



# Advanced Base Plate Technical Background

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## Contents

1	Introduction.....	3
1.1	Component-based Finite Element Method.....	4
1.2	Rigid base plates: problems when designing base plates today .....	5
1.3	Advance base plate design in PROFIS Engineering.....	8
1.3.1	Rigid .....	8
1.3.2	Flexible.....	10
2	Flexible design: CBFEM components.....	12
2.1	Material model for steel.....	12
2.1.1	Plate model.....	13
2.1.2	Mesh convergence.....	14
2.2	Anchors .....	15
2.2.1	Anchors with stand-off .....	16
2.3	Concrete block.....	18
2.3.1	Design model .....	18
2.3.2	Deformation stiffness .....	18
2.4	Welds .....	18
2.4.1	Direct connection of plates .....	19
2.4.2	Weld with plastic redistribution of stress.....	19
2.4.3	Steel material and lamellar tearing .....	19
2.5	Point of applied shear loads .....	20
2.6	National Annexes of the Eurocode considered in design.....	21
2.7	CBFEM application in case of seismic design.....	22
3	CBFEM design: Verification of components according to Eurocode.....	24
3.1	Anchor verifications.....	24
3.1.1	Static load combinations .....	24
3.1.2	Seismic load combinations .....	25
3.2	Base plate .....	27
3.3	Profile.....	29
3.4	Stiffeners.....	30



3.5 Concrete block.....	30
3.5.1 Mesh sensitivity.....	32
3.6 Welds .....	33
3.6.1 No weld.....	33
3.6.2 Fillet welds.....	33
3.6.3 Butt welds.....	34
3.7 Deformations.....	35
3.8 Detailing.....	36
4 Advanced Base Plate design gives the flexibility to choose the right needed solution .....	37
4.1 Anchor design only. Go rigid .....	37
4.2 Full connection design. Go flexible including anchors, baseplate, weld, stiffeners and concrete	37
5 Proposed solution for assessing if the base plate can be considered “close to rigid” by engineering judgement .....	38
5.1 Engineering judgement to define “Close to rigid” base plates behavior.....	38
5.1.1 Anchor forces.....	40
5.1.2 Concrete stresses.....	41
5.1.3 Deformation .....	41
5.2 Hilti recommendation for engineering judgement.....	41
6 Conclusion .....	42
Legal disclaimers.....	43
Figures .....	45
Table.....	47
References.....	48



## 1 Introduction

Every structure needs to be connected to the ground via a foundation. These foundations are in most cases made from concrete. To transfer the loads acting on the structure a connection of the structural steel components (beams, columns, ...) to concrete foundations is required. One of the most popular solutions is fastening the structural steel component welded to a base plate with anchors to the foundation. All relevant elements of this connection need to be designed for the acting loads.

Bar members are preferred by engineers when designing steel structures. However, there are many locations on the structure where the theory of members is not valid, e.g., base plate, anchors connections, welded joints. The structural analysis in such locations requires special attention. The behavior is non-linear and the nonlinearities must be respected, e.g., yielding of the material of plates or profiles, base plate and concrete block, one-sided actions of anchors, welds.

The base plate and anchors needs to be designed at the intersection between steel design, anchor design and concrete design guidelines. The design assumptions for the full connection must not contradict each other. Main example for steel to concrete connection is the assumed behavior of the base plate. Design codes, e.g. EN 1993-1-8 [1], and technical literature offer engineering solution methods. Their general feature is derivation for typical structural shapes and simple loadings. The approach is based on the component method.

Over the last years finite element (FEM) based design revolutionized structural engineering. Nowadays most engineers have access to powerful FEM software packages. Since the term “rigid base plate” can mean everything and nothing there are a lot of design engineers modeling their base plates, concrete and anchors in FEM solutions (sometimes even with a non-linear FEM software utilizing the plastic design according to Eurocode 3). Most likely they are not aware that the anchor design guidelines are based on a rigid base plate assumption.

This document is meant to give guidance and additional details on the Advanced Base Plate solution in Hilti’s PROFIS Engineering software.

## 1.1 Component-based Finite Element Method

Reality behavior of steel to concrete connections cannot be solved by simple beam equations. Component Method (CM) solves the connection as a system of interconnected items – components. The corresponding model is built per each connection type to be able to determine forces and stresses in each component – see the following picture.

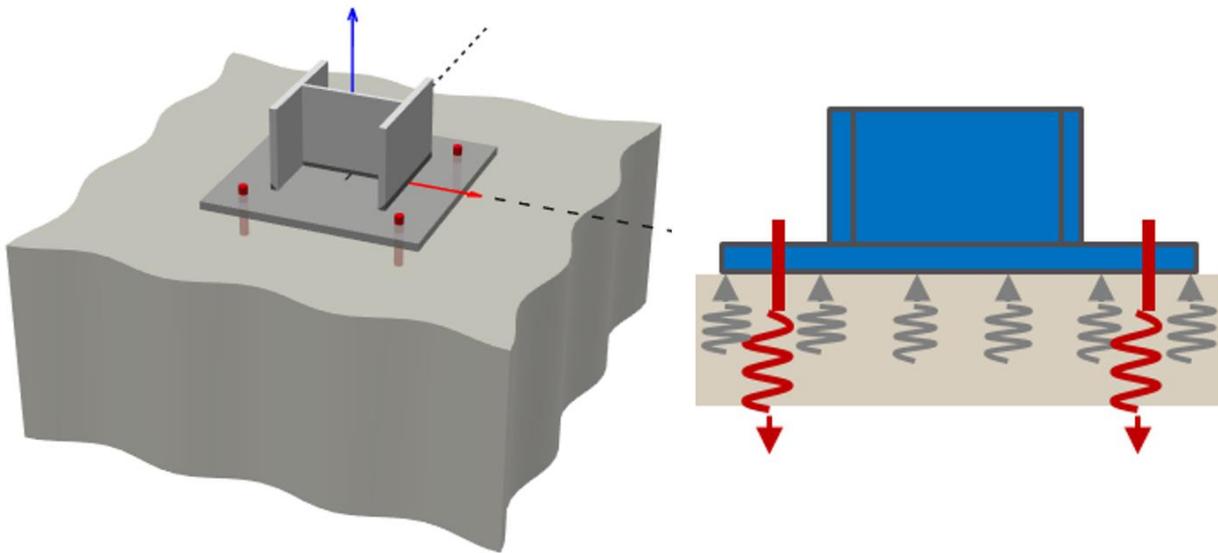


Figure 1. Anchors modeled by springs in steel-to-concrete connection

Concrete is modelled with compression spring, anchor is modelled with tension spring, and steel is defined with shell elements. The mechanical properties of the individual components are defined by EN 1992 for concrete, EN 1993 for steel, and Hilti Technical Data based on laboratory tests for anchors.

Each component's resistance is checked separately using corresponding equations from the code (more information given later in this document).

The method used by *PROFIS Engineering* to simulate reality behavior of base plate is the Component-based Finite Element Method (CBFEM) which is:

- General enough to be usable for most of connections in engineering practice.
- Simple and fast enough in daily practice to provide results in a time comparable to current methods and tools.
- Comprehensive enough to provide structural engineer clear information about connections behavior, stress, strain and reserves of individual components and about overall safety and reliability.

The CBFEM is based on the idea that most of the verified and very useful parts of CM should be kept. The weak point of CM – its generality when analyzing stresses of individual components – was replaced by modeling and analysis using Finite Element Method (FEM). The connection is divided into main components: profile, stiffeners, welds, plate, concrete and anchors.

This method has been proved by a verification and validation process of benchmark cases, both numerical and experimental, source [2].

## 1.2 Rigid base plates: problems when designing base plates today

Rigid base plate assumptions from ETAG/EN/ACI guidelines are usually not top of mind for engineers – however anchor guidelines are explicitly valid for rigid base plates only.

There is no clear definition available when a base plate can be considered rigid.

The current design software gives a solution to the load distribution on the anchors, but the assumptions behind these calculations are not transparent at all, giving a black box feeling.

The main influencing effects of non-rigid base plate:

- Reduction of inner lever arm. In case the base plate cannot be considered rigid the inner lever arm between resulting tension and resulting compression decreases. Limiting case to consider is a very thin plate where the center of compression will be underneath the compressed flange of the I-profile. A reduction in inner lever arm will lead to an increase in anchor forces.

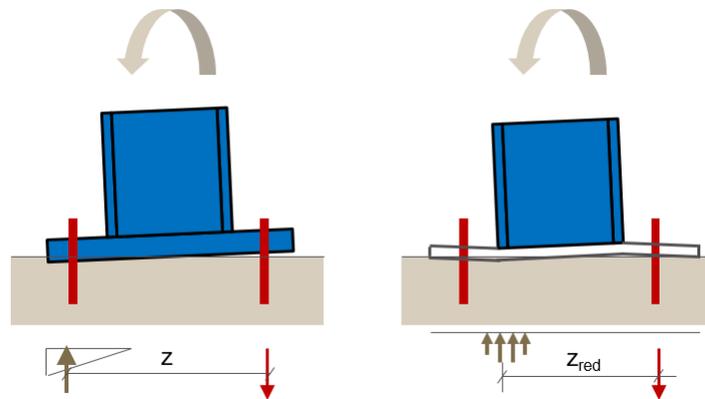


Figure 2. Reduction of inner lever arm for non-rigid base plates

- Prying effects. For non-rigid base plates with certain geometries prying forces can be observed. These forces will increase the anchor forces coming from the acting loading (tension or bending moments).

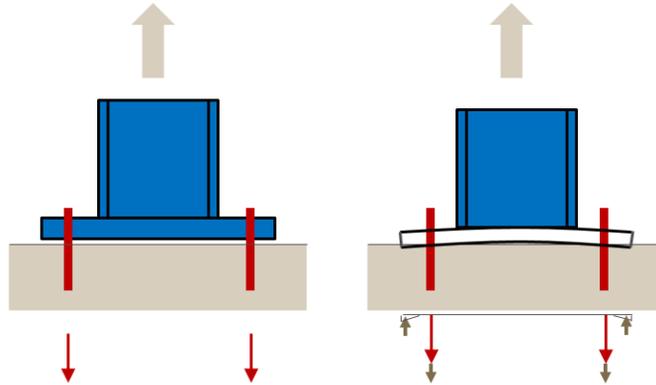


Figure 3. Increase of anchor forces due to prying effects

- Different load distribution in anchor groups. In case of different distances of fasteners to the profile and non-rigid base plates the load distribution between the single anchors will be different, e.g. in a 3x3 anchor arrangement the center anchor will get much more load than the outer ones in case the base plate is non-rigid.

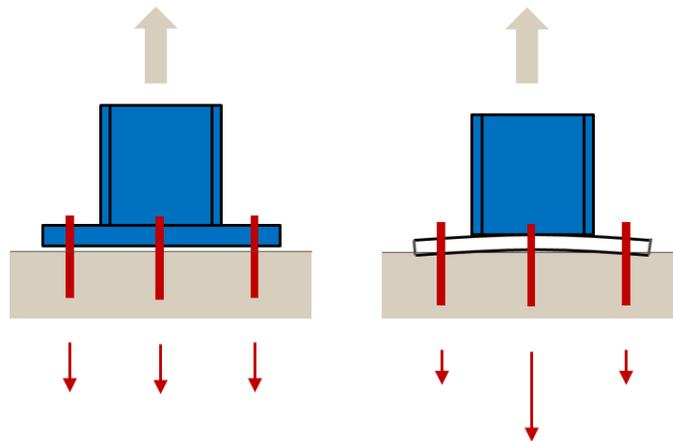


Figure 4. Increase of anchor forces due to different load distribution for non-rigid base plates

- Different concrete stress distribution. In case of non-rigid base plate, the compression stress will be concentrated underneath the profile. This will lead to higher concrete stresses.

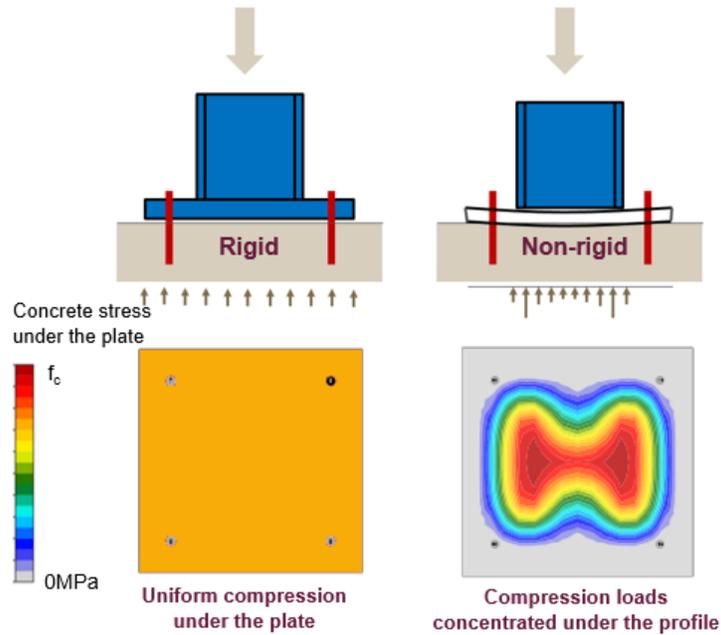


Figure 5. Different concrete stress distribution

- SLS considerations. For cantilever beams a non-rigid base plate will create more displacement because there is more rotation in the base plate.

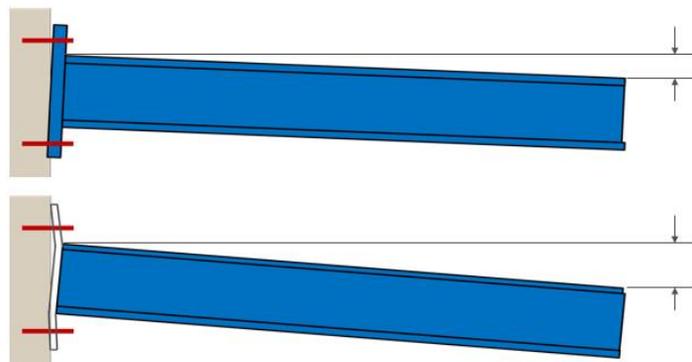


Figure 6. Displacement of cantilever beam in case of rigid and non-rigid base plate

Depending on the loading and the geometry, one or more of these effects will apply and will change the anchor forces in the connection. See the next example which compares rigid and flexible base plate.

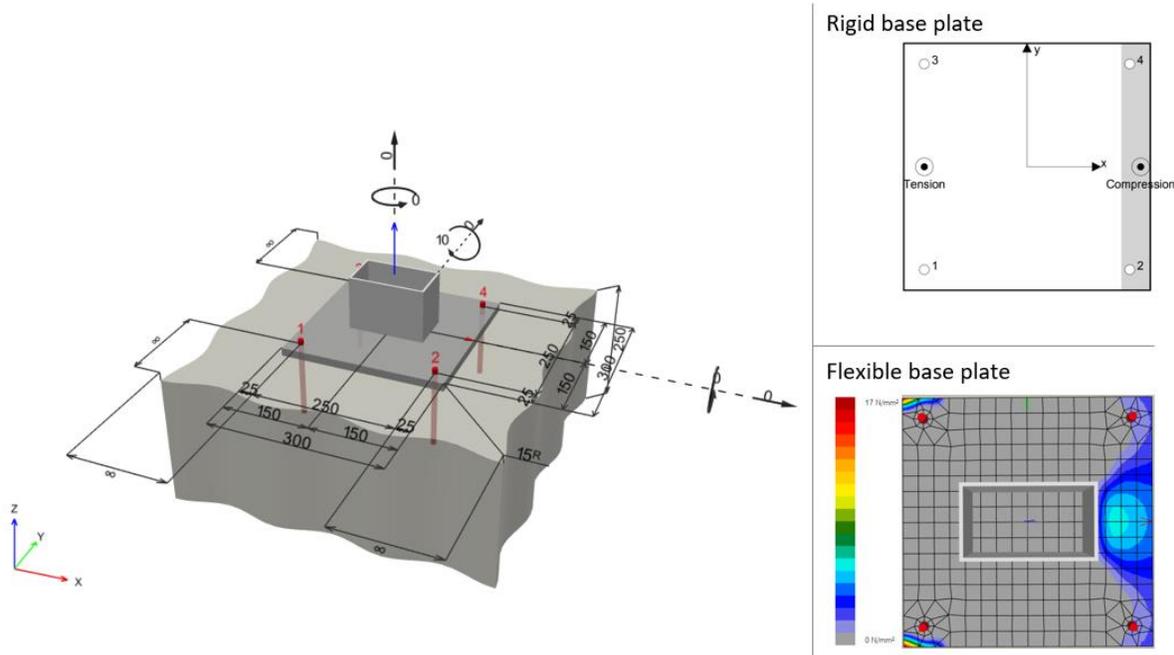


Figure 7. Example comparing rigid vs flexible

In this example the reduction of the lever arm and prying forces lead to higher anchor forces. This is not a theoretical approach – also in physical tests this behavior is being observed.

### 1.3 Advance base plate design in PROFIS Engineering

After observing the lack of detail in the steel-to-concrete connections, alternative methods have been developed with more accurate results and closer to reality. In the next figure, it is shown the different methods to calculate.



Figure 8. Base plate design options for determination of the load distribution to the anchors

#### 1.3.1 Rigid

PROFIS Anchor and PROFIS Engineering with selection of rigid derive the anchor forces acting on individual fasteners from an applied load. The assumptions for the rigid design options based on the current anchor design guidelines ( [3], [4], [5], [6], [7] and [8]) are:

- No deformation of the plate (plane surfaces remain plane).
- Strains are distributed linearly through the cross-section of the baseplate (corresponding to the “Bernoulli Hypothesis” in reinforced concrete).
- Relevant mechanical properties for design are fastener cross-section ( $A_s$ ) and fastener modulus of elasticity ( $E_s$ ).
- Stiffness of the concrete is characterized by its modulus of elasticity.

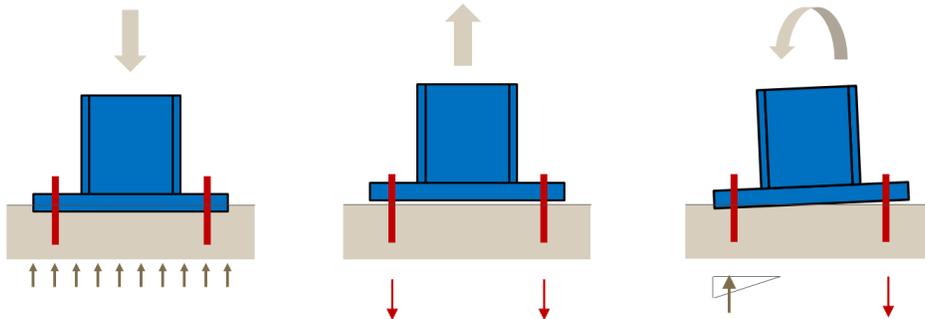


Figure 9. Rigid base plate behavior

In the first step (1 in Figure 10 below), the rigid base plate method calculates the resultant anchor forces and concrete stresses, based on the rigid assumptions. Then (step 2 in Figure 10 below), it turns them into loading vectors and apply these to the base plate, to determine the plate moments. From the plate moments the thickness is being calculated using the yield strength of the base plate (step 3 in Figure 10 below)

At the end, the user is responsible to check if the assumption of a rigid base plate was met (step 4 in Figure 10 below). Unfortunately, this step is not done properly all the time.

## FIRST STEP OF THE BASEPLATE DESIGN IS DONE WITH THE ASSUMPTION OF A RIGID BASEPLATE

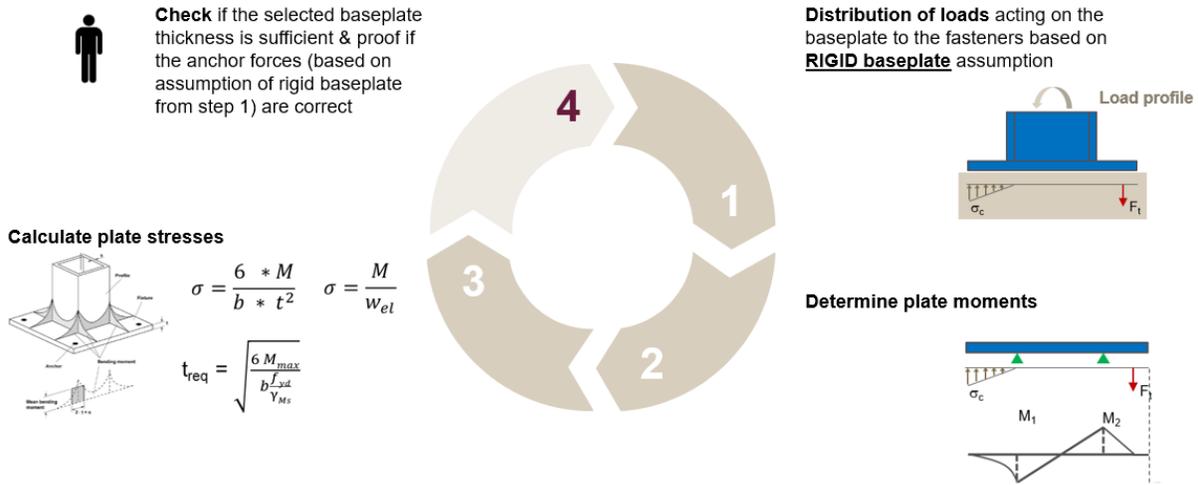


Figure 10. Steps taken in the rigid design

### 1.3.2 Flexible

The real behavior of base plates may be rigid or non-rigid. Although, the anchor codes require the base plate to be rigid.

Illustrated below are two limit examples of a plate with no deformations (rigid), and a case with deformations (non-rigid). However, there is currently no clear definition of a rigid base plate.

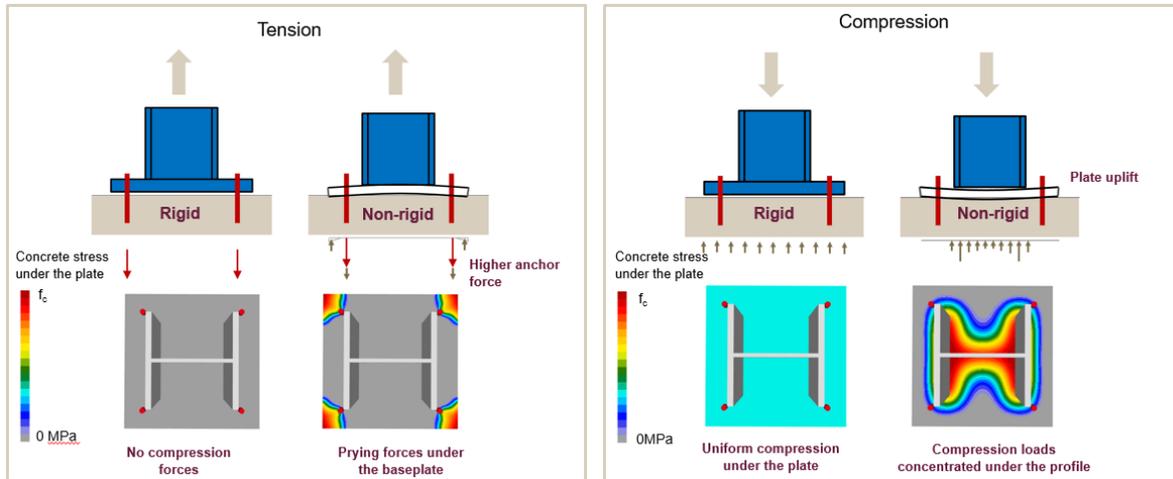


Figure 11. Two examples comparing non-rigid base plate

In a real behavior of a base plate, all the components geometry and mechanical properties influence the load distribution (profile, welds, stiffeners, plate, anchors and concrete).

Then Flexible design based in component method, according to component method considers the full connection design is explained in the flowing chapters.

Also, *PROFIS* helps users solve the rigid base plate, by checking how close to a rigid situation their design is.

## 2 Flexible design: CBFEM components

FEM is a general method commonly used for structural analysis. Usage of FEM for modeling of connections of any shapes seems to be ideal (Virdi 1999 [9]).

An elastic-plastic analysis is required, as the steel ordinarily yields in the structure. In fact, the results of the linear analysis are useless for connection design when high loads are applied.

FEM models are used for research purposes of connection behavior, which usually apply spatial elements and measured values of material properties.

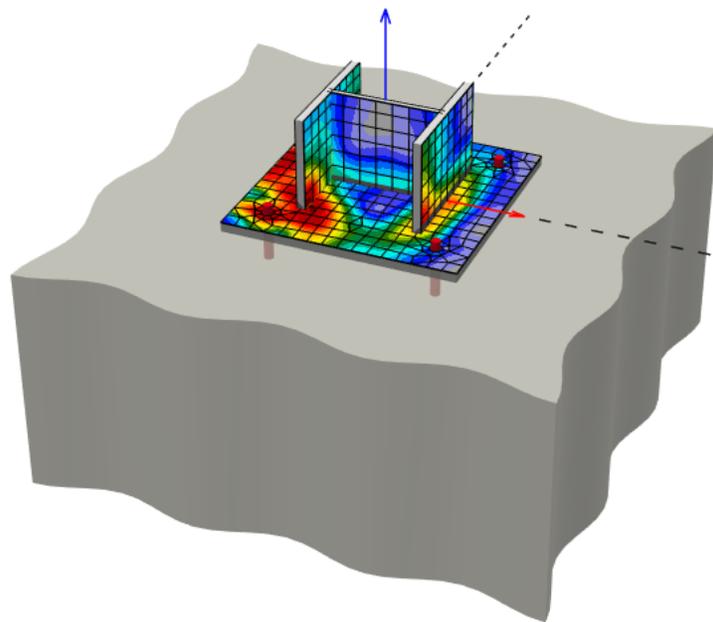


Figure 12. FEM model of a connection for research. It uses spatial 3D elements for both plates and anchors

The fasteners – anchors and welds – are the most difficult in the point of view of the analysis model. Modeling of such elements in general FEM programs is difficult because the programs do not offer required properties. Thus, special FEM components had to be developed to model the welds and anchors behavior in the connection.

In the case of connections, the geometrically nonlinear analysis is not necessary unless plates are very slender. Plate slenderness can be determined by eigenvalue (buckling) analysis. The geometrically nonlinear analysis is not implemented in the software.

### 2.1 Material model for steel

The most common material diagrams which are used in finite element modeling of structural steel are the ideal plastic or elastic model with strain hardening and the true stress-strain diagram.

The plates in Hilti *PROFIS Engineering* are modeled with elastic-plastic material with a nominal yielding plateau slope according to EN 1993-1-5 [10], App C, Part C.6. The material behavior is based on von Mises yield criterion. It is assumed to be elastic before reaching the yield strength,  $f_y$ .

The ultimate limit state criterion for regions not susceptible to buckling is reaching the limiting value of the principal membrane strain. The value of 5% is recommended EN 1993-1-5 [10], App C, Part C.8 Note 1.

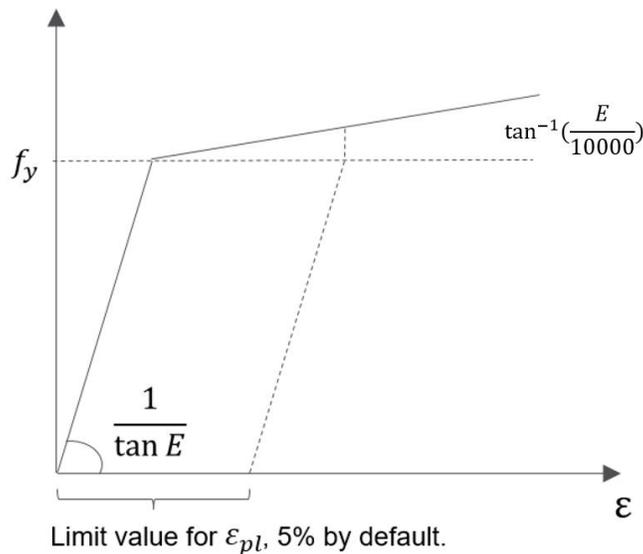


Figure 13. Material diagrams of steel in numerical models

The limit value of plastic strain is often discussed. In fact, the ultimate load has low sensitivity to the limit value of plastic strain when the ideal plastic model is used, [11].

### 2.1.1 Plate model

Shell elements are recommended for modeling of plates in FEA of structural connection. Four-node quadrangle shell elements with nodes at its corners are applied. Six degrees of freedom are considered in each node: 3 translations ( $u_x, u_y, u_z$ ) and 3 rotations ( $\varphi_x, \varphi_y, \varphi_z$ ).

Rotations perpendicular to the plane of the element are considered. Complete 3D formulation of the element is provided. The out-of-plane shear deformations are considered in the formulation of the flexural behavior of an element based on Mindlin hypothesis. The MITC4 elements are applied, see Dvorkin (1984) [12]. The shell is divided into five integration layers through thickness of the plate at each integration point and plastic behavior is analyzed in each point. It is called Gauss-Lobatto integration. The nonlinear elastic-plastic stage of material is analyzed in each layer based on the known strains.

## 2.1.2 Mesh convergence

There are some criteria for the mesh generation in the connection model. The sensitivity analysis considering mesh discretization should be performed by the user for complicated geometries. In general, larger mesh size leads to faster calculation time but less accurate results, smaller mesh size lead to slower calculation but higher accuracy in results.

All plates of a steel-to-concrete connection have a common division into elements. The size of generated finite elements is limited. The minimal element size is set to 10 mm and the maximal element size to 50 mm (can be changed by the user in advanced settings). Meshes on flanges and webs are independent of each other. The default number of finite elements is set to 8 elements to the large profile dimension, as shown in the following figure. The user can modify the default values in advanced settings, [11].

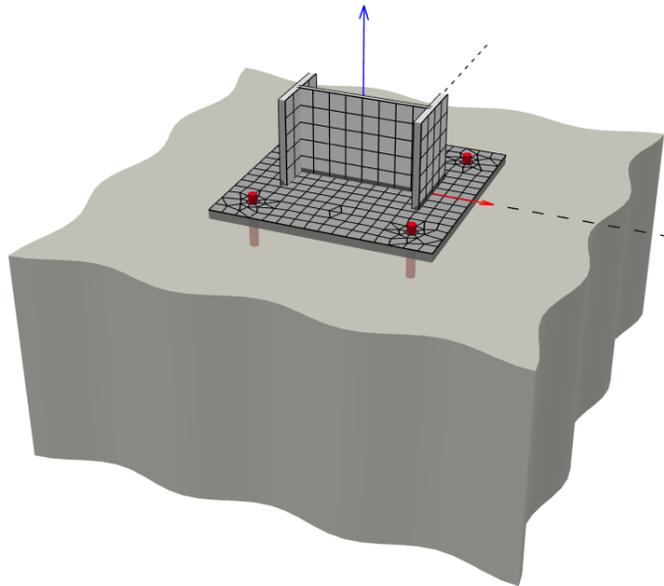


Figure 14. The mesh on a column and baseplate with constraints between the web and the flange

The relation between the profile mesh and the plate mesh is defined as follows:

$$\text{mesh in largest side of base plate} = 2 \cdot \text{number of profile mesh elements}$$

Then, default finite element size is set to 16 elements as shown in the figure.

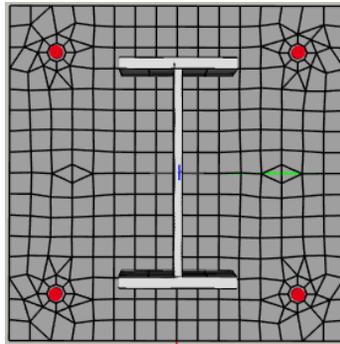


Figure 15. The mesh on a baseplate with 16 elements along its width

The following example of a concrete-to-base plate connection shows the influence of mesh size on the base plate stress. It is loaded by a bending moment as shown in the following figure, the number of the finite elements along the cross-section height varies from 8 to 40 (leading to a division of the larger plate size by a factor from 16 to 80) and the results are compared. It is recommended to subdivide the base plate length into 16 elements (highlighted in red in the next chart) as smaller elements only slow down the calculation but don't improve accuracy. It is the responsibility of the user to define the mesh size for the application at hand.

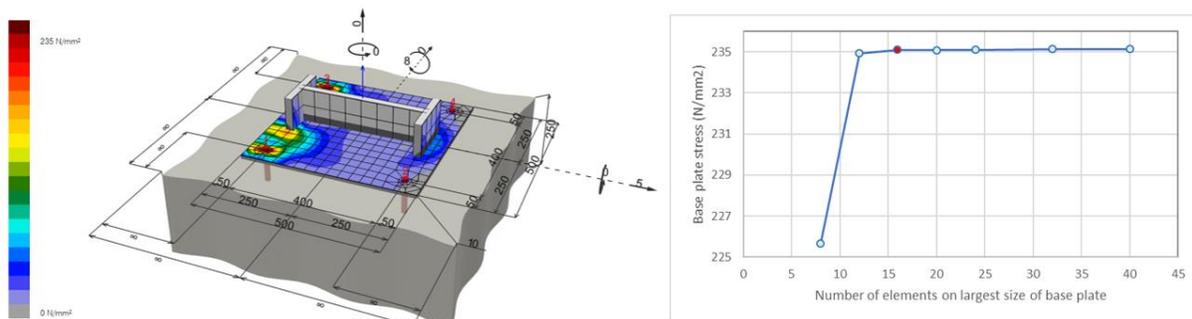


Figure 16. Influence of mesh size of base plate stress

## 2.2 Anchors

Anchor material properties are based on experimental Hilti research for the product assessments. The anchor stiffness is a product specific characteristic which differs depending on the selected product, loading conditions, diameter, material and embedment depth.

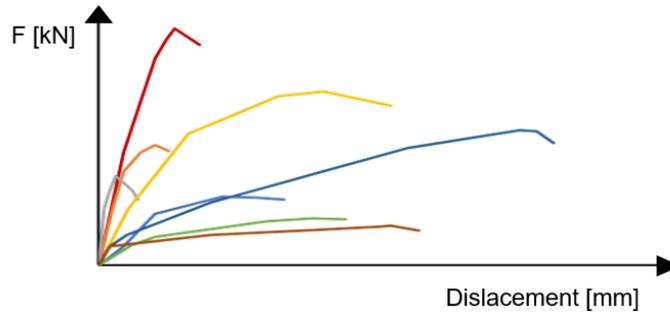


Figure 17. Schematic load displacement

### 2.2.1 Anchors with stand-off

The CBFEM model described in this document is suitable for fastenings with a base plate on concrete and with stand-off in case of grouting under the plate. The grout should have at least the same resistance as the concrete base material.

It is assumed that the grout can resist the compressive stress while tensile stress is transmitted to the anchors. The internal load distribution is determined by finite element model.

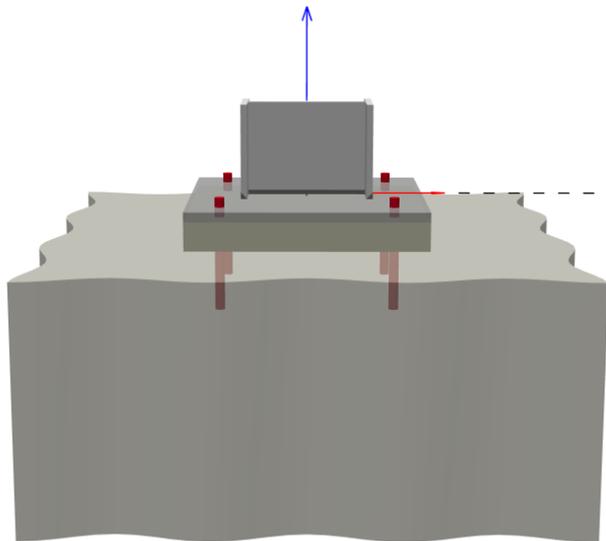


Figure 18. Anchors with stand-off with grouting

PROFIS users must select, in this case, standoff with grout.

If the compressive strength of the grout is smaller than the concrete under it, then PROFIS conservatively assumes that the grout cannot transmit compression forces into the concrete.

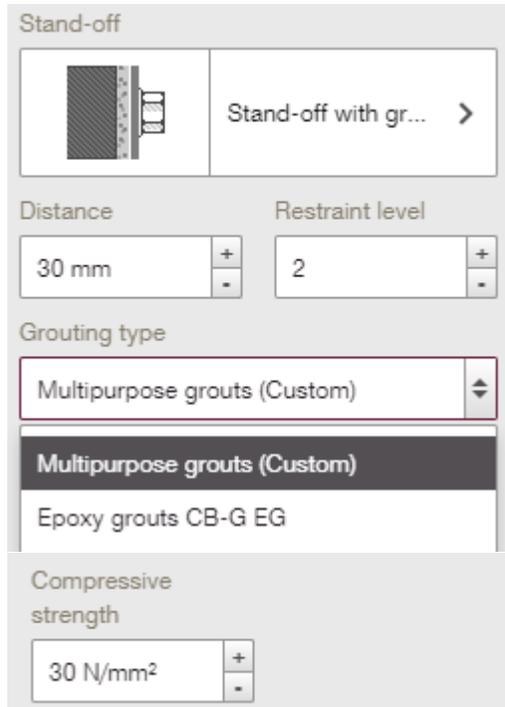


Figure 19: Selection of standoff with grout

The anchor resistance verifications produced by PROFIS Engineering consider increase of shear load due to the lever arm of the shear load, as per the calculation example below.

**1.4.1 Steel failure (with lever arm)**

$$V_{Sd} \leq V_{Rd,s}^M = \frac{V_{Rk,s}^M}{\gamma_{M,s,b}} \quad \text{ETAG 001 Annex C, Table 5.2.3.1}$$

$$V_{Rk,s}^M = \frac{\alpha_M \cdot M_{Rk,s}}{l} \quad \text{ETAG 001 Annex C, Eq. (5.5)}$$

$$M_{Rk,s} = M_{Rk,s}^0 \cdot \left(1 - \frac{N_{Sd}}{N_{Rd,s}}\right) \quad \text{ETAG 001 Annex C, Eq. (5.5a)}$$

$$l = e_c + \frac{t}{2} + a_3 \quad \text{ETAG 001 Annex C, Eq. (4.2)}$$

$l$ [mm]	$\alpha_M$			
40.0	2.00			
$N_{Sd} / N_{Rd,s}$	$1 - N_{Sd} / N_{Rd,s}$	$M_{Rk,s}^0$ [kNm]	$M_{Rk,s} = M_{Rk,s}^0 \cdot (1 - N_{Sd} / N_{Rd,s})$ [kNm]	
0.076	0.924	0.092	0.085	
$V_{Rk,s}^M = \alpha_M \cdot M_{Rk,s} / l$ [kN]	$\gamma_{M,s,b}$	$V_{Rd,s}^M$ [kN]	$V_{Sd}$ [kN]	
4.251	1.500	2.834	1.256	

Figure 20: Example of steel shear resistance of Hilti Anchor with lever arm

## 2.3 Concrete block

### 2.3.1 Design model

In CBFEM, it is convenient to simplify the concrete block as 2D contact elements. The connection between the concrete and the base plate resists in compression only. Compression is transferred via Winkler-Pasternak subsoil model which represents deformations of the concrete block. The tension force between the base plate and concrete block is carried by the anchor bolts, [11].

### 2.3.2 Deformation stiffness

The stiffness of the concrete block may be predicted for the design of column bases as an elastic hemisphere. A Winkler-Pasternak subsoil model is commonly used for a simplified calculation of foundations. The stiffness of subsoil is determined using modulus of elasticity of concrete and the effective height of a subsoil as, [11]:

$$k = \frac{E_c}{(a_1 + \nu) \sqrt{\frac{A_{eff}}{A_{ref}}}} \left( \frac{1}{\frac{h}{a_2} + a_3} + a_4 \right)$$

where:

- $k$  – stiffness of concrete subsoil in compression
- $E_c$  – modulus of elasticity of concrete
- $\nu$  – Poisson's coefficient of the concrete block
- $A_{eff}$  – effective area in compression
- $A_{ref} = 10 \text{ m}^2$  – reference area
- $d$  – smaller side of rectangular base plate
- $a_1 = 1.65; a_2 = 0.5; a_3 = 0.3; a_4 = 1.0$  – coefficients

SI units must be used in the formula, the resulting unit is  $\text{N/m}^3$ .

## 2.4 Welds

Several options how to treat welds in numerical models exist. It is possible to use different mesh descriptions, different kinetic and kinematic variables and constitutive models. The different types of geometric 2D and 3D models and thereby finite elements with their applicability for different accuracy levels are generally used. Most often used material model is the common rate-independent plasticity model based on von Mises yield criterion. Two approaches which are used for welds are described, [11].

## 2.4.1 Direct connection of plates

This first option of weld model between plates is rigid connection by links between meshes of connected plates. The connection is called multi point constraint (MPC) and relates the finite element nodes of one plate edge to another plate. The finite element nodes are not connected directly. The advantage of this approach is the ability to connect meshes with different densities. The constraint allows to model midline surface of the connected plates with the offset, which respects the real plate thickness. This type of connection is used for full penetration butt welds, [11].

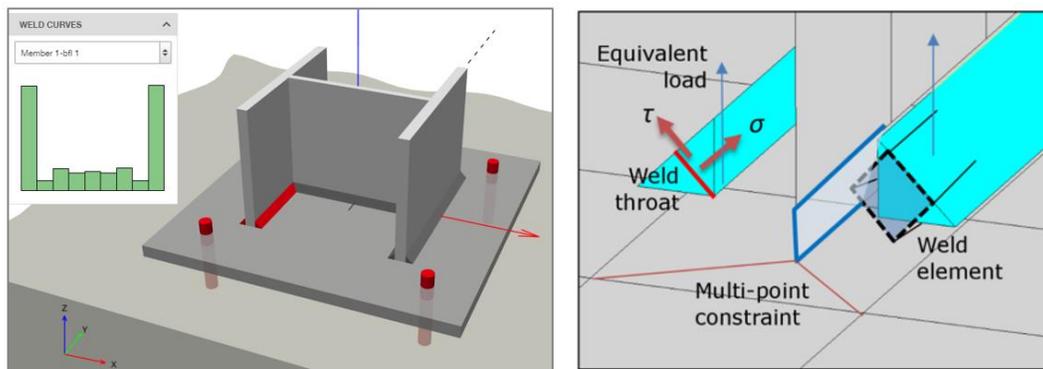


Figure 21. Constraint between weld element and mesh nodes, source [11]

## 2.4.2 Weld with plastic redistribution of stress

The load distribution in weld is derived from the MPC, so the stresses are calculated in the throat section. This is important for the stress distribution in plate under the weld and for modelling of T-stubs. This model does not respect the stiffness of the weld and the stress distribution is conservative. Stress peaks, which appear at the end of plate edges, in corners and rounding, govern the resistance along the whole length of the weld. To express the weld behaviour an improved weld model is applied. A special elastoplastic element is added between the plates. The element respects the weld throat thickness, position and orientation. The equivalent weld solid is inserted with the corresponding weld dimensions. The nonlinear material analysis is applied and elastoplastic behaviour in equivalent weld solid is considered. [11].

## 2.4.3 Steel material and lamellar tearing

PROFIS Engineering does the connection verification in terms of design. Lamellar tearing is related to the choice of the structural steel materials – depending on the weld, a higher 'Z' value may be needed for the involved components (profile, plate and stiffeners).

The consideration of the steel Z-value to prevent lamellar tearing, is not determined by PROFIS Engineering.

Users can input yield strength and ultimate strength of the main steel, from other structural steel information like 'Z'. To fully define the features of the steel more information would be needed, e.g. next image.

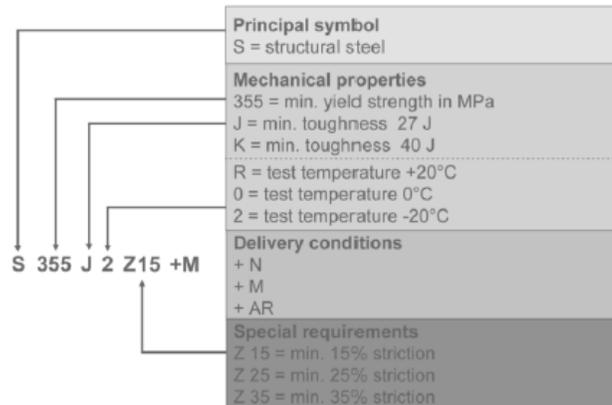


Figure 22. Designation of the steel grade according to EN 10025 (2004). Source: The right choice of steel [13]

## 2.5 Point of applied shear loads

The shear load at the base plate is transferred to the anchors according to EN1992-4 section 6.2.2.3.

Fastenings are considered to act without lever arm, if all conditions below are satisfied:

- The fixture is made from steel and is in contact with the fastener over a length of  $0.5t_{fix}$ .
- Using a levelling mortar with thickness  $t_{gROUT} \leq 0.5d$  under at least the full dimensions of the fixture on a rough concrete surface as an intermediate layer. The strength of the mortar shall be that of the base concrete, but not less than  $30N/mm^2$ .

When the above is not satisfied, then the shear force on the fastenings is assumed to act with lever arm. The considered point of applied load for shear with lever arm is the center of the plate.

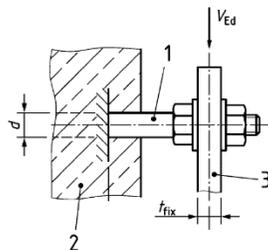


Figure 23: Point of applied load for shear with lever arm

## 2.6 National Annexes of the Eurocode considered in design

The national annexes of EC3 & EC2 affect the advance base plate (ABP) default values for  $\gamma_{M0}$ ,  $\gamma_{M1}$ ,  $\gamma_{M2}$  and concrete factor  $\alpha_{cc}$ .

Most countries in Europe have published a National Annex which provides guidance to engineers regarding which safety factors should be used for design. The default values in PROFIS Engineering are according to the national annex for each region used in design.

However, the user can also edit these values in the advanced settings (check the image below).

**Steel calculation method and parameters** ^

EN based  
 AISC and LRFD based

Steel safety factor gamma M0	Steel safety factor gamma M1	Steel safety factor gamma M2
1	1.1	1.25

**Advanced baseplate calculation settings** ^

**Analysis and checks**

View steel stress / strain results:  
 ULS steel stress / strain  
 Maximum steel stress / strain

**Concrete block**

Joint coefficient Bj	Effective area (influence of mesh size)	Load distribution angle of concrete block	$\alpha_{cc}$
0.67	0.1	33.7 °	0.85

Figure 24. Advanced settings to change safety factors

## 2.7 CBFEM application in case of seismic design

Earthquake resistance steel buildings shall be designed in accordance with one of the following dissipative behavior:

- Low dissipative behavior (concept a)
- Medium or high dissipative behavior (concept b)

Table 1: requirements on cross sectional class of dissipative elements depending on ductility class (medium and high) and reference to behavior factor  $q$  (Table 6.3, EN1998-1)

Design concept	Structural ductility class	Range of reference values for behavior factor $q$
Concept a) Low dissipative seismic behavior	DCL	$\leq 1.5-2$
Concept b) Dissipative seismic behavior	DCM	$\leq 4$
	DCH	Only limited by values of EN1998 table 6.2

In PROFIS Engineering Load Type Seismic should be selected to prove earthquake load combinations.



Figure 25: Selection of seismic load combinations

The value of  $q$  should be determined by the user outside of PROFIS. Based on the value of  $q$  and Table 1, elastic or capacity design should be selected.

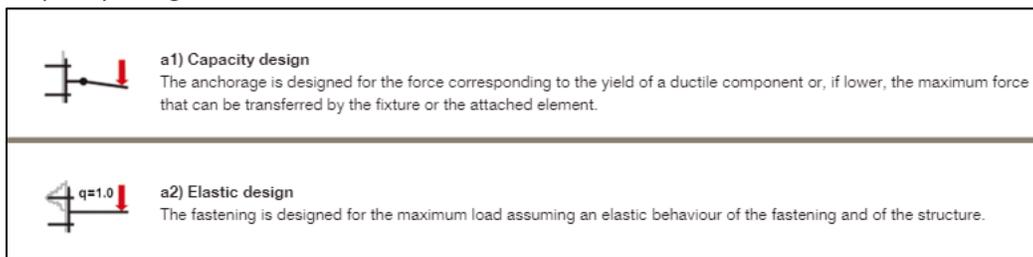


Figure 26: Capacity and elastic design

The CBFEM design described in this document is applicable for concept a), and the resistance of the members and of the connections should be evaluated in accordance with EN 1993 without any additional requirements (section 6.1.2(4) of EN 1998-1).

In this case, PROFIS Engineering proceeds to connection design for seismic load combinations, as described below.



Table 2: CBFEM details in case of seismic (DCL,  $q < 1.5-2$ )

Component	Load distribution definition	Resistance definition
Steel	2.1 Material model for steel	3.3 Profile 3.2 Base plate 3.4 Stiffeners
Anchors	2.2 Anchors	3.1 Anchor verifications
Concrete block	2.3 Concrete block	3.5 Concrete block
Welds	2.4 Welds	3.6 Welds

CBFEM application in case of seismic design has been investigated and assessed by Hilti, [14].

### 3 CBFEM design: Verification of components according to Eurocode

CBFEM method combines advantages of general Finite Element Method (FEM) and Component Method (CM) mentioned in EN1993-1-8 [1]. The stresses and internal forces derived from the CBFEM model are used in checks of all components. Individual components are checked according to Eurocode EN 1993-1-8 [1]. In general, all steel materials must be larger than 4mm, which is a limit to fully comply with EN1993-1-1 [15] & EN1993-1-8 [1].

PROFIS determines the design at the level of the connection. Critical sections for design, i.e. buckling, are not determined in the verification of the connection, precisely because PROFIS would need to know more about the surrounding structure, and not only the node.

#### 3.1 Anchor verifications

##### 3.1.1 Static load combinations

The user can choose to perform anchor design per following European design codes:

- Eurocode 2-4.
- ETAG 001 Annex C.
- EOTA TR029.
- *fib* design bulletin 58.

Verifications are performed according to user selected guideline for steel and concrete failure modes.

Table 3. Required verifications for headed and post-installed fasteners in tension

	Failure mode	Single fastener	Group of fasteners	
			Most loaded fastener	group
	<b>Steel failure of fastener</b>	$N_{Ed} \leq N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}}$	$N_{Ed}^h \leq N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}}$	
	<b>Concrete cone failure</b>	$N_{Ed} \leq N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}}$		$N_{Ed}^g \leq N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}}$
	<b>Pull-out failure of fastener</b>	$N_{Ed} \leq N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}}$	$N_{Ed}^h \leq N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}}$	
	<b>Combined pull-out and concrete failure</b>	$N_{Ed} \leq N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}}$		$N_{Ed}^g \leq N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}}$
	<b>Concrete splitting failure</b>	$N_{Ed} \leq N_{Rd,sp} = \frac{N_{Rk,sp}}{\gamma_{Msp}}$		$N_{Ed}^g \leq N_{Rd,sp} = \frac{N_{Rk,sp}}{\gamma_{Msp}}$

Table 4. Required verifications for headed and post-installed fasteners in shear

	Failure mode	Single fastener	Group of fasteners	
			Most loaded fastener	group
1 	Steel failure of fastener w/o lever arm	$V_{Ed} \leq V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{Ms}}$	$V_{Ed}^h \leq V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{Ms}}$	
2 	Steel failure of fastener w/ lever arm	$V_{Ed} \leq V_{Rd,s,M} = \frac{V_{Rk,s,M}}{\gamma_{Ms}}$	$V_{Ed}^h \leq V_{Rd,s,M} = \frac{V_{Rk,s,M}}{\gamma_{Ms}}$	
3 	Concrete pry-out failure	$V_{Ed} \leq V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{Mc}}$		$V_{Ed}^g \leq V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{Mc}}$
4 	Concrete edge failure	$V_{Ed} \leq V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{Mc}}$		$V_{Ed}^g \leq V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{Mc}}$

### 3.1.2 Seismic load combinations

Per EN1992-4/EOTA TR045, the design resistance of a fastening in case of seismic, shall be performed per failure mode, according to:

$$R_{d,eq} = \frac{R_{k,eq}}{\gamma_{M,eq}}$$

Where:

$$R_{k,eq} = \alpha_{gap} \alpha_{eq} R_{k,eq}^0$$

$\gamma_{M,eq}$  is safety factors related to seismic failure mode

And:

$R_{k,eq}^0$  is the basic characteristic resistance for a given failure mode

$\alpha_{gap}$  is defined in product ETA

$\alpha_{eq}$  is defined in the tables below

Table 5: Reduction factors for  $\alpha_{eq}$  in tension, per EN1992-4

	Failure mode	Single fastener	Fastener group
1 	<b>Steel failure of fastener</b>	1	1
2 	<b>Concrete cone failure</b> Headed fastener and undercut fastener	1	0.85
		0.85	0.75
3 	<b>Pull-out failure of fastener</b>	1	0.85
4 	<b>Combined pull-out and concrete failure</b>	1	0.85
5 	<b>Concrete splitting failure</b>	1	0.85

Table 6: Reduction factors for  $\alpha_{eq}$  in shear, per EN1992-4

	Failure mode	Single fastener	Fastener group
1 	<b>Steel failure of fastener w/o lever arm</b>	1	0.85
2 	<b>Steel failure of fastener w/ lever arm</b>	1	0.85
3 	<b>Concrete pry-out failure</b> Headed fastener and undercut fastener	1	0.85
		0.85	0.75
4 	<b>Concrete edge failure</b>	1	0.85

### 3.2 Base plate

The resulting equivalent stress (Von Mises Stress) and plastic strain are calculated on plates as per the FEM model defined in chapter 2.

#### Stress

The use of Von Mises stress is also indicated to determine the maximum stress level in the cross section.

$$\sigma_{Ed,eq} \leq f_{yd} \text{ EN 1993-1-6 [16], section 6.2}$$

*PROFIS Engineering* and the EN 1993-1-5 [10], App C, Part C.8 Note 1 allow stress slightly higher than  $f_{yd}$ , because the steel yielding stress level is not 100% constant. There is a very small increase of stress during this phase.

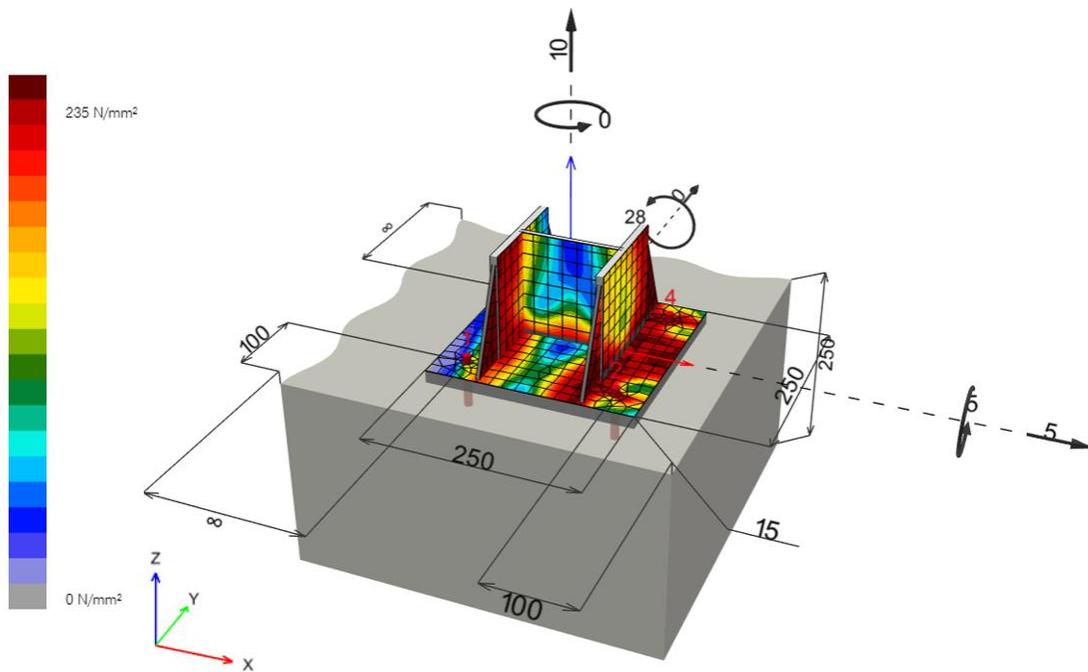


Figure 27. Example of steel stress

#### Strains

Ultimate limit state criteria for plated structural elements should verify the principal membrane strains against a limiting value of strain ( $\epsilon$ ).

$$\epsilon \leq \epsilon_{lim} \text{ as per EN 1993-1-5 [10], App C, Part C.8 Note 1}$$

Where  $\epsilon_{lim}$  is defined by the user. PROFIS Engineering presents a default value of  $\epsilon_{lim}$ , considering a max. value of 5% EN 1993-1-5 [10], App C, Part C.8 Note 1.

The engineer is responsible to check the max. strain limits, and if there are any National regulations for  $\epsilon_{lim}$  in specific markets. This information would be in the NA for EN 1993-1-5 [10], App C, Part C.8 Note 1.

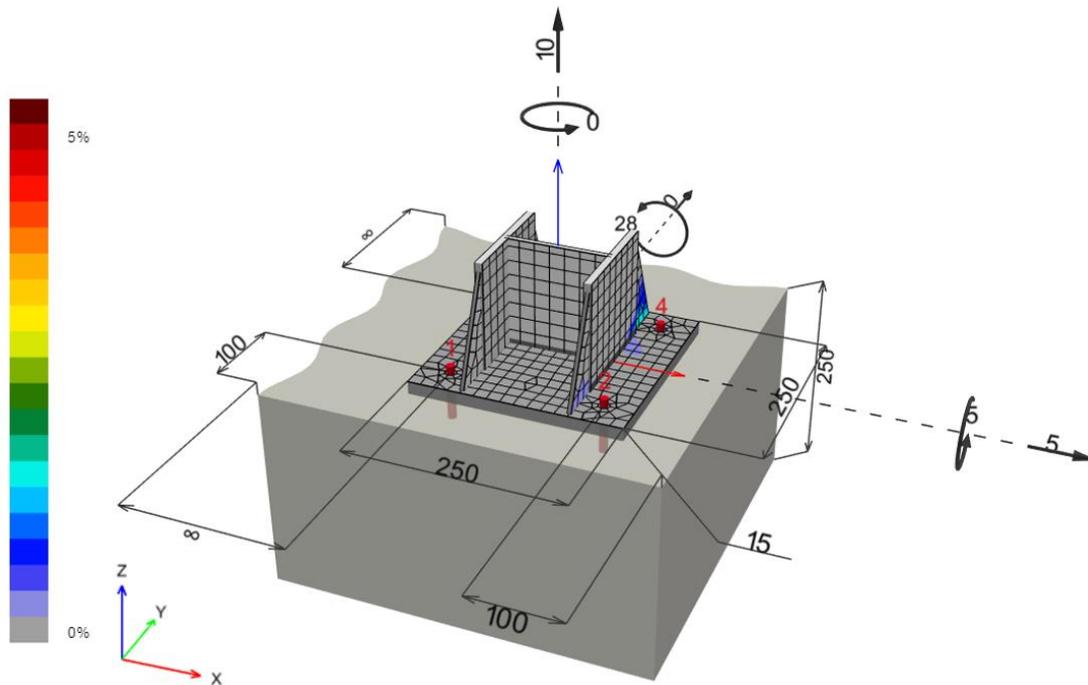


Figure 28. Example plastic strain

Since the plate elements are divided into 5 layers, elastic/plastic behavior is investigated in each layer separately. A verification of stress level and equivalent plastic strain is performed by the program- the calculation output relates to the most critical verification for all 5 layers.

The limiting criteria is 5%- as suggested in Eurocode EN 1993-1-5 [10], App C, Part C.8 Note 1. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

Part	Load combination	Material	$f_y$ [N/mm <sup>2</sup> ]	$\gamma_{M0}$	$\epsilon_{lim}$ [%]	$\sigma_{Ed}$ [N/mm <sup>2</sup> ]	$\epsilon_{Pl}$ [%]	Status
Plate	Combination 1	S 235	235.00	1.00	5.00	235.20	0.10	OK
Profile	Combination 1	S 235	235.00	1.00	5.00	235.55	0.26	OK

Figure 29. Equivalent stress and plastic strain

CBFEM method can provide steel stress higher than the yield strength. The reason is the slight inclination (1) of the plastic branch of the stress-strain diagram which is used in the analysis to improve the stability of interaction calculation.

This is not a problem in practical design. At higher loads, the equivalent plastic strain is rising and the connection fails while exceeding the plastic strain limit.

### 3.3 Profile

A part of the profile is modeled to ensure that the stress distribution in the profile is “settled” in order to be transferred to the welds. However, the software does not replace the verification on the superstructure because is not doing any buckling or stability assessment. But is checking the stresses and strain in a certain section of the profile.

#### Strains

Ultimate limit state criteria for plated structural elements should verify the principal membrane strains against a limiting value of strain ( $\varepsilon$ ).

$$\varepsilon \leq \varepsilon_{lim} \text{ as per EN 1993-1-5 [10], App C, Part C.8 Note 1}$$

Where  $\varepsilon_{lim}$  is defined by the user. *PROFIS Engineering* presents a default value of  $\varepsilon_{lim}$ , considering a max. value of 5% EN 1993-1-5 [10], App C, Part C.8 Note 1.

The engineer is responsible to check the max. strain limits, and if there are any National regulations for  $\varepsilon_{lim}$  in specific markets. This information would be in the NA for EN 1993-1-5 [10], App C, Part C.8 Note 1.

#### Stress

The use of Von Mises stress is also indicated to determine the maximum stress level in the cross section.

$$\sigma_{Ed,eq} \leq f_{yd} \text{ EN 1993-1-6 [16], section 6.2}$$

Both *PROFIS Engineering* and the EN 1993-1-5 [10], App C, Part C.8 Note 1 allow stress slightly higher than  $f_{yd}$ . This because the steel yielding stress level is not 100% constant. There is a very small increase of stress during this phase.

### 3.4 Stiffeners

Similar the plate and profile components, *PROFIS Engineering* checks for the stiffeners the equivalent stress (or Von Mises stress) and plastic strain. This check does not replace the frame design which is required for steel structures (including buckling check of the stiffeners).

#### Strains

Ultimate limit state criteria for plated structural elements should verify the principal membrane strains against a limiting value of strain ( $\varepsilon$ ).

$$\varepsilon \leq \varepsilon_{lim} \text{ as per EN 1993-1-5 [10], App C, Part C.8 Note 1}$$

Where  $\varepsilon_{lim}$  is defined by the user. *PROFIS Engineering* presents a default value of  $\varepsilon_{lim}$ , considering a max. value of 5% EN 1993-1-5 [10], App C, Part C.8 Note 1.

The engineer is responsible to check the max. strain limits, and if there are any National regulations for  $\varepsilon_{lim}$  in specific markets. This information would be in the NA for EN 1993-1-5 [10], App C, Part C.8 Note 1.

#### Stress

The use of Von Mises stress is also indicated to determine the maximum stress level in the cross section.

$$\sigma_{Ed,eq} \leq f_{yd} \text{ EN 1993-1-6 [16], section 6.2}$$

*PROFIS Engineering* and the EN 1993-1-5 [10], App C, Part C.8 Note 1 allow stress slightly higher than  $f_{yd}$ , because the steel yielding stress level is not 100% constant. There is a very small increase of stress during this phase.

### 3.5 Concrete block

The resistance of concrete in 3D compression is determined based on EN 1993-1-8 [1] by calculating the design bearing strength of concrete in the connection,  $f_{jd}$ , under the effective area,  $A_{eff}$ , of the base plate. The design bearing strength of the joint,  $f_{jd}$ , is evaluated according to Cl. 6.2.5 in EN 1993-1-8 [1] and Cl. 6.7 in EN 1992-1-1. The grout quality and thickness is introduced by the connection coefficient,  $\beta_{jd}$ . For grout quality equal or better than the quality of the concrete block,  $\beta_{jd} = 1.0$  is expected. The effective area,  $A_{eff,cm}$  under the base plate is estimated to be of the shape of the column cross-section increased by additional bearing width,  $c$ .

$$c = t \sqrt{\frac{f_y}{3f_i\gamma_{M0}}}$$

where  $t$  is the thickness of the base plate,  $f_y$  is the base plate yield strength and  $\gamma_{M0}$  is the partial safety factor for steel.

The effective area is calculated by iteration until the difference between the additional bearing widths of current and previous iteration  $|c_i - c_{i-1}|$  is less than 1 mm.

The area where the concrete is in compression is taken from results of FEA. This area in compression,  $A_{eff,FEM}$ , allows determining the position of the neutral axis.

The intersection of the area in compression,  $A_{eff,FEM}$ , and the effective area,  $A_{eff,cm}$ , allows to assess the resistance for generally loaded column base of any column shape with any stiffeners and is labeled  $A_{eff}$ . The average stress  $\sigma$  on the effective area,  $A_{eff}$ , is determined as the compression force divided by the effective area. Check of the component is in stresses  $\sigma \leq f_{jd}$

Concrete resistance at concentrated compression:

$$f_{jd} = \beta_j k_j \frac{f_{ck}}{\gamma_c}$$

Average stress under the base plate:

$$\sigma = \frac{N}{A_{eff}}$$

Utilization in compression [%]:

$$Ut = \frac{\sigma}{f_{jd}}$$

where:

- $f_{ck}$  – characteristic compressive concrete strength
- $\beta_j = 0.67$  – factor of grout quality editable in Code setup
- $k_j$  – concentration factor
- $\gamma_c$  – safety factor for concrete
- $A_{eff}$  – effective area on which the column normal force  $N$  is distributed

Variables						
N [kN]	$A_{eff}$ [mm <sup>2</sup> ]	$\beta_j$	$k_j$	$\alpha_{cc}$	$f_{ck}$ [N/mm <sup>2</sup> ]	$\gamma_c$
62.225	3,133	0.67	3.00	0.85	25.00	1.50
Results						
$\sigma$ [N/mm <sup>2</sup> ]	$f_{jd}$ [N/mm <sup>2</sup> ]	Utilization [%]		Status		
19.86	28.47	70		OK		

Figure 30. Contact stress in concrete

### 3.5.1 Mesh sensitivity

This procedure of assessing the resistance of the concrete in compression is independent on the mesh of the base plate as can be seen in the figures bellow. It is shown in the example of concrete in compression assessment according to EC.

Two cases were investigated: loading by pure compression of 1200 kN and loading by a combination of compressive force 1200 kN and bending moment 90 kN.

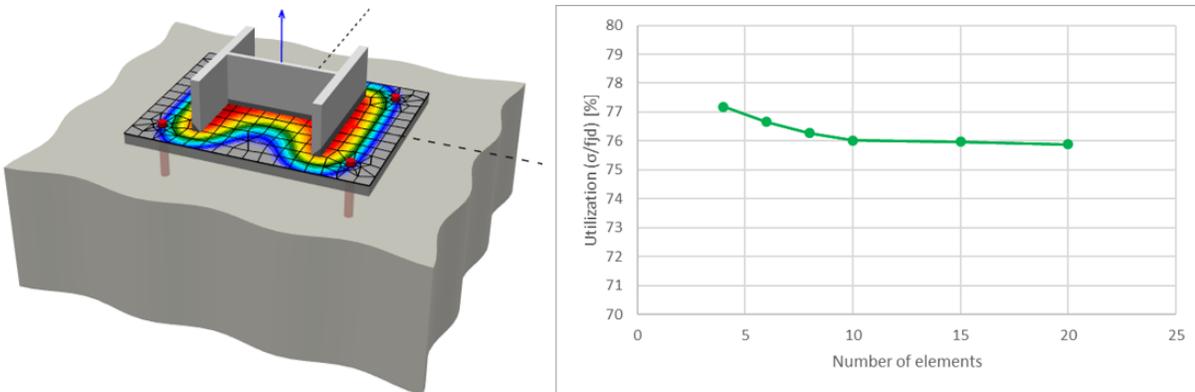


Figure 31. Influence of number of elements on prediction of resistance of concrete in compression in case of pure compression

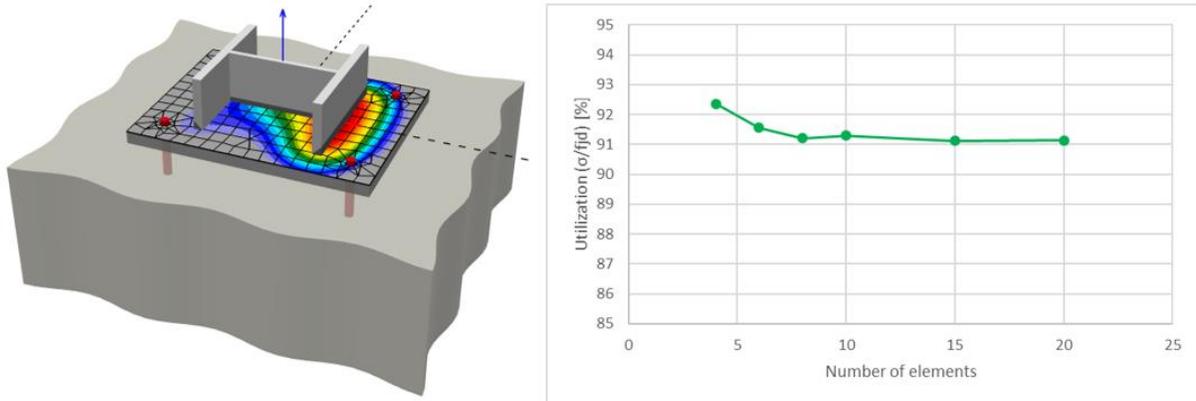


Figure 32. The influence of number of elements on prediction of resistance of concrete in compression in case of compression and bending

### 3.6 Welds

Three welding options are available, considering weld materials as per EN 1991-1 [17]. User may select to model the connection between profile / stiffeners and plate.

#### 3.6.1 No weld

If the steel components are not welded together, then it is assumed that there is no transfer of loads between them. Thus, the weld is not modelled – the elements don't share nodes.

#### 3.6.2 Fillet welds

All loads are transferred via the weld. The fillet weld is modelled as a special weld element, which has an equivalent cross section area as the weld.

#### Design resistance

The plastic strain in weld is limited to 5% as in the plate (EN1993-1-5 [10], App. C, Par. C.8, Note 1). The stress in the throat section of a fillet weld is determined according to EN 1993-1-8 [1], Cl. 4.5.3. using the directional method. Stresses are calculated from the stresses in weld element. Bending moment around the weld longitudinal axis is not considered.

$$\sigma_{\omega,Ed} = \sqrt{\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{\parallel}^2)}$$

$$\sigma_{\omega,Rd} = \frac{f_u}{\beta_{\omega} \gamma_{M2}}$$

## Weld utilization

$$U_t = \min \left\{ \frac{\sigma_{\omega,Ed}}{\sigma_{\omega,Rd}}, \frac{\sigma_{\perp}}{0.9f_u/\gamma_{M2}} \right\}$$

where:

- $\beta_{\omega}$  – correlation factor (EN 1993-1-8 [1] – Table 4.1)
- $f_u$  – ultimate strength, chosen as the lower of the two connected base materials
- $\gamma_{M2}$  – safety factor (EN 1993-1-8 [1]; editable in Code setup)

The plastic strain in weld is limited to 5% as in the plate EN1993-1-5 [10], App. C, Par. C.8, Note 1. The stress in the throat section of a fillet weld is determined according to EN 1993-1-8 [1]. Stresses are calculated from the stresses in weld element. Bending moment around the weld longitudinal axis is not considered.

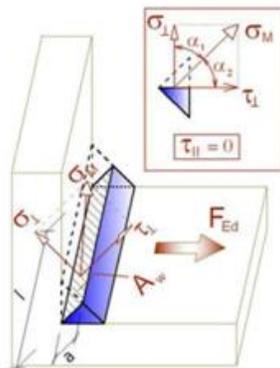


Figure 33. Decomposition of weld loads, source [11]

All values required for check are printed in tables.  $U_t$  is the utilization of the most stressed element. Since plastic redistribution of stress in weld is used, it is the decisive utilization.  $U_{tc}$  provides information about utilization along the weld length. It is the ratio of actual stress at all elements of the weld to the design resistance of the stress of the whole length of the weld, [11].

### 3.6.3 Butt welds

User can select butt welds, which correspond to full penetration.

According to Eurocode 3-1-8, section 4.7.1 for full penetration butt welds the design is implicitly verified by the resistance of the weakest element in the connection.

### 3.7 Deformations

PROFIS Engineering calculates and provides the deformations in every point of the connection including the location of the anchors. Since the anchors are joined with the plate, the deformation on the plate is the same as the deformation on the anchors.

A check on deformations can be done by including SLS loads and verifying the displacements. If a user wants to calculate the deformations under the SLS load combination, it's a question of inputting the SLS load combination into PROFIS Engineering and check these displacements.

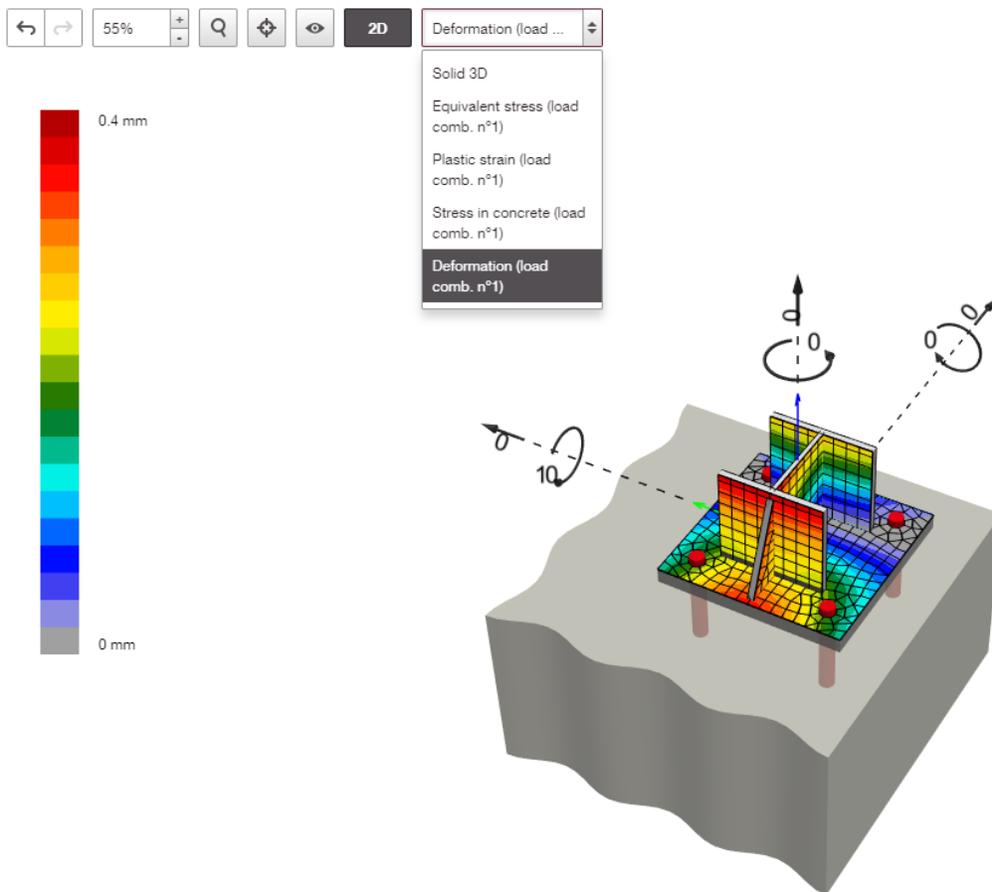


Figure 34. Deformation



### 3.8 Detailing

Detailing checks of minimum distance between anchors are performed always.

Hilti Anchor ETAs prescribe dimensions from:

- Minimum distance between anchors
- Minimum distance between anchors and concrete edge

These options may not be changed by the user. The minimum distances guarantee that the Hilti Anchor is able to resist the loads, including concrete related failure modes (e.g. concrete edge failure).

Program settings prescribe dimensions from:

- Minimum distance between anchor and edge of plate
- Minimum distance between anchor and profile

The default values for min. end distance are compliant with EN 1993-1-8 [1], table 3.3.



## 4 Advanced Base Plate design gives the flexibility to choose the right needed solution

Depending on what design would want to be calculated, advance base plate module gives the freedom to choose the right method.

### 4.1 Anchor design only. Go rigid

Go rigid for anchor design saving time on calculation with improvements such as structure software interfaces, usability, etc. But you need to ensure that the base plate is behaving like a rigid plate.

### 4.2 Full connection design. Go flexible including anchors, baseplate, weld, stiffeners and concrete

There are three options for a full connection design:

- a) Go flexible with verified rigidity of base plate, in case of wanting to apply Eurocode rules strictly. This is the right option for taking the “guessing” out of the base plate solution. But it might get either a negligible displacement of the base plate compared the anchors displacement. Or go with advance base plate getting 0% deviation, and it might lead to uneconomical base plate thicknesses.
- b) Go flexible and optimize base plate with max. rigidity deviation of 10%. Improve the connection design with engineering judgement backed by extensive testing, satisfying the verification engineer with a comprehensive design report for the entire connection.
- c) Go Flexible and optimize base plate according to the engineer own judgment. With one click do complete connection design and iterate quickly to find the most suitable solution. Considering that anchor design formulas are only valid if the base plate can be considered rigid.

## 5 Proposed solution for assessing if the base plate can be considered “close to rigid” by engineering judgement

Rigid base plate is a theoretical approach. In real-life no member is fully rigid - if you load it enough the base plate will deform. “Close to rigid” base plates behave similar than rigid base plates acknowledging the real behavior. If the behavior is very similar, the anchor design guidelines can be applied by engineering judgement.

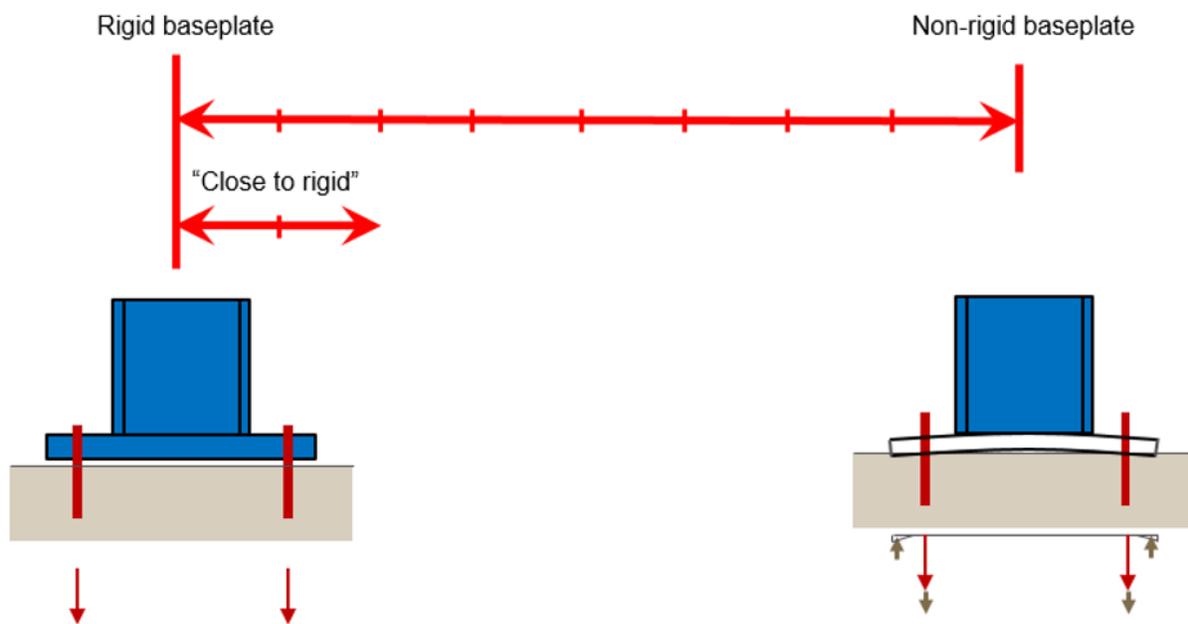


Figure 35. Definition of “Close to rigid” base plates

### 5.1 Engineering judgement to define “Close to rigid” base plates behavior

Whereas no check of the actual base plate rigidity was previously carried out, this is now performed at the end of the calculation process (Figure 36. Rigid and Flexible comparison window). The anchor loads calculated according to the elasticity theory are compared with the anchor loads taking account of equilibrium and compatibility conditions on the basis of realistic assumptions of the load-displacement behavior and the stress-strain curves of the individual components, and assuming a rigid base plate. By comparing the anchor loads of both methods, the gap between theory (rigid base plate) and practice within a software can be determined.

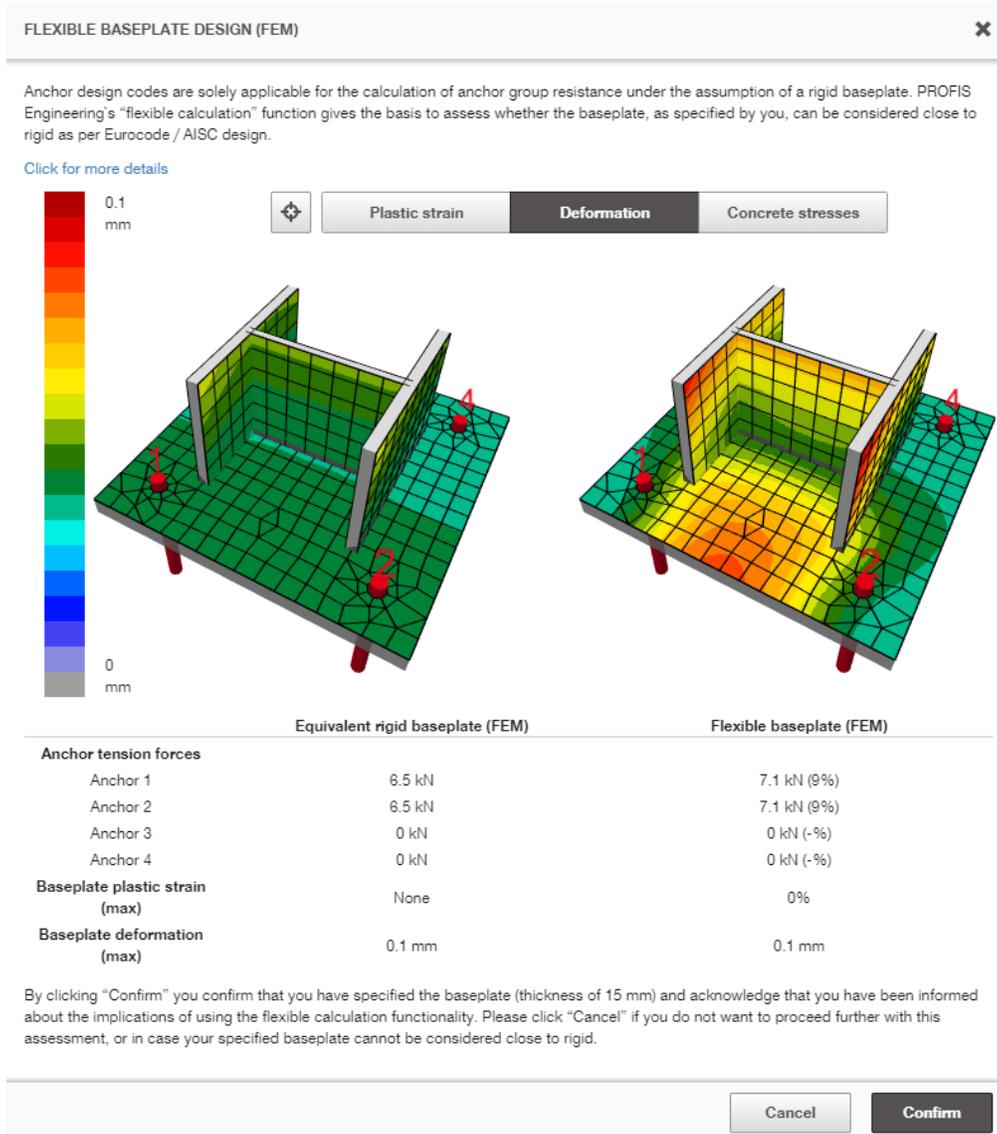


Figure 36. Rigid and Flexible comparison window

Figure 37. Increase of anchor forces associated with a flexible baseplate, from [18], shows experimentally determined mean failure loads of anchor groups ( $N_{u,test}$ ) related to the calculated failure load taking into account the elasticity theory ( $N_{u,E-theory}$ ) as a function of highest loaded anchor of the group based on non-linear assumptions, related to the calculated value of the most loaded anchor, determined on the basis of the elasticity theory. These tests consider anchor groups with four and nine anchors under uniaxial and biaxial bending in non-cracked concrete. In the tests, cast-in headed studs and post-installed anchors were used.

Figure 37 was chosen in order to estimate to what extent a deviation of the most heavily loaded anchor of a group from the calculation according to the elasticity theory affects the overall group’s load capacity. Based on the available test values, with a deviation of the anchor loads of the most loaded anchor of a group of approx. 10% - 15% – between rigid and realistic baseplate assumptions – the mean failure load of the group fixture corresponds to the mean group failure load calculated according to the elasticity theory. This means that in the case of an approx. 10% - 15% deviation in the anchor loads between the rigid plate and the flexible plate, no negative influence on the load bearing capacity could be observed in the tests.

The comparison of the rigid and the realistic base plate in combination with the above investigations should help the planning engineer to make an assessment of the existing base plate thickness that does not contradict the applicable guidelines.

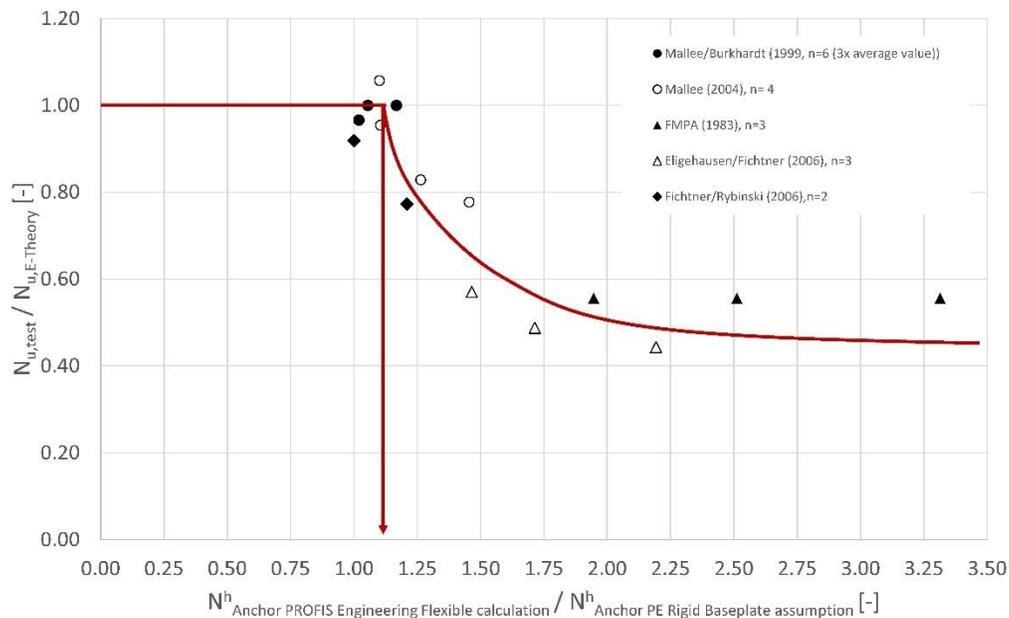


Figure 37. Increase of anchor forces associated with a flexible baseplate

If there are other engineering judgement considerations to go beyond the 10% value proposed here, please apply your own engineering judgement.

Looking at the influencing parameters of non-rigid base plates (see chapter 1.3), the following parameters are important to define if a base plate can be considered “close to rigid”.

### 5.1.1 Anchor forces

The following three effects will lead to an increase in anchor forces:

- Reduction of inner lever arm, Figure 2.



- Prying effects, Figure 3.
- Different load distribution in anchor groups, Figure 4.

If the anchor forces are similar to the rigid base plate results one can assume that the plate behaves in a very similar compared to a rigid plate – in this case the plate can be considered “close to rigid”.

Some literature on this point to be able to take an engineering judgement are: Mallée/Burkhardt [19], Fichtner [20].

The anchor force results of a rigid vs non-rigid base plate will be accepted up to engineering judgement by the engineer who is designing.

#### 5.1.2 Concrete stresses

EN 1993-1-8 [1] defines how concrete stresses should be evaluated based on the T-stub model, which is also valid for non-rigid base plates. Therefore, a check of the concrete stresses coming from our “close to rigid” FEM calculation is sufficient.

#### 5.1.3 Deformation

It’s up to the engineer and/or the owner to decide which deformation in serviceability limit state will be acceptable.

## 5.2 Hilti recommendation for engineering judgement

Based on the information given above the engineer needs to decide if the base plate can be considered “close to rigid” on a case by case basis to ensure that the scope of the anchor design guidelines is at least met by engineering judgement.

Due to the fact there is no general rule applicable for all cases, Hilti is not giving a recommendation how much deviation from the rigid base plate can be accepted.



## 6 Conclusion

The assumptions used for the design are the responsibility of the engineer. Different limitations in the anchor and structural steel design guidelines require to introduce the “close to rigid” base plate to have a safe solution for anchor design. This base plate will behave very similar to the rigid base plate assumption prescribed in the anchor design guidelines by engineering judgement.

This technical background document shows the main influencing factors for the engineer to judge if his base plate is in accordance with the scope of the anchor design guidelines. The summarized literature should help the engineer to take this engineering judgement call.

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### **Advance Baseplate Design:**

- *Use of Advance Baseplate Design with PROFIS Engineering- Desktop Application requires an active internet connection*
- *Realistic baseplate/ stresses/ strains/ deformation/ design proofs as per EN1993*
- *Design of weld as per EN1993-1-8*
- *Design of stiffeners as per EN1993*
- *Concrete bearing capacity check as per EN1992*
- *Anchor design as per ETAG/ TR 029*
- *Anchor stiffness data is based on extensive tests conducted by Hilti and Hilti engineering judgement*



- *By using the realistic calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified base plate may not behave rigidly. Please, have the results validated by a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.*

## Figures

Figure 1. Anchors modeled by springs in steel-to-concrete connection.....	4
Figure 2. Reduction of inner lever arm for non-rigid base plates .....	5
Figure 3. Increase of anchor forces due to prying effects.....	6
Figure 4. Increase of anchor forces due to different load distribution for non-rigid base plates.....	6
Figure 5. Different concrete stress distribution .....	7
Figure 6. Displacement of cantilever beam in case of rigid and non-rigid base plate .....	7
Figure 7. Example comparing rigid vs flexible .....	8
Figure 8. Base plate design options for determination of the load distribution to the anchors .....	8
Figure 9. Rigid base plate behavior .....	9
Figure 10. Steps taken in the rigid design .....	10
Figure 11. Two examples comparing non- rigid base plate.....	11
Figure 12. FEM model of a connection for research. It uses spatial 3D elements for both plates and anchors.....	12
Figure 13. Material diagrams of steel in numerical models.....	13
Figure 14. The mesh on a column and baseplate with constraints between the web and the flange .....	14
Figure 15. The mesh on a baseplate with 16 elements along its width .....	15
Figure 16. Influence of mesh size of base plate stress.....	15
Figure 17. Schematic load displacement.....	16
Figure 18. Anchors with stand-off with grouting .....	16
Figure 19: Selection of standoff with grout.....	17
Figure 20: Example of steel shear resistance of Hilti Anchor with lever arm .....	17
Figure 21. Constraint between weld element and mesh nodes, source [11] .....	19
Figure 22. Designation of the steel grade according to EN 10025 (2004). Source: The right choice of steel [13] .....	20
Figure 23: Point of applied load for shear with lever arm .....	20
Figure 24. Advanced settings to change safety factors.....	21
Figure 25: Selection of seismic load combinations .....	22
Figure 26: Capacity and elastic design .....	22
Figure 27. Example of steel stress .....	27
Figure 28. Example plastic strain.....	28
Figure 29. Equivalent stress and plastic strain .....	28
Figure 30. Contact stress in concrete .....	32
Figure 31. Influence of number of elements on prediction of resistance of concrete in compression in case of pure compression .....	32
Figure 32. The influence of number of elements on prediction of resistance of concrete in compression in case of compression and bending.....	33
Figure 33. Decomposition of weld loads, source [11].....	34
Figure 34. Deformation .....	35
Figure 35. Definition of "Close to rigid" base plates .....	38



Figure 36. Rigid and Flexible comparison window ..... 39  
Figure 37. Increase of anchor forces associated with a flexible baseplate ..... 40



## Table

Table 1: requirements on cross sectional class of dissipative elements depending on ductility class (medium and high) and reference to behavior factor $q$ (Table 6.3, EN1998-1).....	22
Table 2: CBFEM details in case of seismic (DCL, $q < 1.5-2$ ) .....	23
Table 3. Required verifications for headed and post-installed fasteners in tension .....	24
Table 4. Required verifications for headed and post-installed fasteners in shear .....	25
Table 5: Reduction factors for $\alpha_{eq}$ in tension, per EN1992-4 .....	26
Table 6: Reduction factors for $\alpha_{eq}$ in shear, per EN1992-4.....	26

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