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Baseplate Rigidity and Anchorage Design

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Abstract

The anchorage design with baseplates according to EN 1992-4 [1] and ACI 318 [2] postulates that the baseplates need to be sufficiently rigid, because the linear distribution of anchor forces is required for calculating the concrete failure resistance of anchor groups.

With the precondition that the baseplate is sufficiently rigid, a linear strain distribution under the baseplate is assumed in Eurocode [1] for calculating the anchor tension forces in the anchor group. But there are no workable provisions in [1] to check the required baseplate rigidity.

The research results about the stiffness criteria for rigid baseplate assumption in [3,4] confirm the statements in [5,6] that the linear strain assumption is unrealistic for normal baseplate thicknesses. That means, with normal baseplate thicknesses in the practice, the non-linear anchor tension force distribution will take place and has to be considered for the anchorage design. In this case, additional proofs to [1, 2] may be necessary in verifying the concrete failure resistance of anchor groups [3,4].

For anchor groups with narrow baseplate under shear load perpendicular to the long axis, there may be similar stiffness requirements on baseplate as for tension load.

In this paper, the rigid baseplate assumption for anchorage design is examined by calculation examples. The required thickness of baseplate to satisfy the linear strain assumption is studied by variation of tension load and bending moment. The proposed additional proof of anchor group resistance with non-linear anchor tension force distribution [3,4] is extensively verified by 40 test results. This additional proof is illustrated by design examples.

A realistic calculation model for baseplate under shear load is presented. A stiffness criterium for baseplate under shear load is proposed based on the calculation results of this model.

Keywords: anchorage design, baseplate rigidity, anchor stiffness, tension, shear, concrete failure

1. Introduction

For anchorage design according to EN1992-4 [1] and ACI 318 [2], the baseplates need to be sufficiently rigid under tension and shear loads respectively on the fixture.

For baseplates under tension load and bending moment, the EN1992-4 postulates that the baseplates should have the rigidity level to result in a linear anchor tension force distribution as in beam bending where "plane sections remain plane". In calculating the concrete cone failure resistance of anchor groups, the coefficient to take account of the adverse effect of anchor force eccentricity is based on the linear distribution of anchor forces calculated using the above rigid baseplate assumption. However, apart from a guiding principle for dealing with cases where the rigid baseplate assumption does not hold, there are no workable provisions in EN1992-4 to classify the baseplate rigidity.

Based on the realistic 3D Finite Element Analysis (FEA), a stiffness criterion is proposed in [3, 4] to evaluate the baseplate rigidity under tension and bending moment. But there are always arguments about the linear strain assumption according to EN1992-4 and the understandable rigid baseplate assumption.

The research results [3, 4, 5, 6] indicate that the linear strain assumption is practically unrealistic. In general cases, the anchor tension force distribution is non-linear. But the design methods according EN 1992-4 and ACI 318 do not cover the non-linear anchor tension force distribution under the baseplates.

The rigidity requirements on baseplate under shear load has not been investigated until now.

This paper presents calculation examples to analyze and examine the following topics relating to the aforementioned linear anchor force distribution assumption:

- Relevant anchor tension stiffness for elastic design of anchorages.

- The differences between the linear strain assumption and the rigid baseplate assumption

- Main influence parameters on baseplate bending rigidity.

- Verification of the design method proposed in [3, 4] for anchor groups with non-linear anchor tension force distribution by test results.

- Stiffness condition for baseplate under shear load.

2. Studies on rigid baseplate assumption

2.1 Baseplate subjected to tension load and bending moment on the fixture

2.1.1 Simulation of steel-to-concrete connections with baseplate-

An anchorage system with baseplate on concrete connected by attached profile and anchors (Fig. 1) can be simulated realistically by elastic analysis with the following Finite Element Analysis (FEA) model [3, 4, 7].

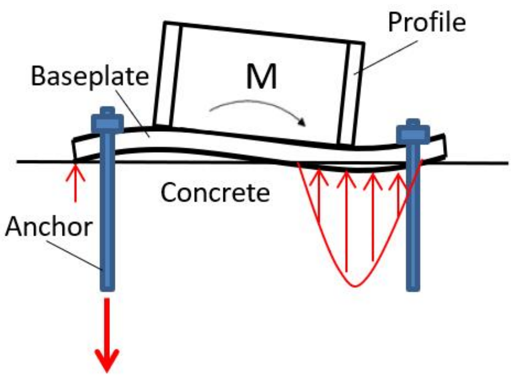


Fig. 1 Schematic representation of anchorage with elastic baseplate

- Baseplate is assumed to be bedded elastically on the concrete and modelled using plate bending elements.

- The anchors are represented by elastic springs taking only tension force.

- The concrete areas under baseplate are represented by elastic springs taking only compression force.

- The profile is simulated by shell elements connected to the plate elements.

This above FEA model has been verified by laboratory tests in [3] and can be used to evaluate the behavior of the anchorage system accurately.

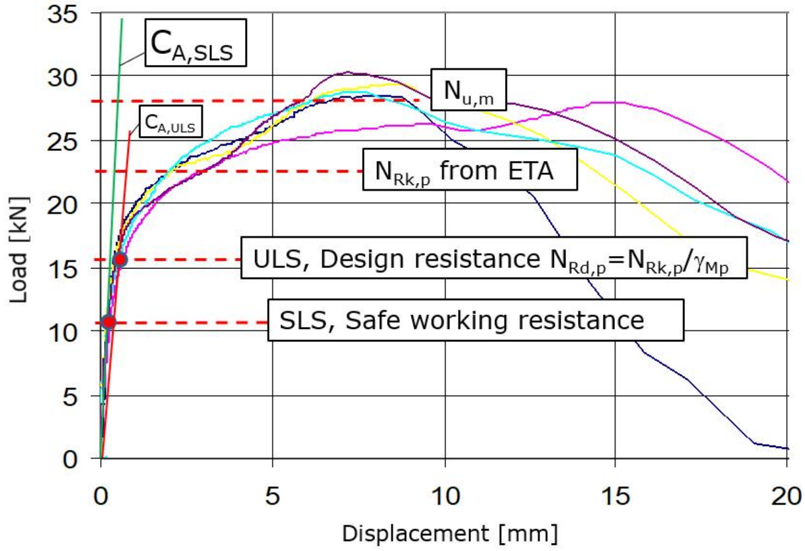
2.1.2 Anchor tension stiffness and concrete bedding factor

The anchor tension stiffness used in the FEA as a spring constant can be determined by pull-out test of single anchors in concrete. The mean stiffness $C_{A,SLS}$ at the load level of Serviceability Limit State (SLS, Fig. 2) in uncracked concrete is decisive in the anchorage design for the following reasons.

- In the elastic anchorage design, the FEA is conducted with one stiffness value for anchor and concrete respectively. Due to the non-linearity of the anchor stiffness between SLS and ULS (Ultimate Limit State), the higher stiffness of SLS and ULS is decisive for the anchorage design, because the higher anchor stiffness results in higher anchor tension force in general.

- In working conditions of the anchorage, the load on anchors does not exceed the load level of SLS. For verification of SLS, the anchor tension forces at SLS can be determined by reducing the anchor tension forces of FEA at ULS linearly with a factor between design and characteristic action load. That is, if the anchor stiffness at ULS is used for the FEA in design of anchorages, the anchor tension forces for verification of SLS have to be determined by a second FEA using the anchor stiffness at SLS. Then, it is no longer linear elastic design.

- In practice, most anchors are located in uncracked concrete in SLS, although they are designed with cracked concrete.



C_A: Anchor tension stiffness evaluated from pull-out tests

Fig. 2 Relevant anchor stiffness for anchorage design

The anchor tension stiffness C_g for anchorage design can be expressed as in equations (2.1) and (2.2) according to the bearing mechanisms of different anchor types (Fig. 3), neglecting the washer thickness.

- For headed stud with smooth shaft (Fig. 3 a)), the stiffness can be derived from the elongation of shaft and the displacement of head due to concrete deformation under tension load on the anchor to equation (2.1).

$$C_g = \frac{1}{\frac{h_{ef}+t_{fix}}{E_S \cdot A_S}+\frac{1}{C_C \cdot A_H}} \qquad (2.1)$$

with

E_s : E-modulus of anchor rod

A_s : Shaft cross sectional area

A_H : Bearing area of the head

h_{ef} : Effective anchorage depth

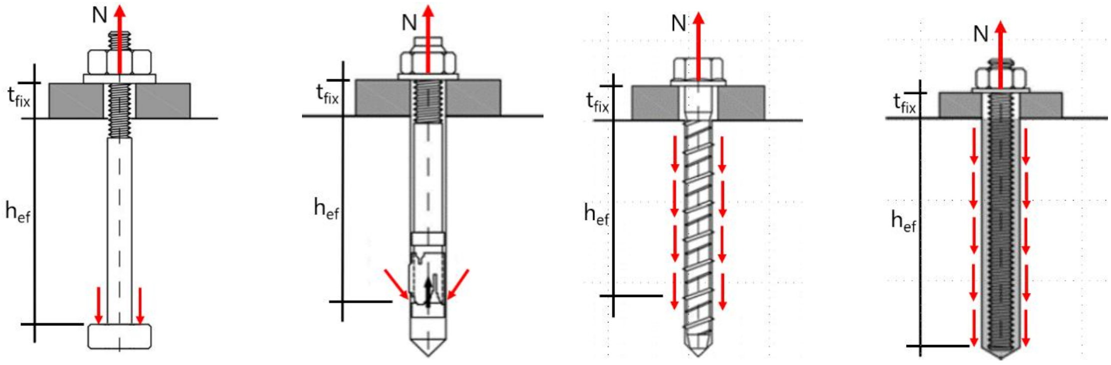
t_{fix} : Baseplate thickness

C_c : Concrete bedding factor (Fig. 4)

- For post-installed anchors (Fig. 3 b)-d)), the anchor stiffness may be expressed by equation (2.2).

$$C_g = \frac{E_s \cdot A_s}{\frac{h_{ef}}{\varphi} + t_{fix}} \qquad (2.2)$$

with stiffness factor φ to consider the slip, the tension stiffening of anchor body or the influence of concrete strength.



a) Headed stud b) Expansion anchor c) Screw anchor d) Bonded anchor

Fig. 3 Bearing mechanism of common anchor types

Based on the evaluation of test results in [8, 9] and the theoretical derivation in [10], the following stiffness factors may be proposed if no laboratory tests have been carried out to obtain such values.

$\varphi = 0.3$ for Torque-controlled expansion anchors in low strength concrete [8]

$\varphi = 0.5$ for Torque-controlled expansion anchors in high strength concrete [9]

$\varphi = 1.0$ with $h_{ef} \leq 8d$, for cast-in place rebar, bonded anchors and concrete screws [10]

To accommodate the scatter of anchor stiffness and based on fatigue loading test results [11], an additional safety factor of 1.35 may be adopted in the process of baseplate design.

The stiffness of concrete in compression C_c may be expressed by concrete bedding factor evaluated by test results with $C_c = D / (\Delta \cdot A_c)$ (Fig. 4).

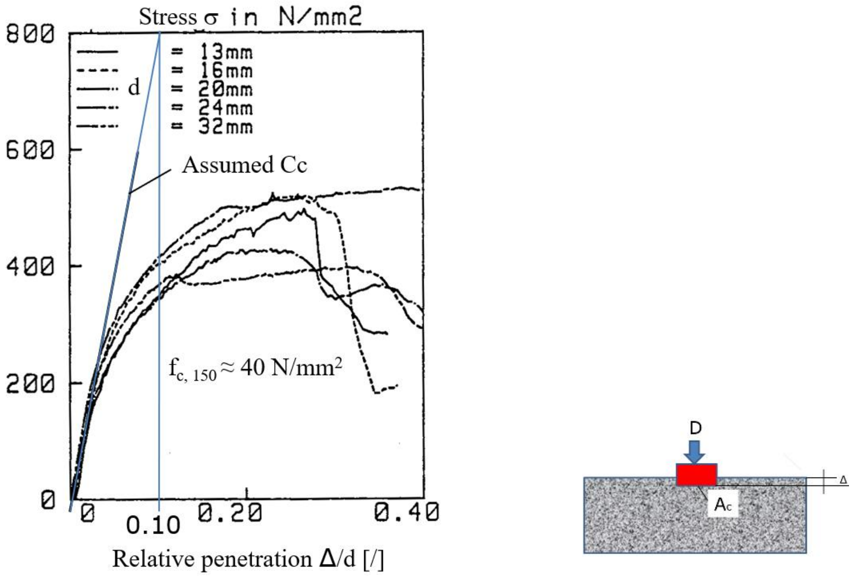


Fig. 4 Load-displacement behavior of concrete ($f_{ck,cube}$=40 N/mm²) under localized compression [12]

Based on the test results in concrete $f_{ck,cube} \approx 40$ N/mm³ with different compressive area [12], the concrete bedding factor may be determined by equation (2.3) depending on the diameter d of compression area.

$$C_c = 800 / (0.10 \cdot d) \qquad (2.3)$$

with d: Stamp diameter in the tests.

e.g. d=13mm, C_c=800/(0.10*13) =615 N/mm³, about 15$f_{ck,cube}$ = 600 N/mm³

d =32mm, C_c=800/(0.10*32)= 250 N/mm³, about 6$f_{ck,cube}$ = 240 N/mm³

The parametric studies in [13] showed that the concrete bedding factor does not have so much influence on the anchor tension force distribution in baseplate. The following concrete bedding factor may be proposed for the FEA in anchorage design.

$$C_c = b \cdot f_{ck,cube} \qquad (2.4)$$

with b =15 for calculating the anchor tension stiffness according to equation (2.1) and the concrete bedding factor for FEA of baseplates.

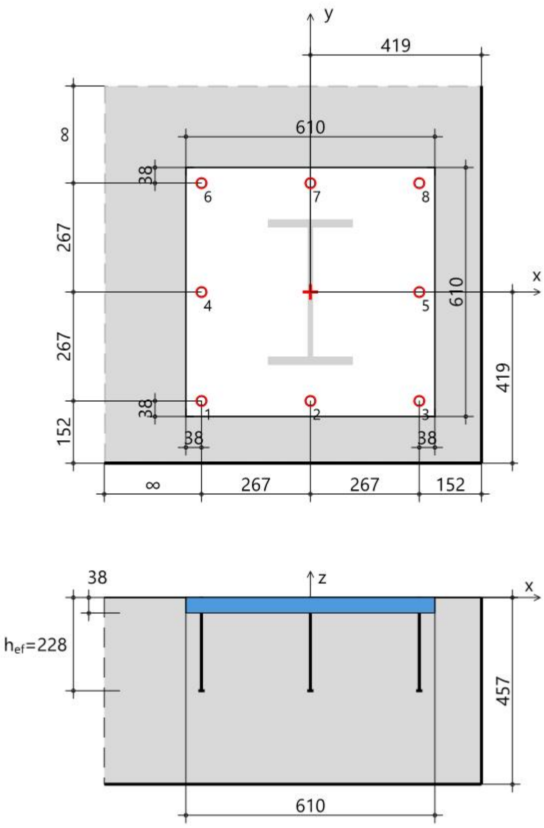
2.1.3 Discussion about rigid baseplate assumption by calculation examples

The rigid baseplate assumption is normally understood as baseplate without deformation itself. That is, under tension load and bending moment, the baseplate rotates and translates depending on the anchor and concrete stiffness. But in EN 1992-4 [1], the rigid baseplate assumption for anchorage design is defined as linear strain assumption analogous to the reinforced concrete beam section with “plane sections remain plane”.

In order to see the differences between the linear strain assumption and the rigid baseplate assumption, three calculation examples are studied. The anchorage parameters are shown in Fig. 5. The rigid baseplate assumption is simulated by elastic baseplate with an increased E-modulus of $E = 2.1x10^{14}$ N/mm² so that there are effectively no bending deformations.

The results of the 3 calculations displayed in Fig. 6 a) - c) show that the linear strain assumption (Fig. 6 a)) produces some higher anchor tension force than the real rigid baseplate assumption (Fig. 6 b), c)). With the rigid baseplate assumption, the anchor stiffness has only a small influence on the calculated highest anchor tension force (Fig. 6 b), c)).

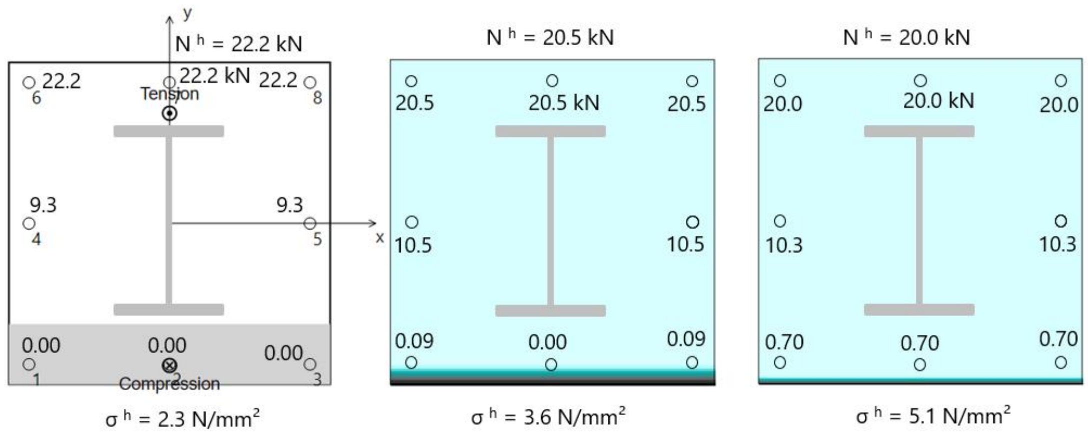
In the current anchorage design practice, the linear strain assumption is taken as rigid baseplate assumption which is regulated in EN1992-4 [1] for anchor tension force calculation with the precondition that the baseplate is sufficiently rigid. In the following sections, the linear strain assumption will be treated as equivalent to the rigid baseplate assumption.



Action load: $M_{Ed,x}$ = 40,7 kNm, $V_{Ed,y}$ = - 89 kN
Attached profile: AISC W14x53
Baseplate material S235, f_{yk}=235 N/mm²
Concrete: f_{ck} = 31.0 N/mm² with edge reinforcement and closely spaced stirrups
Headed stud d22x200

Fig. 5 Design example with 8 headed studs [14]

N^h: highest anchor tension force; σ^h: highest concrete compressive stress



a) Linear strain distribution b) Rigid plate, C_g = 186.1 kN/mm c) Rigid plate, C_g = 49.5 kN/mm

Fig. 6 Comparison of anchor tension forces and concrete compressive stresses under baseplate resulted from linear strain and rigid baseplate assumptions ($f_{c,cube}$=38.3 N/mm²)

2.1.4 Stiffness criteria to check the baseplate rigidity

Because the E-modulus of the baseplate is a constant value depending on the material of the baseplate, the baseplate rigidity can be changed only by baseplate thickness and by the supported profile with optionally additional stiffeners.

In order to study the influence parameter on the baseplate rigidity, the anchor tension force distributions are calculated with variation of baseplate thickness, connection profile and anchor stiffness by the FEA described in section 2.1.1 using the same example shown in Fig. 5. The calculated highest anchor tension force in anchor #7 is displayed in Fig. 7 with varying baseplate thicknesses. For comparison, the calculated highest anchor tension force by rigid baseplate assumption and its +5% tolerance are also shown in the diagram.

From Fig.7, the following interesting points may be observed:

- With the practical baseplate thickness t_{fix} = 30-40 mm, the bending stress in the baseplate satisfies the stress criterion $\sigma_s^h \leq f_{yk}/1.35$= 174 N/mm² (Tab. 1). But the anchor tension force distribution is non-linear. The highest anchor tension force is much higher than that based on rigid baseplate assumption with a factor of more than twice (48.5/22.2 = 2.2) (see Fig. 8 a) and Fig. 6 a)).

- Baseplate rigidity depends not only on plate thickness, but also the relative anchor stiffness (including the base material property and the type of anchoring mechanism) and profile type and size. For example, to achieve an equivalent rigid baseplate, t_{fix}=100mm is required for profile HP14x117 whereas to achieve the same for W14x53, a higher thickness of 110mm is required. This is just the stiffness criterion in equation (2.5) proposed in [3] to assess the required baseplate rigidity for anchorage design according to the current regulations [1, 2] in addition to the stress criterion that the baseplate does not yield.

$$(N_e^h - N_r^h)/ N_r^h \leq 5\% \qquad (2.5)$$

where N_r^h and N_e^h are the highest anchor tension forces in the anchor group calculated according to rigid baseplate assumption and FEA, respectively.

- In this example, the required baseplate thickness to meet the conditions of rigid baseplate assumption is so large that is unrealistic in practice (Fig. 9).

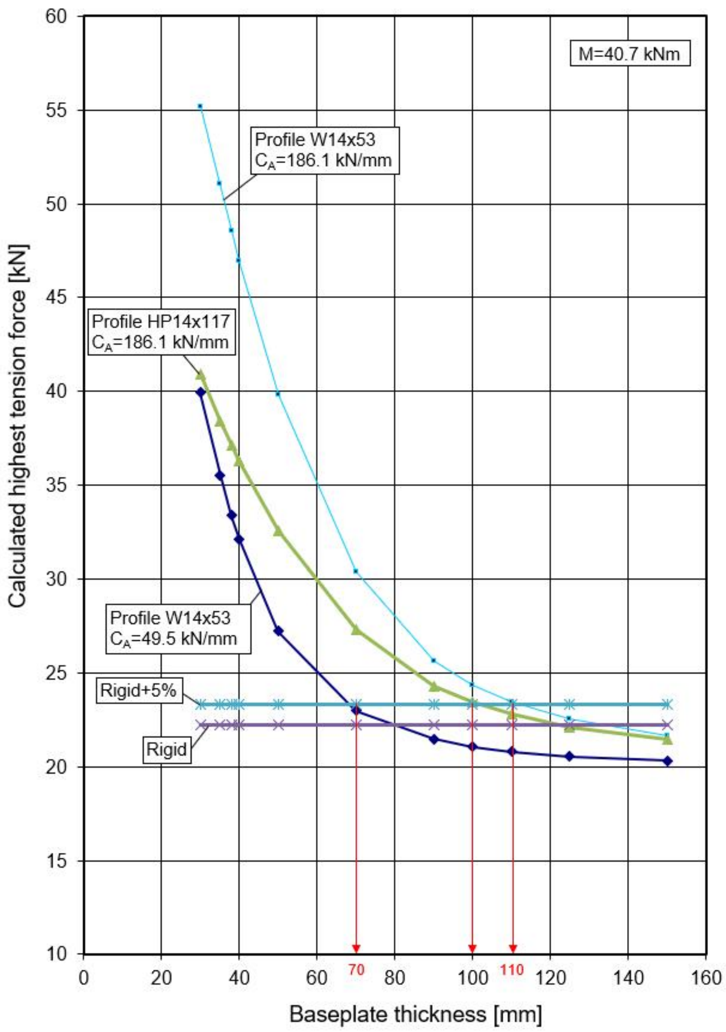


Fig. 7 Highest tension in anchor #7 vs baseplate thickness with M=40.7 kNm

The calculated bending stress deviation in baseplate from the different anchor stiffnesses 186.1 kN/mm and 49.5 kN/mm show that the proposed additional safety factor 1.35 for baseplate design can cover the scatter of the anchor stiffness (see Tab. 1 a) and c)).

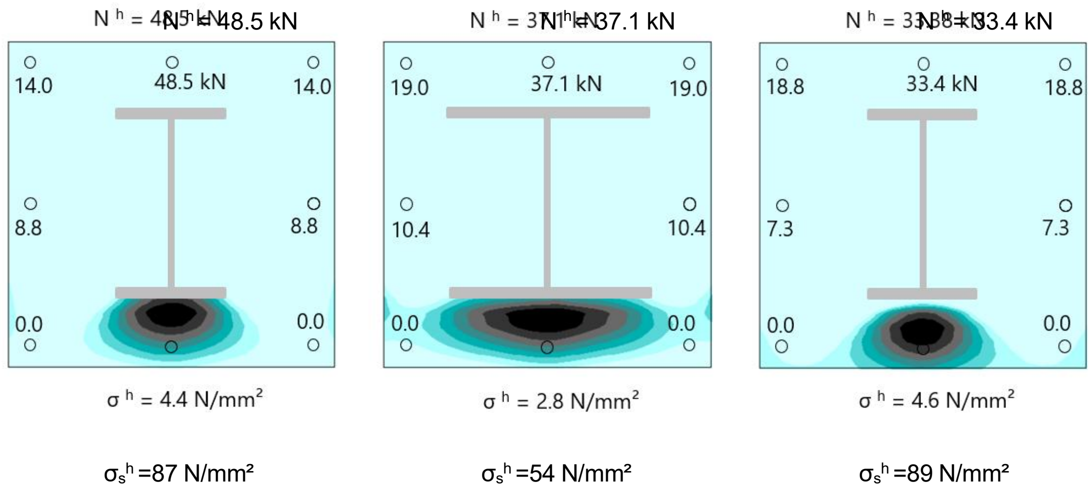
Baseplate t_{fix} [mm]	Bending stress $\sigma_s{}^h$ [N/mm²]	
	a)	c)
30	134	143
35	102	106
38	87	89
40	79	84

a) W14x53, C_g = 186.1 kN/mm

c) W14x53, C_g = 49.5 kN/mm

Tab. 1 Calculated highest bending stress in baseplate in variation of anchor stiffness C_g

N^h: highest anchor tension force; σ^h: highest concrete compressive stress; $\sigma_s{}^h$: highest bending stress in baseplate



a) W14x53, C_g = 186.1 kN/mm   b) HP14x117, C_g = 186.1 kN/mm   c) W14x53, C_g = 49.5 kN/mm

Fig. 8 Comparison of anchor tension forces and concrete compressive stresses under baseplate calculated with t_{fix} = 38 mm, $f_{c,cube}$=38.3 N/mm²

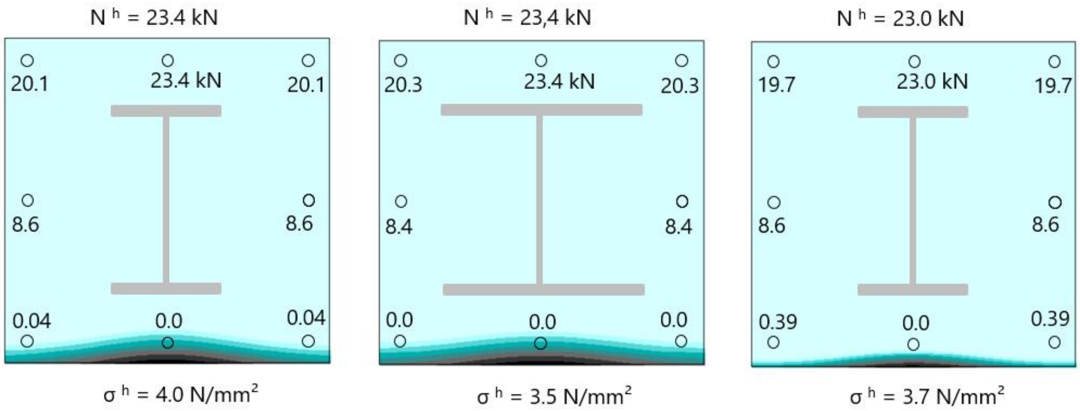


Plate thickness t_{fix} = 110 mm t_{fix} = 100 mm t_{fix} = 70 mm

a) W14x53, C_g = 186.1 kN/mm b) HP14x117, C_g = 186.1 kN/mm c) W14x53, C_g = 49.5 kN/mm

Fig. 9 Comparison of anchor tension forces and concrete compressive stresses under equivalent rigid baseplate according to equation (2.5)

2.1.5 Parametric studies for rigid baseplate assumption

Since putting the stiffness criterion according to equation (2.5) into practical use in 2016 [15], there have been questions raised by users why a relatively lower tension load and bending moment acting on the baseplate arise a large thickness to satisfy the criterion for rigid baseplate assumption. To address this question, parametric studies on influence of loads on the required thickness of flexurally rigid baseplate were carried out. In Fig. 10-12, the required baseplate thicknesses according to the stiffness criterion in equation (2.5) for the example in Fig. 5 are displayed in variation of loadings on the baseplate.

The results of FEA in Fig. 10-12 show that the action load on the baseplate has only a relatively small influence on the required thicknesses of flexurally rigid baseplate. It appears to be unconceivable in Fig. 11 showing that a very low tension load N => 0 kN on the baseplate arises just the same large baseplate thickness as N =100 kN. But this may be the reality that the rigid baseplate is mainly a material property which does not so much depend on the loads.

Fig. 12 shows that the anchor stiffness has the most influence on the required thickness of rigid baseplate.

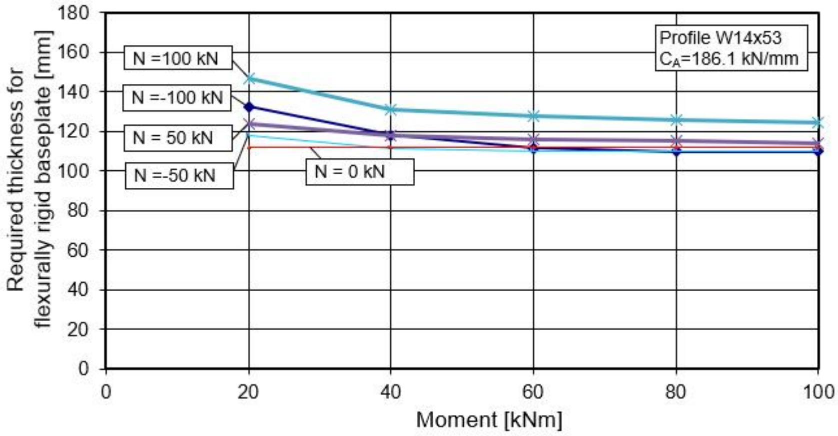


Fig. 10 Influence of bending moment and tension load on the required thickness of equivalent rigid baseplate

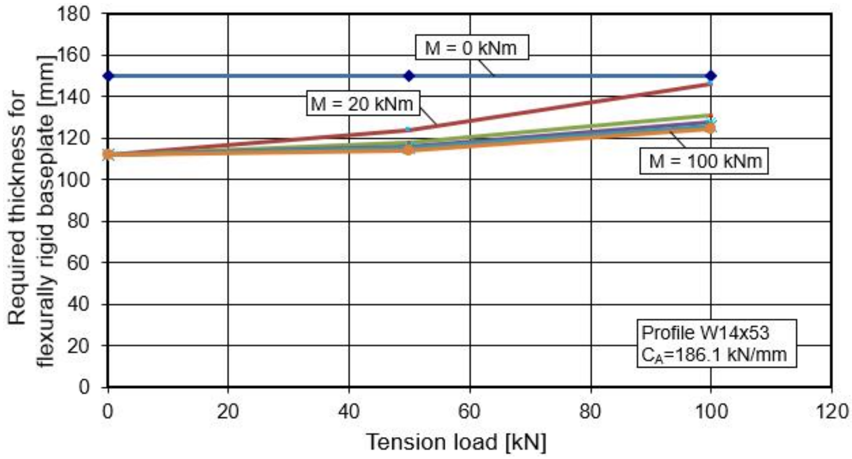


Fig. 11 Influence of tension load and bending moment on the required thickness of equivalent rigid baseplate

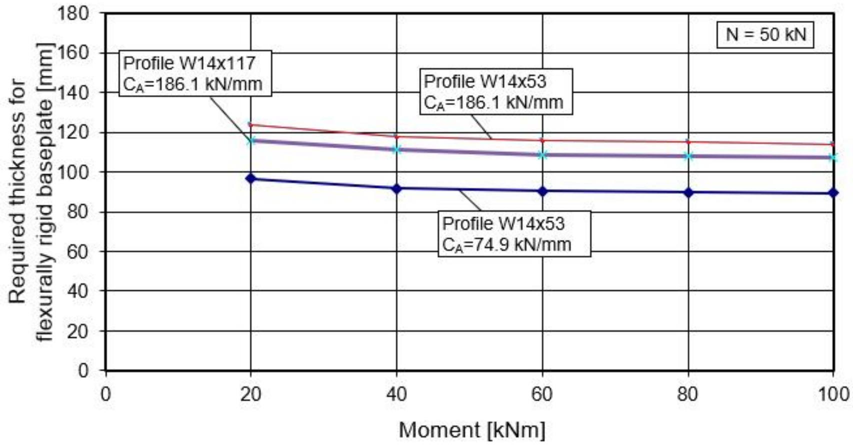


Fig. 12 Influence of bending moment, attached profile and anchor stiffness on the required thickness of equivalent rigid baseplate

2.2 Baseplate subjected to shear loading on the fixture

2.2.1 Simulation of baseplates under shear loading

Just like tension and bending, the baseplate under shear loadings needs to be sufficiently rigid to use the design methods according to EN 1992-4 [1] and ACI 318 [2]. In most cases, e.g. baseplates with width/length w/l > 1/3, the baseplate rigidity for shear loading may not be critical [16]. But for anchorage near edge with narrower baseplate, the required rigidity of baseplate may need to be considered.

To study the elastic behavior of baseplate under shear loadings, the baseplate is simulated by plane stress elements. The anchors are simulated by elastic springs connected to the baseplate. The shear forces from the profile are distributed in proportion to the contact area of profile to the baseplate. The stiffness of profile is neglected conservatively in the study.

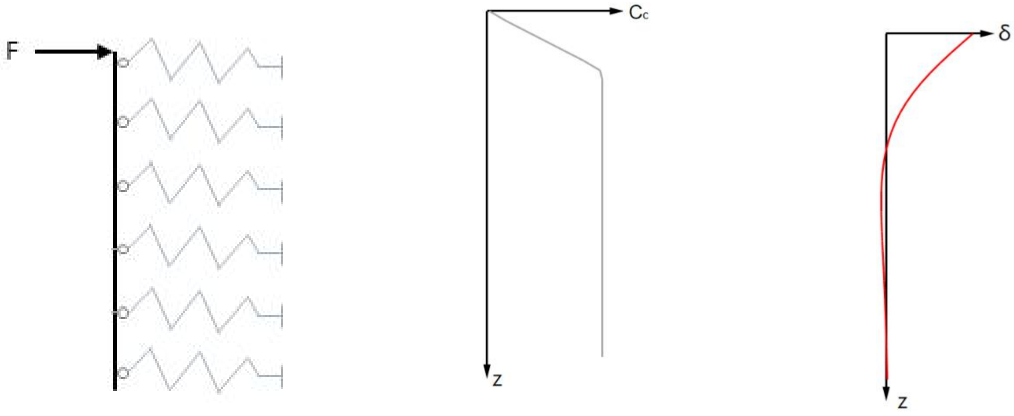
2.2.2 Anchor shear stiffness

The anchor shear stiffness C_{AV} used in the FEA as a spring constant under shear loading may be determined by simulation with a beam of anchor rod embedded elastically in concrete (Fig. 13). The concrete bedding factors are determined according to equation (2.4) with b iven in Tab. 2.

Tab. 2 Proposed coefficient b for determining the concrete bedding factor $C_c = b \cdot f_{c,cube}$

Anchor d [mm]	8	10	12	16	20	24	27	30	36
b [-]	15	15	15	12.5	10	8.3	7.5	6.6	5

To consider the limited confinement of concrete near the concrete surface, the concrete bedding factor C_c near concrete surface is reduced linearly (Fig. 13 b)) to a depth of 1.5d for bonded anchors (Fig. 2.3d)) and 1.75 d for wedge anchors (Fig. 2.3 b)) where d is the anchor diameter.



a) Elastically bedded beam b) Distribution of concrete bedding factor c) Anchor displacement

Fig. 13 Simulation of anchor in concrete under shear load

The simulated anchor shear stiffnesses are verified by 12 test series, 6 of which for wedge anchors M8 – M24 and other 6 for bonded anchors M8 – M24 respectively [17]. In Fig. 14, the simulated anchor stiffness without gap is compared with test results with gap. Without gap, the simulated anchor shear stiffness may reflect the anchor shear stiffness realistically.

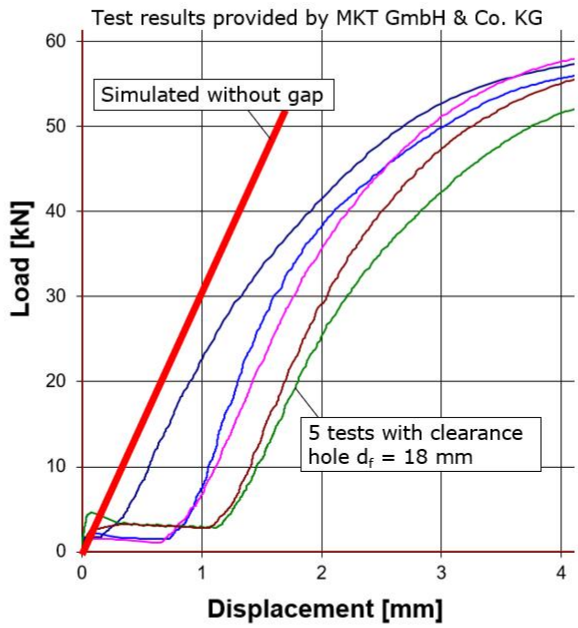


Fig. 14 Comparison of simulated (without gap) with experimental (with gap max. 2 mm) load-displacement behavior of bonded anchor M16 under shear load, $f_{c,cube} = 35.9$ N/mm²

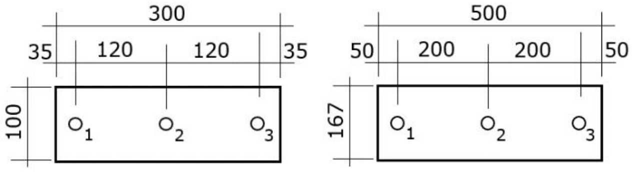
2.2.3 Calculation examples for stiffness criteria to check the baseplate rigidity

Using a similar approach to comparing the anchor tension forces resulted from FEA and rigid baseplate assumption for evaluating the baseplate rigidity, calculations were made to study the effect of non-linear distribution of anchor shear forces for narrower baseplates. The baseplates used are shown in Fig. 15 ($w/l = 1/3$, $t_{fix} \approx d$) with a shear load of 12 kN. The following configurations have been used in the calculations:

- Bonded anchors M16 h_{ef} =125mm, concrete C20/25, baseplate 300x100x15 mm, simulated shear stiffness 28.9 kN/mm without gap

- Bonded anchors M30 h_{ef} =300mm, concrete C20/25, baseplate 500x167x30 mm, simulated shear stiffness 53.0 kN/mm without gap

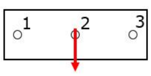
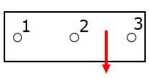
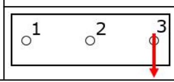
The calculated shear force distributions are summarized in Tab. 3. The results show that the critical load action point is the shear load around the middle anchor #2 perpendicular to the anchor line (line No 1 and 7 in Tab. 3). The deviation of anchor shear force on anchor #2 between rigid baseplate assumption and realistically elastic baseplate is about 6%, i.e. $(V_e^h - V_r^h)/V_r^h \approx 6\%$. That means the geometrical condition $w/l \geq 1/3$ and $t_{fix} \approx d$ may be proposed as stiffness criterion to check the required baseplate rigidity for shear loading.

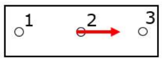
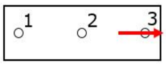
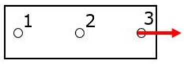
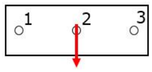


a) M16 t_{fix} =15 mm b) M30 t_{fix} =30 mm

Fig. 15 Baseplates used for the calculations in Tab. 3

Tab. 3 Comparison of calculated shear force distribution between rigid and elastic baseplate with w/l=1/3 (Fig. 15)

LineNo	Size	Load location V= 12 kN	Plate rigidity	Shear force [kN] on anchor No		
				1	2	3
1	M16		Rigid	4.0	4.0	4.0
			Elastic	3.88	4.24	3.88
2			Rigid	1.0	4.0	7.0
			Elastic	0.95	4.11	6.95
3			Rigid	-2.0	4.0	10.0

			Elastic	-1.94	3.88	10.1
4			Rigid	4.0	4.0	4.0
			Elastic	3.98	4.03	3.98
5			Rigid	4.0	4.0	4.0
			Elastic	3.96	4.03	4.03
6			Rigid	4.0	4.0	4.0
			Elastic	3.93	3.98	4.09
7	M30		Rigid	4.0	4.0	4.0
			Elastic	3.89	4.23	3.89

2.3 Summary of stiffness criteria to check the rigid baseplate assumption under tension and shear loading

For anchorages with baseplate under tension load and bending moment on the fixture, the stiffness criteria according to equation (2.5) [3] can evaluate the baseplate rigidity clearly. This approach is easily applicable in practice for the following reasons:

- The highest anchor tension force N_e^h in the anchor group results anyway from the FEA in baseplate design in which the highest bending stress in the baseplate has to be calculated.

- The highest anchor tension force N_r^h in the anchor group results from the calculation according to [2] or [3] using rigid baseplate assumption for anchorage design.

- With the calculated values N_e^h and N_r^h , the rigidity of the baseplate can be assessed using equation (2.5).

The calculation examples in section 2.1.4 and the parametric studies in section 2.1.5 confirm the statements in [5, 6] that the linear strain/rigid baseplate assumption is unrealistic for the majority of anchorage designs in practice.

For anchorages with baseplate under shear loading, the baseplate rigidity may need to be checked only for narrower baseplates with w/l<1/3 and t_{fix} < d.

3. Anchorage design with realistically elastic baseplate

3.1 Baseplate subjected to tension load and bending moment on the fixture

3.1.1 Criteria for additional proof of concrete cone failure resistance of anchor groups

The examples of FEA in section 2.1.4 (Fig. 7, 8) show that the anchor tension force distribution is non-linear for normal range of applicable baseplate thickness, e.g. t_{fix}= 38 mm in the design example [14]. In this case, an additional proof is required to check the concrete cone failure resistance of the anchor group under tension loading, because the baseplate is not rigid enough to use the calculation method for concrete cone failure resistance of anchor group according to [1, 2] directly [3,4].

The following criteria may be used to check if the additional proof is required (Fig. 16) in design of anchor groups with elastic baseplate.

- Neglecting the small anchor tension forces N_i in the anchor group with $N_i / N^h \leq 5\%$ where N^h is the highest anchor tension force in the group. This measure is for the purpose to avoid over-conservative results of the additional proof due to the small anchor tension force in the group increasing the number of tensioned anchors n in the equation (3.2).

- Check all calculated anchor tension forces N_i in the anchor group with $N_i / N^h > 5\%$ whether they are in a line or in a plane. If a tension force of an anchor in the group is not in the line or in the plane with $\Delta Ni / Ni > 5\%$ (Fig. 16), the additional proofs are required as proposed in [3,4] for concrete cone failure and for combined pull-out and concrete cone failure (only bonded anchors) by modifying the load eccentricity factor, e.g. for concrete cone failure as follows.

$$\psi_{ec,N} = 1.0 \tag{3.1}$$

$$N_{Ed}^{\ h} \leq N_{Rd,c,e} = N_{Rd,c} / n \tag{3.2}$$

with $\psi_{ec,N}$: Reduction factor to consider the load eccentricity in the anchor group in equation (7.1) of EN1992-4 [1] or in equation (17.4.2.1b) of ACI 318-14 [2].

$N_{Ed}^{\ h}$: Highest anchor tension force in the group

$N_{Rd,c}$: Design resistance of the anchor group at concrete failure with $\psi_{ec,N}$ =1,0

n: number of anchors in tension in the group

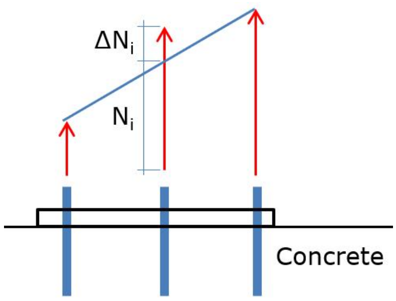


Fig. 16 Anchor tension force deviation ΔNi /Ni for evaluating the baseplate rigidity

This additional proof is verified by 40 laboratory tests by comparing the tested peak load with corresponding calculated design resistance with additional proof in next section 3.1.2 and explained in detail in the section 3.1.3 with a design example.

3.1.2 Validation of the proposed additional proof

For non-linearly distributed anchor tension forces in the anchor group, the additional proof to the linearly distributed anchor tension forces is proposed according to equations (3.1) and (3.2) for calculation of design resistance at concrete cone failure and combined bond and concrete cone failure.

To check the safety level of the additional proof, 40 test results are analyzed.

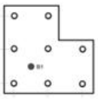
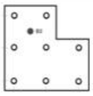
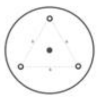
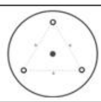
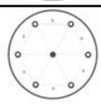
In Tab. 4-6, the peak load $N_{u,test}$ from tests is compared with the design resistance $N_{Rd,c}$ from the calculation. The required mean safety factor is determined by $N_{u,test}$ /$N_{Rd,c}$ =1.5/0.75=2.0, where 1.5 is the required safety factor for the concrete cone failure resistance and 0.75 is the factor for characteristic value to mean value of concrete cone resistance.

In Tab. 4 and 5, the tests were loaded until to failure by tension through a rod connected to baseplate [21]. In the test series G84 (Tab. 4), the baseplate should have been yielded at 6.6 kN (assumed bending stress in baseplate 355 N/mm²) which is much lower than the peak load at failure 46.1 kN. In the calculation, it is assumed that the baseplate remains elastic up to the concrete design resistance of 16.3 kN. In all other tests, the baseplates should have been remained elastic until to the concrete design resistance just as assumed in the calculations.

In the calculations with elastic baseplates, the anchor stiffness factor φ is assumed 0.5 for the torque-controlled expansion anchors and 1.0 for the bonded anchors.

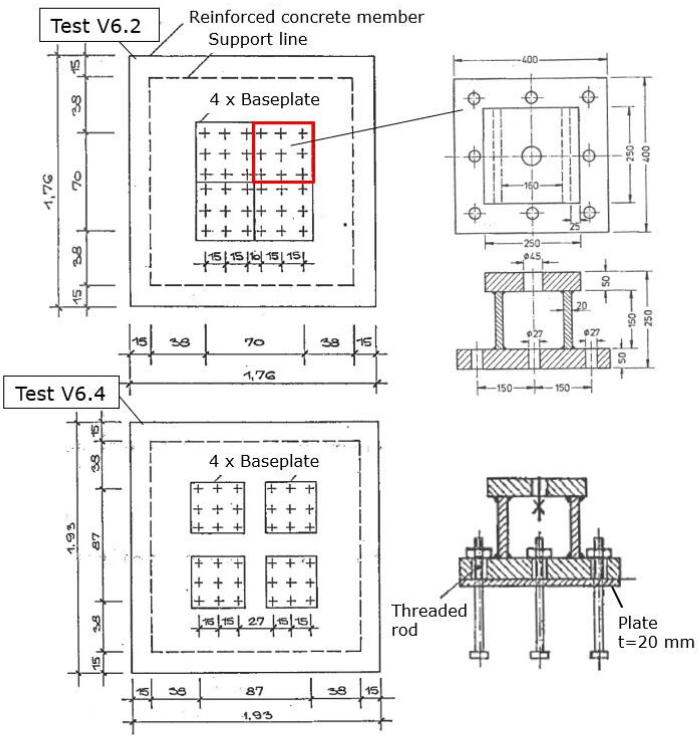
The comparison of calculated safety factors $N_{u,test}/N_{Rd,c}$ between rigid baseplate assumption and elastic baseplate in Tab. 4 and 5 shows that the elastic baseplate with the proposed additional proof is conservative (mean safety factor > 2.0 and minimum safety factor >1.5) while the rigid baseplate assumption is clearly unconservative in tests G83 and G84 in Tab. 4 with mean safety factor =1.9< 2.0 and minimum safety factor =1.1 < 1.5.

Tab. 4 Calculation of test results [21] based on [18]

	Test conditions and test results							Calculated design resistance $N_{Rd,c}$		Safety factor		
Test series	Baseplate l x b x t	Loading point	Bonded anchor	fc,cube N/mm²	hef mm	Spacing mm	$N_{u,test}$ kN	Rigid plate kN	Elastic plate kN	$N_{u,test}/N_{Rd,c}$ Rigid pl.	Elastic pl.	Required
G81	350 x 60 x 40			31,4	60	90	73,5	43,8	27,8	1,7	2,6	
G82	350 x 60 x 40		M12	30,3	60	90	48,8	22,4	22,3	2,2	2,2	
G83	350 x 60 x10			31,4	60	90	51,6	43,8	20,9	1,2	2,5	
G84	350 x 60 x 5			31,4	60	90	46,1	43,8	16,3	1,1	2,8	
G91	Side length 220, 140, 140, 220, 360, 360 t=50		M16	37	60	140	105,4	48,1	49,5	2,2	2,1	
G92			M16	37	60	140	113,4	41,5	52,3	2,7	2,2	
G101	D=210 t=25		M16	37	60	140	89,8	46,8	46,8	1,9	1,9	mean 2,0 min. 1,5
G201			M16	70	60	140	128,9	59,5	59,5	2,2	2,2	
G301	D=300 t=40		M12	30,3	60	120	127,3	72,5	72,5	1,8	1,8	
G401			M12	31,3	60	120	70,5	33,9	34,0	2,1	2,1	
								Mean value of 10 tested groups		1,9	2,2	2
								Minimum		1,1	1,8	1,5

Tab. 5 Calculation of test results [21] based on [18, 19]

	Test conditions and test results								Calculated design resistance $N_{Rd,c}$		Safety factor		
Test series	Baseplate lxbxt	Loading point	Anchor type	fc,cube N/mm²	hef mm	Spacing mm	Edge c mm	$N_{u,test}$ kN	Rigid plate kN	Elastic plate kN	$N_{u,test}/N_{Rd,c}$ Rigid pl.	Elastic pl.	Required
G11	270x30x30		Torque-controlled expansion anchor M12	25,8	55	98		56,1	30,5	16,8	1,8	3,3	mean 2,0 min. 1,5
G12				25,8	55	98	50	48,1	24,8	13,4	1,9	3,6	
G13				25,8	55	98		40,7	19,2	20,3	2,1	2,0	
G22				30,3	55	98	50	33,1	17,3	18,3	1,9	1,8	
G23				30,3	55	98	50	41,6	17,3	18,3	2,4	2,3	
G31	160x160x30			25,8	55	98		63,4	35,4	35,4	1,8	1,8	
G32				25,8	55	98		46,4	22,0	18,2	2,1	2,5	
G51	400x120x50		Bonded anchor M16	66,6	70	120		103,9	59,1	42,4	1,8	2,4	
G52				66,6	70	120		83,3	38,3	35,2	2,2	2,4	
G53				66,6	70	120		70,4	30,3	23,2	2,3	3,0	
G54	400x120x25			66,6	70	120		61,8	30,3	26,9	2,0	2,3	
									Mean value of 11 tested groups		2,0	2,5	2
									Minimum		1,8	1,8	1,5



Dimensions mixed in m, cm and mm

Fig. 17 shows the test conditions of 16 tested anchor groups in 4 tests which are evaluated in Tab. 6. Each test included 4 anchor groups with 9 headed studs d22. With special test set-up, the 4 anchor groups are loaded at same time in the test [22].

- 4 anchor groups loaded in tension in same time

- Each group with 9 headed studs d22, h_{ef}=185mm

- Uncracked concrete, $f_{c,\,cube} \approx 24$ N/mm²

Fig. 17 Anchor layout, baseplate and supported profile in tests [22]

In the calculation with elastic baseplate, an equivalent baseplate thickness 51 mm is used with the same moment of inertia from 2 plates of 50mm and 20mm together (Fig. 18). The anchor stiffness is calculated according to equation (2.1) to C_g=124.8 kN/mm. Because the calculation is done with a part group (Fig. 17, 18) with a fictive edge distance of 50 mm arising an edge reduction factor $\psi_{s,N}$ = 0.745, the design resistance is modified with the reduction factor $\psi_{s,N}$ = 1.0 with $N_{Rd,c}$ = 80.9/0.754 = 107.3 kN (Tab. 6 test V6.2.2) [22]. As shown in Fig. 19, the baseplate thickness 51 mm cannot create a uniformly distributed anchor tension force. The tension force for the anchor #5 at the plate centrum compared to anchor #1 on the plate corner amounts to a factor of 10.405 kN / 8.283 kN =1.26.

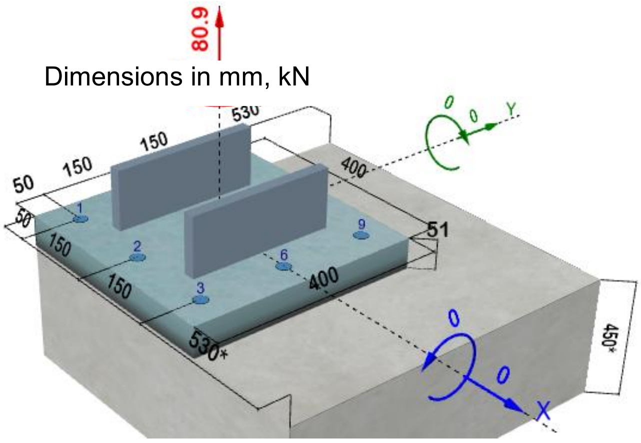
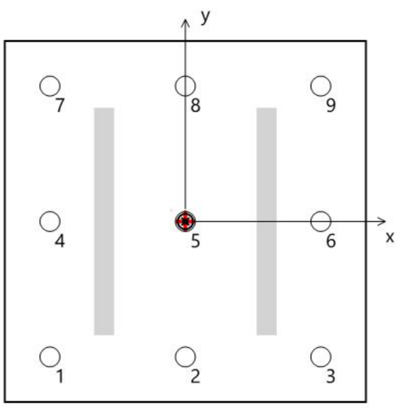


Fig. 18 Simulated baseplate and profile in the calculation [25]



Anchor No.	Tension [kN]
1	8.283
2	9.271
3	8.283
4	9.411
5	10.405
6	9.411
7	8.282
8	9.271
9	8.283

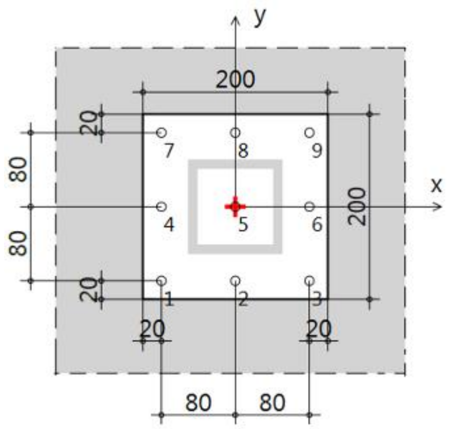
Fig. 19 Calculated anchor tension force distribution [25]

Tab. 6 Calculation of test results from [22] based on [20]

	Test results of each group			Calculated design resistance $N_{Rd,c}$		Safety factor		
Test	fc,c150	Spacing	$N_{u,test}$	Rigid baseplate	Elastic baseplate	$N_{u,test}/N_{Rd,c}$		
series	N/mm²	mm	kN	kN	kN	Rigid plate	Elastic plate	Required
V6.2.1	22,6	100/150	198	121,30	105,2	1,63	1,88	mean 2,0 min. 1,5
			235	121,30	105,2	1,94	2,23	
			196	121,30	105,2	1,62	1,86	
			210	121,30	105,2	1,73	2,00	
V6.2.2	23,7	100/150	229	124,23	107,3	1,84	2,13	
			229	124,23	107,3	1,84	2,13	
			225	124,23	107,3	1,81	2,10	
			223	124,23	107,3	1,80	2,08	
V6.4.1	23,6	150/270	295	159,75	138,2	1,85	2,13	
			306	159,75	138,2	1,92	2,21	
			309	159,75	138,2	1,93	2,24	
			310	159,75	138,2	1,94	2,24	
V6.4.2	23,6	150/270	258	159,75	138,2	1,62	1,87	
			263	159,75	138,2	1,65	1,90	
			262	159,75	138,2	1,64	1,90	
			286	159,75	138,2	1,79	2,07	
				Mean value of 16 tested groups		1,78	2,06	2,0
				Minimum		1,62	1,86	1,5

The calculated safety factors $N_{u,test}/N_{Rd,c}$ on rigid baseplate assumption and on elastic baseplate in Tab. 6 show that the elastic baseplate with the proposed additional proof is conservative with mean safety factor = 2.06 > 2.0 and minimum safety factor = 1.86 > 1.5 while the rigid baseplate assumption is slightly unconservative with mean safety factor =1.78 < 2.0 for the tested baseplate thickness of 51 mm.

In Tab. 7, three tests with anchor groups of 9 headed studs as shown in Fig. 20 a) are calculated and evaluated. Because the highest anchor tension force $N_{test}{}^{h}$ at peak load is recorded for each anchor in the tests as shown in Fig. 20 b) for baseplate thickness 10 mm, the safety factor for elastic baseplate is calculated with the measured value $N_{test}{}^{h}$ in Tab. 7 with $N_{test}{}^{h} / N_{Rd,c,e}$.

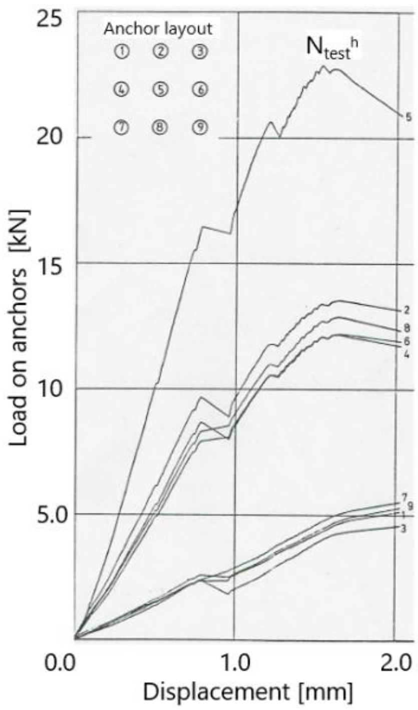


Concrete $f_{c,cube}$ = 43.4 N/mm²
headed stud d16, h_{ef} =65 mm.

Baseplate with stand-off installation, distance a=300mm, connected with threaded rod M16

3 tests with plate thickness 10, 15 and 20 mm respectively.

a) Test condition



b) Measured anchor tension forces at plate thickness 10 mm

Fig. 20 Test condition and measured anchor tension forces at t_{fix}=10 mm [23]

Tab. 7 Calculation of test results [23] based on [20]

	Test results			Calculated resistance $N_{Rd,c}$		Safety factor		
Test	Baseplate t	$N_{u,test}$	N_{test}^{h}	Rigid plate	Elastic pl. $N_{Rd,c,e}$	$N_{u,test}/N_{Rd,c}$; $N_{test}^{h}/N_{Rd,c,e}$		
	mm	kN	kN	kN	kN	Rigid plate	Elastic plate	Required
1	10	93	22,84	90,70	10,1	1,03	2,27	mean 2,0 min. 1,5
2	15	94	19,77	90,70	10,1	1,04	1,96	
3	20	92	17,32	90,70	10,1	1,01	1,72	
				Mean value of 3 tests		1,03	1,98	2,0
				Minimum		1,01	1,73	1,5

The test recalculations in Tab. 4-7 show that the additional proof is conservative for verification of non-linearly distributed anchor tension forces in the anchor group with mean safety factor of 1.98 ≈ 2.0 while the rigid baseplate assumption is clearly unconservative with mean safety factor of 1.03, much lower than required 2.0.

3.1.3 Design example based on EN 1992-4 [1]

For the design example in Fig. 5 using headed studs from ETA-03/0039, the calculated design resistance of the anchor group at concrete cone failure without considering the additional proof satisfies the requirement with the utilization $\beta_{N,c}$ =0.555 <1.0 (Tab. 8). As shown in Fig. 8a), the anchor tension force distribution is clearly not in a plane. The verification of concrete cone resistance needs to be carried out by the additional proof according to equations (3.1) and (3.2). which result in an utilization $\beta_{N,c}$ =1.228 >1.0 (Tab. 9).

Tab.8 Proof of concrete cone failure without considering the non-linear anchor tension force distribution [25]

$N_{Rk,c}=N^0_{Rk,c} \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N} \cdot \psi_{M,N}$ $\quad$ $N^0_{Rk,c} = k_1 \cdot (f_{ck})^{0.5} \cdot h_{ef}^{1.5}$ [N] $\quad$ $\psi_{A,N} = A_{c,N}/A^0_{c,N}$ $\quad$ $N_{Rd,c} = N_{Rk,c}/\gamma_{Mc}$

$N^0_{Rk,c}$ [kN]	$A_{c,N}$ [mm²]	$A^0_{c,N}$ [mm²]	$\psi_{A,N}$	k_1	γ_{Mc}	h_{ef} [mm]	$s_{cr,N}$ [mm]	$c_{cr,N}$ [mm]
170.598	977114	467856	2.088	8.9	1.5	228.0	684.0	342.0

$\psi_{s,N}$	$\psi_{re,N}$	$e_{N,x}$ [mm]	$e_{N,y}$ [mm]	$\psi_{ec,N,x}$	$\psi_{ec,N,y}$	$\psi_{ec,N}$	$\psi_{M,N}$	$N_{Rk,c}$ [kN]	$N_{Rd,c}$ [kN]	N_{Ed} [kN]	$\beta_{N,c}$
0.834	1.0	0.0	56.6	1.0	0.858	0.858	1.0	254.840	169.893	94.322	0.555

Tab. 9 Additional proof of concrete cone failure considering the non-linear anchor tension force distribution [25]

$N^0_{Rk,ec} = N_{Rk,c} / \psi_{ec,N}$ $\quad$ $N_{Rk,ec} = N^0_{Rk,ec} / n$ $\quad$ $N_{Rd,ec} = N_{Rk,ec}/\gamma_{Mc}$

γ_{Mc}	$N_{Rk,c}$ [kN]	$\psi_{ec,N}$	$N^0_{Rk,ec}$ [kN]	n	$N_{Rk,ec}$ [kN]	$N_{Rd,ec}$ [kN]	N_{Ed} [kN]	$\beta_{N,ec}$
1.50	254.840	0.858	297.035	5	59.407	39.605	48.625	1.228

3.2 Baseplate subjected to shear loading

3.2.1 Additional proof for concrete failure resistance

For narrower baseplate with width/length (w/l) <1/3, e.g. w/l=1/5 as shown in Fig. 21, the rigidity of the baseplate under shear action may not be enough for using the design method according to [1, 2] directly, because the real anchor shear force in critical cases can deviate from the linear distribution significantly.

Tab. 10 shows the comparison of calculated anchor shear force distribution between rigid baseplate assumption and elastic baseplate. On anchor #2, the realistic anchor shear force from the elastic baseplate amounts to about 20% higher than that from the rigid baseplate assumption.

For this case, the following additional proofs may be proposed for pry-out failure and concrete edge failure modes by modifying the load eccentricity factor for these failure types respectively, e.g. for edge failure as follows.

$$\psi_{ec,V} = 1.0 \quad (3.3)$$

$$V_{Ed}^{h} \leq V_{Rd,c}/n \quad (3.4)$$

with $\psi_{ec,V}$: Reduction factor to consider the load eccentricity in the anchor group in equation (7.47) of [1] and (17.5.2.1b) of [2]

V_{Ed}^{h}: Highest anchor shear force in the group

$V_{Rd,c}$: Design resistance of the anchor group at concrete edge failure with $\psi_{ec,V} = 1,0$

n: number of anchors in shear in the group

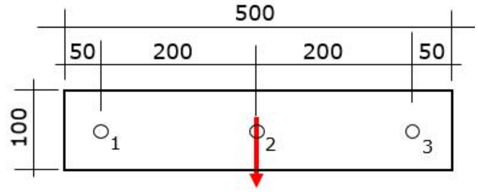


Fig. 21 Baseplates (t_{fix}=15mm for M16, t_{fix}=30mm for M30) used for the calculations in Tab. 10

Tab.10 Comparison of calculated shear force distribution between rigid and elastic baseplate with w/l=1/5 (without gap, Fig. 21)

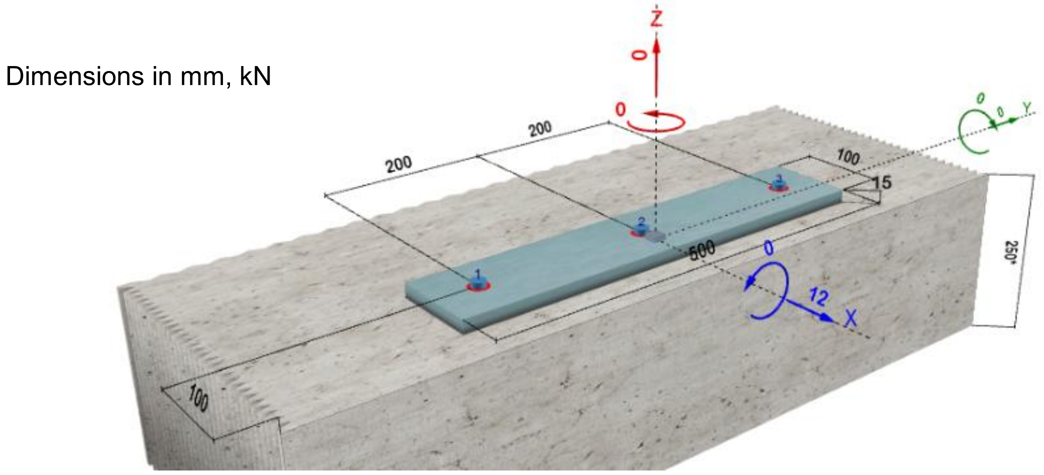
Size	Baseplate	Plate rigidity	Shear force [kN] in anchor #		
			1	2	3
M16	500x100x15	Rigid	4.0	4.0	4.0
		Elastic	3.59	4.82	3.59
M30	500x100x30	Rigid	4.0	4.0	4.0
		Elastic	3.62	4.76	3.62

M16 h_{ef} = 125 mm; M30 h_{ef} = 300 mm

3.2.2 Design example based on EN1992-4 [1]

The proposed additional proofs for non-rigid baseplate under shear loading are demonstrated by a design example shown in Fig. 22 with the anchor shear forces displayed in Tab. 10.

The calculations of concrete failure resistances according to EN 1992-4 [1] with rigid baseplate assumption are shown in Tab. 11 for pry-out failure and in Tab. 12 for edge failure respectively. The results calculated using recommended additional proofs are appended to each table.



Cracked concrete C20/25, bonded anchor M16, h_{ef} =125 mm with gap filling

Fig. 22 Basic anchorage data of the design example [25]

Tab. 11 Calculation of pry-out failure resistance without considering the non-linear anchor force distribution [25]

Pry-out failure

$N_{Rk,c}=N^0_{Rk,c} \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,V,cp}$ $\quad N^0_{Rk,c} = k_1 \cdot (f_{ck})^{0.5} \cdot h_{ef}^{1.5}$ [N] $\quad \psi_{A,N} = A_{c,N}/A^0_{c,N}$ $\quad V_{Rk,cp}=k_8 \cdot N_{Rk,c}$ $\quad V_{Rd,cp} = V_{Rk,cp}/\gamma_{Mc}$

$N^0_{Rk,c}$ [kN]	$A_{c,N}$ [mm²]	$A^0_{c,N}$ [mm²]	$\psi_{A,N}$	$\psi_{s,N}$	$\psi_{re,N}$	h_{ef} [mm]	$s_{cr,N}$ [mm]	$c_{cr,N}$ [mm]	k_1	k_8	γ_{Mc}
48.125	222813	140625	1.584	0.86	1.0	125.0	375.0	187.5	7.7	2.0	1.5

$e_{V,cp,x}$ [mm]	$e_{V,cp,y}$ [mm]	$\psi_{ec,V,cp,x}$	$\psi_{ec,V,cp,y}$	$\psi_{ec,V,cp}$	$N_{Rk,c}$ [kN]	$V_{Rk,cp}$ [kN]	$V_{Rd,cp}$ [kN]	V_{Ed} [kN]	$\beta_{V,cp}$
0.0	0.0	1.0	1.0	1.0	65.576	131.152	87.435	12.000	0.137

Result calculated with additional proof analogous to equation (3.3) and (3.4):

$\beta_{V,cp} = V_{Ed}^{\ h} / (V_{Rd,cp}/n) = 4.82/(87.345/3) = 0.166$

Tab.12 Calculation of edge failure resistance without considering non-linear anchor shear force distribution [25]

Concrete edge failure, direction x+

$V_{Rk,c} = V^0_{Rk,c} \cdot \psi_{A,V} \cdot \psi_{s,V} \cdot \psi_{h,V} \cdot \psi_{\alpha,V} \cdot \psi_{ec,V} \cdot \psi_{re,V}$ $\quad V^0_{Rk,c} = k_9 \cdot d^{\alpha} \cdot l_f^{\beta} \cdot (f_{ck})^{0.5} \cdot c_1^{1.5}$ [N] $\quad \psi_{A,V} = A_{c,V}/A^0_{c,V}$ $\quad V_{Rd,c} = V_{Rk,c}/\gamma_{Mc}$

$l_f = \min(h_{ef}, 12d)$ $\quad \alpha = 0.1 \cdot (l_f / c_1)^{0.5}$ $\quad \beta = 0.1 \cdot (d / c_1)^{0.2}$

h_{ef} [mm]	k_9	f_{ck} [N/mm²]	γ_{Mc}	c_1 [mm]	c'_1 [mm]	α	β	$V^0_{Rk,c}$ [kN]	$\psi_{s,V}$	d [mm]	l_f [mm]
125.0	1.7	20	1.5	100.0	-	0.112	0.069	14.486	1.000	16.0	125.0

$A_{c,V}$ [mm²]	$A^0_{c,V}$ [mm²]	$\psi_{A,V}$	$\psi_{h,V}$	$\psi_{\alpha,V}$	e_V [mm]	$\psi_{ec,V}$	$\psi_{re,V}$	$V_{Rk,c}$ [kN]	$V_{Rd,c}$ [kN]	V_{Ed} [kN]	$\beta_{V,c}$
105000	45000	2.333	1.000	1.000	0.0	1.000	1.000	33.800	22.533	12.000	0.533

Result calculated with additional proof according to equation (3.2) and (3.3):

$\beta_{V,c} = V_{Ed}^{\,h} / (V_{Rd,c} / n) = 4.82/(22.533/3) = 0.642$

3.3 Baseplate subjected to combined tension and shear loading

To illustrate the effect of the elastic baseplate model with proposed additional proof on the verification of anchor group under combined tension and shear loading, a well-documented design example is selected to compare the results between the rigid and elastic baseplate methods.

The example has been extracted from [14] and shown previously in Fig. 5, all design checks based on EN 1992-4 [1] regarding tension, shear and combined tension and shear loading with and without recommended additional proofs are displayed in Tab. 13, 14 and 15 respectively.

For cast-in-place headed studs in the design example under shear load, it is assumed that the concrete under the baseplate is compacted well and there is no gap around the studs. The shear load is distributed uniformly on the 8 studs. After calculation of edge failure resistances of each anchor rows [24], the back-anchor row is decisive for the design (Tab. 14).

Tab. 13 Check of tension resistance based on [1] with proposed additional proof

Tension loading	Related anchor	Action [kN]	Resistance [kN]	Utilization [%]	Status
Steel failure	7	48.625	119.333	40.7	√
Pull-out	7	48.625	87.833	55.4	√
Concrete cone failure	4,5,6,7,8	94.322	169.893	55.5	√
Concrete cone failure e *)	7	48.625	39.605	**122.8**	X

*) additional proof for the fastening with elastic baseplate

Tab. 14 Check of shear resistance based on [1] with extension according to [24]

Shear loading	Related anchor	Action [kN]	Resistance [kN]	Utilization [%]	Status
Steel failure (without l. arm)	1,2,3,4,5,6,7,8	11.125	85.600	13.0	√
Pry-out	1,2,3,4,5,6,7,8	89.000	428.339	20.8	√
Concrete edge failure (y-)	1,2,3,4,5,6,7,8	89.000	103.025	**86.4**	√

Tab. 15 Check of combined tension and shear resistance based on [1] with proposed additional proof

Combined tension and shear loading

- With consideration of related anchors and separation of steel and concrete failure

	Anchor	Tension (β_N)	Shear (β_V)	Condition	Utilization [%]	Status
Steel	7	0.407	0.130	$\beta^2_N + \beta^2_V \leq 1.0$	18.3	√
Concrete	7	1.228	0.864	$\beta_N + \beta_V \leq 1.2$	**174.3**	X

As shown in Tab. 13, the utilization of the anchorage under tension loading meets the current code requirements with max. β_N =55.5% < 100%. However, considering the elastic behavior of the baseplate and with the recommended additional proof, the max. calculated β_N =122.8% > 100%, results in an unsafe design.

The utilization of the anchorage under combined tension and shear loading with the recommended additional proof (Tab. 15) exceeds the design requirement significantly with

$(\beta_N + \beta_V)/1.2 = (1.228+0.864)/1.2 = 1.743/1.2= 145.3\% > 100\%.$

4. Conclusions

For elastic design of anchorages in concrete, the anchor tension and shear forces under design actions on baseplate need to be determined with sufficient accuracy [1]. The calculations of tested anchor groups in [3, 7] show that the 3D FEA considering the stiffness of anchors, concrete, baseplate and attached profile can simulate the behavior of steel-to-concrete connections accurately. With this simulation method, the anchor tension and shear forces can be determined realistically. Comparing the anchor force distributions resulted from the realistic 3D FEA with those based on the rigid baseplate assumption, the baseplate rigidity can be evaluated.

For baseplate under tension load and bending moment, the anchor tension stiffness at SLS determined by pull-out test in uncracked concrete is decisive for the elastic design. In absence of anchor tension stiffness from tests, a general anchor tension stiffness depending on anchor types, effective anchorage depth and concrete strength has been proposed.

The main influence parameters on the baseplate rigidity are the thickness of baseplate, the attached profile/stiffeners and the anchor stiffness. The action load does not have so high influence on the baseplate rigidity.

The linear strain distribution under baseplate is an unrealistic assumption because an unpractical baseplate thickness will be required to satisfy the rigidity level imposed by this assumption.

For most practical cases, non-linear anchor tension force distribution will take place. To use the design methods according to [1, 2], an additional proof is necessary for verifying the concrete cone failure of anchor groups. This additional proof has been recommended in [3,4] and verified in this paper by 40 laboratory test results with anchor groups under centric and eccentric tension load on the baseplates. For anchor groups with combined tension load and bending moment, this additional proof may be conservative. The additional proof can extend the elastic anchorage design based on rigid baseplate assumption to non-linearly distributed anchor tension forces and eliminate the safety gaps in [1, 2] resulted from the rigid baseplate assumption.

The required baseplate rigidity under shear load may be defined by the geometrical conditions with narrowness and thickness, i.e. $w/l \geq 1/3$ and $t_{fix} \approx d$. For narrow baseplate $w/l<1/3$ and $t_{fix} < d$, an addition proof has been recommended.

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• 1st - Plane sections do not remain plane

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