum value of concrete compressive strength of the two concrete layers, measured on cylinders
f_{yk}	=	Characteristic yield strength of the shear connector
μ_e	=	Friction coefficient
σ_n	=	Lowest expected compressive stress resulting from an eventual normal force acting on the interface (compression has a positive sign)
κ_{1e}	=	Interaction coefficient for tensile force activated in the shear connector
κ_{2e}	=	Interaction coefficient for flexural resistance in the shear connector
α_{k1}	=	Modification factor for material properties of the connector
α_{k2}	=	Modification factor for geometry of the connector
ρ	=	Reinforcement ratio of the steel of the shear connector crossing the interface
σ_A	=	Steel stress associated to the relevant failure mode, (see section 3.14)
γ_c	=	Safety factor for concrete; 1,50 as given in EN 1992-4 for strengthening of existing structures
γ_s	=	Safety factor for steel; 1,15 as given in EN 1992-4 for supplementary reinforcement
b_i	=	Width of the interface of the composed section
ν_e	=	Coefficient for reduction of concrete strength
β_c	=	Coefficient for the strength of the compression strut

Coefficients and parameters for different surface roughness							
Surface characteristics of interface	c_a	c_r	κ_{1e}	κ_{2e}	β_c	μ_e $f_{ck} \geq 20$	μ_e $f_{ck} \geq 35$
Very rough, (including shear keys [1]) $R_t \geq 3{,}0$ mm	0,5	0,2	0,5	0,9	0,5	0,8	1,0
Rough, $R_t \geq 1{,}5$ mm	0,4	0,1	0,5	0,9	0,5	0,7	
Smooth (concrete surface without treatment after vibration or slightly roughened when cast against formwork)	0,2	0	0,5	1,1	0,4	0,6	
Very smooth (steel, plastic, timber formwork)	0,025	0	0	1,5	0,3	0,5	
[1] Shear keys should satisfy the geometrical requirements given in Fig.9							

$\nu_e =$ $\quad 0{,}55 \cdot \left(\frac{30}{f_{ck}}\right)^{1/3} < 0{,}55$

α_{k1} = given in the European Technical Assessment of the connector

α_{k2} = given in the European Technical Assessment of the connector

Product specific coefficients according to TR066		
Anchor	α_{k1}	α_{k2}
HCC-B	0.8	1.3
HUS3	0.8	1.0
HCC-K	N.A	N.A
HCC-HIT-V	N.A	N.A
REBAR	N.A	N.A

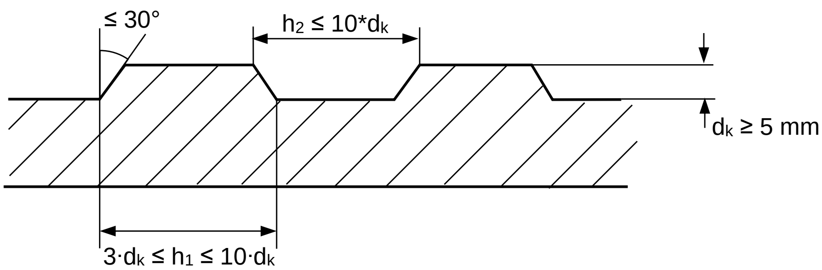


Fig. 9: Geometry of shear keys

3.5 Design shear strength at the interface, HILTI method

According to the Hilti method the governing equation for interface resistance is:

$$v_{Rd} = \{\mu_h\ (\sigma_n + \rho\ \min(\ \sigma_A\ , \kappa_{1h}\ \sigma_s\ ) + \kappa_{2h}\ \rho\ \sqrt{f_{yd}\ f_{cd}}\ \} b_j \leq \beta_c\ \nu_h\ f_{cd}\ b_j$$

Pull-out dowel concrete strut

Where:

f_{cd}	=	minimum value of concrete design compressive strength of the two concrete layers, measured on cylinders
f_{yd}	=	design yield strength of the shear connector
μ_h	=	Friction coefficient
σ_n	=	Lowest expected compressive stress resulting from an eventual normal force acting on the interface (compression has a positive sign)
κ_{1h}	=	contribution factor for the friction mechanism
κ_{2h}	=	contribution factor for the dowel mechanism
ρ	=	Reinforcement ratio of the steel of the shear connector crossing the interface
σ_A	=	Steel stress associated to the relevant failure mode (see section 3.14)
b_i	=	Width of the interface of the composed section
ν_h	=	Effectiveness factor for the concrete according to fib MC2010, Eq. (7.3-51)
β_c	=	Coefficient for the strength of the compression strut
σ_s	=	Effective steel stress in the connector

The following definitions are valid for the considered method:

$$\mu_h = 0.3\ \sqrt[3]{\left(\frac{f_{cd}}{\sigma_c + \sigma_n}\right)^2}$$

$$\sigma_c = \rho\ \sigma_s$$

$$\sigma_s = \frac{\sigma_A}{0.80}$$

$$\nu_h = 0.55\left(\frac{30}{f_{ck}}\right)^{\frac{1}{3}} < 0.55 \qquad \text{(Fib MC2010 7.3-51)}$$

In the following are reported the tables with the parameters needed for the application of the Hilti method

Parameter κ_{1h} for different interface characteristics: monotonic loading	
	$\boldsymbol{6d < l_{emb} < 20\,d}$
Mechanically roughened (1/4-in. amplitude), normal strength concrete	0.60
Mechanically roughened (1/4-in. amplitude), lightweight or high strength concrete	0.40
Smooth Interface	0.40
Very Smooth Interface, steel formed	0.20
Smooth Interface, lightweight concrete	0.20
Smooth Interface, no cohesion	0.10
Rough Interface, external compressive stress	0.70
Smooth Interface, external compressive stress	0.50
Interface with Shear Keys	0.80

Parameter κ_{1h} for different interface characteristics: cyclic loading, maximum resistance		
	$\boldsymbol{l_{emb} > 20d}$	$\boldsymbol{10d < l_{emb} < 20d}$
Mechanically roughened (1/4-in. amplitude), normal strength concrete	0.60	$0.02\frac{l_{emb}}{d} + 0.2$
Mechanically roughened (1/4-in. amplitude), lightweight or high strength concrete	0.40	$0.02\frac{l_{emb}}{d}$
Smooth Interface	0.20	
Very Smooth Interface, steel formed	N/A	
Smooth Interface, lightweight concrete	N/A	
Smooth Interface, no cohesion	N/A	
Rough Interface, external compressive stress	0.70	
Smooth Interface, external compressive stress	0.50	
Interface with Shear Keys	0.40	N/A

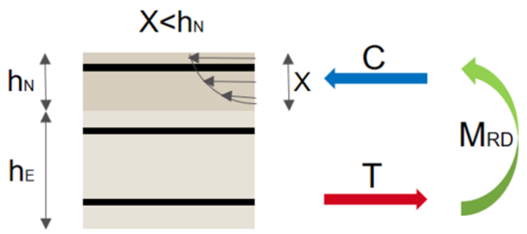
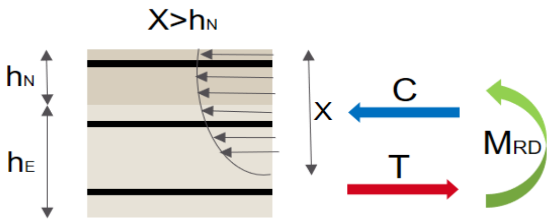
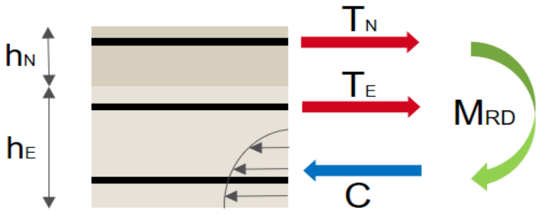
Parameter κ_{1h} for different interface characteristics: cyclic loading, resistance after three cycles	
	$\boldsymbol{10d \leq l_{emb} \leq 12d}$
Cyclic Loading: Resistance of Mechanically roughened interface, without external compressive stress, after three loading cycles: Applied displacement smax≥1.00mm	0.10
Cyclic Loading: Resistance of Mechanically roughened interface, with external compressive stress, after three loading cycles: Applied displacement 0.20mm<smax<1.00mm	0.20
Cyclic Loading: Resistance of Mechanically roughened interface, with external compressive stress, after three loading cycles: Applied displacement smax<0.20mm, or smax>1.00mm	0.10

Parameter κ_{2h} for different normalized embedment depth	
$\frac{h_{ef}}{d} > 8$	0.70
$6 \leq \frac{h_{ef}}{d} \leq 8$	$0.1\frac{l_{emb}}{d} - 0.1$
$\frac{h_{ef}}{d} = 6$	0.5
Residual interface resistance	$0.5\,\kappa_2$

3.6 Design shear force acting longitudinally at interface, v_{Ed}

Normally, the design shear force v_{ed} is calculated from the bending resistance of the cross-section (shear failure of the member should not be the governing factor). The design shear force v_{ed} can also be calculated from the change of the compression and/or tension (shear load v_{ed}) force in the concrete overlay.

In this case, the definition of the acting longitudinal shear is made starting from the value of shear defined with the structural analysis. It is possible to account for both for the case of positive and negative bending moments. The formulas used are the following:

<table>
<tr><th colspan="2">Computation of the longitudinal shear on the interface</th></tr>
<tr><td colspan="2">Positive bending moment, compressed part of the slab all contained in the overlay depth</td></tr>
<tr><td>
</td><td>$$v_{ED,i} = \frac{V}{z}$$ $$z = 0{,}9\ d$$</td></tr>
<tr><td colspan="2">Positive bending moment, compressed part of the slab not fully contained in the overlay depth</td></tr>
<tr><td>
</td><td>$$v_{ED,i} = \frac{V}{z}\frac{h_n}{x}$$ $$z = 0{,}9\ d$$</td></tr>
<tr><td colspan="2">Negative bending moment:</td></tr>
<tr><td>
</td><td>$$v_{ED,i} = \frac{V}{z'}\frac{A_{s,n}}{A_{s,n} + A_{s,ex}}$$ $$z' = 0{,}9\ d'$$</td></tr>
</table>

Where:

$v_{ED,i}$	=	longitudinal shear
V	=	shear acting on the considered section
x	=	depth of the compressed part of the composite slab
$A_{s,n}$	=	area of the reinforcements in the new slab
$A_{s,ex}$	=	area of the reinforcements on top of the existing slab
z	=	internal lever harm of the composite slab
d	=	effective depth of the slab, positive bending moment
d'	=	effective depth of the slab, negative bending moment

3.7 Shear force to be transferred at overlay perimeter

At the perimeter of a new concrete overlay, the maximum tensile force F_{cr} must be taken into account in the design. In fact, here the most severe action on the interface can be related to the effect of shrinkage. Considering the cracking force for the design of the interface it is conservatively defined the maximum possible action on the connection. Particular attention must be paid to constraining the moment arising from F_{cr}:

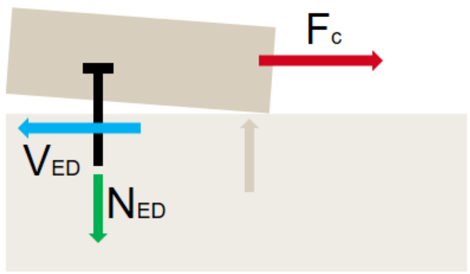


Fig. 10: Equilibrium condition at the edges of the slab

3.7.1 Verification according to EOTA TR066 indications

$$F_{cr} = V_{Ed}^{*} = h_N \cdot b_j \cdot f_{ctd} \qquad \text{(TR066 2.3)}$$

$$N_{ed}^{*} = \frac{V_{ed}^{*}}{6} = \frac{h_N \cdot b_j \cdot f_{ctd}}{6} \qquad \text{(TR066 2.5)}$$

$$\begin{cases} l_e = 3 \cdot h_N & for\ very\ rough\ surfaces \\ l_e = 6 \cdot h_N & for\ rough\ surfaces \\ l_e = 9 \cdot h_N & for\ smooth\ surfaces \end{cases} \qquad \text{(TR066 2.1)}$$

Where:

l_e	=	Width of the lateral strip considered as subjected to the highest shrinkage forces
h_N	=	Thickness of the overlay
f_{ctd}	=	Design tension resistance of concrete

3.7.2 Verification according to HILTI method indications

$$F_{cr} = V_{Ed}^{*} = h_N \cdot b_j \cdot k * f_{ct,eff}$$

$$N_{ed}^{*} = \frac{V_{ed}^{*}}{6}$$

$$\begin{cases} l_e = 3\, h_N & for\ rough\ surfaces \\ l_e = 6\, h_N & for\ sand\ blasted\ and\ smooth\ surfaces \end{cases}$$

Where:

l_e	=	Width of the lateral strip considered as subjected to the highest shrinkage forces
h_N	=	Thickness of the overlay
k	=	Coefficient to allow for non-uniform self-equilibrating stresses = 0.8 for h_N=30 cm
$f_{ct,eff}$	=	Tensile strength of overlay effective at the time when the cracks may first be expected to occur as per [1], Section 7.3.2 (for general cases: $f_{ct,eff}$ = 3 N/mm^2)

3.8 Central regions without shear connectors

Where the shear stresses are low, shear connectors need not be used in the central region (area) of the overlay if the load is predominantly static and if connectors are positioned around the perimeter. The resistance of the interface without shear connectors can be computed with two different methods: the one proposed by EOTA TR066 or the Hilti method.

3.8.1 Verification of an interface without shear connectors according to EOTA TR066

$$\tau_{Rd,ct} = c_a \cdot f_{ctd} + \mu_e \cdot \sigma_n \leq 0{,}5 \cdot \nu_e \cdot f_{cd} \qquad \text{(TR066 2.9)}$$

The coefficients and the parameters present in the equation are defined in section 3.4

3.8.2 Verification of an interface without shear connectors according to Hilti method

$$\tau_{Rd,ct} = \mu_h \sigma_n \; \leq \beta_c \, \nu_h \, f_{cd}$$

The coefficients and the parameters present in the equation are defined in section 3.5

3.9 Redundant load transfer

The design applies to redundant load transfer; which means that a local load must be transferred by at least 3 shear connectors.

3.10 Fatigue loading

3.10.1 General observations

(1) Bond interfaces subject to substantial changes in stress, i.e. not to predominantly static forces, must be designed to withstand fatigue.
(2) Bonds subject to fatigue must always be roughened.
(3) The effect of the superposition of fatigue loading and static loading is not addressed in this work

3.10.2 Proof for fatigue

Fatigue loadings are accounted differently according to the method considered:

- According to Hilti method the effect of fatigue loading is taken into account by means of the use of the relevant parameters (Section 3.5) in the formula for the definition of longitudinal shear resistance

- According to EOTA TR066 fatigue is taken into account by means of a reduction coefficient η_{sc}

$$\Delta\tau_{Ed} \leq \eta_{sc} \cdot \tau_{Rd} \qquad \text{(TR066 2.13)}$$

Without the effect of static loadings:

$$\Delta\tau_{Ed} = \tau_{Ed,\max} \qquad \text{(TR066 2.14)}$$

$\eta_{sc} = 0{,}4$ Or otherwise given in the European Technical Assessment of the shear connector for interfaces with use of shear connectors

Where:

$\Delta\tau_{Ed}$	=	Shear stress acting as fatigue relevant loading
η_{sc}	=	Factor for fatigue loading
$\tau_{Ed,max}$	=	Upper shear stress acting as fatigue relevant loading
τ_{Rd}	=	Resisting shear stress

3.11 Seismic loading

Seismic loadings are verified only according to the Hilti method. In this case, the verification of the interface is made using the parameters related to the case of “cycling loading maximum resistance” (see section 3.5). The anchor verification is made with reference to the C1 category bond strength as defined in the ETA of the product. For the failure mechanism considered in the proofs (See section 3.14) the following reduction coefficients are considered:

Reduction coefficient of the design resistances of the anchors	
Steel failure	1,00
Combined concrete cone/pull-out failure	0,85
Pull-out failure	1,00
Concrete cone failure	0,75
Concrete cone failure for cast-in headed anchors	0,85
Splitting failure	0,85

3.12 Serviceability limit state

As an approximation in normal cases, the additional deformation of a strengthened bending element may be determined using the monolithic cross-section, and then increased as follows:

$$W_{eff} = \gamma\, W_{calc}$$

W_{eff}	=	additional deformation calculated for the reinforced section considering the elasticity of the shear connectors
W_{calc}	=	additional deformation calculated for the reinforced section assuming a perfect bond
γ	=	factor reported in the following table
s_d	=	displacement of connectors under the mean permanently acting load $F_p \approx 0.5\, F_{uk}$

The displacements reported in the table can be used for more precise calculations:

Surface treatment	Mean roughness R_t [mm]	γ	s_d [mm]
High-pressure water jets / scoring	> 3.0	1.0	$\approx 0{,}005\ \emptyset$
Sand-blasting / chipping hammer	> 0.5	1.1	$\approx 0{,}015\ \emptyset$

∅ = diameter of shear connectors

3.13 Additional rules and design details

3.13.1 Mixed surface treatment

The surface treatments used for a building component may differ only if the non-uniform stiffness arising in the bond is taken into account (also see **Error! Reference source not found.**, displacement s_d). Note that a non-cracked interface, i.e., rigid bond, is assumed for interfaces with low shear stress that do not require connectors in the central region

3.13.2 Minimum amount of reinforcement at the interface

If shear connectors cannot be omitted, the following minimum reinforcement ratio must be provided in the interface:

$$\rho_{\min.} = 0.12\,\frac{f_{ctm}}{f_{yk}} \geq 0.0005 \qquad \text{(FIB MC2010)}$$

3.13.3 Layout of connectors

(1) The connectors must be positioned in the load-bearing direction of the building component with respect to the distribution of the applied shear force in such a way that the shear force at the interface can be constrained, and de-bonding of the new concrete overlay prevented.
(2) If the new concrete overlay is on the tension side of the load-bearing component, the connectors must be distributed to accord with the grid spacing of the longitudinal reinforcement without any allowance being made for anchorage length
(3) The connector spacing in the load-bearing direction may not be larger than 6 times the thickness of the new concrete overlay, or 800 mm.

3.13.4 Recommendation for overlay placement

Pre-treatment:
A primer consisting of thick cement mortar is recommended.
Before the cement mortar primer is applied, the old concrete should be adequately wetted 24 hours in advance, and thereafter at suitable intervals. Before applying the primer, the concrete surface should be allowed to dry to such an extent that it has only a dull moist appearance.
The mortar used as a primer should consist of water and equal parts by weight of Portland cement and sand of 0/2 mm particle size. This is applied to the prepared concrete surface and brushed in.

Overlay:
The concrete mix for the overlay should normally be such as to ensure low-shrinkage (W/C $\leq$ 0.40). The overlay must be placed on the still fresh primer, i.e. wet on wet.

Curing:
Careful follow-up is necessary to ensure an overlay of adequate durability. Immediately after placement, the concrete overlay must be protected for a sufficiently long period (at least five days) against drying out and excessive cooling.

3.13.5 Recommendation for surface treatment

The roughness of the interface has a decisive influence on the shear force that can be transferred. For design purposes, the characteristic dimension is the mean depth of roughness, R_t, measured according to the sand-patch method [9]. It must be borne in mind that R_t is a mean value, and thus the difference between the peaks and valleys is about 2 R_t.
It is recommended that a mean roughness, R_t, be stipulated when specifying the surface treatment. Prior to approving the treatment, a sample surface must be made up and this checked using the sand-patch method.

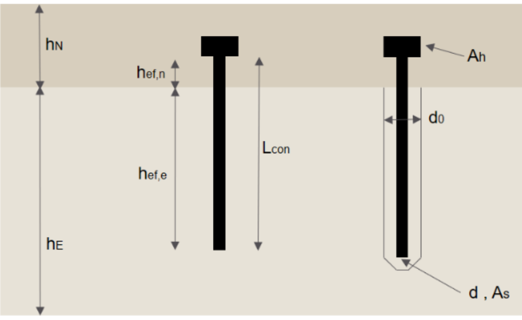
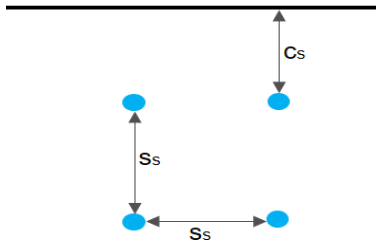
3.14 Anchorage of the shear connectors in the existing and new concrete

3.14.1 General observations

The anchorage of the shear connectors must be verified both in the existing slab and in the overlay according to the prescriptions of prEN 1992-4. The shear connectors are considered alternatively as post installed anchors in the existing concrete slab and as cast-in anchors in the overlay. The minimum resistance between the two is considered for the definition of the ultimate load carrying capacity of the connectors. From the ultimate load carrying capacity, considering the effective cross section of the connectors, it is defined the stress σ_A used for interface verifications

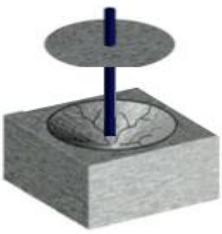
3.14.2 Installation geometry

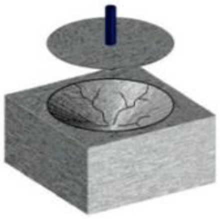
To guarantee the efficiency of the connection, the shear connectors are to be anchored sufficiently in the existing concrete and in the overlay. In the following, it is reported the description of the geometrical parameters involved in the proofs needed for the verification of the anchors.

Setting details for shear connectors			
	L_{con}	Length of shear connector	[mm]
	$h_{ef,e}$	Embedment depth in the existing slab	[mm]
	$h_{ef,n}$	Embedment depth in the overlay	[mm]
	h_E	Thickness of the existing slab	[mm]
	h_N	Thickness of the overlay	[mm]
	d_0	Drill bit diameter	[mm]
	d	Anchor stud diameter	[mm]
	A_s	Effective steel area of the anchor	[mm]
	A_h	Area of the anchor head	[mm]
	c	Anchor edge distance	[mm]
	s	Anchors spacing	[mm]

3.14.3 Design proofs for the connector

The design proofs considered for the connectors are the ones defined by the prEN 1992-4.
Due to the characteristic of the design problem, it was chosen to neglect the possibility of concrete blowout failure. This decision is justified by the geometry of the problem.
The following table summarizes the design proofs that are carried out.

Design proofs considered for connectors verification:	
	**Steel failure resistance** Resistance depending on the connector properties, independent from the embedment in the concrete element. Notation: $N_{Rd,s}$
	**Combined concrete cone failure/pull-out resistance** Resistance depending on the connector properties and on concrete element properties, verified for the anchorage in the existing slab Notation: $N_{Rd,cp}$

	**Splitting failure** Resistance depending on the connector properties and on concrete element properties, verified independently for both overlay and existing slab Notation: $N_{Rd,sp}$
	**Concrete cone failure** Resistance depending on the connector properties and on concrete element properties, verified independently for both overlay and existing slab Notation: $N_{Rd,c}$
	**Pull-out failure** Resistance depending on the connector properties and on concrete element properties, verified for the embedment in the overlay Notation: $N_{Rd,p}$

3.14.4 Geometrical boundary conditions for existing concrete and concrete overlays

The connectors layout is organized in two different regions, the central part of the slab and the lateral one. In the central part it is considered that the connectors are installed following a regular square grid. In the lateral region it is assumed that the connectors are organized in rows. The width of the lateral region is assumed as equal to l_e (see section 3.7).
In the following it is reported a general scheme of the connectors layout.

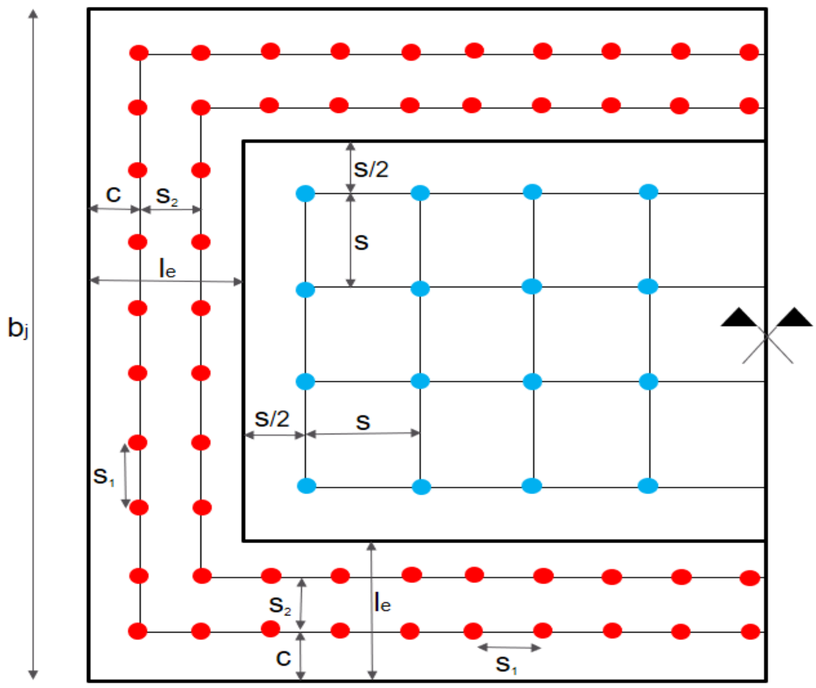


Fig. 11: Connector layout

According to the described geometry

$$s_2 = \frac{2(l_e - c)}{2r - 1}$$

r	=	Number of the lateral rows of anchors
c	=	Distance of the first row of anchors from the free edge

4 Examples

4.1 Description of the design case, structural analysis

In the following, it is presented an example about the design of the connection between an existing concrete slab and an overlay. We consider the case of a continuous beam with two equal spans.

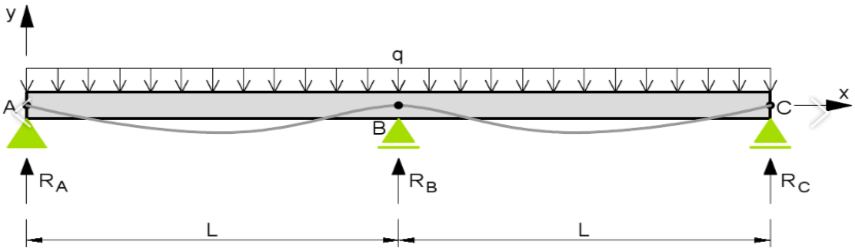


Fig. 12: Static scheme of the beam for the design example

It is assumed that the value of the shear at the central and lateral supports are already available from structural analysis:

$$V_A = 79{,}9\ kN$$

$$V_B = 133{,}1\ kN$$

The geometry of the composite section made with the concrete overlay is the following:

Geometrical details:			
Thickness of new slab (overlay)	h_N =	[mm]	100
Thickness of existing slab	h_E =	[mm]	200
Net cover on top of existing slab	Cs =	[mm]	45
Effective depth of existing slab	d =	[mm]	155
Width of the slab	bj =	[mm]	5000

It is assumed that the reinforcements in the existing slab are known while the presence of reinforcements in the overlay is neglected. The reinforcements that are present at the bottom of the existing slab are Φ16 with spacing 250 mm while the top reinforcements are made with Φ20 with the same spacing.

The concrete class is C20/25 for the existing slab and C25/30 for the overlay. The reinforcements are made with steel B450C

From the analysis of the section at ultimate limit state, considering the stress block approximation for the description of the concrete in compression and the elastic perfectly plastic constitutive model for steel, it is possible to evaluate the ultimate bending moment resistance and the position of the neutral axis in the section.

The steel areas on top and bottom of the existing slab are respectively:

$$A_{se,-} = 7853{,}98\ mm^2$$

$$A_{se,+} = 5026{,}55\ mm^2$$

Under the hypothesis of planarity of the sections and perfect bond concrete-steel, imposing the translational equilibrium, it is possible to find the depth of the neutral axis X and the strains in the reinforcements. Considering the case of positive bending moment resistance, we get:

Neutral axis position and steel bars strains:

Positive bending moment				
Depth of the neutral axis	X =	[mm]	75.5	
Top steel strains	εs_{Ase-} =	[%]	0.32	Steel yielded in tension
Bottom steel strains	εs_{Ase+} =	[%]	0.83	Steel yielded in tension

These values have been obtained under the conservative hypothesis (for the interface shear verification) of having the same class of concrete both for the overlay and for the existing slab. The class of concrete considered was the higher one in order to maximize the longitudinal shear.

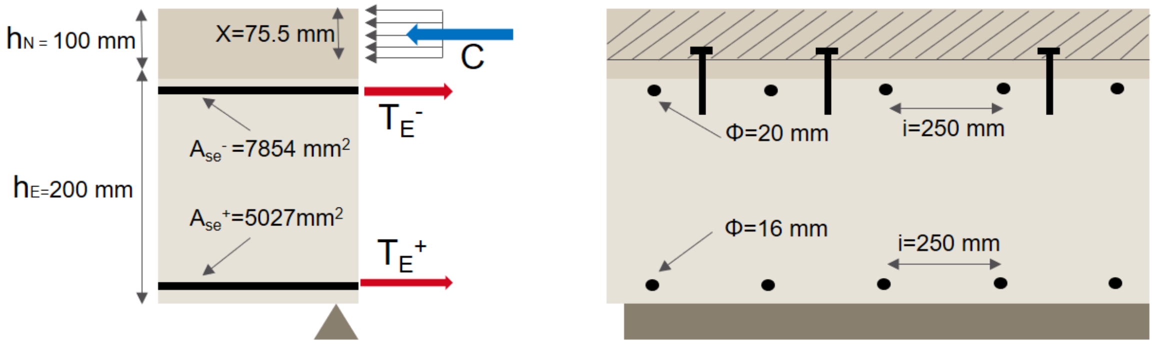


Fig. 13: Details of the beam for the design example

4.2 Data for design of the connection at external supports

After the preliminary definition of the quantities of interest, it is now possible to try to get the best solution for the design problem related to the longitudinal shear connection of new and existing slab. First, it is necessary to define the situation that we want to analyse. Since we consider that the compressed part of the composite slab is in the upper portion of the structure, we chose the case of positive bending moment.

Then it is necessary to define the depth of the compressed part of the slab, as defined in the previous computations X=75.5 mm. It was chosen to neglect the beneficial compression on the interface due to the external load so $\sigma_N = 0$. As already stated the shear is $V = 79{,}9\ kN$

The geometrical data describing the problem are:

Geometrical details:			
Thickness of new slab (overlay)	h_N =	[mm]	100
Thickness of existing slab	h_E =	[mm]	200
Slab width	b_j =	[mm]	5000
Reinforcements diameter on top of the existing slab	Φ_{ex} =	[mm]	20
Reinforcements spacing on top of the existing slab	$s_{s,ex}$ =	[mm]	200
Embedment depth in the new slab	$h_{ef,n}$ =	[mm]	55

Furthermore, it is assumed not to have sufficient reinforcements in order to limit the crack width to 0.3 mm, both in existing and new slab.

It is also necessary to define the state of concrete (cracked or un-cracked), the concrete class for both new and existing slab and the properties of the interface. In this example:

Concrete element properties:	
Concrete state	cracked
Existing slab concrete class	C20/25
New slab concrete class	C25/30
Surface treatment	Very smooth

Than it is needed to define the parameters of our design: first of all the choice of the connector and of the resin and then the geometrical parameters describing the positioning of the connectors:

type of connector	HCC-B 14-180
type of resin	HIT-RE 500-V3

Anchor spacing	$s =$	[mm]	300
Spacing between anchors close to the edge	$s_1 =$	[mm]	100
Anchors edge distance	$c =$	[mm]	100
Number of rows of anchors close to the edge	$r =$	[-]	3

Finally it is required the selection of the method to be used for carrying out the verification of the interface. This method must be available for the chosen type of connectors and for the considered loading conditions. In this example, it was chosen to use the FIB model.

From the choice of connector and of the resin, it is possible to find all the needed data from the documentation of the product. These data are reported in the table below:

Data from the relevant documentation:			
Hole diameter	$d_0 =$	[mm]	16
Anchor diameter	$d =$	[mm]	14
Anchor head diameter	$d_h =$	[mm]	42
Anchorage cross section area	$A_s =$	[mm^2]	83,00
Safety coefficient for concrete cone	$\gamma_{Mc} =$	[-]	1.5
Safety coefficient for splitting	$\gamma_{Msp} =$	[-]	1.8
Safety coefficient for pull-out	$\gamma_{Mp} =$	[-]	1.5
Safety coefficient for steel failure	$\gamma_{MS} =$	[-]	1.2
Steel resistance of the anchorage	$f_{uk} =$	[MPa]	400,00
Design resistance of the anchor steel	$f_{yd} =$	[MPa]	333,33
Minimum existing slab thickness for splitting	$h_{min\ E} =$	[mm]	157
Minimum new slab thickness for splitting	$h_{min\ N} =$	[mm]	87
Critical spacing for splitting in new slab	$s_{cr,spN} =$	[mm]	70
Critical edge distance for splitting in new slab	$c_{cr,spN} =$	[mm]	70
Critical spacing for splitting in existing slab	$s_{cr,spE} =$	[mm]	70
Critical edge distance for splitting in existing slab	$c_{cr,spE} =$	[mm]	70
Bond strength in existing cracked concrete	$\tau_e =$	[MPa]	8.50
Bond strength in new cracked concrete	$\tau_n =$	[MPa]	8.67
Bond strength in uncracked concrete C20/25	$\tau_{Rk,ucr} =$	[MPa]	14.00

4.3 Determination of longitudinal shear

In this case, since $h_N > X$ the compressed part of the slab is totally contained into the new slab. The longitudinal shear is computed as:

$$v_{ED,i} = \frac{V}{z} = \frac{79{,}9\ kN}{0{,}2295\ m} = \quad 348{,}15\ kN/m \quad (B2.5\ 5.1.2)$$

With:

$$z = 0{,}9\ d = \quad 229{,}50\ mm$$

4.4 Verification of connectors: central part, existing slab

The load carrying capacity of the connector is defined taking into account separately the geometry of the installation in the central part of the slab and in the external part close to the edge.

First, the central part of the slab is considered, taking into account that the tensile resistance of the connector is the minimum one between the values obtained considering the anchorage in the existing and in the new slab.

In the existing slab the connector is a post installed bonded anchor, the computation of the load carrying capacity requires to consider different failure modes:

4.4.1 Steel failure

The steel failure resistance of the connector is taken directly from the values given in the relevant documentation

$$N_{Rk,s} = \quad 33{,}20\ kN$$

From this, the design values is obtained as:

$$N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}} = \frac{33{,}20\ kN}{1{,}2} = \quad 27{,}67\ kN$$ (EN1992-4 Table 7.1)

4.4.2 Combined concrete cone/pull-out verification

$$N^0_{Rk,p} = \pi\ d\ h_{ef,e}\ \tau_{Rk} = \pi\ 14mm * 125mm * 8{,}5MPa = \quad 46{,}73\ \ kN$$ (EN1992-4 7.3.1)

$$s_{cr,Np} = 7.3\ d\ \sqrt{\tau_{Rk,ucr}} = 7.3 * 14mm\ \sqrt{14\ MPa} = \quad 382.39\ \ mm$$ (EN1992-4 7.5)

$$s_{cr,Np} \leq 3\ h_{ef} = \quad 375\ \ mm$$ (EN1992-4 7.5)

$$s_{cr,Np} = \quad 375\ \ mm$$

$$c_{cr,Np} = \frac{s_{cr,Np}}{2} = \frac{375\ mm}{2} = \quad 187.5\ \ mm$$ (EN1992-4 5.6)

$$A^0_{p,N} = (s_{cr,Np})(s_{cr,Np}) = (375\ mm)^2 = \quad 140625\ \ mm^2$$ (EN1992-4 7.3.2)

$$A_{p,N} = \left(\min(s, s_{cr,Np})\right)^2 = (\min(300\ mm; 375\ mm))^2 = \quad 90000\ \ mm^2$$

$$\psi_{s,Np} = 0.7 + 0.3\frac{c}{c_{cr,Np}} \leq 1 = \quad since\ c = \infty \quad 1$$ (EN1992-4 7.10)

$\psi_{ec,Np} = \frac{1}{1+2\frac{e_n}{s_{cr,Np}}} = \quad since\ e_n = 0$ 1 (EN1992-4 7.12)

Assuming conservatively n=8 for the maximum number of anchors:

$$\psi^0_{g,Np} = \sqrt{n} - (\sqrt{n} - 1)\left(\frac{d\ \pi\ \tau_{Rk}}{k_8\sqrt{h_{ef}\ f_{ck}}}\right)^{1.5} \geq 1 =$$

$$= \sqrt{8} - (\sqrt{8} - 1)\left(\frac{\pi * 14\ mm * 8.5\ MPa}{7.7\sqrt{125\ mm * 20 MPa}}\right)^{1.5} \geq 1 = \qquad 1.079 \qquad \text{(EN1992-4 7.8-7.9)}$$

$$\psi_{g,Np} = \psi^0_{g,Np} - \left(\frac{S}{S_{cr,Np}}\right)^{0.5}\left(\psi^0_{g,Np} - 1\right) \geq 1 =$$

$$= 1.079 - \left(\frac{300mm}{375mm}\right)^{0.5}(1.079 - 1) \geq 1 = \qquad 1.008 \qquad \text{(EN1992-4 7.7)}$$

$$\psi_{re,Np} = 0.5 + \frac{h_{ef}}{200} \leq 1 = \qquad 1 \qquad \text{(EN1992-4 7.11)}$$

It is then possible to define the characteristic and the design resistances:

$$N_{Rk,p} = N^0_{Rk,p}\ \frac{A_{p,N}}{A^0_{p,N}}\ \psi_{s,Np}\ \ \psi_{g,Np}\ \ \psi_{ec,Np}\ \ \psi_{re,Np} =$$

$$= 46{,}73\ kN * \frac{90000\ mm^2}{140625\ mm^2} * 1 * 1 * 1 * 1 = \qquad 30.16\ kN \qquad \text{(EN1992-4 7.3)}$$

$$N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}} = \frac{30.16\ kN}{1.5} = \qquad 20.10\ kN \qquad \text{(EN1992-4 Table 7.1)}$$

4.4.3 Concrete cone breakout verification

$$N^0_{Rk,c} = k_g\sqrt{f_{ck}}\ h^{1.5}_{ef} = 7.7\sqrt{20\ MPa}\ (125\ mm)^{1.5} = \qquad 48{,}13\ kN \qquad \text{(EN1992-4 7.14)}$$

$$s_{cr,N} = 3\ h_{ef} = 3 * 125\ mm = \qquad 375\ mm \qquad \text{(EN1992-4 7.14)}$$

$$c_{cr,N} = \frac{s_{cr,N}}{2} = \frac{375\ mm}{2} = \qquad 187{,}5\ mm$$

$$A_{c,N} = \left(\min(s, s_{cr,N})\right)^2 = \left(\min(300\ mm; 375\ mm)\right)^2 = \qquad 90000\ mm^2$$

$$A^0_{c,N} = (s_{cr,N})(s_{cr,N}) = (375\ mm)^2 = \qquad 140625\ mm^2 \qquad \text{(EN1992-4 7.15)}$$

$$\psi_{s,N} = 0.7 + 0.3\frac{c}{c_{cr,N}} \leq 1 = \qquad 1 \qquad \text{(EN1992-4 7.16)}$$

$$\psi_{ec,N} = \frac{1}{1+2\frac{e_n}{s_{cr,N}}} = \qquad 1 \qquad \text{(EN1992-4 7.17)}$$

$$\psi_{re,N} = 0.5 + \frac{h_{ef}}{200} \leq 1 = \qquad 1 \qquad \text{(EN1992-4 7.11)}$$

$$N_{Rk,c} = N^0_{Rk,c}\ \frac{A_{c,N}}{A^0_{c,N}}\ \psi_{s,N}\ \ \psi_{ec,N}\ \ \psi_{re,N} =$$

$$= 48{,}13\ kN * \frac{90000\ mm^2}{140625\ mm^2} * 1 * 1 * 1 * 1 = \qquad 30.80\ kN \qquad \text{(EN1992-4 7.13)}$$

$$N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}} = \frac{30.80\ kN}{1{,}5} = \qquad 20.53\ kN \qquad \text{(EN1992-4 Table 7.1)}$$

4.4.4 Splitting verification

The concrete is assumed to be cracked, so the splitting failure verification is not needed

4.4.5 Summary

The lowest resistance of the anchorage in the existing slab is associated to the steel failure $N_{Rd,s} = 27{,}67\ kN$

4.5 Verification of connectors: central part, new slab

It is now necessary to evaluate also the anchorage in the new slab. In this case, the connector can be seen as a cast-in headed stud.

4.5.1 Pull-out verification

$A_h = \frac{\pi}{4}\,(d_h^2 - d^2) = \frac{\pi}{4}\,((42\ mm)^2 - (14\ mm)^2) =$	$1231.54\ mm^2$	(EN1992-4 7.2)
$N_{RK,p} = k_1\ A_h\ \ f_{ck} = 7.5 * 1231.54\ mm^2 * 25\ MPa =$	$230{,}91\ kN$	(EN1992-4 7.1)
$N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}} = \frac{230{,}91\ kN}{1.5} =$	$153{,}94\ kN$	(EN1992-4 Table 7.1)

4.5.2 Concrete cone breakout verification

$N_{Rk,c}^0 = k_g\ \sqrt{f_{ck}}\ h_{ef}^{1.5} = 7.7\ \sqrt{25\ MPa}\ (55\ mm)^{1.5} =$	$18{,}15\ kN$	(EN1992-4 7.14)
$s_{cr,N} = 3\ h_{ef} = 3 * 55\ mm =$	$165\ \ mm$	(EN1992-4 7.14)
$c_{cr,N} = \frac{s_{cr,N}}{2} = \frac{165\ mm}{2} =$	$82{,}5\ \ mm$	
$A_{c,N} = \left(\min\left(s, s_{cr,N}\right)\right)^2 = \left(\min(300\ mm; 165\ mm)\right)^2 =$	$27225\ mm^2$	
$A_{c,N}^0 = (s_{cr,N})(s_{cr,N}) = (165\ mm)^2 =$	$27225\ mm^2$	(EN1992-4 7.15)
$\psi_{s,N} = 0.7 + 0.3\frac{c}{c_{cr,N}} \leq 1\ =$	1	(EN1992-4 7.16)
$\psi_{ec,N} = \frac{1}{1+2\frac{e_N}{s_{cr,N}}} =$	1	(EN1992-4 7.17)
$\psi_{re,N} = 0.5 + \frac{h_{ef}}{200} \leq 1\ =$	1	(EN1992-4 7.11)
$N_{Rk,c} = N_{Rk,c}^0\ \ \frac{A_{c,N}}{A_{c,N}^0}\ \ \psi_{s,N}\ \ \psi_{ec,N}\ \ \psi_{re,N} =$		
$= 18{,}15\ kN * \frac{27225\ mm^2}{27225\ mm^2} * 1 * 1 * 1 * 1 =$	$18{,}15\ kN$	(EN1992-4 7.13)
$N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}} = \frac{18{,}15\ kN}{1{,}5} =$	$12{,}10\ kN$	(EN1992-4 Table 7.1)

4.5.3 Splitting verification

Since the concrete is assumed to be cracked, the splitting failure verification is not needed.

4.5.4 Summary

The lower resistance of the anchorage in the new slab is related to the concrete cone breakout, this value is also smaller than the minimum one coming from the verification of the connection with the existing slab. In conclusion, we can consider that the failure mode of the connector is associated to concrete cone breakout in the new slab and the tension design resistance is $N_{Rd,c} = 12{,}10\ kN$

4.6 Verification of connectors: lateral part, existing slab

A similar procedure is than followed for the verification of the connectors of the external rows, close to the edges of the concrete slab. Only the parameters that differs from the ones computed for the central part need to be recomputed. In this case the verification is carried out considering a strip of anchors perpendicular to the free edge of the slab. We start considering the existing slab:

4.6.1 Steel failure

Since the connectors are assumed to be the same both for lateral and central part of the plate the steel failure resistance is obviously the same

4.6.2 Combined concrete cone/pull-out verification

First, it is needed to define the spacing of the anchors. We compute the width of the lateral strip subjected to higher shrinkage stresses as:

$$l_e = \begin{cases} 3\ h_N & rough\ surfaces \\ 6\ h_N & smooth\ surfaces \end{cases}$$

For this example:

$$l_e = 6 * 100mm = \qquad 600mm \qquad \text{(B2.5, 3.3.1)}$$

$$S_2 = \frac{2(l_e - c)}{2r-1} = \frac{2(600\ mm - 100\ mm)}{2*3-1} = \qquad 200\ mm$$

Then it is possible to define the idealized failure area:

$$A_{p,N} = \left\{\min(c, c_{cr,Np}) + (r-1)\min(s_2, s_{cr,Np}) + \min\left(\frac{s_{cr,Np}}{2}, \frac{s_2}{2}\right)\right\} * \min(s_1, s_{cr,Np})$$

$$A_{p,N} = \left\{\min(100; 187{,}5) + (3-1)\min(200; 375) + \min\left(\frac{375}{2}, \frac{200}{2}\right)\right\} * \min(100; 375)$$

$$= \qquad 60000\ mm^2$$

$$\psi_{s,Np} = 0.7 + 0.3\frac{c}{c_{cr,Np}} \leq 1 \quad = 0.7 + 0.3\frac{100\ mm}{187{,}5\ mm} = \qquad 0.86 \qquad \text{(EN1992-4 7.10)}$$

$$\psi_{ec,Np} = \frac{1}{1+2\frac{e_n}{s_{cr,Np}}} = \ (since\ e_n = 0) \qquad 1 \qquad \text{(EN1992-4 7.12)}$$

We can now define the characteristic and the design resistances:

$$N_{Rk,p} = N_{Rk,p}^0\ \frac{A_{p,N}}{A_{p,N}^0}\ \psi_{s,Np}\ \ \psi_{g,Np}\ \ \psi_{ec,Np}\ \ \psi_{re,Np} =$$

$$= 46{,}73\ kN * \frac{60000\ mm^2}{140625\ mm^2} * 0{,}86 * 1 * 1 * 1 = \qquad 17{,}29\ kN \qquad \text{(EN1992-4 7.3)}$$

$$N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}} = \frac{17{,}29\ kN}{1.5} = \qquad 11{,}52\ kN \qquad \text{(EN1992-4 Table 7.1)}$$

4.6.3 Concrete cone breakout verification

$$A_{c,N} = \left\{\min\left(c, c_{cr,N}\right) + (r-1)\min\left(s_2, s_{cr,N}\right) + \min\left(\frac{s_{cr,N}}{2}, \frac{s_2}{2}\right)\right\} * \min\left(s_1, s_{cr,N}\right)$$

$$A_{c,N} = \left\{\min(100; 187{,}5) + (3-1)\min(200; 375) + \min\left(\frac{375}{2}, \frac{200}{2}\right)\right\} * \min(100; 375)$$

$$= \quad 60000\ mm^2$$

$$\psi_{s,N} = 0.7 + 0.3\frac{100}{187{,}5} \leq 1 \quad = \quad 0{,}86 \qquad \text{(EN1992-4 7.16)}$$

$$\psi_{ec,N} = \frac{1}{1+2\frac{e_N}{s_{cr,N}}} = \quad 1 \qquad \text{(EN1992-4 7.17)}$$

$$N_{Rk,c} = N^0_{Rk,c}\ \frac{A_{c,N}}{A^0_{c,N}}\ \psi_{s,N}\ \ \psi_{ec,N}\ \ \psi_{re,N} =$$

$$= 48{,}13\ kN * \frac{60000\ mm^2}{140625\ mm^2} * 0{,}86 * 1 * 1 * 1 = \quad 17{,}66\ kN \qquad \text{(EN1992-4 7.13)}$$

$$N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}} = \frac{17{,}66\ kN}{1{,}5} = \quad 11{,}77\ kN \qquad \text{(EN1992-4 Table7.1)}$$

4.6.4 Splitting verification

Since the concrete is assumed to be cracked, the splitting failure verification is not needed.

4.6.5 Summary

The lower resistance of the anchorage to the existing slab is associated to the combined pull-out/concrete cone failure. $N_{Rd,p} = 11{,}52\ kN$.

4.7 Verification of connectors: lateral part, new slab

It is now necessary to evaluate also the anchorage in the new slab. In this case, the connector can be seen as a cast in headed stud

4.7.1 Pull-out verification

The verification for pull-out of a cast in headed stud is referred to the single anchor, so in this case the resistance is the same as in the central part of the slab.

4.7.2 Concrete cone breakout verification

$$A_{c,N} = \left\{\min\left(c, c_{cr,N}\right) + (r-1)\min\left(s_2, s_{cr,N}\right) + \min\left(\frac{s_{cr,N}}{2}, \frac{s_2}{2}\right)\right\} * \min\left(s_1, s_{cr,N}\right)$$

$$A_{c,N} = \left\{\min(100; 82{,}5) + (3-1)\min(200; 165) + \min\left(\frac{165}{2}, \frac{200}{2}\right)\right\} * \min(100; 165)$$

$$= \quad 49500\ mm^2$$

$$\psi_{s,N} = 0.7 + 0.3\frac{100\ mm}{82{,}5\ mm} \leq 1 \quad 1 \qquad \text{(EN1992-4 7.16)}$$

$$\psi_{ec,N} = \frac{1}{1+2\frac{e_N}{s_{cr,N}}} = \quad 1 \qquad \text{(EN1992-4 7.17)}$$

$$N_{Rk,c} = N^0_{Rk,c}\ \frac{A_{c,N}}{A^0_{c,N}}\ \psi_{s,N}\ \ \psi_{ec,N}\ \ \psi_{re,N} =$$

$= 18{,}15\ kN * \frac{49500\ mm^2}{27225\ mm^2} * 1 * 1 * 1 * 1 =$ 33,00 kN (EN1992-4 7.13)

$N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}} = \frac{33{,}00\ kN}{1{,}5} =$ 22,00 kN (EN1992-4 Table7.1)

4.7.3 Splitting verification

Since the concrete is assumed to be cracked, the splitting failure verification is not needed.

4.7.4 Summary

The resistance of the connectors placed close to the edges is governed by the combined concrete cone/pull-out failure in the existing slab. The ultimate resistance is $N_{Rd,p} = 11{,}52\ kN$

The following tables summarize the resistance of the connectors:

Design resistance, central part of the slab				
Steel failure resistance	$N_{Rd,s}$	=	27,67	kN
Existing slab				
Combined concrete cone/pull-out resistance	$N_{Rd,p}$	=	20,10	kN
Concrete cone breakout resistance	$N_{Rd,c}$	=	20,53	kN
New slab				
Pull-out resistance	$N_{Rd,p}$	=	153,94	kN
Concrete cone breakout resistance	$N_{Rd,c}$	=	12,10	kN

Failure mode: Concrete cone breakout in the new slab

Resistance: $N_{rd} = 12{,}10\ kN$

Design resistance, lateral part of the slab				
Steel failure resistance	$N_{Rd,s}$	=	27,67	kN
Existing slab				
Combined concrete cone/pull-out resistance	$N_{Rd,p}$	=	11,53	kN
Concrete cone breakout resistance	$N_{Rd,c}$	=	11,77	kN
New slab				
Pull-out resistance	$N_{Rd,p}$	=	153,94	kN
Concrete cone breakout resistance	$N_{Rd,c}$	=	22,00	kN

Failure mode: Combined concrete cone/pull-out in the existing slab

Resistance: $N_{rd} = 11{,}53\ kN$

4.8 Verification of the interface in the central part of the slab

The method of verification chosen for this example is the one proposed by FIB in model code 2010.

First, it is computed the longitudinal shear resistance of the interface without shear connectors:

$$v_{RD,ct} = (c_a\, f_{ctd} + \mu_e\, \sigma_n)\, b_j \leq (0.5\, \nu_e\, f_{cd}) b_j$$

$$= (0{,}025 * \min(1.20\ MPa; 1{,}03\ MPa) + 0.5 * 0) * 5000\ mm$$
$$\leq \big(0.5 * 0{,}55 * \min(13{,}3\ MPa; 16{,}7\ MPa)\big) * 5000\ mm$$

$$= \qquad 128.94\, \frac{kN}{m}$$ (MC 2010, 7.3-50)

Since:

$$v_{ED,i} = 348{,}15\ \frac{kN}{m} \geq\ v_{RD,ct} = 128.94\frac{kN}{m}$$

The shear connectors are needed in the central part of the slab. The resistance of the interface is recomputed taking into account the presence of the connectors. The geometry considered for the connectors installation is the one already defined (see section 4.2).

From the verification of the connectors in the central part of the slab we have $N_{Rd,c} = 12{,}10\ kN$

$$\sigma_A = \frac{N_{rd}}{A_S} = \frac{12{,}10\ kN}{83\ mm^2} = \qquad 146\ MPa$$

$$\rho = \frac{A_s}{s^2} = \frac{83\ mm^2}{(300\ mm)^2} = \qquad 0{,}0912\ \%$$

It is now possible to use the formula for the computation of the interface shear resistance according to FIB:

$$v_{RD} = \left(c_r\ f_{ck}^{\frac{1}{3}} + \mu_e\, \sigma_n +\ \rho\, \mu \min(\sigma_A\ , k_{1e} f_{yd}) + k_{2e}\, \rho\, \sqrt{f_{yd}\, f_{cd}}\right) b_j\ \ \leq (\beta_c\, \nu\, f_{cd}) b_j$$

$$= \Big(0 * \min(20MPa; 25MPa)^{\frac{1}{3}} + 0.5 * 0 + \frac{0.091}{100} * 0.5 * \min(146\ MPa;\ \ 0 * 333{,}3\ MPa) + 1.5$$
$$* \frac{0.091}{100} \sqrt{333{,}3\ MPa *\ \min(13{,}3MPa; 16{,}7MPa)}\ \Big) * 5000\ mm$$
$$\leq (0{,}3 * 0{,}55 * \min(13{,}3MPa; 16{,}7MPa)) * 5000\ mm$$

$$= \qquad 460.53\ \frac{kN}{m}$$ (MC 2010, 7.3-51)

In this case $v_{ED,i} = 348{,}15\ \frac{kN}{m} \leq V_{RD} = 460.53\ kN/m$ so the selected design option for the shear connection can be effectively used.

4.9 Verification of the connection close to the edges

It is necessary to define the actions on the interface close to the edge. In this position the most relevant source of longitudinal shear load is the effect of shrinkage. It is assumed as maximum shear due to shrinkage a force equal to the cracking force in the new slab. This assumption is conservative since the design of the interface for this force is equivalent to impose that the connection cannot experience shear failure since for higher actions the cracking happens before reaching the shear resistance.

$f_{ct,eff} = f_{ctm} = 0.3 f_{ck}^{\frac{2}{3}} = 0.3 \cdot 25^{\frac{2}{3}} =$ 2.6 MPa (MC 2010, 7.2.3.1.1)

$l_e = 6h_N =$ 600 mm (B2.5, 3.3.1)

$F_{cr} = h_N \; b_j \; k \; f_{ct,eff} = 100\,mm * 5000\,mm * 0{,}8 * 2{,}6\,Mpa =$ 1026 kN (B2.5, 3.3.1)

$V_{ed} = F_{cr} =$ 1026 kN (B2.5, 3.3.1)

$v_{ed} = \frac{V_{ed}}{l_e} = \frac{1026\,kN}{600mm} =$ 1709.98 kN/m (B2.5, 3.3.1)

Considering the rotational equilibrium of the end portion of the upper slab close to the edge, it is possible to approximate conservatively the tension load acting on the connectors as:

$N_{T,ed} = \frac{F_{cr}}{6} = \frac{1026\,kN}{6} =$ 171.00 kN (B2.5, 3.3.1)

As in the previous computations, also in this case the longitudinal shear resistance of the interface without connectors is considered:

$v_{ED} = 1709.98\,\frac{kN}{m} \geq v_{RD,ct} =$ 128.94 $\frac{kN}{m}$

So the shear connectors are needed also to sustain the shear at the edges.

In this case $N_{Rd,p} = 11{,}52\,kN$:

$\sigma_A = \frac{N_{rd}}{A_S\,r} = \frac{11{,}52\,kN}{83\,mm^2 * 3} =$ 46.29 MPa

$\rho = \frac{A_S\,r}{l_e\,s_1} = \frac{83\,mm^2 * 3}{600\,mm * 100\,mm} =$ 0,42 %

$$v_{RD} = \left(c_r \; f_{ck}^{\frac{1}{3}} + \mu\,\sigma_n + \rho\mu_e \; \min(\sigma_A\,, k_{1e} f_{yd}) + k_{2e}\,\rho\,\sqrt{f_{yd}\,f_{cd}} \right) b_j \;\; \leq (\beta_c\,\nu_e\,f_{cd})$$

$$= \Big(0 * \min(20MPa; 25MPa)^{\frac{1}{3}} + 0.5 * 0 + \frac{0.42}{100} * 0.5 * \min(37{,}13\,MPa;\; 0 * 333{,}3\,MPa) + 1.5 * \frac{0.42}{100} \sqrt{333{,}3\,MPa * \min(13{,}3MPa; 16{,}7MPa)} \Big) * 5000\,mm \leq (0{,}3 * 0{,}55 * \min(13{,}3MPa; 16{,}7MPa)) * 5000\,mm$$

$=$ 2072.40 kN/m (MC 2010, 7.3-51)

In this case $v_{ED} = 1709.98\,\frac{kN}{m} \leq v_{RD} = 2072.40\,kN/m$ so the connection is verified for shear.

At the edges, also the tensile resistance of the connectors must be considered:

$N_{T,RD} = N_{rd}\,\frac{b_j}{s_1} = 11{,}52\,kN * \frac{5000\,mm}{100\,mm} =$ 576.33 kN

Since $N_{T,ed} = 171.00\,kN \leq N_{T,RD} = 576.33\,kN$ the connection is verified also for tension

The obtained results are reported in the figure below:

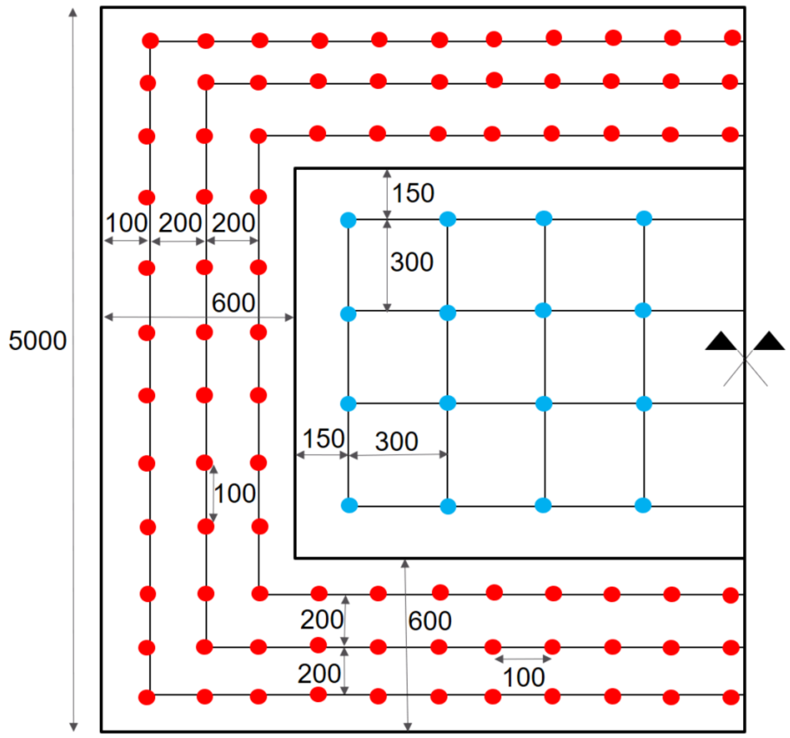


Fig. 14: Layout of the connectors (measurements in millimetres)

4.10 Verification of the connection for a rough interface

It is now of interest the design of a connection similar to the one already designed in the previous paragraphs but, in this case, assuming a very rough surface. The verifications are still carried out according to FIB Model Code 2010. All the data used in the computations remains the same as for the previous design.

As already done, first it is computed the longitudinal shear resistance of the interface without shear connectors:

$$v_{RD,ct} = (c_a \, f_{ctd} + \mu_e \, \sigma_n) \, b_j \leq (0.5 \, v_e \, f_{cd}) b_j$$

$$= (0{,}5 * \min(1.20 \, MPa; 1{,}03 \, MPa) + 0.8 * 0) * 5000 \, mm$$
$$\leq \left(0.5 * 0{,}55 * \min(13{,}3 \, MPa; 16{,}7 \, MPa)\right) * 5000 \, mm$$

$$= \qquad 2578{,}8 \, \frac{kN}{m}$$ (MC 2010, 7.3-50)

Since:

$$v_{ED,i} = 348{,}15 \, \frac{kN}{m} < v_{RD,ct} = \qquad 2578.8 \, \frac{kN}{m}$$

It is possible to avoid the installation of the connectors in the central part of the plate. Even if they are not needed for static reasons, it is recommended to insert at least two connectors for square meter, for constructive reasons.

Close to the edges of the concrete slab the connectors are always needed since it is necessary to sustain the tension normal to the interface, coming from the effect of the restrained shrinkage of the new layer of concrete

It was chosen for the design of the connectors close to the edge to make reference to a configuration with only one row of edge anchors with spacing $s_1 = 300\ mm$. For the distance from the edge c it was chosen the maximum one allowed, equal to 150 mm.

Also in this case it is necessary to define the actions on the interface:

$V_{ed} = F_{cr} =$ 1026 kN (B2.5, 3.3.1)

$l_e = 3h_N =$ 300 mm (B2.5, 3.3.1)

$v_{ed} = \frac{V_{ed}}{l_e} = \frac{1026\ kN}{300mm} =$ 3419.95 kN/m (B2.5, 3.3.1)

It is possible to approximate conservatively the tension load acting on the connectors as:

$N_{T,ed} = \frac{F_{cr}}{6} = \frac{1026\ kN}{6} =$ 171.00 kN (B2.5, 3.3.1)

As in the previous computations, also in this case the longitudinal shear resistance of the interface without connectors is considered:

$v_{ED} = 3419.95\ \frac{kN}{m} \geq v_{RD,ct} =$ 2578.8 $\frac{kN}{m}$

So the shear connectors are needed to sustain the shear at the edges.

In this case $N_{Rd,N} = 12.10\ kN$:

$\sigma_A = \frac{N_{rd}}{A_S\ r} = \frac{12.10\ kN}{83\ mm^2 * 3} =$ 145.79 MPa

$\rho = \frac{A_S\ r}{l_e\ s_1} = \frac{83\ mm^2 * 1}{300\ mm * 300\ mm} =$ 0,09 %

$$v_{RD} = \left(c_r\ f_{ck}^{\frac{1}{3}} + \mu_e \sigma_n + \rho\mu\ \min(\sigma_A\ , k_{1e} f_{yd}) + k_{2e}\ \rho\ \sqrt{f_{yd}\ f_{cd}}\ \right) b_j \ \leq (\beta_c\ v_e\ f_{cd})$$

$$= \left(0.2 * \min(20MPa; 25MPa)^{\frac{1}{3}} + 0.8 * 0 + \frac{0.09}{100} * 0.8 * \min(37{,}13\ MPa;\ \ 0.5 * 333{,}3\ MPa) + 0.9 * \frac{0.09}{100} \sqrt{333{,}3\ MPa * \ \min(13{,}3MPa; 16{,}7MPa)}\ \right) * 5000\ mm \leq (0{,}5 * 0{,}55 * \min(13{,}3MPa; 16{,}7MPa)) * 5000\ mm$$

$=$ 3528.55 kN/m (MC 2010, 7.3-51)

In this case, $v_{ED} = 3419.95\ \frac{kN}{m} \leq v_{RD} = 3528.55\ kN/m$ so the connection is verified for shear.

At the edges, the tensile resistance of the connectors must be considered:

$N_{T,RD} = N_{rd}\ \frac{b_j}{s_1} = 12.10\ kN * \frac{5000\ mm}{300\ mm} =$ 201.68 kN

Since $N_{T,ed} = 171.00\ kN \leq N_{T,RD} = 201.68\ kN$ the connection is verified also for tension

The obtained results are reported in the figure below:

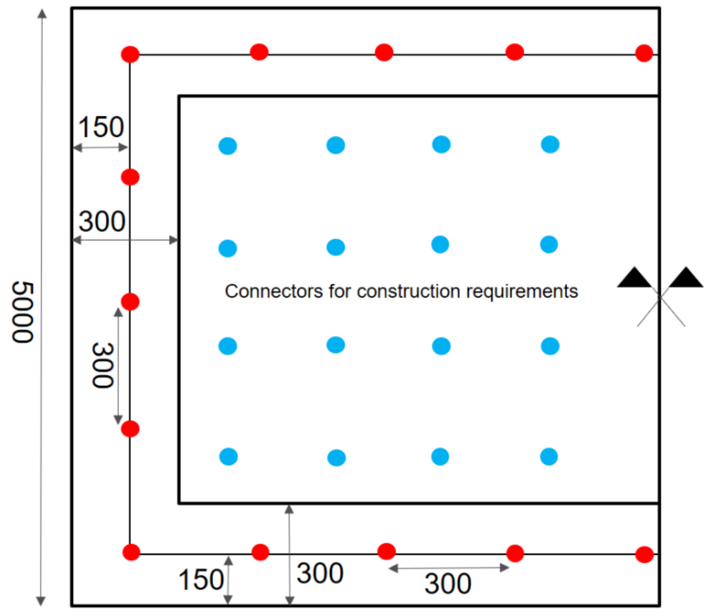


Fig. 15: Layout of the connectors (measurements in millimetres)

4.11 Alternative design solution using the Hilti method

In the following, an alternative solution is proposed based on the use of hooked rebar connectors verified by means of the Hilti method for interface verification. The design problem considered is the same as the one solved in the previous part, considering a very smooth interface between the existing concrete and the concrete overlay.

The data regarding this new design solution are reported below:

type of connector	Rebar Φ10
type of resin	HIT-RE 500-V3

Anchor spacing	$s =$	[mm]	300
Spacing between anchors close to the edge	$s_1 =$	[mm]	120
Anchors edge distance	$c =$	[mm]	100
Number of rows of anchors close to the edge	$r =$	[-]	3
Embedment depth in the new slab	$h_{ef,n} =$	[mm]	60
Length of the connector	$L=$	[mm]	120

From the choice of connector and of the resin, it is possible to find all the needed data from the documentation of the product. These data are displayed in the table below:

Data from the relevant documentation:			
Hole diameter	d_0 =	[mm]	12
Anchor diameter	d =	[mm]	10
Anchorage cross section area	A_s =	[mm²]	78,54
Safety coefficient for concrete cone	γ_{Mc} =	[-]	1.5
Safety coefficient for splitting	γ_{Msp} =	[-]	1.5
Safety coefficient for pull-out	γ_{Mp} =	[-]	1.5
Safety coefficient for steel failure	γ_{MS} =	[-]	1.4
Steel resistance of the anchorage	f_{uk} =	[MPa]	540,00
Design resistance of the anchor steel	f_{yd} =	[MPa]	357,14
Minimum existing slab thickness for splitting	$h_{min\ E}$ =	[mm]	90
Minimum new slab thickness for splitting	$h_{min\ N}$ =	[mm]	90
Critical spacing for splitting in new slab	$s_{cr,spN}$ =	[mm]	192
Critical edge distance for splitting in new slab	$c_{cr,spN}$ =	[mm]	96
Bond strength in existing cracked concrete	τ_e =	[MPa]	8.50
Bond strength in new cracked concrete	τ_n =	[MPa]	8.67
Bond strength in uncracked concrete C20/25	$\tau_{Rk,ucr}$ =	[MPa]	14.00

4.12 Verification of connectors: central part, existing slab

As already computed, the longitudinal shear is: $348{,}15\ kN/m$

The load carrying capacity of the connector is defined taking into account separately the geometry of the installation in the central part of the slab and in the external part close to the edge.

First, the central part of the slab is considered, taking into account that the tensile resistance of the connector is the minimum one between the values obtained considering the anchorage in the existing and in the new slab.

In the existing slab the connector is a post installed bonded anchor, the computation of the load carrying capacity requires to consider different failure modes:

4.12.1 Steel failure:

The steel failure resistance of the connector is taken directly from the values given in the relevant documentation

$N_{Rk,s} =$ $43{,}00\ kN$

From this, the design values is obtained as:

$N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}} = \frac{43{,}00\ kN}{1{,}4} =$ $30{,}71\ kN$ (EN1992-4 Table 7.1)

4.12.2 Combined concrete cone/pull-out verification

$N^0_{Rk,p} = \pi\ d\ h_{ef,e}\ \tau_{Rk} = \pi\ 10mm * 60mm * 8{,}5MPa =$ $16{,}02\ kN$ (EN1992-4 7.3.1)

$$s_{cr,Np} = 7.3\ d\ \sqrt{\tau_{Rk,ucr}} = 7.3 * 10mm\ \sqrt{14\ MPa} = \quad 273{,}14\ mm \quad \text{(EN1992-4 7.5)}$$

$$s_{cr,Np} \leq 3\ h_{ef}\ = \quad 180\ mm \quad \text{(EN1992-4 7.5)}$$

$$s_{cr,Np} = \quad 180\ mm$$

$$c_{cr,Np} = \frac{s_{cr,Np}}{2} = \frac{180\ mm}{2} = \quad 90\ mm \quad \text{(EN1992-4 5.6)}$$

$$A^0_{p,N} = (s_{cr,Np})(s_{cr,Np}) = (180\ mm)^2 = \quad 32400\ mm^2 \quad \text{(EN1992-4 7.3.2)}$$

$$A_{p,N} = \left(\min(s, s_{cr,Np})\right)^2 = (\min(300\ mm; 180\ mm))^2 = \quad 32400\ mm^2$$

$$\psi_{s,Np} = 0.7 + 0.3\frac{c}{c_{cr,Np}} \leq 1 = \quad since\ c = \infty \quad 1 \quad \text{(EN1992-4 7.10)}$$

$$\psi_{ec,Np} = \frac{1}{1+2\frac{e_n}{s_{cr,Np}}} = \quad since\ e_n = 0 \quad 1 \quad \text{(EN1992-4 7.12)}$$

Assuming conservatively n=8 for the maximum number of anchors:

$$\psi^0_{g,Np} = \sqrt{n} - (\sqrt{n} - 1)\left(\frac{d\ \pi\ \tau_{Rk}}{k_8\ \sqrt{h_{ef}\ f_{ck}}}\right)^{1.5} \geq 1 =$$

$$= \sqrt{8} - (\sqrt{8} - 1)\left(\frac{\pi * 10\ mm * 8.5\ MPa}{7.7\ \sqrt{60\ mm * 20 MPa}}\right)^{1.5} \geq 1 = \quad 1 \quad \text{(EN1992-4 7.8-7.9)}$$

$$\psi_{g,Np} = \psi^0_{g,Np} - \left(\frac{S}{S_{cr,Np}}\right)^{0.5} (\psi^0_{g,Np} - 1) \geq 1 =$$

$$= 1.00 - \left(\frac{300mm}{180mm}\right)^{0.5} (1 - 1) \geq 1 = \quad 1 \quad \text{(EN1992-4 7.7)}$$

$$\psi_{re,Np} = 0.5 + \frac{h_{ef}}{200} \leq 1 \ = \quad 1 \quad \text{(EN1992-4 7.11)}$$

It is then possible to define the characteristic and the design resistances:

$$N_{Rk,p} = N^0_{Rk,p}\ \frac{A_{p,N}}{A^0_{p,N}}\ \psi_{s,Np}\ \ \psi_{g,Np}\ \ \psi_{ec,Np}\ \ \psi_{re,Np} =$$

$$16{,}02\ kN * \frac{32400\ mm^2}{32400\ mm^2} * 1 * 1 * 1 * 1 = \quad 16{,}02\ kN \quad \text{(EN1992-4 7.3)}$$

$$N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}} = \frac{16{,}02\ kN}{1.5} = \quad 10{,}68\ kN \quad \text{(EN1992-4 Table 7.1)}$$

4.12.3 Concrete cone breakout verification

$$N^0_{Rk,c} = k_g\ \sqrt{f_{ck}}\ h^{1.5}_{ef} = 7.7\ \sqrt{20\ MPa}\ (60\ mm)^{1.5} = \quad 16{,}00\ kN \quad \text{(EN1992-4 7.14)}$$

$$s_{cr,N} = 3\ h_{ef} = 3 * 60\ mm = \quad 180\ mm \quad \text{(EN1992-4 7.14)}$$

$$c_{cr,N} = \frac{s_{cr,N}}{2} = \frac{180\ mm}{2} = \quad 90\ mm$$

$$A_{c,N} = \left(\min(s, s_{cr,N})\right)^2 = \left(\min(300\ mm; 180\ mm)\right)^2 = \quad 32400\ mm^2$$

$$A^0_{c,N} = (s_{cr,N})(s_{cr,N}) = (180\ mm)^2 = \quad 32400\ mm^2 \quad \text{(EN1992-4 7.15)}$$

$\psi_{s,N} = 0.7 + 0.3 \frac{c}{c_{cr,N}} \leq 1 \quad =$ 1 (EN1992-4 7.16)

$\psi_{ec,N} = \frac{1}{1+2\frac{e_n}{s_{cr,N}}} =$ 1 (EN1992-4 7.17)

$\psi_{re,N} = 0.5 + \frac{h_{ef}}{200} \leq 1 \ =$ 1 (EN1992-4 7.11)

$N_{Rk,c} = N_{Rk,c}^0 \ \frac{A_{c,N}}{A_{c,N}^0} \ \psi_{s,N} \ \ \psi_{ec,N} \ \ \psi_{re,N} =$

$= 16{,}00\ kN * \frac{32400\ mm^2}{32400\ mm^2} * 1 * 1 * 1 * 1 =$ $16{,}00\ kN$ (EN1992-4 7.13)

$N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}} = \frac{16{,}00\ kN}{1{,}5} =$ $10{,}67\ kN$ (EN1992-4 Table 7.1)

4.12.4 Splitting verification

The concrete is assumed to be cracked, so the splitting failure verification is not needed

4.12.5 Summary

The lowest resistance of the anchorage in the existing slab is associated to concrete cone breakout. $N_{Rd,c} = 10{,}67\ kN$

4.13 Verification of connectors: central part, new slab

It is now necessary to evaluate also the anchorage in the new slab. In this case, the connector is a cast-in hooked rebar.

4.13.1 Pull-out verification

$e_h = \min\big(max\big(h_{ef,n} - d, 3d\big),\ 4{,}5d\big) =$ $45\ mm$ (ACI code)

$N_{Rk,p} = 0{,}9\ f_{ck}\ e_h\ d =$ $10{,}13\ kN$ (ACI code)

$N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mc}} = \frac{10{,}13\ kN}{1{,}5} =$ $6{,}75\ kN$ (EN1992-4 Table 7.1)

4.13.2 Concrete cone breakout verification

$N_{Rk,c}^0 = k_g \sqrt{f_{ck}}\ h_{ef}^{1.5} = 7.7 \sqrt{25\ MPa}\ (\ 60\ mm)^{1.5} =$ $20{,}68\ kN$ (EN1992-4 7.14)

$s_{cr,N} = 3\ h_{ef} = 3 * 60\ mm =$ $180\ \ mm$ (EN1992-4 7.14)

$c_{cr,N} = \frac{s_{cr,N}}{2} = \frac{180\ mm}{2} =$ $90\ \ mm$

$A_{c,N} = \big(\min\big(s, s_{cr,N}\big)\big)^2 = \big(\min(300\ mm; 180\ mm)\big)^2 =$ $32400\ mm^2$

$A_{c,N}^0 = (s_{cr,N})(s_{cr,N}) = (180\ mm)^2 =$ $32400\ mm^2$ (EN1992-4 7.15)

$\psi_{s,N} = 0.7 + 0.3 \frac{c}{c_{cr,N}} \leq 1 \quad =$ 1 (EN1992-4 7.16)

$\psi_{ec,N} = \frac{1}{1+2\frac{e_n}{s_{cr,N}}} =$ 1 (EN1992-4 7.17)

$\psi_{re,N} = 0.5 + \frac{h_{ef}}{200} \leq 1 \ =$ 1 (EN1992-4 7.11)

$$N_{Rk,c} = N^0_{Rk,c}\ \frac{A_{c,N}}{A^0_{c,N}}\ \psi_{s,N}\ \ \psi_{ec,N}\ \ \psi_{re,N} =$$

$$= 20{,}68\ kN * \frac{32400\ mm^2}{32400\ mm^2} * 1 * 1 * 1 * 1 = \qquad 20{,}68\ kN \qquad \text{(EN1992-4 7.13)}$$

$$N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}} = \frac{20{,}68\ kN}{1{,}5} = \qquad 13{,}79\ kN \qquad \text{(EN1992-4 Table 7.1)}$$

4.13.3 Splitting verification

Since the concrete is assumed to be cracked, the splitting failure verification is not needed.

4.13.4 Summary

The lower resistance of the anchorage in the new slab is related to the pull-out, this value is also smaller than the minimum one coming from the verification of the connection with the existing slab. In conclusion, we can consider that the failure mode of the connector is associated to pull-out in the new slab and the tension design resistance is $N_{Rd,c} = 6{,}75\ kN$

4.14 Verification of connectors: lateral part, existing slab

A similar procedure is than followed for the verification of the connectors of the external rows, close to the edges of the concrete slab. Only the parameters that differs from the ones computed for the central part need to be recomputed. In this case the verification is carried out considering a strip of anchors perpendicular to the free edge of the slab. We start considering the existing slab:

4.14.1 Steel failure

Since the connectors are assumed to be the same both for lateral and central part of the plate the steel failure resistance is obviously the same

4.14.2 Combined concrete cone/pull-out verification

First, it is needed to define the spacing of the anchors. We compute the width of the lateral strip subjected to higher shrinkage stresses as:

$$l_e = \begin{cases} 3\ h_N & \quad rough\ surfaces \\ 6\ h_N & \quad smooth\ surfaces \end{cases}$$

For this example:

$$l_e = 6 * 100mm = \qquad 600mm \qquad \text{(B2.5, 3.3.1)}$$

$$S_2 = \frac{2(l_e - c)}{2r-1} = \frac{2(600\ mm - 100\ mm)}{2*3-1} = \qquad 200\ mm$$

Then it is possible to define the idealized failure area:

$$A_{p,N} = \left\{\min\left(c, c_{cr,Np}\right) + (r-1)\min\left(s_2, s_{cr,Np}\right) + \min\left(\frac{s_{cr,Np}}{2}, \frac{s_2}{2}\right)\right\} * \min\left(s_1, s_{cr,Np}\right)$$

$$A_{p,N} = \left\{\min(100; 90) + (3-1)\min(200; 180) + \min\left(\frac{180}{2}, \frac{200}{2}\right)\right\} * \min(120; 180)$$

$$= \qquad 64800\ mm^2$$

$$\psi_{s,Np} = 0.7 + 0.3\frac{c}{c_{cr,Np}} \leq 1 = 0.7 + 0.3\frac{100\ mm}{90\ mm} \leq 1 = \qquad 1 \qquad \text{(EN1992-4 7.10)}$$

$$\psi_{ec,Np} = \frac{1}{1 + 2\frac{e_n}{s_{cr,Np}}} = \ (since\ e_n = 0) \qquad 1 \qquad \text{(EN1992-4 7.12)}$$

We can now define the characteristic and the design resistances:

$$N_{Rk,p} = N^0_{Rk,p}\ \frac{A_{p,N}}{A^0_{p,N}}\ \psi_{s,Np}\ \ \psi_{g,Np}\ \ \psi_{ec,Np}\ \ \psi_{re,Np} =$$

$$= 16{,}02\ kN * \frac{64800\ mm^2}{32400\ mm^2} * 1 * 1 * 1 * 1 = \qquad 32{,}04\ kN \qquad \text{(EN1992-4 7.3)}$$

$$N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}} = \frac{32{,}04\ kN}{1.5} = \qquad 21{,}36\ kN \qquad \text{(EN1992-4 Table 7.1)}$$

4.14.3 Concrete cone breakout verification

$$A_{c,N} = \left\{\min\left(c, c_{cr,N}\right) + (r-1)\min\left(s_2, s_{cr,N}\right) + \min\left(\frac{s_{cr,N}}{2}; \frac{s_2}{2}\right)\right\} * \min\left(s_1, s_{cr,N}\right)$$

$$A_{c,N} = \left\{\min(100; 90) + (3-1)\min(200; 180) + \min\left(\frac{180}{2}; \frac{200}{2}\right)\right\} * \min(120; 180)$$

$$= \qquad 64800\ mm^2$$

$$\psi_{s,N} = 0.7 + 0.3\frac{100}{90} \le 1 = \qquad 1 \qquad \text{(EN1992-4 7.16)}$$

$$\psi_{ec,N} = \frac{1}{1+2\frac{e_N}{s_{cr,N}}} = \qquad 1 \qquad \text{(EN1992-4 7.17)}$$

$$N_{Rk,c} = N^0_{Rk,c}\ \frac{A_{c,N}}{A^0_{c,N}}\ \psi_{s,N}\ \ \psi_{ec,N}\ \ \psi_{re,N} =$$

$$= 16{,}00\ kN * \frac{64800\ mm^2}{32400\ mm^2} * 1 * 1 * 1 * 1 = \qquad 32{,}00\ kN \qquad \text{(EN1992-4 7.13)}$$

$$N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}} = \frac{32{,}00\ kN}{1{,}5} = \qquad 21{,}34\ kN \qquad \text{(EN1992-4 Table7.1)}$$

4.14.4 Splitting verification

Since the concrete is assumed to be cracked, the splitting failure verification is not needed.

4.14.5 Summary

The lower resistance of the anchorage to the existing slab is associated to the concrete cone breakout failure. It is now necessary to evaluate also the anchorage in the new slab. In this case the connector can be seen as a cast in hooked rebar.

4.15 Verification of connectors: lateral part, new slab

4.15.1 Pull-out verification

The verification for pull-out of a cast in headed stud is referred to the single anchor, so in this case the resistance is the same as in the central part of the slab.

4.15.2 Concrete cone breakout verification

$$A_{c,N} = \left\{\min\left(c, c_{cr,N}\right) + (r-1)\min\left(s_2, s_{cr,N}\right) + \min\left(\frac{s_{cr,N}}{2}, \frac{s_2}{2}\right)\right\} * \min\left(s_1, s_{cr,N}\right)$$

$$A_{c,N} = \left\{\min(100; 90) + (3-1)\min(200; 180) + \min\left(\frac{180}{2}, \frac{200}{2}\right)\right\} * \min(120; 180)$$

$$= \qquad 64800\ \ mm^2$$

$\psi_{s,N} = 0.7 + 0.3 \frac{100\ mm}{90\ mm} \leq 1$ 1 (EN1992-4 7.16)

$\psi_{ec,N} = \frac{1}{1+2\frac{e_n}{s_{cr,N}}} =$ 1 (EN1992-4 7.17)

$N_{Rk,c} = N^0_{Rk,c} \frac{A_{c,N}}{A^0_{c,N}} \ \psi_{s,N} \ \ \psi_{ec,N} \ \ \psi_{re,N} =$

$= 20{,}68\ kN * \frac{64800\ mm^2}{32400\ mm^2} * 1 * 1 * 1 * 1 =$ $41{,}36\ kN$ (EN1992-4 7.13)

$N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}} = \frac{41{,}36\ kN}{1{,}5} =$ $27{,}58\ kN$ (EN1992-4 Table7.1)

4.15.3 Splitting verification

Since the concrete is assumed to be cracked, the splitting failure verification is not needed.

4.15.4 Summary

The resistance of the connectors placed close to the edges is governed by the pull-out failure in the new slab. The ultimate resistance is $N_{Rd,p} = 20{,}25\ kN$

4.16 Verification of the interface in the central part of the slab

The method of verification chosen for this example is the Hilti method (also known as Palieraki method).

First, it is computed the longitudinal shear resistance of the interface without shear connectors: in this case, since the external stress acting normal to the interface can be assumed equal to zero, the shear resistance without connectors, according to Hilti method is null.

$v_{Rd,ct} = \mu_h\ \sigma_n\ b_j \leq \beta_c\ \nu_h\ f_{cd}\ b_j =$ $0\ \frac{kN}{m}$

The shear connectors are needed in the central part of the slab. The resistance of the interface is recomputed taking into account the presence of the connectors. The geometry considered for the connectors installation is the one already defined (see section 4.11).

From the verification of the connectors in the central part of the slab we have $N_{Rd,p} = 6{,}75\ kN$

$\sigma_A = \frac{N_{rd}}{A_s} = \frac{6{,}75\ kN}{78{,}54\ mm^2} =$ $85{,}94\ MPa$

$\rho = \frac{A_s}{s^2} = \frac{78{,}54\ mm^2}{(300\ mm)^2} =$ $0{,}09\ \%$

It is now possible to use the formula for the computation of the interface shear resistance according to Hilti method:

$$v_{Rd} = \{\mu_h\ (\sigma_n + \rho\ \min(\ \sigma_A\ , \kappa_{1h}\ \sigma_s\ ) + \kappa_{2h}\ \rho\ \sqrt{f_{yd}\ f_{cd}}\ \}b_j \leq \beta_c\ \nu_h\ f_{cd}\ b_j$$

Where we assume, considering the selected surface condition (Very smooth) and the type of loading (static monotonic):

$k_{1h} =$ 0,2

$k_{2h} =$ 0,5

$$\mu_h = 0{,}3\sqrt[3]{\left(\frac{f_{cd}}{\frac{\rho\,\sigma_A}{0{,}8}+\sigma_n}\right)^2} = 0{,}3\sqrt[3]{\left(\frac{13{,}3\,MPa}{\frac{0{,}09}{100}*\frac{85{,}94}{0{,}8}+0}\right)^2} = \qquad 8{,}16$$

$$v_{Rd} = \left\{8{,}16*\left(0+\frac{0{,}09}{100}\min\left(85{,}94\,MPa;\ 0{,}2*\frac{85{,}94\,MPa}{0{,}8}\right)\right)+0.5 * \frac{0{,}09}{100}\sqrt{357{,}14\,MPa*\min(13{,}3MPa;16{,}7MPa)}\right\}*5000\,mm \le (0{,}3*0{,}55*\min(13{,}3MPa;16{,}7MPa))*5000\,mm$$

$$= 915{,}39\ \frac{kN}{m}$$

In this case $v_{ED,i} = 348{,}15\ \frac{kN}{m} \le V_{RD} = 915{,}39\ kN/m$ so the selected design option for the shear connection can be effectively used.

4.17 Verification of the connection close to the edges

It is necessary to define the actions on the interface close to the edge. As for the previous example:

$v_{ed} = \frac{V_{ed}}{l_e} =$ 1709.98 kN/m (B2.5, 3.3.1)

$N_{T,ed} = \frac{F_{cr}}{6} =$ 171.00 kN (B2.5, 3.3.1)

As in the previous computations, also in this case the longitudinal shear resistance of the interface without connectors is null.

So the shear connectors are needed also to sustain the shear at the edges. In this case $N_{Rd,p} = 20{,}25\ kN$:

$\sigma_A = \frac{N_{rd}}{r\,A_S} = \frac{20{,}25\ kN}{3*78{,}54\ mm^2} =$ $85{,}94\ MPa$

$\rho = \frac{A_S\,r}{l_e\,s_1} = \frac{78{,}54\ mm^2*3}{600\ mm*120\ mm} =$ 0,33 %

$$v_{Rd} = \{\mu_h\,(\sigma_n + \rho\min(\sigma_A, \kappa_{1h}\,\sigma_s) + \kappa_{2h}\,\rho\sqrt{f_{yd}\,f_{cd}}\}b_j \le \beta_c\,v_h\,f_{cd}\,b_j$$

$k_1 =$ 0,2

$k_2 =$ 0,5

$$\mu_h = 0{,}3\sqrt[3]{\left(\frac{f_{cd}}{\frac{\rho\,\sigma_A}{0{,}8}+\sigma_n}\right)^2} = 0{,}3\sqrt[3]{\left(\frac{13{,}3\,MPa}{\frac{0{,}33}{100}*\frac{85{,}94}{0{,}8}+0}\right)^2} = \qquad 3{,}38$$

$$v_{Rd} = \left\{3{,}38*\left(0+\frac{0{,}33}{100}\min\left(85{,}94\,MPa;\ 0{,}2*\frac{85{,}94\,MPa}{0{,}8}\right)\right)+0.5 * \frac{0{,}33}{100}\sqrt{357{,}14\,MPa*\min(13{,}3MPa;16{,}7MPa)}\right\}*5000\,mm \le (0{,}3*0{,}55*\min(13{,}3MPa;16{,}7MPa))*5000\,mm$$

$$= \qquad 1752{,}42\ \frac{kN}{m}$$

In this case $v_{ED} = 1709.98\ \frac{kN}{m} \le v_{RD} = 1752{,}42\frac{kN}{m}$ so the connection is verified for shear.

At the edges, also the tensile resistance of the connectors must be considered:

$$N_{T,RD} = N_{rd} \frac{b_j}{s_1} = 20{,}25\ kN * \frac{5000\ mm}{120\ mm} = \qquad 843{,}75\ kN$$

Since $N_{T,ed} = 171.00\ kN \leq N_{T,RD} = 843{,}75\ kN$ the connection is verified also for tension

The obtained results are reported in the figure below:

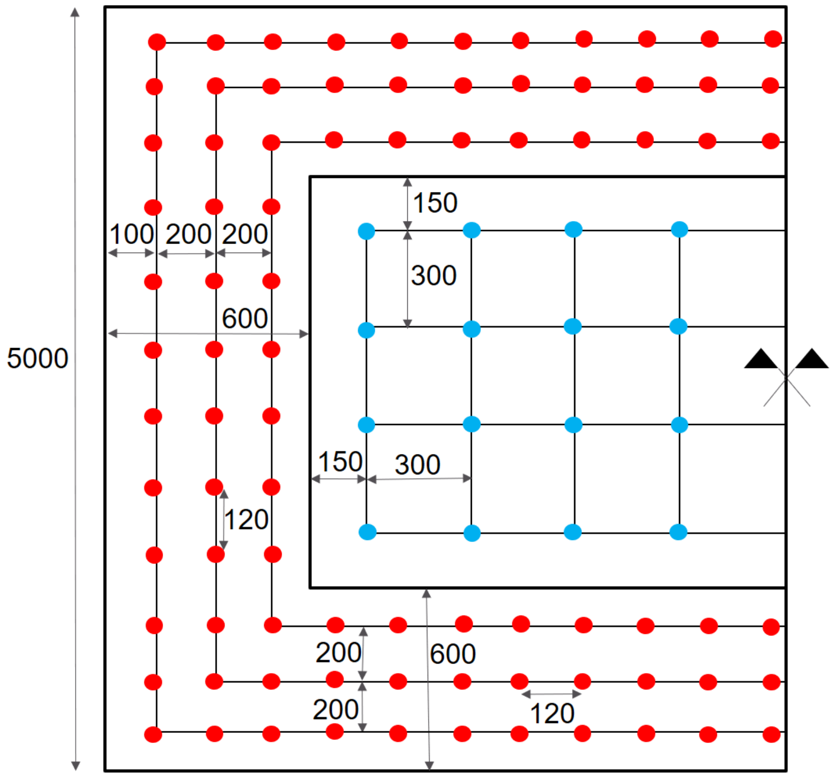


Fig. 16: Layout of the connectors (measurements in millimetres)

5 Notations

Design and characteristic resistances:

f_{ck}	=	minimum value of concrete compressive strength of the two concrete layers, measured on cylinders
f_{yk}	=	characteristic yield strength of the shear connector
f_{cd}	=	minimum value of concrete design compressive strength of the two concrete layers, measured on cylinders
f_{yd}	=	design yield strength of the shear connector
f_{ctd}	=	design tension resistance of concrete
$f_{ct,eff}$	=	tensile strength of overlay effective at the time when the cracks may first be expected to occur as per [1], Section 7.3.2 (for general cases: $f_{ct,eff} = 3$ N/mm^2)
f_{uk}	=	ultimate characteristic steel resistance of the anchorage

Safety coefficients:

γ_{Mc}	=	safety coefficient for concrete cone
γ_{Msp}	=	safety coefficient for splitting
γ_{Mp}	=	safety coefficient for pull-out
γ_{MS}	=	safety coefficient for steel failure
γ_c	=	safety factor for concrete; 1,50 as given in EN 1992-4 for strengthening of existing structures
γ_s	=	safety factor for steel; 1,15 as given in EN 1992-4 for supplementary reinforcement

Stresses:

σ_n	=	lowest expected compressive stress resulting from an eventual normal force acting on the interface (compression has a positive sign)
σ_A	=	steel stress associated to the relevant failure mode, (see section 3.14)
σ_s	=	effective steel stress in the connector
$\Delta\tau_{Ed}$	=	shear stress acting as fatigue relevant loading
$\tau_{Ed,max}$	=	upper shear stress acting as fatigue relevant loading
τ_{Rd}	=	resisting shear stress
τ_e	=	bond strength in existing concrete
τ_n	=	bond strength in new concrete
$\tau_{Rk,ucr}$	=	bond strength in non-cracked concrete C20/25

Forces:

F_{cr}	=	cracking force for the overlay
$N_{Rd,s}$	=	steel failure resistance of the connector
$N_{Rd,cp}$	=	combined concrete cone failure/pull-out resistance of the connector
$N_{Rd,sp}$	=	splitting failure resistance of the connector
$N_{Rd,c}$	=	concrete cone failure resistance of the connector
$N_{Rd,p}$	=	pull-out failure resistance of the connector
V	=	shear acting on the considered section
v_{ED}	=	acting longitudinal shear
v_{RD}	=	resisting longitudinal shear

Deformations, displacements and strains:

W_{eff}	=	additional deformation calculated for the reinforced section considering the elasticity of the shear connectors
W_{calc}	=	additional deformation calculated for the reinforced section assuming a perfect bond
s_d	=	displacement of connectors under the mean permanently acting load $F_p \approx 0.5\ F_{uk}$
$\varepsilon s_{\ Ase-}$	=	top steel strains [%]
$\varepsilon s_{\ Ase+}$	=	bottom steel strains [%]

Areas:

$A_{s,n}$	=	area of the reinforcements in the new slab
$A_{s,ex}$	=	area of the reinforcements on top of the existing slab
A_s	=	effective steel area of the anchor
A_h	=	area of the anchor head

Length:

b_i	=	width of the interface of the composed section
x	=	depth of the compressed part of the composite slab
z	=	internal lever harm of the composite slab
d	=	effective depth of the slab, positive bending moment
d‘	=	effective depth of the slab, negative bending moment
l_e	=	width of the lateral strip considered as subjected to the highest shrinkage forces
h_N	=	thickness of the overlay
h_E	=	thickness of the existing slab
$h_{ef,e}$	=	embedment depth in the existing slab
$h_{ef,n}$	=	embedment depth in the overlay
L_{con}	=	length of shear connector
d_0	=	drill bit diameter
d	=	anchor stud diameter
d_h	=	anchor head diameter
c	=	anchor edge distance
s	=	central anchors spacing
s_2	=	anchor rows spacing at the edges
s_1	=	anchor in-row spacing at the edges
$h_{min\ E}$	=	minimum existing slab thickness for splitting
$h_{min\ N}$	=	minimum new slab thickness for splitting
$s_{cr,spN}$	=	critical spacing for splitting in new slab
$c_{cr,spN}$	=	critical edge distance for splitting in new slab
$s_{cr,spE}$	=	critical spacing for splitting in existing slab
$c_{cr,spE}$	=	critical edge distance for splitting in existing slab
$S_{s,ex}$	=	reinforcements spacing on top of the existing slab
X	=	depth of the neutral axis
Φ_{ex}	=	reinforcements diameter on top of the existing slab
c_s	=	Net cover

Coefficients and ratios:

c_r	=	coefficient for adhesive bond resistance in a reinforced interface
μ_e	=	friction coefficient according to Eota Method
κ_{1e}	=	interaction coefficient for tensile force activated in the shear connector
κ_{2e}	=	interaction coefficient for flexural resistance in the shear connector
α_{k1}	=	modification factor for material properties of the connector
α_{k2}	=	modification factor for geometry of the connector
ρ	=	reinforcement ratio of the steel of the shear connector crossing the interface
ν_e	=	coefficient for reduction of concrete strength
β_c	=	coefficient for the strength of the compression strut
μ_h	=	friction coefficient according to Hilti method
κ_{1h}	=	contribution factor for the friction mechanism
κ_{2h}	=	contribution factor for the dowel mechanism
ν_h	=	effectiveness factor for the concrete according to fib MC2010, Eq. (7.3-51)
k	=	coefficient to allow for non-uniform self-equilibrating stresses = 0.8 for h_N=30 cm
η_{sc}	=	factor for fatigue loading
γ	=	factor for displacements
r	=	number of the lateral rows of anchors
ρ_{min}	=	minimum reinforcement ratio for the interface

6 Literature

[1] EC 2; Design of concrete structures: ENV 1992-1-1: 2004;

[2] Technisches Handbuch der Befestigungstechnik für Hoch- und Ingenieurbau, HILTI

[3] FIB model code 2010

[4] ACI 318-11 - Building Code Requirements (Imperial)

[5] EOTA etag-001-annex-c-10-08-01

[6] Elizabeth Vintzileou and Vasiliki Palieraki, Giovacchino Genesio, Roberto Piccinin: Shear behaviour of interfaces within repaired/strengthened RC elements subjected to cyclic actions, *fib* International Workshop on Advanced Materials and Innovative Systems in Structural Engineering: Seismic Practices (IWAMISSE), November 16, 2018; Istanbul, Turkey

[7] EOTA TR066 design concrete shear connection