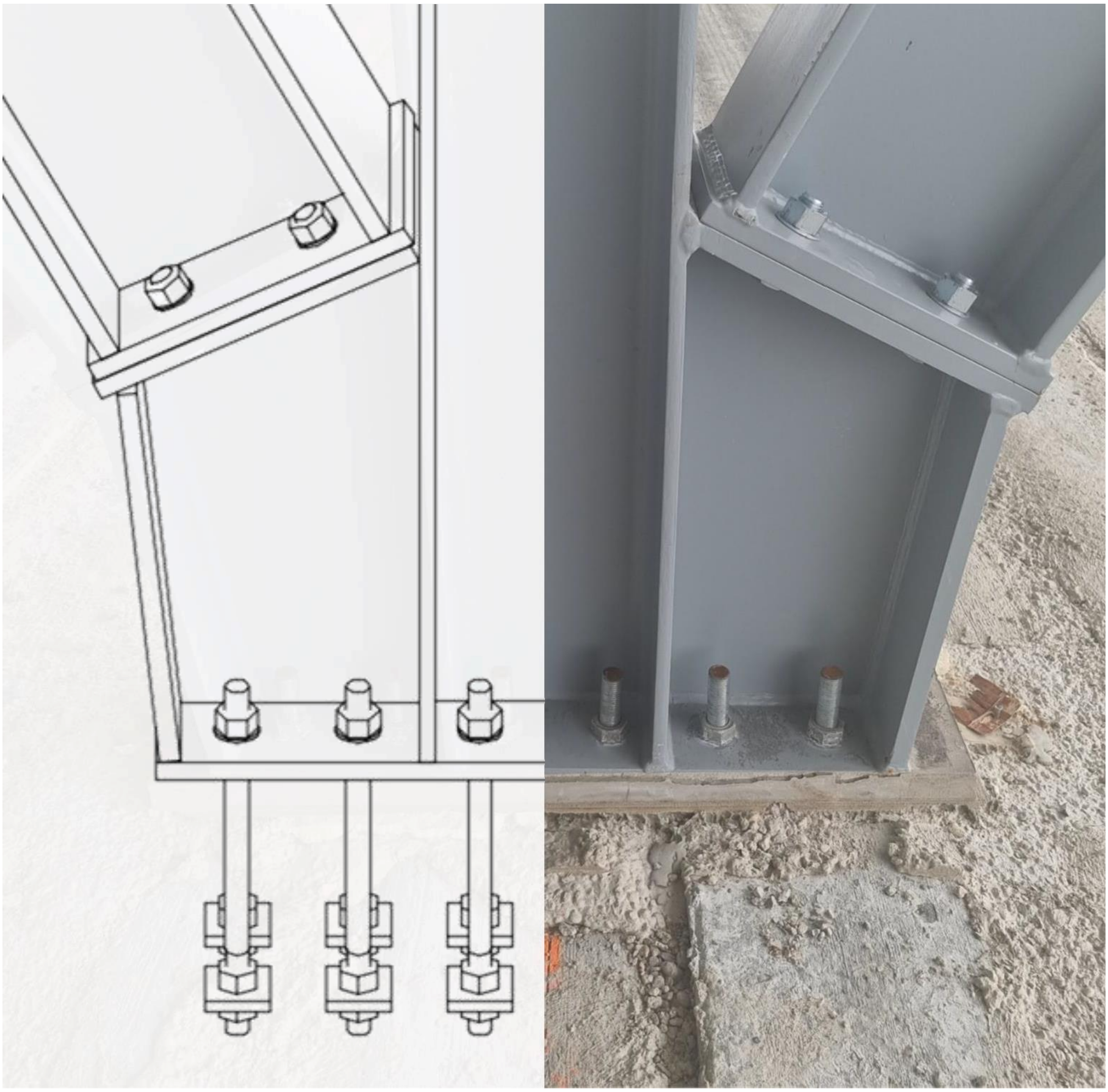
BV/BmS Opleidingen samenwerking Betonvereniging en Bouwen met Staal

Graduate research

SHEAR CAPACITY OF ANCHORS WITH MORTAR-FILLED ADJUSTMENT SPACE



Student/number: Arco de Gelder - 17803

Three Formulas for the Steel Engineers under the sky,
three for the Concrete Engineers in their halls of stone,
Two for the Engineers over sea,
one for the Fisherman to the east
In the Land of Construction where the Shadows lie.
One Formula to rule them all, One Formula to find them,
One Formula to bring them all and in the darkness bind them
In the Land of Construction where the Shadows lie.

(After J.R.R. Tolkien's epigraph of *The Lord of the Rings)*

Foreword

About ten years ago Fabian de Vos (Hilti) asked me how I dealt with the adjustment space at anchors. I had some vague ideas, but nothing concrete. That's where the seed for this research was planted. In recent years the issue has not let go of me, partly because the question is occasionally asked by controlling parties. I lacked the time and especially the knowledge to do proper research into this. Partly due to the course in structural design from BV/BmS, I was able to give this issue a try. It is one of the many issues that have been dormant for years among many (detailed) structural engineers and that is one of the reasons why I have tackled this issue in order to contribute my share to the development of the profession.

Reading Guide

After the introduction in Chapter 0, the report is laid out in five sections:

- Theory - Chapters 2 and 3 discuss the theory of the shear capacity of anchors for both without and with set space. These chapters also provide an overview of the current equations available to determine the shear capacity of anchors.
- *Practice* - Chapter 4 presents the current practice of anchor connections. Based on a survey and using current calculation software, this was mapped out. This chapter also laid the foundation of the calculation model assumptions.
- Experiments - In Chapter 5, the experimental results of L. Bouwman et al [1], K. Mcbride [2] and R. Mallée [3] were juxtaposed with current standards and theory. What do these results say about the method of computation and what differences stand out? From these tests the first contours of a responsible upper and lower limit for the shear capacity of anchors follow.
- Computational model - For Chapters 6 and 7, a finite element method model (hereafter EEM model) was set up based on the mechanics, current practice and assumptions of the tests with the aim of properly simulating the behavior of shear-loaded anchors with a mortar joint. If this model shows a behavior consistent with the mechanics and the tests, a responsible computational value of the shear capacity of anchors can be determined. These sections describe the construction and results of the EEM model.
- Justification - At the end of the paper, Chapter 8 answers the main and sub-questions and describes the final conclusion, proposal and some recommendations.

When citing sources, if necessary, after the reference number, the page or paragraph number is given in the form of p.X. In the graphs, in the legend, numbers of the equations from this paper are cited in the form of (x).

Thanksgiving

This research did not come about by me alone.

I thank God for all he has given me in health, talent, perseverance, the people around me and most of all his indispensable grace!

I thank my wife Sanne who made it possible for me to pursue my studies with our family in this way, always had to be understanding and patient about it and did. Thank you for who you are! My sister Petra for helping me to write and check this document and all the other pieces I have written over the years for my study(s) and everything we have been privileged to do together. My colleague Cees den Dunnen (ADS) who for years has been my mentor and sounding board for everything work- and study-related and under whom I was able to grow from intern to the engineer I am today. Thanks for the space and time I have received from ADS for my studies and research in all these years.

Regarding the research, I would like to thank René Braam in his role as study supervisor for the fine coaching, his contacts within TU Delft and the sounding board. Ab van der Bos (Diana) and Pim van der Aa (Diana) for their help in preparing the EEM model and providing the software. Marcel van Odenhoven for talking about this topic, reading along and sorting out and sending various documents. Floris de Lange (Fischer) and Jan Poppe (Fischer) with the help regarding the Fischer connect-it article. Furthermore, I thank Marjan Pols (Hilti), Coert Domen (Van Rossum), Bastiaan Hoefnagel (Pelecon), Henk van Vliet (Pelecon), Dick van Gemerden (PT) who all contributed in their own way in providing and finding documents. I also thank all respondents of my survey and everyone from my work, study and private who I was allowed and able to harass for six months about my graduation.

And finally, the most important, the man who broke open the cupboard in the basement of TU Delft with a crowbar because it contained a Stevin research that had been sort of lost and was crucial to my research: thanks!

Capelle aan den IJssel, 2022
Arco de Gelder

Summary

This report investigates the influence of a setting space filled by casting mortar on the shear capacity of a steel anchor. Within the current codes NEN-EN 1993-1-8 and NEN-EN 1992-4 various methods with different boundary conditions are described and this causes in practice when determining this shear capacity ambiguities and discussions what the correct method is. In this study the equations from the current codes are described, what the differences between them are, how constructors deal with this in daily practice and which equations are used by the various calculation software. The equations show that the shear capacity must be reduced by the headroom and this can best be captured in a β-factor. The research further focuses on the description of β. For this description, daily practice, safety and simplicity of design and the structural behavior of the whole connection are taken into account.

Underlying the comparisons are past experiments conducted by the Stevin Laboratory at TU Delft, Fischer and the University of Florida, among others. These results have been examined and compared with each other and the prevailing codes. For a broader picture, the comparison and an underlying experiment for the determination of shear capacity of bolts when using steel fills in steel-steel joints has also been included. Here, the same mechanical behavior takes place.

In order to visualize the complex behavior of the interplay of forces in the anchor caused by the shear force and the moment created by the setting space, an EEM model was created in Diana for this study. This model is consistent with the experimental results and the equations investigated and can therefore be used to draw conclusions. Several conservative assumptions were made for the boundary conditions of the model in order not to unjustifiably overvalue the actual shear capacity of the anchors. Some examples include: no adjusting nut under the footplate so that the longest bending length of the anchors is controlled, the friction factors according to the codes being lower than will have occurred in the tests, and a more brittle behavior of the anchor material than shown in tensile tests.

EEM model results show that from the current equation for shear capacity of anchors:

- NEN-EN 1993-1-8 is closest to the test results at the high adjustment space around 3*diameter and gives a safe underestimate at lower adjustment spaces.
- NEN-EN 1992-4 with mortar joint has various incorrect boundary conditions and is described with a decreasing β, which can be unwise due to the increase in the installation space caused by setting. During the design process one is not always sure what the exact height of the adjustment space after installation is.
- NEN-EN 1992-4 without a filled joint gives an unnecessarily low estimate of the shear capacity. Because this equation is known to be conservative, it is regularly ignored within, for example, calculation software. The subsequent failure to apply one of the aforementioned equations leads to a large overestimation of the shear capacity and should therefore not be applicable to headroom-filled joints.

From this investigation it follows the proposal to extend the current equation for shear of bolts and anchors according to NEN-EN 1993-1-8 Table 3.4 with a reduction factor β to which the following boundary conditions apply:

- From a joint height greater than 1/3*diameter, the shear capacity should be reduced.
- The mortar joint height must not exceed 3*diameter or 0.2*the smallest width of the base plate in accordance with NEN-EN 1992-1-1.
- Steel shims may only be used in steel-concrete connections if the connecting surfaces are parallel to each other and no open position is present on any side after installation. A maximum of three plates applies here.

If these boundary conditions are met, the following equations apply for the β-factor:

- For a filled gap by mortar joint or single steel shim

$$\beta = 0.745 - 0.0005 * {}_{fyb}$$

- for multiple steel shims

$$\beta = \frac{9d}{8d * 3tp}$$

This proposal allows the following equations to be dropped:

- NEN-EN 1993-1-8 equation 3.3
- NEN-EN 1993-1-8 equation 6.2
- NEN-EN 1992-4 section 7.2.2.3.1

There is room due to the conservative assumptions in the EEM model to introduce additional influence factors that accurately describe the behavior. Based on the experimental and modeling results, recommendations for this have been made that require further investigation.

Content

1. Introduction

When connecting steel to concrete, it is often necessary to provide adjustment space. Most people will be familiar with adjustment space under a column footplate where an adjustment nut is often used under the footplate. Below is perhaps a less familiar situation described. The situation below was the reason for investigating the ambiguity regarding the shear capacity of anchors with mortar-filled adjusting space.

When a precast concrete structure is installed, the executing and assembling parties have an allowable tolerance on the final position of +/- 20 millimeters (hereafter mm). On this basis, the product and its assembly are sold. Thus, when placing a steel beam or column against a precast concrete element, the possibility exists that the steel element may be 20 mm too long or 20 mm too short (Figure 1-1).

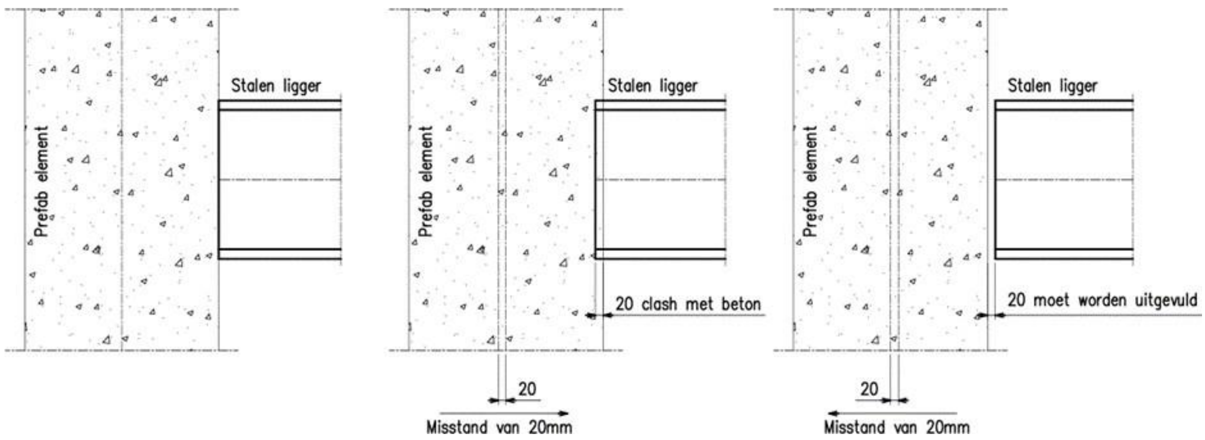


Figure 1-1 Steel against concrete without set space
From left to right: Neutral position/ wrongness toward the element/ wrongness away from the element

Applying 20 mm of adjustment space in the neutral position is a practical necessity in many situations. If the concrete element is offset by 20 mm, this distance is reduced to 0 mm or increased to 40 mm (Figure 1-2). The space between the steel beam and the concrete can be filled with specially designed mortar (Figure 1-3) or with steel plates. In such a situation, the anchors or bolts used to secure the steel element to the concrete cannot transfer the shear force directly to the concrete but must transfer the shear force across the setting space (shift of the shear force resulting in an eccentricity moment).

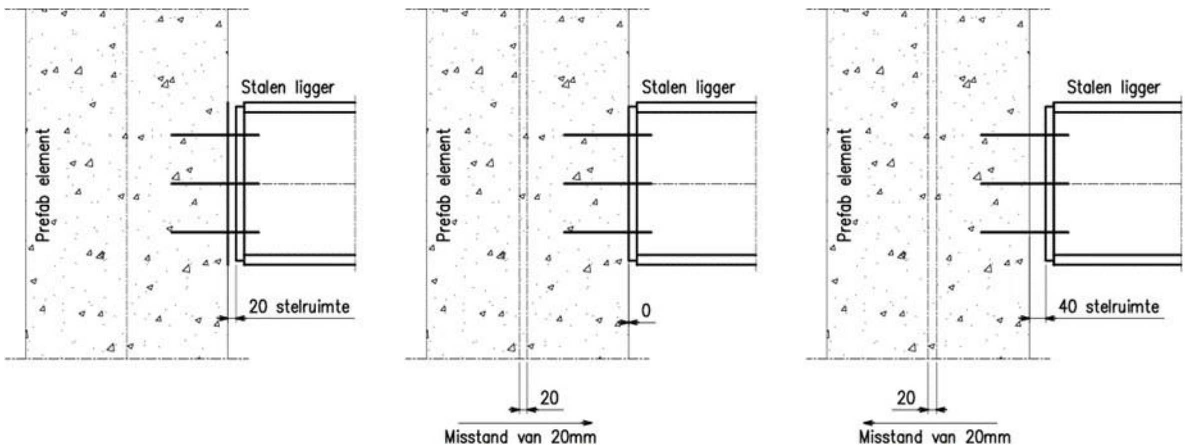


Figure 1-2 Steel against concrete with set space
From left to right: Neutral position/ wrongness toward the element/ wrongness away from the element

It is not always clear to structural engineers what a realistic value of the shear capacity of anchors with a filled headspace is. The level of shear capacity is the subject of a discussion in the literature, which has led to the coexistence in Europe of two standards in force that differ in terms of the shear capacity of anchors. NEN-EN 1993-1-8 [4] prescribes the shear capacity of anchors from an already older (1989) Stevin study [1]. NEN-EN 1992-4 [5], a fairly new (2018) standard, prescribes the shear capacity in which the influence of the mortar joint is not taken into account or under strict boundary conditions. This thesis research will look at this discussion and what can be a responsible use of the current calculation rules. Only the shear capacity of the steel anchor will be considered. Testing the shear capacity of concrete is outside the scope of this study.

An answer to the following question is sought within this study:
What reduction may or should be applied to the shear capacity of a steel anchor or bolt where the setting space after it has been filled out by mortar or steel plates?

In doing so, it examines:
- or NEN-EN 1993-1-8 equation 6.2 *shear capacity of anchors,* is in effect.
- Whether NEN-EN 1993-1-8 equation 3.3 *the reduction of shear capacity when shims are used,* can be used.
- with which the equations of NEN-EN 1992-4 section 7.2.2.3.1 apply.

Several sub-questions are relevant:
- Is the behavior of a bolt inserted into a sleeve similar to that of an anchor inserted or post-inserted?
- To what bolt and anchor diameters are the current equations applicable?
- is NEN-EN 1993-1-8 equation 6.2 convertible to an equation similar to NEN-EN 1993-1-8 equation 3.3, namely an equation in which the diameter of the anchor and the size of the setting space affect the reduction?
- Can a universal equation for the reduction of shear capacity for steel-steel and steel-concrete joints with an incompressible filler be described?
- how to deal with misalignment between steel-concrete? In case of misalignment, a different shape of the setting space occurs, resulting in a smaller setting space on one side than on the other side.
- What is the shear capacity of (collapsed) anchors at an adjustment space of more than 60 mm?

Figure 1-3 Box to steel-concrete joint removed after filling and curing

Definition of anchors and bolts
In the remainder of this study, "anchors" refers to all types of fasteners used in concrete, including sleeves into which a bolt is turned (Figure 1-4 a, b, c and d).

'Bolts' refers to the fastener between steel and steel (Figure 1-4 e).

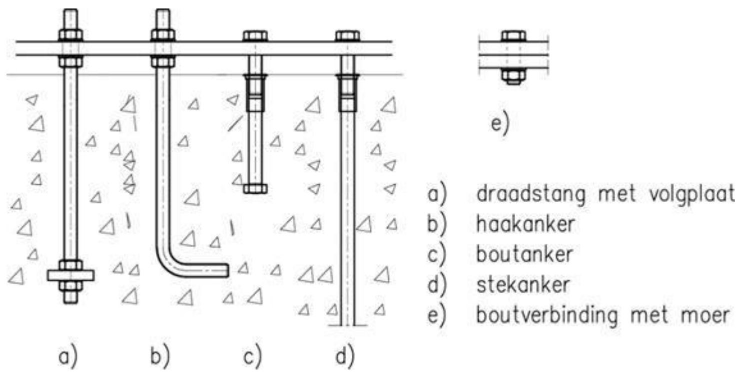


Figure 1-4 Types of fasteners

2. Shear force on anchors, backgrounds

2.1. Introduction

What is the actual shear capacity of an anchor? What happens when a gap is present between the steel and the concrete? What happens when this setting space is filled with mortar? What is the shear capacity of an anchor with a filled set space? Which of the current standards comes closest to the actual behavior? There are different ideas about this and that is the core and reason for the research.

In this chapter, Section 2.2 considers the concept of anchoring. Section 2.3 gives the scope of the study and the chapter concludes with in sections 2.4 and 2.5 how the shear capacity of a bolt or anchor is determined according to mechanics and standards. Chapter 3 discusses the situation that in joining to concrete, headroom is often used. Section 3.1 describes the influence of headroom on anchors. Section 3.2 describes the shear capacity of anchors without filled headroom and Section 3.3 describes that with filled headroom. The different shear capacities are visualized in Section 3.4, and Section 3.5 makes the connection between Chapters 2 and 3. In Section 3.6, a preliminary conclusion based solely on theory is drawn to give space to practice in Section 4.

2.2. Anchor

Parts, such as columns and beams, are fastened together within steel construction with welds and/or bolted connections. These are the two most common fasteners. The steel structure is also connected to the concrete in this way. This could be done by welding if a slab is poured in, but usually this is done by securing the steel structure with the help of anchors. In everyday practice, this is called anchoring. These anchors are either poured in beforehand or are installed afterwards. Holes are then drilled into the concrete and the anchors may be secured by glue or mechanically.

Fasteners can be loaded in a variety of ways. The three basic loads are a normal force in the form of tension or compression, a transverse force in the flat plane, and a bending moment (Figure 2-1). Torsion can also occur, but for bolt and anchor connections it generally translates into a shear force. When pressure (Nsd) is applied to the footplate, the anchors are unloaded and only the concrete is subjected to a compressive force [6, p. 71] and when a tensile force (Nsd) is applied, the anchor is pulled axially. This creates various interactions in the base plate in combination with the anchor and the concrete. Both the compressive force on the concrete and the tensile force on the anchors are otherwise beyond the scope of this study. During shear, a transverse force, through the anchors, is transferred from the steel footplate to the concrete. If there is no adjustment space between the footing plate and the concrete, the shear force will be transmitted directly, as with steel-steel connections.

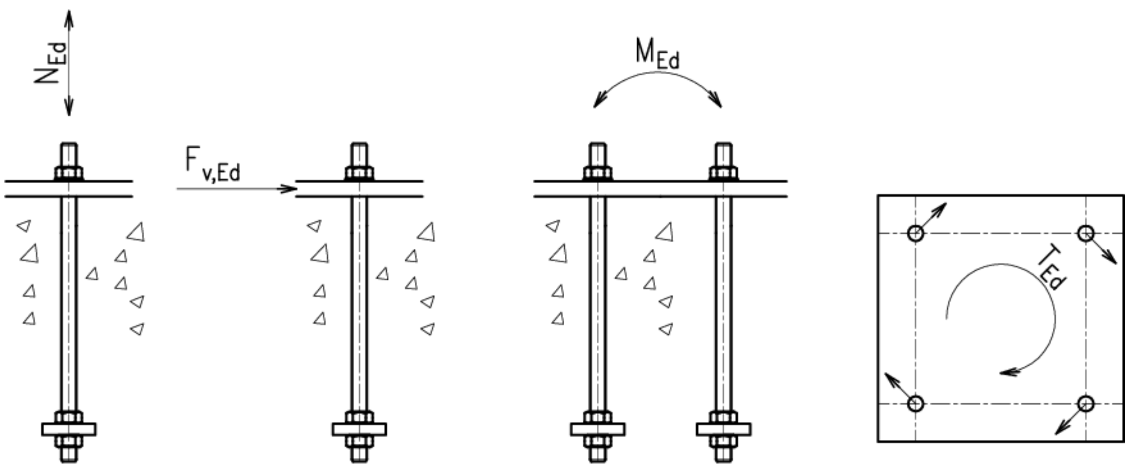


Figure 2-1 Possibilities of loading on anchors in the ideal situation that there is no adjusting space available

2.3. Delineation

This study examined common practice in the use of anchors. It includes the following:

Anchor class:	4.6 and 8.8		For these anchors, fub < 1000 N/mm^2
Anchor dimension:	M12-M36		
Standards:	EN 1993-1-8	[4]	
	NEN-EN 1992-4	[5]	

- When referring to the standard without specifying which one it specifically refers to, these two standards are concerned in this study [4] [5].
- The investigation focuses only on the failure of the steel anchor in the form of steel fracture, i.e., the limit that the maximum strain due to the occurring load has been reached. Checking the concrete for shear is outside the scope of this study.

- Many standards have their own symbols. For simplicity, when quoting equations, the notation according to [4] adhered to, see also the symbol list.
- In [4] and [5], the capacities are determined either directly or indirectly as computational values. The computational value is determined by dividing the characteristic capacity by a partial safety factor (material factor). For [4] this is γm2 and for
  [5] γMs. When determining the shear capacity of bolts and anchors of class 4.6 and 8.8, both are 1.25. To make things easy to compare with each other and to be in line with common practice, it was decided to express everything in the calculation value and keep the indices γm2 as partial safety factor.

2.4. Shear capacity of an anchor/bolt without adjustment space

The basis for determining the characteristic shear capacity of steel is as follows: the resistance to shear, also called shear capacity, is the allowable shear strength (τ) of the material over the surface that resists shear. The shear strength (τ) is proportional to the tensile strength (f_u) and, according to Von Mises' criterion, it is $f_u/\sqrt{3}$. This results in the following equation:

$$F_{v,k} = A_v * \frac{f_u}{\sqrt{3}} \tag{1}$$

In the current Eurocode, NEN-EN 1993-1-8, this equation is included in Table 3.4 as a calculation value for steel anchors and bolts in the following form:

$$F_{v,Rd} = \frac{\alpha_v * A_s * f_{ub}}{\gamma_{m2}} \tag{2}$$

$$A_s = \frac{\pi}{4} * d_s^2$$

Herein, γm2 is the aforementioned partial safety factor with a value of 1.25. In equation (2), the ratio "$f_u/\sqrt{3}$" is expressed as αv, which may have a different value in different situations, but in basic terms, this value for bolts and anchors is 0.6, which approximates 0.577 (1/√3). This value has also been examined using tests. These tests show an αv of 0.68, but because steel in practice has 110% to 120% of its assumed capacity, the value is reduced to 0.6 [7, p. 4.1.2.2. a]. Numerically, that looks like this: 0.68/120% = 0.566 and that approximates 0.577. Results of these tests can be found in the book by R. Eligehausen et al [7].

The value for the ratio of shear strength to tensile strength of anchors and bolts is widely accepted and can therefore be found in many literature and standards.

αv	=	0,6	[4, p. 3.6.1 table 3.4] [5, p. 7.2.2.3.1] (designation αv = k6) [8, p. 17.7.1.2]
αv	=	1/√3	[9, p. 10.3.3] [10, p. 30]

2.5. When does αv deviate?

Variations of αv are known within current standards for the different bolt and anchor classes.

For bolts, this is the case for an anomalous bolt class for which the tensile strength (fub) relative to yield strength (fyb) is significantly higher. As a result, αv decreases to 0.5 [4, p. 3.6.1 Table 3.4]. As a result, the basic assumption that the shear strength is equal to $f_u/\sqrt{3}$ does not hold and the reduction falls higher. The same happens for a non-deviating class when testing anchors according to NEN-EN 1992-4 [5]. In this standard, all classes of anchors with a permissible tensile strength greater than 500 N/mm^2 get a lower αv. So where within NEN-EN 1993-1-8 [4] for class 8.8 αv is equal to 0.6, within NEN-EN 1992-4 the αv for class 8.8 is reduced to 0.5. It is not clear where this lower estimate of this ratio for material with a tensile strength greater than 500 N/mm^2 comes from. The book by R. Eligehausen et al [7] also does not indicate this reduction. They only recommend a lower αv when placing multiple anchors in line. This is not a material reduction, but a placement reduction, and for clarity it might be better to reflect this in a separate index, as is done in ETAG-001 Annex C [11]. However, ETAG-001 Annex C equation 5.4 does have an αv of 0.5, but this applies to all classes of anchors. The ETAG-001 Annex C has since been replaced by NEN-EN 1992-4 and in this replacement of the standard, apparently αv = 0.6 if 'fub < 500 N/mm^2 ' and αv = 0.5 if '500 N/mm^2 ≤ fub ≤ 1000 N/mm^2 ' has been chosen.

αv can also deviate if anchors rather than bolts are tested according to NEN-EN 1993-1-8. αv then decreases because at anchors with headroom also a tensile force occurs in the anchors and here the shear/tensile strength ratio also incorporates the reduction due to the influence of headroom. This is further elaborated in sections 3.3.1 and 5.1.1.

3. Shear capacity of an anchor/bolt with adjustment space

3.1. The moment created by adjusting space

When steel members are connected against concrete, such a connection is often provided with adjustment space between the steel and concrete. In construction, there are accepted tolerances with which to build, and this means that a deviation may occur in the level dimensions and position of concrete elements, relative to an initial position (Figure 3-1). This is overcome by incorporating adjustability into the structure wherever possible. Under footplates of columns and between end plates of beams a free space is provided. In this space, footplates are often provided with a so-called adjusting nut, so that the structure can be adjusted in height to accommodate, for example, a gradient in the foundation.

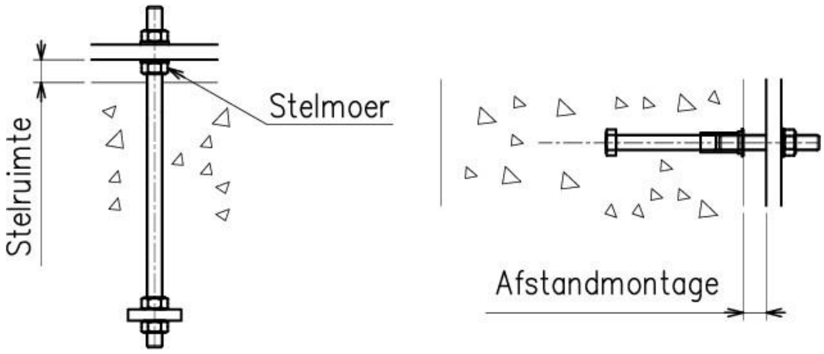


Figure 3-1 Vertical spacing to adjust structure to height/ horizontal spacing to accommodate misalignment and misalignment

Due to the distance mounting, a moment is created by the shear force. The moment is basically the force ($F_{v,Ed}$) across the arm (l_a) (Figure 3-2 a). The force herein is the shear force from the steel and the arm (hereafter bending length) is the distance from the center of the plate to a suitable depth in the concrete to transmit the shear force to the concrete. This distance, a kind of "notional clamping depth," is normally equal to 0.5*diameter (hereafter d) (a3). This bending length becomes shorter if a clamping nut is used. The clamping of the anchor is then directly at the top of the concrete and a3 then becomes equal to 0 mm (Figure 3-2 b). The bending length is defined this way in both [5] and [7].

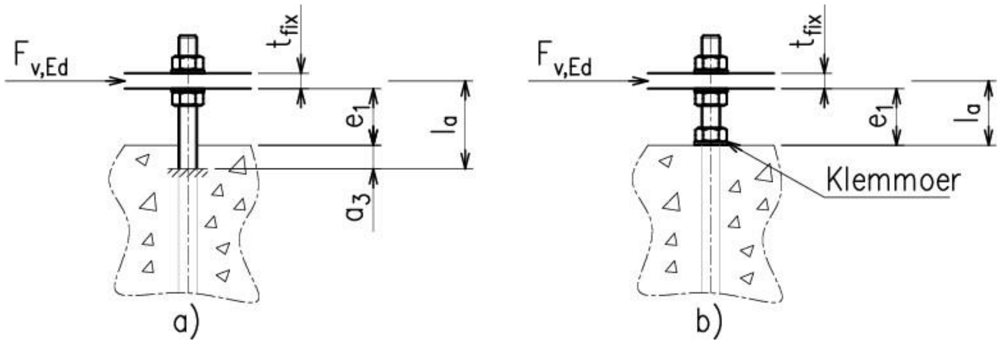


Figure 3-2 Definition of arm length of anchors in remote mounting

3.2. Shear capacity of an anchor without filled adjusting space

If no mortar is used to fill the gap, the moment created by the shear force can only be absorbed by the anchors themselves. In NEN-EN 1992-4 [5, p. 7.2.2.3.2.] this method is elaborated. The background of this method is the mechanical behavior of an anchor clamped on one or two sides (Figure 3-3).

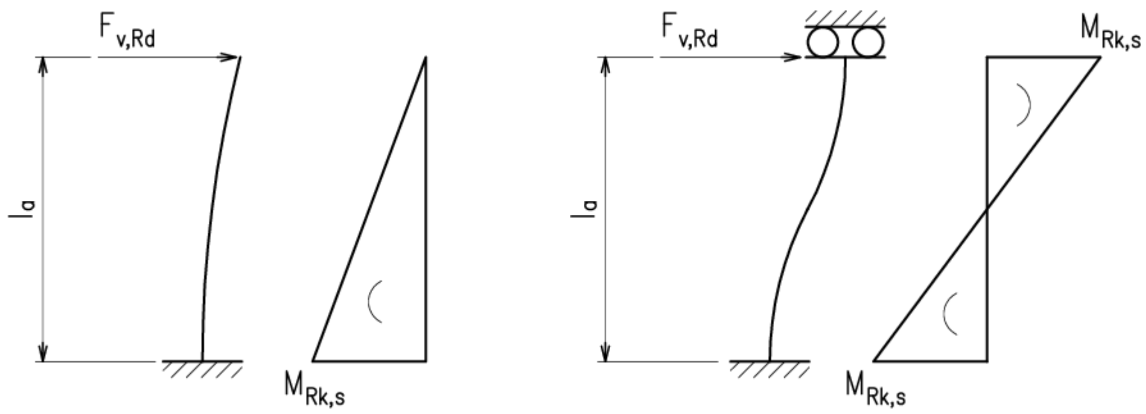


Figure 3-3 Deformation and moment line. For the situation on the left, αM = 1 and for the right, αM = 2

The shear force ($F_{v,Rd}$) required to create the moment ($M_{Rk,s}$) follows from the number of restraints (1 or 2) that resist bending along the length (l_a) between the restraints (Figure 3-3):

$$F_{v,Rd} = \frac{\alpha M * M_{Rk,s}}{l_a * \gamma_{m2}} \tag{3}$$

In the above equation, the length (la) is formed by the actual adjusting space + 0.5*tfix + 0.5*d. So this is an arithmetic headroom. With an arithmetic headroom of 45 mm, the actual headroom is around the common value of 30 mm.

The plastic moment resistance of a circular cross section has the same value in [4] and [5] for a plastic calculation:

$$M_{Rk,s} = 1.5 * W_{el} * f_{yb} \tag{4}$$

This value is derived from tests [7]. These tests show that 95% of the studs reach at least the plastic bending resistance at failure. 5% of these tests fall within the range of 90%-100% of the plastic bending resistance according to (4). Therefore, the assumption was made that 0.9*Wpl*fyb is the safe plastic bending resistance of a circular cross section. Between Wpl and W_{el} is a form factor that is approximately equal to 1.7 for a round cross section, see equation (5). If the form factor is entered into the assumption, equation (6) follows which is then rounded to the notation according to equation (4):

$$\alpha = \frac{W_{pl}}{W_{el}} = \frac{\frac{d_s^3}{6}}{\frac{\pi}{32} * d_s^3} = \frac{32}{6 * \pi} = 1.697 \tag{5}$$

$$W_{pl} = \frac{d_s^3}{6}; \; W_{el} = \frac{\pi}{32} * d_s^3$$

$$M_{Rk,s} = 0.9 * 1.7 * W_{el} * f_{yb} = 1.53 * W_{el} * f_{yb} \sim 1.5 * W_{el} * f_{yb} \tag{6}$$

There is no obvious reason why for the determination of plastic bending resistance the elastic bending resistance is taken as the starting point. Thus, for clarity and correctness, it seems logical to use the notation according to equation (7):

$$M_{Rk,s} = 0.9 * W_{pl} * f_{yb} \tag{7}$$

3.3. Shear capacity of an anchor with filled adjustment space

3.3.1. EN 1993-1-8

NEN-EN 1993-1-8 article 6.2.2. has the following equation for the shear capacity of anchors:

$$F_{v,Rd} = \frac{\alpha_{bc} * A_s * f_{ub}}{\gamma_{m2}} \tag{8}$$

$$\alpha_{bc} = 0.44 - 0.0003 * f_{yb}$$

αbc is almost the same index as αv in equation (2). Like αv, αbc indicates the reduction of the allowable tensile stress. In addition, αbc also incorporates the interaction with the set space.

This form of writing is a reformulation of the old equation from NEN 6772 [12, p. 11.7.2.3.3.] and has its origin in the Stevin study by L. Bouwman et al. [1]. Herein, a proposal for the maximum allowable shear force on anchors was made based on the interaction formula of tensile and shear force at anchors then in force. This equation was used because the theoretical model assumes that due to shear in the anchor and the deformation that the anchor undergoes, a compressive force is created in the mortar. For a correct balance and because the mortar joint cannot absorb a tensile force, the tensile force must be absorbed by the anchors. Thus, the sum of the tensile force in the anchors equals compressive force on the mortar. Thus, the tension in the anchor increases during spacer installation not only because of the shear force that occurs, but also because of the tensile force generated from the eccentricity moment. The maximum shear capacity will decrease as a result. This also follows from the Von Mises criterion for spatial stresses. The reduction factor αbc already incorporates the interaction of the shear capacity with the additional tensile force occurring. This is therefore the reason why with this equation the occurring shear load on anchors with set space does not have to be combined with an external occurring tensile force. "If an external tensile force is present, it will not directly increase the tensile stress in the anchor but will mainly have the effect of reducing the contact pressure between the footplate and the joint." [1, p. 41]. Section 5.1.1 elaborates on this further.

For bolts in steel-steel joints without set space, the occurring tensile and shear forces do need to be combined because in equation (2) the full stress cross-section is allocated to absorb the shear force. In
[4], the interaction formula of bolts in steel-steel joints on shear and tensile strength has been elaborated, but this is further beyond the scope of this study.

3.3.2. NEN-EN 1992-4

Distance mounting with mortar joint from 0.5*d

In NEN-EN 1992-4 article 7.2.2.3.1, a distinction is made on the basis of the height of the setting space. If the height is small enough, the shear capacity of the anchor does not have to be reduced. This applies to a mortar-filled setting space up to 0.5*d.

If the headroom increases, the shear capacity must be reduced according to (9) if it holds that:

- the concrete can be considered uncracked;
- at least two anchors are placed consecutively in the direction of shear force;
- there is no pulling force or moment acting on the footplate;
- the pitch between the anchors is at least 10*d;
- the mortar joint is less than or equal to 40 mm and less than 5*d;
- the entire footplate has a mortar joint; and
- the strength of the casting mortar is not less than 30N/mm^2 .

$$F_{v,Rd} = (1 - 0.01 * t_{mortel}) \frac{\alpha v * As * fub}{\gamma_{m2}} \tag{9}$$

The basic expression for shear capacity remained the same but an additional reduction is added based on tmortar. With this reduction, the shear capacity decreases as the setting space increases.

Distance mounting greater than 40 mm or 5*d

If the distance mounting exceeds 40 mm or 5*d or if any of the other boundary conditions are not met, then equation (9) is no longer applicable. NEN-EN 1992-4 then gives no other method for spacer mounting with mortar but refers to equation (3).

3.3.3. ACI-318-19

The U.S. Code [8] has the following equation according to Article 17.7.1.2.1 when a mortar joint is used:

$$F_{v,Rd} = 0.8 * \frac{\alpha v * As * fub}{\gamma_{m2}} \tag{10}$$

An 80% reduction is added to the allowable shear capacity of an anchor when a mortar joint is used. No maximum height for the mortar joint is given. No distinction is made within ACI-318-19 between the different classes of anchors, as is the case in both Eurocodes, and therefore αv = 0.6 always applies.

3.3.4. Shear capacity of bolts with padding by steel plates

NEN-EN 1993-1-8 article 3.6.1 (12) provides guidelines for reducing the shear capacity of bolts when fillers are used in steel-steel connections. There may be a correlation between steel-steel and steel-concrete connections. Therefore, this comparison is included in this study:

$$F_{v,Rd} = \beta_p * \frac{\alpha v * As * fub}{\gamma_{m2}} \tag{11}$$

$$\beta_p = \frac{9d}{8d + 3t_p}$$

This equation should be used when shims are used that together are thicker than 1/3*d.

3.4. Visual display comparisons

To get a picture with all the previous comparisons, they are shown in the graphs below for both M20 4.6 (Figure 3-4) and M20 8.8 (Figure 3-5).

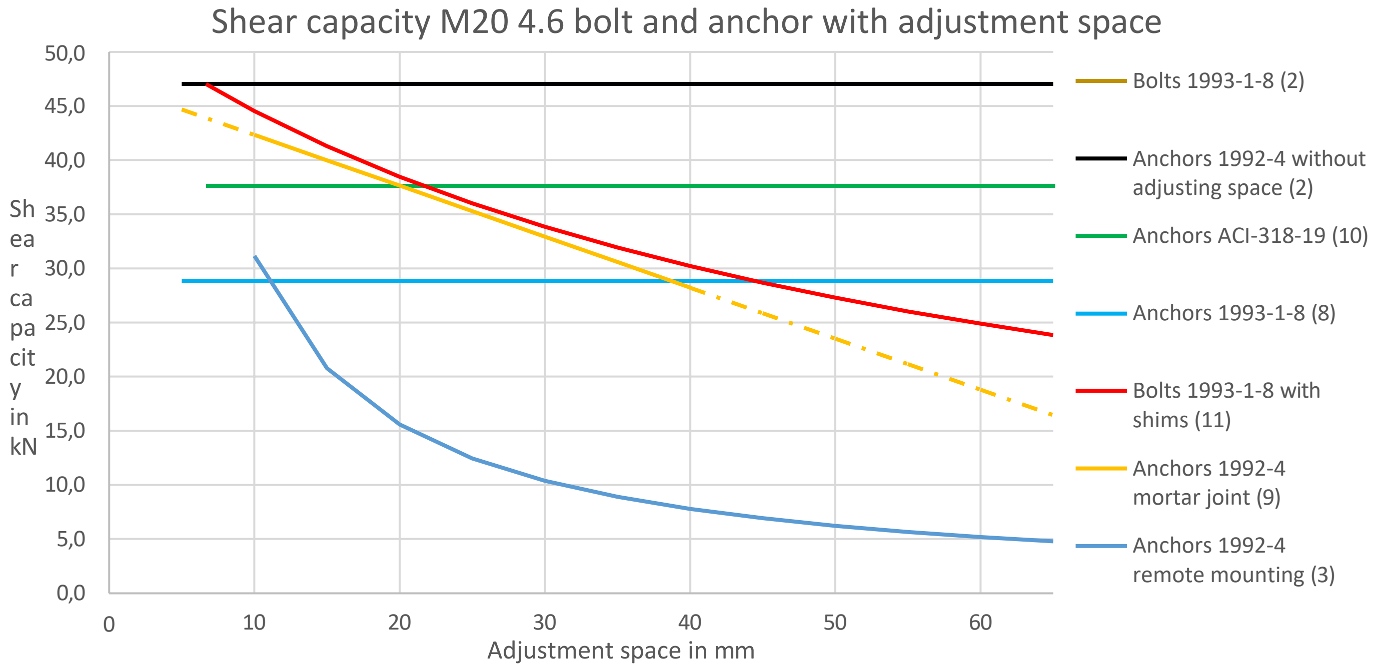


Figure 3-4 Shear capacity M20 4.6 bolt and anchor with adjustment space

The base values of bolts and anchors (brown and black) without set space are given as a reference and overlap for class 4.6. This is because in both standards [4] [5] αv is equal to 0.6. These are not in effect when applying spacer mounting but show the maximum allowable shear capacity of the cross-section without adjusting space, which may be worked with according to [4] [5].

The following stand out:

- The shear capacity of anchors according to equation (9) and (11) give almost the same results. In (9) the setting space is filled by casting mortar and in (11) by steel plates.
- The shear capacity of anchors according to equations (2) and (10) is constant and is independent of mortar thickness.
- The shear capacity of anchors according to equation (3) is the lower limit of the current codes. Thereby, equation (3) assumes an arithmetic actuating space greater than the actual actuating space, see section 3.2. Thus, the shear capacity of equation (3) at an arithmetic headroom of 45 mm is comparable to the shear capacity of other equations at an actual headroom of 30 mm. The shear capacity according to equation (3) at this actuating space is more than four times lower than the shear capacity according to the three closest equations (8), (9) and (10)(11).

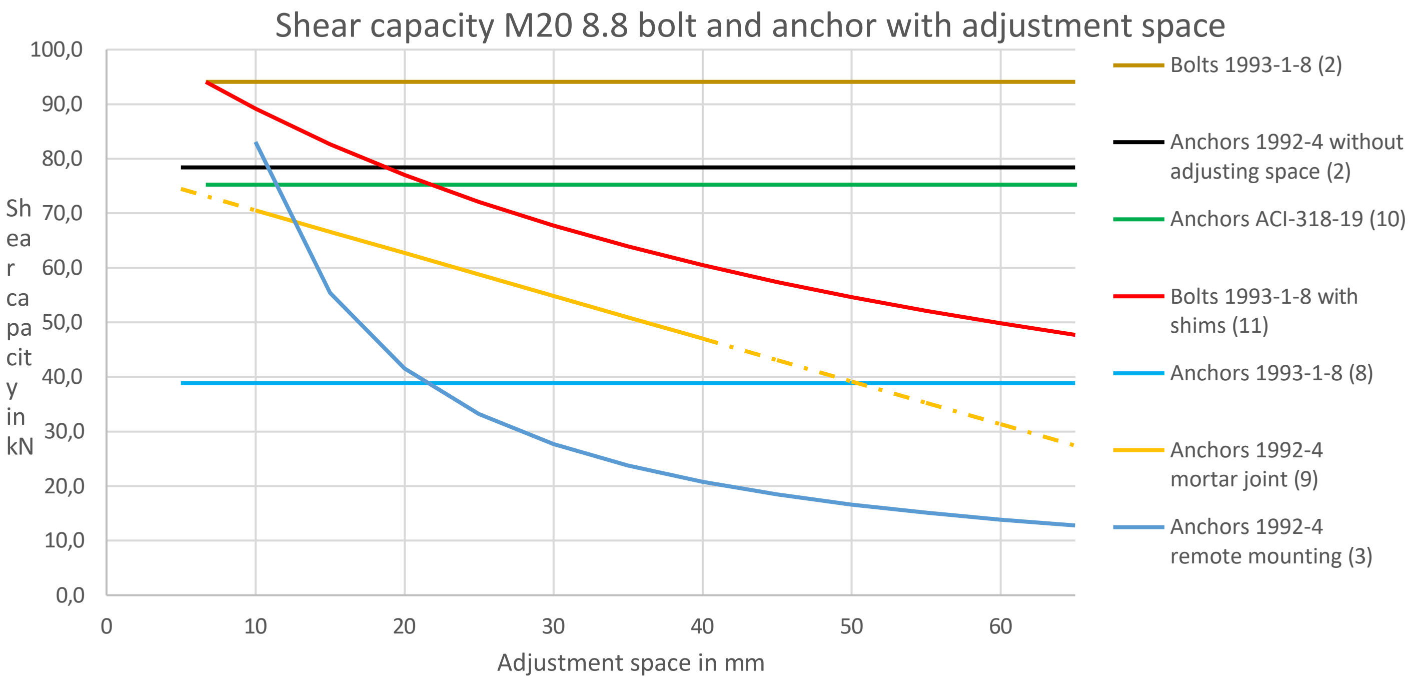


Figure 3-5 Shear capacity M20 8.8 bolt and anchor with adjustment space

The following stand out:

- The base shear capacity of Class 8.8 anchors according to NEN-EN 1992-4 is 20% lower than that of NEN-EN 1993-1-8 because, according to NEN-EN 1992-4, αv then equals 0.5 instead of 0.6.
- The shear capacity of anchors according to equations (9) and (11), in which the setting space is filled by casting mortar or steel plates, differ by 20% as with bolts. For the same αv (as for class 4.6), these equations would give almost the same results.
- The shear capacity of anchors with an adjustment space according to equation (8) is comparatively lower for class 8.8 than for class 4.6. This is because αv in this equation is lower for class 8.8 than for class 4.6:
  - αv for class 8.8 = 0.44 - 0.0003*640 = 0.25
  - αv for class 4.6 = 0.44 - 0.0003*240 = 0.37
- The shear capacity according to equations (2) and (10) is constant and is independent of mortar thickness.
- The shear capacity of anchors according to equation (3) is the lower limit of the current codes. Thereby, equation (3) assumes an arithmetic actuating space greater than the actual actuating space, see section 3.2. Thus, the shear capacity of equation (3) at an arithmetic headroom of 45 mm is comparable to the shear capacity of other equations at an actual headroom of 30 mm. At this actuating space, the shear capacity according to equation (3) is more than twice as low as the shear capacity according to equation (8), three times as low as that of equation (9), and four times as low as that of equations (10) and (11).

3.5. Coherence of chapters 2 and 3

What is a safe value for the shear capacity of anchors or bolts with mortar-filled adjustment space? There are large arithmetic differences between the capacities of different equations, and these even increase to a factor of 4 (Figure 3-4).

The calculation method according to NEN-EN 1992-4, which is based on ETAG [11] and explained in the book by R. Eligehausen et al. [7], determines the shear capacity of an anchor based only on the bending capacity without considering the mortar present, see equation (3). This is considered conservative by various parties and structural engineers [13] [2]. But [7] mentions that the shear capacity according to the report of Bouwman et al. [1] then again cannot be adopted because the implementation of mortar joints is not reliable enough to state that no bending will occur in the anchors.

An article by Hilti [13] shows the differences between equation (3) and (9). Figure 3-6 shows that from equation (9) *shear capacity of anchors with a mortar-filled joint*, it can be seen that up to 40 mm mortar joint height more shear capacity can be assigned. In equation (9), height affects the shear capacity linearly. This is also incorporated in equation (11) *the reduction in shear capacity of bolts with steel fills*, and in equation (3) *bending resistance of anchors without mortar joint*. It can also be seen that the increased shear capacity stops abruptly at 40 mm. So what about this for different anchor diameters? Is the decrease in shear capacity, at an absolute mortar joint height, for an M16 bolt the same as for an M36 bolt? Or can it be expected that larger joint heights would be permissible for larger diameters? And why at 40 mm would the shear capacity of the anchor suddenly drop back to that of an anchor loaded only on bending without considering the mortar joint?

For the equations described, all kinds of limits arise where the equations are no longer applicable or become out of context. It would therefore be desirable if the actual shear behavior of mortar-filled spacing of footing slabs could be described.

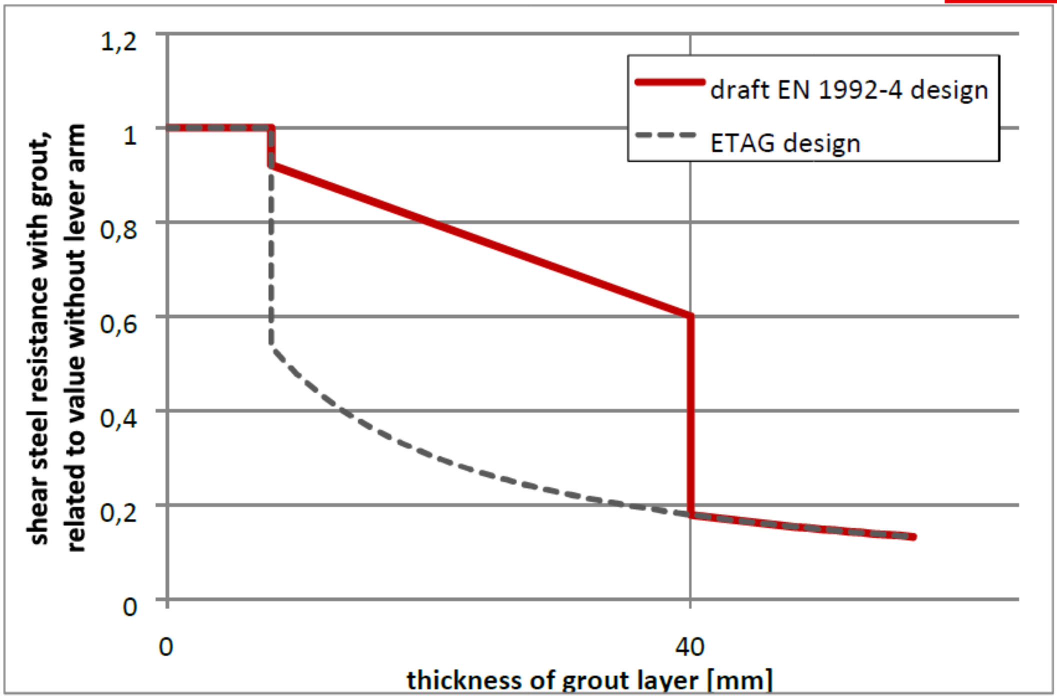


Figure 3-6 Shear capacity according to equation (9) (red line segments) with allowable mortar layer thickness up to 40 mm

3.6. Interim conclusion

There is not yet a conclusive answer for the shear capacity of an anchor with mortar-filled set space. This is also evident from the literature:

- Currently, there is no generally accepted theory to determine the complex interaction of shear force, tensile force and moment for a shear-loaded anchor [7, p. 4.1.2.2]. This is also mentioned as a discussion later in the book [7, p. 6.1.2.2].
- "Lack of information and uniformity in determining shear capacity for anchors with set space gives reason for experimental research." [2, p. VI]

But a clear line can be found in what is known. This shows itself well in the writing of equation (11) *the reduction in shear capacity of bolts with steel fillings*. In this equation, the shear capacity of the bolt is determined with an additional reduction for the headroom in the form of βp. All described equations of shear capacity, including NEN-EN 1992-4 without mortar joint, use this notation in direct or indirect form. Therefore, equation (12) is further used for this study: a calculation value of the shear capacity ($Fv_{,Rd/\gamma m2)}$ reduced by set space (β) which is determined based on a factor (αv) of the allowable tensile stress (fub) over the shear surface ($_{As}$).

$$F_{v,Rd} = \beta * \frac{\alpha v * As * fub}{\gamma_{m2}} \tag{12}$$

For the remainder of this study, the question is what is a responsible value of β?

- Is this a constant value according to equations (8) and (10)?
- Is this a decreasing value according to equations (3), (9) and (11)?
- Is it possible that this value increases even as the thickness of the mortar joint increases?

4. The current practice of remote mounting

Chapter 3 concluded with an equation and a still unknown reduction value β. For good modeling, it is necessary to include daily practice. Therefore, among other things, a survey was conducted. Appendix 1 contains the complete results. This chapter will summarize the results. Section 4.1 briefly discusses the group of respondents. Section 4.2 describes the parameters relevant to practice. Section 4.3 highlights the calculation methods used. And finally, Section 4.4 describes the potential risks and to what extent they may occur in practice.

4.1. Survey justification

In order to gain a good and broad insight into the practice of anchoring, an attempt was made to enlist the broadest possible group of respondents in terms of location and function to complete the survey. Therefore, principal structural engineers, detail structural engineers, steel fabricators and engineering firms were written to, within and outside the Netherlands. These are not only parties known to the graduate student, but also companies and individuals who responded through various contacts or in response to a general call via LinkedIn. This resulted in 46 respondents. The responses to the initial questions show a decent spread in function, type of company and location. Partly because of the wide spread across the country, the daily practice is well reflected because responses were received from various clusters of companies, which as a rule often work together.

4.2. Parameters steel-concrete connection

The connection of steel against concrete consists of several components with various parameters. These parameters all affect the shear capacity of the anchor to a greater or lesser extent. Below is the framework that will later be used for the EEM model as well.

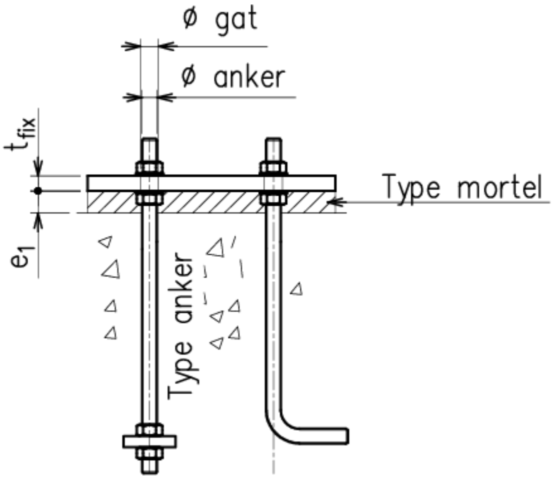


Figure 4-1 Parameters at steel-concrete connection

Foot plate thickness tfix	= 15 mm	(practical footplate thickness)
Material quality footplate	= S235	(standard material quality)
Anchor dimension	= M12-M30	(used in the experiments)
Anchor class	= 4.6 and 8.8	(used in the experiments)
Hole clearance	= +2 mm and +4 mm	(see section 4.2.1)
Setting space e1	= variable	(see section 4.2.2)
Type of filling	= K30, K50, K70 and steel shims	(see sections 4.2.3 and 4.2.4)
Type of anchor	= straight anchor with follower plate	(see section 4.2.5)
Tightening torque	= "hand tight" according to NEN-EN 1090 [14]	(see section 4.2.6)

Several parameters are of lesser importance for the shear capacity of the anchors. Choices were made for these values that correspond to daily practice.

4.2.1. Hole clearance

To secure the structure to anchors, the steel element is slid over the anchors. To make this fit, various hole diameters can be chosen, each of which individually affects the shear capacity of the anchors. The current production standard NEN-EN 1090-2 [14] defines nominal diameters (Table 4-1). Now this deals specifically with bolt or pin connections rather than anchor connections. No specific diameter is given in this standard for anchors in base plates [6, p. 8.2.] NEN-EN 1992-4 [5] does give a guideline for anchor holes (Table 4-2) and it is broadly similar to that for normal round holes in [14].

Nominal bolt or pin diameter *d* (mm)	12	14	16	18	20	22	24	27 to 36
Normal round holes	1		2					3
Oversized round holes	3		4				6	8
Short slotted holes (on the total length)	4		6				8	10
Long slotted holes (on the overall length)	1,5 *d*							

table 4-1 NEN-EN 1090-2 - table 11 - Nominal hole clearance for bolts and pins (mm)

Dimensions in millimeters													
Diameter anchor	6	8	10	12	14	16	18	20	22	24	27	30	> 30
Hole in steel plate	7	9	12	14	16	18	20	22	24	26	30	33	d+3

table 4-2 NEN-EN 1992-4 - table 6.1 - Hole clearance for anchors

Among other things, the following has been written about it:

- "The anchor holes cannot be oversized ... permissible diameters for clearance holes is dc < 1.2 d" [7, p. 3.7.1.2;]
- "The diameter of the clearance hole in the fixture should be <1.2 d" [15, p. 99]; and
- "Should multiple anchors bear then the hole clearance (...) should be 0.1 dst" [16, p. 30].

From the survey, about 50% of the respondents apply 4 mm larger holes. This is mainly to avoid fitting problems during assembly. For M20 anchors this results in holes $\emptyset 24$ mm and for M16 in $\emptyset 20$ mm. For M20 this does fit and for M16 this does not fit with what the book by R. Eligehausen et al. [7] and the HERON article by A.M. Gresnigt et al. [15] mention about this, namely that holes should be smaller than or equal to 1.2*d. However, according to [5] and CUR10 [6, p. 8.2], these are oversized holes for both sizes of anchors. Because the various literature gives leeway on hole size (1.1*d and 1.2*d), it is unclear what should now be called oversized and when and what should be reduced.

Apart from this ambiguity, the "problem" posed by oversized holes can be solved during assembly. The structure can be assembled with large washers that are welded off after assembly, but this is rarely used in practice. Holes can also be filled with a suitable filler. Hole filling is also prescribed by various anchoring calculation software. This is prescribed in cases such as deviating anchor patterns, large anchor numbers and oversized holes (Figure 4-2). In these situations, it is more likely that not all anchors will be on when a shear force occurs. Filling prevents this. The survey shows that about 50% of the respondents apply this filling only when the software prescribes it. In practice, however, this software is used only for post-applied anchors and not for collapsed anchors. Thus, a significant proportion of oversized holes will not be filled in practice.

On the design side, the problem of oversized holes can also be overcome. According to the book by R. Eligehausen et al [7], for oversized holes, it may be assumed that all anchors are engaged but the shear capacity of the anchors must be determined based on anchors on bending according to equation (3), resulting in a very low allowable shear force. Within NEN-EN 1993-1-8 [4], for shear capacity, nothing is included in this and the full shear capacity may be used for both equations (2) and (8). In [4], when using oversized holes, only the shear capacity must be reduced.

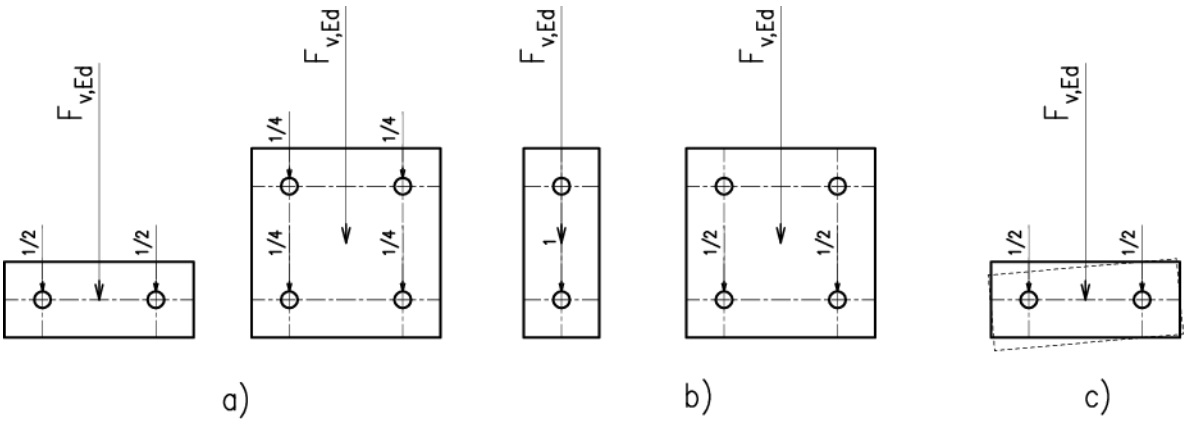


Figure 4-2 a) distribution of forces at normal holes b) possible distribution at oversized holes c) twisting due to oversized holes

4.2.2. Setting space

The survey shows that about two-thirds of respondents use an actuating clearance of 30 mm and one-third use an actuating clearance of 40 mm or more. Increasingly, larger setting spaces are being used, in part because of the new ISO4023 standard where nuts are higher than in the old DIN934 standard. Different adjusting spaces will be used in modeling to get a good idea of the importance of this parameter for the reduction value β.

4.2.3. Type of filling - Casting mortar

In the 1989 study [1], several types of mortar are examined. Over time, newer types have become available on the market and the current designations are K30, K50, K70 and K100. Over half of the respondents reported prescribing K70. This mortar is also widely used to fill spar pipes (gains) in precast concrete structures and is the most common mortar sold, according to some suppliers. After 28 days, this mortar has a compressive strength of more than 90 N/mm^2 . Some respondents indicated prescribing K50 for lighter structures. Some mortar specifications are attached in Appendix 3. Various grades will be examined for the EEM model because the lower compressive strength of the mortar could reduce the capacity of shear-loaded anchors.

4.2.4. Type of filling - Steel shims

In practice, it may be convenient to fill the distance between the steel and the concrete not with a casting mortar but with steel plates. For example, because the setting space is poorly accessible to apply a box around it and fill it with mortar, as with a steel beam installed under a floor. Steel fills, despite filling the gap with non-compressible steel, have the following concerns:

- If friction is a significant proportion of the shear capacity, then the shear capacity with steel plates will be able to be lower because a lower coefficient of friction for steel-steel than for steel-concrete must be maintained according to current standards.
- In the execution of such a connection, there is a chance that the head plate and the concrete may not be parallel to each other. As a result, the connection may remain open on one side. If clamping of the gap affects the shear capacity, no open position should be present after installation (Figure 4-3).

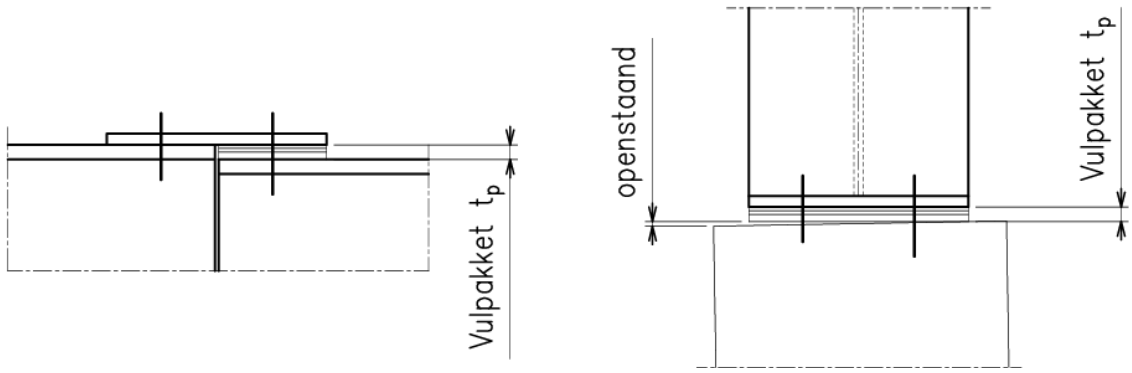


Figure 4-3 left) side view of girder: steel fills at steel-steel connection that can generally be closed right) top view of girder: open position between steel and concrete due to rotation of concrete column, for example

4.2.5. Type of anchor

The survey shows that half of the respondents prescribe hook anchors and the other half prescribe anchors with follower plates for anchors loaded to a shear force. Thus, although the literature [6] favors anchors with follower plates for anchors loaded on shear and/or tensile force, daily practice is divided in this regard. In the thesis research, concrete collapse will not be investigated and this means that the type of anchor and the whole mechanism of the anchor in the concrete are not important in the EEM model. By the way, both types of anchors will be treated indirectly, since hook anchors are provided in class 4.6 and anchors with follower plate in class 8.8 and both classes will be investigated.

4.2.6. Tightening torque

Of the respondents whose role requires them to state something about tightening anchors, the majority indicated that they use the standard 'hand-tight' torque for this purpose according to NEN-1090-2 [14]. The tightening torque will be significant when using an adjusting nut for deformation at (oversized) holes. Tightening the nuts will create a clamping surface with the base plate. The shear force will initially be transmitted directly to the anchors through friction in the clamping surface. When the friction between the washer and the steel plate is overcome, the plate will begin to shift until the anchors are engaged. Thus, pre-tensioning primarily affects when shifts start to occur in the footing, but will not affect the ultimate failure value and deformation of the anchors. The application of the mortar joint will always take place after the steel structure has been assembled, so the joint itself will never be under that pre-tensioning. Limited attention will be paid to this further in this study.

4.3. Calculation method

4.3.1. NEN-EN 1993-1-8 or NEN-EN 1992-4

In current practice, several interviews reveal, there is a perception that NEN-EN 1993-1-8 applies to long anchors and NEN-EN 1992-4 applies to post-applied short anchors [6, p. T.4.1(2)]. By and large, this picture is not strange, but it is also not entirely justified. For verification of concrete capacity, this distinction is understandable because different calculation rules apply for the different anchoring principles, for example regarding breakout, edge breakage and the application of split reinforcement. But the shear behavior of steel anchors is independent of the length of the anchors. No other mechanical behavior takes place in the steel. In CUR10 [6, p. T.4.1(2)] it is indicated that no distinction should be made whether they are long or short anchors, but that this should be based on anchoring principle. The report indicates that in terms of steel failure, there is no difference in shear capacity between short or long anchors. The survey shows that over two-thirds of the respondents usually apply NEN-EN 1993-1-8 [4] when checking the shear capacity. The anchoring principle or the length of the anchors does not seem to underlie the choice.

4.3.2. How does the current software compute?

For the steel and concrete checks, the anchoring software uses NEN-EN 1992-4, which is the only available standard for concrete capacity. For calculating the shear capacity of anchors, this contradicts the practice of manual checks based on [4]. Without margin of adjustment, the shear capacities in both standards are almost the same (see Section 3.4) and the difference mainly occurs when margin of adjustment is applied.

Hilti's software uses equation (3) *bending resistance of anchors without a mortar joint,* or (9) *shear capacity of anchors with a mortar-filled joint,* depending on whether the boundary conditions of equation (9) are met. This equation is applied even with a single anchor row in the direction of force. This seems an overestimation of the stated boundary conditions. B+Btec, Fischer and Halfen always apply equation (3) and within the software do not use the higher shear capacity according to (9). Thus, the shear capacity according to the different software packages generally falls very low and this is not always adopted by users. The survey shows that half of the respondents ignore the shear capacity in this software by not entering any shear capacity in the calculation software. Half of them indicate that they then check the shear capacity manually using equation (8).

Appendix 2 details the verification of the calculation method of the shear capacity of a steel anchor according to the various calculation software. Included in this check are Class 5.8 anchors. These are otherwise outside the scope of this study, but the anchor suppliers do not supply Class 4.6 anchors, nor does Halfen supply Class 8.8 anchors. Therefore, class 5.8 has been added so that the calculation methods can be compared.

		Suppliers			
	Setting space	B+Btec	Halfen	Fischer	Hilti
$Fv_{,Rd}$ (kN) Anchor M20	**5.8**				
	0	48.8	48.8	59.2	49.04
	15	17.28	17.33	17.28	17.31
	30	11.52	11.56	11.52	11.54
	8.8				
	0	78.4	x	78.4	78.4
	15	27.68	x	27.68	27.69
	30	18.45	x	18.45	18.46

Table 4-3 Results of M20 anchors with different setting spaces from four different types of calculation software based on equation (3)

4.4. What are the risks of too low anchoring capacity?

Could it be wrong to calculate with too low anchoring capacity? Surely it is extra capacity that does not compromise building safety? However, some of the respondents indicated that they ignore the setting space in the anchor software and do not test it separately. If the adjustment space is disabled within that software, the steel inspection is performed according to NEN-EN 1992-4 equation (2). By not manually checking the shear capacity of anchors with headroom - by using equation (8), for example - the shear capacity is actually overestimated (Figure 4-4). If the concrete is not dimensional, there is an unjustified overestimation of the anchor's shear capacity.

Nevertheless, in practice, steel failure of the anchor will not occur easily. The concrete will be more likely to fail at higher occurring shear forces. These checks are always performed by the software with or without set space and therefore unknowingly overestimate the anchors in which the concrete is normative. Within this study, it is assumed that these checks are programmed correctly.

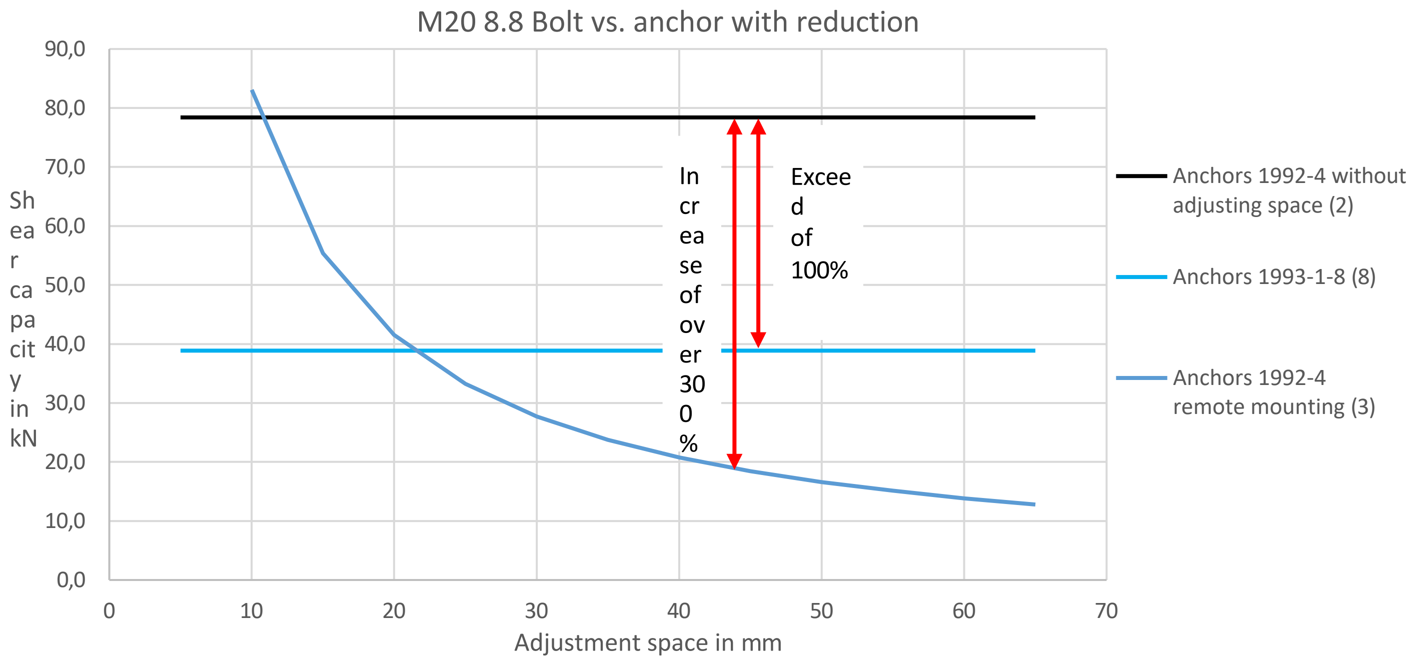


Figure 4-4 Capacity ratio anchor controls

It may also be less justifiable in other respects to "allow an undervaluation in the calculated anchor capacity. As Mr. Berkelder put it, "*Dutch practice has almost always, in the case of column bases and footplates important to steel fabricators, shown great and rather unsubtle caution. (...) The constellation painted above led to large footplates, (...) To a greater extent than was probably necessary, stiffened column foot constructions came about, which for economic and for architectural reasons was not desirable*." [16].

With capacity differences of a factor of 2 to 4 as explained in Section 3.4, the above certainly occurs: more labor is required, as well as more materials such as additional anchors and additional splice and confinement reinforcement. These are all economic consequences.

Structural and pre-assembly challenges may also arise:

- Footplates will be larger due to larger anchor groups and, larger plates will get in the way more quickly structurally.
- In the concrete, more anchors will cause faster fit problems in the applied reinforcement.
- When more anchors are used, it becomes more difficult to slide the base of a steel structure over the anchors.
- More anchors will also have to be placed closer to the concrete edge more often, and there the shear capacity of the concrete is lower.

And constructive even follows a downward spiral:

- By applying more anchors, suppliers will be more likely to apply oversized holes. Oversized holes affect deformation and possibly shear capacity.
- Larger numbers of anchors must be filled to ensure shear capacity of which it is not certain that this is always done correctly.
- With more anchors, according to ETAG [11], the advice is to reduce shear capacity, which may result in the need for more anchors again. Incidentally, this requirement has not been adopted in the current codes.
- Anchors placed close to the concrete edge will require edge reinforcement that both increases cost and has a negative impact spatially.

Thus, should it turn out that the current equations carry too much safety, it is not automatically justified to apply them as well. Finding the appropriate reduction value β is therefore relevant.

5. Experimental research on anchors with mortar joint

This chapter will reflect on various experiments that have been conducted:

- Laboratory experiments by L. Bouwman, A. Gresnigt and A. Romeijn, "Investigation of the attachment of steel footplates to concrete foundations," TU-Delft Stevin Laboratory, Delft, 1989 [1].
- PhD research by K. Mcbride, "Steel shear strength of anchors with stand-off base plates," University of Florida, Florida, 2013 [2].
- Experiments conducted and published by Fischer: R. Mallée, "Größere Querlasten," *Fischer connet it,* no. 5 August 2005 [3].
- Research by P. Dusicka, G. Lewis and C. Smith, "Effect on fillers on steel girder field splice perfomance," Portland state University, Portland, 2012 [17].

Section 5.1 will describe the computational models of the tests and then in Section 5.2 the results from these tests will be juxtaposed with the current equations as detailed in Chapters 2 and 3. The tests by L. Bouwman et al. were the basis of equation (8) as currently given in NEN-EN 1993-1-8 [4]. The doctoral research of K. Mcbride confirms equation (10) given in ACI-318-19 [8]. In Fischer's experiment, a new proposal for determining the shear capacity of anchors is given by R. Mallée. Several conclusions and points of interest will also be cited from the graduate research of A.M.P. den Deurwaarder [18], who himself also conducted several experiments.

Section 5.3 will describe all the conclusions and findings of the various studies related to the grout, and Section 5.3.3 will highlight findings and conclusions important to this study from the experiment [17] that, among other things, underlies equation (11) *the reduction in shear capacity of bolts with steel fillers*. If there is a correlation between the failure behavior of bolts with steel fills and anchors in concrete with a mortar-filled joint then perhaps those conclusions could also apply to those of anchor joints.

Sections 5.4 and 5.5 focus on the influence of hole clearance and anchor deformation based on the tests. Section 5.6 describes the conclusions based on the tests.

5.1. Calculation models

All experiments start with a theoretical framework and hypothesis regarding the behavior of anchors when applying a mortar joint. For each study, this is summarized in the sections below.

The behavior of the joint can be basically described as follows. The occurring load $F_{v,Ed}$ can be absorbed in the form of shear (V1, V2) in the anchors (Figure 5-1 a). Due to the distance (e), $F_{v,Ed}$ also causes an additional moment on the anchors. The material that offers the most resistance to this bending in terms of stiffness will begin to absorb this load. This can be absorbed by the moment capacity of the anchors (M_{sd1}, M_{sd2}) and/or by a tensile/compressive torque (D, T_1/T_2) created between the mortar and the anchors (Figure 5-1 b and c). If the moment $F_{v,Ed} * e$ is (partially) absorbed by the tensile /pressure torque creates an additional shear resistance in the form of friction (13) where μ is the coefficient of friction.

$$F_W = D * \mu \qquad (13)$$

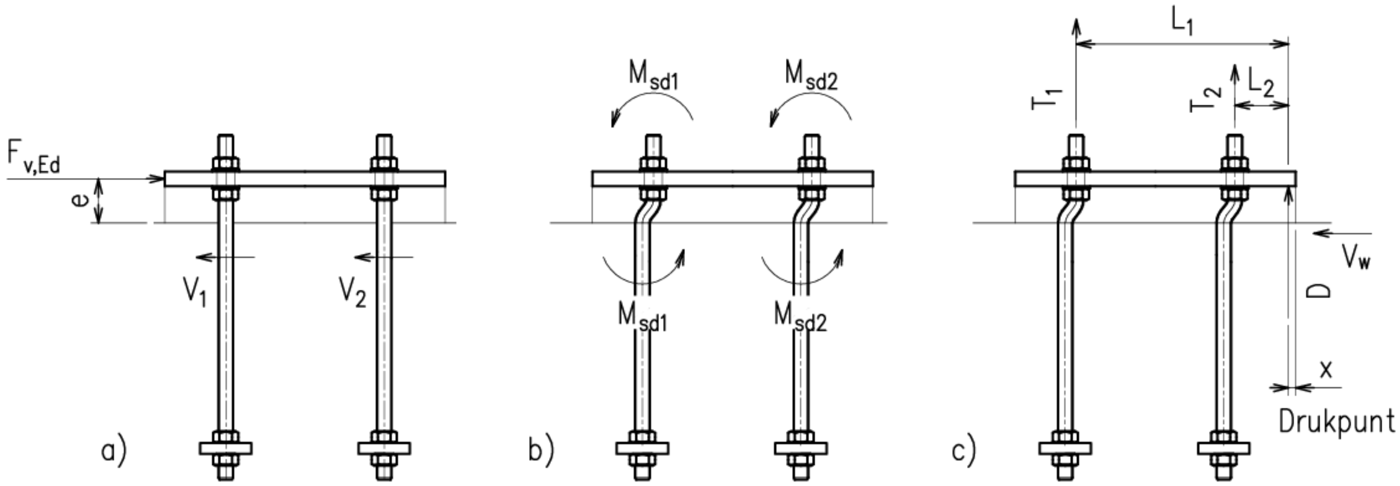


Figure 5-1 Variants of force progression in steel-concrete connection

For all test setups, the shear force was set at the center of the plate, with the exception of Mcbride's eccentrically loaded tests (one series).

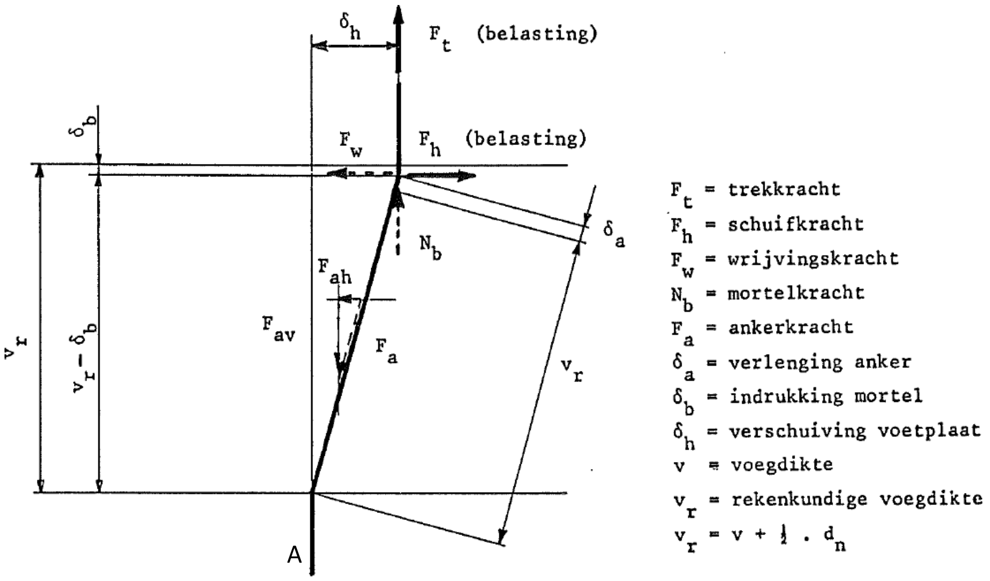
5.1.1. Bouwman et al.

The study by L. Bouwman et al [1] states that the behavior of the anchor with mortar joint is very complex. Various parameters such as bending stiffness, hole clearance, preload, resistance of the grout material and the degree of trapping make describing a relationship between the load and the displacement not very meaningful [1, p. 32]. Therefore, we turn to a simpler behavior to be described. [1] states the following (Figure 5-2): the transverse force acting on the plate makes the foot want to slide. Before the foot will slide, friction (Fw) must first be overcome. After the friction is overcome, the anchor will shift in the horizontal direction (δh). The sliding will pull the slab toward the earth. The mortar will prevent this deformation in the vertical direction (δb) (depending on the stiffness of the mortar) and this creates a compressive force on the mortar (Nb). Because the anchor is blocked in the vertical direction and the shortening of the mortar (δb) is almost equal to 0 mm according to research, the anchor lengthens (δa). This elongation translates into a tensile force in the anchor (Fa) and makes equilibrium with Fh and Nb.

With that given, an old interaction equation is used to determine the reduction in tensile strength. If part of the strength is used to absorb the tensile force, the remaining part can absorb less shear force. By allowing the maximum allowable tensile force to act in the interaction equation, the smallest residual portion remains for the shear force. This results in a reduction of 0.30 for class 4.6 anchors and 0.20 for class 8.8 anchors. If the partial safety factor γm2 (1.25) is incorporated into this value, the reduction becomes 0.375 and 0.25, respectively. These are the old values according to NEN 6772 [12]. In the current NEN-EN 1993-1-8 an interpolation is written between these values of 0.375 and 0.25 in the form of equation (8), αbc= 0.44-0.0003*fyb. To stay in the notation of β*αv, as stated at the end of section 3.6, i.e., the αv split from the β as a reduction factor for the headroom, the equation prepared by L. Bouwman et al. gives the following β at an αv of 0.6:

- For class 4.6, β = 0.375/0.6 = 0.625
- For class 8.8, β = 0.25/0.6 = 0.42

From the hypothesis of [1] comes a proposal of a constant β lower than that in ACI-318-19 equation (10) where β is equal to 0.8.



Balance in A:

Σh=0 Fh = Fw + Fah

Σv=0 with only a shear forceFav = Nb

Afbeelding 3.1-3 : Schema rekenmodel (elastisch stadium).

Σv=0 with an additional external tensile forceFav = Nb + Ft

ΣM=0 (Nb + Ft) * δh - (Fh - Fw) * Vr

The moment equilibrium does not include δb because according to L. Bouwman et al. this distance is negligible.

Figure 5-2 Figure 3.1-3: schematic computational model (elastic model) [1, p. 32].

The computational model is further deepened in the study with a plastic model. The basic behavior remains the same as described above, but now it is possible for plastic stretching to occur in the anchors.

The boundary in the elastic model is the situation where a fiber reaches the extreme stress fy. In the plastic model, the entire cross section will be able to reach the maximum stress fy (Figure 5-3). Therefore, in the plastic model, the anchors have a higher failure value and therefore a larger deformation will also occur.

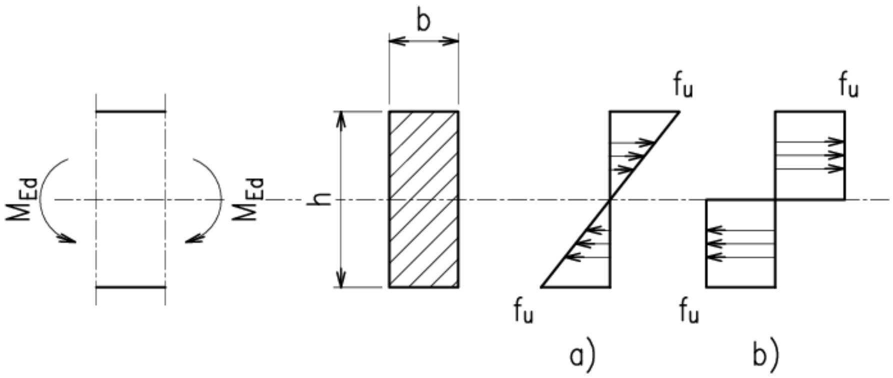


Figure 5-3 Stress gradient due to a moment on a rectangular cross section
a) elastic gradient; extreme fiber reaches maximum yield stress fy
b) plastic gradient; all fibers reach maximum yield stress as the outermost fibers stretch and do not tear

5.1.2. Mcbride

The thesis by Mcbride [2] extensively discusses the method of L. Bouwman et al [1]. In it, four focal points of L. Bouwman et al are highlighted with subsequent comments:

- L. Bouwman et al. assume that anchors collapse purely on tensile stress, whereas it will be a combination of a shear and tensile stress.
- L. Bouwman et al. indicate that the ratio of tensile to shear force does not affect the anchor failure force. This is often true, but certainly not in all circumstances.
- There are three possible horizontal equilibrium situations in which friction does or does not contribute:
  1) The shear capacity of the joint is equal to only the frictional force.
  2) The shear capacity is the sum of the frictional force and the shear capacity of the anchors.
  3) The shear capacity is only that of the anchors without a share of friction.

  L. Bouwman et al. assume the second form, which is a combination of frictional and shear resistance. Again, this is often the case, but not in all circumstances.
- The maximum allowable force will depend on the plastic strain, while L. Bouwman et al. assume that it depends on the maximum yield stress. The resistance belonging to the maximum strain is not necessarily equal to the resistance belonging to the maximum yield stress, and that is where additional shear capacity is potentially available.

Using a free-body diagram, the study determines internal equilibrium. Here it is also concluded that with a rigid grout, bending on the anchors cannot occur unless an excessive external tensile force acts on the footing. Initially, this tensile force will subtract from the compressive force of the mortar. It is stated that, as long as a compressive force is present on the mortar, the tensile force in an anchor is barely increased by the moment from the shear force. Thus, the compressive force on the mortar acts as a prestress that must first be overcome (situation 2). In an example calculation in which low mortar stiffness is calculated, it is shown that 93% of the external tensile force is subtracted from the compressive force and the tensile force in the anchors increases by only 7%. When the external tensile force is so high that no compressive force is present on the mortar, no friction is also present. The occurring shear force will then be completely absorbed by the anchors (situation 3). Thus, as long as a compressive force is present, internal or external, friction will contribute to the total shear capacity (situation 2). The external compressive force may also be so large that the anchors are not loaded at all because the frictional resistance of the joint is higher than the shear force occurring (situation 1). The displacement (Figure 5-2 δh) will be nearly 0 mm in this situation.

5.1.3. Mallée

In the article by R. Mallée [3] treating an experiment conducted by Fischer, a hypothesis of the problem is not specifically established. Based on the experimental results, a proposal is made by R. Mallée to quantify the experimental results. The reason is that R. Mallée observes that the shear capacity according to equation
(3) *bending resistance of anchors without mortar joint* is so low compared to the test results that it does not describe the actual behavior well. This is the conclusion based on the results from 27 tests of anchor groups consisting of four anchors of two rows with two different stitches. In the proposal, based on the findings of the tests, the behavior of the rear row anchors is carried forward to additional anchor rows if they are applied. The proposal is as follows: the front row of anchors (row 1 Figure 5-4 a) is loaded on bending and the rear row(s) (2 and 3) on shear, which results in equation (14). For a pattern of four anchors distributed over two rows (see Figure 5-4 c), this yields a combination of two anchors that are not reduced (and where pure shear is the failure mechanism) and two anchors whose plastic bending resistance is included. A group effect is created in which equations (2) and (3) are combined.

$$F_{v,Rd\,boutgroep} = n i_{,j} * F_{v,Rd} + n1 * \frac{\alpha M * MRk,s}{l * \gamma_{m2}} \quad (14)$$

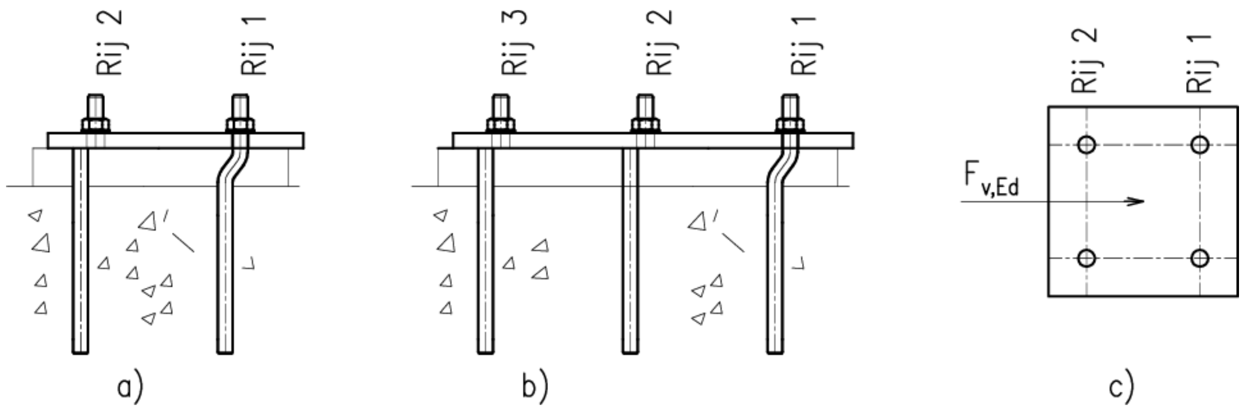


Figure 5-4 Failure mechanism by anchor row according to research R. Mallée; row 1: anchors on pure bending; row 2 and above: anchors on pure shear

5.2. Results

From the three studies, the results are shown separately in Figure 5-5, Figure 5-6, Figure 5-7 and Figure 5-8. The results of [1] and [3] have been translated to the standardized f_u of the respective anchor class and the fracture value found has also been corrected with this ratio. Thus, for all tests the average material properties were corrected to characteristic values.

Example:
Trial D10 research L. Bouwman et al [1].
Tensile capacity test specimen f_u = 1196 N/mm^2 , tensile capacity class 8.8 f_u = 800 N/mm^2
Correction factor = 1196/800 = 1.50
Breaking capacity test of 2xM20 = 295.0 kN
Corrected shear capacity per anchor = 29.05/2 = 147.5kN 147.5/1.5 = 98.3 kN

For the results of [2], a visual representation was made showing not the breaking capacity of the anchors but the reduction factor of the shear capacity of a filled joint (β). The American standard of bolts and the ratio of fub/fyb of these bolts, is difficult to translate to the European standard. In these results β has already been determined indirectly, see section 5.2.3.

Finally, Figure 5-9 shows all equations and results expressed as β, the ratio between the fractional value found and the computational value of the shear capacity. The arrested shear capacity is based on equation (2) with α_v of 0.6. Thus, the calculated β belongs to the computational value of the shear capacity. If the characteristic shear capacity would be used (without partial safety factor), a lower β follows, but the above can be used interchangeably. It follows from standards [4] and [12] that the partial safety factor applies only to the terms α_v, A_s and f_u, but not to β, since it has already been increased from [1] by the partial safety factor γm2.

Example:
For class 4.6, β equals 0.3 [1, p. 152].
*Fundamentally, it follows β = 0.3*1.25 = 0.375* [12, p. 11.7.2.3.3.]

The referenced tables can be found in Appendix 4.

5.2.1. Bouwman et al.

The fracture and deformation of the anchors were examined in three series of tests. The first series was not included in this thesis research because the unreinforced concrete block in which the anchors were poured had a lower resistance than the maximum shear force of the anchors at which they would break. These test results should not be used by the researchers to determine the shear capacity of the anchors. In the second series, reinforced concrete was used and the anchors were loaded with a shear force only. In the third series, this was combined with an external tensile force. From these tests it is concluded that the calculation rules in force at the time are either very conservative or far too favorable.

Table 4-1 (Appendix 4), Figure 5-5 and Figure 5-6 show the test results where steel failure of the anchor is the failure mechanism.

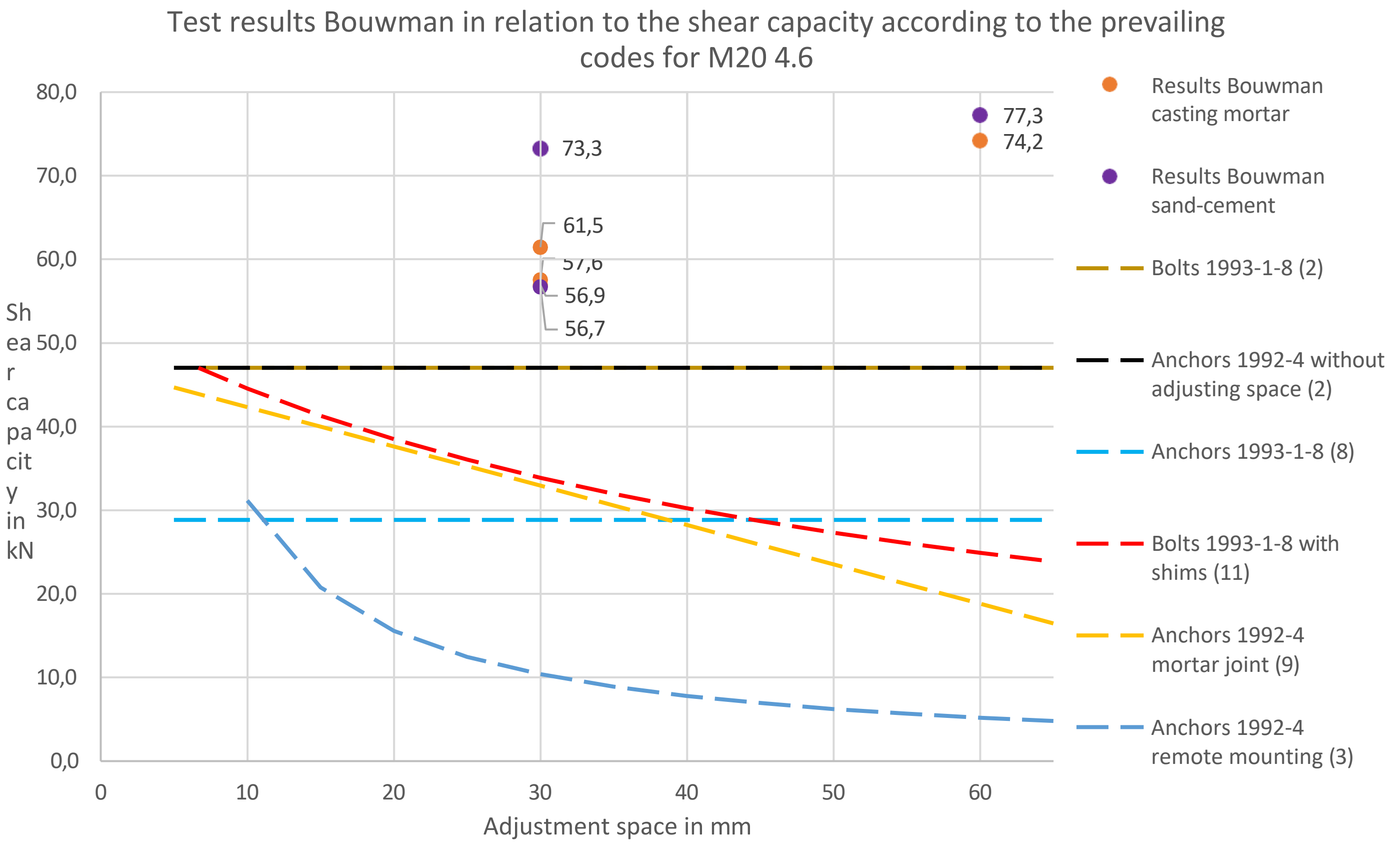


Figure 5-5 Visual representation test results L. Bouwman et al. [1] of M20 4.6 anchors in relation to prevailing code expressed in kN

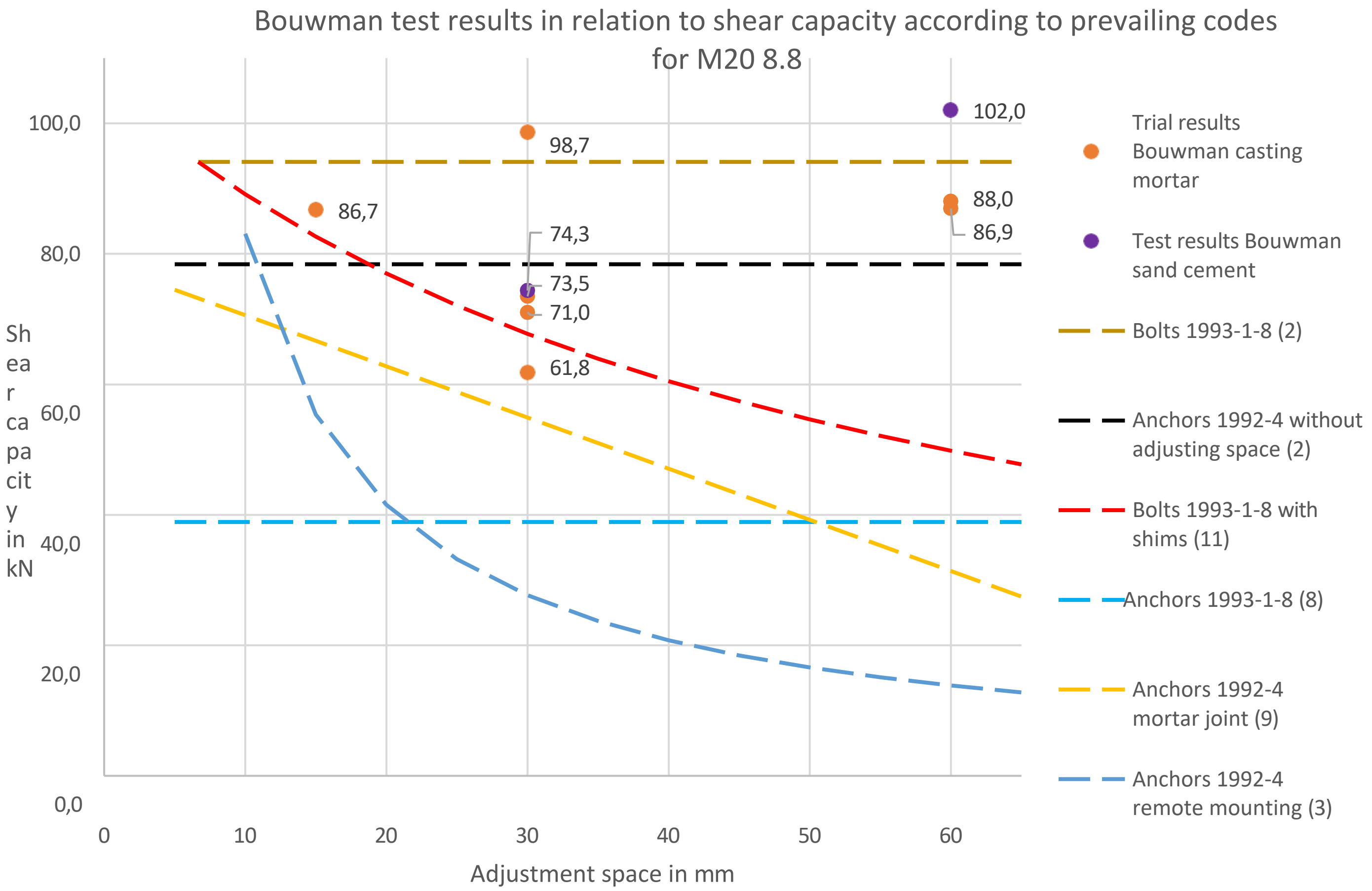


Figure 5-6 Visual representation test results L. Bouwman et al [1] of M20 8.8 anchors in relation to prevailing code expressed in kN

5.2.2. Mcbride

K. Mcbride's research [2] addresses in detail what was also cited in the sections above with a focus on the U.S. code. Four test cases are examined in this research:

1) A single shear force, both without and with setting space without mortar joint
2) Torsion on anchor group, both without and with set space with and without mortar joint
3) Circular pattern of anchors simulating a traffic sign situation with set space with and without mortar joint
4) Eccentric loading, both without and with set space with and without mortar joint

Converting these test results to a European equivalent is complex, and it is not entirely clear what certain indices from this study represent. Also unclear is the tensile capacity of the anchors used in these tests. Fastener F1554-G55 has a yield strength (fy) of 380 N/mm^2 and an allowable tensile strength ($_{fu}$) between 516 N/mm^2 and 621 N/mm^2 . But the exact tensile strength of the tested samples is unknown. These have been used for research [1] and
[3] is well known. According to NEN-EN 1992-4, an αv of 0.5 should be applied to this material, but these results rather indicate that this αv will probably be 0.6. An important fact about these tests is that they were carried out with oversized holes. Thus, the shear capacity of the anchors could be lower than in the other tests with normal holes (see Section 4.2.1).

In the conclusion of [2, p. 5.5.1], the experimental results are given in Vu/Tu-. The results would thus come out to be between 0.55Tu and 0.7Tu. These values are compared with the value of 0.48Tu following from ACI-318-19 equation
(10) [8]. 0.48 is determined by β*αv and is equal to 0.8*0.6. β can thus be determined from Vu/Tu. This brings the β for the group tests to 0.55/0.6 = 0.85 and 0.7/0.6 = 1.1. It is then concluded by K. Mcbride that the β of 0.8 stated in [8] may be considered conservative. All experimental results can be converted to β in this way.

Table 4-2Table 4-1 (Appendix 4) and Figure 5-7 show the test results where steel failure of the anchor is the failure mechanism.

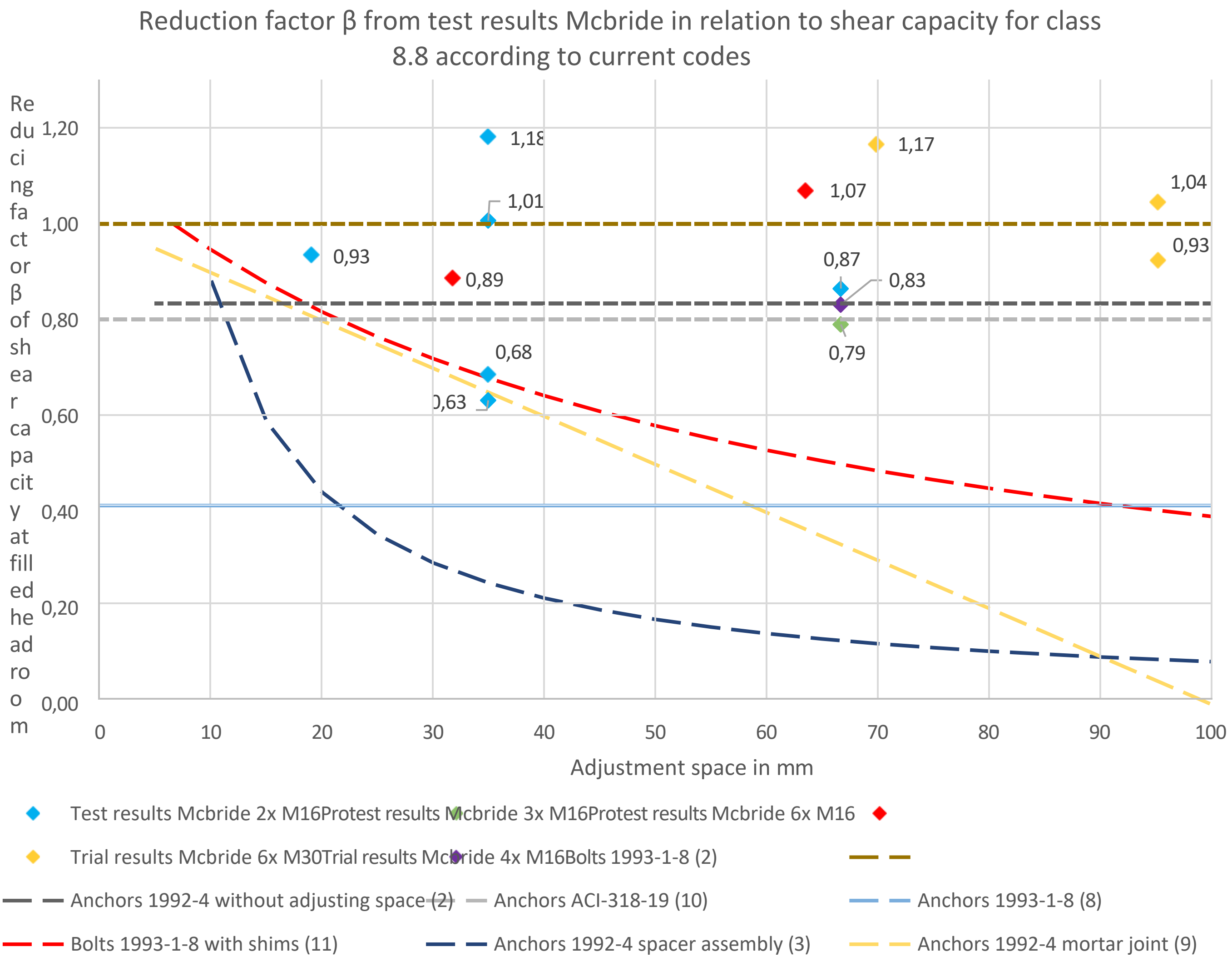


Figure 5-7 Visual representation test results K. Mcbride [2] of 8.8 anchors in relation to prevailing code expressed in β

5.2.3. Mallée

Twenty-nine tests were conducted in the study by Fischer [3]. Two tests were carried out for one loose anchor without adjusting space to determine a reference breaking value. The remaining 27 tests were tested in various heights of set space with two different pitch sizes, 65 mm and 190 mm. Teflon film was placed between the mortar and the steel so that no friction could be absorbed. M12 adhesive anchors were used with holes of d=14 mm in the base plates which corresponds to the specification of normal holes.

The applied adhesive anchor is not a standard class anchor. However, the material properties of the anchor are given. The tensile strength (fub) = 713 N/mm^2 and the yield strength (fyb) = 541 N/mm^2 . Scaling up the material properties and results to 8.8 anchors, in consultation with Fischer, does not do injustice to the conclusions that will be drawn from them. At a yield strength (fyb) of 640 N/mm^2 , the tensile strength (fub) scales up to 640/541*713 = 840 N/mm^2 , which is an acceptable deviation from the expected 800 N/mm^2 .

Table 4-3 (Appendix 4) and Figure 5-8 show the test results where steel failure of the anchor is the failure mechanism.

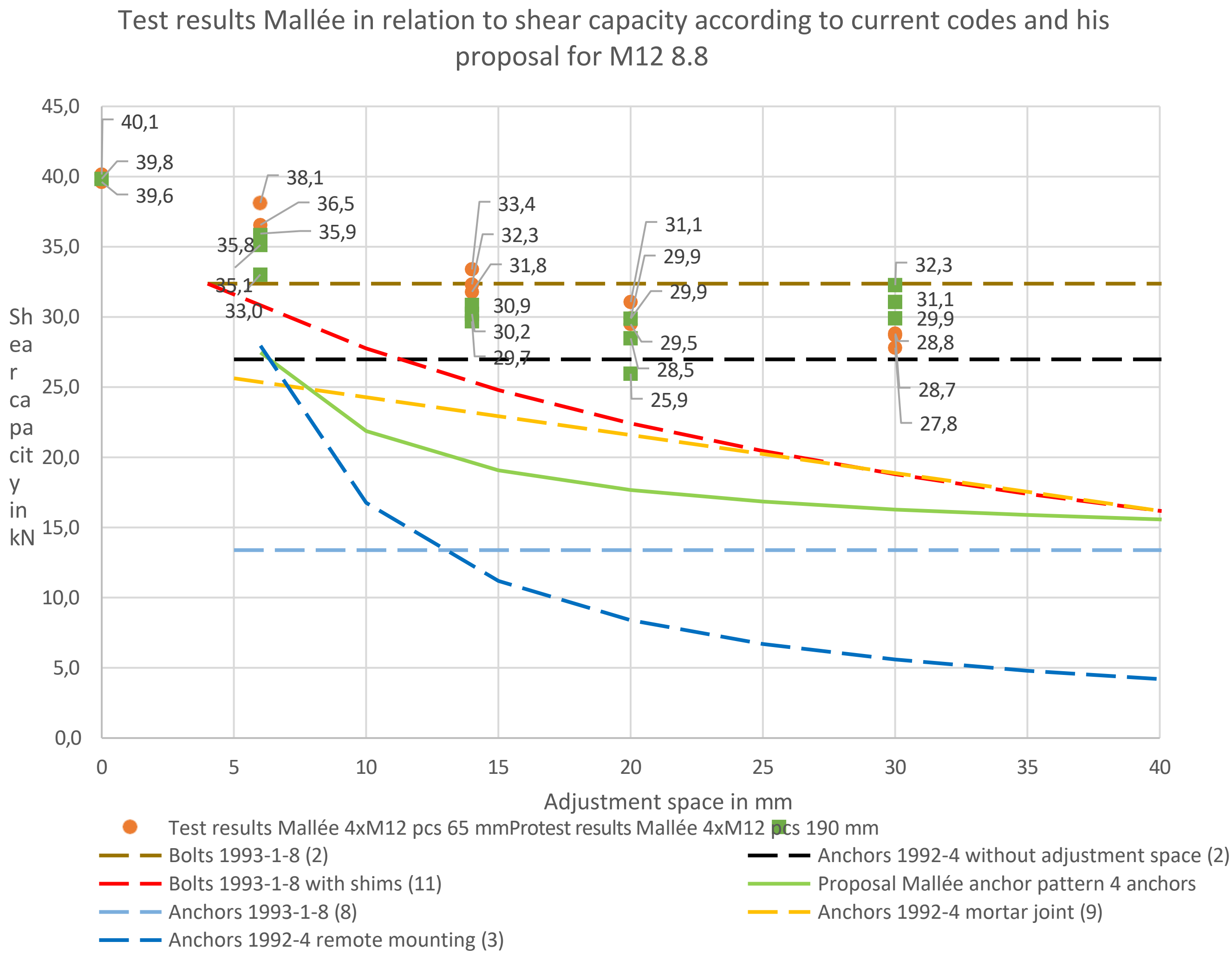


Figure 5-8 Visual representation test results performed by Fischer [2] of M12 8.8 anchors in relation to prevailing code in kN

5.2.4. Visual display of all test results

All experimental results were converted to a β based on an αv of 0.6 and shown in Figure 5-9. In this way, all results can be compared. Also shown is equation (14) of R. Mallée for an anchor group of two anchors consisting of two rows and six anchors consisting of three rows in the direction of force. For this equation, the β of the mean value of an anchor is shown.

The proposal by L. Bouwman et al [1] imposes a restriction on the use of anchor diameters. Their proposal applies only to M12 to M24 anchors because tests have only been conducted with M20 anchors. By including the M12, M16 and M30 tests from the other studies, a broader basis is created to determine β with high certainty for common anchor sizes from M12 to M36.

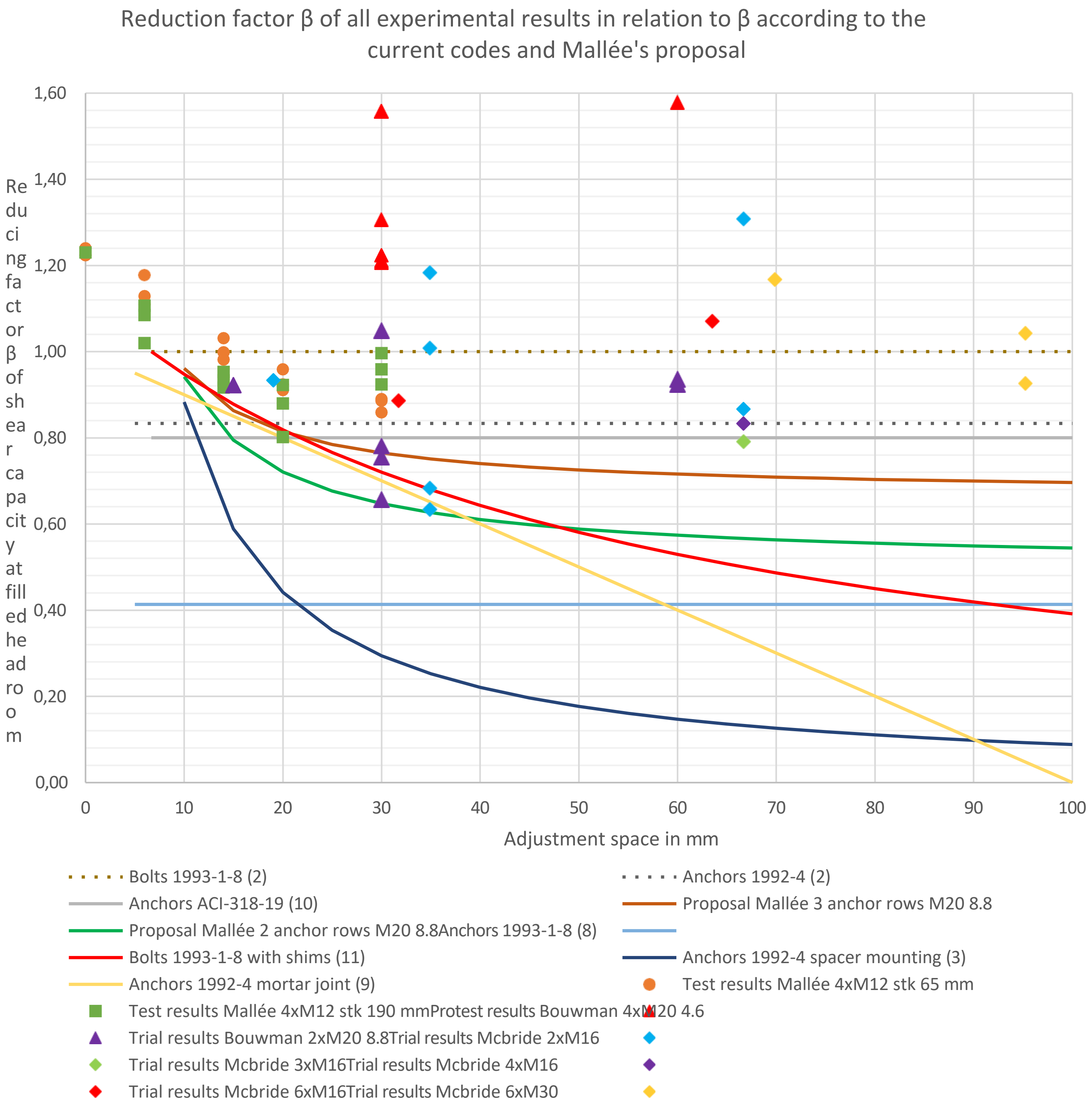


Figure 5-9 Visual representation of all experimental results examined expressed in relation to prevailing code expressed in β

5.3. Add

The same conclusion follows from all experiments: the shear capacity is increased by applying a non-compressible joint. The equations of NEN-EN 1992-4 for spacer assembly result in an unfairly low shear capacity.

5.3.1. Mortar type

The type of mortar joint affects the shear capacity. Undersanding with sand cement, according to the tests of A.M.P. de Deurwaarder [18], is significantly less stiff and strong than the joint filled with poured mortar. L. Bouwman et al [1] also concludes this behavior, but less firmly. There is a difference in the deformation behavior. Whereas when undersabbing with sand cement, the front of the base plate moves downward and thus compresses the mortar, this does not occur with cast mortar. Because of this compression in sand-cement joints, a lower frictional resistance is also achieved. The mortar joint should have sufficient compressive capacity and stiffness, but with the current practice of high-quality poured mortars, this is no longer a problem.

5.3.2. Arithmetic joint height

The studies reveal important concerns regarding the computational headroom reported earlier in Section 3.1 (Figure 5-10 a).

The study by R. Mallée [3] shows that due to the mortar joint, the "fictitious" trapping depth of 0.5*d is no longer in effect (Figure 5-10 b). This is concluded from the tests performed. The mortar joint traps the concrete, preventing the concrete from breaking out at the top. This results in a shorter bending length. While this does not affect the anchors whose full shear capacity is included, it does increase the shear capacity of the front flexure-loaded anchors. Figure 5-9 shows R. Mallée's proposal according to these shorter bending lengths (green and brown lines).

The study by A.M.P. den Deurwaarder [18] shows that for anchors without a filled joint, not further considered in this study, the arithmetic adjusting space with adjusting nuts can be assumed from the bottom of adjusting nut (Figure 5-10 c). This would increase the allowable shear capacity according to equation (3) for column bases by decreasing the arithmetic adjusting space by half a base plate (1/2*tfix) and a nut height+ring thickness (≈ 1*d) (Section 3.1).

Combining the two conclusions results in a significantly smaller bending length (Figure 5-10 d). Consequently, the shear capacity of the anchors loaded on bending will be higher.

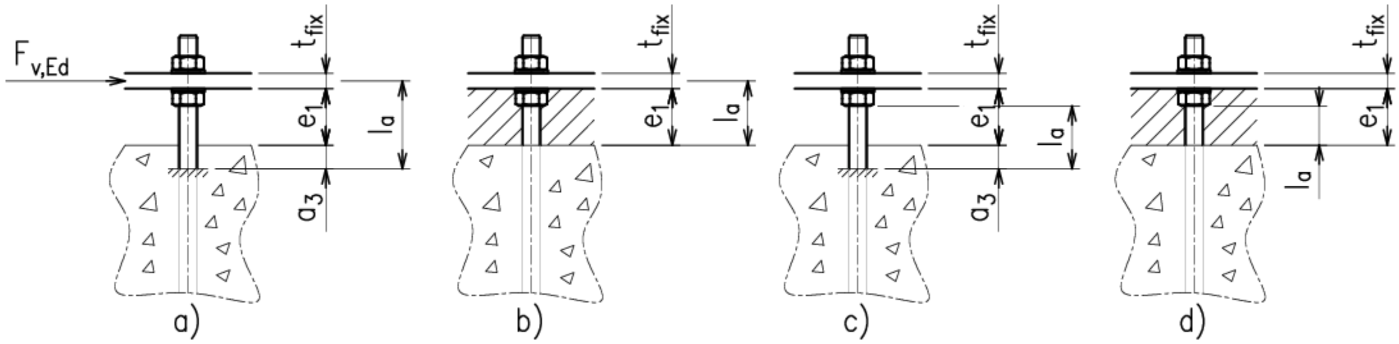


Figure 5-10 Arithmetic set space
a) arithmetic headroom in accordance with NEN-EN 1992-4
b) Mathematical headroom according to R. Mallée for mortar-filled joints
c) Mathematical setting space according to A.M.P. den Deurwaarder for setting nuts without mortar-filled joints
d) computational headroom if b) and c) were to be combined

5.3.3. Steel shims

In the study by K. Mcbride [2], one test arrangement with steel fills was examined. From this it is tentatively concluded that the anchors exhibit the same behavior with steel fills as with cast mortar. In this test, three stacks of loose shims were used, thus avoiding the problem of Section 4.2.4. The first shift, because the frictional resistance of steel-steel is lower than steel-concrete, does occur earlier [2, p. 255]. However, based on the tests performed, few results are available.

Figure 3-5 shows the current equation (11). It shows a decreasing shear capacity as the headroom increases. This equation, as explained in Section 3.3.4, is for steel-steel connections and it is therefore important to find the correlation between the tests of steel-steel and steel-concrete connections. By P. Dusicka et al [17], shear tests on steel-steel connections with steel fills and without prestressed bolts were carried out. In these tests, different numbers of bolt rows, filler thicknesses and numbers of fills were investigated. The tests were conducted with a bolt diameter of ⅞ inch (22.4 mm). This will have an approximate ds of 20 mm. For M20, d = 20 mm and ds = 17.7 mm, see Table 5-1 in Section 5.6.2. ds is thus 2.3 mm smaller than d. For the tested bolt diameter, ds ≈ 22.4 - 2.3 mm = 20.1 mm will then apply.

The following conclusion is drawn: the application of steel fillings reduces the shear capacity of the bolts and the deformation increases. This is consistent with tests previously performed by others [19]. Those tests were conducted up to a filler height of 1 inch (25 mm) and therefore in this study [17] the tests were conducted up to 2 inches (50 mm). The results show that beyond 1 inch fill height, the shear capacity of the bolts starts to increase again (Figure 5-12). A trapping effect of the shim occurs, preventing bending on the bolts. On the front side, a compressive force is created on the shims and the balancing tensile force must be absorbed by the bolts (Figure 5-11). The sum of all the tensile force in the bolts must equal the compressive force in the steel plates. The total shear resistance is equal to the shear capacity of the bolts and the frictional resistance created by the compressive force. The frictional resistance is determined by multiplying the compression force by the coefficient of friction μ, see equation (13).

The above follows from the tests where one shim was used. In the tests where multiple shims are used, the shear capacity after 1 inch does not increase but decreases, in accordance with equation (11). The situation with multiple shims can be disregarded in comparison with the test results of the steel-concrete joints. This is because in this situation there are multiple surfaces on which friction can occur because fills can shift among themselves. In a monolith-filled joint, this behavior is not present.

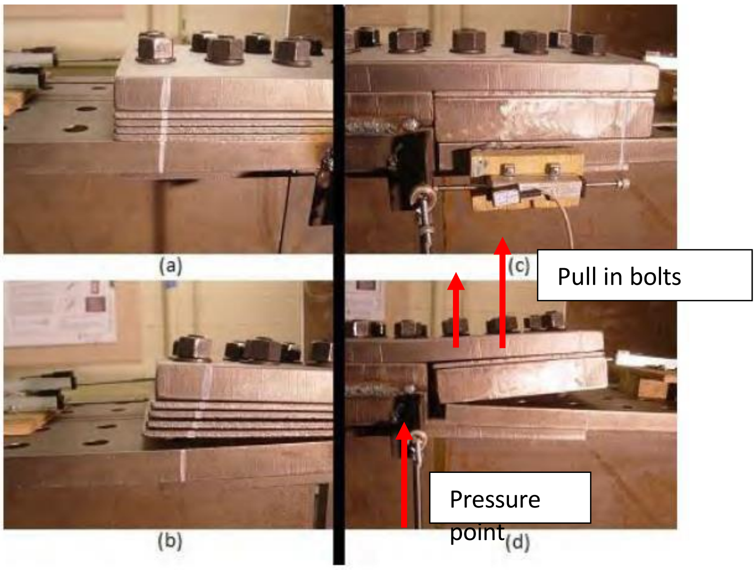


Figure 5-11 Deformation of 1-inch trial with multiple fills before (a) and after (b) the trial; and 2-inch trial with one fill before (c) and after (d) the trial [17, p. Figure 36]

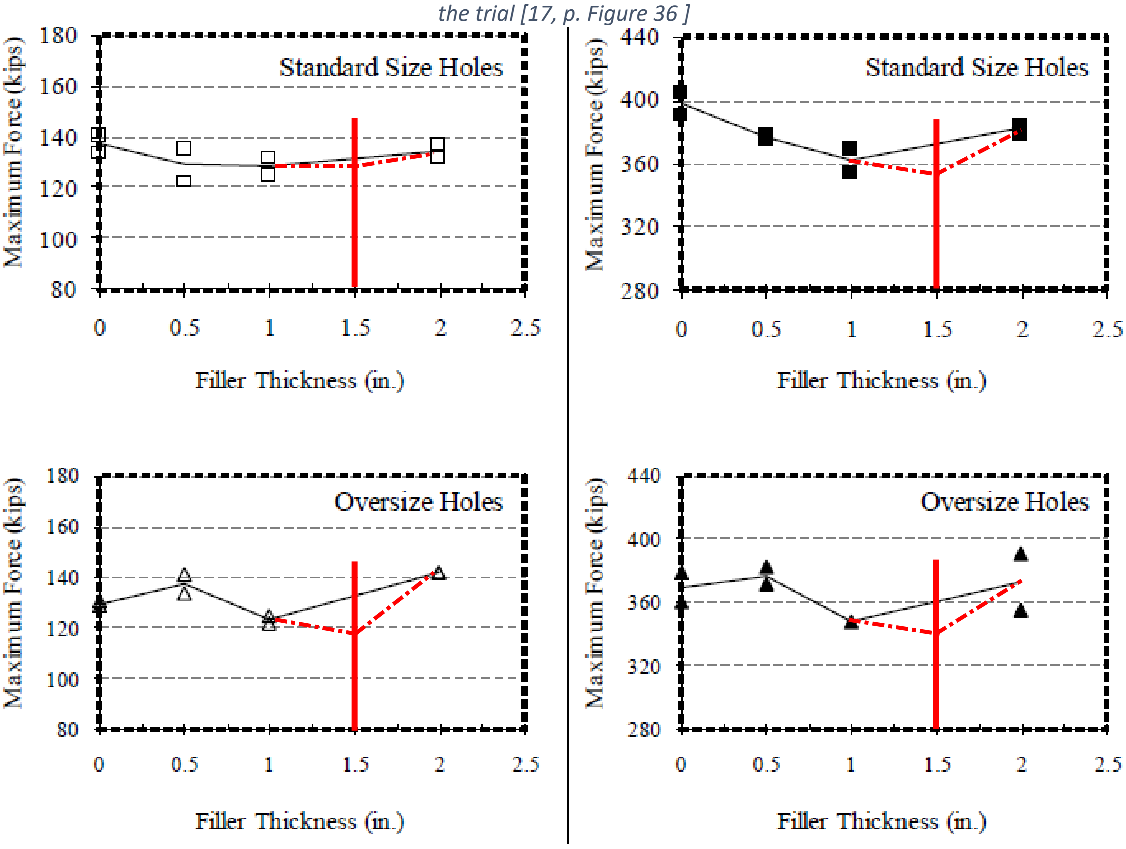


*Figure 5-12 Hypothetical course of maximum shear capacity if at 2*ds (1.5 Inch) the lowest failure value were to occur, projected in the test results of P. Dusicka et al. [17, p. Figure 9]*

5.4. Hole clearance

The studies by L. Bouwman et al [1] and R. Mallée [3] used the normal hole diameter described in [14]. Thus, these studies do not provide guidance for oversized holes. However, in the study by K. Mcbride [2], all tests were performed with oversized holes. 'According to [7], the shear capacity for oversized holes should not be determined on the basis of equation (2), but on the basis of equation (3) *bending resistance of anchors without mortar joint*. However, that seems very conservative based on the test results. Even the lowest test results do not show such a large reduction as suggested in [7].' Also, the study on the influence of steel shims in steel-steel joints [17] shows that shear capacity hardly decreases (Figure 5-13). However, deformation does increase significantly.

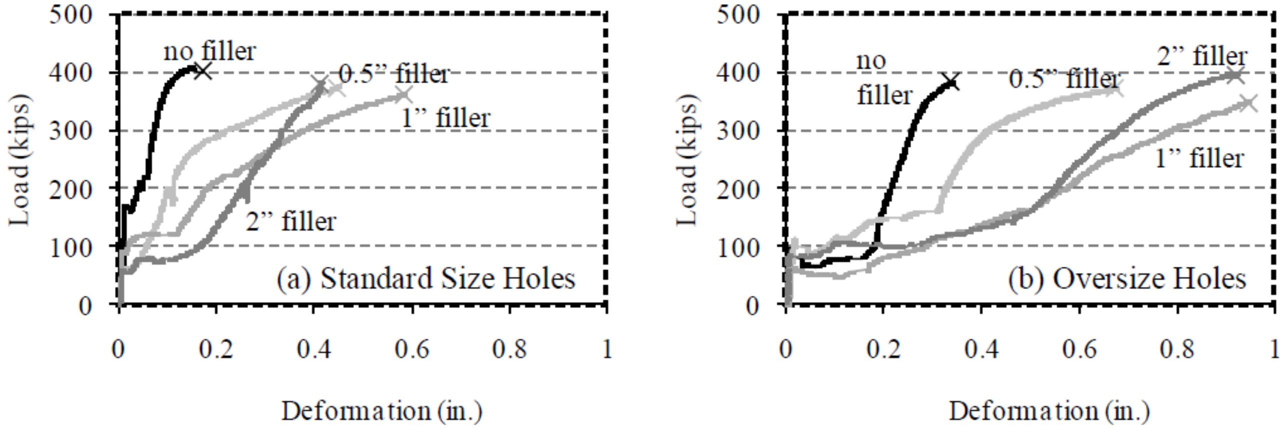


Figure 5-13 Example of deformation of three bolt rows with different shim thickness [17, p. Figure 7].

5.5. Distortion

In the tests conducted, including those of steel-steel joints, the deformations that occurred were also measured. These deformations run up to 30 mm in some tests. Such deformations are undesirable or even inadmissible for various practical reasons.

The various studies summarily write the following about this:

- The CUR10: "For reinforcement steel, requirements for elongation at break are given in Annex C of NEN-EN 1992-1-1 [20]. For threaded studs, the requirements apply in accordance with NEN-ISO 891-1. For short anchors, the requirements are taken from Annex B of CEN/TS 1992-4-1. The breaking strain requirement of 12% is very high compared to the required breaking strain of reinforcing steel." [6, p. T4.2(2)
- One of the recommendations by L. Bouwman et al [1] for further research is the following: "At the use stage, displacements in the order of 3 to 6 mm could occur. This will mostly involve cracks in the grout. No criteria were found regarding the permissible displacement. It is recommended that criteria be established in dependence on the nature of the construction..." [15, p. 167]
- The study by P. Dusicka et al [17] indicates that oversized holes create additional bending in the bolts because the head can twist more freely in a wider hole than in a normal hole. The head is supported worse (Figure 5-14).
- In the past, NEN 3880-Part C, among others, gave an advisory value to allow a maximum deformation of 0.1*d with the following note: "The deformation of the anchor is mainly determined by the crushing of the concrete by exceeding the compressive strength. It is at the discretion of the structural engineer, which deformation is acceptable. The test results show a strong spread with respect to the deformation. The value δ = 0.1*d should therefore be regarded only as a global indication."

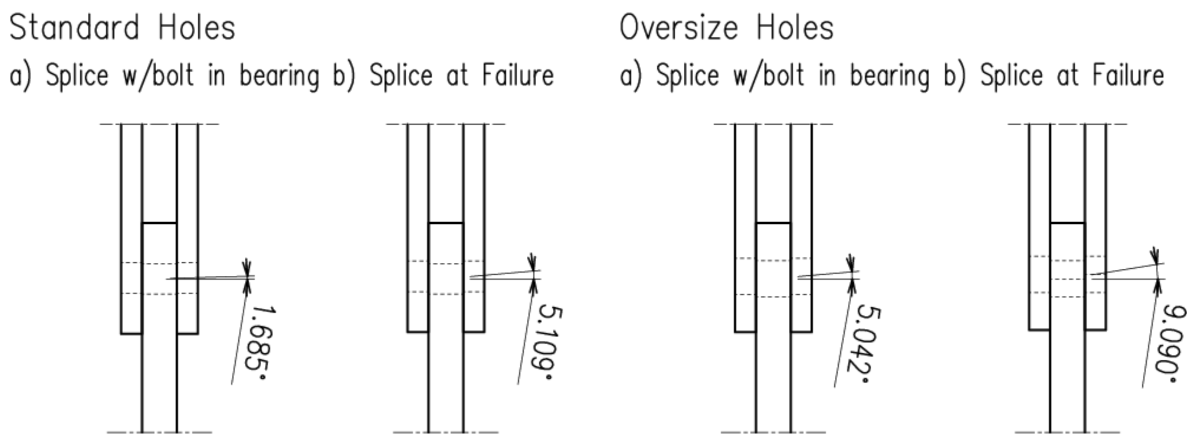


Figure 5-14 Theoretical angle due to bolt deformation at shear for normal and oversized holes [17, p. Figure 31]

5.6. Conclusions

In this section, a common thread has been drawn through all the experimental results so that an upper and lower bound is created. For each section, the various concerns and conclusions will be described.

5.6.1. Calculation models

The various studies show basically the same behavior of the connection as described at the beginning of Section 5.1. The study by Mcbride [2] goes into this most deeply and introduces quite complex equations in the study that describe quite accurately the behavior for the three situations described. These equations were not presented in this study. These equations include the friction factor (μ) and that is where the complexity lies for daily practice. There is discussion about the actual friction factor and it is therefore questionable whether it is advisable to include it as a variable in practical engineering formulas. For this study, these formulas are therefore disregarded.

Mallée's study [3] provides a fairly practical equation (14) for determining the shear capacity. However, equation (14) has the disadvantage that for a single anchor row, the shear capacity becomes equal to equation (3) *bending resistance of anchors without mortar joint*. This corresponds to the boundary condition of equation (9) *shear capacity of anchors with a mortar-filled joint*. Based on this proposal, then, the question is whether a single-row anchor pattern is desirable (Figure 5-15). The American equivalent of NEN-EN 1090-2 [14], the OSHA 29 CFR 1926 Part R [21], states that a structural column must have at least four anchors (two rows) and that only for structural posts two anchors (one row) may be used. According to [21], the definition of a style is that it weighs only 300 lbs (136 kg), is not axially loaded and is placed only vertically. Thus, structural members must be secured with four anchors, which is a logical requirement in light of Mallée's findings.

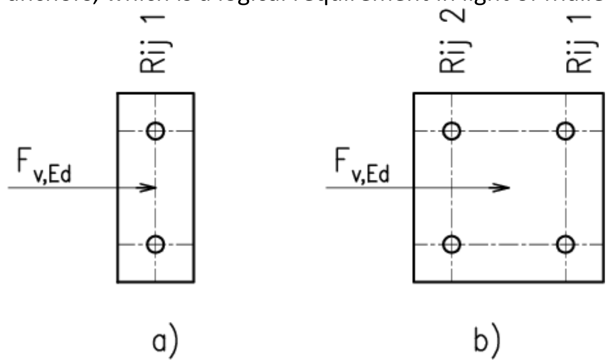


Figure 5-15 Load classification by anchor row for one and two rows Row 1: anchors with bending resistance; row 2: anchors with shear resistance

5.6.2. Results

Several trends emerge in the different tests [1], [2] and [3] that apply to all test results. These will be named first. Then, for the individual trials, points of interest that specifically follow from those trial series will be cited.

- The experimental results complement each other. A good picture emerges of low to high set spaces and anchor sizes from M12 to M30.
- From the test results, the lowest shear capacity appears to occur at a set space of 2*ds. At Bouwman, the lowest value for tests with M20 is at 30 mm. At Mcbride, the lowest value for tests with M16 is at 35 mm. At Mallée, the lowest value for tests with M12 is at 20 mm (Table 5-1).

	Research	Adjustment space at lowest test result (mm) According to figure 5-5 to figure 5-8	d (mm)	Axis (mm²)	ds (mm)	2*ds(mm)
M12	Mallée	20	12	84,3	10,4	20,7
M16	Mcbride	35	16	157	14,1	28,3
M20	Bouwman	30	20	245	17,7	35,3
M30	Mcbride	95	30	561	26,7	53,5

A_s = *shaft area*, $d_s = \sqrt{(A_s*4/\pi)}$

Table 5-1 Adjustment space in relation to shaft diameter

- Initially, the shear resistance of the joint decreases and then increases again. This can be explained as the actuating space (e) increases and the eccentricity moment ($F_{v,Ed}*e$) therefore also increases. A larger moment creates a larger compressive and tensile force (D and T_1) and the larger compressive force also results in a higher frictional resistance (Vw). The total shear capacity (V1, V2 + Vw) then increases. The tensile stress created by the tensile force ($_{T1}$) in the anchor will increase proportionally less rapidly than the shear stress decreases. This is a logical behavior given the mechanical relationship in which the shear capacity of steel is equal to $f_u/\sqrt{3}$. Thus, the total shear capacity of the joint increases. The EEM model will have to show the equilibrium situation and the gradient between the maximum shear force and acting tensile force.

- Compared to equation (3) *bending on anchors without a filled joint,* the anchor capacity found is significantly higher when the setting space is filled with mortar. Especially for the larger setting spaces, this increases to as much as 5x the capacity according to this equation (Figure 5-9). For example, at 60 mm of setting space, β according to (3) is about 0.15 and the β found from the various tests is not lower than 0.8.
- All test results fall above equation (14) for two anchor rows with a total of four anchors of Mallée in which for the β of the shear capacity in the determination the bolt value of NEN-EN 1993-1-8 was used and not that of NEN-EN 1992-4. In NEN-EN 1993-1-8 the αv is 0.6 and in NEN-EN 1992-4 it is 0.5. An αv of 0.6 does not seem to do injustice to the shear capacity.
- The shear capacity according to equations (9) and (11), seen for the lowest test results, gives an excessive capacity. Especially when the standard deviation on the test results are considered.

Bouwman

- All the results found are significantly higher than the calculation rules proposed in the study. The smallest safety for M20 4.6 is 56.7/28.9 = 1.96 and for M20 8.8 it is 61.8/38.9 = 1.59 (Figure 5-5, Figure 5-6 and the shear capacity according to equation (8)). Because only a small number of tests were performed of each arrangement, a standard deviation must be taken into account making this safety probably appropriate. Nevertheless, the safety for class 4.6 seems too high compared to that for class 8.8.
- The M20 8.8 test series by L. Bouwman et al. tested two M20 anchors in a single anchor row in the direction of force. It should follow from Mallée's proposal that the shear capacity of these anchors is equal to that of anchors without a filled joint. From the results of these tests, such a drop in capacity does not appear to occur and thus the single anchor row proposal and also the boundary condition of equation (9) seem unnecessarily conservative.
- The results of the sand-cement tests show a wider spread and, therefore, a greater uncertainty.

Mcbride

- The 6xM30 tests have higher β relative to the set spaces than the 2xM16 tests and are more consistent in terms of β with the results of 6xM16. This seems to confirm Mallée's proposal that the average shear capacity of the anchors increases when multiple rows are applied.
- Test results 2xM16 (Table 4-3 in Appendix 4) show a lower β than 0.8. It is unclear why the study does not hold these lower values alongside equation (10). With a lower number of anchor rows, a β of 0.8 seems too high.

Mallée

- In the tests by R. Mallée [3], two different pitch patterns of the anchors were applied. It follows from the results that for the large pitch the shear capacity is lower than for the small pitch. The article indicates that no explanation can be given for this from the results and that more tests are necessary. It is possible that a smaller pitch results in a greater compressive force on the mortar which increases the proportion of friction and therefore the shear capacity of the joint is higher. This fits into the analysis of L. Bouwman [1] for balancing the occurring load.
- Around 45 mm of adjusting space, and - if this were the arithmetic adjusting space - a physical adjusting space of about 30 mm (see Section 5.3.2), the four equations (9), (10), (11) and (14) including the proposal of R. Mallée [3] give almost the same shear capacity.
- In Mallée's tests, the series with a pitch of 65 mm (5.4*d) does not meet one of the boundary conditions of equation (9), namely that the pitch must be greater than 10*d. However, the results of these adhesive anchors are higher than the tests with a pitch of 190 mm (15.83*d). Within the Class 4.6 tests of L. Bouwman et al. the pitch is also smaller than 10*d and again the tests do not fall proportionally lower than the other tests that do meet this condition. Based on these results, the boundary condition does not appear to apply to the shear capacity of anchors.
- R. Mallée's proposal based on three anchor rows with a total of six anchors gives lower shear capacity than the test results of six anchors. The test results that fall on the limit of equation (14) (Figure 5-8) are from setups with one anchor row.

5.6.3. Add

Casting mortar provides the best test results with respect to the shear capacity of the anchors. Cast mortar is low shrinkage and also stiffer than, for example, a sand-cement joint. Since NEN-EN 1090-2 [14] was introduced in 2018, the type of grout is no longer a concern for this type of research. This standard [14, p. 5.9] indicates that only specific types of mortar may be used: cement-based mortar, special mortar or fine-grained concrete. The survey shows that the majority of respondents indicated that K70 is used and almost all respondents indicated that cast mortar is used. An important property of poured mortar is that it has a swelling coefficient of 0.5-2% (Exhibit 3) and thus expands after curing. This is opposite to ordinary cement or concrete where the liquid mass actually shrinks after curing (Figure 5-16). As the poured mortar expands, it compresses and adheres to the surfaces of the concrete and steel and there will be less or no loose grout. The tests by A.M.P. den Deurwaarder [18] show that with cast mortar, friction occurs directly. This can be explained by the fact that with a cast mortar joint, the eccentricity moment due to the tensile
/pressure torque in the mortar and anchors can be absorbed faster than with sand-cement mortar. Because of the denser and stiffer mortar, no or less physical deformation is required before pressure can be deposited on the mortar.

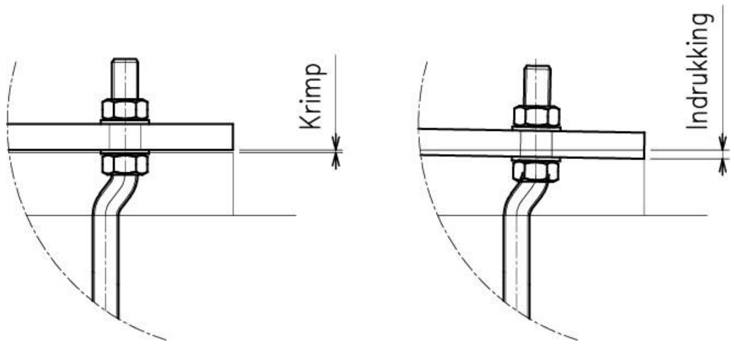


Figure 5-16 Physical characteristics of non-shrink and weak joints

Figure 5-12 shows that when filling with steel shims, the lowest shear capacity of the bolts is found approximately at 2*ds (2*20 = 40 mm = 1.56 inches), and this appears to be the case for oversized holes as well. The tests were performed for 1 and 2 inches, but for this bolt size, perhaps the worst value is at 1.5 inches. This is purely hypothetical, of course, but the results give room that this could be the case. Their conclusion shows that - depending on the adjustment space - there is apparently different behavior in the bolt. Up to a certain filling height, the bolt will have the greatest stiffness which will cause it to contribute to the shear capacity in the form of bending. As the headroom increases, the stiffness lags behind, increasing the deformation and causing the joint to compress at the front and, for equilibrium, to pull at the rear on the bolts, as has also been stated by L. Bouwman et al. [1], K. Mcbride [2] and R. Mallée [3] at anchor joints (Figure 5-11). Thus, it strongly appears that the same behavior occurs in steel-steel joints during spacer assembly as in steel-concrete joints.

Both the test results for steel-steel and steel-concrete seem to have a valley. And the lowest valley will be the upper limit of the allowable shear capacity. The lowest test results will numerically give the highest allowable β by which to reduce the shear capacity $F_{v,Rd}$.

5.6.4. Hole clearance

The results of the various tests show that the size of the hole clearance does not significantly affect the shear capacity but does affect the deformation of the anchors.

5.6.5. Distortion

A few things do matter when determining β when it comes to deformation:

- Oversized holes have a minor effect on the shear capacity of the anchor, but have a significant effect on deformation.
- Large deformations occur in the experiments. If there are requirements for this, it may affect the allowable shear capacity. At present, no concrete requirements have been articulated in the standard and the level of shear capacity can be determined based on the breaking strength or maximum elongation of the anchors. If the deformation of the anchor would be leading for the maximum allowable shear force, it might be appropriate to add a correction factor for hole clearance for the shear capacity - just as for thrust capacity [4, p. 3.6.1 - Table 3.4 footnote 1].

For unclear reasons, the recommendations and proposals were never translated into concrete requirements in codes [4] and [5]. This thesis study, in part due to lack of concrete requirements, is specifically concerned with an acceptable upper and lower limit of shear capacity of anchors rather than deformation. The results of the EEM model do show the deformation (δ) at the maximum strain and break value. Also shown is the acting load at the recommended δ according to Section 5.5.

6. EEM model

In this chapter, section 6.1 describes the EEM model. Section 6.2 presents the overview of the analyses made and, finally, section 6.3 describes the results of this model. This model was created to initially simulate the behavior of the tests based on the calculation value according to the current standards. If there is an agreement between the results of the model compared to those of the tests, then the model can be used to determine a computational limit of the shear capacity of anchors and of the β sought. Chapter 7 will describe the conclusions and findings of the EEM model.

6.1. Model

Using a python script, Diana generates a model including loads, supports, interfaces, computational methods and outcome types sought (Figure 6-1). This script allows many different analyses to be performed easily by adjusting desired parameters in the script. Appendix 5.1 shows the section of the script where parameters can be adjusted.

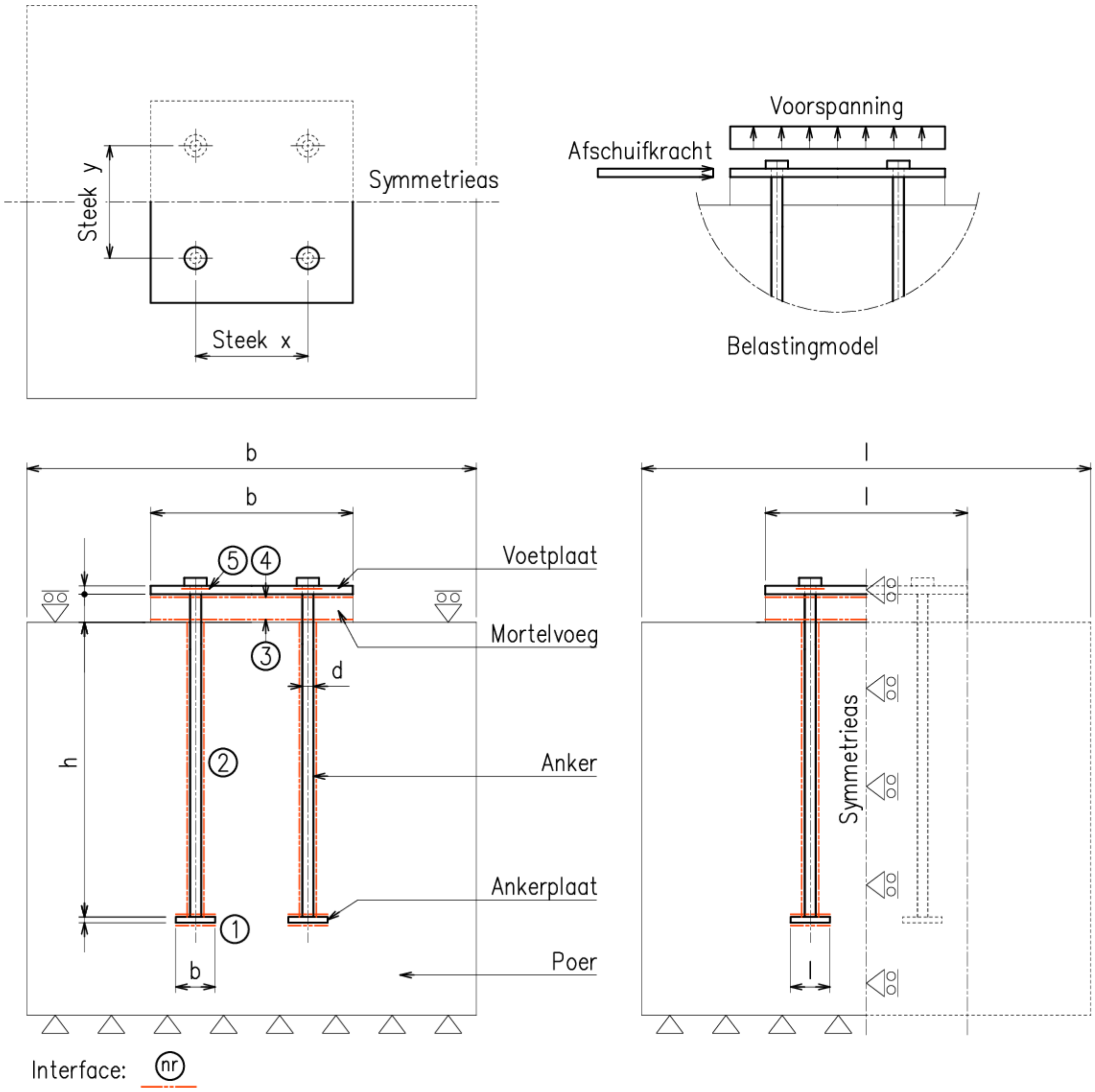


Figure 6-1 Drawing of parameters and components as the EEM model is generated by Diana

6.1.1. Construction

The model consists of four basic components: footplate, anchors, mortar joint and pile. It was chosen not to use an adjusting nut under the footplate. Without an adjusting nut, the EEM model corresponds to the problem outlined in the introduction. Without adjusting nut also results in the longest and therefore most unfavorable bending length (see Section 5.3.2). The following sections highlight the choices made regarding the basic components. All materials, interfaces, etc. cited are fully detailed in Appendix 0. To keep the number of elements small, a half model is generated by the script. The model has a pure axis of symmetry and can therefore be halved.

In addition to the basic components, the model contains a variety of subcomponents to create, for example, a pure element grid or contact surfaces. These are necessary to make the analysis pure and faster. The model is available from the "highest" point on the z-axis constructed from planes that are then given a thickness. Working from the top, Diana is forced to create an even grid across the z-axis (Figure 6-2). This results in higher purity of results for several reasons. The two main reasons are:

- The extrusion of the surfaces forces the EEM model to create cubic elements. This prevents the generation of pyramidal or other triangular-shaped elements. Such elements behave more rigidly locally than cubic elements. Integration points of such elements are more likely to create peak stresses, causing cracks to form locally in the concrete sooner and plastic strain to be reached more quickly. With cubic elements, the results become more accurate.
- The integration points do not extend in the x/y plane. This also contributes to the accuracy of the results.

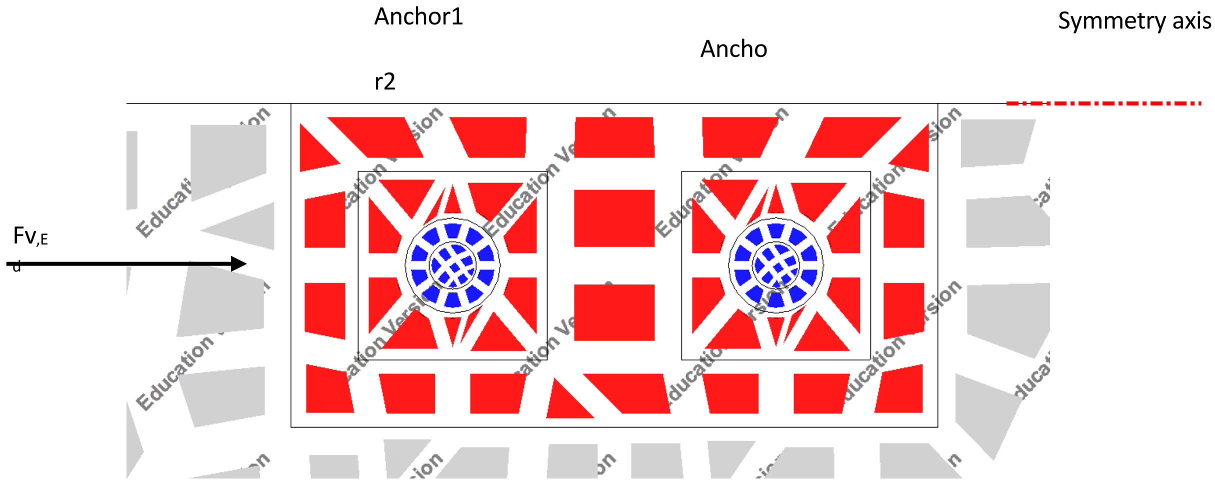


Figure 6-2 Top view of the element grid from the 3D EEM model

6.1.2. Support centers

At the bottom of the pile, the model carries all vertical and horizontal loads. To prevent the pile from rotating, because the horizontal load can only be transferred at the bottom, the pile is clamped on top at the outside edges. This arrangement was chosen because the tests described in Chapter 5 were also constructed this way. The slab, mortar joint and the pile are given a fulcrum at right angles to the plane to stop the shift about the axis of symmetry.

6.1.3. Dimensions

For the base parts, all outer dimensions (w, l) and thickness (h) can be adjusted. For the anchors, pitch (x and y), length (h) and follower plate size (b, l) can also be adjusted.

6.1.4. Materials

For the base plates, material classes S235 and S355 can be used. Basically, S235 has been retained for this. For the analyses with M30 anchors, S355 has been retained so that the head plate of this analysis does not become normative.

For the anchors, classes 4.6 and 8.8 can be applied. In the analyses, mainly class 8.8 was used because it was also used in most of the test runs. A series with class 4.6 was also analyzed to validate the model using the test series of L. Bouwman et al. [1]. In the study of [1, p. 133], the maximum elongations for this class are cited in the recommendation. The minimum elongation at failure for class 4.6 is 25% and for class 8.8 it is 12%. After an integration point reaches this strain, the absorbable stress at the increase in strain immediately decreases to 0 N/mm^2 . This is an underestimate of the actual capacity and therefore the material will behave more brittle, among other things (Appendix 5.2 and Figure 6-3).

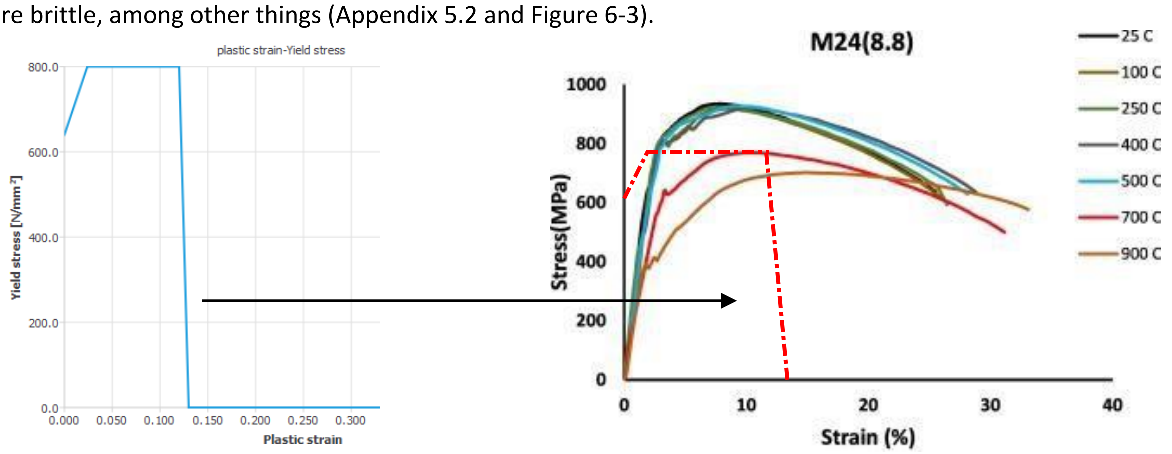


Figure 6-3 left) mechanical properties from the Class 8.8 EEM model;
right) mechanical properties at different temperatures from tensile tests for M24 8.8 [22]

For the mortar joint, class K30, K50 and K70 can be applied. The survey shows that K70 is mainly applied and this grade was therefore applied in all analyses. To determine the influence of mortar class on shear capacity, analyses were also performed with class K30. For all classes, the maximum compressive strength maintained is equal to the denominator of the class. Thus, for K70, 70 N/mm^2 was retained for this purpose. This is a conservative value because most K70 casting mortar is supplied with a higher compressive strength of up to 95 N/mm^2 (Appendix 3). The material model of the mortar conforms to guideline [23]. This guideline - issued by the Department of Public Works - describes the settings and calculations of, among other things, the behavior of concrete C70. In the case of the material behavior under pressure, this guideline was deviated from with respect to the plastic gradient. With the standard prescribed behavior, when the concrete has reached the maximum pressure at which the concrete would fail, the capacity of the concrete decreases to 0 N/mm^2 and the deformation of the footing plate would also react to this as if there were no longer any material at the location of the failed mortar joint, the footing plate would then suddenly lie free when concrete failed. However, the tests show that mortar joint breaks into pieces and physically remains in place. Thus, the footing plate is still kept from tipping over. It was therefore decided to make the material behavior ideally plastic so that after the mortar breaks into pieces, the footing plate cannot tip over.

All common concrete strength classes can be used for the pile. In the analyses, C30/37 was used because it is currently the most common grade. The material behavior subject to tensile and compressive forces develops linearly elastic. No dominant cracks (microcracks excepted) are expected in the concrete that impair the overall behavior of the shear capacity of the anchors. At the pressure point of the anchor against the concrete at the top of the pile, spalling could be expected, but based on Mallée's proposal [3] (Section 5.3.2), that behavior is disregarded. Therefore, linear-elastic behavior can be applied. The results best approximate the "fictitious" anchor trapping depth of 0.5*d (a3) (Figure 3-2 and Figure 6-4). Also, for an EEM model, if more nonlinear-elastic material behavior is applied, it becomes more complex to find an accurate solution. By avoiding nonlinear-elastic behavior whenever possible, the overall accuracy of the results is increased.

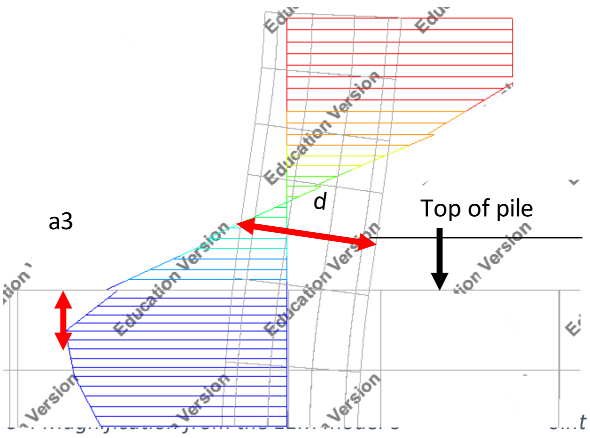


Figure ... Magnification from the EEM model ... t of the anchor in the concrete

6.1.5. Interfaces

All components in the EEM model are linked by means of so-called interfaces. In such an interface can be indicated how a coupling should deal with the various loads that may occur therein.

Number	Parts		Properties of transition
1	anchor plate	poer	anchor plate can transfer compressive force to the concrete, but no shear or tensile force
2	anchor rod	poer	axially, this plane cannot transfer shear force to the concrete
3	mortar joint	poer	friction and compressive force can be transmitted, but not tensile force
4	mortar joint	footplate	friction and compressive force can be transmitted, but not tensile force
5	nut	footplate	clutch can transmit compressive force, but not tensile force

Table 6-1 Description of the interface in the EEM model, for numbers see Figure 6-1

For transmission of shear force by friction, the Mohr-Coulomb criterion is adopted. The plane between steel and concrete cannot incorporate friction in the basic analyses. In the execution of the joint, there is uncertainty whether friction is present and, if so, how high or low the frictional resistance is. This may be because, for example, the mortar joint is not properly filled, but also certain types of preservation, such as hot-dip galvanizing, have low friction angles. Therefore, in Mallée's tests [3], this plane was coated with Teflon film in order to force that no friction can be absorbed. The generally accepted friction angle of 30° (μ = 0.58) was adopted for the transition between the mortar joint and pile (concrete-concrete).

Apart from Mallée's tests, where the transition between steel and concrete is controlled by Teflon film, it is unknown what the friction angles were in the tests. Therefore, EEM analyses were also performed with increased friction angles. For the transition between the mortar joint and the pile, the friction angle was increased to 45° (μ = 1), and for the transition between the mortar joint and steel, a friction angle of 30° (μ = 0.58) was used. These are overestimated friction angles given the standard, but the possibility exists that they may have been physically present in the tests. By performing the analyses with the prescribed and with the increased friction angles, the influence of friction is depicted.

6.1.6. Occurring taxes

In the analyses, the maximum shear capacity of the joint is sought. For this purpose, a distributed load of 100kN is applied to the entire side of the plate so that it acts on the center of the footplate, as in the tests. Thus, a shear load of 50kN is present in the EEM model because a half model is built. The EEM model will be able to represent the behavior of the node for each load step. A load factor is associated with each load step. This load factor is the magnification/decrease of 100kN.

A tensile force was also applied to the plate in the model as a distributed load in order to simulate the prestress in the anchors. For this preload value, 25% of the computational tensile capacity of the anchors was retained as a numerical interpretation of the concept of hand-tightening. Without an adjusting nut, a structure with adjustment space can only be tightened in, for example, a vertical orientation where a joint is also present on the other side. The spring created by pretensioning is not located between the adjusting nuts (Figure 6-5 a) but throughout the joint (Figure 6-5 b). This spring is not modeled in the EEM model and therefore the applied tensile force creates a permanent tensile stress in the anchor. In the situation with an adjusting nut, no tensile force is initially present in the anchor shaft under the nut and, without shear force, it is still unloaded. In the EEM model, however, 25% of the computational tensile capacity is already present in the anchor shaft in advance. This model choice is an unfavorable interpretation of the tightening torque and therefore underestimates the maximum capacity of the anchor.

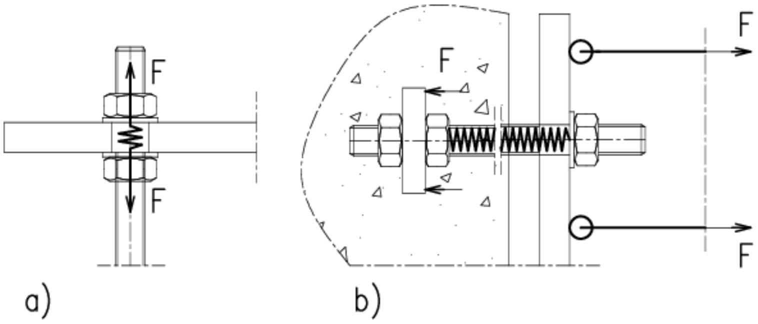


Figure 6-5 Springs model at prestressing
a) balancing springs between pressure on footplates created by pull on anchor shaft between nuts
b) without adjusting nut, preload force will have to balance between anchor in concrete and steel element

6.1.7. Calculation method EEM model

The models are calculated in two phases. These phases simulate the assembly sequence as performed in practice.

In the first stage, the mortar joint is omitted, preload is applied to the anchors, and all parts are given their own weight. The model has very small displacements due to these loads, but these are calculated.

In the second stage, the mortar joint is added. This creates a close fit between the initial deformations and the mortar joint. In practice, the mortar joint is also applied only after the steel element is assembled. By doing this in two stages, it is avoided that the model initially already has small empty spaces between the steel plate and the mortar joint. Since this transition surface cannot absorb tensile, a slight displacement of the prestress would create an opening between the steel plate and the mortar joint. This gap would result in a large inaccuracy and increase in calculation time. This is avoided in this way. In the second stage, the horizontal force (shear force) is also applied to the side of the base plate. The horizontal force is applied based on "arc length control. This is an indirect displacement-controlled calculation method. If the horizontal load were applied in standard step sizes and the last step was beyond the top of the capacity, the model cannot properly calculate the last step (Figure 6-6 a). As a result, the results become inaccurate and the picture also becomes incomplete. Because different effects are going to occur in this model, as shown in the studies in Chapter 5, the model will need to be able to describe different stages. The friction and interaction of tensile and shear stresses will always find a different equilibrium depending on the deformation. With "arc length control," the EEM analysis is able to self-determine the step size of the load and also step back (Figure 6-6 b). In this way, the nonlinear behavior of the joint can be mapped and the complete force-displacement diagram including the peaks can be calculated.

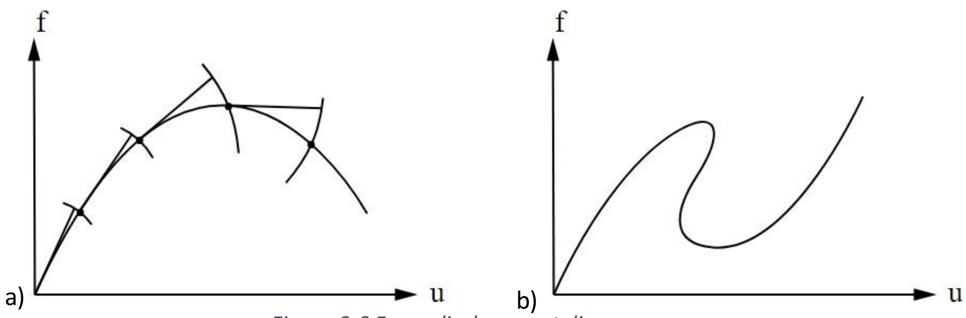


Figure 6-6 Force-displacement diagrams
left: Standard step sizes; right: full arc length controlled with steps back

6.2. Calculations performed

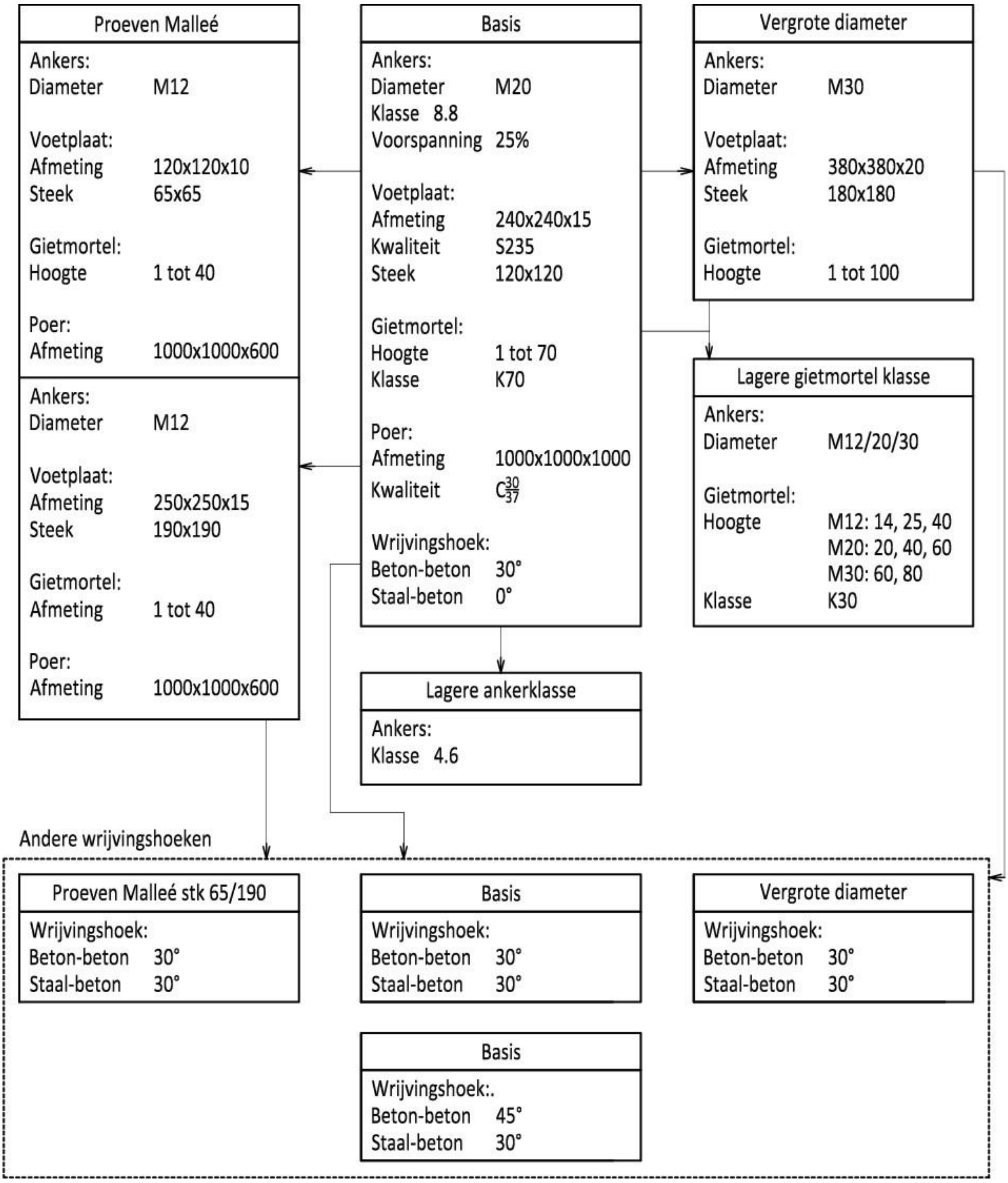


Figure 6-7 Overview of the calculation performed in Diana, all dimensions in mm

Figure 6-7 describes the parameters of the analyses performed. From the base, only the adjusted paramaters are shown for the follow-up configuration.

6.3. Results

This section presents the results of the EEM model. The reliability of the EEM model based on the results will also be explained. The results will be presented in the β sought relative to the margin of adjustment. In Section 5.2 it was elaborated that for β the partial safety factor $\gamma m2$ is not applicable. The found value of the shear force is divided by the calculation value of the shear capacity of bolts and anchors according to equation (2). This directly results in the β associated with the calculation value of the shear capacity. The corresponding deformation with respect to the setting space will also be shown.

For each series where the diameter, anchor class, mortar type, pitch and all other dimensions are the same, results will be presented for four parameters.

I) *Maximum plastic strain in anchor*. The moment an integration point in the anchor reaches the maximum strain of the anchor class, one can speak of technical failure. An increase in load will result in breakage of the anchor.

II) *Maximum buckling load.* After an integration point in the anchor reaches maximum strain, the anchor will begin to constrict. Because there will be bending in the anchors, the maximum strain will occur on the outside of the anchor and a portion of the anchor may still take up additional capacity until the entire cross-section of the anchor reaches maximum strain. The anchor will deform more and the friction will absorb a greater proportion of the shear force occurring. The total shear capacity of the joint increases. The maximum failure load provides insight into whether the EEM model shows the same behavior as the tests. These results are not relevant for a responsible computational value because large deformations occur here, the connection rests largely on friction, and failure of the anchor is not a responsible limit. Because it is unknown what the actual friction angles were in the tests, the increased friction angles were used in the analyses of the EEM model for this failure load. This gives a clear picture of what influence friction can have on the total shear capacity.

III) *δx of 3 mm top of footplate*. This is the smallest value of deformation from the recommendation of L. Bouwman et al. [1] (see section 5.5). To properly reflect the relationship of these results, relative to the other results, lowest value was chosen.

IV) *δx of 0.1*d top of pile.* This is an advisory value for the size of the deformation of the anchor at the top of the pile, see section 5.5 and NEN-3880 [8]. At higher set spaces, there are results where at the maximum failure value no δ of 0.1*d occurs. In these situations, the maximum failure value has been retained.

Appendix 0 shows numerical and visual results of all analyses. The visual results also include the results of the tests. Of some analyses, the visual outputs of the EEM model are also added.

For the first analysis series of M20 8.8, results with additional friction are shown twice:

- *Maximum load with friction head plate* - a friction angle of 30° is maintained for the concrete-concrete and steel-concrete coupling.
- *Maximum load with increased friction concrete* - the friction angle between concrete-concrete is increased to 45°.

These two sets of results reflect well the impact of friction on the maximum shear capacity. They approximate in behavior the experimental results because in all but Mallée's tests, friction will have been present under the footplate. For the other analyses, only the *maximum load with friction head plate* is shown. These results already accurately represent the behavior of the tests and, because it represents the breaking value, it is not relevant to show the *maximum load with increased friction concrete* for all tests.

6.3.1. Results single analysis

For the basic analysis of 4xM20 8.8 with 40 mm of adjustment space and different friction angles, the results are shown and explained in this section (Table 6-2 through 6-5). In this way, the data of all analyses of Section 6.2 are shown in Appendix 0. Table 6-2 shows the basic structure of the model.

4xM20 8.8 base plate 240x240x15 pitch 120x120						
Shear force (kN)	Traction (kN)	Preload/ anchor (kN)	Mortar class	Setting space (mm)	Friction concrete-concrete (°)	Friction steel-concrete (°)
100	0	35,25	K70	40	30	0
100	0	35,25	K70	40	30	30
100	0	35,25	K70	40	45	30

table 6-2 Basic structure of model

Of the four parameters sought, the corresponding load steps, load factors, elongations and δ are shown (Table 6-3):

I) Maximum plastic strain in anchor				II) Maximum collapse load				III) δ = 3 mm top of footplate				IV) δ = 0.1*d top of pile			
Load-step	Load-factor	Rack in anchor (%)	δ (mm)	Load-step	Load-factor	Rack in anchor (%)	δ (mm)	Load-step	Load-factor	Rack in anchor (%)	δ (mm)	Load-step	Load-factor	Rack in anchor (%)	δ (mm)
50	1,965	11,8	9,23*	54	1,987	15.1	9,99	17	1,453	2,1	3,0*	54	1,987	15,1	2,0*
62	2,081	12,0	9,35	67	2,108	15,6	10,12*	21	1,509	2,2	3,0	63	2,090	12,6	2,0
62	2,230	12,1	9,35	67	2,251	15,8	10,07*	20	1,550	2,0	3,0	65	2,246	14,3	2,0

* Presented values in Figure 6-8 and Figure 6-9

Table 6-3 Value of parameters sought

For parameter I, the internal forces (tensile, shear and moment) are shown (Table 6-4). Based on these values, the following can be determined:

- The external shear force that occurs must be equal to V1+V2+Vw. Subtracting the shear force of anchor1 and anchor2 (V1 and V2) from the total shear force occurring (load factor* shear force) leaves the shear force absorbed by friction (Vw).
- The compressive force (D) on the mortar is equal to the tensile force of anchor1 and anchor2 ($_{T1}$ and $_{T2}$) minus the prestressing force.
- The friction factor is equal to Fw/D, see equation (13).

Appearing shear force (kN)	Traction (kN)		Shear force (kN)		Moment (kNm)		Friction (kN)	Share friction (%)	Pressure force on mortar (kN)	Friction factor
	Anchor1	Anchor2	Anchor1	Anchor 2	Anchor 1	Anchor2				
196,45	52,98	42,56	41,05	44,76	0,780	0,764	12,42	13	25,04	0,50
208,14	53,59	41,72	41,88	48,78	0,780	0,757	13,41	13	24,81	0,54
223,00	53,58	43,65	41,46	45,74	0,781	0,763	24,30	22	26,73	0,91

Table 6-4 Internal forces and friction factor of parameter I

From the acting load follows β (Table 6-5) based on equation (12).

Reduction factor due to headroom				
Bolt Capacity EN 1993-1-8	β I)	β II)	β III)	β IV)
94,1	0,52*	0,53	0,39*	0,53*
94,1	0,55	0,56*	0,40	0,56
94,1	0,59	0,60*	0,41	0,60

* Presented values in Figure 6-8 and Figure 6-9

Table 6-5 β of the sought parameters based on equation (12)

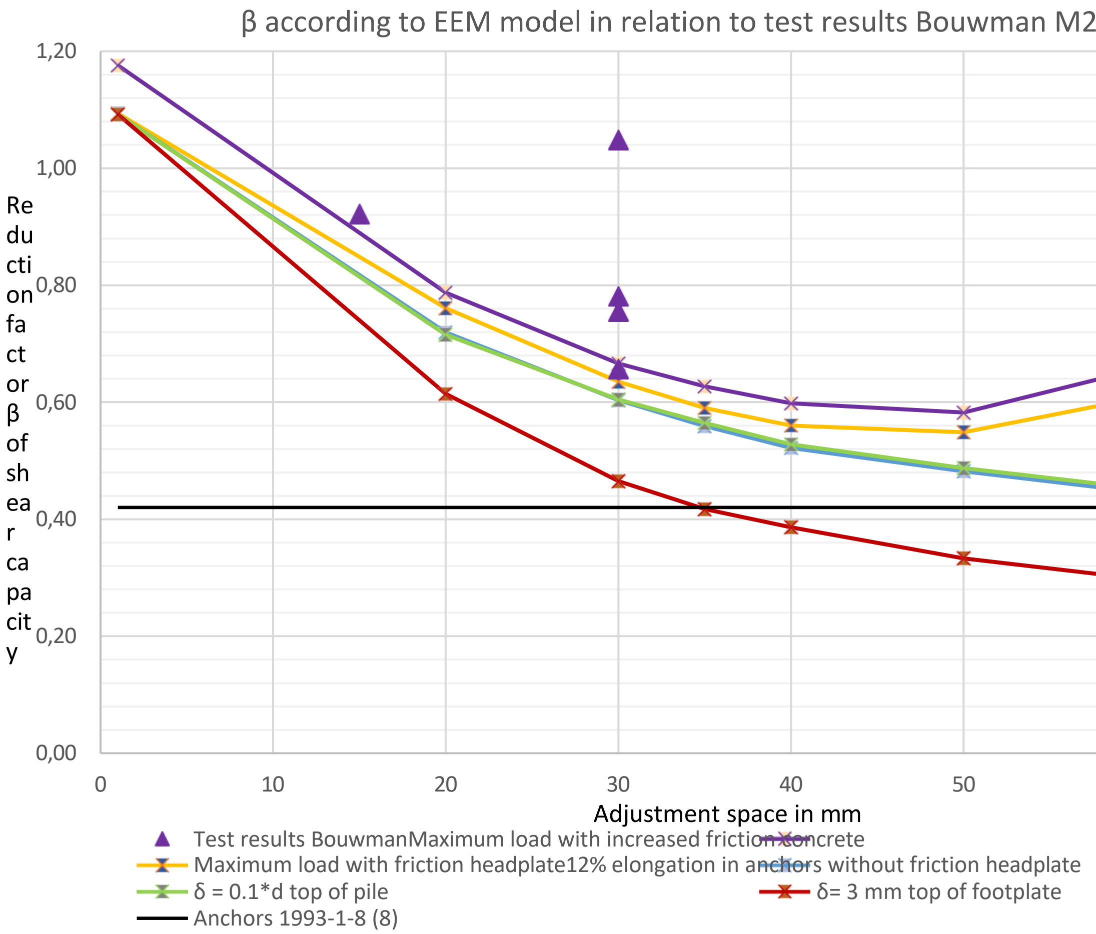


Figure 6-8 β according to EEM model in relation to test results Bouwman M20 8.8

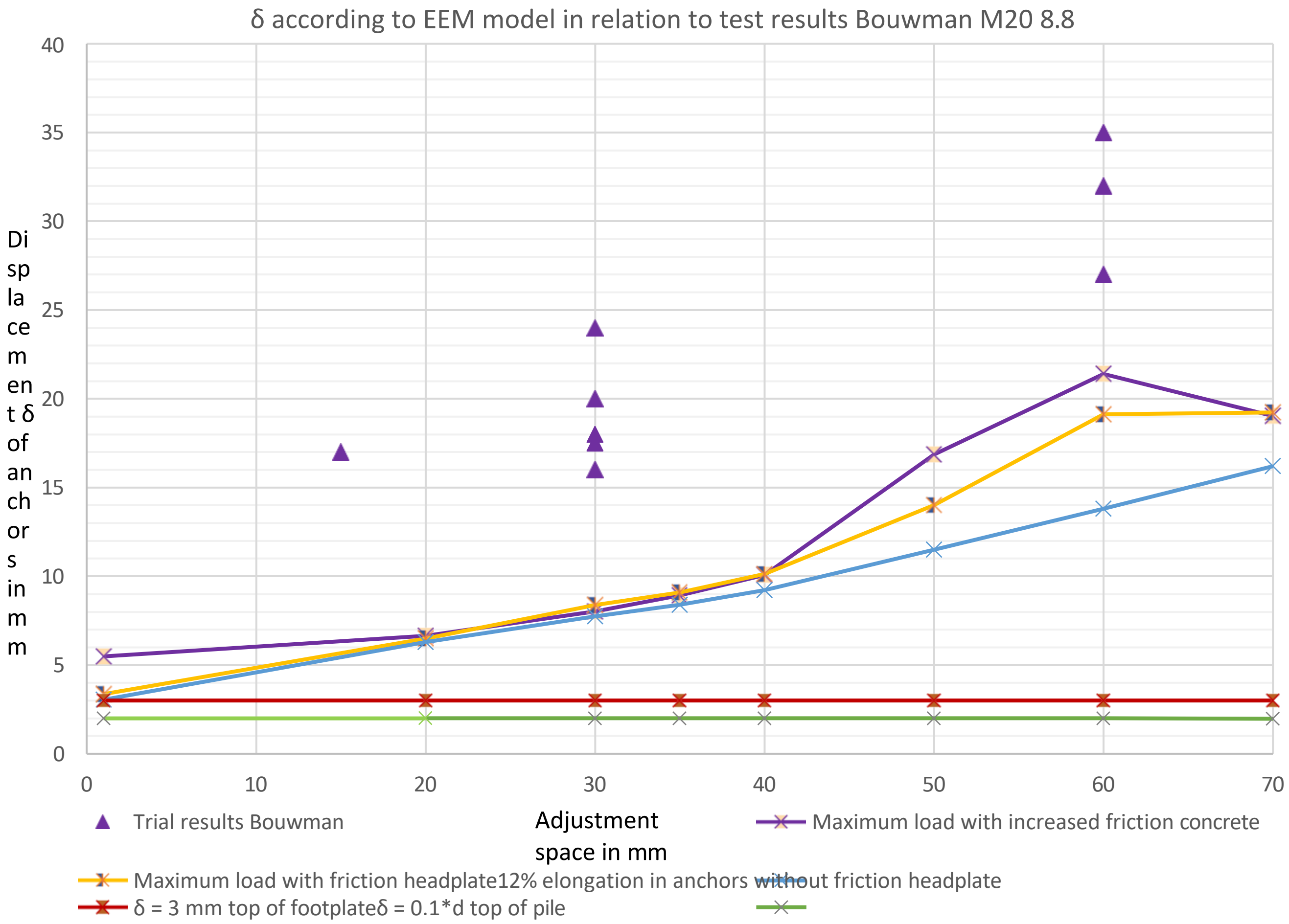


Figure 6-9 δ according to EEM model in relation to test results Bouwman M20 8.8

6.3.2. Control results EEM model

Shear capacity anchors

In the analysis of the EEM model with 1 mm of headroom, the maximum shear capacity of anchor1 is 102.1 kN (Table 6-6). The model incorporates the characteristic material properties. This means that the occurring shear force should approximate equation (1) *characteristic shear capacity*.

$$F_{v,k} = A_v * \frac{f_u}{\sqrt{3}} = \frac{\pi}{4} * 17,7^2 * \frac{800}{\sqrt{3}} * 10^{-3} = 113.64\ kN$$

Containing: $A_v = \pi/4 * ds2$; ds M20 = 17.7 mm ; fub 8.8 = 800 N/mm^2

That the EEM model shows a (approximately 10%) lower shear capacity may have two causes or a combination of both:

- In the anchors, a tensile/compressive force torque of 20.19 kN is already present at 1 mm of clearance. As a result, a tensile force of 48.94 kN occurs in anchor1. The maximum shear capacity should take into account the interaction of a shear and tensile force (see Section 5.1.1).
- It can be seen from the analysis that the ratio for the tensile strength (αv) is lower than 1/√3 and thus also lower than the widely accepted 0.6. Nevertheless, the shear capacity found in the analysis is higher than the computational value of an anchor without set space. It could well be that αv has been rounded up in equation (2) to (partially) correct the partial factor, just as was done for β (Section 5.2.

Despite the lower result relative to the characteristic shear capacity, it is higher than the computational value of the shear capacity, this results in a β = 1.09. In this, the EEM model gives reliable results.

4xM20 8.8 base plate 250x250x15 pitch 120x120						
Shear force (kN)	Traction (kN)	Preload/ anchor (kN)	Mortar class	Setting space (mm)	Friction concrete-concrete (°)	Friction steel-concrete (°)
100	0	35,25	K70	1	30	0

I) Maximum plastic strain in anchor				II) Maximum collapse load				III) δ = 3 mm top of footplate				IV) δ = 0.1*d top of pile			
Load-step	Load-factor	Rack in anchor (%)	δ (mm)	Load-step	Load-factor	Rack in anchor (%)	δ (mm)	Load-step	Load-factor	Rack in anchor (%)	δ (mm)	Load-step	Load-factor	Rack in anchor (%)	δ (mm)
16	4,110	0,069	3,05	16	4,110	0,069	3,05	16	4,110	0,069	3,0	16	4,110	0,069	2,0

Results by anchor - Maximum plastic strain										
Appearing shear force (kN)	Traction (kN)		Shear force (kN)		Moment (kNm)		Friction (kN)	Share friction (%)	Pressure force on mortar (kN)	Friction factor
	Anchor1	Anchor 2	Anchor1	Anchor2	Anchor1	Anchor2				
410,98	48,94	41,75	102,10	100,70	0,000	0,000	2,73	1	20,19	0,14

Reduction factor due to headroom				
Bolt Capacity EN 1993-1-8	β I)	β II)	β III)	β IV)
94,1	1,09	1,09	1,09	1,09

Table 6-6 Numerical results 4xM20 8.8 K70 with 1 mm adjusting gap

Moment capacity anchors

The maximum moment that can occur in an anchor for M20 8.8 with a casting mortar class K70 occurs from a setting space starting at 40 mm. For the moment, this should approximate the moment capacity according to equation (7) without the correction factor of 0.9 and based on a maximum tensile force because in situation I) the entire anchor deforms plastically and the spatial stress according to the Von Misses criterion can be maintained. This is shown as a correction in the following expression,

$$M_{Rk,s} = \cancel{0.9} * W_{pl} * f_{y\cancel{b}}\ \cancel{f_{ub}} = \frac{17,7^3}{6} * 800 * 10^{-6} = 0.74 \sim 0.78$$

Containing: $W_{pl} = d_s^3/6$; ds M20 = 17.7 mm ; fub 8.8 = 800 N/mm^2

This capacity of the moment approximates the outcome of the model, and again a reliable picture of the EEM model emerges.

Progression of internal forces and friction

Table 6-7 shows the internal forces of a whole series of M20 8.8 anchors with K70 mortar without friction under the head plate. The following points emerge from it confirming the reliability of the model:

- The occurring shear force creates a moment on the joint due to the setting space. Part of the moment is absorbed by both anchors. The part that is not absorbed creates a compressive force on the mortar joint. This compressive force increases as the set space increases. However, this moment count lacks the share of the anchors. The moment theorem including resistance of the anchors can be written as follows:
  $\Sigma M = 0$ ☎D + F_t) * $\delta h + M_1 + M_2 - (F_h - F_w) * V_r$
  The moment theorem including anchor resistance is consistent with the description by L. Bouwman et al. [1, p. 3.1.1] that there is a complex interaction of loads on an anchor and it is consistent with the finding of R. Mallée [3] that bending will be present in the forward anchors.

- In this analysis, the maximum friction angle between concrete-concrete is 30° (μ = 0.58). For 35 mm of set space, μ = 0.53. No friction higher than the specified friction occurs in the model.
- The proportion absorbed by friction (Vw) increases as the actuating clearance increases. Nevertheless, the friction factor decreases after 40 mm of setting space. The total friction that the mortar joint can absorb depends on the maximum shear strength of the concrete. The frictional resistance is limited by the maximum allowable shear stress of the concrete and the area activated by the deformation (Figure 6-10).

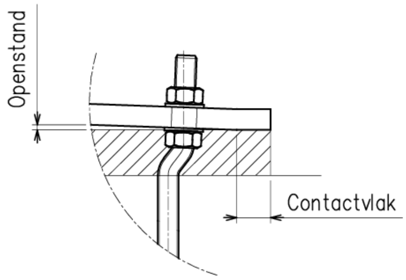


Figure 6-10 Contact surface compressive force

- As the adjustment space increases, the tensile force in the anchors and the maximum absorbable moment of the anchors increases.

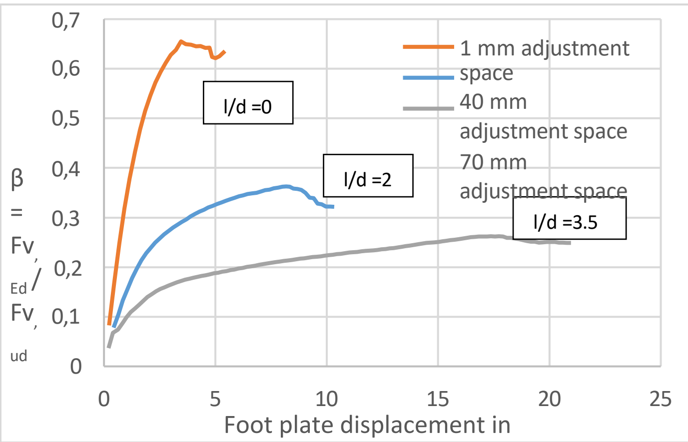
  The maximum absorbable shear capacity decreases due to interaction with the other forces.

	Results by anchor - maximum plastic strain										
Setting space (mm)	Appearing shear force (kN)	Traction (kN)		Shear force (kN)		Moment (kNm)		Friction (kN)	Share friction (%)	Press mortar (kN)	Friction factor
1	410,98	48,94	41,75	102,10	100,70	0,001	0,006	2,73	1	20,19	0,14
20	270,66	50,59	46,81	61,37	60,28	0,668	0,660	13,68	10	26,90	0,51
30	227,17	50,89	43,33	49,81	51,40	0,744	0,729	12,38	11	23,72	0,52
35	210,42	51,83	41,81	44,66	48,37	0,757	0,740	12,18	12	23,14	0,53
40	196,45	52,98	42,56	41,05	44,76	0,780	0,764	12,42	13	25,04	0,50
50	181,29	56,10	44,49	36,57	40,74	0,793	0,787	13,34	15	30,09	0,44
60	168,16	60,22	50,75	32,59	37,39	0,803	0,801	14,10	17	40,47	0,35
70	162,55	68,79	63,79	30,20	35,54	0,810	0,807	15,54	19	62,08	0,25

Table 6-7 Internal forces of M20 8.8 K70 without friction under head plate

Force displacement diagram

From the analysis of the EEM model, a force-displacement diagram can be created from the load-steps taken (Figure 6-11). To its right, a force-displacement diagram from the tests of K. Mcbride [2] is shown. Both diagrams show the ratio of the actuating space to the diameter. The course of the diagram from the EEM model corresponds to that of the tests, and in it, too, the model shows a reliable picture.



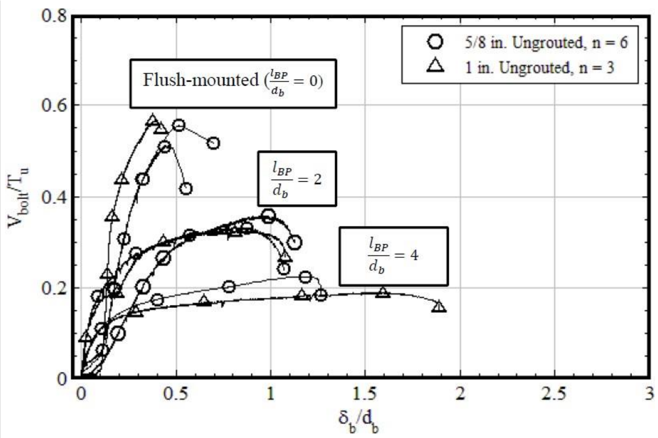


Figure 6-11 Force-displacement diagram of three setting spaces of M20 8.8 with K70 from EEM model and from Mcbride's tests.

7. Conclusions and findings EEM model

Using the results from the EEM model, as presented in Appendix 0, conclusions will be drawn in this chapter. To this end, Section 7.1 will compare the results with the findings and experimental results of the studies from Chapter 5. Section 7.2 will compare the found β of series I and III with the equations from Chapter 3. In Section 7.3, the deformation as shown by the EEM model will be considered in relation to the findings from Section 5.5. Finally, Section 7.4 will describe the remaining conclusions and findings.

7.1. Points of interest and validation of EEM model

Comparing all test results from Chapter 5 creates a "framework" in which the EEM model results should be located to be representative. The EEM model (EEM) shows the following:

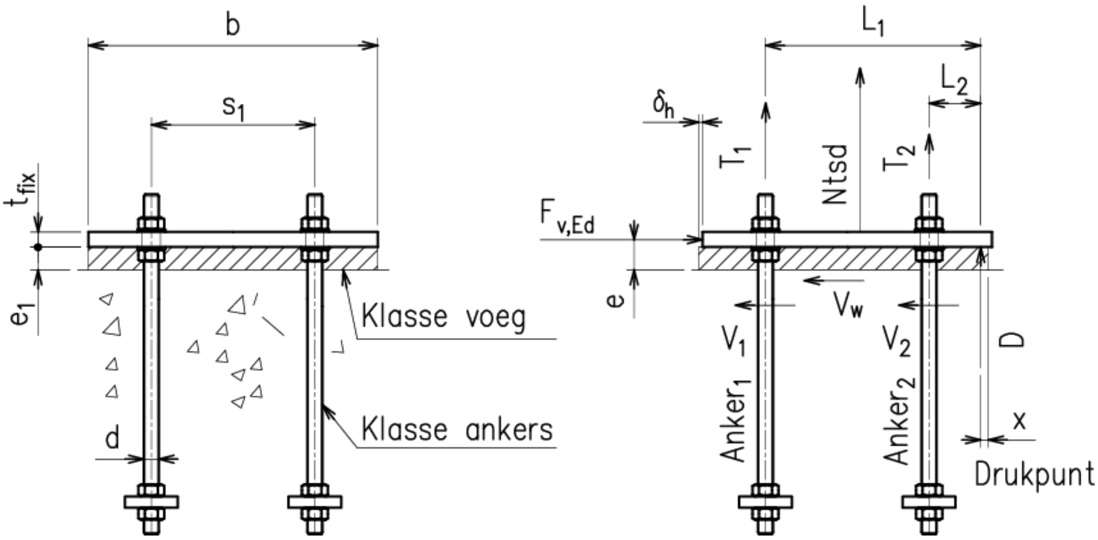


Figure 7-1 Parameters affecting the force action of a steel-concrete connection

1a) *The various studies have indicated that no bending will occur on the anchors. The trapping action of the mortar prevents this.*
EEM) This is not evident from the results of the EEM model. All analyses with set space show that there is a tensile force, a shear force and a moment in the anchors.

1b) *As the mortar joint increases in height (e1), the moment created by the shear force ($Fv_{,Ed}$) will increase. A larger moment creates a higher compressive force on the mortar and therefore, in accordance with equation (13), more frictional resistance (Fw) will also be present in the joint.*
EEM) The results clearly show that the compressive force on the concrete is increasing. Consequently, the proportion absorbed by friction also increases. Due to the frictional resistance, in series I beyond a setting space of 2*ds the shear capacity decreases only slightly. For the high friction resistances (series II), the shear capacity even increases from 3*ds, which is consistent with various test results. However, the maximum allowable shear stress of the material where the friction occurs must be sufficient for the occurring compressive force to also result in an increase in the shear capacity.

1c) *Can Mallée's assumption that the front anchors exhibit a bending behavior and the rear anchors only a sliding behavior be validated?*
EEM) For the two anchor rows, the difference of the internal moment is small. There is a larger difference in the maximum shear force absorbed by an anchor. In this, it is found that anchor2 absorbs more shear force than anchor1. This is because the proportion of tensile force in anchor1 is greater. The behavior described by Mallée in which anchor1 does not absorb bending and anchor2 barely absorbs shear is not shown in this EEM model.

2) *The friction results in an increased shear capacity and thus an increase in the overall joint resistance (Fw).*
EEM) The total shear capacity - as the headroom increases - is determined by friction for a significant proportion, 20% (see Appendix Section 5.3.2 Table 5-1). In the analyses with the increased friction (series II), this proportion increases to 30% (see Appendix Section 5.3.2 Table 5-2). In the M12 8.8 analysis with small pitch, the proportion for series I is even 30% and of series II 40% (see Appendix Section 5.3.4 Table 5-6 and Table 5-7).

3) *With a large pitch of the anchors (s1) and/or a larger base plate (b) in the direction of the shear force, a lower tensile force (T_1) occurs in the rear anchors and therefore a lower compressive force (D) on the mortar. The lower compressive force will create less friction (Fw) and reduce the shear capacity.*
EEM) By increasing the pitch of the anchors in the footplate, the compressive force generated from the eccentricity moment is proportionally reduced. The proportion of shear absorbed by friction decreases as a result. The total shear capacity - despite the smaller share of friction with a larger pitch - does not decrease but increases. This is different from what would be expected based on Mallée's test results [3]. See Section 7.4.2 for the detailed elaboration of these experimental results with respect to the EEM model.

4) *At about 2*ds, the lowest shear capacity of the complete joint will occur. This is an important point to determine β in a simple but responsible way. Today, the actuating height to which steel-concrete connections are often designed is 2*d (about 15% larger than 2*ds). In this way, the setting space does not become unnecessarily large, but large enough to be able to set with a nut.*
EEM) The analysis of the Series II EEM model shows that the lowest shear capacity is not necessarily at one specific point, but more on a plateau between 2*d and 3*d. These results are still consistent with the experimental results. However, no plateau appears in the experimental results because not as many different heights of setting spaces were tested. For series I, this point of lowest shear capacity is not present because this analysis included the computational coefficient of friction between the mortar joint and the pile. This coefficient of friction has a responsibly low value in the standards in force, but with very high probability, the friction proportion in the tests has never been in so low.

5) *The test results show that the shear capacity of Class 4.6 anchors could be reduced comparatively much less than Class 8.8 anchors.*
EEM) The analysis of the EEM model shows that the shear capacity reduction of class 4.6 anchors is smaller than that for class 8.8 anchors. This is consistent with the experimental results of Bouwman [1]. This seems to be caused by two effects:

- In a mandrel loaded for bending, the fibers at the extreme distance from the neutral line will reach maximum strain first. Due to the high allowable strain (25%) in Class 4.6 anchors, the extreme fiber maintains the maximum allowable stress, until failure, longer than in Class 8.8. Class 4.6 will therefore be better able to utilize a larger portion of the cross-section at the maximum stress.
- The proportion of friction is fairly similar in absolute terms for Class 4.6 and 8.8 anchors, but relatively for Class 4.6 it is a larger contribution to the total shear capacity.

7.2. β based on EEM model

Several choices were deliberately made in the EEM model that underestimate the real situation so that the shear capacity is not falsely overestimated:

- In the model, there is no adjusting nut under the footplate. As a result, the anchor does not clamp in as well and the anchors have a long bending length, see Section 5.3.2.
- In series I, there is no frictional resistance between mortar joint and base plate.
- The capacity of the anchor, after the maximum elongation occurs, drops abruptly to 0 N/mm^2 where in reality it is gradual.
- The preload is permanent as an external pull.

Despite these estimates, the found β of series I and IV at an adjustment space of 60 mm approaches the β of equation (8) (Figure 7-2 I) blue and IV) green, these fall exactly on top of each other for M20 8.8., for context, series II) yellow is also added). Series I may have too high a shear capacity to assume as safe because maximum strain occurs in this series, but series IV is at or below this maximum strain in various analyses with other parameters (see Appendix 0) and best approaches the β from equation (8).

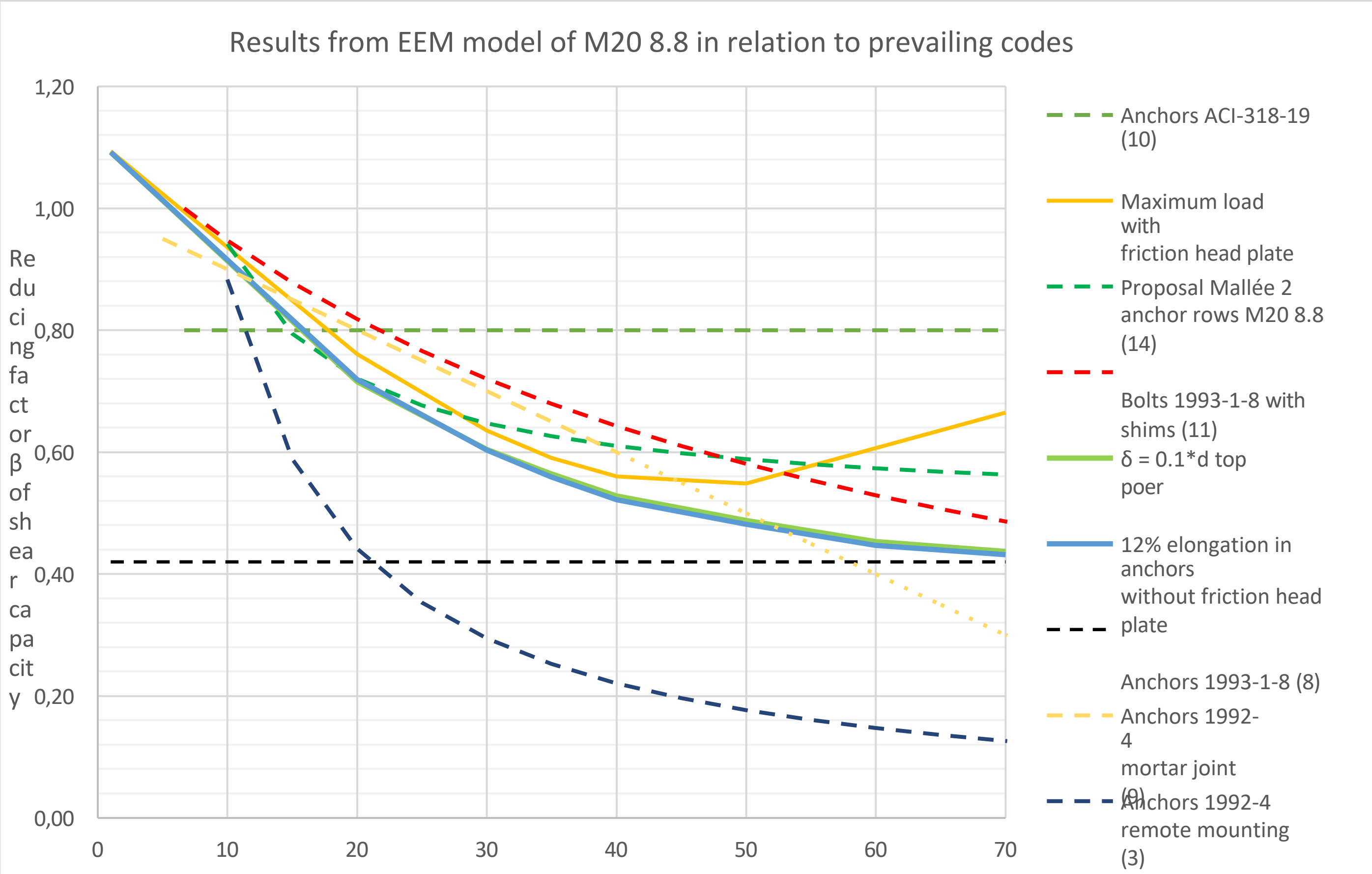


Figure 7-2 Results from EEM model of M20 8.8 in relation to the equations from Chapter 3

These analyses show (Figure 7-2) that the shear capacity of:

- equation (10) is far too favorable and, relative to the EEM model, overestimates the shear capacity;
- equation (14) describes the initial trajectory well, but then overestimates the shear capacity;
- equation (11) approximates the same trend as the EEM model, but slightly overestimates the shear capacity;
- equation (8) until the setting space of 60 mm underestimates the shear capacity and then approaches the found shear capacity;
- equation (9) describes the descending behavior linearly and therefore slightly overestimates the shear capacity between 0.5*d to 2*d and underestimates it from 3*d onward; and
- equation (3) is very conservative and underestimates shear capacity by a factor of 2 to 3 for all set spaces.

7.3. Distortion

Although this thesis research is not so much about deformation (see section 5.6.5), some issues regarding deformation from the EEM model will be explained. From the results relative to the test results, it can be concluded that the EEM model behaves too rigidly. This is because the shear capacity from the tests is proportionally closer to the calculated shear capacity than the deformation from the tests is to the calculated deformation. For the shear capacity the deviation is between a factor of 1.2 and 1.5 and for the deformation it is in between a factor of 1.6 and 2.0 of the results found (Figure 7-3). This means that drawing conclusions regarding the deformation can only be done with the caveat that the behavior of the EEM model is too stiff.

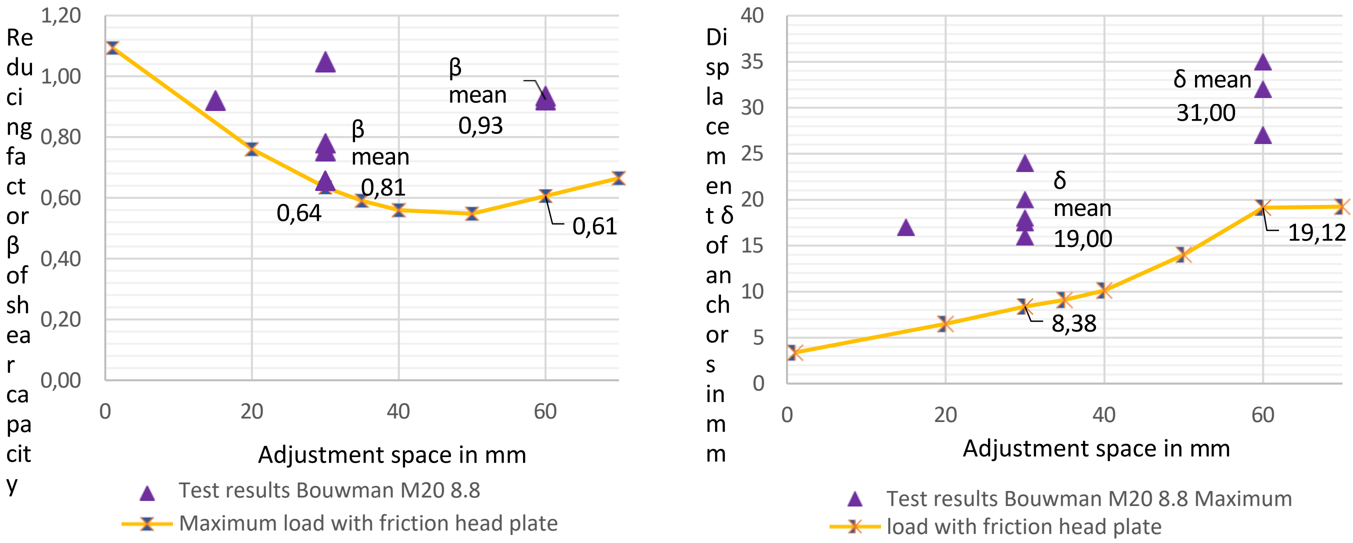


Figure 7-3 Deviation of test results from EEM model

That the EEM model differs in terms of shear capacity is partly due to the conservative assumptions made. But the fact that the EEM model behaves comparatively stiffer is largely due to the fact that the EEM model does not include hole clearance. The hole clearance lacks the physical displacement due to the clearance itself, and the hole clearance will also, to some extent, allow the shaft to twist, causing more deformation to occur (Section 5.6.4). Also, the model will behave more rigidly due to the choice of material behavior made at the maximum strain.

The results show that significant deformations already occur at 1 mm of headroom. For both series III and series IV, the maximum allowable shear capacity is already reached at 1 mm of headroom. The deformation can take place in the height of the adjustment space because there is physically no room for bending there. The large shear force in the anchor at a small adjusting space, must activate a larger concrete surface to transmit the shear force to the concrete than at larger adjusting spaces where the shear capacity proportionally decreases significantly. Due to the larger area required, the "fictitious" trapping occurs deeper than a 0.5*d (Figure 7-4). Due to the angular displacement, which causes the shear force, and a deeper point over which the anchor rotates, large deformation occurs. At higher concrete strength classes, the required butt plane will be smaller and the anchor will exhibit a stiffer behavior and deform less.

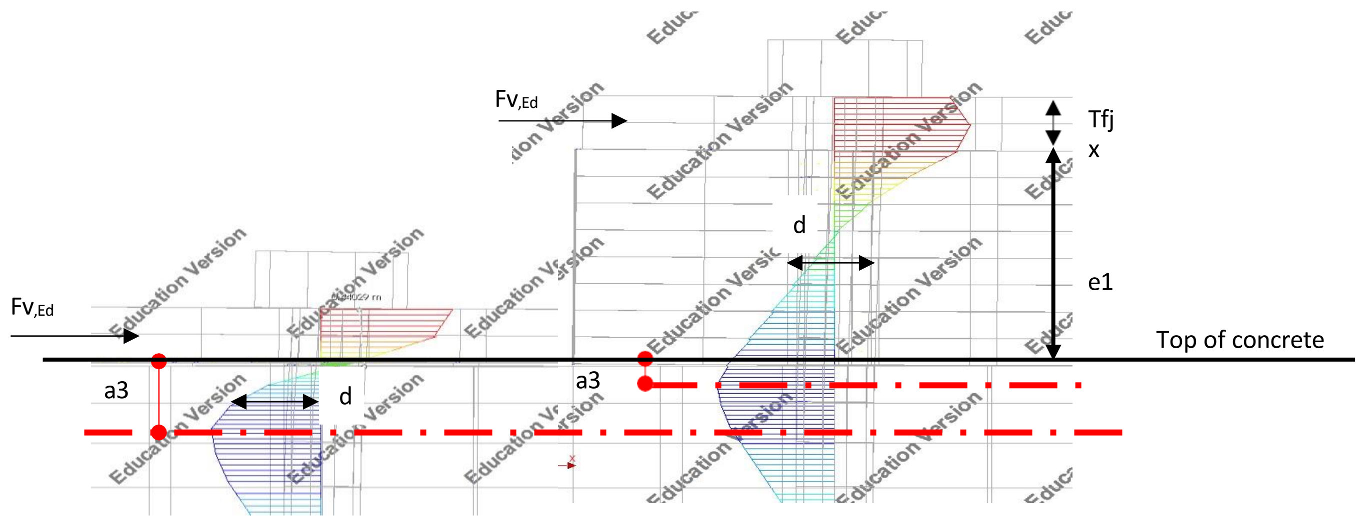


Figure 7-4 Representation of bending moment in an anchor due to a shear force $Fv_{,Ed}$. The point at which the internal moment starts to decrease is the depth of the "fictitious" clamping (a3) and the point at which the anchor will rotate left) 1 mm adjusting space (e1); right) 70 mm adjusting space (e1)

The EEM model results described two deformation-dependent sequences: III and IV. For both series the shear capacity belongs to the serviceability limit state (hereafter BGT) in which normally the deformation is tested. This is different for series I and II where the shear capacity found belongs to the ultimate limit state (hereafter UGT). Between the BGT and UGT there is a partial factor ($\gamma g_{,q}$). Its magnitude depends on the type of load and the degree of safety. Thus, in terms of shear capacity, series I and II are difficult to compare with III and IV. The shear capacity of series IV seems comparatively low. If series I is corrected to the BGT for, say, consequence class 1 where γg = 1.08 and γq = 1.35, the deformation, which occurs at the corrected shear force, can be represented from the model (Figure 7-5 and Table 7-1). For γq belonging to variable loads - for example, a shear force from a wind load on a column base - the maximum allowable shear capacity approaches the maximum proposed deformation. For γg belonging to permanent loads - for example, the dead weight of a floor on horizontally placed anchor - the occurring deformation remains high. For higher risk classes, γg and γq are higher and then the occurring deformation is smaller. With respect to deformation, it seems that at higher safety classes the occurring deformation need not be a boundary condition for the allowable shear force.

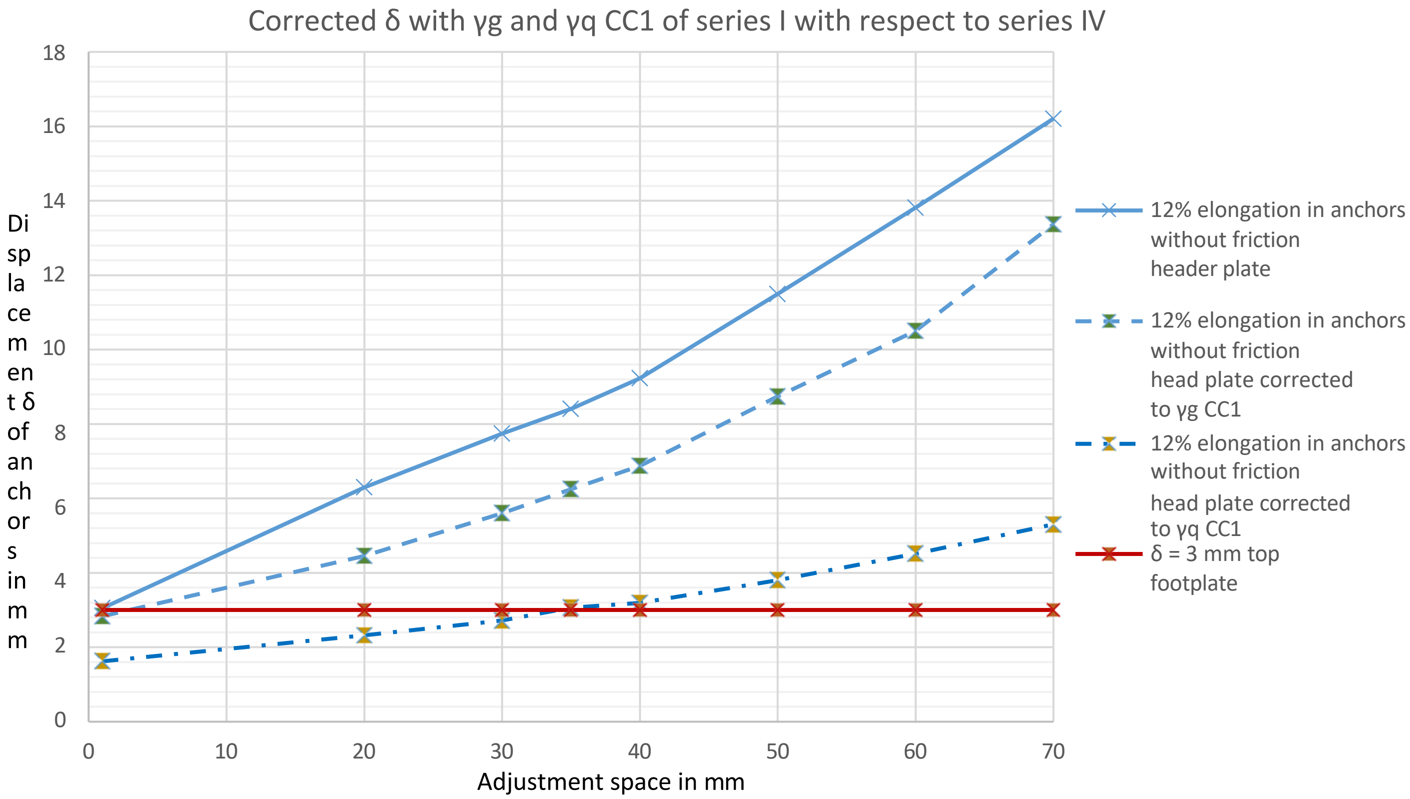


Figure 7-5 Corrected series I to BGT for M20 8.8

Adjustment space (mm)	Load factor	Loadstep	δ (mm)	Load factor γq = 1.35	Loadstep	δq (mm)	Load factor γg = 1.08	Loadstep	δg (mm)
1	4,110	16	3,05	3,044	8	1,62	3,805	13	2,83
20	2,707	34	6,30	2,005	12	2,32	2,506	22	4,45
30	2,272	41	7,74	1,683	14	2,72	2,103	28	5,60
35	2,104	45	8,40	1,559	16	3,06	1,948	32	6,25
40	1,965	50	9,23	1,455	17	3,19	1,819	36	6,87
50	1,813	64	11,50	1,343	21	3,80	1,679	48	8,74
60	1,682	77	13,82	1,246	25	4,51	1,557	58	10,50
70	1,626	89	16,21	1,204	29	5,30	1,505	73	13,37

Table 7-1 Corrected series I to BGT for M20 8.8

7.4. Other conclusions and findings

7.4.1. Other conclusions

- For all tests, at a headroom of 1 mm, β is greater than 1. The shear capacity is higher than the arithmetic shear capacity according to equation (2) with αv = 0.6. From the results of the EEM model, there is no reason to doubt this ratio number. Therefore, based on this model, it can be concluded that αv = 0.5 according to NEN-EN 1994-4 [5] is an underestimate of the actual shear capacity.

- The mortar class has a minor impact on the shear capacity when the setting space is small (>2*d). As the setting space increases and more compressive force is applied to the mortar, the impact of the mortar class is greater (Table 7-2 and Appendix Section 5.3.7. Figure 5-20, Figure 5-21, Figure 5-22). Therefore, the recommendation is not to apply class K30 mortar at a setting space greater than 3*d. This is because with a proportionally larger set space, the shear capacity of the anchors decreases too much.

Diameter	Setting space (mm)	β K70	β K30	Decrease
M12	15	0,74	0,73	1%
M12	25	0,6	0,58	3%
M12	40	0,47	0,42	12%
M20	20	0,72	0,7	3%
M20	40	0,52	0,5	4%
M20	60	0,45	0,42	7%
M30	60	0,57	0,55	4%
M30	80	0,48	0,45	7%

Table 7-2 Decrease in shear capacity at lower mortar class

- Up to an actuating margin of 1/3*d, the shear capacity of the anchors need not be reduced. In fact, it follows from the EEM model that up to this headroom the reduction is greater than or equal to 1. For 4.6 anchors, this is even true up to 3/4*d (Figure 7-6). For simplicity within the calculation rules, however, it is advisable to take one value for this. This value of 1/3*d corresponds to the boundary condition of equation (11) *the reduction of shear capacity of bolts with steel fills.*

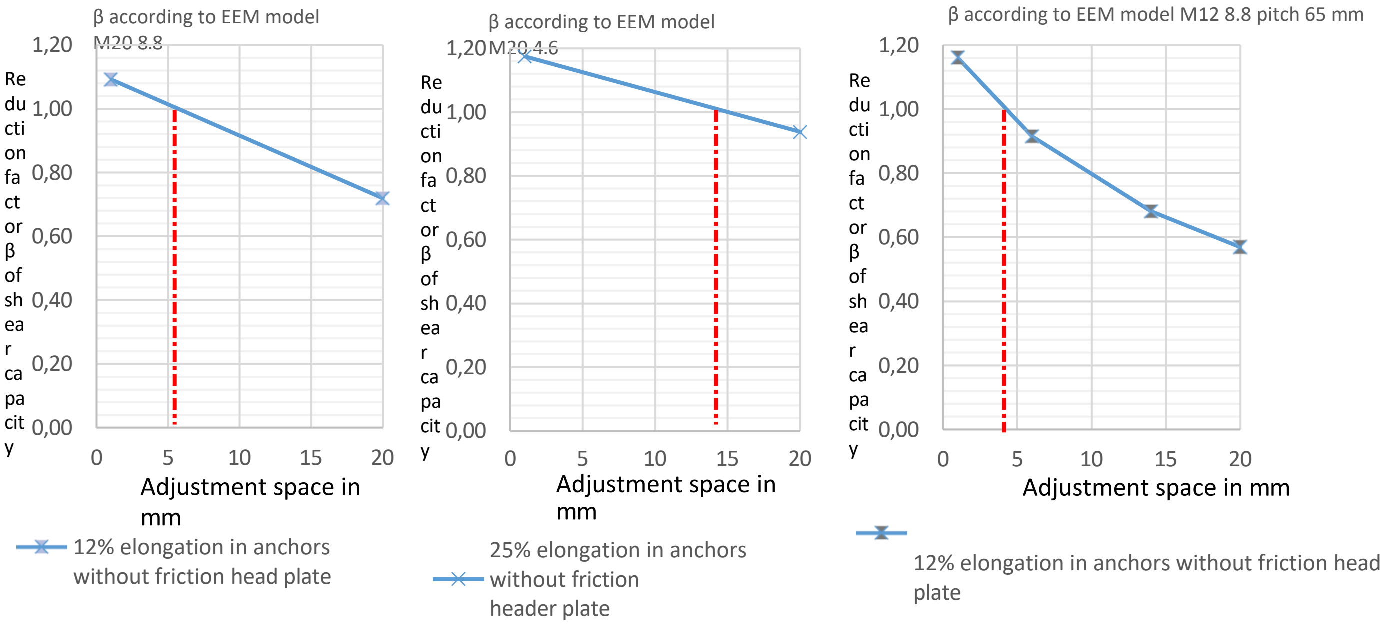


Figure 7-6 Height of set space where β equals 1

- For an actuating space greater than 3*d, without considering friction under the footplate, more than 20% of the shear capacity is provided by the frictional resistance (blue column Table 7-3). If friction under the footplate is included, the share of friction is at least 30% of the shear capacity (green column). There are no quantified requirements regarding the proportion of friction to the shear capacity. However, for the following reasons, it is prudent that friction share be limited:
    - An external tensile force on the anchors will reduce the compressive force from the eccentricity moment on the mortar and therefore the frictional resistance. The loss of shear capacity will be partly compensated by the fact that the external tensile force, through the 2de order effect due to δh, has a lowering effect on the internal moment of the anchors. But it cannot be quantified from the EEM model results how this relates to each other.
    - The quality of the casting mortar joint depends largely on the execution. For example: contamination of the joint, improper or complete application of the poured mortar, a pouring box that does not close properly, poured mortar that is not properly prepared, or the roughness of the surface of the concrete.

A proportion of up to 20% friction seems a responsible assumption. At that proportion, the anchors contribute twice as much to the total shear capacity (40% per anchor) than the friction. If frictional resistance is lower or not present at all, the anchors must provide 25% additional shear capacity (from 40% to 50% per anchor). With a share of friction of 30%, the anchors have a share of 35% per anchor. If friction is eliminated, the anchors should provide 40% additional shear capacity (from 35% to 50% per anchor). Considering the partial safety factor of the anchor $\gamma m2 = 1.25$ and in the fundamental occurring load a smallest partial safety factor $\gamma g = 1.08$, a potential loss of 40% is not justified.

Adjustment space (mm)	Setup space / d	4xM20 8.8 without friction footplate	4xM20 8.8 with friction footplate	4xM20 8.8 with increased friction footplate	4xM20 4.6 without friction footplate	4xM20 4.6 with friction footplate	Adjustment space (mm)	Setup space / d	M12 8.8 pitch 65 without friction footplate	M12 8.8 pitch 65 with friction footplate	M12 8.8 pitch 190 without friction footplate	M12 8.8 pitch 190 with friction footplate	Adjustment space (mm)	Setup space / d	M30 8.8 pitch 180 without friction footplate	M30 8.8 pitch 180 with friction footplate
1	0,1	1%	5%	12%	3%	3%	1	0,1	1%	8%	1%	10%	1	0,0	1%	4%
20	1,0	10%	10%	15%	7%	6%	6	0,5	8%	9%	6%	6%				
30	1,5	11%	10%	17%	11%	12%	14	1,2	10%	10%	10%	10%	40	1,3	7%	8%
35	1,8	12%	11%	20%	13%	17%	20	1,7	13%	13%	11%	14%				
40	2,0	13%	13%	22%	13%	22%	25	2,1	16%	18%	14%	20%	60	2,0	10%	14%
50	2,5	15%	16%	26%	16%	34%	30	2,5	20%	24%	15%	18%				
60	3,0	17%	21%	33%	19%	39%	35	2,9	25%	32%	19%	23%	80	2,7	15%	17%
70	3,5	19%	31%	35%	22%	41%	40	3,3	31%	34%	21%	29%	100	3,3	19%	25%

Table 7-3 Share of friction in total shear capacity joint

7.4.2. Relationship results M12 class 8.8 pitch 65 and 190 mm

The EEM model shows reliable results relative to the test results, except for the different pitch sizes of the tests of R. Mallée [3]. The results of the arrangement with M12 anchors and a pitch 65 mm or 190 mm compare differently in the EEM model than in the test results of Mallée [3]. In the results of [3], the shear capacity of the tests with a pitch of 65 mm is higher than that of the tests with a pitch of 190 mm. On the contrary, the EEM model shows that the shear capacity is higher for a pitch of 190 mm (Figure 7-7).

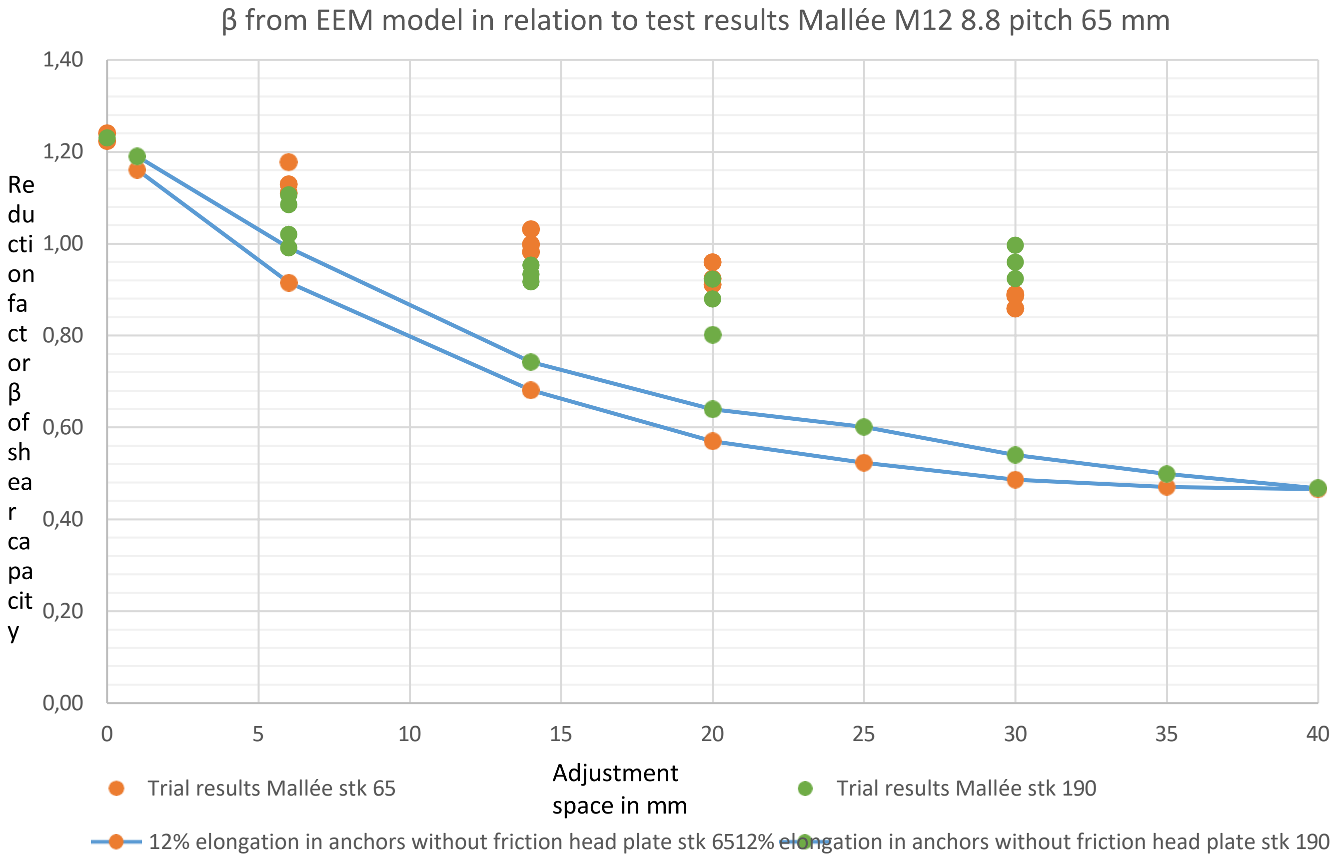


Figure 7-7 Test and analytical results of M12 8.8 anchors with a pitch of 65 and 190 mm

With the larger pitch, the arm (L1) between the anchor and the pressure point is larger. Consequently, the eccentricity moment causes a smaller pressure on the mortar. This is also evident from the results of the EEM model (Appendix 5.3.4 and 5.3.5). The proportion absorbed by friction (Fw) decreases as a result, but the tensile force acting in the anchor is also lower. This leaves more capacity available in the anchor to absorb the shear force. In the EEM model, with the larger pitch, the anchors can absorb more shear than decreases in friction.

There is a possibility that test results were inadvertently flipped. Indeed, when the test results are inverted, the deviation with the EEM model, except for the 30-mm tests, is almost the same (Table 7-4).

	Trial results Mallée		EEM model results.				
	Mean β		β		Testing/analysis ratio.		Difference
Adjustment space (mm)	Pitch 65 mm	pitch 190 mm	Pitch 65 mm	pitch 190 mm	Pitch 65 mm	pitch 190 mm	
0	1,23	1,23	1,16	1,19	6%	3%	3%
6	1,14	1,07	0,92	0,99	24%	8%	16%
14	1,00	0,93	0,68	0,74	47%	26%	21%
20	0,93	0,87	0,57	0,64	63%	36%	28%
30	0,88	0,96	0,49	0,54	81%	78%	3%
	Reversed trial results Mallée						
0	1,23	1,23	1,16	1,19	6%	4%	2%
6	1,07	1,14	0,92	0,99	17%	15%	2%
14	0,93	1,00	0,68	0,74	37%	35%	2%
20	0,87	0,93	0,57	0,64	52%	46%	7%
30	0,96	0,88	0,49	0,54	97%	63%	35%

Table 7-4 Mallée test results relative to EEM model results

8. Accountability

This chapter formulates the answer to the main question:
What reduction may or should be applied to the shear capacity of a steel anchor or bolt where the setting space has been filled out afterwards by mortar or steel plates?

Section 8.1 draws conclusions from the study. Following that, sections 8.2 and 8.3 answer the sub-questions and the main question. Subsequently, section 8.4 provides recommendations for further research and a possible extension of the proposal to better define the actual shear capacity.

8.1. Conclusions

The following conclusions can be drawn from the study:

1. With respect to the codes in force:

a. NEN-EN 1992-4 [5] equation (9) *shear capacity of anchors with mortar-filled headroom* rather simply describes the sloping behavior but thereby runs the risk - as with the proposal of R. Mallée [3] - of unjustifiably assigning too high a shear capacity if the headroom in practice turns out to be higher (see further explanation conclusion 4c). Also, the equation in its current form has many boundary conditions that this study does not show to be valid (see 2a, 2c, 2d, 3a, 3b, 4a and 5a);

b. NEN-EN 1993-1-8 [4] equation (8*) shear capacity of anchors with mortar-filled headroom* up to a headroom of 3*d underestimates the shear capacity. Due to the underestimation, the influence of deformation, oversized holes and tensile/ shear force interaction is less relevant. The intention of L. Bouwman et al [1] was to cover these factors in a responsibly simple manner in a reduction factor of the shear capacity. The outcome of this equation, at, a set space of 3*d, is consistent with the results of the EEM model; and

c. NEN-EN 1992-4 [5] equation (3*) bending resistance of anchors without mortar joint* significantly underestimates the shear capacity. Suppliers of post-applied anchors apply this equation within their software. The setting space, because the outcome is so conservative, is often ignored by users of this software. If the user does not subsequently test the anchors using equation (8) or (9), a significant overestimation by a factor of 2 subsequently occurs. Because the shear capacity in equation (3) is so underestimated, it should not be used for safety reasons (ignoring set space when testing), cost (more anchors and reinforcement required) and performance (problems with anchor group dimensions and placement).

2. With respect to the joint:

a. in [4] for equation (8) no maximum setting space is defined. For mortar joints loaded under pressure, the adjustment space should not exceed 0.2*the smallest length of the base plate. But that would allow for large dimensions of footplates to accommodate large joints. Within [5], equation (9) has the following conditions: *the mortar joint must be less than or equal to 40 mm and less than 5*d.* To make the mortar joint limit absolutely equal to 40 mm is a rigid requirement given the results of the tests and analysis. The results show that the diameter of the anchors is of in influence on the possible height of the setting gap. For M12 anchors, the described plateau of lowest shear capacity is located at a smaller setting space than for M30 anchors. The second requirement of a maximum allowable headroom of 5*d is too high. Combined with the first requirement, it is only applicable for M8 anchors or smaller, but if the 40 mm requirement does not apply, for M20 anchors this would result in an actuating clearance of 100 mm. Within the tests, no adjusting spaces larger than 3*d were tested. However, analyses for larger values were performed in the EEM model and showed that from 3*d the shear capacity of the anchors decreases and the friction share in the shear capacity increases. For implementation reasons, this is an undesirable situation. Therefore, in order to keep the reliability of the connection and the occurring deformation controllable, the maximum headroom for anchors loaded to shear force should not exceed 3*d.

In summary: for the headroom, the smallest value of 3*d or 0.2*smallest length of the footplate should be used. For adjusting spaces larger than 3*d, anchors should no longer be used to transfer a shear force and other solutions in the form of a reinforced joint or a shear cam will have to be sought;

b. applying multiple steel shims has a greater negative impact on shear capacity of bolts or anchors than if the joint were filled with a single steel plate or homogeneous casting mortar. This is evident from the study of steel shims [17]. Equation (11) *shear capacity of bolts when applying multiple steel shims* is more cleanly described in terms of formulation because herein the influence of the set space is a separate factor. This occurs in direct or indirect way in various formulas as well, but naming this factor as a separate β provides more insight into the behavior of the joint. Therefore, it is advisable to rewrite all existing equations related to shear in the form of equation (12);

c. the boundary condition associated with equation (9) that *the entire footplate be provided with a mortar joint* is a logical condition. Because friction has a share in the shear capacity, the entire footplate should be provided with a filled joint. With a mortar joint, the space between the post and the footplate can be properly filled even if the component to be connected is not parallel to the post. Should the gap between the steel base plate and the poer be filled with a single or multiple steel shims, it must be guaranteed that the two connecting surfaces are parallel to each other. If this cannot be guaranteed in the execution, the gap must be filled with casting mortar.

d. the boundary condition associated with equation (9) that *the compressive strength of the casting mortar should not be less than 30 N/mm^2* is a good condition. The survey conducted shows that K70 casting mortar with a compressive strength of 95 N/mm^2 is often used. Nevertheless, the various standards allow the use of a filler material up to a minimum quality of 30 N/mm^2 . The analysis of the EEM model shows a small loss of shear capacity at the lower casting mortar grade K30. However, as long as the setting space does not exceed 3*d, the loss of shear capacity at casting mortar class K30 is still acceptable.

3. With respect to anchors:

a. the boundary condition belonging to equation (9) that *at least two anchors should be placed consecutively in the direction of the shear force* is not necessary in view of the test results. The tests show almost the same reduction in shear capacity for anchors in a single row as with two rows. The requirement from the U.S. [21] that four anchors should be used for structural elements is a good proposal for execution reasons but with respect to shear capacity, it does not appear to be necessary and the precondition can be dropped;

b. the boundary condition belonging to equation (9) that *the pitch between anchors is at least 10*d is* an unnecessarily onerous requirement. From the results of the tests of L. Bouwman et al. (130 mm for M20) [1] and R. Mallée (65 mm for M12) [3] where the pitch between the anchors is significantly less than 10*d, no significant lower shear capacity in the anchors appears to be present compared to the other tests and equations. The influence of the pitch will undoubtedly have an effect for the breakout of the concrete, but this does not apply to the shear capacity of the steel anchor. This boundary condition should also be dropped;

c. the equations for determining the shear capacity are applicable for M12 through M36 anchors with classes 4.6 to 8.8. The tests show that for M12, M16, M20 and M30 anchors the shear capacities found have almost the same absolute reduction. The shear capacity reduction from the tests of one or two anchor rows is generally between 0.8 and 1 times the shear capacity. The EEM model also shows that the M12, M20 and M30 anchors exhibit similar behavior. The proposal is thus valid for M12 through M30 anchors and, given the stability of the results, it can be extended to M39 in accordance with NEN-EN 15048-1 [24];

d. the notation for determining the bending resistance of bolts and anchors can be described more simply in the form of equation (7). Describing the plastic bending resistance using the form factor and the elastic bending resistance is unclear and results in unnecessary rounding.

$$M_{Rk,s} = 0.9 * W_{pl} * f_{yb}$$

e. the type of anchor as worked out in Figure 1-3 does not affect the shear capacity of the anchor. Different types of anchors were used in the tests. In the study by L. Bouwman et al [1], hook anchors and anchors with follower plate were tested. In the study by K. Mcbride [2], anchors with follower plate were tested and in the study by R. Mallée [3], adhesive anchors were tested. As mentioned earlier, corresponding results follow from the test results. On this basis, there is no need to distinguish between anchor types for the shear capacity of an anchor; and

f. bolts threaded into sleeves (Figure 1-3 c) probably have the same shear capacity as other types of anchors. No experimental results were available to relate these to the other types of anchors. Also, it was not possible during the study to get in touch with suppliers of co-drilled sleeves to clarify this through, for example, an interview. In the calculation software of one supplier (Halfen), equation (3) from [5] is applied. Within [5], no distinction is made according to anchor type for the determination of shear capacity. From this it was concluded for this study that the same method can be used for determining the shear capacity of bolts in sleeves.

4. With respect to the β sought, it holds that

a. The shear capacity of the anchors only needs to be reduced from an actuating margin of 1/3*d or more. According to the test results and the results from the EEM model, β is greater than 1 for an expansion gap smaller than 1/3*d. The boundary condition belonging to equation (9) that *the shear capacity needs to be reduced from an expansion gap > 0.5*d* is too optimistic in view of the results from the EEM model. To create equality and simplicity in determining the shear capacity for bolts and anchors, the value from the boundary condition of equation (11) has been adopted, being 1/3*d;

b. a value of 0.8 ACI-318-19 [8] is too low given the test results and the results from the EEM model. A very large proportion of the results have failure at a lower shear capacity than is allowed with this reduction. There are factors that can have a positive influence on the shear capacity (Section 8.4.2), but the responsible lower limit sought cannot include these factors; and

c. the proposal of R. Mallée [3] gives a good limit of testing, but a non-constant β is only justified if the joint height is already known at design time and cannot become higher than assumed in the design. However, an important fact of headroom is precisely that it varies and can deviate significantly from a design due to execution tolerances. Thus, the height of the adjusting spaces may end up at too high a point, which puts Mallée's proposal at risk (Figure 8-1 red). In general, a decreasing β is therefore not desirable and also underestimates the influence of a higher level of adjusting space. The tests and the EEM model show that the shear capacity does not decrease or decreases less from 2*d. This is present in R. Mallée's proposal, but not in comparison
(9) and (11), unnecessarily underestimating the shear capacity at higher actuating spaces. On the contrary, a constant β underestimates lower actuating spaces (Figure 8-1 black), but this is a responsible value with respect to an unknown end height of the actuating space.

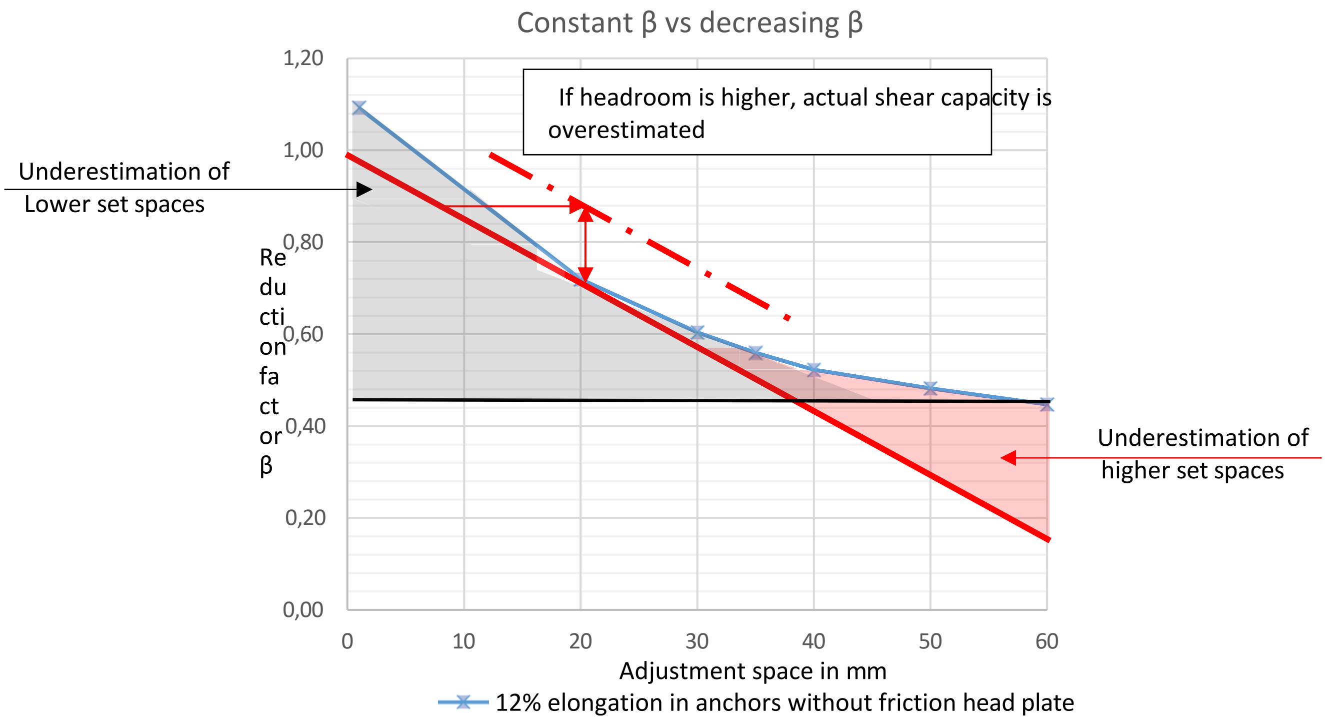


Figure 8-1 Constant β relative to decreasing β

5. With respect to the occurring loads and capacity:

a. In view of the test results of Bouwman [1] and Mcbride [2], the precondition belonging to equation (9) that *no tensile force or moment should act on the footplate* is not applicable. The shear capacity of anchors does not appear to be lower from the test results with additional additional loads. This was not investigated in the prepared EEM model, but given the slightly lower shear capacity at 1 mm of headroom and the internally occurring tensile force, the anchors will be sensitive to the interaction with a tensile force. This is also natural in relation to the Von-Misses criterion. If the allowable shear capacity is increased from the current equations, the influence of the interaction will have to be taken into account.

8.2. Subquestions

Is the behavior of a bolt inserted into a sleeve similar to that of an anchor that is cast in or inserted after completion? Both standards, [4] and [5], do not distinguish between anchors protruding me a out of the concrete or sleeves into which a threaded rod or bolt is inserted. No other calculation method is used in the calculation software of a sleeve supplier. No literature was found where this particular configuration is described and also contact with a supplier of pre-cast sleeves proved very difficult. Given that the standards and the calculation software make no distinction in the type of anchor and the steel-steel and steel-concrete tests show that the mechanical behavior that occurs is the same, it seems justified to assume that the same behavior applies to sleeves.

To what bolt and anchor diameters are the current equations applicable?
The determination of the shear capacity is applicable to anchors in class 4.6 to 8.8 with a diameter of M12 to M39 in accordance with NEN-EN 15048-1 [24]. Both the tests and the EEM model show that for M12 to M30 anchors the shear capacities found have almost the same absolute reduction and show similar behavior. Due to the reliability of the results, there is no reason to doubt that this would not be applicable up to M39.

Is NEN-EN 1993-1-8 equation 6.2 convertible to an equation similar to NEN-EN 1993-1-8 equation 3.3, namely an equation in which the diameter of the anchor and the size of the setting space affect the reduction?
Converting to the same notation is not applicable for anchors with a homogeneous set joint. The test results and the EEM model show that an anchor has the lowest shear capacity at an adjusting gap around 2*d to 3*d. This plateau is already present in the analyses where only computational friction is present between the mortar joint and pile. If there is increased friction between the concrete and also friction between the base plate and the mortar joint, the shear capacity already increases at 2.5*d. Therefore, for anchors and bolts with a homogeneous joint, β can be determined based on the lowest shear capacity. In Equation 3.3, β decreases as the set space increases. This is also the case when applying multiple steel fills, due to the shifting of the fills among themselves. Therefore, this notation cannot be adopted.

Can a universal equation for the reduction of shear capacity for steel-steel and steel-concrete joints with an incompressible filler be described?
If the joint is performed homogeneously (poured joint or a single steel plate), the test results of steel-steel and steel-concrete tests show the same mechanical behavior. Due to the tensile/pressure torque created by the shear force at a joint and frictional resistance created in the process, the bolt or anchor at 2*d to 2.5*d has the lowest shear capacity. Therefore, a universal equation can be described for steel-steel and steel-concrete.

How should misalignment between steel-concrete be handled? In case of misalignment, a different shape of the adjustment space occurs, resulting in a smaller adjustment space on one side than on the other side.
The spacing affects the shear capacity of the anchors. When the gap is filled, the moment created by the shear force is absorbed partly by the anchors in the form of bending and partly by the mortar joint in the form of a compressive force on the mortar and a tensile force on the anchors. The compressive force on the mortar creates additional shear capacity in the form of friction. Because the friction has a share in the shear capacity, it is important that the entire base plate have a filled joint. With a mortar joint, even if the component to be connected is not parallel to the paving slab, the space between the paving slab and the footing slab can be properly filled. Should the gap be filled with a single or multiple steel shims, the two connecting surfaces must be parallel to each other. If this cannot be guaranteed in the execution, the gap should be filled with casting mortar.

Due to skew, the bending length of the anchors is not equal among themselves. The anchors with a short bending length will behave more rigidly and therefore absorb a larger share of the shear force. This could have a lowering effect on the total shear capacity of the joint given that the stiffer anchors may fail before the "longer" anchors have reached their full shear capacity. Whether this effect occurs and to what extent needs further investigation.

What is the shear capacity of (cast-in) anchors at an adjustment space greater than 60 mm?
The shear capacity of anchors at 60 mm of actuating space is found to depend on the diameter of the anchor. The previously described plateau of lowest shear capacity is present at all diameters but depends on the ratio of the actuating space to the diameter. The shear capacity according to Equation 6.2 can be maintained provided that an additional boundary condition is included that the maximum actuating space should not exceed 3*d. With a larger headroom than 3*d, more than 20% of the shear capacity is provided by the frictional resistance. Due to a variety of implementation-related issues, the friction may be low. Therefore, it is irresponsible to let the shear capacity depend to a large extent on the friction. Also, at a set space greater than 3*d, the deformation of the anchors is significant and the lower quality casting mortar will no longer be strong enough.

8.3. Main question and proposal

To create simplicity and do justice to the test results and daily practice, this section describes the lower limit of shear capacity of anchors with a filled joint. This answers the main question: **What reduction may or should be applied to the shear capacity of a steel anchor or bolt where the setting space is filled out afterwards by mortar or steel plates?**

There is no evidence that the proposal of L. Bouwman et al [1] does injustice to the shear capacity of anchors. In fact, in all follow-up experiments [2] and [3], their proposal can rather be seen as somewhat too conservative. The present equations (8) according to NEN-EN 1993-1-8 [4] is a lower limit that captures the following effects: maximum allowable deformation, oversized holes and the tensile/ shear force interaction. This equation broken down into the proposed factors αv and β results in one unified equation (15) for the shear capacity of bolts and anchors and would thus eliminate equations (2), (8), (9) and. For completeness of the proposal below, the requirement regarding αv of the deviating bolt classes, which were not investigated in this study, is taken from NEN-EN 1993-1-8 Table 3.4 [4].

$$F_{v,Rd} = \beta * \frac{\alpha v * As * fub}{\gamma_{m2}} \qquad (15)$$

With respect to αv,:

- when the shear plane passes through the thread of the bolt (A is then the tensile stress cross section A_x of the bolt):
  - For bolt classes 4.6, 5.6 and 8.8: αv = 0.6;
  - for bolt classes 4.8, 5.8, 6.8 and 10.9: αv = 0.5; and
- when the shear plane passes through the shaft (without thread) of the bolt (A is then the gross cross-sectional area of the bolt):
  - αv = 0.6

With respect to β,:

- from a joint height greater than 1/3*d the shear capacity should be reduced;
- the mortar joint height may not exceed 3*d or 0.2*the smallest width of the base plate in accordance with NEN-EN 1992-1-1; and
- steel shims in steel-concrete connections may only be used if the connecting surfaces are parallel to each other and after installation there is no open position on any side. A maximum of three plates applies here.

If these boundary conditions are met, the following equations apply for the β-factor:

- For a filled gap by mortar joint or single steel shim

$$\beta = 0.745 - 0.0005 * f_{yb}$$

- for multiple steel shims

$$\beta = \frac{9d}{8d * 3tp}$$

8.4. Recommendations

8.4.1. In relation to the proposal

1. The EEM model did not investigate the interaction of the shear force with an external tensile force and/or moment. In the study by L. Bouwman et al [1], the influence of the tensile force was incorporated into the maximum allowable shear force. The EEM model, despite several conservative assumptions relative to the computational rules that follow from the study of [1], gives corresponding results at higher set spaces. In the EEM model, however, the complex behavior of all occurring loads was checked. For a broader validation of the proposal, it is advisable to investigate the influence of an external tensile force and/or moment and assess whether the interaction is important for the maximum allowable shear force. Based on the studies [1], [2] and [3], the following behavior can be expected (see Figure 7-1 for the indices):
    a. With increasing external tensile force ($Nt_{,sd}$) on the footplate, the tensile force on the rear anchors ($_{T1}$) will increase slightly, but the compressive force (D) on the mortar will decrease significantly. For this, the applied tensile force that serves as a prestressing force will need to be modeled in the EEM model such that it responds as an actual prestress in a spring system and not as an external permanent tensile force as in the EEM model of this study.
    b. The shear capacity of the anchors will change (decrease or increase) when the external tensile force ($Nt_{,sd}$) is so large that no compressive force (D>0) is present on the mortar. The degree of shear capacity decline depends on the deformation (δh) due to the shear force. The external tensile force ($Nt_{,sd}$) will develop an opposing torque ($Nt_{,sd*\delta h}$) due to that deformation that will reduce the eccentricity moment of the shear force. Therefore, the internal bending stress of the anchors will be lower, increasing the shear capacity of the anchors.
2. In addition, other issues in the EEM model have been underexplored. For validation of the proposed boundary conditions of equation (15)(12), the following items could still be added in an EEM model:
    a. *steel filler(s)* - This allows verification that in steel-concrete joints, the behavior of a single or multiple steel fillers exhibit the same behavior as in steel-steel tests; and
    b. *misalignment in the base plate* - Due to misalignment, the adjusting space of the anchors is not equal among themselves. Anchors with a short adjusting space will behave more rigidly and therefore absorb a larger share of the shear force. This could have a lowering effect on the shear capacity of the entire connection given that the stiffer anchors may fail before the "longer" anchors have reached their full shear capacity.
3. The proposal does not consider maximum allowable deformation. Within the current codes [4] and [5] there are no requirements for this. Nevertheless, this study agrees with the recommendations of several other studies (see also Section 5.5). Limiting the maximum allowable deformation related to, for example, the function of the structure can be a good first step in this direction.

8.4.2. Possible upper limit

Equation (15) can be expanded to include other factors that affect shear capacity, see equation (16). These are assumptions based on various results, findings and conclusions from the literature and this research without also having been considered in an EEM model. It is therefore advisable to investigate the following three parameters:

$$F_{v,Rd} = \alpha n * \alpha h * \beta \alpha i * \frac{\alpha v * As * fub}{\gamma m2} \qquad (16)$$

αn = influence factor of adjusting nuts

- Without adjusting nutαn = 1
  With adjusting nut αn= 1.2 (1*d short anchor length for bending) With clamping nut and adjusting nutαn = 1.5 (2*d short anchor length for bending)

αh = influence factor for hole clearance

- Normal holesαh = 1
  Oversize holesαh = 0.8 (as per butt reduction for oversize holes)

αi = influence factor of the number of anchor rows

- 1 or 2 anchor rows αi= 1 3
  anchor rows or more αi= 2/i
  Wherein i = number of anchor rows

αn = Influence factor of adjusting nuts

The bending length would decrease according to R. Mallée [3] by applying casting mortar and according to A. den Deurwaarder [18] by applying an adjusting nut (Figure 8-2 a and Section 5.3.2). From the deformation behavior of the EEM model, the pressure point still appears to be at a certain depth (a3) in the pile with which the proposal of [3] does not appear to be valid. The proposal of [18] cannot be validated with the EEM model because this adjusting nut is missing. If this suggestion is correct it has an increasing effect on the shear capacity. In NEN-EN 1992-4 [5], a shortened bending length may also be used when applying a clamping nut (Figure 8-2 b and Section 3.1). This combined results in the bending length decreasing by 0.5*tfix + 1*d for the adjusting nut and by 0.5*d for the clamping nut (Figure 8-2 c). But from the suggestion of [18], it can be stated that at the clamping nut, the bending length decreases by 0.5*d + 1*d (Figure 8-2 d).

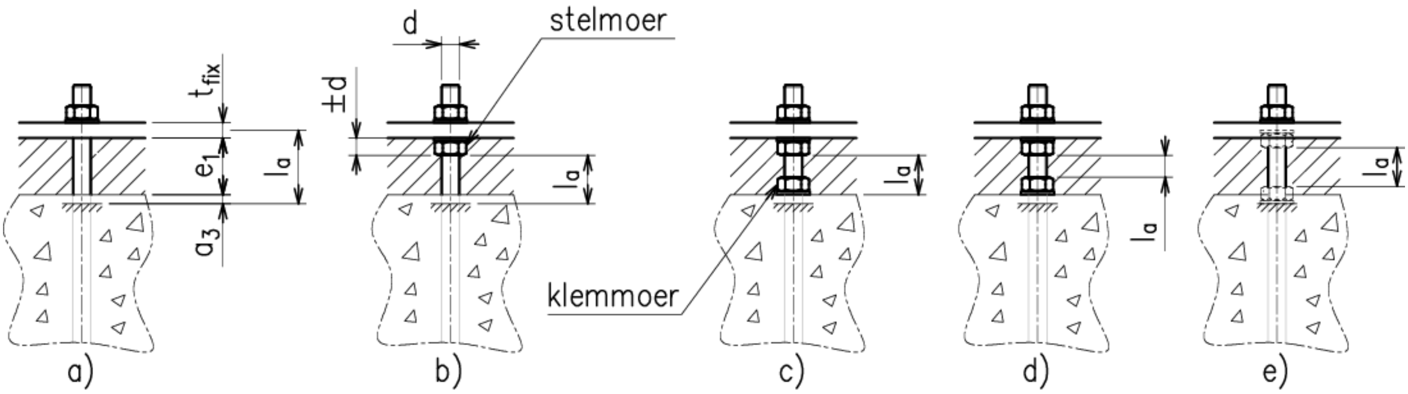
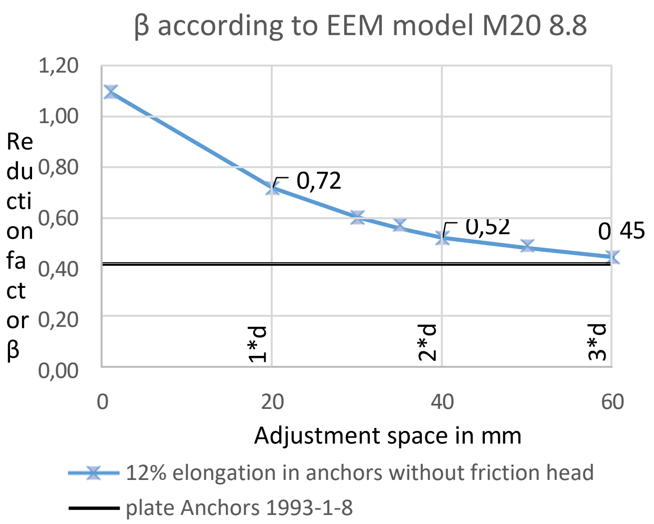


Figure 8-2 Recommendation for examination of computational headroom for filled headroom
a) mathematical setting space according to NEN-EN 1992-4 without setting and clamping nut
b) arithmetic adjusting space according to A. den Deurwaarder with adjusting nut
c) possible mathematical margin of adjustment by combination of suggestion A. den Door value and clamping nut according to NEN-EN 1992-4
d) possible arithmetic adjusting space based on suggestion A. den Door value on adjusting and clamping nut
e) computational leeway in the recommendation

In order not to overestimate the reduction of the nuts, the recommendation is to reduce the bending length per nut/ring combination by only 1*d (Figure 8-2 e). Simply reasoned, the shear capacity of the EEM model associated with the corrected setting space, in which the bending length is equal to Figure 8-2 a, then applies. When one adjusting nut or clamping nut is applied, the bending length decreases by 1*d and the increase in shear capacity is 20% (0.50/0.60). When both nuts are applied, the overall bending length decreases by 2*d and the increase in shear capacity is 50% (0.50/0.74) (Figure 8-3). Applying the reduction to the results of the investigated EEM model does not unnecessarily overestimate the shear capacity.



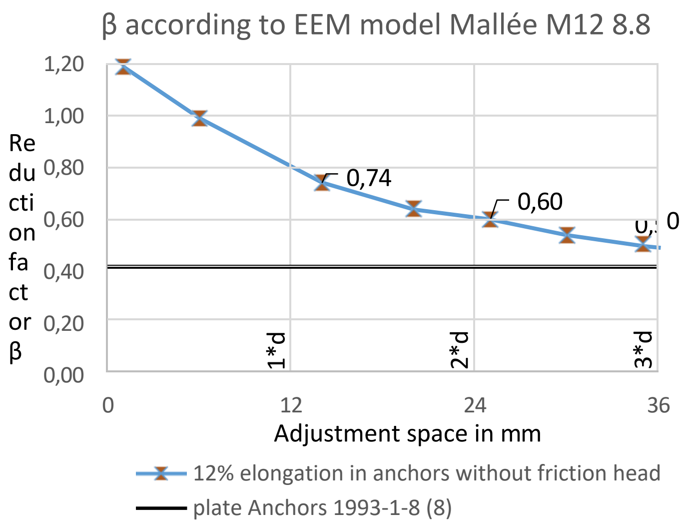


Figure 8-3 Difference in shear capacity of 8.8 anchors for different setting spaces

αh = Influence factor for the hole clearance

The current proposal and current codes do not include a correction factor for the impact of oversized holes. Nevertheless, it is a fact that oversized holes are often used in practice. The experimental results of K. Mcbride [2] and the research of P. Dusicka et al. [17] show that the shear capacity of anchors or bolts with oversized holes is not lower. However, the deformation does increase significantly. If more extensive examination of the deformation shows that it is not acceptable, a correction factor can be added that will lower the allowable shear capacity and thus also lower the deformation occurring. At this stage, the correction factor for oversized holes according to NEN-EN 1993-1-8 [4, p. Table 3.4 paragraph 1] is adopted as a first step, but this is an arbitrary value.

αi = Influence factor of the number of anchor rows

In the proposal of R. Mallée [3], equation (14), the average anchor capacity increases, if multiple rows are applied. This assumption seems to correspond to the test results of Mcbride [2] in which the shear capacity per anchor, in the tests with a higher number of anchors than four, is higher. If in equation (14) the anchor row, which is loaded on bending only, is excluded from the shear capacity, the equation can easily be described as β = (i - 1)/i, where i equals the number of anchor rows. With a high headroom, the proportion of the shear capacity of the anchor row loaded on bending is small relative to the full shear capacity (see equation (3) in Figure 3-5). The β associated with two anchor rows thus becomes equal to (2 - 1)/2 = 0.5. This is a higher β than shown by the analysis of the EEM model and equation (15). Herein, for class 8.8, β = 0.438 and for class 4.6, β = 0.625. If this absolute difference of β is corrected with Mallée's proposal, the corrected β per anchor row follows (see Table 8-1). A factor of 2/i can be included for the influence of the number of anchor rows. This deviates proportionally more for class 4.6 than for class 8.8, but for both classes this factor is an underestimation with respect to the proposal [3].

		corrected β			β^αi		deviation of β^αi by corrected β	
i	(i-1)/i	4.6	8.8	αi = 2/i	4.6	8.8	4.6	8.8
2	0,50	0,625	0,428	1,00	0,625	0,428	0%	0%
3	0,67	0,79	0,59	0,67	0,73	0,57	-8%	-4%
4	0,75	0,88	0,68	0,50	0,79	0,65	-10%	-3%
5	0,83	0,96	0,76	0,40	0,83	0,71	-14%	-6%
6	0,86	0,98	0,78	0,33	0,85	0,75	-13%	-4%
7	0,88	1,00	0,80	0,29	0,87	0,78	-13%	-2%
8	0,89	1,01	0,82	0,25	0,89	0,81	-12%	-1%

i = number of anchor rows

Table 8-1 Potential magnification factor of β for the number of anchor rows

Validation of upper limit

To endorse the above findings from the literature review and experimental results, an EEM model should be created in which the following should be added:

- *adjusting nut and clamping nut*, thus the impact on the bending length can be visualized and what shear capacity belongs to the lower bending length;
- *hole clearance*, thus the deformation will increase significantly. By analyzing the EEM model with and without oversized holes, a correction factor can be determined based on these deformations; and
- *multiple anchor rows,* thus the increasing effect on shear capacity could be validated.

Reflection

When I started my studies, this was already one of the possible subjects on which I wanted to graduate. Knowing this so far in advance gave me peace of mind on the one hand, and on the other hand I may have expected too much of it. When, after submitting the proposal, it was initially rejected on the grounds that it would be too complex and too little information would be known about it and that it was not integral enough in relation to the program, the motivation for graduation disappeared for a while. Nevertheless, after a good talk, graduation was allowed to begin. But then: were the warnings justified, is it too much, is it too complex, did I make too much hay, when are you stubborn, when do you listen?

After the approval interview with my thesis supervisor who said I should go for it, I started the literature review with fresh courage. More has been written on the subject than was known to me beforehand, and without the help of various contacts, I would not have found several sources either. But the more I read the more elusive the subject became and the more questions arose. A constant feeling between hope and fear. I read things that convinced me it was a good idea to start looking into the subject, but I also found out more and more that it is too complex to just draw new conclusions. Also, getting in touch with suppliers of collapsed shells and NEN committees, for example, proved impractical. How is it that if there are ambiguities about a subject in daily practice that parties who can play a role in it do not pick up on them? Why was seemingly nothing done with past research, am I naive if I think you want to move the profession forward together?

Stopping the literature review at the right time and starting to write was something I had to accept. Had I read enough, what if I missed an important source? On the advice of René Braam, I started writing early and I am grateful to him for that. In my own planning this was only at the end of the research and then it would not have been completed on time and in this form. Also, good writing takes a lot of time and in writing this piece I learned a lot, all the more through the editing work of Petra Barkema and René Braam's pointers.

I also thought a little too easily about the time frame in which I could make a good EEM model. Creating such a model needs time to mature and grow on you so that the model is also correct. Correct as in suitable for the study. A model soon seems finished, but if the results are not good or clean, it needs to keep being scraped. Until even a week and a half before the deadline for the draft report and I found out that a friction parameter had been entered incorrectly. But we just consider that part of the entertainment. Without Diana's support I would not have been able to get the model so reliable.

Graduation involved choosing between what my job is as a graduate student and trying to solve a problem for a field. Apparently you can quickly become less naive yourself. Still, the will and ambition to contribute something is also a good motivator, but listening to people at the right time who say something is too much or not the right path is also good. So, on the one hand, not agreeing to reject a proposal, on the other hand, not being stubborn and knowing when other people can assess something better. Am I now a master of shear-loaded anchors? I don't think so, but no doubt I will know more than anyone else. I now dare to make statements about a safe framework.

How to proceed now? No idea. I still consider myself a simple structural designer doing a poor job for what it's worth. In the work field I meet people I look up to for their (sometimes apparent) knowledge and am regularly surprised at the (great) ignorance of others. Some time later, several people I looked up to are disappointed in their skills and I find new people to look up to. There is so much I do not understand, do not comprehend, and with regularity I think I must learn more, study more and try harder, and that struggle will probably last the rest of my life.

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Symbols

a3 -distance between surface and indentation of an anchor in case of remote mounting d
-nominal diameter of bolt or anchor
ds -calculated diameter of bolt or anchor e1 -physical
headroom
fub -tensile strength of bolt or anchor fyb
-fluid limit of bolt or anchor k6
-see αv
la - arithmetic headroom
tfix -thickness of connecting plate
tmortar -height of physical mortar layer
tp -thickness of the package of steel fillings

Axis -voltage cross section of bolt or anchor Av -
shear surface
D -internal compressive force acting in a steel-concrete joint that follows from $Fv_{,Ed*e}$ $Fv_{,k}$ -
characteristic value of the shear capacity
$Fv_{,Ed}$ -calculated value of the shear force for the ultimate limit state $Fv_{,Rd}$ -
calculated value of the shear capacity of a bolt or anchor
$Fv_{,Rd,M}$ -calculated value of the shear capacity of a bolt or anchor due to a spacer Li -length of lever
arm from anchori to pressure point D
MRk,s -characteristic resistance to bending of bolt or anchors
MEd -calculated value of the bending moment for the ultimate limit state NEd
-calculated value of the normal force for the ultimate limit state $Nt_{,Ed}$ -
calculated value of the tensile normal force for the ultimate limit state
Ti -internal tensile force in an anchor that occurs in a steel-concrete connection that follows from $Fv_{,Ed*e}$ Vi
- proportion of the shear capacity in a steel-concrete joint absorbed by an anchor VRd,s -see
$Fv_{,Rd}$
Vw -proportion of shear capacity in a steel-concrete joint absorbed by friction Wel -elastic resistance
to bending
Wpl -plastic resistance to bending

α - ratio of plastic and elastic resistance to bending of cross sections αbc -
reduction factor for the shear capacity of anchors according to NEN-EN 1993-1-8 αM -
number of indentations of an anchor in case of distance mounting
αv -ratio between the shear and tensile capacity of steel bolts and anchors
β -reduction factor for the shear capacity of anchors at set space
βp -reduction factor for shear capacity of bolts with steel shims
δ - displacement
δi -displacement in a designated axis direction
μ -coefficient of friction following from tan (angle of friction) of the material.
τ - shear stress
γq -partial safety factor for permanent loads
γm2 -partial safety factor for bolts and anchors with the value of 1.25
γMs - partial safety factor for bolts and anchors with the value of 1.25 if: 1.0 · fub/fyb ≥
1.25 if fu ≤ 800 N/mm^2 and fyb/fub ≤ 0.8
γq -partial safety factor for variable loads

BV/BmS Opleidingen samenwerking Betonvereniging en Bouwen met Staal

GRADUATE RESEARCH

SHEAR CAPACITY OF ANCHORS WITH MORTAR-FILLED ADJUSTMENT SPACE

Student/No.: Arco de Gelder - 17803

Attachments

1. Survey Results

Figure 1-1 Distribution of survey participants across the Netherlands and Belgium

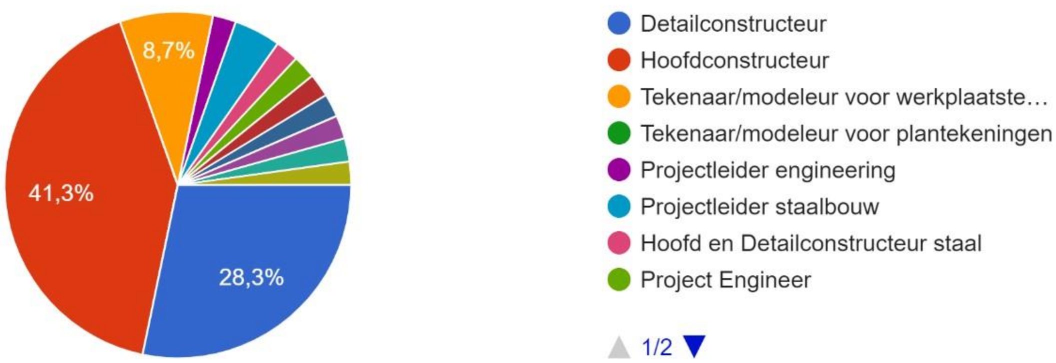
Responses to the 2de tab are not shown. This is an answer from a single respondent and almost always a variant of the other answers with a nuance that was important to the respondent.

Bij wat voor type bedrijf werkt u.

46 antwoorden

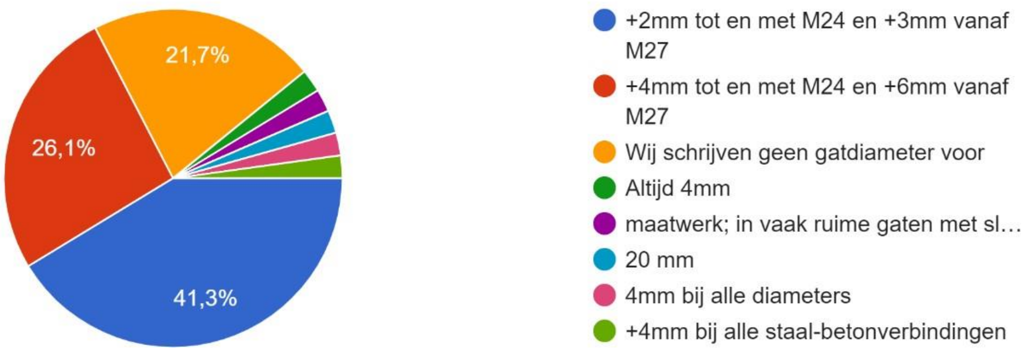
Welke functiebeschrijving past het meest bij uw werkzaamheden.

46 antwoorden



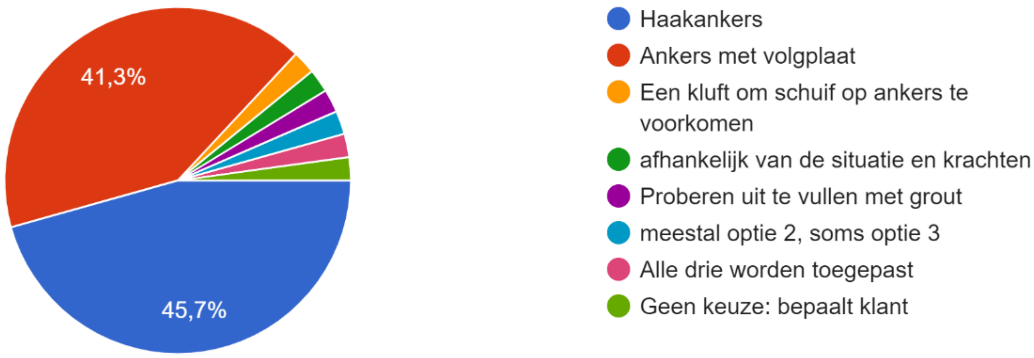
Welke gatspelling wordt er binnen jullie bedrijf toegepast bij voetplaten in staal-beton verbindingen.

46 antwoorden



Welke stelruimte word in basis gebruikt voor een M20 anker.
46 antwoorden

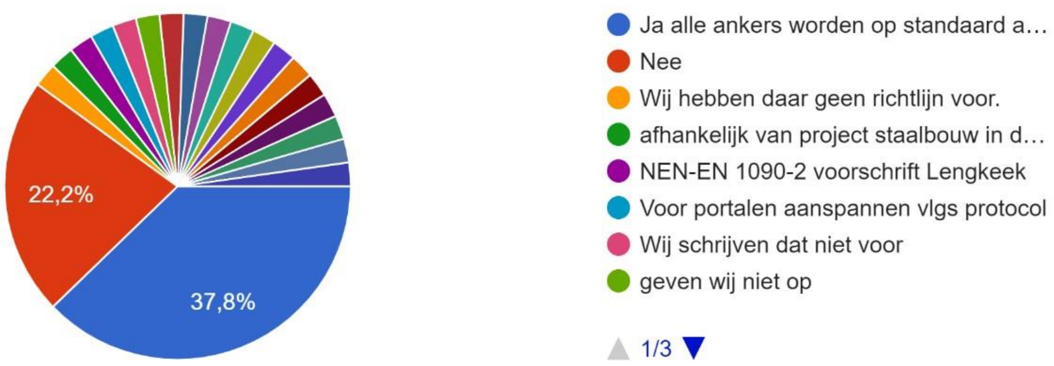
Wat voor type anker wordt in basis gebruikt bij voetplaten die op schuif worden belast, zoals bij een windverband.
46 antwoorden



De schuifcapaciteit van ankers kunnen volgens 2 codes worden getoetst. Welke code wordt in basis gebruikt binnen jullie bedrijf.
46 antwoorden

Binnen diverse software zoals die van Hilti, Fischer en IDEA kan stelruimte met mortel aan en uit worden gezet. Negeert u deze mortelstelruimte wel ...de capaciteit van de ankers op schuif tegen valt.
46 antwoorden

Worden ankers gemonteerd met een standaard aanhaalmoment "handvast" of heeft uw montage hier geen richtlijn voor.
45 antwoorden



Worden de gaten in voetplaten in het werk afgevuld om speling bij ankers te voorkomen met bijvoorbeeld lijm of houdt u toezicht als u dit voorschrijft dat dit ook gebeurd?
46 antwoorden

Do you or your company know of damage cases where anchors have broken on shear, do you know what caused this breakage and in what situation it happened? This is purely steel fracture and not concrete collapse.

No I am not familiar with this.	No.	not yet experienced
no	No	No but since wind braces sometimes fall outside the schematization of the main structure, I can imagine that there are once something goes wrong.
No never experienced it.	No	No concrete damage but always wrangling
No	No examples so to me shooting in.	No
No	No usually a combination, no pure slide	N/A.
No claim known	No	No never experienced
No, no known claims.	No	Never occurred.
No known case.	No	No
no	No	No damage cases known. Preferred application of hook anchors 4.6, because compared to 8.8 they have more deformation capacity have.
No out of 15000 details no situation experienced	no	no, no known cases.
no, not experienced in 28y	No	No
no, only in case of fire damage	No, going to check internally.	No, we are unable to provide claims
No	No, not known	No
no cases experienced in 30 years.	No	

What type of mortar is used to fill the setting space or do you prescribe to use .

This question was asked as an open-ended question and the responses were manually compiled into a summary.

Do not prescribe mortar	1
Shrinkable mortar	8
K30	1
K50	2
K60	1
K70	19
K80	1
HIT-RE500 or all around welding	1
Minimum compressive strength concrete	4
Other	3

2. Perform anchoring software

2.1. Control calculation software

For the shear capacity without set space, equation (2) is applied. Except for the calculation of class 5.8 anchors from Fischer, an αv of 0.5 is used in accordance with [5, p. 7.2.2.3.1] (fub > 500 N/mm^2). For the Class 5.8 anchor of Fischer, an αv = 0.6 for fub < 500 N/mm^2 is applied.

$$F_{v,Rd} = \frac{\alpha v * As * fub}{\gamma m2} = \frac{0,5 * 245 * 500}{1,25} = 49\ kN$$

$$F_{v,Rd} = \frac{\alpha v * As * fub}{\gamma m2} = \frac{0,6 * 245 * 500}{1,25} = 58.8\ kN$$

For the shear capacity with set space in uncracked concrete, equation (3) is applied. In all software, the degree of trapping αM adjustable. This is further elaborated below. For cracked concrete, equation (9) is used.

Arithmetic adjustment space l at 15 mm = 15 + 0.5*20 + 0.5*10 = 30 mm.
Arithmetic adjustment space l at 30 mm = 30 +-0.5*20 + 0.5*10 = 45 mm.

$$F_{v,Rd} = \frac{\alpha * M_{MRk,s}}{l * \gamma_{m2}} = \frac{2 * 1.5 * \frac{\pi}{32} * 17.7^3 * 400}{30 * 1,25} = 17.42\ kN$$

5.8
15 mm
adjustment space

$$F_{v,Rd} = \frac{\alpha * M_{MRk,s}}{l * \gamma_{m2}} = \frac{2 * 1.5 * \frac{\pi}{32} * 17.7^3 * 400}{45 * 1,25} = 11.61\ kN$$

5.8
30 mm
adjustment space

$$F_{v,Rd} = \frac{\alpha * M_{MRk,s}}{l * \gamma_{m2}} = \frac{2 * 1.5 * \frac{\pi}{32} * 17.7^3 * 640}{30 * 1,25} = 27.87\ kN$$

8.8
15 mm
adjustment space

$$F_{v,Rd} = \frac{\alpha * M_{MRk,s}}{l * \gamma_{m2}} = \frac{2 * 1.5 * \frac{\pi}{32} * 17.7^3 * 640}{45 * 1,25} = 18.58\ kN$$

8.8
30 mm
adjustment space

All software comes out equal on roundings among themselves. With the exception of the shear capacity of anchors without headroom in Fischer's calculation. Below are the shear capacity calculations of the different software.

		Suppliers			
	Setting space	B+Btec	Halfen	Fischer	Hilti
$Fv_{,Ed}$ (kN) Anchor M20	**5.8**				
	0	48.8	48.8	59.2	49.04
	15	17.28	17.33	17.28	17.31
	30	11.52	11.56	11.52	11.54
	8.8				
	0	78.4	x	78.4	78.4
	15	27.68	x	27.68	27.69
	30	18.45	x	18.45	18.46

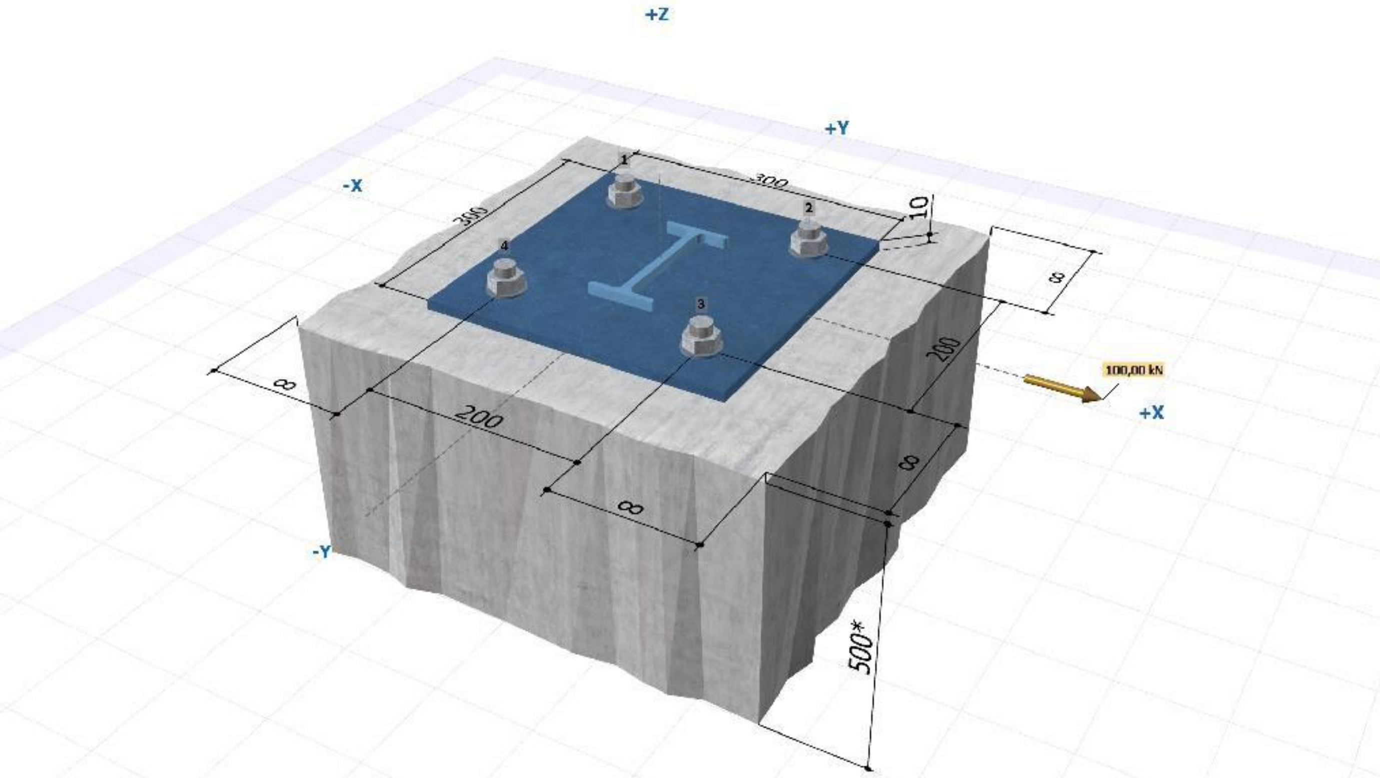
Table 2-1 Results of M20 anchors with different setting space from four different types of calculation software

2.2. B+Btec

M20 5.8 direct

2/8/2022 DesignFiX 3.4.8061.5

Leverancier:	**B+BTec Export Division** Munterij 8 | 4762 AH Zevenbergen Telefoon: +31 168 331 260, E-mail: info@bbtectools.com
Product:	**B+BTec Export Division** **Injectie systeem BIS-HY - Materiaal: 5.8 EV** M20 Draadstang: l = 90 + 10 + 21 + 5 = 126 mm
Ontwerpmethode: Goedkeuring:	ETAG 001, TR 029 ETA-16/0958 | 2/20/2017
Basismateriaal:	Ongescheurd beton C30/37 Geen of normale wapening Temperatuur: Korte termijn = 40°C | Lange termijn = 24°C
Installatie:	Voorsteekmontage | Hamerboren | Reiniging met perslucht (min. 6 bar | CAC) | Doorvoergat niet opgevuld. | Droog/nat
Verankeringsdiepte:	h_{ef} = 90 mm
Ankerplaat: Profiel:	Rechthoek 300 • 300 • 10 I-profiel - Grootte: I 140
Last: Eenheden:	Statisch/quasi-statisch Belasting [kN, kNm] | Afmetingen [mm], * Niet op schaal



Ankerlasten [kN]:

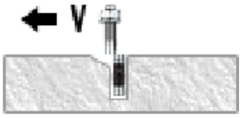
Anker	N	V	Vx	Vy
1	0.00	25.00	25.00	0.00
2	0.00	25.00	25.00	0.00
3	0.00	25.00	25.00	0.00
4	0.00	25.00	25.00	0.00

Beton:

Max. compressie	-	[‰]
Max. betondrukspanning	-	
Resulterende trekkracht	-	
xZ / yZ:	- / -	
Resulterende drukkracht	-	
xD / yD:	- / -	

V Afschuifkrachten

Staalbreuk (Zonder hefboomarm)

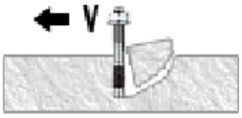


$\boxed{V^{h}_{Sd} \leq V_{Rk,s} / \gamma_{Ms}}$

25.00 $\leq$ 61.00 / 1.25 = 48.80 kN

Benuttingspercentage $\beta_{V,s}$ = 51.2%

Beton achteruitbreken:



Verificatie in overeenstemming met ETAG 001, TR 029, Vergelijking (5.7a)

$\boxed{V_{Rk,cp} = k \cdot N^{0}_{Rk,c} \cdot (A_{c,N} / A^{0}_{c,N}) \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N,x} \cdot \psi_{ec,N,y}}$

$\boxed{V^{g}_{Sd} \leq V_{Rk,cp} / \gamma_{Mcp}}$

De bevestiging bestaat uit 1 groep.

Ankernr. 1, 2, 3, 4

$V_{Rk,cp}$ = 2.0 · 52.45 · (220900 / 72900) · 1.000 · 1.000 · 1.000 · 1.000 = 317.90 kN

100.00 $\leq$ 317.90 / 1.50 = 211.93

Benuttingspercentage $\beta_{V,cp}$ = 47.2%

Verificatie in overeenstemming met ETAG 001, TR 029, Vergelijking (5.7)

$\boxed{V_{Rk,cp} = k \cdot N^{0}_{Rk,p} \cdot (A_{p,N} / A^{0}_{p,N}) \cdot \psi_{s,Np} \cdot \psi_{re,Np} \cdot \psi_{ec,Np,x} \cdot \psi_{ec,Np,y} \cdot \psi_{g,Np}}$

$\boxed{V^{g}_{Sd} \leq V_{Rk,cp} / \gamma_{Mcp}}$

De bevestiging bestaat uit 1 groep.

Ankernr. 1, 2, 3, 4

$V_{Rk,cp}$ = 2.0 · 82.33 · (220900 / 72900) · 1.000 · 1.000 · 1.000 · 1.000 · 1.000 = 498.96 kN

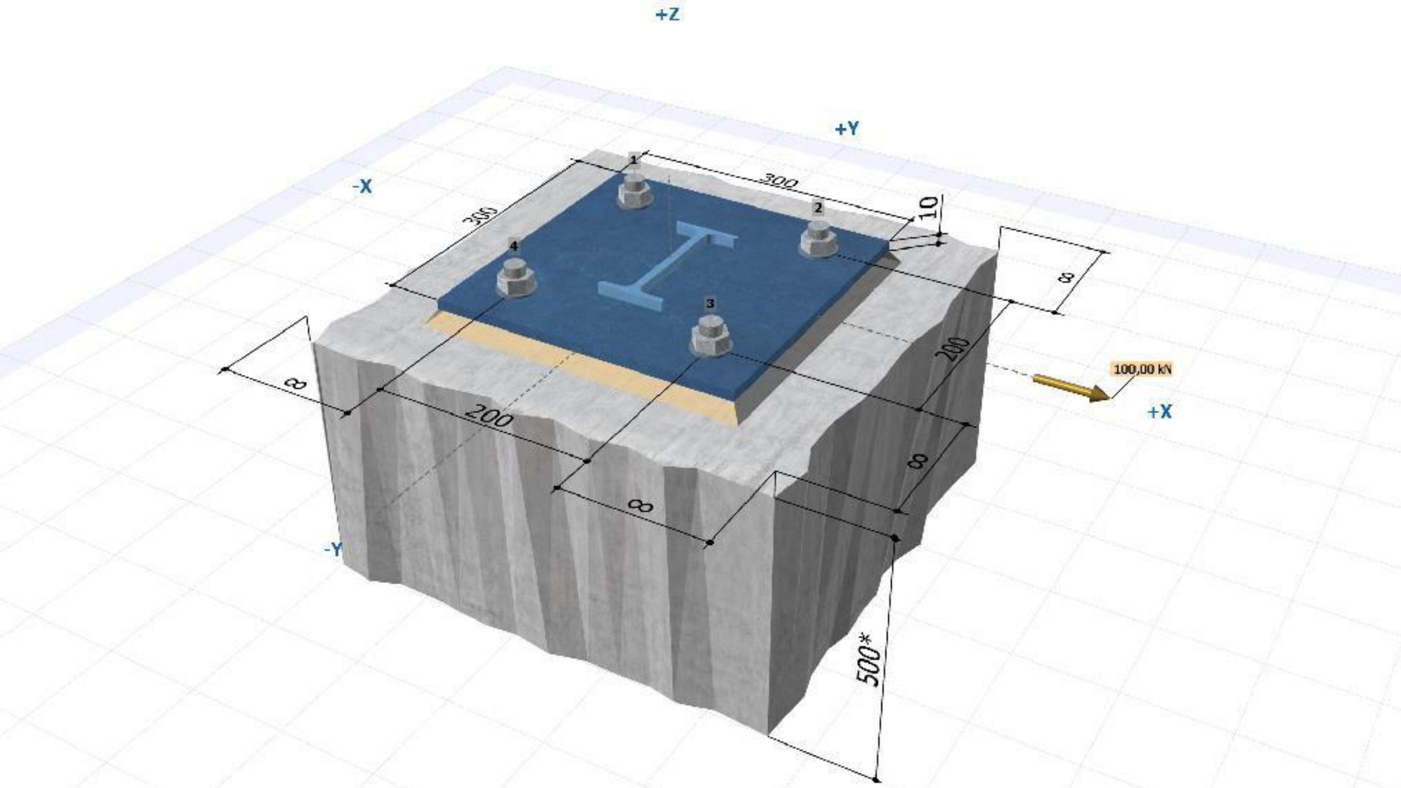
100.00 $\leq$ 498.96 / 1.50 = 332.64

Benuttingspercentage $\beta_{V,cp}$ = 30.1%

M20 5.8 15 mm adjustment space

3/4/2022 DesignFiX 3.4.8061.10

Leverancier:	**B+BTec Export Division** Munterij 8 | 4762 AH Zevenbergen Telefoon: +31 168 331 260, E-mail: info@bbtectools.com
Product:	**B+BTec Export Division** **Injectie systeem BIS-HY - Materiaal: 5.8 EV** M20 Draadstang: l = 90 + 25 + 21 + 5 = 141 mm
Ontwerpmethode: Goedkeuring:	ETAG 001, TR 029 ETA-16/0958 | 2/20/2017
Basismateriaal:	Ongescheurd beton C35/45 Dichte wapening Temperatuur: Korte termijn = 40°C | Lange termijn = 24°C
Installatie:	Voorsteekmontage | Hamerboren | Reiniging met perslucht (min. 6 bar | CAC) | Doorvoergat niet opgevuld. | Droog/nat
Verankeringsdiepte: Buiging:	h_{ef} = 90 mm Met ondersabeling | Afstand a = 15 mm Hefboomarm l = 30 mm | Mate van weerstand α_M = 2.0
Ankerplaat: Profiel:	Rechthoek 300 • 300 • 10 I-profiel - Grootte: I 140
Last: Eenheden:	Statisch/quasi-statisch Belasting [kN, kNm] | Afmetingen [mm], * Niet op schaal



Ankerlasten [kN]:

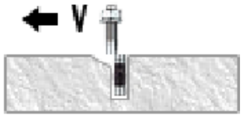
Anker	N	V	Vx	Vy
1	0.00	25.00	25.00	0.00
2	0.00	25.00	25.00	0.00
3	0.00	25.00	25.00	0.00
4	0.00	25.00	25.00	0.00

Beton:

Max. compressie	-	[‰]
Max. betondrukspanning	-	
Resulterende trekkracht	-	
xZ / yZ:	- / -	
Resulterende drukkracht	-	
xD / yD:	- / -	

Afschuifkrachten

Staalbreuk (Met hefboomarm)



$$\boxed{V_{Rk,s} = \alpha_M \cdot M^0_{Rk,s} \cdot (1 - N_{Sd} / (N_{Rk,s} / \gamma_{Ms})) / l}$$

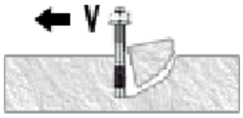
$$\boxed{V^h_{Sd} > V_{Rk,s} / \gamma_{Ms}}$$

$$\boxed{V_{Rk,s} = 2.0 \cdot 324.00 \cdot (1 - 0.00 / (122.00 / 1.50)) / 30 = 21.60 \text{ kN}}$$

25.00 > 21.60 / 1.25 = 17.28 kN

Benuttingspercentage $\beta_{V,s}$ = 144.7%

Beton achteruitbreken:



Verificatie in overeenstemming met ETAG 001, TR 029, Vergelijking (5.7a)

$$\boxed{V_{Rk,cp} = k \cdot N^0_{Rk,c} \cdot (A_{c,N} / A^0_{c,N}) \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N,x} \cdot \psi_{ec,N,y}}$$

$$\boxed{V^g_{Sd} \leq V_{Rk,cp} / \gamma_{Mcp}}$$

De bevestiging bestaat uit 1 groep.

Ankernr. 1, 2, 3, 4

$V_{Rk,cp} = 2.0 \cdot 57.85 \cdot (220900 / 72900) \cdot 1.000 \cdot 0.950 \cdot 1.000 \cdot 1.000 = 333.05$ kN

100.00 $\leq$ 333.05 / 1.50 = 222.04

Benuttingspercentage $\beta_{V,cp}$ = 45.0%

Verificatie in overeenstemming met ETAG 001, TR 029, Vergelijking (5.7)

$$\boxed{V_{Rk,cp} = k \cdot N^0_{Rk,p} \cdot (A_{p,N} / A^0_{p,N}) \cdot \psi_{s,Np} \cdot \psi_{re,Np} \cdot \psi_{ec,Np,x} \cdot \psi_{ec,Np,y} \cdot \psi_{g,Np}}$$

$$\boxed{V^g_{Sd} \leq V_{Rk,cp} / \gamma_{Mcp}}$$

De bevestiging bestaat uit 1 groep.

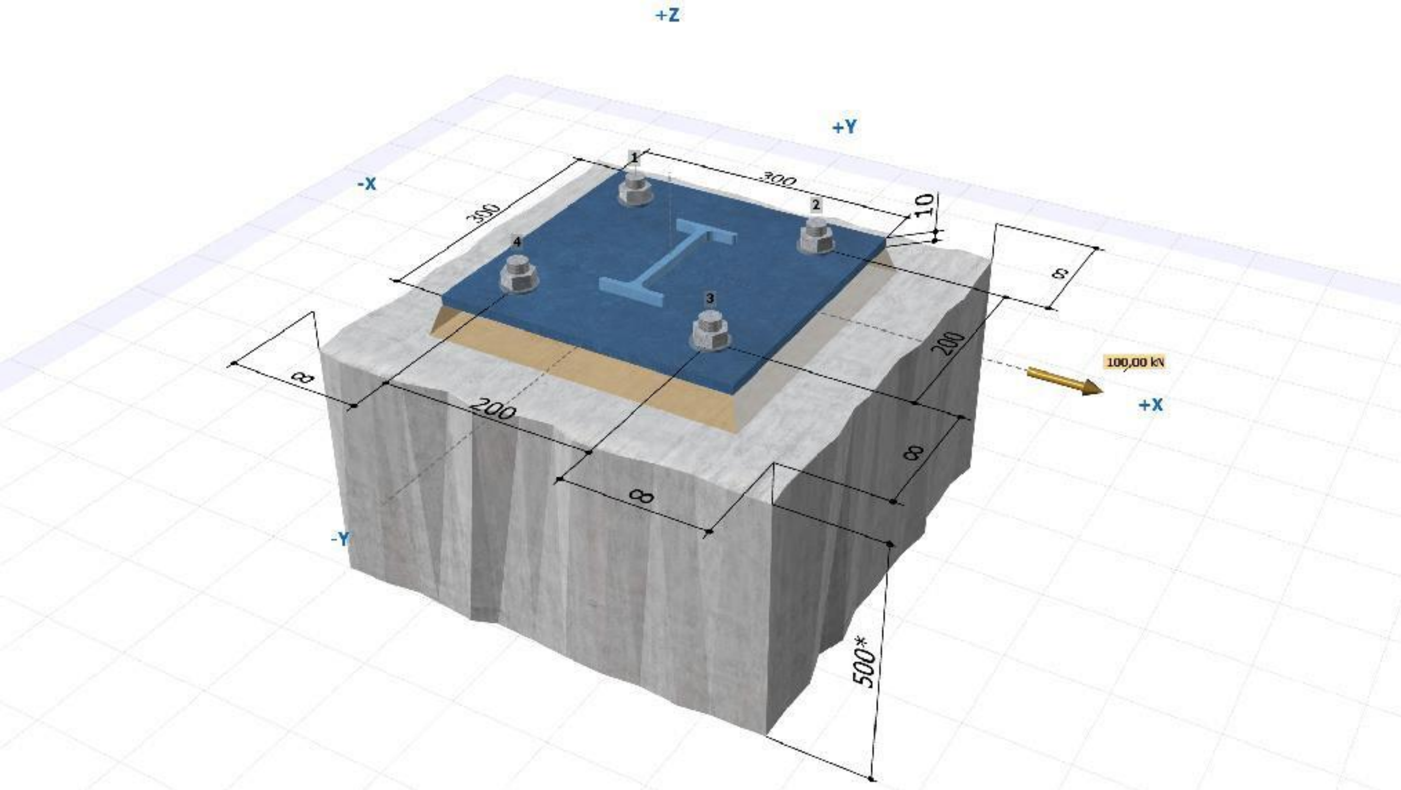
Ankernr. 1, 2, 3, 4

$V_{Rk,cp} = 2.0 \cdot 84.71 \cdot (220900 / 72900) \cdot 1.000 \cdot 0.950 \cdot 1.000 \cdot 1.000 \cdot 1.000 = 487.69$ kN

100.00 $\leq$ 487.69 / 1.50 = 325.13

Benuttingspercentage $\beta_{V,cp}$ = 30.8%

Leverancier:	**B+BTec Export Division** Munterij 8 | 4762 AH Zevenbergen Telefoon: +31 168 331 260, E-mail: info@bbtectools.com
Product:	**B+BTec Export Division** **Injectie systeem BIS-HY - Materiaal: 5.8 EV** M20 Draadstang: l = 90 + 40 + 21 + 5 = 156 mm
Ontwerpmethode:	ETAG 001, TR 029
Goedkeuring:	ETA-16/0958 | 2/20/2017
Basismateriaal:	Ongescheurd beton C35/45 Dichte wapening Temperatuur: Korte termijn = 40°C | Lange termijn = 24°C
Installatie:	Voorsteekmontage | Hamerboren | Reiniging met perslucht (min. 6 bar | CAC) | Doorvoergat niet opgevuld. | Droog/nat
Verankeringsdiepte:	h_{ef} = 90 mm
Buiging:	Met ondersabeling | Afstand a = 30 mm Hefboomarm l = 45 mm | Mate van weerstand α_M = 2.0
Ankerplaat:	Rechthoek 300 • 300 • 10
Profiel:	I-profiel - Grootte: I 140
Last:	Statisch/quasi-statisch
Eenheden:	Belasting [kN, kNm] | Afmetingen [mm], * Niet op schaal



Ankerlasten [kN]:

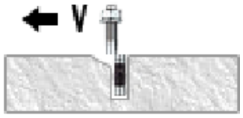
Anker	N	V	Vx	Vy
1	0.00	25.00	25.00	0.00
2	0.00	25.00	25.00	0.00
3	0.00	25.00	25.00	0.00
4	0.00	25.00	25.00	0.00

Beton:

Max. compressie	-	[‰]
Max. betondrukspanning	-	
Resulterende trekkracht	-	
xZ / yZ:	- / -	
Resulterende drukkracht	-	
xD / yD:	- / -	

Afschuifkrachten

Staalbreuk (Met hefboomarm)



$\boxed{V_{Rk,s} = \alpha_M \cdot M^0_{Rk,s} \cdot (1 - N_{Sd} / (N_{Rk,s} / \gamma_{Ms})) / l}$

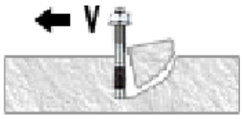
$\boxed{V^h_{Sd} > V_{Rk,s} / \gamma_{Ms}}$

$\boxed{V_{Rk,s} = 2.0 \cdot 324.00 \cdot (1 - 0.00 / (122.00 / 1.50)) / 45 = 14.40 \text{ kN}}$

25.00 > 14.40 / 1.25 = 11.52 kN

Benuttingspercentage $\beta_{V,s}$ = 217.0%

Beton achteruitbreken:



Verificatie in overeenstemming met ETAG 001, TR 029, Vergelijking (5.7a)

$\boxed{V_{Rk,cp} = k \cdot N^0_{Rk,c} \cdot (A_{c,N} / A^0_{c,N}) \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N,x} \cdot \psi_{ec,N,y}}$

$\boxed{V^g_{Sd} \leq V_{Rk,cp} / \gamma_{Mcp}}$

De bevestiging bestaat uit 1 groep.

Ankernr. 1, 2, 3, 4

$V_{Rk,cp} = 2.0 \cdot 57.85 \cdot (220900 / 72900) \cdot 1.000 \cdot 0.950 \cdot 1.000 \cdot 1.000 = 333.05$ kN

100.00 $\leq$ 333.05 / 1.50 = 222.04

Benuttingspercentage $\beta_{V,cp}$ = 45.0%

Verificatie in overeenstemming met ETAG 001, TR 029, Vergelijking (5.7)

$\boxed{V_{Rk,cp} = k \cdot N^0_{Rk,p} \cdot (A_{p,N} / A^0_{p,N}) \cdot \psi_{s,Np} \cdot \psi_{re,Np} \cdot \psi_{ec,Np,x} \cdot \psi_{ec,Np,y} \cdot \psi_{g,Np}}$

$\boxed{V^g_{Sd} \leq V_{Rk,cp} / \gamma_{Mcp}}$

De bevestiging bestaat uit 1 groep.

Ankernr. 1, 2, 3, 4

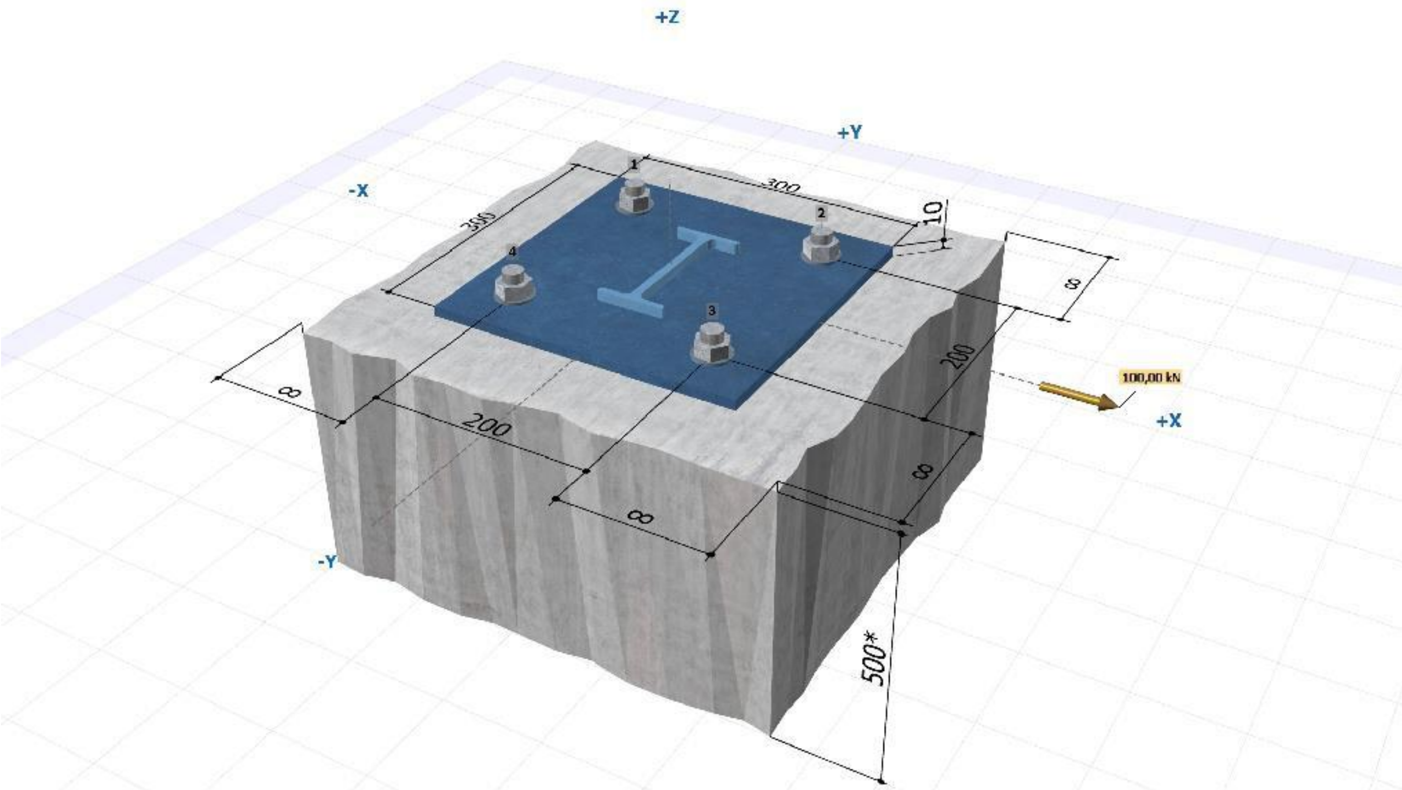
$V_{Rk,cp} = 2.0 \cdot 84.71 \cdot (220900 / 72900) \cdot 1.000 \cdot 0.950 \cdot 1.000 \cdot 1.000 \cdot 1.000 = 487.69$ kN

100.00 $\leq$ 487.69 / 1.50 = 325.13

Benuttingspercentage $\beta_{V,cp}$ = 30.8%

M20 8.8 direct

Leverancier:	**B+BTec Export Division** Munterij 8 | 4762 AH Zevenbergen Telefoon: +31 168 331 260, E-mail: info@bbtectools.com
Product:	**B+BTec Export Division** **Injectie systeem BIS-HY - Materiaal: 8.8 EV** M20 Draadstang: l = 90 + 10 + 21 + 5 = 126 mm
Ontwerpmethode:	ETAG 001, TR 029
Goedkeuring:	ETA-16/0958 | 2/20/2017
Basismateriaal:	Ongescheurd beton C30/37 Geen of normale wapening Temperatuur: Korte termijn = 40°C | Lange termijn = 24°C
Installatie:	Voorsteekmontage | Hamerboren | Reiniging met perslucht (min. 6 bar | CAC) | Doorvoergat niet opgevuld. | Droog/nat
Verankeringsdiepte:	h_{ef} = 90 mm
Ankerplaat:	Rechthoek 300 • 300 • 10
Profiel:	I-profiel - Grootte: I 140
Last:	Statisch/quasi-statisch
Eenheden:	Belasting [kN, kNm] | Afmetingen [mm], * Niet op schaal



Ankerlasten [kN]:

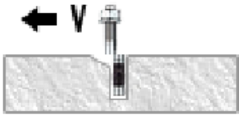
Anker	N	V	Vx	Vy
1	0.00	25.00	25.00	0.00
2	0.00	25.00	25.00	0.00
3	0.00	25.00	25.00	0.00
4	0.00	25.00	25.00	0.00

Beton:

Max. compressie	-	[‰]
Max. betondrukspanning	-	
Resulterende trekkracht	-	
xZ / yZ:	- / -	
Resulterende drukkracht	-	
xD / yD:	- / -	

Afschuifkrachten

Staalbreuk (Zonder hefboomarm)

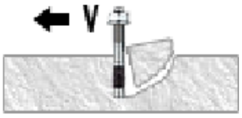


$\boxed{V^{h}_{Sd} \leq V_{Rk,s} / \gamma_{Ms}}$

25.00 $\leq$ 98.00 / 1.25 = 78.40 kN

Benuttingspercentage $\beta_{V,s}$ = 31.9%

Beton achteruitbreken:



Verificatie in overeenstemming met ETAG 001, TR 029, Vergelijking (5.7a)

$\boxed{V_{Rk,cp} = k \cdot N^{0}_{Rk,c} \cdot (A_{c,N} / A^{0}_{c,N}) \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N,x} \cdot \psi_{ec,N,y}}$

$\boxed{V^{g}_{Sd} \leq V_{Rk,cp} / \gamma_{Mcp}}$

De bevestiging bestaat uit 1 groep.

Ankernr. 1, 2, 3, 4

$V_{Rk,cp}$ = 2.0 · 52.45 · (220900 / 72900) · 1.000 · 1.000 · 1.000 · 1.000 = 317.90 kN

100.00 $\leq$ 317.90 / 1.50 = 211.93

Benuttingspercentage $\beta_{V,cp}$ = 47.2%

Verificatie in overeenstemming met ETAG 001, TR 029, Vergelijking (5.7)

$\boxed{V_{Rk,cp} = k \cdot N^{0}_{Rk,p} \cdot (A_{p,N} / A^{0}_{p,N}) \cdot \psi_{s,Np} \cdot \psi_{re,Np} \cdot \psi_{ec,Np,x} \cdot \psi_{ec,Np,y} \cdot \psi_{g,Np}}$

$\boxed{V^{g}_{Sd} \leq V_{Rk,cp} / \gamma_{Mcp}}$

De bevestiging bestaat uit 1 groep.

Ankernr. 1, 2, 3, 4

$V_{Rk,cp}$ = 2.0 · 82.33 · (220900 / 72900) · 1.000 · 1.000 · 1.000 · 1.000 · 1.000 = 498.96 kN

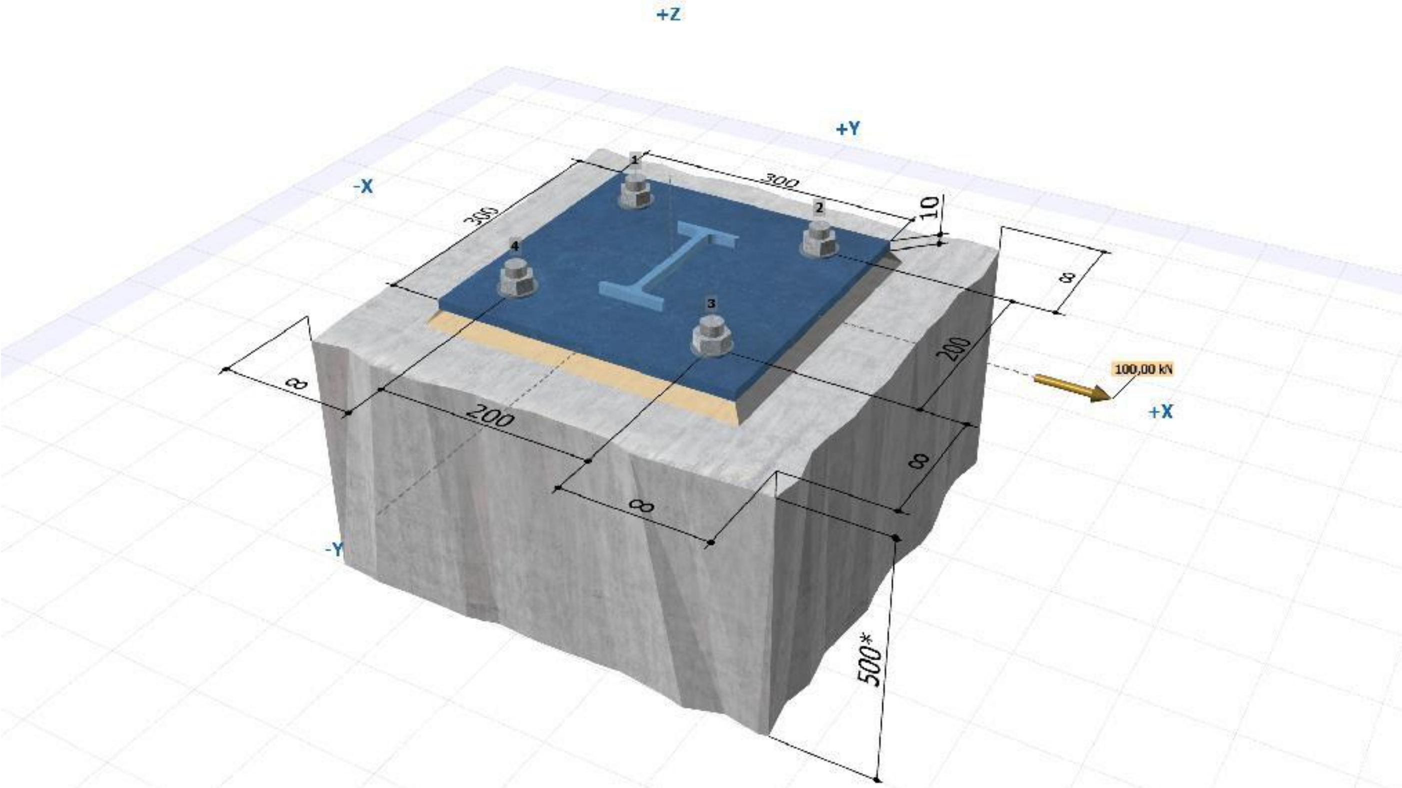
100.00 $\leq$ 498.96 / 1.50 = 332.64

Benuttingspercentage $\beta_{V,cp}$ = 30.1%

M20 8.8 15 mm adjustment space

3/4/2022 DesignFiX 3.4.8061.16

Leverancier:	**B+BTec Export Division** Munterij 8 | 4762 AH Zevenbergen Telefoon: +31 168 331 260, E-mail: info@bbtectools.com
Product:	**B+BTec Export Division** **Injectie systeem BIS-HY - Materiaal: 8.8 EV** M20 Draadstang: l = 90 + 25 + 21 + 5 = 141 mm
Ontwerpmethode:	ETAG 001, TR 029
Goedkeuring:	ETA-16/0958 | 2/20/2017
Basismateriaal:	Ongescheurd beton C35/45 Dichte wapening Temperatuur: Korte termijn = 40°C | Lange termijn = 24°C
Installatie:	Voorsteekmontage | Hamerboren | Reiniging met perslucht (min. 6 bar | CAC) | Doorvoergat niet opgevuld. | Droog/nat
Verankeringsdiepte:	h_{ef} = 90 mm
Buiging:	Met ondersabeling | Afstand a = 15 mm Hefboomarm l = 30 mm | Mate van weerstand α_M = 2.0
Ankerplaat:	Rechthoek 300 • 300 • 10
Profiel:	I-profiel - Grootte: I 140
Last:	Statisch/quasi-statisch
Eenheden:	Belasting [kN, kNm] | Afmetingen [mm], * Niet op schaal



Ankerlasten [kN]:

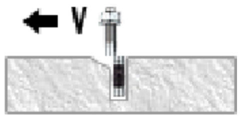
Anker	N	V	Vx	Vy
1	0.00	25.00	25.00	0.00
2	0.00	25.00	25.00	0.00
3	0.00	25.00	25.00	0.00
4	0.00	25.00	25.00	0.00

Beton:

Max. compressie	-	[‰]
Max. betondrukspanning	-	
Resulterende trekkracht	-	
xZ / yZ:	- / -	
Resulterende drukkracht	-	
xD / yD:	- / -	

V Afschuifkrachten

Staalbreuk (Met hefboomarm)



$V_{Rk,s} = \alpha_M \cdot M^0_{Rk,s} \cdot (1 - N_{Sd} / (N_{Rk,s} / \gamma_{Ms}))//$

$V^h_{Sd} \leq V_{Rk,s} / \gamma_{Ms}$

$V_{Rk,s} = 2.0 \cdot 519.00 \cdot (1 - 0.00 / (196.00 / 1.50)) / 30 = 34.60$ kN

$25.00 \leq 34.60 / 1.25 = 27.68$ kN

Benuttingspercentage $\beta_{V,s} = 90.3\%$

Beton achteruitbreken:

Verificatie in overeenstemming met ETAG 001, TR 029, Vergelijking (5.7a)

$V_{Rk,cp} = k \cdot N^0_{Rk,c} \cdot (A_{c,N} / A^0_{c,N}) \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N,x} \cdot \psi_{ec,N,y}$

$V^g_{Sd} \leq V_{Rk,cp} / \gamma_{Mcp}$

De bevestiging bestaat uit 1 groep.

Ankernr. 1, 2, 3, 4

$V_{Rk,cp} = 2.0 \cdot 57.85 \cdot (220900 / 72900) \cdot 1.000 \cdot 0.950 \cdot 1.000 \cdot 1.000 = 333.05$ kN

$100.00 \leq 333.05 / 1.50 = 222.04$

Benuttingspercentage $\beta_{V,cp} = 45.0\%$

Verificatie in overeenstemming met ETAG 001, TR 029, Vergelijking (5.7)

$V_{Rk,cp} = k \cdot N^0_{Rk,p} \cdot (A_{p,N} / A^0_{p,N}) \cdot \psi_{s,Np} \cdot \psi_{re,Np} \cdot \psi_{ec,Np,x} \cdot \psi_{ec,Np,y} \cdot \psi_{g,Np}$

$V^g_{Sd} \leq V_{Rk,cp} / \gamma_{Mcp}$

De bevestiging bestaat uit 1 groep.

Ankernr. 1, 2, 3, 4

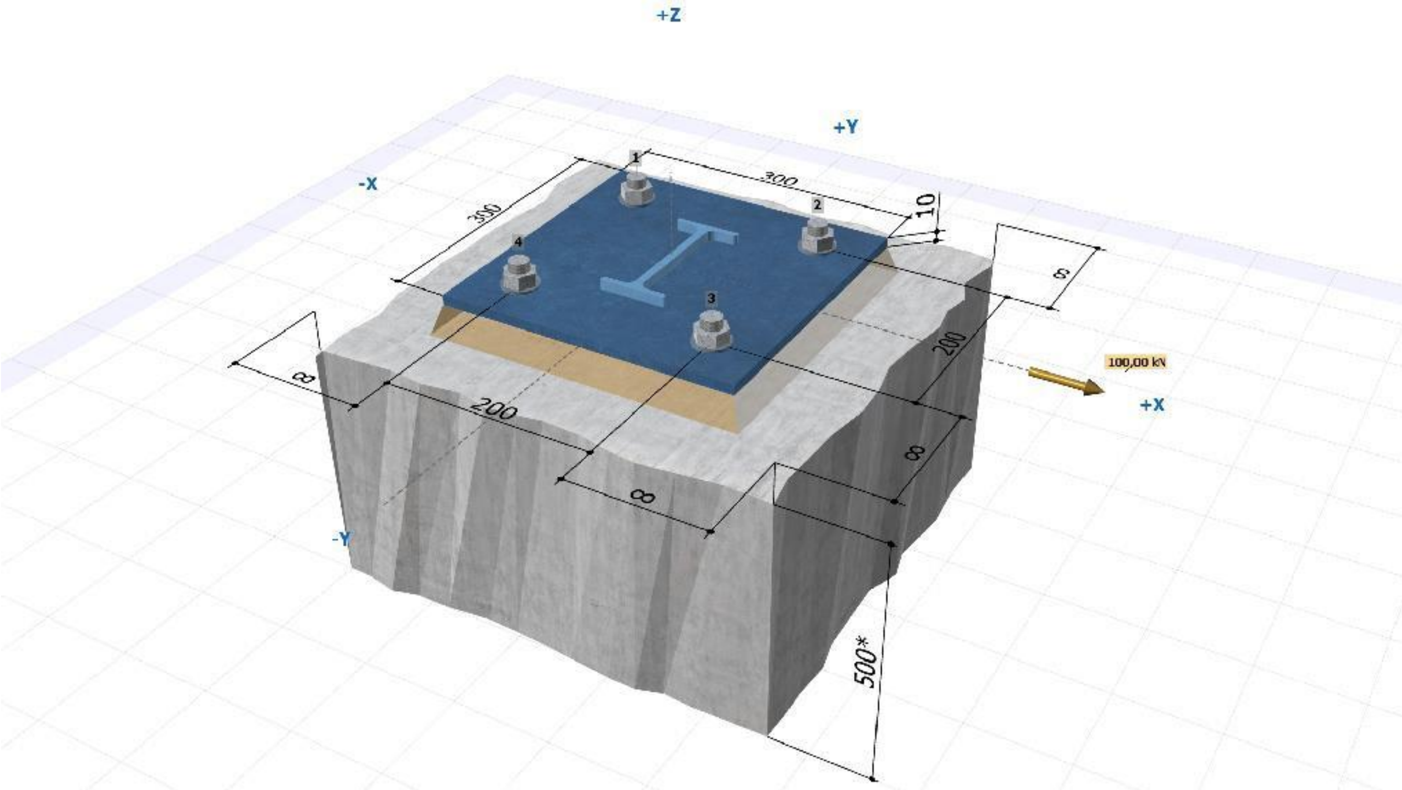
$V_{Rk,cp} = 2.0 \cdot 84.71 \cdot (220900 / 72900) \cdot 1.000 \cdot 0.950 \cdot 1.000 \cdot 1.000 \cdot 1.000 = 487.69$ kN

$100.00 \leq 487.69 / 1.50 = 325.13$

Benuttingspercentage $\beta_{V,cp} = 30.8\%$

M20 8.8 30 mm adjustment space

Leverancier:	**B+BTec Export Division** Munterij 8 | 4762 AH Zevenbergen Telefoon: +31 168 331 260, E-mail: info@bbtectools.com
Product:	**B+BTec Export Division** **Injectie systeem BIS-HY - Materiaal: 8.8 EV** M20 Draadstang: l = 90 + 40 + 21 + 5 = 156 mm
Ontwerpmethode:	ETAG 001, TR 029
Goedkeuring:	ETA-16/0958 | 2/20/2017
Basismateriaal:	Ongescheurd beton C35/45 Dichte wapening Temperatuur: Korte termijn = 40°C | Lange termijn = 24°C
Installatie:	Voorsteekmontage | Hamerboren | Reiniging met perslucht (min. 6 bar | CAC) | Doorvoergat niet opgevuld. | Droog/nat
Verankeringsdiepte:	h_{ef} = 90 mm
Buiging:	Met ondersabeling | Afstand a = 30 mm Hefboomarm l = 45 mm | Mate van weerstand α_M = 2.0
Ankerplaat:	Rechthoek 300 • 300 • 10
Profiel:	I-profiel - Grootte: I 140
Last:	Statisch/quasi-statisch
Eenheden:	Belasting [kN, kNm] | Afmetingen [mm], * Niet op schaal



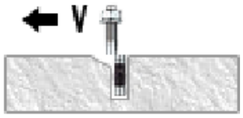
Ankerlasten [kN]:

Anker	N	V	Vx	Vy
1	0.00	25.00	25.00	0.00
2	0.00	25.00	25.00	0.00
3	0.00	25.00	25.00	0.00
4	0.00	25.00	25.00	0.00

Beton:		
Max. compressie	-	[‰]
Max. betondrukspanning	-	
Resulterende trekkracht	-	
xZ / yZ:	- / -	
Resulterende drukkracht	-	
xD / yD:	- / -	

Afschuifkrachten

Staalbreuk (Met hefboomarm)



$V_{Rk,s} = \alpha_M \cdot M^0_{Rk,s} \cdot (1 - N_{Sd} / (N_{Rk,s} / \gamma_{Ms})) / l$

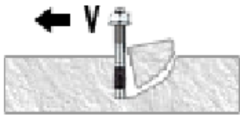
$V^h_{Sd} > V_{Rk,s} / \gamma_{Ms}$

$V_{Rk,s} = 2.0 \cdot 519.00 \cdot (1 - 0.00 / (196.00 / 1.50)) / 45 = 23.07$ kN

25.00 > 23.07 / 1.25 = 18.45 kN

Benuttingspercentage $\beta_{V,s} = 135.5\%$

Beton achteruitbreken:



Verificatie in overeenstemming met ETAG 001, TR 029, Vergelijking (5.7a)

$V_{Rk,cp} = k \cdot N^0_{Rk,c} \cdot (A_{c,N} / A^0_{c,N}) \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N,x} \cdot \psi_{ec,N,y}$

$V^g_{Sd} \leq V_{Rk,cp} / \gamma_{Mcp}$

De bevestiging bestaat uit 1 groep.

Ankernr. 1, 2, 3, 4

$V_{Rk,cp} = 2.0 \cdot 57.85 \cdot (220900 / 72900) \cdot 1.000 \cdot 0.950 \cdot 1.000 \cdot 1.000 = 333.05$ kN

$100.00 \leq 333.05 / 1.50 = 222.04$

Benuttingspercentage $\beta_{V,cp} = 45.0\%$

Verificatie in overeenstemming met ETAG 001, TR 029, Vergelijking (5.7)

$V_{Rk,cp} = k \cdot N^0_{Rk,p} \cdot (A_{p,N} / A^0_{p,N}) \cdot \psi_{s,Np} \cdot \psi_{re,Np} \cdot \psi_{ec,Np,x} \cdot \psi_{ec,Np,y} \cdot \psi_{g,Np}$

$V^g_{Sd} \leq V_{Rk,cp} / \gamma_{Mcp}$

De bevestiging bestaat uit 1 groep.

Ankernr. 1, 2, 3, 4

$V_{Rk,cp} = 2.0 \cdot 84.71 \cdot (220900 / 72900) \cdot 1.000 \cdot 0.950 \cdot 1.000 \cdot 1.000 \cdot 1.000 = 487.69$ kN

$100.00 \leq 487.69 / 1.50 = 325.13$

Benuttingspercentage $\beta_{V,cp} = 30.8\%$

2.3. Fischer

M20 5.8 direct

C-FIX
1.0.36.0
Database versie
1.0.36.0
Datum
07/02/2022

Ontwerp specificaties

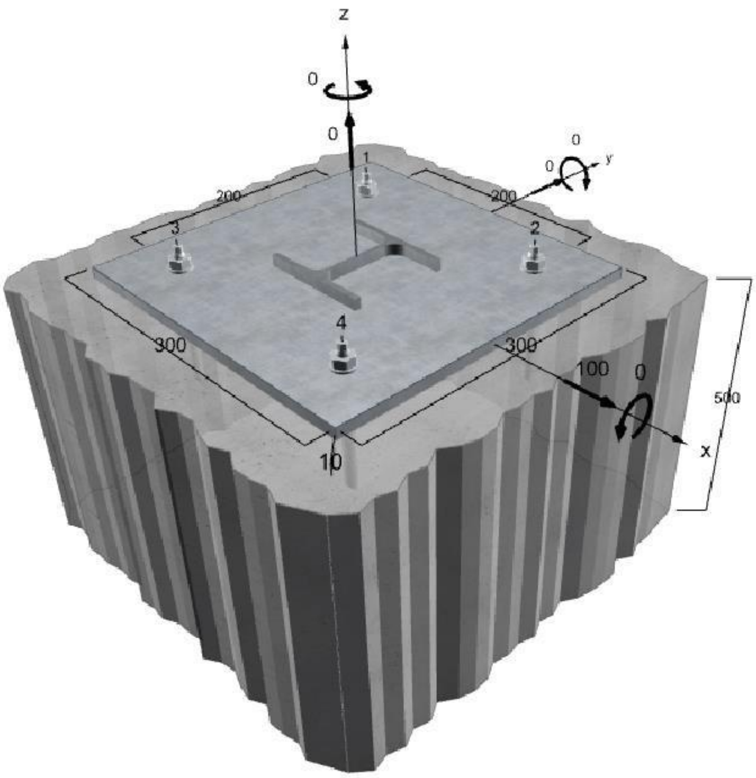
Anker

Anker systeem	fischer Superbond injectiesysteem FIS SB met ankerstang FIS A of RG M
Injectiemortel	FIS SB 390 S
Bevestigingselement	RG M20 x 220
Verankeringsdiepte	90 mm
Ontwerp gegevens	ETA-12/0258

Geometrie / Belastingen / Eenheden

mm, kN, kNm

Rekenwaarde van de optredende krachten (inclusief veiligheidsfactoren)



Niet op schaal

Data

Ontwerp methode	EN 1992-4
Ondergrond	Normaal beton, C30/37
Betonsituatie	Ongescheurd, Droog boorgat
Temperatuur bereik	24 °C Lange duur temperatuur, 40 °C Korte duur temperatuur
Wapening	Geen of gewone wapening, Randwapening met beugels, With reinforcement to control splitting
Boormethode	Hamer boren
Installatie soort	Doorsteek montage

Tussenruimte	Gevuld
Soort belasting	Statisch of quasi-statisch
Ankerplaat locatie	Ankerplaat koud tegen ondergrond
Ankerplaat afmetingen	300 mm x 300 mm x 10 mm
Soort Profiel	HEAA

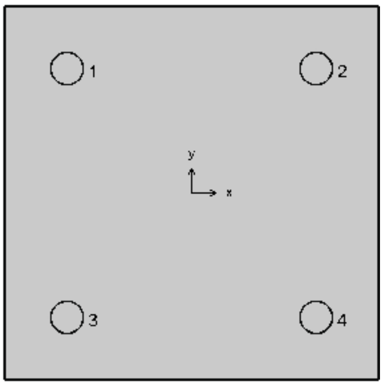
Rekenwaarde van de belastingen *)

#	N_{Ed} kN	$V_{Ed,x}$ kN	$V_{Ed,y}$ kN	$M_{Ed,x}$ kNm	$M_{Ed,y}$ kNm	$M_{T,Ed}$ kNm	Soort belasting
1	0,00	100,00	0,00	0,00	0,00	0,00	Statisch of quasi-statisch

*) Inclusief de benodigde veiligheidsfactoren

Resultante anker krachten

Anker nr.	Trek belasting kN	Afschuif belasting kN	Afschuif belasting x kN	Afschuif belasting y kN
1	0,00	25,00	25,00	0,00
2	0,00	25,00	25,00	0,00
3	0,00	25,00	25,00	0,00
4	0,00	25,00	25,00	0,00



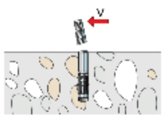
Max. betondruksterkte	0,00 ‰
Max. betondruksterkte	0,00 N/mm²
Resulterende trek krachten	0,00 kN, X/Y positie (0 / 0 mm)
Resulterende druk krachten	0,00 kN, X/Y positie (0 / 0 mm)

Weerstand bij afschuifbelastingen

Onderbouwing	Belasting kN	Capaciteit kN	Uitnutting β_V %
Staalbezwijken zonder hefboomsarm *	25,00	59,20	42,2
Beton achteruit breken	100,00	207,84	**48,1**

* Maatgevende anker

Staalbezwijken zonder hefboomsarm



$$V_{Ed} \leq V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{Ms}}$$

$$V_{Rk,s} = k_7 \cdot V^0_{Rk,s} = 1{,}00 \cdot 74{,}00\,kN = 74{,}00\,kN$$

$V_{Rk,s}$ kN	γ_{Ms}	$V_{Rd,s}$ kN	V_{Ed} kN	$\beta_{V,s}$ %
74,00	1,25	59,20	25,00	42,2

C-FIX
1.0.36.0
Database versie
1.0.36.0
Datum
04/03/2022

Ontwerp specificaties

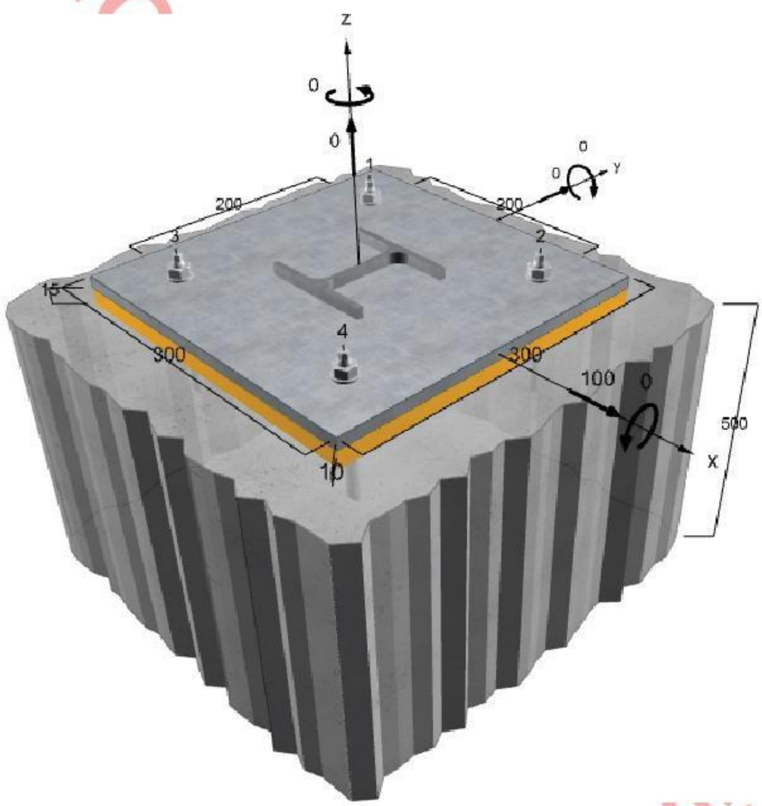
Anker

Anker systeem	fischer Superbond injectiesysteem FIS SB met ankerstang FIS A of RG M
Injectiemortel	FIS SB 390 S
Bevestigingselement	RG M20 x 500
Verankeringsdiepte	400 mm
Ontwerp gegevens	ETA-12/0258

Geometrie / Belastingen / Eenheden

mm, kN, kNm

Rekenwaarde van de optredende krachten (inclusief veiligheidsfactoren)



Niet op schaal

Data

Ontwerp methode	EN 1992-4
Ondergrond	Normaal beton, C30/37
Betonsituatie	Ongescheurd, Droog boorgat
Temperatuur bereik	24 °C Lange duur temperatuur, 40 °C Korte duur temperatuur
Wapening	Geen of gewone wapening, Randwapening met beugels, With reinforcement to control splitting
Boormethode	Hamer boren
Installatie soort	Doorsteek montage

Tussenruimte	Gevuld
Soort belasting	Statisch of quasi-statisch
Ankerplaat locatie	Ankerplaat met vullaag
Ankerplaat afmetingen	300 mm x 300 mm x 10 mm
Soort Profiel	HEAA

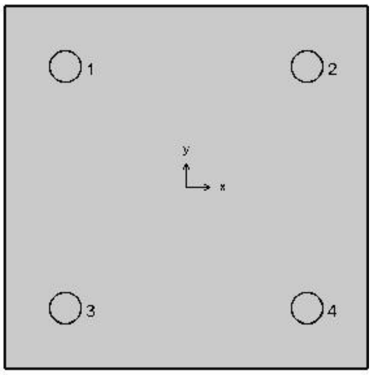
Rekenwaarde van de belastingen *)

#	N_{Ed} kN	$V_{Ed,x}$ kN	$V_{Ed,y}$ kN	$M_{Ed,x}$ kNm	$M_{Ed,y}$ kNm	$M_{T,Ed}$ kNm	Soort belasting
1	0,00	100,00	0,00	0,00	0,00	0,00	Statisch of quasi-statisch

*) Inclusief de benodigde veiligheidsfactoren

Resultante anker krachten

Anker nr.	Trek belasting kN	Afschuif belasting kN	Afschuif belasting x kN	Afschuif belasting y kN
1	0,00	25,00	25,00	0,00
2	0,00	25,00	25,00	0,00
3	0,00	25,00	25,00	0,00
4	0,00	25,00	25,00	0,00



Max. betondruksterkte	0,00 ‰
Max. betondruksterkte	0,00 N/mm²
Resulterende trek krachten	0,00 kN, X/Y positie (0 / 0 mm)
Resulterende druk krachten	0,00 kN, X/Y positie (0 / 0 mm)

Weerstand bij afschuifbelastingen

Onderbouwing	Belasting kN	Capaciteit kN	Uitnutting β_V %
Staalbezwijken met hefboomsarm *	25,00	17,28	**144,7**
Beton achteruit breken	100,00	874,73	11,4

* Maatgevende anker

Staalbezwijken met hefboomsarm

$$V_{Ed} \leq V_{Rd,s,M} = \frac{V_{Rk,s,M}}{\gamma_{Ms}}$$

$$V_{Rk,s,M} = \frac{\alpha_M \cdot M_{Rk,s}}{l_a} = \frac{2 \cdot 324{,}00\,Nm}{30\,mm} = 21{,}60\,kN$$

$$M_{Rk,s} = M^0_{Rk,s} \cdot (1 - N_{Ed} / N_{Rd,s}) = 324{,}00\,Nm \cdot (1 - 0{,}00\,kN / 82{,}00\,kN) = 324{,}00\,Nm$$

$V_{Rk,s,M}$ kN	γ_{Ms}	$V_{Rd,s,M}$ kN	V_{Ed} kN	$\beta_{V,s,M}$ %
21,60	1,25	17,28	25,00	144,7

UITNUTTINGSPERCENTAGE LIGT BOVEN

C-FIX
1.0.36.0
Database versie
1.0.36.0
Datum
04/03/2022

Ontwerp specificaties

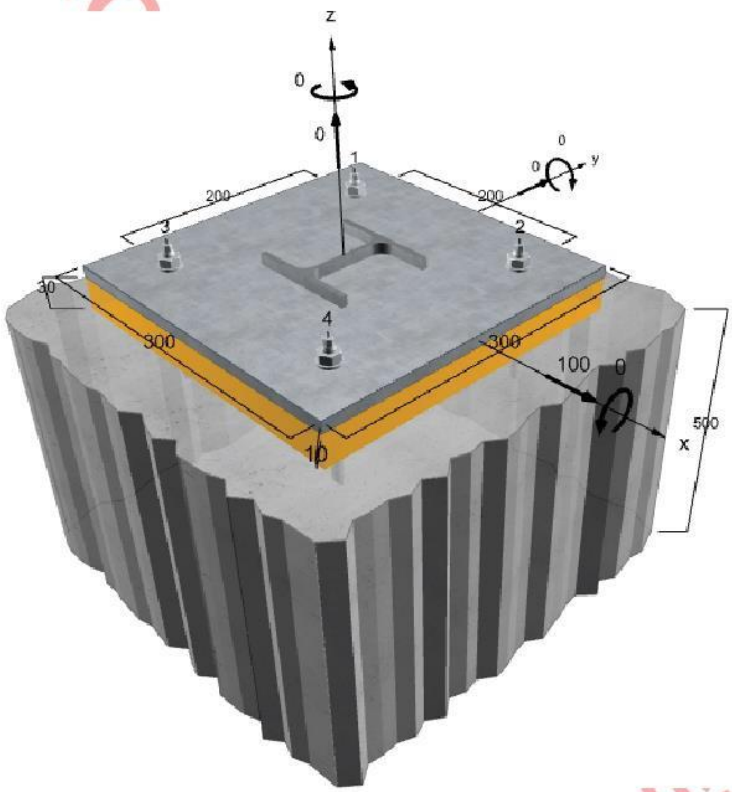
Anker

Anker systeem	fischer Superbond injectiesysteem FIS SB met ankerstang FIS A of RG M
Injectiemortel	FIS SB 390 S
Bevestigingselement	RG M20 x 500
Verankeringsdiepte	400 mm
Ontwerp gegevens	ETA-12/0258

Geometrie / Belastingen / Eenheden

mm, kN, kNm

Rekenwaarde van de optredende krachten (inclusief veiligheidsfactoren)



Niet op schaal

Data

Ontwerp methode	EN 1992-4
Ondergrond	Normaal beton, C30/37
Betonsituatie	Ongescheurd, Droog boorgat
Temperatuur bereik	24 °C Lange duur temperatuur, 40 °C Korte duur temperatuur
Wapening	Geen of gewone wapening, Randwapening met beugels, With reinforcement to control splitting
Boormethode	Hamer boren
Installatie soort	Doorsteek montage

Tussenruimte	Gevuld
Soort belasting	Statisch of quasi-statisch
Ankerplaat locatie	Ankerplaat met vullaag
Ankerplaat afmetingen	300 mm x 300 mm x 10 mm
Soort Profiel	HEAA

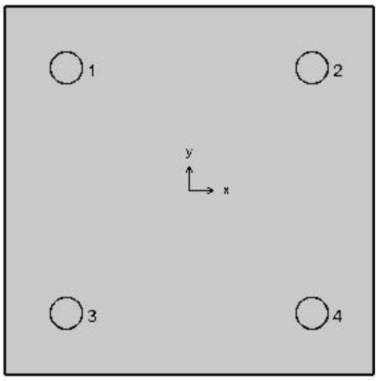
Rekenwaarde van de belastingen *)

#	N_{Ed} kN	$V_{Ed,x}$ kN	$V_{Ed,y}$ kN	$M_{Ed,x}$ kNm	$M_{Ed,y}$ kNm	$M_{T,Ed}$ kNm	Soort belasting
1	0,00	100,00	0,00	0,00	0,00	0,00	Statisch of quasi-statisch

*) Inclusief de benodigde veiligheidsfactoren

Resultante anker krachten

Anker nr.	Trek belasting kN	Afschuif belasting kN	Afschuif belasting x kN	Afschuif belasting y kN
1	0,00	25,00	25,00	0,00
2	0,00	25,00	25,00	0,00
3	0,00	25,00	25,00	0,00
4	0,00	25,00	25,00	0,00



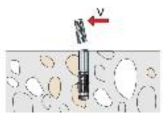
Max. betondruksterkte	0,00 ‰
Max. betondruksterkte	0,00 N/mm²
Resulterende trek krachten	0,00 kN, X/Y positie (0 / 0 mm)
Resulterende druk krachten	0,00 kN, X/Y positie (0 / 0 mm)

Weerstand bij afschuifbelastingen

Onderbouwing	Belasting kN	Capaciteit kN	Uitnutting β_V %
Staalbezwijken met hefboomsarm *	25,00	11,52	**217,0**
Beton achteruit breken	100,00	874,73	11,4

* Maatgevende anker

Staalbezwijken met hefboomsarm



$$V_{Ed} \leq V_{Rd,s,M} = \frac{V_{Rk,s,M}}{\gamma_{Ms}}$$

$$V_{Rk,s,M} = \frac{\alpha_M \cdot M_{Rk,s}}{l_a} = \frac{2 \cdot 324,00 Nm}{45 mm} = 14,40 kN$$

$$M_{Rk,s} = M^0_{Rk,s} \cdot (1 - N_{Ed} / N_{Rd,s}) = 324,00 Nm \cdot (1 - 0,00 kN / 82,00 kN) = 324,00 Nm$$

$V_{Rk,s,M}$ kN	γ_{Ms}	$V_{Rd,s,M}$ kN	V_{Ed} kN	$\beta_{V,s,M}$ %
14,40	1,25	11,52	25,00	217,0

UITNUTTINGSPERCENTAGE LIGT BOVEN

M20 8.8 direct

C-FIX
1.0.36.0
Database versie
1.0.36.0
Datum
07/02/2022

Ontwerp specificaties

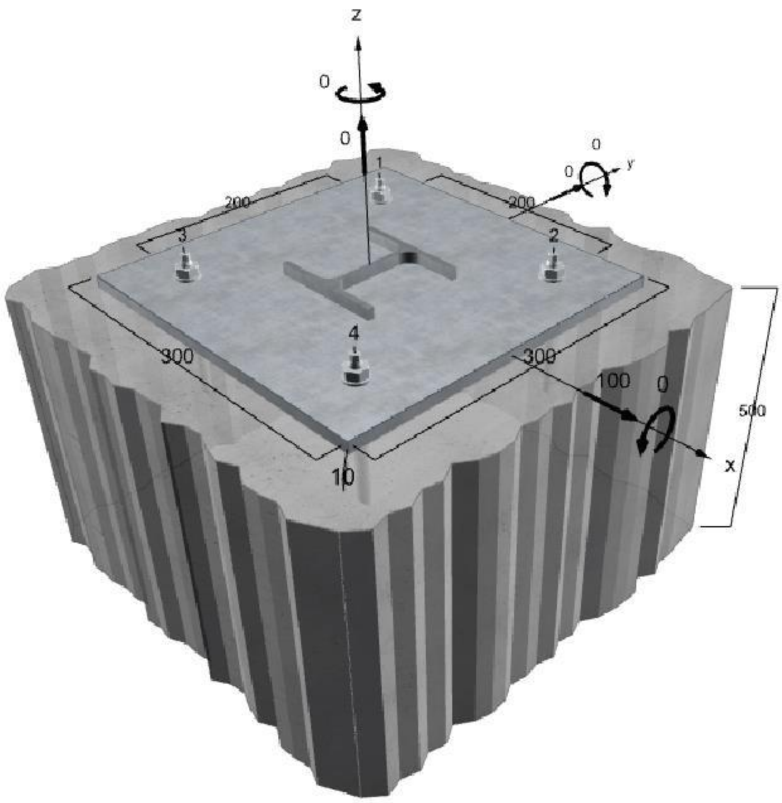
Anker

Anker systeem	fischer Superbond injectiesysteem FIS SB met ankerstang FIS A of RG M
Injectiemortel	FIS SB 390 S
Bevestigingselement	FIS A M 20 x 245 8.8
Verankeringsdiepte	90 mm
Ontwerp gegevens	ETA-12/0258

Geometrie / Belastingen / Eenheden

mm, kN, kNm

Rekenwaarde van de optredende krachten (inclusief veiligheidsfactoren)



Niet op schaal

Data

Ontwerp methode	EN 1992-4
Ondergrond	Normaal beton, C30/37
Betonsituatie	Ongescheurd, Droog boorgat
Temperatuur bereik	24 °C Lange duur temperatuur, 40 °C Korte duur temperatuur
Wapening	Geen of gewone wapening, Randwapening met beugels, With reinforcement to control splitting
Boormethode	Hamer boren
Installatie soort	Doorsteek montage

C-FIX
1.0.36.0
Database versie
1.0.36.0
Datum
07/02/2022

Tussenruimte	Gevuld
Soort belasting	Statisch of quasi-statisch
Ankerplaat locatie	Ankerplaat koud tegen ondergrond
Ankerplaat afmetingen	300 mm x 300 mm x 10 mm
Soort Profiel	HEAA

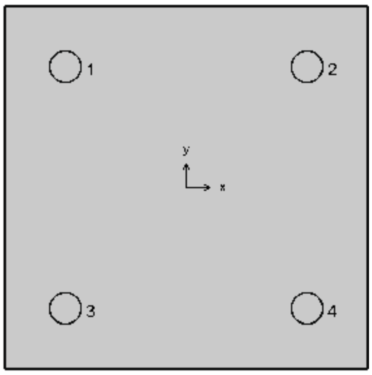
Rekenwaarde van de belastingen *)

#	N_{Ed} kN	$V_{Ed,x}$ kN	$V_{Ed,y}$ kN	$M_{Ed,x}$ kNm	$M_{Ed,y}$ kNm	$M_{T,Ed}$ kNm	Soort belasting
1	0,00	100,00	0,00	0,00	0,00	0,00	Statisch of quasi-statisch

*) Inclusief de benodigde veiligheidsfactoren

Resultante anker krachten

Anker nr.	Trek belasting kN	Afschuif belasting kN	Afschuif belasting x kN	Afschuif belasting y kN
1	0,00	25,00	25,00	0,00
2	0,00	25,00	25,00	0,00
3	0,00	25,00	25,00	0,00
4	0,00	25,00	25,00	0,00



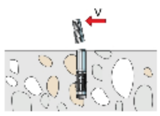
Max. betondruksterkte	0,00 ‰
Max. betondruksterkte	0,00 N/mm²
Resulterende trek krachten	0,00 kN, X/Y positie (0 / 0 mm)
Resulterende druk krachten	0,00 kN, X/Y positie (0 / 0 mm)

Weerstand bij afschuifbelastingen

Onderbouwing	Belasting kN	Capaciteit kN	Uitnutting β_V %
Staalbezwijken zonder hefboomsarm *	25,00	78,40	31,9
Beton achteruit breken	100,00	207,84	**48,1**

* Maatgevende anker

Staalbezwijken zonder hefboomsarm



$$V_{Ed} \leq V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{Ms}}$$

$$V_{Rk,s} = k_7 \cdot V^0_{Rk,s} = 1{,}00 \cdot 98{,}00\,kN = 98{,}00\,kN$$

$V_{Rk,s}$ kN	γ_{Ms}	$V_{Rd,s}$ kN	V_{Ed} kN	$\beta_{V,s}$ %
98,00	1,25	78,40	25,00	31,9

M20 8.8 15 mm adjustment space

C-FIX
1.0.36.0
Database versie
1.0.36.0
Datum
04/03/2022

Ontwerp specificaties

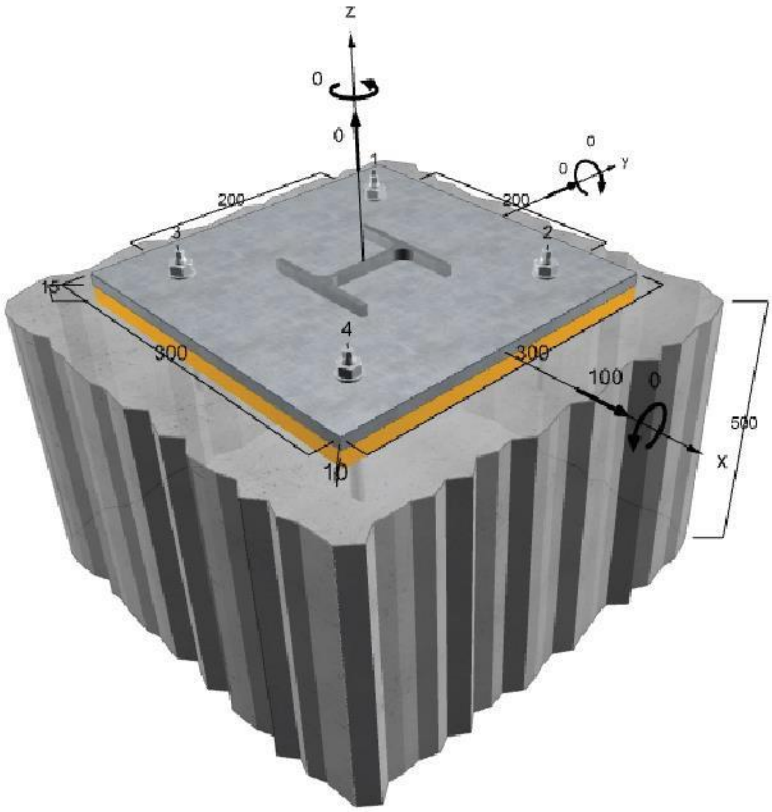
Anker

Anker systeem	fischer Superbond injectiesysteem FIS SB met ankerstang FIS A of RG M
Injectiemortel	FIS SB 390 S
Bevestigingselement	FIS A M 20 x 245 8.8
Verankeringsdiepte	90 mm
Ontwerp gegevens	ETA-12/0258

Geometrie / Belastingen / Eenheden

mm, kN, kNm

Rekenwaarde van de optredende krachten (inclusief veiligheidsfactoren)



Niet op schaal

Data

Ontwerp methode	EN 1992-4
Ondergrond	Normaal beton, C30/37
Betonsituatie	Ongescheurd, Droog boorgat
Temperatuur bereik	24 °C Lange duur temperatuur, 40 °C Korte duur temperatuur
Wapening	Geen of gewone wapening, Randwapening met beugels, With reinforcement to control splitting
Boormethode	Hamer boren
Installatie soort	Doorsteek montage

Tussenruimte	Gevuld
Soort belasting	Statisch of quasi-statisch
Ankerplaat locatie	Ankerplaat met vullaag
Ankerplaat afmetingen	300 mm x 300 mm x 10 mm
Soort Profiel	HEAA

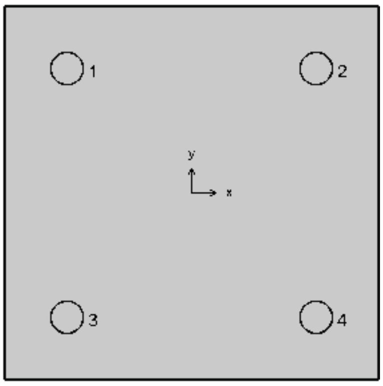
Rekenwaarde van de belastingen *)

#	N_{Ed} kN	$V_{Ed,x}$ kN	$V_{Ed,y}$ kN	$M_{Ed,x}$ kNm	$M_{Ed,y}$ kNm	$M_{T,Ed}$ kNm	Soort belasting
1	0,00	100,00	0,00	0,00	0,00	0,00	Statisch of quasi-statisch

*) Inclusief de benodigde veiligheidsfactoren

Resultante anker krachten

Anker nr.	Trek belasting kN	Afschuif belasting kN	Afschuif belasting x kN	Afschuif belasting y kN
1	0,00	25,00	25,00	0,00
2	0,00	25,00	25,00	0,00
3	0,00	25,00	25,00	0,00
4	0,00	25,00	25,00	0,00



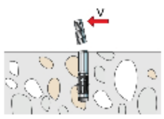
Max. betondruksterkte	0,00 ‰
Max. betondruksterkte	0,00 N/mm²
Resulterende trek krachten	0,00 kN, X/Y positie (0 / 0 mm)
Resulterende druk krachten	0,00 kN, X/Y positie (0 / 0 mm)

Weerstand bij afschuifbelastingen

Onderbouwing	Belasting kN	Capaciteit kN	Uitnutting β_V %
Staalbezwijken met hefboomsarm *	25,00	27,68	**90,3**
Beton achteruit breken	100,00	207,84	48,1

* Maatgevende anker

Staalbezwijken met hefboomsarm



$$V_{Ed} \leq V_{Rd,s,M} = \frac{V_{Rk,s,M}}{\gamma_{Ms}}$$

$$V_{Rk,s,M} = \frac{\alpha_M \cdot M_{Rk,s}}{l_a} = \frac{2 \cdot 519{,}00\,Nm}{30\,mm} = 34{,}60\,kN$$

$$M_{Rk,s} = M^0_{Rk,s} \cdot (1 - N_{Ed} / N_{Rd,s}) = 519{,}00\,Nm \cdot (1 - 0{,}00\,kN / 130{,}67\,kN) = 519{,}00\,Nm$$

$V_{Rk,s,M}$ kN	γ_{Ms}	$V_{Rd,s,M}$ kN	V_{Ed} kN	$\beta_{V,s,M}$ %
34,60	1,25	27,68	25,00	90,3

C-FIX
1.0.36.0
Database versie
1.0.36.0
Datum
04/03/2022

Ontwerp specificaties

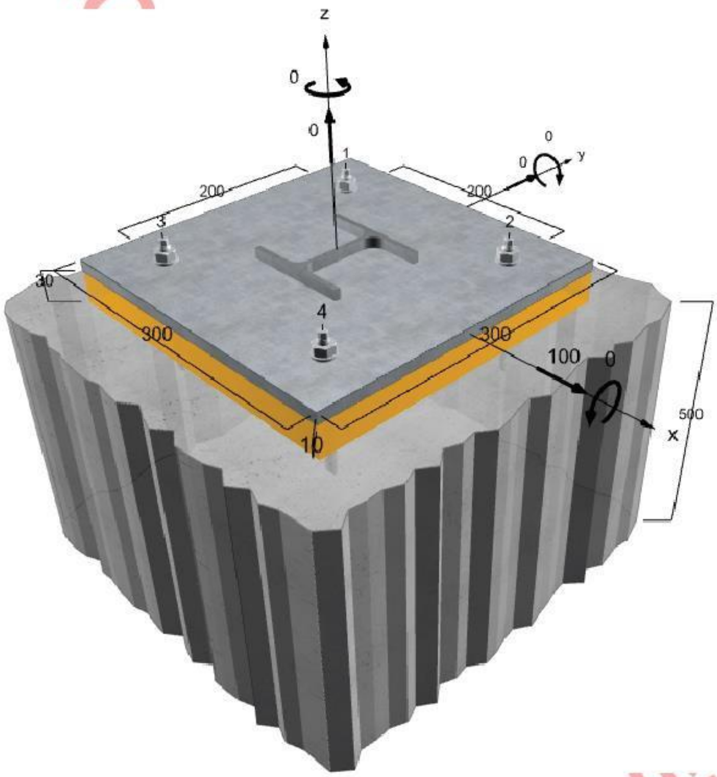
Anker

Anker systeem	fischer Superbond injectiesysteem FIS SB met ankerstang FIS A of RG M
Injectiemortel	FIS SB 390 S
Bevestigingselement	FIS A M 20 x 1000 8.8
Verankeringsdiepte	400 mm
Ontwerp gegevens	ETA-12/0258

Geometrie / Belastingen / Eenheden

mm, kN, kNm

Rekenwaarde van de optredende krachten (inclusief veiligheidsfactoren)



Niet op schaal

Data

Ontwerp methode	EN 1992-4
Ondergrond	Normaal beton, C30/37
Betonsituatie	Ongescheurd, Droog boorgat
Temperatuur bereik	24 °C Lange duur temperatuur, 40 °C Korte duur temperatuur
Wapening	Geen of gewone wapening, Randwapening met beugels, With reinforcement to control splitting
Boormethode	Hamer boren
Installatie soort	Doorsteek montage

Tussenruimte	Gevuld
Soort belasting	Statisch of quasi-statisch
Ankerplaat locatie	Ankerplaat met vullaag
Ankerplaat afmetingen	300 mm x 300 mm x 10 mm
Soort Profiel	HEAA

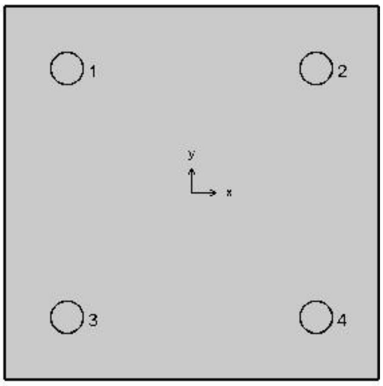
Rekenwaarde van de belastingen *)

#	N_{Ed} kN	$V_{Ed,x}$ kN	$V_{Ed,y}$ kN	$M_{Ed,x}$ kNm	$M_{Ed,y}$ kNm	$M_{T,Ed}$ kNm	Soort belasting
1	0,00	100,00	0,00	0,00	0,00	0,00	Statisch of quasi-statisch

*) Inclusief de benodigde veiligheidsfactoren

Resultante anker krachten

Anker nr.	Trek belasting kN	Afschuif belasting kN	Afschuif belasting x kN	Afschuif belasting y kN
1	0,00	25,00	25,00	0,00
2	0,00	25,00	25,00	0,00
3	0,00	25,00	25,00	0,00
4	0,00	25,00	25,00	0,00



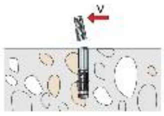
Max. betondruksterkte	0,00 ‰
Max. betondruksterkte	0,00 N/mm²
Resulterende trek krachten	0,00 kN, X/Y positie (0 / 0 mm)
Resulterende druk krachten	0,00 kN, X/Y positie (0 / 0 mm)

Weerstand bij afschuifbelastingen

Onderbouwing	Belasting kN	Capaciteit kN	Uitnutting β_V %
Staalbezwijken met hefboomsarm *	25,00	18,45	**135,5**
Beton achteruit breken	100,00	874,73	11,4

* Maatgevende anker

Staalbezwijken met hefboomsarm



$$V_{Ed} \leq V_{Rd,s,M} = \frac{V_{Rk,s,M}}{\gamma_{Ms}}$$

$$V_{Rk,s,M} = \frac{\alpha_M \cdot M_{Rk,s}}{l_a} = \frac{2 \cdot 519{,}00\,Nm}{45\,mm} = 23{,}07\,kN$$

$$M_{Rk,s} = M^0_{Rk,s} \cdot (1 - N_{Ed} / N_{Rd,s}) = 519{,}00\,Nm \cdot (1 - 0{,}00\,kN / 130{,}67\,kN) = 519{,}00\,Nm$$

$V_{Rk,s,M}$ kN	γ_{Ms}	$V_{Rd,s,M}$ kN	V_{Ed} kN	$\beta_{V,s,M}$ %
23,07	1,25	18,45	25,00	135,5

2.4. Halfen

M20 5.8 direct

Ing. bureau: *

Door: *

Projectnaam:

Functie:

Projectnummer: Datum: 4-3-2022

HALFEN
A CRH COMPANY

Invoerwaarde: **Bladzijde 1 / 3**

Beton:
- Ongescheurd (Drukzone)
- Drukvastheid: C30/37
- Langdurige/kortstondig temperatuur ≤ 50/80 °C

Wapening:
- dichtheid bewapening
- Met rand- en ophangwapening

Ankerbuiging:
- Het te bevestigende deel is niet gemaakt van metaal of met drukvaste tussenlaag e = 15 mm (drukvastheid ≥ 30N/mm²)
- Mate van inklemming $\alpha_M = 2{,}00$

Montage voorschriften:
- Gat geboord met boorhamer
- Droog boorgat

Statisch / Overwegend statisch Belasting

Trekbelasting:
- $N_{Sd} = 0{,}00$ kN

Afschuifkracht:
- $V_{x,Sd} = 0{,}00$ kN
- $V_{y,Sd} = 100$ kN

Moment:
- $M_{x,Sd} = 0{,}00$ kNm
- $M_{y,Sd} = 0{,}00$ kNm
- $M_{z,Sd} = 0{,}00$ kNm

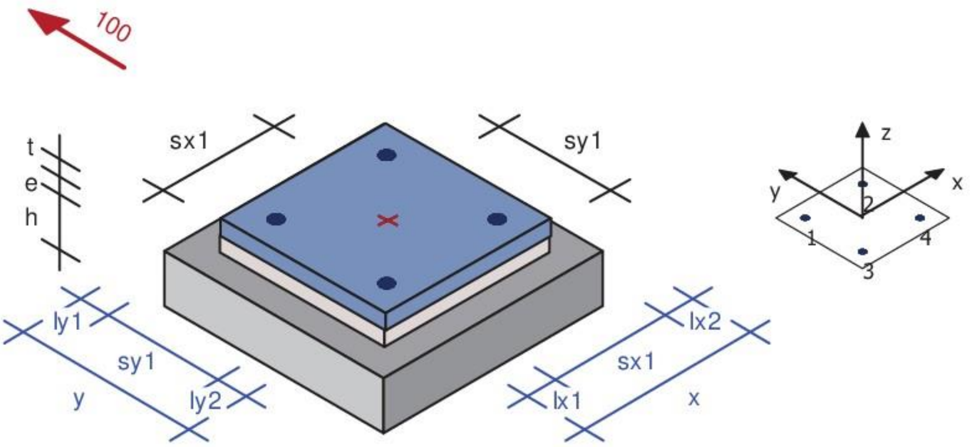
Excentrische belasting
- $e_x = 0{,}0$ mm
- $e_y = 0{,}0$ mm

Ankerplaat:
- x = 300 mm
- y = 300 mm
- l_{x1} = 50 mm
- l_{x2} = 50 mm
- l_{y1} = 50 mm
- l_{y2} = 50 mm
- t = 10 mm

Ankerafstand:
- s_{x1} = 200 mm
- s_{y1} = 200 mm

Randafstand:
- Zonder randafstand

Bouwdeeldikte:
- h = 500 mm



[kN, kNm]

Chemisch Anker HB-V-P + HB-V-A GV (5.8) M20
Ontwerp methode A, ETAG 001, Bijlage C
Goedkeuring ETA-07/0257

De verankering kan niet worden geverifieerd met de ingevoerde gegevens

	Trek β_N [%]	Afschuif β_V [%]	Interactie $\beta_{N,V}$ [%]	Rand voorwaarden:
Statisch / Overwegend statisch Belasting	- [1)]	144,2	- [1)]	OK

[1)] Controle niet noodzakelijk.

De berekening is van toepassing indien de gebruikshandleiding op de laatste pagina in acht worden genomen.

Chemisch Anker HB-V-P + HB-V-A GV (5.8) M20
Goedkeuring ETA-07/0257

Controle bij afschuifkrachten

Optredende belasting

Anker		**1**	**2**	**3**	**4**
V_{Sd}	[kN]	25,00	25,00	25,00	25,00
$V_{x,Sd}$	[kN]	0,00	0,00	0,00	0,00
$V_{y,Sd}$	[kN]	25,00	25,00	25,00	25,00

Staalbreuk zonder hefboomarm

V^h_{Sd}	≤	$V_{Rk,s}$	/	γ_{Ms}	=	$V_{Rd,s}$	Belasting:
25,00	≤	61,00	/	1,25	=	48,80	51,2%

Betonachteruitbreken

V^g_{Sd}	≤	$V_{Rk,cp}$	/	γ_{Mc}	=	$V_{Rd,cp}$	Belasting:
100,00	≤	378,37	/	1,50	=	252,25	39,6%

$N^o_{Rk,c}$	$\Psi_{C\ (C30/37)}$	$\Psi_{A,c,N}$	$\Psi_{s,N}$	$\Psi_{re,N}$	$\Psi_{ec,N}$	k
75,00 kN	1,00	2,52	1,00	1,00	1,00	2,00

$A_{c,N}$	$A^o_{c,N}$	$c_{cr,N}$	$e_{c1,N}$	$e_{c2,N}$
2916 cm^2	1156 cm^2	170,0 mm	0,0 mm	0,0 mm

Betonrandbreuk (ongunstigste rand)

Berekening voor het betonrandbreuk van het anker is niet nodig daar:
a) $c \geq 10h_{ef}$ en $c \geq 60d$
b) Geen dwarskracht richting betonrand.

h_{ef}	d
170 mm	25 mm

M20 8.8 15 mm adjustment space

Ing. bureau:	*
Door:	*
Projectnaam:	
Functie:	
Projectnummer:	

Datum: 4-3-2022

Invoerwaarde:

Beton:
Ongescheurd (Drukzone)
Drukvastheid: C30/37
Langdurige/kortstondig temperatuur ≤ 50/80 °C

Wapening:
dichtheid bewapening
Met rand- en ophangwapening

Ankerbuiging:
Het te bevestigende deel is niet gemaakt van metaal of
met drukvaste tussenlaag e = 15 mm
(drukvastheid ≥ 30N/mm²)
Mate van inklemming $\alpha_M = 2{,}00$

Montage voorschriften:
Gat geboord met boorhamer
Droog boorgat

Statisch / Overwegend statisch Belasting

Trekbelasting:
$N_{Sd} = 0{,}00$ kN

Afschuifkracht:
$V_{x,Sd} = 0{,}00$ kN
$V_{y,Sd} = 100$ kN

Moment:
$M_{x,Sd} = 0{,}00$ kNm
$M_{y,Sd} = 0{,}00$ kNm
$M_{z,Sd} = 0{,}00$ kNm

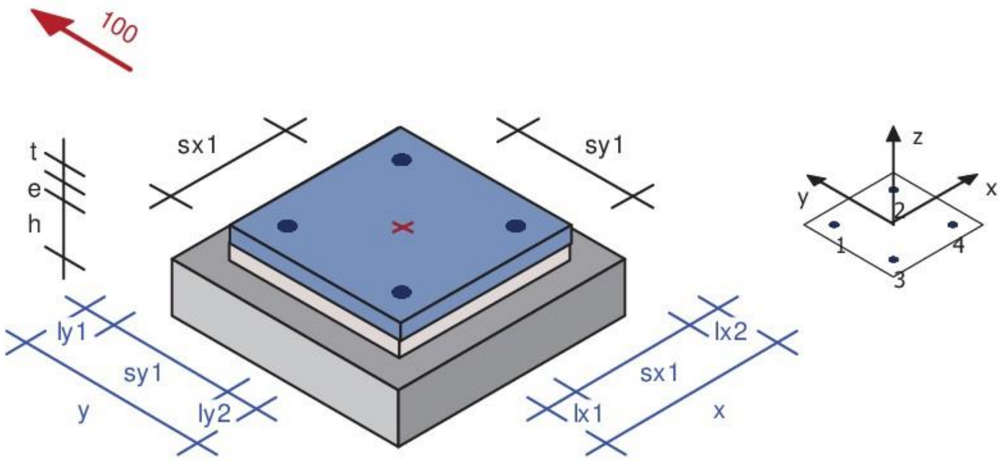
Excentrische belasting
$e_x = 0{,}0$ mm
$e_y = 0{,}0$ mm

Ankerplaat:
x = 300 mm
y = 300 mm
l_{x1} = 50 mm
l_{x2} = 50 mm
l_{y1} = 50 mm
l_{y2} = 50 mm
t = 10 mm

Ankerafstand:
s_{x1} = 200 mm
s_{y1} = 200 mm

Randafstand:
Zonder randafstand

Bouwdeeldikte:
h = 500 mm



[kN, kNm]

Chemisch Anker HB-V-P + HB-V-A GV (5.8) M20
Ontwerp methode A, ETAG 001, Bijlage C
Goedkeuring ETA-07/0257

De verankering kan niet worden geverifieerd met de ingevoerde gegevens

	Trek β_N [%]	Afschuif β_V [%]	Interactie $\beta_{N,V}$ [%]	Rand voorwaarden:
Statisch / Overwegend statisch Belasting	- [1)]	144,2	- [1)]	OK

[1)] Controle niet noodzakelijk.

De berekening is van toepassing indien de gebruikshandleiding op de laatste pagina in acht worden genomen.

Chemisch Anker HB-V-P + HB-V-A GV (5.8) M20
Goedkeuring ETA-07/0257

Controle bij afschuifkrachten

Optredende belasting

Anker		**1**	**2**	**3**	**4**
V_{Sd}	[kN]	25,00	25,00	25,00	25,00
$V_{x,Sd}$	[kN]	0,00	0,00	0,00	0,00
$V_{y,Sd}$	[kN]	25,00	25,00	25,00	25,00

Staalbreuk zonder hefboomarm

V^h_{Sd}	≤	$V_{Rk,s}$	/	γ_{Ms}	=	$V_{Rd,s}$		Belasting:
25,00	≤	61,00	/	1,25	=	48,80		51,2%

Staalbreuk met hefboomarm

V^h_{Sd}	≤	$V_{Rk,s}$	/	γ_{Ms}	=	$V_{Rd,s}$		Belasting:
25,00	>	21,67	/	1,25	=	17,33		144,2%

$1-\beta_{Ns}$	$M^0_{Rk,s}$	$M_{Rk,s}$	l	α_M
1,00	325,00 Nm	325,00 Nm	30,0 mm	2,0

Betonachteruitbreken

V^g_{Sd}	≤	$V_{Rk,cp}$	/	γ_{Mc}	=	$V_{Rd,cp}$		Belasting:
100,00	≤	378,37	/	1,50	=	252,25		39,6%

$N^o_{Rk,c}$	$\Psi_{C\ (C30/37)}$	$\Psi_{A,c,N}$	$\Psi_{s,N}$	$\Psi_{re,N}$	$\Psi_{ec,N}$	k
75,00 kN	1,00	2,52	1,00	1,00	1,00	2,00
$A_{c,N}$	$A^o_{c,N}$	$c_{cr,N}$	$e_{c1,N}$	$e_{c2,N}$		
2916 cm^2	1156 cm^2	170,0 mm	0,0 mm	0,0 mm		

Betonrandbreuk (ongunstigste rand)

Berekening voor het betonrandbreuk van het anker is niet nodig daar:
a) $c \geq 10h_{ef}$ en $c \geq 60d$
b) Geen dwarskracht richting betonrand.

h_{ef}	d
170 mm	25 mm