Fastening Technology Manual
B 2.6

Strengthening Concrete Slabs against Punching Shear using Hilti Tension Ancohrs HZA-P

Principles and Design

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Contents

1 Strengthening against Punching Shear with Hilti Tension Anchors HZA-P

1.1 Application Range

The safety against punching shear of existing concrete slabs is basically determined on the basis of the geometry and the reinforcement of the slab and the column. Such data can be taken from construction drawings if available or they are evaluated in situ by taking out concrete cores and seeking the existing reinforcement.

Post-installed punching shear reinforcement can be applied in two ways: if both the lower and the uppper side of the slab are accessible for work simultaneously, then holes can be drilled through the slab. Steel bars can then be introduced through the holes and be prestressed against the slab by tightening nuts on both sides (fig. 1). An appropriate mortar is then injected into the annular gap through an injection washer, e.g. the Hilti Dynamic set. Thus the steel rods cannot move under shear load and water cannot penetrate into the annular gap [3].

Such methods which include working from the upper side of the slab also have certain drawbacks: The cover of the slab has to be removed (earth, tiles, etc…). Moreover the waterproofing system is penetrated and has to be repaired properly after installation of the reinforcement.

As often the upper side is not accessible for work or is accessible only with a high effort, a method has been developed to apply punching shear reinforcement only from the lower side of the slab. Hilti tension anchors HZA-P are bonded into drill holes inclined towards the column by means of an appropriate adhesive mortar (fig. 2). The drilled holes should protrude until at least the level just below the lowest layer of the upper (tensile) reinforcement, but preferably to the centre of the tensile reinforcement. As the effectiveness of punching shear reinforcement strongly depends on the quality of its anchorage, a reliable adhesive mortar is required and the lower anchorage is carried out with the Hilti Dynamic set.

As penetrating reinforcement according to fig. 1 can be designed like cast-in-place punching shear reinforcement on the safe side, this brochure will in the following present details of the post-installed punching shear reinforcement applied only from the lower side of the slab according to fig. 2.

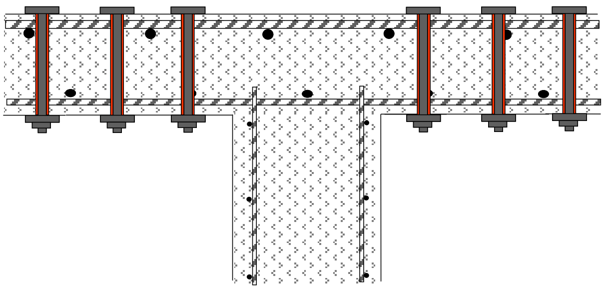


Fig. 1. penetrating post-installed punching shear reinforcement

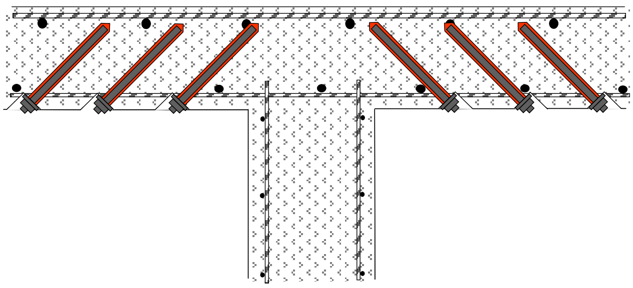
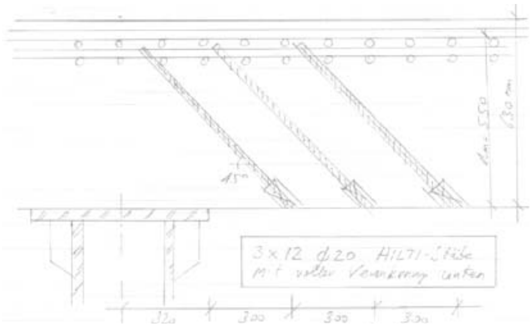


Fig. 2. post-installed punching shear reinforcement applied only from bottom side of the slab

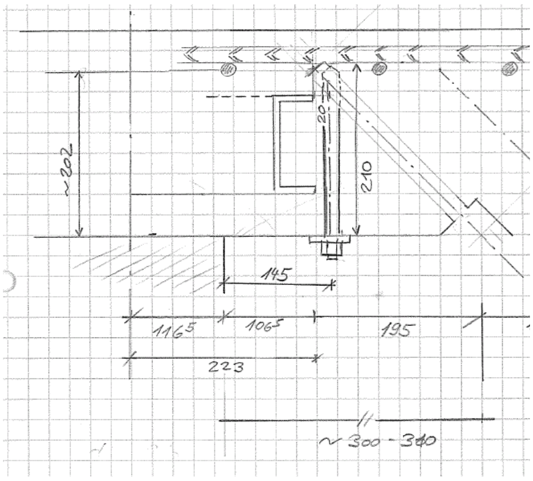
1.2 Advantages of the method

- cost effective reinforcement against punching shear loads
- design according to applicable structural concrete code
- proof of safety level required by structural code
- can be combined with cast-in-place punching shear reinforcement
- simple and fast design with software EXBAR-Punching



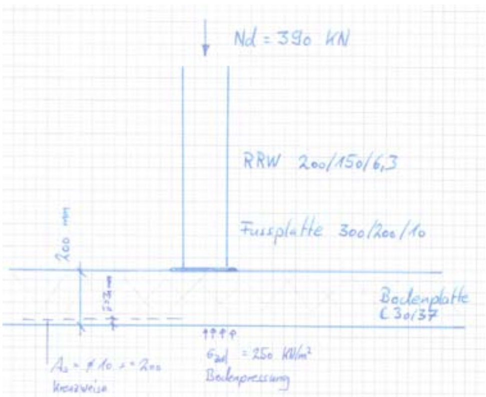
reinforcement of a ceiling

Fig. 3: ceiling reinforcement



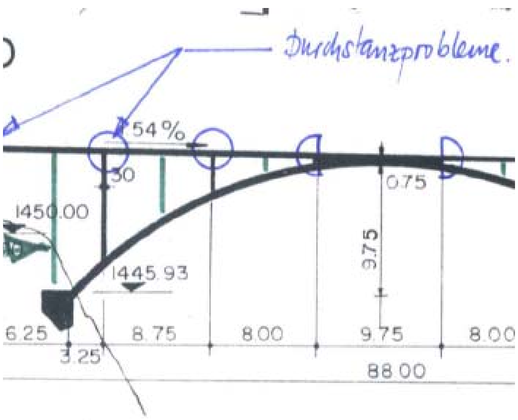
reinforcement of a slab with (insufficient) cast-in-place punching shear reinforcement

Fig. 4: ceiling with cast-in-place shear reinforcement



strengthening of a floor

Fig. 5: floor reinforcement



Strengthening of a bridge deck

Fig. 6: bridge deck reinforcement

2 System description

2.1 Configuration

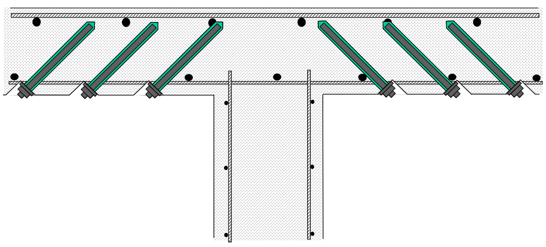


Fig. 8: post-installed punching shear reinforcement

Fig. 9: Hilti tension anchor HZA-P

Hilti Tension Anchors HZA-P in combination with Hilti adhesive mortars are used to install punching shear reinforcement into already hardened concrete slabs.
Inclined holes are hammer drilled into the concrete slab under an angle of 45° and in the direction towards the column. The length of the drilled holes should be such that they reach at least the lowest level of the upper (tensile) reinforcement, but preferably, the holes should end at the level between the tensile reinforcements in the two directions.

Adhesive mortar Hilti HIT-RE 500 or Hilti HIT HY 150 is injected into the drilled holes and the Hilti Tension Anchors HZA-P are set into the mortar filled holes. The Hilti tension anchor consists of a reinforcement bar of diameter 16mm or 20mm in the upper part. The lower part is a smooth shaft with a thread M16 or M20 at the end. For the design, the reinforcement bar is decisive since the smooth shaft and thread are made of steel with higher yield strength than that of the reinforcement bar.

After curing of the mortar, the lower anchor head is installed. The Hilti Dynamic Set consists of an injection washer (diameter 52mm for M16 / 60mm for M20), a spherical washer to eliminate bending of the bar and a nut. In order to create a slip-free anchorage the annular gaps are filled through the injection washer with Hilti HIT-RE 500.

Fig. 10: anchor head external

Fig. 11: anchor head in concrete

Fig. 12: Hilit dynamic set

The anchor head can be installed on the concrete surface with washers inclined at 45° or be embedded in an enlarged part of the drilled hole. The embedded anchorage has the advantage that it can be covered with a fire protection mortar and is not visible after the installation.

The design method presented in section 3 of this report refers to correctly installed punching shear reinforcement with Hilti Tension Anchors HZA. The appropriate installation equipment and procedure are described in section 2.2.

2.2 Installation

2.2.1 Detection and marking of the existing lower reinforcement

An area of at least 180cm x 180cm of the slab around the column is detected with the Ferroscan System PS 200 and the lower reinforcement is marked. Then, the pattern of the anchorages is marked.

Fig. 12: location of reinforcement with Ferroscan PS200

2.2.2 Drilling the Anchor Holes

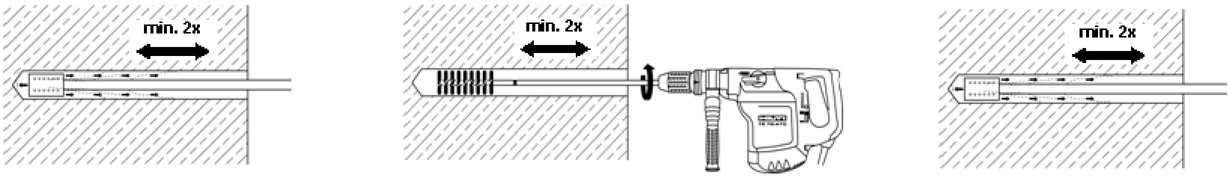
With HZA-P M16: Ø 22mm (TE-YX 22/92), extension of lower part with special drill bit TE-Y-GB 55/59
With HZA-P M20: Ø 25mm (TE-YX 25/92), extension of lower part with special drill bit TE-Y-GB 66/59
The holes and extensions are drilled with the rotary hammer TE 76 and the above drill bits and special drill bits. The holes are drilled under an angle of 45°. The direction is given by a rotation laser PR 25.

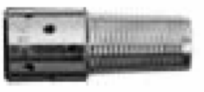
Fig. 13: drilling of the holes

2.2.3 Cleaning the Holes

with HZA-P M16: air nozzle HIT-DL 20, round steel brush HIT-RB 22
with HZA-P M20: air nozzle HIT-DL 25, round steel brush HIT-RB 25
Cleaning of the drilled hole is carried out in the following Steps:

1. blowing out of the hole with compressed air and an air nozzle HIT-DL twice
2. brushing of the hole with TE 76, holder, brush extension HIT-RBS and round steel brush HIT-RB
3. blowing out of the hole with compressed air and an air nozzle HIT-DL twice





HIT-DL 20 or HIT-DL 25, respectively

Cleaning set

Round steel brush HIT-RB	extension HIT-RBS 10/0.7	holder TE-Y	rotary hammer

Fig. 14: equipment for hole cleaning

2.2.4 Injection of the Drilled Hole with Injection Mortar

After cleaning the anchor hole is partially filled with the two-component injection mortar Hilti HIT-RE 500 or Hilti HIT-HY 150. Piston plugs HIT-SZ 22 or HIT SZ-25 are fixed to the mixer extension to ensure proper filling of the hole without air voids. The efficient installation of the anchors is supported by the use of the large cartridges VIC 1400ml and the compressed air injection tool HIT-P 8000-D.

Fig. 15: Injection tool HIT-P 8000-D

2.2.5 Installation of the Punching Shear Reinforcement

After injection of the mortar, the tension anchor HZA-P is manually installed into the drilled hole.

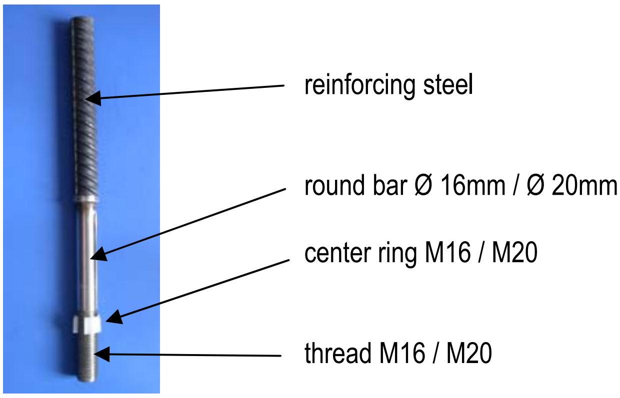


Fig. 16: Hilti tension anchor HZA-P

2.2.6 Installation of Anchor Head

After curing of the injection mortar HIT-RE 500 or HIT-HY 150, respectively, the anchor head is installed, i.e. the injection washer HIT M16, spherical washer C17 und nut M16 or injection washer HIT M20, spherical washer C21 und nut M20 are fixed to the thread. The installation torque moment of 100Nm(HZA-P M16) or 160Nm(HZA-P M20) is then applied.

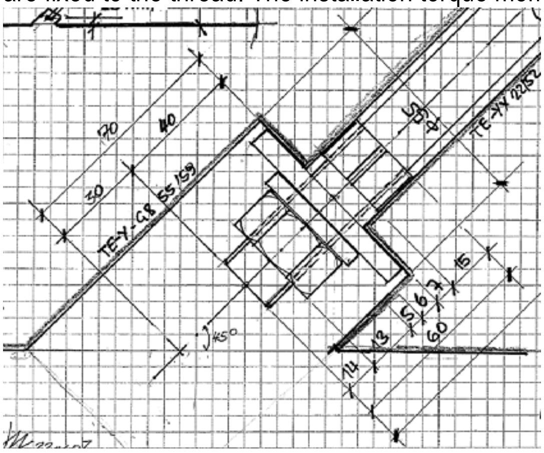


Fig. 17: anchor head and hole extension HZA-P M16

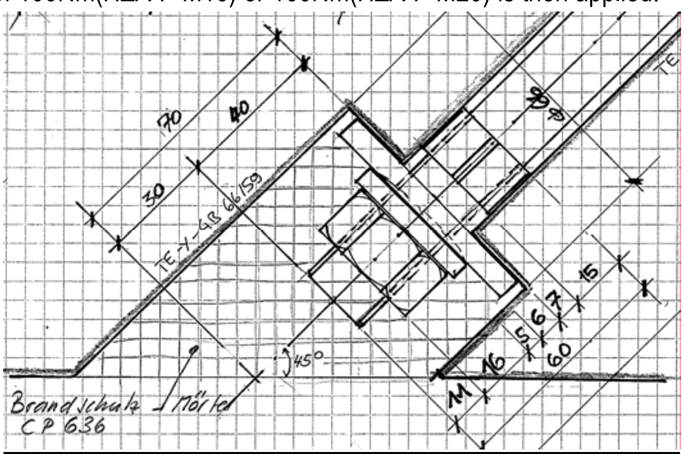


Fig 18: anchor head and hole extension HZA-P M20

2.2.7 Injection of the washer with HIT-RE 500

After application of the torque moment, the washer of the anchor head is injected with adhesive mortar HIT-RE 500.

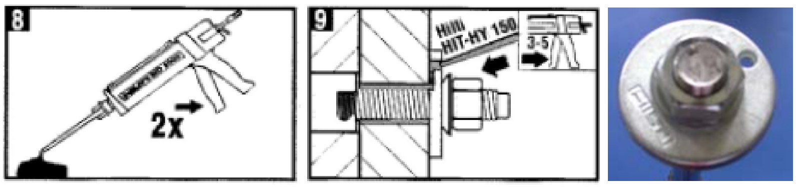


Fig. 19: injection of dynamic set washer

2.2.8 Filling of hole extension with fire protection mortar CP 636

The anchor head is covered with fire protection mortar CP 636.

Fig. 20: fire protection mortar CP 636

3 Design

3.1 General Aspects

The actual punching shear resistance of the non reinforced slab as calculated by the applicable structural concrete code, $V_{Rc,d}$, is the basis of the design. The upper limit of the design resistance of the reinforced slab, $V_{Rd,max}$, is usually also given by the codes in order to avoid crushing of the compressed concrete in the vicinity of the column. If the actual punching shear load, V_d, exceeds the puching shear resistance of the actual slab, $V_{Rc,d}$, but is smaller than the upper limit of the punching shear resistance, $V_{Rd,max}$, then the slab can be reinforced with Hilti tension anchors HZA-P. The design is based on tests which were carried out at the research laboratory of the Hilti Corporation and were scientifically evaluated at the ETH (Federal Institute of Technology) Lausanne in Switzerland. The following design rules are given for punching shear forces on a given column. The calculation of the concrete shear strength and of the critical perimiters is defined in different ways in the different structural concrete codes and will not be reproduced in detail in this brochure.

3.2 Evaluation of the Load Taken up by the Reinforcement

The punching resistance of a concrete slab with shear reinforcement is in most codes given as a combination of the concrete and the steel resistance. The combined steel and concrete resistance is often smaller than the direct addition of both resistances. One (or both) component is usually reduced and the general formula may be written as

$$V_{Rd} = a \cdot V_{Rd,c} + b \cdot A_{sw} \cdot f_{yd} \quad (1)$$

With
- V_{Rd} punching shear resistance of the slab with reinforcement
- $V_{Rd,c}$ punching shear resistance of the slab without reinforcement
- A_{sw} area of shear reinforcement in the direction of the punching shear force
- f_{yd} design strength of the shear reinforcement
- $V_{Rd,s}$ punching shear resistance of the shear reinforcement ($V_{Rd,s}=A_{sw}\cdot f_{yd}$)
- a reduction factor for the concrete strength ($a\leq 1.0$)
- b reduction factor for the steel strength ($b\leq 1.0$)

As the design requires the acting shear load to be inferior to the resistance ($V_d \leq V_{Rd}$) the load to be taken up by the shear reinforcement, $V_{Rd,s,req}$, according to formula (1) can be written as

$$V_{Rd,s,req} \geq \frac{V_d - a \cdot V_{Rd,c}}{b} \quad (2)$$

With post-installed shear reinforcement, the parameters a and b may be inferior to those given in the structural concrete code for cast-in-place shear reinforcement for two reasons: The quality of the anchorage of the post-installed shear reinforcement is different to that of cast-in-place elements and as the approaches vary strongly in different codes, the design concept has been calibrated to the safe side. In the following sub-sections, the approaches of different codes are briefly summarized.

3.2.1 Eurocode 2 (EN 1992-1-1:2004)

According to code: $V_{Rd,sy} = 0.75 \cdot V_{Rd,c} + A_{sw} \cdot f_{ydwd,ef}$; $f_{ywd,ef} = 250 + 0.25d \leq f_{ywd}$ (3)

→ For post- installed punching shear reinforcement with HZA-P: $V_{Rd,s,req} \geq \frac{(V_d - 0.75 \cdot V_{Rd,c}) \cdot f_{yd}}{\min(250 + 0.25d; f_{yd})}$ (4)

3.2.2 American Concrete Institute ACI 318M-05 (metric units)

→ For post- installed punching shear reinforcement with HZA-P: $V_{Rd,s,req} \geq V_d - 0.5 \cdot V_{Rd,c}$ (5)

3.2.3 British Standard BS 8110 (1997)

→ For post- installed punching shear reinforcement with HZA-P: $V_{Rd,s,req} \geq V_d - 0.6 \cdot V_{Rd,c}$ (6)

3.2.4 DIN 1045-1

Code formulation: $V_{Rd,sy} = V_{Rd,c} + \kappa_s \cdot A_{sw} \cdot f_{yd}$ (7)

With κ_s efficiency factor = 0.7+0.3*(d-400)/400

→ For post- installed punching shear reinforcement with HZA-P: $V_{sd,req} = \dfrac{V_d - 0.9 \cdot V_{Rd,c}}{0.7 + 0.3 \cdot (d - 400)/400}$ (8)

3.2.5 öNorm B 4700

Code formulation: $V_{Rd,sy} = V_{Rd,c} + \kappa_s \cdot A_{sw} \cdot f_{yd}$ (9)

With κ_s efficiency factor = 0.5 for standard shear reinforcement

→ For post- installed punching shear reinforcement with HZA-P: $V_{sd,req} = \dfrac{V_d - V_{Rd,c}}{0.5}$ (10)

3.2.6 Singapore Standard CP 65

→ For post- installed punching shear reinforcement with HZA-P: $V_{sd,req} = V_d - 0.55 \cdot V_{Rd,c}$ (11)

3.2.7 SIA 262

→ For post- installed punching shear reinforcement with HZA-P: $V_{sd,req} = V_d - V_{Rd,c}$ (12)

Where $V_{Rd,c}$ is the shear resistance of the non reinforced slab at the effective shear load V_d. According to the failure criterion, this value is inferior to the real shear resistance of the non reinforced slab. Strictly according to the code, normal shear reinforcement has to take up the entire shear load, but evaluation of the tests on slabs reinforced with Hilti HZA-P has shown the validity of the above formulation [7].

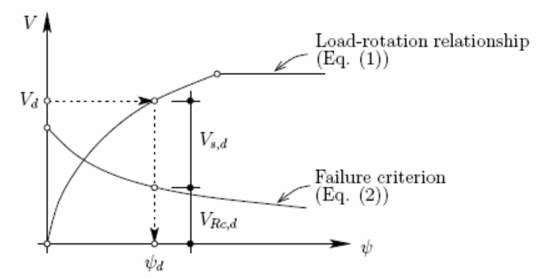


Fig. 21: concrete and steel resistance according in SIA 262

3.3 Design of the Reinforcement with HZA-P

The shear reinforcement is designed satisfying the following condition:

$$V_{s,d} \leq \sum_{i=1}^{n} N_{si,d} \cdot \sin \beta_i \qquad (13)$$

where $N_{si,d}$ ist e factored strength of the shear reinforcement and β_i ist he angle of the shear reinforcement.

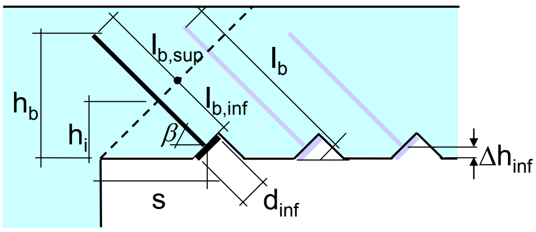


Fig. 22: geometry of reinforcement

The factored strength of the Hilti Tension Anchor HZA-P ($N_{si,d}$) is equal to the minimum of the following values:

$$N_{si,d} = \min(N_{si,el,d}\,; N_{si,pl,d}\,; N_{si,b,d}\,; N_{si,p,d}) \qquad (14)$$

where $N_{si,el,d}$ ist he force in the shear reinforcement that can be activated asssuming an elastic behaviour of the bar. This value, accounting fort he rotation of the slab at SLS (see fig. 23) results:

$$N_{si,el,d} = K_{ai} \cdot \sqrt{\Delta\psi_d \cdot h_i \cdot \sin(\alpha + \beta_i)} \qquad (15)$$

Where α ist he angle of the sritical shear crack (normally set to 45°). In the standard case of reinforcements set under β_i=45° the value of $\sin(\alpha+\beta_i)=1.0$. h_i is the height up to which the reinforcement is assumed to be anchored(Fig. 22). $\Delta\psi_d$ is the decisive rotation of the structure to be reinforced: $\Delta\psi_d = \psi_d - \psi_{SLS}$.

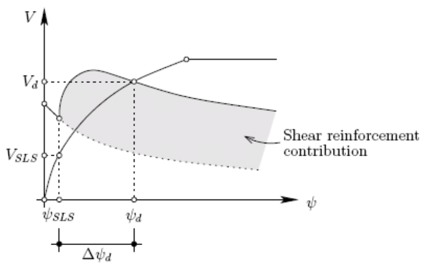


Fig. 23: Definition of $\Delta\psi_d$

$$\psi = \frac{0.000711 \cdot \ell}{d} \cdot \left(\frac{m_0}{m_{Rd}}\right)^{3/2} \qquad (16)$$

ℓ is the span, d the static height of the structure to be reinforced and m_{Rd} the design value of the bending resistance. m_0 is a reference value. For more or less regularly spaced columns (long distance / short distance ≤ 2), the following values can be used:

- inner columns	m_0=V/8	
- border columns	m_0=V/4	(reinforcement parallel to border)
- border columns	m_0=V/8	(upper and lower reinforcement perpendicular to border)
- corner columns	m_0=V/2	

Thus, the decisive rotation for e.g. an inner column can be calculated as

$$\Delta\psi_d = \frac{0.000711 \cdot \ell}{d} \cdot \left(\frac{V_d - V_{SLS}}{8 \cdot m_{Rd}}\right)^{3/2} \qquad (17)$$

K_a is a coefficient depending on the anchorage and is given in the following table:

	HZA-P M16	HZA-P M20
Hilti HIT-RE 500	$K_{ai}\left[MN/m^{0.5}\right]=2.68\sqrt{1+\frac{f_{cc,k}\left[N/mm2\right]-25}{100}}$	$K_{ai}\left[MN/m^{0.5}\right]=3.75\sqrt{1+\frac{f_{cc,k}\left[N/mm2\right]-25}{100}}$
Hilti HIT-HY 150	$K_{ai}\left[MN/m^{0.5}\right]=1.79\sqrt{1+\frac{f_{cc,k}\left[N/mm2\right]-25}{100}}$	$K_{ai}\left[MN/m^{0.5}\right]=2.39\sqrt{1+\frac{f_{cc,k}\left[N/mm2\right]-25}{100}}$

$N_{si,pl,d}$ is the plastic resistance of the reinforcement bar, it's value is:

$$N_{si,pl,d} = A_{si} \cdot f_{yd}$$

$N_{si,b,d}$ is the upper limit of the resistance due to the bond strength. It is assumed that the bar is bonded between the point where it cuts the shear crack and its upper end ($\ell_{b,sup,i}$ see Fig. 22).

$$N_{si,b,d} = \tau_{bd} \cdot d_b \cdot \pi \cdot \ell_{b,sup,i}$$

The design value of the bond strength is evaluated as $\tau_{bd} = \tau_{bd}^0 \cdot f_{B,N}$. τ_{bd}^0 ist he design strength in a concrete of class C20/25 and $f_{B,N}$ takes into account the effective concrete strength. The values are given in the following table.

	Hilti HIT-RE 500	Hilti HIT-HY 150
Bond strength: $\tau_{bd,0}$ =	6.95 MPa	3.11 MPa (HZA-P M16) 2.83 MPa (HZA-P M20)
Influence of concrete strength: $f_{B,N}$ =	$1+\frac{f_{cc,k}-25}{100}$	$1+\frac{f_{cc,k}-25}{212.5}$
	25 MPa ≤ $f_{cc,k}$ ≤ 60MPa	

$N_{si,p,d}$ is the resistance against pullout (by concrete cone failure) of the lower anchorage (Fig. 22):

$$N_{si,p,d} = A_{si} \cdot \frac{0.360}{\gamma_c} \cdot \sqrt{f_{ck}} \cdot \frac{\ell_{b,\inf,i}^{1.5}}{d_{bi}^2}\left(1+\frac{d_{\inf,i}}{\ell_{b,\inf,i}}\right)$$

$\ell_{b,inf,i}$ is the distance between the point where the reinforcement bar intersects the critical shear crack and its lower anchorage plate; $d_{inf,i}$ is the diameter of the lower anchorage plate. It should be noted that this formula is dimension-dependent and SI units should be introduced [MN, m].

3.4 Concrete failure close to column

It should be checked that concrete failure close to the column is not critical. Most codes do so by limiting the maximum shear resistance:

$V_d \leq V_{Rd,max}$

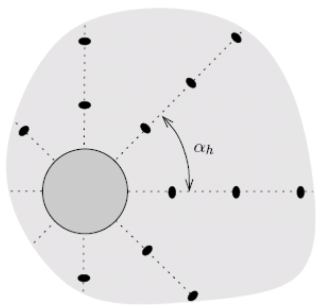
3.5 Punching outside the reinforced area

The size of the reinforced area must be sufficient, so that the punching shear resistance outside the reinforced zone is inferior to the acting shear force on the column minus those forces acting inside the reinforced area. The punching shear resistance outside of the reinforced area is evaluated according to the applicable structural concrete code. It should be noted that the statical height d' is reduced if the lower anchorage is inside the plate for fire protection or esthetic reasons (see fig. 22)

3.6 Rules for good detailing

In order to obtain a good detailing, the following constructive rules should be followed when desinging punching shear reinforcement with Hilti tension anchors HZA-P:

3.6.1 Number of radii



The Hilti Tension Anchors HZA-P are placed along a series of radii where the angle between tem has to be lower or equal than 45°:

$\alpha_h \leq 45°$

Fig 24: angle between radii

3.6.2 Number of reinforcements in a radius

At least, two Hilti Tension Anchors HZA-P should be placed at each radius.

3.6.3 Distance between reinforcements and column

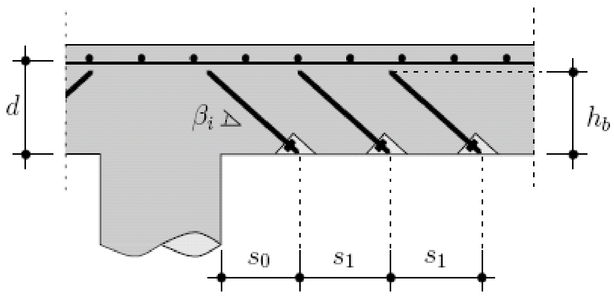


Fig. 25: distance between reinforcements

The distance of the first anchorage to hte border of the column should be lower than or equal to 0.75d:

$s_0 \leq 0.75d$

If a very small value of s_0 is selected, then the capacity of the first reinforcement bar may be strongly reduced. The presented design concept takes this into account. Moreover a small distance s0 may lead to difficulties if there is dense column reinforcement

3.6.4 Radial distance between reinforcements

The distance between two anchorages in a radius should be lower than or equal to 0.75d:

$s_1 \leq 0.75d$

3.6.5 Direction of the drilled holes

The direction of the drilled holes should be at an angle of 45° compared to the slab surface and towards the column:

$\beta_i = 45°$

3.6.6 Length of the drilled holes

The height at which a Hilti Tension Anchor HZA-P should be bonded (h_b) is equal to d:

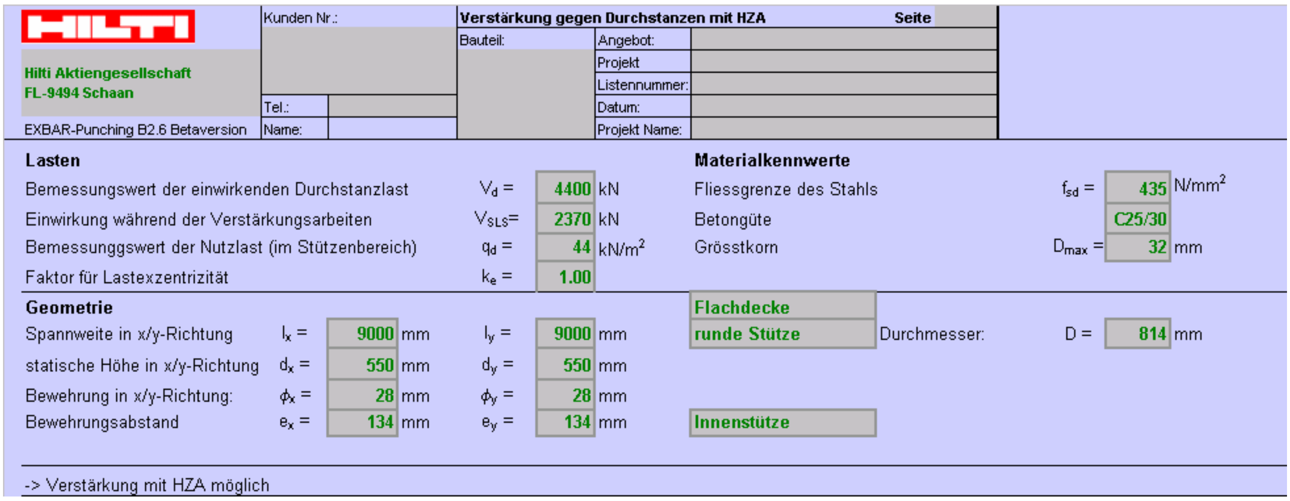
$h_b = d$

In cases where tensile reinforcement is intersected when the slab is being drilled, the bonded height (h_b) can be reduced in order not to cut the ensile reinforcement. The estimate of thestrength of the system should be performed with a value of h_b that accounts for this possibility.

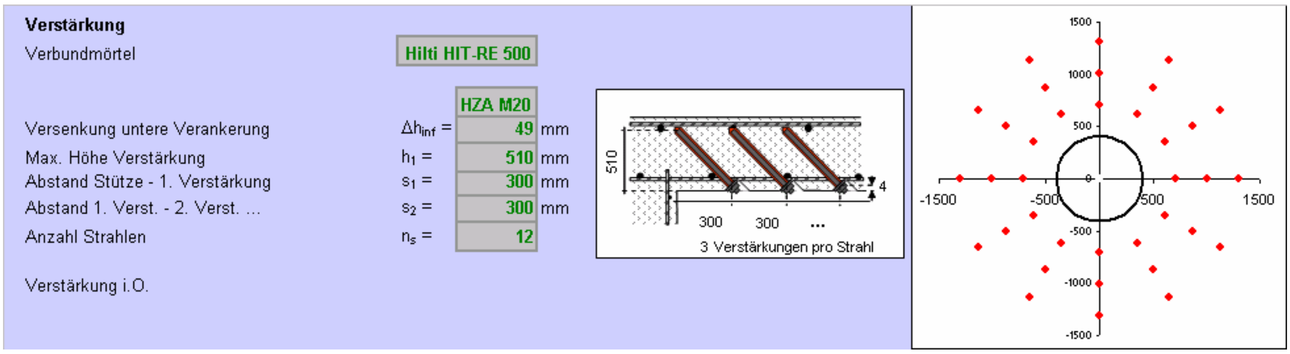
4 Exbar Punching Design Software

4.1 Introduction

EXBAR punching is the design software for the strengthening of structural parts against punching shear with Hilti Tension Anchors HZA-P. It carries out the design according to section 3. The resistance of the non-reinforced structural part, the maximum possible punching resistance (failure of compressed concrete at limit of column) and the punching shear resistance outside of the reinforced area are calculated according to the selected structural concrete code (at the time being ACI 318, BS 8110 and SIA 262 are available). The resistance of the reinforcement is calculated according to the formulae of section 3.3. The user enters all necessary data on the entry screen:



Based on the evaluation of the punching shear resistance of the non-reinforced structural part and on the maximum possible punching shear resistance of the reinforced part, the user is informed, whether reinforcing with Hilti HZA-P is possible. If this is the case, the user can enter data concerning the adhesive mortar, the type of reinforcing bars, the embedment of the lower anchorage Δh_{inf}, the height over which the bars are anchored h_b, the distance between the first anchorage and the column edge s_1, the radial distance between two reinforcements s_2 and the number of radii n_s.



When entering the above data, the user is constantly informed whether the selected reinforcing arrangement is sufficient or not. The number of reinforcements in one radius is automatically selected in such a way that proof of the punching shear resistance outside of the reinforced area can be performed.

Once the user has selected a satisfying reinforcement arrangement, he finds all the necessary design proofs on a separate screen which can be printed and added to a statical calculation document.

5 Examples

5.1 Strengthening of a ceiling (ACI 318)

given

Factored load on column	V_u =	4400	kN	Load on slab	q_u =	44	kN/m^2
Last bei Verstärkung	V_{SLS} =	2370	kN				
yield strength steel	f_{sy} =	435	N/mm^2				
concrete strength	f'_c =	25	N/mm^2				
diameter reinforcement	$d_{b,x}$ =	28	mm	static height	d_x =	550	mm
	$d_{b,y}$ =	28	mm		d_y =	550	mm
Bewehrungsabstand:	e_x =	134	mm	reinforcement ratio	ρ_x =	0.84	%
	e_y =	134	mm		ρ_y =	0.84	%
circular column	D =	814	mm				
interior column							
spans	ℓ_x =	9000	mm	ℓ_y =	9000	mm	

punching shear resistance without reinforcement

area inside punching cone	A_i =		1.46	m^2
effective punching shear load	V'_u =	$V_u - q_u*A_i$ =	4336	kN
perimeter of critical section	b_0 =		4285	N/mm^2
average static height	d =	$(d_x + d_y) / 2$ =	550	mm
ratio of long side / short side of column	β_c =		1	
ACI 318M (11-35)	V_{ca} =	$(1+2/\beta_c)*f'_c$^0.5*b_0*d/6 =	5892	kN
column location: interior column →	α_s =		40	
ACI 318M (11-36)	V_{cb} =	$(\alpha_s d/b_0+2)*f'_c$^0.5*b_0*d/12 =	7006	kN
ACI 318M (11-37)	V_{cc} =	1/3*f'_c^0.5*b_0*d =	3928	kN
punching resistance w/o reinforcement	V_c =	$\min(V_{ca};V_{cb};V_{cc})$ =	3928	kN
maximum punching resistance	$V_{c,max}$ =	f'_c^0.5*b_0*d/2 =	5892	kN

V'_u > V_c → strength of slab not sufficient

V'_u ≤ $V_{c,max}$ → reinforcement possible with Hilti HZA-P

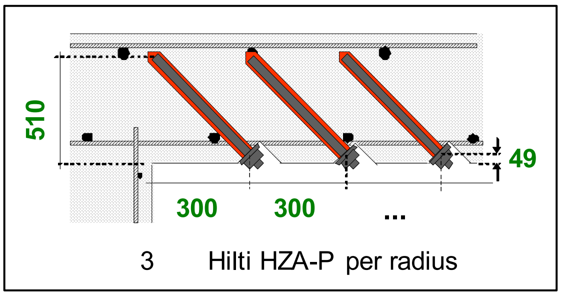
bending resistance of concrete slab

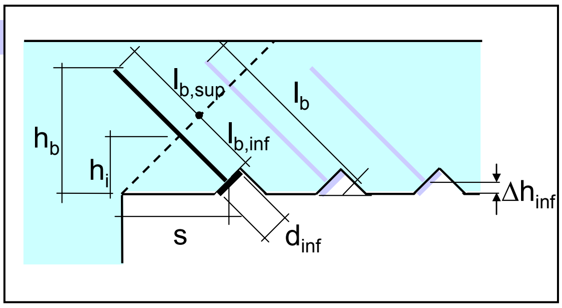
average reinforcement ratio	ρ =	(rx*ry)^0.5 =	0.0084	
inner lever arm	z =	d-0.416*ρ*d*f_{sd}/(0.7*f'_c) =	502	mm
resisting moment	m_u =	ρ*d*f_{sy}*z =	1004	kNm/m

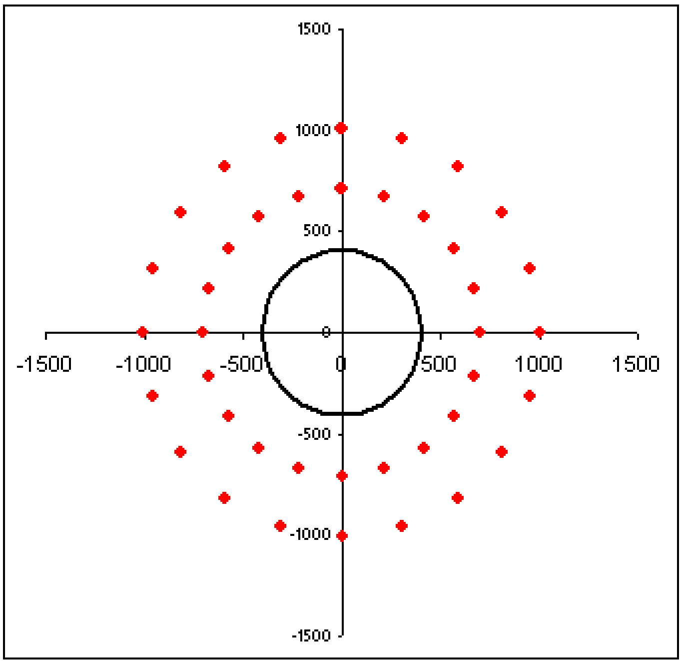
selected reinforcement

adhesive mortar	Hilti HIT-RE 500	drilling depth from conc. Surface	l_b =	721 mm
reinforcement rods	Hilti HZA-P M20	embedment of anchorage	Δh_{inf} =	49 mm
number of radii	n_s = 20	distance column - 1st anchorage	s_1 =	300 mm
number of rods per radius	n_v = 2	distance between anchorages	s_2 =	300 mm

drawings







design proof for the selected reinforcement

reference V_0: interior column →	$V_0^{1.5}$ =	$(V_u^{1.5}-V_{SLS}^{1.5})$ / 22.627	7800	$kN^{1.5}$
factor for deformation	$\Delta\Psi_d$ =	$0.000711*(\ell_x+\ell_y)/2/d*[V_0^{1.5}/m_u^{1.5}]$ =	0.0029	
bond strength	τ_{bd} =	(Hilti Fastening Technology Manual)	7.30	N/mm^2
anhcorage factor	k_a =	Hilti HZA-P M20 Hilti HIT-RE 500 →	3.84	$MN/m^{0.5}$
diameter reinforcement rod	d_b =	Hilti HZA-P M20 →	20	mm
diameter anchor plate	d_{inf} =	Hilti HZA-P M20 →	60	mm
max. depth of anchorage	h_b =	$l_b/\sqrt{2}$ =	510	mm

rod	s [mm]	h_i [mm]	$l_{b,inf}$ [mm]	$l_{b,sup}$ [mm]	$N_{si,pl,d}$ [kN]	$N_{si,el,d}$ [kN]	$N_{si,b,d}$ [kN]	$N_{si,p,d}$ [kN]	$N_{si,d}$ [kN]
1	300	150	143	509	137	79.5	233.4	71.5	71.5
2	600	300	355	297	137	112.4	136.2	230.8	112.4

resistance per radius in direction of bars $N_{Rs,d,s}$ = 183.9 kN

res./radius in dir. of shear load $V_{Rs,d,s} = N_{Rs,d,s}/\sqrt{2}$ = 130.1 kN

used formulae

$N_{si,pl,d} = d_b^2*\pi/4*f_{sy}$ $\quad$ $N_{si,b,d} = \pi*d_b*l_{sup}*\tau_{bd}$

$N_{si,el,d} = k_{ai}*(\Delta\Psi_d*h_i)^{0.5}$ $\quad$ $N_{si,p,d} = 0.28/\gamma_c*f_{ck}^{0.5}*l_{b,inf}^{1.5}*(1+d_{inf}/l_{b,inf})$ $\quad$ [N/mm^2, m, MN]

load carried by shear reinforcement	V_s =	$V_{Rs,d,s}*n_s$ =	2601	kN
load carried by concrete	V_c =	$\min(V_{ca};V_{cb};f'_c{}^{0.5}*b_0*(d-\Delta h_{inf})/6)$ =	1789	kN
total puncing shear resistance	V_n =	V_s+V_c =	4390	kN

V'_u ≤ V_n → ok

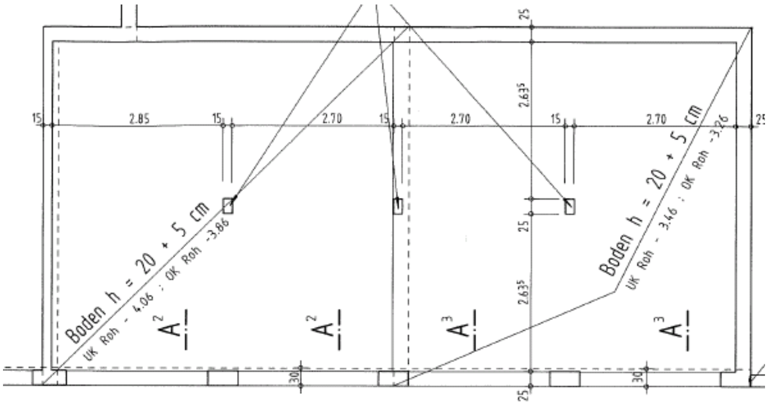
proof of shear resistance outside the reinforced area

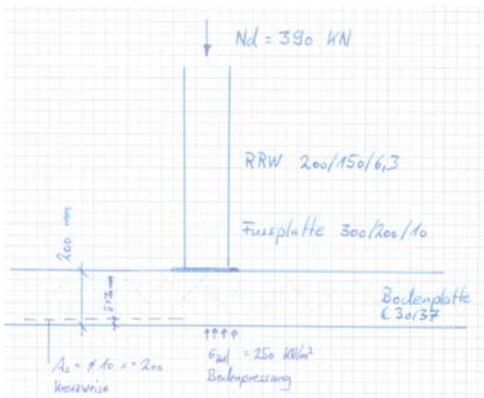
reduced static height	$d' =$	$d-\Delta h_{inf} =$	501	mm
perimeter of critical section	$b_{ext} =$	π (D+dv+2*[s1+(nv-1)*s2]) =	9786	mm
area inside b_{ext}	$A_a =$	$b_{ext}^2/(4*\pi) =$	7.62	m2
punching shear load outside b_{ext}	$V''_u =$	$V_u - A_a*q_u \geq 0 =$	4065	kN
ratio of long side / short side of column	$\beta_c =$		1	
ACI 318M (11-35)	$V_{ca} =$	$(1+2/\beta_c)*f'_c$^0.5*b_{ext}*d'/6 =	12257	kN
column location: interior column →	$\alpha_s =$		40	
ACI 318M (11-36)	$V_{cb} =$	$(\alpha_s d'/b_{ext}+2)*f'_c$^0.5*$b_{ext}$*d'/12 =	8269	kN
ACI 318M (11-37)	$V_{cc} =$	1/3*f'_c^0.5*b_{ext}*d' =	8171	kN
punching resistance outside reinforcement	$V_c =$	$\min(V_{ca};V_{cb};V_{cc}) =$	8171	kN

V'_u ≤ V_c → ok

Additional reinforcement rods should be applied if the radial distance between anchorage points exceeds twice the static height (s_{rad} > 2d).

5.2 Strengthening of a floor (SIA 262)





Design proof from EXBAR-Punching

Hilti Aktiengesellschaft FL-9494 Schaan	Kunden Nr.:	Verstärkung gegen Durchstanzen mit HZA-P		Seite 1
		Bauteil:	Angebot:	
			Projekt	
			Listennummer:	
	Tel.:		Datum:	
EXBAR-Punching B2.6 Betaversion	Name:		Projekt Name:	

Gegeben

Bemessungslast	V_d =	390 kN	Nutzlast bei Stütze	q_d =	250 kN/m²
Last bei Verstärkung	V_{SLS} =	100 kN	Faktor Lastexzentrizität	k_e =	1.00
Fliessgrenze Stahl	f_{sy} =	435 N/mm²	Elastizitätsmodul Stahl	E_s =	205000 N/mm²
Betonqualität		C25/30	Durchm. Grösstkorn	D_{max} =	32 mm
Durchmesser Bewehrung	ϕ_x =	10 mm	statische Höhe	d_x =	170 mm
	ϕ_y =	10 mm		d_y =	170 mm
Bewehrungsabstand:	e_x =	200 mm	Bewehrungsgrad	ρ_x =	0.23 %
	e_y =	200 mm		ρ_y =	0.23 %
rechteckige Stütze	a =	300 mm	b =	200 mm	
Innenstütze					
Spannweiten:	l_x =	2700 mm	l_y =	2700 mm	

Durchstanzwiderstand ohne Verstärkung

Beton: charakteristische Druckfestigkeit	C25/30 ->	f_{ck} =		25	N/mm^2
Bemessungsdruckfestigkeit	->	f_{cd} =		16.5	N/mm^2
Schubspannungsgrenze	->	τ_{cd} =	$0.3/\gamma_c * f_{ck}^{0.5}$ =	1	N/mm^2
mittlere statische Höhe		d =	$(d_x + d_y) / 2$ =	170	mm
Nachweisschnitt		u =	π d +2a + 2b =	1534.1	mm
Fläche im Nachweisschnitt		A_i =	a*b + d(a+b) + dπ =	0.1677	m^2
Last ausserhalb Durchstanzbereich		V'_d =	$V_d - A_i * q_d$ =	348	kN
Vergleichsmoment		m_{0d} =	$V'_d / 8$	44	kN
Biegewiderstand y-Richtung		m_{Rdx} =	$r_x * d_x^2 * f_{sd} * [1-0.416 * \rho_x * f_{sd} / (0.81 * f_{cd})]$ =	28.1	kNm/m
Biegewiderstand x-Richtung		m_{Rdy} =	$r_y * d_y^2 * f_{sd} * [1-0.416 * \rho_y * f_{sd} / (0.81 * f_{cd})]$ =	28.1	kNm/m
Einfluss Grösstkorn		k_{Dmax} =	$48/(D_{max}+16)$ =	1	
plastifizierte Zone im Bruchzustand x		$r_{y,x}$ =	$0.15 l_x * (m_{0d}/m_{Rdx})^{1.5}$ =	0.778	m
plastifizierte Zone im Bruchzustand y		$r_{y,y}$ =	$0.15 l_y * (m_{0d}/m_{Rdy})^{1.5}$ =	0.778	m
massgebende plastifizierte Zone		r_y =	$MAX[r_{y,x}; r_{y,y}]$ =	0.778	m
Beiwert Bauteilgrösse / Biegewiderstand		k_r =	$1/(0.45+0.9 * r_y) \geq 1/(1+2.2d)$ =	0.87	
Verformung im Bruchzustand		Ψ_d =	$0.00474 * r_y / d$ =	0.0217	
Durchstanzwiderstand bei V'_d		$V_{Rc,d}$ =	$k_r * \tau_{cd} * d * u$ =	227	kN

V'_d > V_{Rd} → Durchstanzwiderstand ungenügend

max. Durchstanzwiderstand unverstärkt	$V_{Rd,c}$ =	(iterative Berechnung)	276	kN
max. Durchstanzwiderstand (mit $2k_r$)	$V_{Rd,max}$ =	(iterative Berechnung)	399	kN

V'_d ≤ $V_{Rd,max}$ → Verstärkung mit Hilti HZA-P möglich

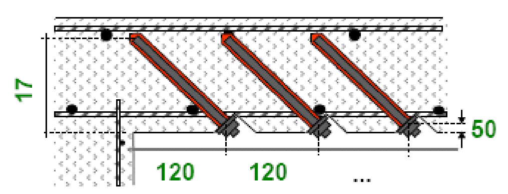
Eingabe und Resultate müssen auf Uebereinstimmung mit den Gegebenheiten auf der Baustelle und auf Plausibilität überprüft werden

Hilti Aktiengesellschaft FL-9494 Schaan	Kunden Nr.:	Verstärkung gegen Durchstanzen mit HZA-P	Seite 2
		Bauteil:	Angebot:
			Projekt
			Listennummer:
	Tel.:		Datum:
EXBAR-Punching B2.6 Betaversion	Name:		Projekt Name:

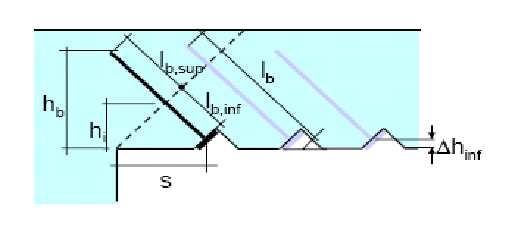
Gewählte Verstärkung

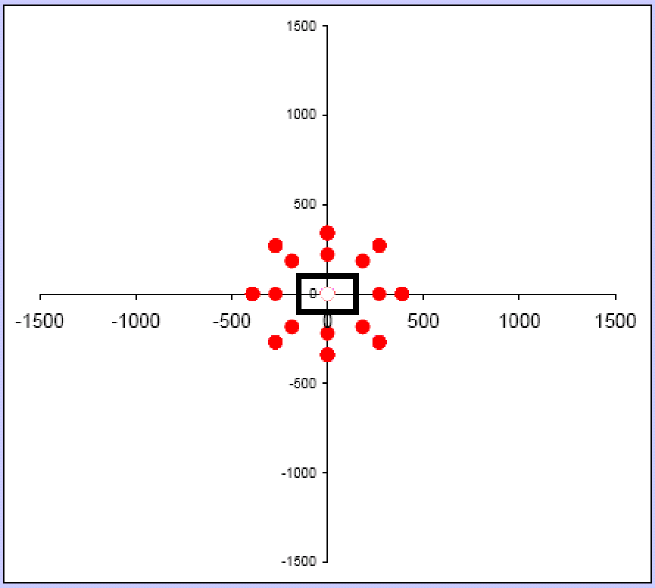
Verbundmörtel	Hilti HIT-RE 500		Bohrtiefe von Betonoberfläche	l_b =	240	mm
Verstärkungsstäbe	Hilti HZA-P M20		Einsenkung der Verankerung	Δh_{inf} =	50	mm
Anzahl Strahlen	n_s =	8	Abstand Stütze - 1. Verstärkung	s_1 =	120	mm
Anzahl Stäbe pro Strahl	n_v =	2	Abstand zwischen Stäben	s_2 =	120	mm

Skizzen



2Hilti HZA-P M20 pro Strahl





Nachweis der gewählten Verstärkung

plastifizierte Zone während Verstärkung	$r_{y,SLS}$ =	$r_y*(V_{SLS}/V'_d)^{1.5}$ =	0.1199 m
Verformung während Verstärkung	Ψ_{SLS} =	$0.00474*r_{y,SLS}/d$ =	0.0033
Verbundfestigkeit	τ_{bd} =	(Hilti Handbuch Befestigungstechnik)	7.3 N/mm^2
Verankerungsfaktor	k_a =	Hilti HZA-P M20 Hilti HIT-RE 500 →	0.73
Durchmesser Verankerungsstab	d_b =	Hilti HZA-P M20 →	20 mm
Durchmesser Verankerungsscheibe	d_{inf} =	Hilti HZA-P M20 →	60 mm
max. Tiefe der Verankerung	h_b =	$l_b/2^{0.5}$ =	170 mm

Stab	s	h_i	$l_{b,inf}$	$l_{b,sup}$	σ_{sd}	$A*\sigma_{sd}$	$N_{R,sup,d}$	$N_{R,inf,d}$	$N_{Rs,d}$
	[mm]	[mm]	[mm]	[mm]	[N/mm²]	[kN]	[kN]	[kN]	[kN]
1	120	60	14	156	405.98	127.5	71.3	8.2	8.2
2	240	120	99	71	435	136.7	32.4	46.7	32.4

Widerstand Verstärkung pro Strahl $V_{Rs,d,s}$ = 40.7

verwendete Formeln:

aktivierte Spannung	σ_{sd} =	$[k_a*E_s*(\Psi_d-\Psi_{SLS})*h_i]^{0.5} \leq f_{sd}$	
Verankerungskraft oben	$N_{R,sup}$ =	$\pi*d_b*l_{sup}*\tau_{bd}$	
Verankerungskraft unten	$N_{R,inf}$ =	$0.28/\gamma_c*f_{ck}^{0.5}*l_{b,inf}^{1.5}*(1+d_{inf}/l_{b,inf})$	[N/mm², m, MN]

Traganteil Durchstanzverstärkung	$V_{Rs,d}$ =	$V_{Rs,d,s}*n_s$ =	325 kN
Durchstanzwiderstand nach Verstärkung	$V_{R,d}$ =	$V_{Rs,d}+V_{Rc,d} \leq V_{Rd,max}$ =	399 kN

$V_{Rd} > V'_d$ → i.O.

Eingabe und Resultate müssen auf Uebereinstimmung mit den Gegebenheiten auf der Baustelle und auf Plausibilität überprüft werden

B 2.6 V Beta 05/2007

6 Test Results

Hilti has performed tests where shear reinforcement HZA-P was bonded into drilled holes inclined towards the column. This is a continuation of a system that has been investigated at the Royal Institute of Technology KTH in Stockholm in 1995 [6].

It is important that the drilled holes proceed up to at least just below the tensile reinforcement of the slab. As the anchorage quality has a strong influence on the efficiency of shear reinforcement, the reinforcing bars were anchored at the bottom of the bar with an anchorage plate and a nut. In a first step beam tests have shown that the number of reinforciement bars and the characteristics of the used adhesive mortar have the strongest influence on the result.

Slab tests carried out subsequently have shown increases of resistance up to the theoretically possible maximum punching shear resistance. The results of these tests were incorporated into a consistent design concept by Professor A. Muttoni at the Swiss Federal Institute of Technology (ETH) in Lausanne.

In addition to the increase in resistance, slabs reinforced with Hilti Tension Anchors HZA-P also provide a significantly increased deformation capacity. The failure is definitely less brittle than that of non-reinforced slabs. Figure 27 shows the comparison of two tests with a relatively high tension reinforcement ratio. The non-reinforced slab failed at a load of abot 1000kN in a very brittle way. On the other hand, the reinforced slab failed outside of the reinforced area at about 1600kN after a clear plastic deformation. This corresponds to an increase of load capacity of 60% and to a doubling of the deformation capacity. Due to the increased deformation capacity, loads can be redistributed to neighbouring columns in case of overloading, which increases the safety of the overall structure.

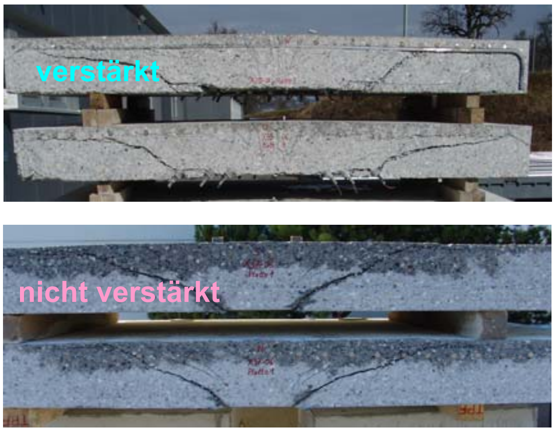


Fig. 26: failure patterns

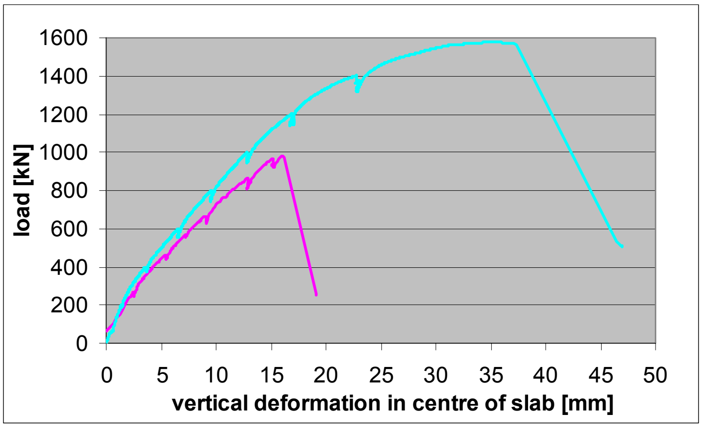


Fig. 27: Load-displacements curves

7 References

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