HILTI

WHITEPAPER

Anchor design accounting for realistic baseplate flexibility

Steel-to-concrete connections

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1. Introduction and background

Steel frameworks represent one of the most prevalent structural systems in contemporary construction, valued for their strength, flexibility and rapid assembly. The reliability of a steel structure depends not only on the strength and stiffness of its beams and columns but equally on the performance and quality of its connections (Fig. 1a). These joints determine how internal forces and moments are transferred between individual members, helping to ensure continuity and integrity across the entire load-bearing system. Moreover, the mechanical behavior of the connections governs the relative displacement between structural components, thereby influencing the overall deformation pattern and global response of the structure [1] .

Steel-to-concrete (S2C) connections need specific design considerations since they form a critical interface between two fundamentally different materials (Fig. 1b). These connections are designed to transfer forces safely from steel members to concrete elements (such as slabs, cores, walls, or foundations) while accommodating differences in stiffness, deformation behavior, and long-term performance.

a) Steel-to-steel connection

b) Steel-to-concrete connection

Fig. 1: Examples of typical connections used in steel framework construction

Designing S2C connections requires simultaneous compliance with multiple structural design standards, each governing a distinct element of the load-transfer system. Eurocode 2 (EC2) [2] regulates the behavior and resistance of the concrete member, Eurocode 3 (EC3) [3] specifies the requirements for the steel components, and Eurocode 2 part 4 (EC2-4) [4] provides the design methodology for anchors in concrete (see Fig. 2). As a result, engineers must ensure that all components of a S2C connection collectively satisfy the provisions of these codes.

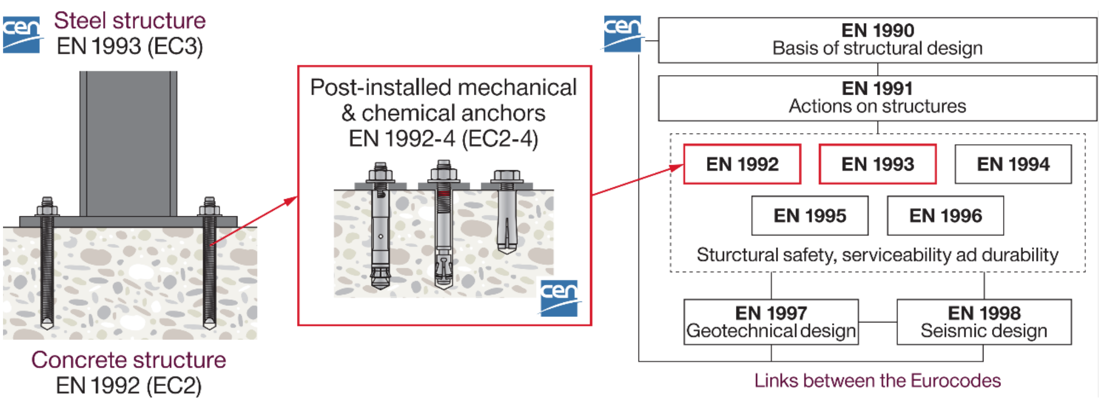


Fig. 2: Regulatory framework for fastening in concrete: links between the Eurocodes and the focus on EC2-4 for anchors

The structural behavior of steel baseplates sits at the intersection of two regulatory domains: steel design under EC3 and anchorage-to-concrete design under EC2-4. Although both codes address the same physical interface, they adopt fundamentally different design philosophies. Eurocode 3 employs a simplified bearing-stress model based on the equivalent T-stub in compression, prescribing an idealized bearing distribution beneath an effective area without assuming a rigid plate; in fact, EC3 explicitly requires verification of plate bending resistance, demonstrating that plate flexibility is intrinsic to its methodology. In contrast, when performing group verifications according to the Concrete Capacity Design (CCD) method [5], the EC2-4 explicitly requires the baseplate to behave as a rigid element, meaning its bending deformation under load must be negligible. If the baseplate can be considered sufficiently rigid, the tensile forces in each anchor may be calculated assuming linear distribution of strains across the baseplate and linear relationship between strain and stresses. This divergence introduces an initial discrepancy in the design philosophy for S2C connections.

Furthermore, within EC2-4, the rigid baseplate assumption, although facilitating standardized hand calculations and efficient design workflows, does not always reflect the actual flexural behavior of baseplates in practice. When real baseplates deviate from ideal rigidity, anchor force predictions and subsequent resistance checks may no longer align with actual structural response.

In practice, many baseplates, like the ones used in industrial assemblies, heavy mechanical installations, and connections with large eccentricities, can experience significant flexural deformation. This bending leads to a nonlinear distribution of forces among anchors and concrete, which the rigid baseplate model cannot capture. When baseplate flexibility is ignored, the resulting design may underestimate actual tension forces in the anchors. Further, because EC 2-4 bases its resistance calculations on force paths derived from the ideal rigid baseplate assumption, significant baseplate deformation can cause the real force transfer to deviate from these assumptions. As a result, **the predicted resistances may be higher than the actual resistances under certain conditions**. This creates a **methodological gap** between codified assumptions and real-world connection behavior.

To fully satisfy EC2-4 design code prescriptions of rigid baseplate behavior, engineers may choose overly conservative detailing choice. This includes thickening baseplates, enlarging anchor groups, or adding stiffeners not because the structural system demands it, but because the design model cannot accurately represent the actual load transfer. These measures increase material use, fabrication effort, and installation complexity without necessarily improving overall structural performance.

In summary, while Eurocodes provide a robust and widely accepted design framework, the assumption of a rigid baseplate may not fully capture all aspects of real structural behavior and may guide detailing choices that are not always aligned with the most economical solutions.

This paper provides an overview of applications where realistic modelling of the structural behavior of post-installed anchorages can help to achieve safer, reliable and optimized S2C connections. It reviews current design practice and highlights limitations of conventional approaches, particularly those related to constraints in existing code methodologies due to baseplate stiffness assumptions. Building on this, the paper introduces **Hilti's new design method**, which offers a more representative prediction of anchor group resistance associated with tension relevant failure modes.

2. Design optimization in steel-to-concrete connections towards a more sustainable built environment

The construction industry is undergoing a fundamental transformation as the climate emergency accelerates the need for resource efficiency and low carbon design. Traditional approaches that rely on heavy overdesign are no longer compatible with sustainability objectives [6]. Across the construction industry, designers are adopting new methodologies that prioritize realistic modelling, lifecycle performance, and material minimization, without compromising safety or constructability.

Within this transition, S2C connections represent a critical but often overlooked opportunity, as they have historically been treated using conservative, highly simplified assumptions. These connections are highly standardized, widely repeated, and directly influence both structural reliability and embodied carbon. As such, they offer substantial potential for optimization at scale.

Post-installed connections are present across structural and non-structural systems, including steel frames, industrial equipment supports, façades, and MEP (Mechanical, Electrical and Plumbing) installations. All S2C connections, regardless of their specific type, scale, or application, share the common requirement of effective transfer of tension, shear, and bending forces between steel components and concrete substrates, while accommodating differing stiffness and deformation characteristics of the two different materials.

In this section, we first introduce the main features of a generic S2C connection, emphasizing the role of each component in the load-transfer mechanism. We then present various applications in which accurately modelling the connection stiffness is essential, either because it significantly influences the behavior of the overall structural system or because it enables optimization of size and cost in standardized, repetitive applications.

2.1 Why stiffness matters in design of post-installed fastening systems

Steel components inevitably bend and deform under load, meaning steel-to-concrete connections are typically not fully rigid in practice. In S2C connections using post-installed anchors, a steel baseplate is fixed to hardened concrete with mechanical or adhesive anchors (Fig. 3).

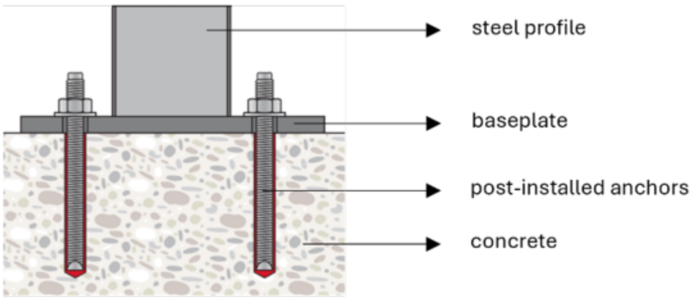


Fig. 3: Schematic arrangement of the components of a steel to concrete connection

The baseplate contributes to the overall shear and rotational stiffness through its bending resistance and its ability to transfer forces into the concrete. The anchors typically play a dominant role: their type, size and embedment depth, slip, bond behavior, steel elongation, largely govern the joint's stiffness. The concrete also significantly influences stiffness through cracking, bearing beneath the plate, and confinement effects around the anchors. Together, these elements form a coupled system whose stiffness depends on anchor type, size, arrangement, embedment, plate thickness, and concrete quality. Because real joints are never perfectly rigid, accurately capturing their structural behavior is essential for realistic drift prediction, reliable load path modelling, and robust steel-to-concrete interaction assessment.

Oversimplifying the stiffness of S2C connections can lead to inaccurate force distribution and deformation predictions, introducing unnecessary conservatism or overlooking critical demands. In contrast, realistic stiffness modelling helps to represent the force transfer mechanisms with greater reliability, leading to safer and more efficient S2C designs. By reducing reliance on overly simplified assumptions and avoiding unnecessary overdesign, stiffness-based modelling aligns structural performance requirements with key objectives of low-carbon and resource-efficient construction.

2.2 Structural baseplates as high-value optimization target

Structural S2C connections are safety-relevant components because they govern load transfer, deformation compatibility, and structural robustness; their failure can compromise global stability and lead to disproportionate collapse, even if individual members remain intact. Their design and installation directly influence load-transfer performance, global stability, and the overall cost efficiency of the structural system (Fig. 4).

a) Steel column-to-concrete connection

b) Steel beam-to-concrete connection

Fig. 4: Example of steel-to-concrete structural connections

The selected connection type (rigid, pinned, or semi rigid) affects steel member sizes and concrete requirements. A rigid column-to-concrete connection provides full rotational restraint and transfers moments into the foundation. This solution can reduce weight of the steelwork but may require larger baseplates, more anchors, and more complex fabrication. Pinned connections, common in low rise metal building systems, result in slightly heavier steel sections but simpler foundations and fewer, smaller anchors.

Current practice often models steel to concrete connections as pinned. Base connections, which are designed as pinned supports, exhibit a non-negligible level of rotational stiffness and strength. Neglecting the stiffness at design stage may result in a significant overestimation of the lateral displacement of the frames. A typical example of out of plane bending of the baseplate under moment loading and the increased rotation at the column base is shown in Fig. 5. This additional displacement is addressed by increasing the flexural stiffness of the frame members thereby increasing the cost of low-rise metal buildings. While conservative, this approach can lead to unnecessary material use at structural level.

Given these multiple interacting effects, S2C connections are more realistically represented as semi-rigid components, motivating the use of stiffness-based modelling approaches rather than idealized rigid or pinned assumptions. Realistic modelling of connection stiffness has a direct impact on global design efficiency. From an economic standpoint, this approach opens the door to substantial material optimization. Engineers can safely design lighter framing systems, while optimizing the thickness of the baseplate and anchor layouts, when possible, and avoid unnecessary foundation enlargement, all without compromising structural reliability. These refinements accumulate across the numerous S2C interfaces typically present in a building or industrial system, with the potential for meaningful reduction in steel tonnage, concrete volume, and installation effort. As the industry moves toward increasingly optimized and sustainable structures, realistic representation of S2C behavior becomes a requirement rather than an optional refinement.

Ultimately, accurate modelling of S2C connections strengthens both safety and economy at system level. It helps to ensure that structural performance predictions better reflect connection behavior, improves the reliability of global analyses, and enables targeted material reductions that support low-carbon design objectives.

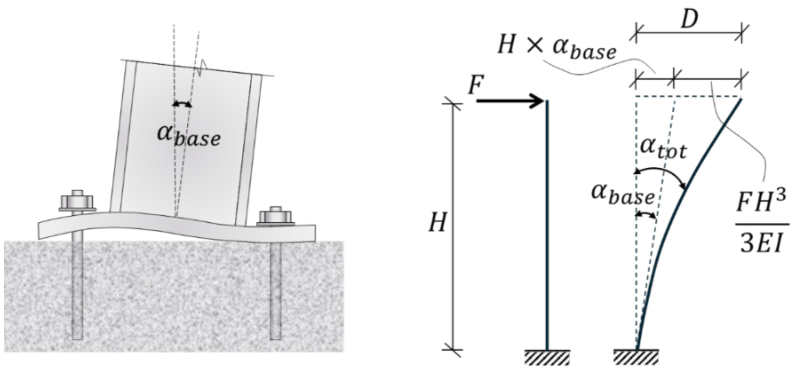


Fig. 5: Example of how the rotation α_{base} due to flexural deformation of the baseplate contributes to the total drift D of a free-standing column of height H

2.3 Post-installed connections: critical in renovation and retrofit

Post-installed baseplates are commonly used in retrofit and renovation work where cast-in anchorage solutions are not feasible. They enable the addition of new structural elements, the strengthening of deficient members, the correction of construction errors, and the restoration of load paths in existing concrete structures. Such solutions are often adopted when design decisions are modified during construction, requiring flexibility in the location or capacity of steel-to-concrete connections after concrete hardening.

In rehabilitation and upgrade projects, designers frequently work with **concrete of uncertain quality, irregular geometries**, and **constrained edge distances**, conditions that deviate substantially from the simplified assumptions of classical design models for anchorages. In such connections, baseplates are relatively thin, heavily loaded, and therefore show pronounced deviations from the idealized linear strain distribution underlying rigid-plate theory.

In these cases, accurate modelling of the S2C connection becomes essential for the design of the anchorage system. The plate thickness required to achieve rigid behavior per EC2-4 is often impractically large; moreover, as the baseplate becomes thinner, the influence of anchor stiffness increases significantly. Therefore, realistic analysis must explicitly include plate flexibility, anchor stiffness, and concrete contact behavior, particularly in statically indeterminate assemblies where these interactions govern the load path.

The need for detailed modelling is even more critical in **seismic retrofitting**, where the goal is to enhance performance and reduce seismic demand without intrusive reconstruction. Strengthening that avoids major alterations to primary structural members are particularly attractive for strategic and heritage structures where preservation is essential. Among the various retrofitting solutions, for the installation of steel braces into existing reinforced concrete frames [7] [8] or local strengthening of beam-to-column joints using steel haunches [9] have proven to be effective strategies for increasing lateral strength and stiffness. These methods rely on post-installed anchors and baseplates to transfer forces into existing reinforced concrete framework, as illustrated in Fig. 6 and Fig. 7, respectively. In these configurations, the steel connection is positioned at the beam–column junction using an L-shaped bracket and gusset-plate assembly, resulting in two perpendicular anchor groups. Due to geometric constraints and the eccentric load path, these groups are prone to uneven force distribution, especially under seismic loading where anchors experience cyclic, reversing tension and shear forces.

In seismic strengthening, where nonlinear behavior governs performance, the displacement of anchors plays a crucial role. Unlike conventional static design, retrofit applications must account for how anchor deformation affects force transfer, energy dissipation, and joint reliability. This makes accurate modelling of anchor group behavior and fixture flexibility essential to ensure the intended seismic performance of the intervention.

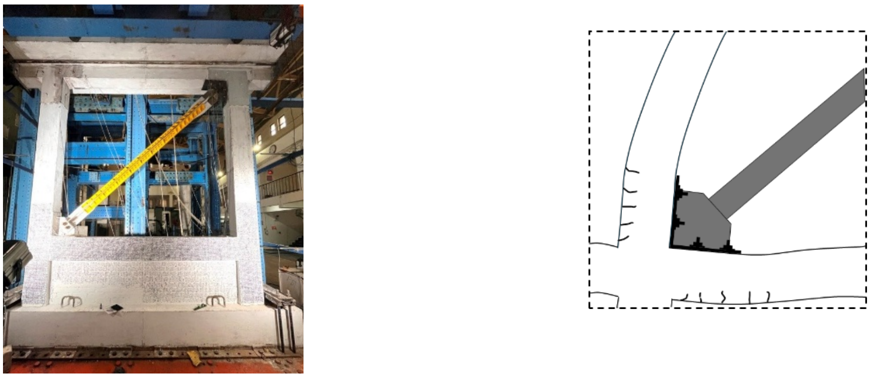


Fig. 6: Diagonal steel brace with its connection and deformed concrete frame

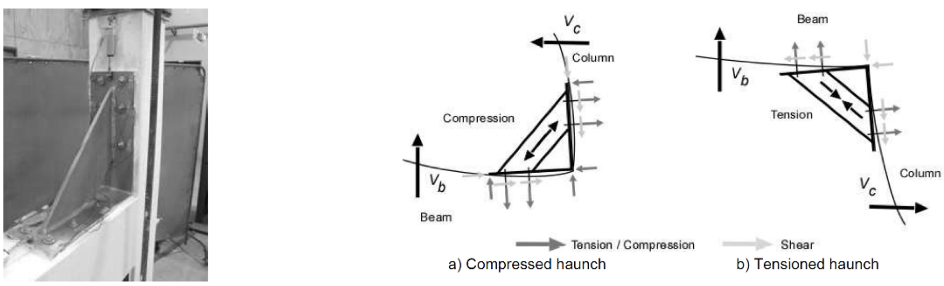


Fig. 7: Steel haunches for local strengthening of beam-to-column reinforced concrete frame and effect of flexural deformation of beam and column on the force distribution on the anchorage [9]

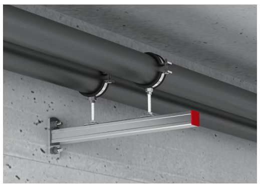
2.4 Non-structural and services fastening: high repetition, high impact

Non-structural connections are fastening points used to support HVAC (Heating, Ventilation, and Air Conditioning), piping, cable trays, lifts, and façade elements. Typical of these fastening are relatively thin baseplates and/or widely spaced anchor configurations that can show non-rigid behavior.

These connections are not part of the load-bearing structural system; however, they are safety relevant because their failure can endanger human life and create significant economic loss. They often include highly repetitive patterns and are installed in large quantities, making them a significant contributor to project cost and carbon footprint. Therefore, even small optimization gains can result in significant carbon and cost reductions.

A description of applications is summarized in Table 1 below to highlight potential benefit of the baseplate optimization.

Table 1: Examples of S2C connections for services fastening

Realistic modelling of services fastening is important for safety relevant applications and helps to deliver cost-optimized solutions in standardized and repetitive applications.		
Application	**Requirement**	**Benefits of optimization**
	Standardized solutions for fastening MEP (Mechanical, Electrical, and Plumbing) systems. Such connections are often subjected to tension load, but some applications require cantilever installation, therefore moment develops.	There is a need to account for flexibility of the baseplate and realistic modelling can help reduce the size of the baseplate, enabling cost efficient solutions.
	Racking is an application where high tension and shear loads need to be transferred down to concrete slabs. They require standardized solutions and repetitive applications.	Thanks to accurate modelling, the reduction of baseplate size has the potential to reduce cost of the application, while preserving safety.
	Systems like aboveground storage tanks and power transformers in high voltage substations must have stable fastening to prevent uneven settling and corrosion.	For these core equipment, unaccounted deformations due to neglected flexibility can lead to failure of the connection, with catastrophic consequences.

3. Current practice for design of tension loaded anchors

The design of anchorages consists of **two fundamental steps**. First, the **forces acting on the individual anchors** within a group are determined. Second, the **resistances corresponding to each relevant failure mode** are calculated, and the minimum resistance among all failure modes, reduced by the appropriate partial safety factors, is taken as the design resistance of the anchorage. The forces acting on the anchors are verified against these resistances. The failure mode with the highest action-to-resistance ratio (utilization) governs the design of the anchorage [10].

For anchorages subjected to tensile loading, when failure occurs due to **steel failure or pullout failure** (excluding combined pullout and concrete cone failure), the resistance of an individual anchor is not affected by adjacent anchors or by the proximity of concrete edges. In such cases, the verification may be performed for the most heavily loaded anchor. The utilization ratio forces/resistance depends only on the magnitude of the acting force, which in turn can differ whether the rigid or flexible baseplate behavior is assumed.

In contrast, when the anchorage capacity is governed by concrete related failure modes, such as **concrete cone failure, combined pullout-concrete cone** (for bonded anchors) or **splitting failure**, the load bearing capacity is strongly influenced by anchor spacing, edge distances, and overall group geometry. In such cases, the verification must be carried out for the **entire anchor group** in accordance with the applicable design provisions. If the load distribution is calculated assuming a flexible baseplate, the resulting action forces may deviate significantly from those predicted under rigid-plate assumptions. Furthermore, the resistance models available in EC2-4 for group verification assume a linear distribution of forces among the anchors and are therefore implicitly valid only for anchorages connected to a rigid baseplate. When the flexibility of the plate cannot be neglected, the actual distribution of forces differs from the one calculated under the rigidity assumption and as a consequence the effective group resistance might decrease.

Enforcing strict baseplate rigidity to satisfy code prescription may lead to overly thick baseplates and potentially uneconomical connection designs. Designers are therefore often willing to accept a degree of baseplate flexibility, provided its impact on the anchorage capacity can be assessed. However, EC2-4 offers only qualitative guidance on what constitutes “sufficient rigidity” and does not provide quantitative criteria, threshold stiffness values, or simplified assessment methods to classify baseplate stiffness.

As a result, a fundamental inconsistency may arise in anchorage verification: actions and resistances are frequently evaluated under different baseplate assumptions. This mismatch can introduce ambiguity into the design process, complicate the interpretation of results, and require substantial engineering judgement to ensure consistent and safe design outcomes.

The aim of this section is to review the main steps involved in the design of anchorages and to highlight where methodological improvements are needed. First, commonly used approaches for determining the forces acting on anchors are examined, including both simplified methods that deliver linear force distributions and more advanced analysis strategies that account for the stiffness contributions of all connection components and can capture realistic non-linear force distributions. The importance of accurately modelling anchor stiffness for stiffness-based calculations is then discussed. Next, the definition of a rigid baseplate according to EC2-4 is presented, along with an overview of the practical methods engineers currently use to assess baseplate rigidity. The procedure for determining the concrete cone resistance according to EC2-4 is then summarized, and the key parameters influencing this resistance are introduced. Finally, the limitations of the current standard in considering realistic baseplate stiffness are discussed, establishing the motivation for developing an improved design strategy.

3.1 Calculations of loads on the anchors

Anchor forces in baseplate connections can be determined using two fundamentally different modelling philosophies: simplified methods where the baseplate is modelled as a rigid body and advanced methods that incorporate the flexibility of the baseplate. The choice between these approaches can have a significant influence on the predicted anchor forces, the resulting required baseplate thickness, and the overall design economy.

According to EC2-4, the load distribution to the fasteners may be calculated analogous to the elastic analysis of reinforced concrete. This approach is fully consistent with the Bernoulli hypothesis, which assumes that plane sections remain plane under the action of axial forces and bending moments, resulting in a linear strain distribution. By adopting this rigid-body assumption and the corresponding linear strain profile, EC2-4 enables a simplified analytical determination of anchor tension forces.

Anchor design software PROFIS Engineering applies this approach when the Rigid assumption is selected under the load distribution menu (Fig. 8). In this mode, anchor tension and shear forces are calculated assuming a rigid baseplate, considering the baseplate's plan dimensions (length and width) but not its actual flexural stiffness or deformation. As a result, the anchor force distribution is governed entirely by rigid-body mechanics, rather than by the real structural behavior of the S2C connection.

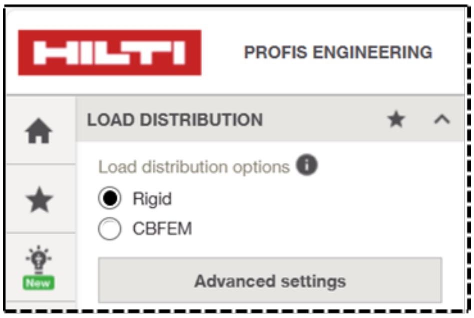


Fig. 8: Menu from PROFIS Engineering where the user can select the method for the calculation of load distribution on the anchors

To overcome the limitations inherent in the rigid body assumption, advanced methods have been developed to provide a more realistic representation of the load transfer within anchor groups. These strategies do not assume that the baseplate remains plane under loading. Instead, they explicitly account for the flexibility of the baseplate and all steel components, the deformability of the concrete support, and the (potentially non-linear) stiffness characteristics of the anchors. In these enriched models, anchor forces arise from the compatibility of deformations between the plate, the supporting concrete, and each individual anchor, rather than solely from simplified assumptions. As a result, **the load distribution within an anchor group is governed by the relative stiffnesses of all components**, allowing phenomena such as prying action, local plate bending, differential anchor stiffness, and interaction with the concrete surface to be captured realistically. These effects become particularly relevant for flexible baseplates, large load eccentricities, and configurations where the rigid-baseplate assumption of EC2-4 does not reflect actual structural behavior.

For anchorages not fully covered by existing standards, engineers may use three-dimensional finite element (3D FE) analyses. In these models, all components (steel parts, concrete, and anchors) are represented explicitly using finite elements. This modelling strategy is the most general and is commonly used in research, as it can simulate complex interactions otherwise neglected by simplified models. However, it is also computationally intensive, it requires specialized software and advanced expertise to preserve predictive accuracy and deliver reliable results. Therefore, it might not be the most common approach chosen in routine engineering practice.

To bridge the gap between simplified methods and full 3D FE modelling, the Component Based Finite Element Method (**CBFEM**) was introduced and implemented in PROFIS Engineering in 2018 [11]. In CBFEM approach the steel components (baseplate, profile, stiffeners) are modelled with finite elements, the concrete support is represented by compression-only springs, and the anchors are implemented as tension-only springs (Fig. 9).

This hybrid strategy combines the accuracy of FEM with the robustness of component-based modelling commonly used in engineering practice. In CBFEM, the load transfer between components is governed by their relative stiffness and it captures the influence of baseplate thickness and flexibility on the force distribution among the anchors. When compared to traditional methods based on simplified assumptions, the method allows simulation of the physical behavior of the whole connection, helping to deliver a more realistic and potentially non-linear distribution of forces among the anchors [12].

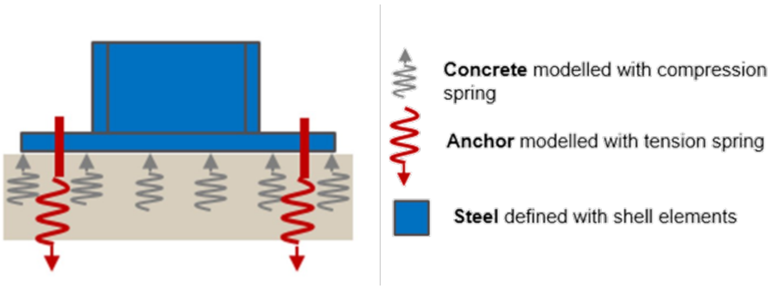


Fig. 9: Conceptual representation of the connection model as implemented in PROFIS Engineering for calculation of anchor load distribution with CBFEM.

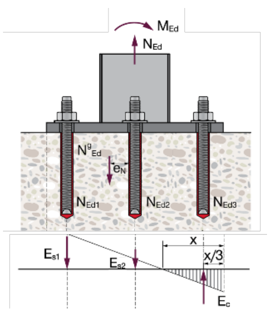
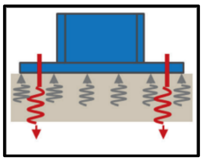
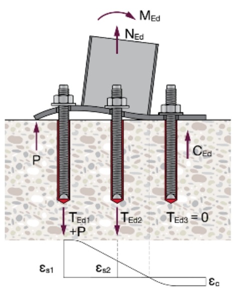
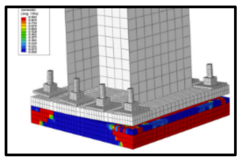
Within the stiffness-driven CBFEM framework, the resulting tension forces on the anchors depend on a number of parameters, including:

- the magnitude and combination of applied loads,
- the geometry and material properties of all connection components, and
- the stiffness characteristics of the anchors.

Overall, the selection of the modelling approach has a decisive impact on the predicted anchor forces and the resulting baseplate design. While rigid baseplate methods offer simplicity and alignment with EC2-4 assumptions, they may not adequately represent the behavior of flexible baseplates. Advanced stiffness-based approaches such as CBFEM provide a more realistic load distribution and support safer and more economical anchorage design.

Table 2 below summarizes the strategies discussed above to calculate anchor loads, highlighting the progression from rigid-body assumptions to stiffness-based methods.

Table 2: Summary of approaches used for calculating the anchor loads.

<table>
<tr><th>Rigid baseplate</th><th colspan="2">Flexible baseplate</th></tr>
<tr><td></td><td colspan="2">  </td></tr>
<tr><td>Strain compatibility
Methods that assume linear strains in the baseplate (rigid baseplate behavior only).
Calculations can be done by hand, like steel reinforcement in concrete sections (Bernoulli hypothesis = plane section under bending).</td><td>Spring modelling
Steel components are modelled with finite elements, anchors as tension-only springs and concrete as compression-only bedding.
Good balance between complexity and engineering accuracy.
Requires anchor stiffness as input.</td><td>3D FEM
Each component is modelled with finite elements (concrete, steel components, anchor).
The most accurate, but complex and time consuming. Requires advanced expertise to preserve predictive accuracy and deliver reliable results.</td></tr>
<tr><th>Linear anchor forces distribution</th><th colspan="2">Non-Linear anchor forces distribution</th></tr>
<tr><td>Method used in PROFIS Engineering
“Rigid”</td><td>“Calculation engine” of PROFIS Engineering
“CBFEM”</td><td>Recommended for complex cases and when other methods cannot capture physics.</td></tr>
</table>

3.2 Anchor stiffness: model and parameters characterization

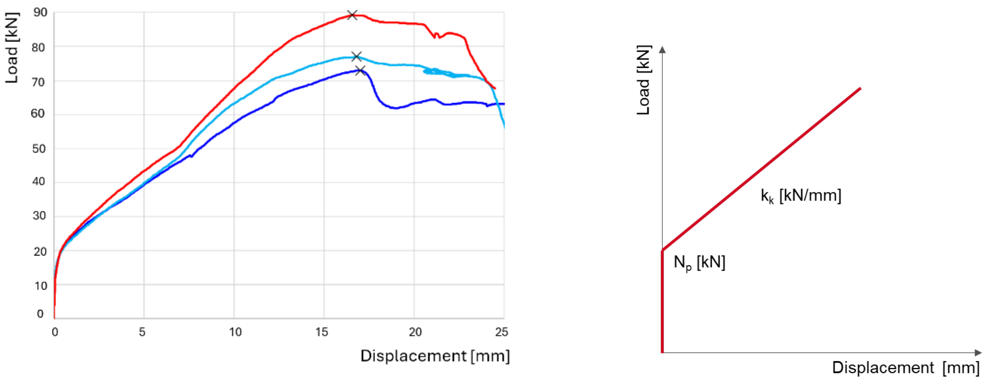
Design approaches accounting for non-linear force distribution (such as spring models) require explicit consideration of the baseplate stiffness and the fastener stiffness when evaluating anchorage behavior. It has been shown that for all parameters held constant, the anchor stiffness plays major influence on the performance of the anchorage [13]. Within the framework of spring modelling strategies, anchors may be represented using simplified linear elastic springs with constant stiffness, or by non-linear spring models whose stiffness varies with load and deformation, as shown in [14] and [15]. The choice of representation has direct implications for predicted connection behavior, particularly in applications where displacement control or dynamic response are critical to performance.

One of the principal open questions in applying displacement-based calculations is the selection of an appropriate model to represent anchor stiffness and the reliable characterization of its parameters. Anchor stiffness is influenced by multiple interacting mechanisms, including steel elongation, concrete deformation, and interface slip, which vary significantly among anchor types. As a result, oversimplified stiffness assumptions may lead to large discrepancies in predicted displacements, load distribution within anchor groups, and overall structural response.

Currently, there is no harmonized approach enforced by codes for characterizing anchor stiffness or defining the parameters of spring-based anchor models. When modeling anchors using FEM, assumptions regarding anchor stiffness often differ and are proprietary, which leads to differences in the predicted connection behavior. Analytical stiffness estimations based solely on basic mechanics can be conceptually oversimplified, and in some cases overly conservative, failing to capture the actual physical response of the anchor–concrete system.

The only available guideline that provides specific prescriptions for mechanical anchors is EAD330232-02 [16]. However, this document exhibits significant shortcomings, most notably the omission of anchor prestressing effects. Neglecting prestressing can lead to lower stiffness and underestimated anchor forces and, consequently, unconservative design outcomes. Unconservative outcomes may also arise for chemical anchors when load–displacement relationships derived from qualification tests, typically conducted without prestressing, are directly implemented in structural models, despite the fact that prestress is applied after installation.

These limitations of the assessment method proposed in the EAD330232-02, along with the consequences for the design outcomes, highlight the need for models that explicitly incorporate prestressing effects in a consistent and transparent manner. In practice, stiffness is a measured property that can be obtained from standardized tension tests. Within this domain, the response of the anchor can be represented using a bi-linear spring model, which provides a simplified yet physically meaningful approximation of the load-displacement curve (Fig. 10). This model divides the response into two linear segments: an initial (steep) stiffness branch followed by a secondary (reduced) stiffness branch to realistically capture how anchors deform under increasing load. This model may provide a practical compromise: it is simple and computationally efficient.



a) Load-displacement curve of a mechanical anchor

b) Bi-linear load-displacement model

Fig. 10: Idealization of the load-displacement curve of a mechanical anchor in uncracked concrete to the bi-linear load-displacement model.

For the definition of the model, two parameters need to be assessed:

- Prestress N_p, which is the contribution due to the pre-stress force caused by the applied installation torque (considering relaxation)
- Stiffness k_k, which is the slope of the load-displacement calculated between two defined load levels

Stiffness of Hilti anchors, as implemented in PROFIS Engineering CBFEM, is characterized by a **bi-linear spring model** that includes prestress contribution to initial stiffness to realistically model anchor performance in uncracked concrete. Model parameters have been established by reviewing existing data, newly conducted tests, and using a refined characterization approach that builds upon the requirements of the EAD330232-02 while addressing its limitations.

3.2.1 Assumptions of cracked and uncracked concrete for stiffness derivation

Concrete exhibits very low tensile strength, therefore, cracking is inevitable under service conditions, whether caused by external loading or by restrained movements. Consequently, anchors installed in initially uncracked concrete are highly likely to intersect a crack once the concrete member undergoes cracking [11]. Furthermore, due to long-term relaxation, the pre-stress is typically almost completely lost when a crack forms or opens at the anchor location. As a result, the load-displacement behavior in cracked concrete can be significantly different (much lower initial stiffness, that is) from the uncracked case. The degree of stiffness reduction also depends on the relationship between anchor type, anchor diameter and crack width.

Since the presence of cracks substantially reduces the performance of most fastening systems, it is generally required to assume that anchors will be located in cracked concrete. This is conservative because the concrete resistance in cracked concrete (under static loading and assumed crack width of 0.3 mm) is about 30% lower than uncracked concrete.

On the action side, however, the situation is reversed. Assuming cracked concrete could be unconservative if the anchors are not located in cracks, because the lower stiffness would lead to reduced anchor forces (lower utilization). Since it cannot be assumed that a crack exists or develops at every anchor location, **uncracked concrete conditions should be considered when calculating the anchor forces**. For this reason and in alignment with [17], Hilti has decided to use a conservative approach and use the anchor stiffness evaluated in uncracked concrete conditions also for the cracked concrete.

3.3 Definition of close-to-rigid baseplate

The procedure for the design of fastenings as outlined in EC2-4 is based on the assumption of rigid baseplate behavior. This assumption is considered valid provided that: (i) the baseplate remains in the elastic range under design actions, exhibiting a linear stress-strain relationship, and (ii) baseplate deformations are small compared to axial displacements of the anchors. The latter is valid only if the flexural rigidity of the fixture is large relative to the axial stiffness of the anchors, such that, at the design load level the deformation of the baseplate is small compared to the anchor axial displacement [18] .

Beyond the qualitative prescriptions given in EC2-4, baseplate flexibility checks are therefore required to be performed. The von Mises stress criterion is commonly used to check if the stress level on the plate $\sigma_{steelBP}$ stays below the yielding limit $\sigma_{y,d}$

$$\sigma_{steelBP} \leq \sigma_{y,d} \tag{1}$$

Although not part of EC2-4 itself, the use of a von Mises yield condition as in Equation (1) is aligned with EC3-1-5 [3] material modeling provisions for steel connections (e.g., finite-element-based verification of plate behavior).

Further baseplate flexibility checks can be performed explicitly by assessing deformation compatibility between the baseplate and the anchors. This may be achieved by comparing the elastic bending deformation of the baseplate at the anchor locations with the corresponding anchor axial elongation, or by means of stiffness-based numerical models that account for the relative stiffness of all connection components. Only when baseplate deformations are demonstrated to be negligible with respect to anchor displacements, the rigid-baseplate assumption and the associated linear force distribution can be considered valid.

Because anchor axial displacement is inversely related to anchor stiffness, a plate of fixed thickness may satisfy the deformation requirement for some anchor types but fail it for stiffer anchors. Current design approaches generally do not calculate fixture bending deformation, meaning this second criterion is often not evaluated in practice.

In typical engineering practice, only the plastic strain (elasticity) check is applied when evaluating baseplate rigidity. However, this approach is insufficient for validating rigid-plate behavior because:

- Stiffness is not directly related to yield strength
- Fulfilling the elasticity condition does not guarantee negligible deformation
- EC2-4 requires both elastic behavior and negligible deformation to justify linear strain distribution

Research studies have shown that these criteria alone do not ensure that the baseplate behaves as rigid and therefore do not guarantee the validity of CCD-based linear anchor force distribution assumptions [19].

The common assumption of a perfectly rigid baseplate is rarely justified in professional practice. In fact, the required thickness for the rigid baseplate can be so great that it would be impractical for many applications. Therefore, when EC2-4 conditions cannot be fulfilled, deformation due to bending should not be neglected and accurate modelling of all components contributions should be considered when designing S2C connections.

Referring to the paper by Fitz et al. [11], PROFIS Engineering has recommended a 10 % anchor forces deviation as acceptable to consider the baseplate close to rigid. If the deviation is above 20%, a second warning message is displayed for the user to confirm that they consider the baseplate as rigid and perform the anchor design verification as per EC2-4. In this criterion, the anchor forces are calculated for both the rigid baseplate assumption and for an arbitrary thick (flexible) baseplate. The deviation between the corresponding forces is determined, and the **highest difference** in anchor forces is considered for assessing the deviation from the rigid hypothesis as below

$$\Delta_{max} = \max_i \left| \frac{N_i^{flex} - N_i^{rig}}{N_i^{rig}} \right| \leq \alpha_{lim} \tag{2}$$

where N_i^{flex} and N_i^{rig} are the tension forces on fastener i calculated assuming non-rigid (flexible) baseplate behavior and rigid baseplate behavior, respectively, and α_{lim} is a limiting value.

This straightforward criterion can provide reliable results when all anchors develop comparable tension forces, such as in symmetric anchorages (1x2 or 2x2 group) subjected to concentric tension load, and where prying effects remain limited. In these situations, a deviation of no more than 10% is recommended as acceptable to assume that the baseplate behaves as sufficiently rigid body, allowing the designer to apply the EC2-4 formulation for anchor resistance.

However, this criterion alone might not be effective for assessing the overall linear distribution of forces among all anchors when bending governs the behavior. Under significant bending moments, the rigid assumption can predict very small or even zero tension forces in some anchors. For such anchors, the relative error becomes extremely large, even if their contribution to the group resistance is negligible. As a result, the deviation measure in Equation (2) may drive the design toward unnecessarily thick baseplates, based on anchors that do not meaningfully influence the load-carrying mechanism of the group. Engineering judgement of the designer is needed to rule out these anchors from the decision.

3.4 Concrete cone resistance according to EC2-4

Concrete cone failure, also referred to as concrete breakout in tension, is among the most critical failure modes considered in the design of anchors in concrete. Under tensile loading, the concrete surrounding an anchor can fracture in a brittle manner, forming a cone-shaped breakout surface extending from the anchor head or bonded zone to the concrete surface. While pullout failure, governed by local bearing or bond mechanisms, can also occur, it typically exhibits a more progressive response and can usually be prevented through adequate embedment and detailing. In contrast, concrete cone failure is controlled by the concrete substrate, offers limited warning before failure, and therefore often governs anchor design.

Concrete cone group failure occurs when multiple anchors loaded in tension form overlapping breakout cones that combine into a single, enlarged failure surface. Unlike the isolated cone of a single anchor, a group cone reflects the collective interaction of anchors, resulting in a reduced effective breakout area and often a lower total resistance than the sum of individual capacities (Fig. 11).

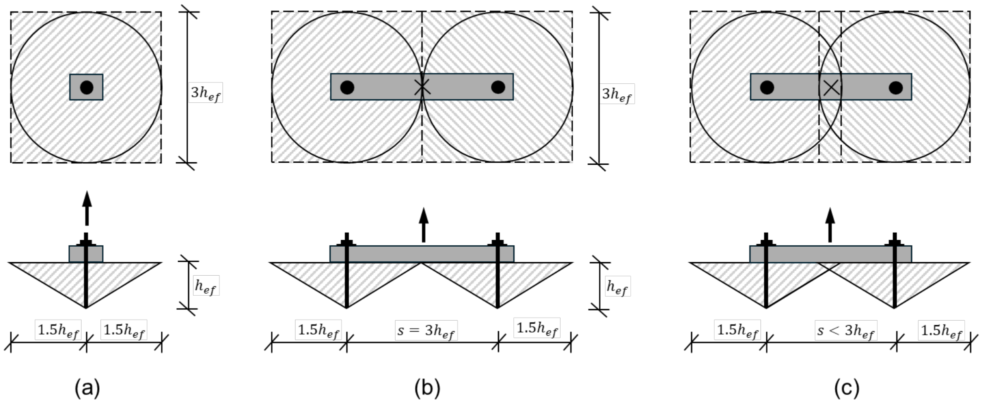


Fig. 11: Idealized concrete cone failure surface of a tension loaded (a) single anchor, (b) anchor group with spacing $s = 3h_{ef}$, (c) anchor group with spacing $s < 3h_{ef}$.

EC2-4 incorporates this behavior through the Concrete Capacity Design (CCD) method, which forms the foundation of its concrete breakout resistance calculations. The CCD method estimates the ultimate concrete breakout resistance of an anchor group as

$$N_{Rk,c} = N^0_{Rk,c} \frac{A_{c,N}}{A^0_{c,N}} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N} \cdot \psi_{M,N} \quad (3)$$

where

$$N^0_{Rk,c} = k_1 \cdot \sqrt{f_{ck}} \cdot h_{ef}^{1.5} \quad (4)$$

is the characteristic concrete cone resistance of a single anchor not influenced by concrete edges or adjacent anchors. Here, k_1 is the coefficient given in the corresponding European Technical Assessment of the product with different values for cracked and uncracked concrete conditions, h_{ef} effective embedment depth in mm and f_{ck} characteristic cylinder compressive strength in N/mm^2.

In Equation (3), $A^0_{c,N} = 9(h_{ef})^2$ is the reference projected area of a single anchor located sufficiently distant (i.e., with edge distance $c_{cr,N} = 1.5\, h_{ef}$), while $A_{c,N}$ is the actual projected area for the group considering the overlapping area of adjacent anchors and all edge distances.

The factors ψ in Equation (3) account for the influence of free edges and introduce information on load distribution effects within the group.

Although EC2-4 provides a simplified method that assumes linear force distribution among anchors (subject to fixture rigidity), experimental evidence shows that breakout failure often initiates at the most highly stressed anchors, typically those closest to edges or those with least confinement. When uneven load sharing is expected, such as when the fixture is not sufficiently rigid, or when eccentricity is present, the simplified linear distribution assumption may become less reliable. In these cases, advanced modelling approaches are required not only to predict anchor forces but also to correctly evaluate the group breakout resistance, as load distribution directly influences the shape and size of the effective breakout surface.

3.5 Gaps in current design method for S2C connections

Previous sections have described the implications of neglecting baseplate flexibility in the design of S2C connections, highlighting how this assumption can influence anchor forces, governing failure modes, and ultimately the safety and economy of the anchorage design. While specifiers nowadays can fully benefit from realistic calculations of anchor forces, there is a need to bridge the gaps in the current design strategy.

On the one hand the rigid hypothesis can lead to underestimation of anchor forces, on the other hand the resistance calculated from EC2-4 formulations can deliver overestimation of the anchorage resistance due to not accounting for the effect of the baseplate flexibility (e.g. number of anchors in tension might change, eccentricity). Therefore, calculating as rigid a baseplate that exhibits non-rigid behavior can lead to unconservative design results, even when the deformation of the baseplate remains elastic. This is because the reduction of the lever arm and potential prying forces are not adequately considered.

The flowcharts shown in Fig. 12 summarize the procedure for determining anchor forces and the anchor resistances, whereas outlines the limitations associated with neglecting baseplate flexibility in the design process and the corresponding implications. The limitations of the current design method in considering realistic baseplate stiffness have been discussed in this section and summarized in Table 3, establishing the motivation for developing the improved design strategy.

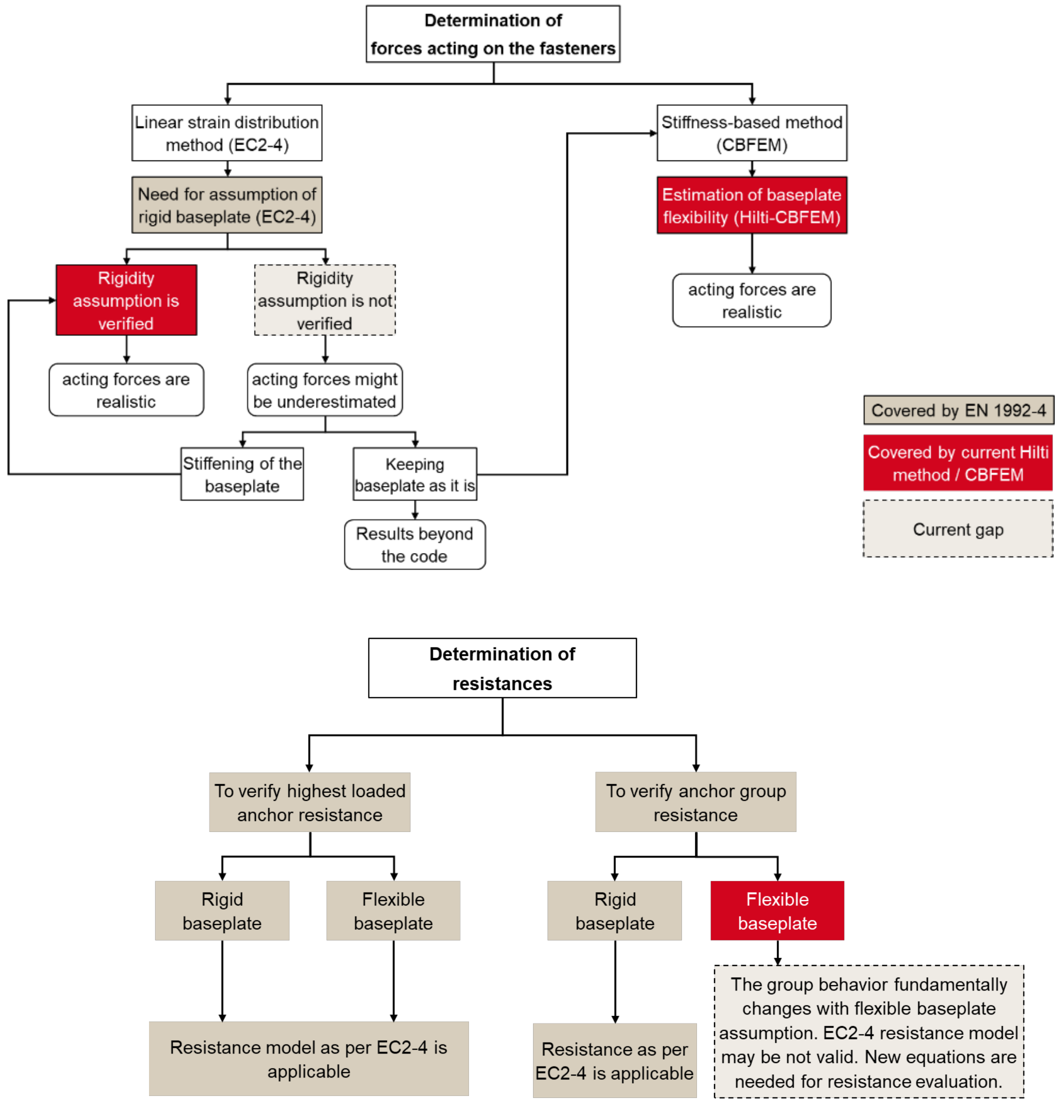


Fig. 12: Flowcharts representing the design procedure for the design of anchorages.

Table 3: Summary of limitations associated with neglecting baseplate flexibility in the design process and the corresponding implications.

Mismatch between assumptions for Action and Resistance calculations	• **Actions** (anchor forces) are often computed assuming flexible baseplate behavior (e.g., CBFEM). • **Resistances** in EC2-4 are based on rigid baseplate behavior. • This inconsistency creates ambiguity and requires engineering judgement.
Lack of quantitative criteria for "close-to-rigid" baseplate definition	• EC2-4 states that baseplates must be "sufficiently rigid" but provides no numerical limits, formulas, or thresholds. • Designers cannot quantify how much flexibility is acceptable, nor how much deviation is still within code intent.
Rigid baseplate assumption may lead to overly thick, over-engineered baseplates	• Because resistance models were calibrated for rigid plates, ensuring strict rigidity may require excessively thick plates. • Designers often accept some flexibility but lack guidance to evaluate its impact reliably.
Group failure modes depend on geometry and baseplate stiffness	• For concrete-related failure modes (cone, splitting, combined pullout-cone), resistances depend on anchor spacing, edge distances, and plate stiffness. • Rigid-plate-based resistance models may not reflect the real behavior of flexible baseplates.
Limited guidance on verifying anchor groups under flexible baseplate behavior	• EC2-4 resistance models assume linear force distribution, valid only with rigid baseplates. • When the baseplate is flexible, actual load distribution can differ significantly, yet the code offers no alternative method.
Need for engineering judgement and potential for inconsistent designs	• Because the code lacks quantitative criteria and mixes flexible and rigid assumptions, engineers must rely heavily on judgement. • This increases the risk of inconsistent, overly conservative, or potentially unconservative designs.

4. Experimental insight toward new group resistance modelling

As discussed in the Section 3, current design provisions, such as those in EC2-4, generally assume idealized rigid baseplate behavior and linear load distribution among anchors; however, many practical S2C connections use relatively thin or flexible baseplates that deviate significantly from these assumptions. Such deviations influence anchor load sharing and plate deformation, ultimately altering the shape and capacity of the concrete breakout body of post-installed anchor groups. To better understand these interactions, experimental tests were performed in Hilti's accredited testing laboratory to compare rigid and flexible baseplate configurations under otherwise identical conditions. By observing how differences in plate stiffness alter breakout geometry, and anchor force distribution, the tests provide valuable insight into failure mechanisms that are not captured by simplified rigid plate models. The findings help clarify the limitations of current design assumptions and support the development of more realistic models for S2C connections.

The anchor group was arranged in 3x2 configuration and consisted of six bonded anchors with threaded rods of size M16 and an embedment depth of 100 mm. These anchors were connected to a baseplate of size 380 mm × 230 mm, tested in two thicknesses: 80 mm, representing a theoretically rigid baseplate, and 15 mm, representing a flexible baseplate able to deform but not yield under the applied load. Both were made of S355 steel. As the attachment and loading point, HEB100 profiles with steel grade S355 were welded to the baseplates. The anchorage has been loaded concentrically in tension, ensuring that all anchors were subjected to pure tension and eliminating additional influences such as external eccentricities. The anchor group was installed near the concrete edge to allow clear visual observation of the breakout bodies both from above and from the side (see Fig. 13).

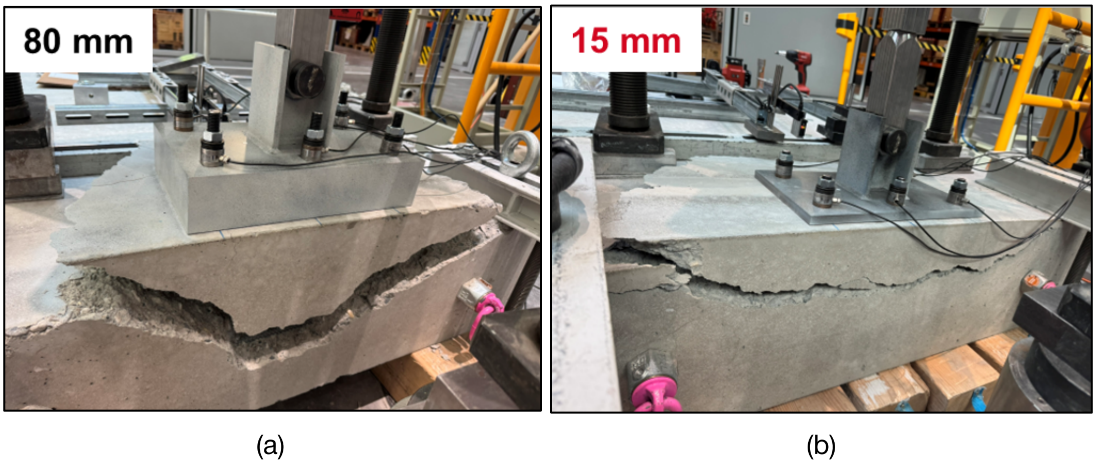


Fig. 13: Concrete breakout bodies for (a) the rigid baseplate with 80 mm thickness and (b) the flexible baseplate with 15 mm thickness

Fig. 13 shows the altered shape of the concrete cone breakout bodies observed in anchorages with rigid and flexible baseplates. In the rigid plate configuration (Fig. 13a), the anchorage fails suddenly producing one large, well-defined breakout body; crack initiation occurred almost simultaneously at all anchors, producing a uniform, symmetric breakout body consistent with the classical concrete cone model. In contrast, the flexible plate (Fig. 13b) allows the formation of primary cracking at the middle anchors, where plate deformation and anchor forces are highest; these cracks then propagate outward toward the edge anchors, resulting in a more progressive and irregular failure. These tests showed that the changes in load distribution significantly influence the behavior and the concrete cone resistance of the anchor groups.

The tests confirmed that the resistance of the anchorage with a flexible baseplate is significantly reduced compared to that with a rigid plate (Fig. 14a). This is because, in the case of the anchor group with a rigid plate, all six anchors are activated simultaneously and carry approximately the same amount

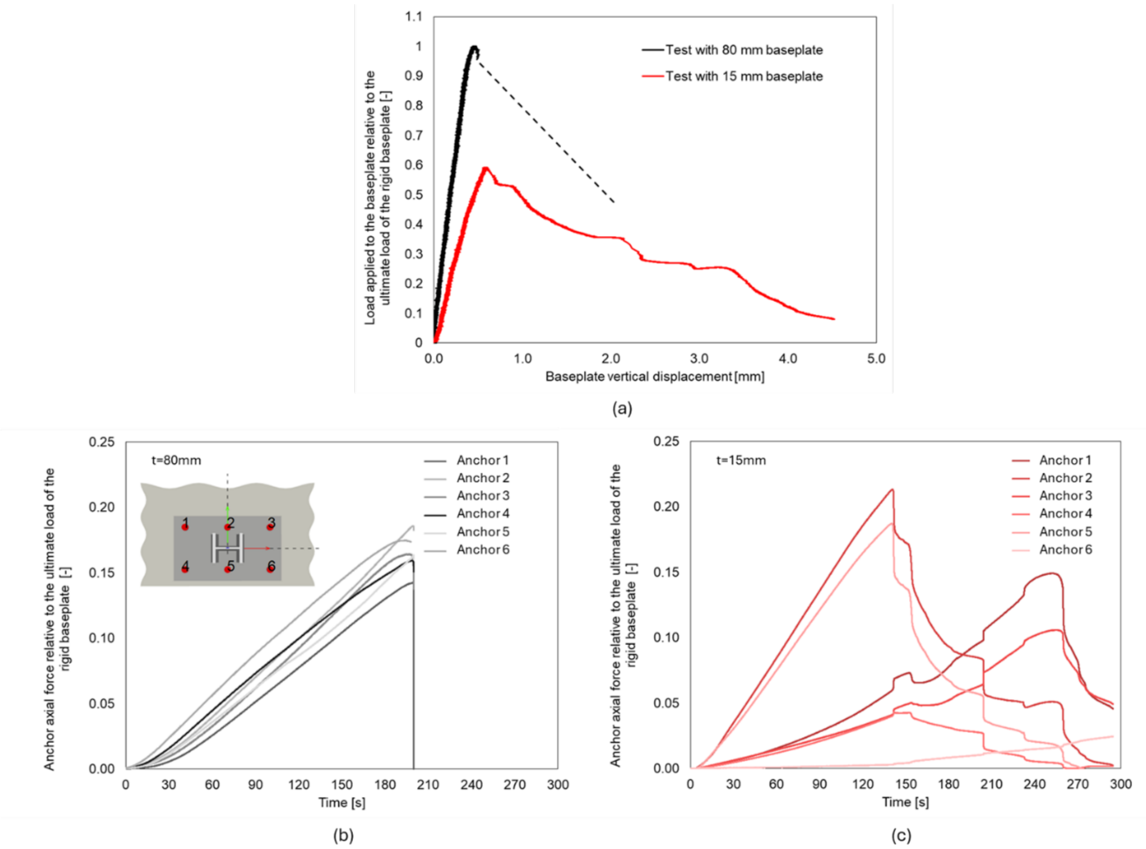


Fig. 14: (a) Load-displacement curves for the rigid baseplate (80mm thick) and the flexible baseplate (15 mm thick); variation tension force on the anchors over time for (b) rigid baseplate and (c) flexible baseplate

of tension. This behavior is illustrated in the load–time diagram in Fig. 14b. In contrast, the anchors in a flexible plate are not loaded equally (Fig. 14c). As the plate bends under increasing tension load, the anchors closest to the load application (middle row) become active first, while the outer anchors engage only after additional displacements. Because the activation of the anchors farther from the load application is delayed, their contribution to the overall group resistance is reduced.

The comparison of the load–displacement curves in Fig. 14a for the tests with rigid and flexible baseplates further confirms that the load distribution changes: the stiffness (load/displacement ratio) of the group with the flexible plate is lower because the outer anchors are not fully utilized before reaching the ultimate load.

In summary, calculating the realistic load distribution among the anchors of a group is essential and represents the state of the art, e.g., using FEM, such as PROFIS Engineering CBFEM. Yet, when calculating the anchor group resistance, this behavior is not accounted for in existing design codes and guidelines such as EC2-4, which continue to rely on simplified assumptions such as rigid plate behavior and uniform/linear load sharing. The experimental findings presented here clearly demonstrate the limitations of these assumptions and underline the need for more realistic modelling approaches that capture the interaction between baseplate flexibility, anchor stiffness, and concrete breakout behavior. These insights provide a foundation for advancing future design methods toward greater accuracy and structural reliability.

5. Hilti method for the design of anchors

Supported by both experimental and conceptual evidence, this section introduces the new Hilti design method intended to address the limitation identified in the existing design procedure. **The new approach incorporates the physical mechanisms associated with baseplate flexibility while remaining compatible with established design workflow.**

This section presents the underlying assumptions, formulation, and implementation of the new design method. Within the new Hilti design framework, the calculation of anchor force distribution is performed using the CBFEM approach as implemented in PROFIS Engineering. As described in Section 3.2, the experimentally derived anchor stiffness is used in the analysis and serves as key input for the anchor force calculations. Together with **the new analytical baseplate rigidity checks and the updated resistance model**, the new Hilti anchorage design strategy can provide a more realistic representation of anchorage behavior in cases where baseplate flexibility influences the S2C connection response.

5.1 Criteria to check baseplate close to rigid behavior – Analytical assessment method

The qualitative definition of a sufficiently stiff baseplate used by the codes requires that the deformation of the baseplate is small compared to the anchor displacements, in addition to the requirement that the baseplate should remain linear elastic [4]. This requirement indirectly ensures that the forces among the anchors of the group are distributed in a linear manner, which is the primary requirement to apply the CCD method for design of anchorages.

An analytical assessment method, formulated as a set of criteria to evaluate whether the tension force distribution for an anchorage can be considered linear with a given anchor pattern, baseplate, attached profile and load case, was proposed by Bokor et al. [19]. A necessary precondition for applying this method is that the baseplate remains elastic and does not yield. Other than that, the following three criteria need to be fulfilled at the same time [20].

5.1.1 Criterion 1. Assessing the deviation of forces in the highest loaded anchor

In this criterion, the anchor forces are determined for both the rigid and the flexible baseplate assumptions, and the evaluation focuses only on **the highest loaded anchor**. The deviation from the rigid baseplate hypothesis is then calculated as below

$$\Delta_{max} = \left| \frac{N_{max}^{flex} - N_{max}^{rig}}{N_{max}^{rig}} \right| \leq \alpha_{lim} \tag{5}$$

where N_{max}^{flex} and N_{max}^{rig} are the tension force in the highest loaded anchor calculated assuming a given fixture thickness (flexible) and with rigid fixture, respectively. This criterion ensures that the highest loaded anchor is not overloaded by more than a certain threshold, for example 5%. It is consistent with EC2-4, where for failure modes such as steel failure or pullout failure, verification is carried out for the most highly loaded anchor within the group.

However, this criterion alone might not be efficient for assessing the overall linear distribution of forces among all anchors, as it considers only a single anchor. In addition, depending on anchorage configuration and loading conditions, the anchor with the highest tension force under the rigid baseplate assumption might not be the same anchor that is the most highly loaded when baseplate flexibility is taken into account. Strictly speaking, Equation (5) should be applied to the same anchor in both the rigid and flexible analyses to ensure a meaningful comparison. Therefore, in such cases, the anchor to be assessed is defined as the one carrying the highest tension force under the rigid baseplate assumption.

5.1.2 Criterion 2. Assessing the deviation of resulting anchor forces and their sum

Depending on the anchorage configuration, baseplate thickness, geometric dimensions, and applied loading, the baseplate may pry against the concrete surface, resulting in additional tension forces developing in the anchors. As a consequence, the sum of all anchor forces can exceed the total applied tension. The effect of prying for a given baseplate thickness can be evaluated by comparing the total anchor tension obtained with the flexible baseplate assumption to the corresponding value computed under the rigid baseplate hypothesis. In this case, the deviation is calculated as follows

$$\Delta_{sum} = \left| \frac{\sum_{i=1}^{n} N_i^{flex} - \sum_{i=1}^{n} N_i^{rigid}}{\sum_{i=1}^{n} N_i^{rigid}} \right| \leq \alpha_{lim} \tag{6}$$

Equation (6) is intended to ensure that the anchor group designed with given baseplate thickness is neither overloaded nor underloaded relative to the same anchorage analyzed assuming rigid baseplate behavior. This criterion provides a smeared, global verification of the anchor group forces, as it is based on the deviation in **the sum of all anchor tensions**. However, this criterion only evaluates the total tension in the anchor group and does not reflect how the forces are distributed among individual anchors. A baseplate may satisfy the total-force requirement while still producing a non-linear or uneven force distribution, which differs from the linear behavior expected under the rigid baseplate assumption.

5.1.3 Criterion 3. Assessing the deviation from linearity of anchor force distribution

While the evaluation of the highest loaded anchor (criterion 1) and the comparison of the total anchor group forces (criterion 2) provide useful indicators of the baseplate behavior, these criteria alone do not ensure the tension forces across individual anchors follow the expected linear distribution associated with a rigid baseplate. In systems where the baseplate exhibits significant flexibility, the tension pattern can become highly non-linear, even if both checks above are satisfied.

A practical way to evaluate linearity is to compare the slope of the tension diagram across the anchor group. Such slope assessment can be based on an anchor pair that carry non-negligible tension in the rigid model. First, the force difference between the highest and the lowest loaded anchors is calculated assuming the rigid baseplate hypothesis, $N_{max}^{rigid} - N_{min}^{rigid}$. Second, the force difference is calculated between the same anchors with a given baseplate stiffness, $N_{max}^{flex} - N_{min}^{flex}$ (flexible assumption).

The ratio between these two differences is then calculated as below

$$\Delta_{slope} = \left| \frac{N_{max}^{flex} - N_{min}^{flex}}{N_{max}^{rigid} - N_{min}^{rigid}} - 1 \right| \leq \alpha_{lim} \tag{7}$$

Here Δ_{slope} measure how much the slope of the tension diagram deviates from the rigid case. Satisfying this equation ensures that the forces on the fasteners are distributed such that a concentric tension load applied to the fixture will resolve into almost equal axial tension in all fasteners, and an eccentric tension or bending moment applied to the fixture results in an almost linear load distribution.

The threshold value (α_{lim}) for all three criteria proposed in the work of Bokor et al. [19] is based on a limited number of calculations, assessing the convergence of deviations of all three criteria in function of the baseplate thickness. Considering the investigated cases, a threshold value such as α_{lim}= 10% seems reasonable, however, for more strict evaluations, 5% or smaller value can be recommended.

The three analytical criteria are now implemented in PROFIS Engineering to check how closely a baseplate satisfies the assumption of rigid behavior. In this approach, **the threshold varies with utilization**, allowing it to be applied in a more or less stringent manner depending on the design demand. When the design utilization under the assumption of rigid baseplate exceeds 90%, a threshold of 5% value is applied; satisfying this limit is taken as indication that the baseplate behaves as close-to-rigid. For cases where the utilization is below 90%, a less restrictive threshold of 10% is used [17]. Note, however, that these threshold values serve as recommendations; the final classification of the baseplate as “close-to-rigid” or “flexible” ultimately relies on the engineering judgment of the user.

5.2 New resistance model to account for flexibility in the group concrete cone capacity

As discussed in previous sections and experimentally validated in Section 4, baseplate flexibility has a significant influence on the force distribution within anchor groups. When the baseplate does not behave as a rigid body, the tension forces acting on individual anchors may differ substantially from those predicted by conventional design approaches that assume rigid baseplate behavior. This deviation arises because realistic baseplate deformations can cause certain anchors to attract higher tension forces compared to the uniform or idealized distributions typically derived under rigid assumptions. Depending on the design configuration, this effect may lead to a reduction in the overall anchorage resistance relative to the resistance calculated using the Concrete Capacity Design (CCD) method, which implicitly assumes a rigid baseplate. When the actual flexibility of the baseplate and the surrounding load-transfer mechanism are considered, the increased tension in one or more anchors can govern the anchorage capacity and may trigger tension-related failure modes earlier than predicted by simplified rigid-baseplate models.

These findings highlight the need for a design methodology capable of capturing realistic anchor force distributions in a way that is both accurate and practical for engineering applications. Such a method must (1) determine the anchor force distribution while explicitly accounting for the stiffness contribution of all relevant components (including the baseplate, anchors, steel profile, stiffeners, and concrete substrate) and (2) incorporate a resistance model that reflects these realistic force distributions, particularly with respect to tension-governed failure modes.

The proposed Hilti method addresses this need by employing a Component-Based Finite Element Method (CBFEM), as implemented in PROFIS Engineering, to calculate the anchor force distribution for both rigid and flexible baseplates. Baseplate flexibility is then assessed using the new analytical criteria introduced in Section 5.2. Furthermore, building on [20], a new resistance model is proposed intended to integrate the influence of baseplate flexibility, enabling a more representative and reliable evaluation of anchorage performance.

The main steps of the Hilti Method, summarized in Table 4, are described in detail below.

The design procedure begins with the definition of geometry, loading, and boundary conditions for the connection. This includes the anchor layout, baseplate dimensions, steel profile characteristics, and concrete member properties.

Step1: Once the fundamental parameters are established, calculation of the anchor force distribution is carried out with CBFEM method as implemented in PROFIS Engineering. This method captures the interaction between components, including the baseplate deformation, anchor stiffness, and steel section, and produces a force pattern that reflects the actual load-transfer mechanism. Calculation of anchor forces $N_1^{flex}, N_2^{flex}, \dots, N_i^{flex}$ is carried out for the baseplate thickness as defined in the original design and anchor forces $N_1^{rig}, N_2^{rig}, \dots, N_i^{rig}$ are calculated by modifying the CBFEM model parameters to simulate the ideal rigid baseplate behavior. This approach allows both sets of anchor forces to be determined using the same modelling framework.

Table 4: Steps in Hilti method for design of anchorages accounting for flexible baseplate behavior

		Rigid Baseplate	Flexible Baseplate
Step 1	Calculations of anchor forces	$N_1^{rig}, N_2^{rig}, \dots, N_i^{rig}$	$N_1^{flex}, N_2^{flex}, \dots, N_i^{flex}$
Step 2	Check “close to rigid”	The specifier classifies the baseplate as flexible	
Step 3	Calculations of tributary areas	$A_1^{rig}, A_2^{rig}, \dots, A_i^{rig}$	$A_1^{flex}, A_2^{flex}, \dots, A_i^{flex}$
Step 4	Tributary area ratio for each anchor	$\psi_1 = \frac{A_1^{flex}}{A_1^{rig}},\ \psi_2 = \frac{A_2^{flex}}{A_2^{rig}}, \dots,\ \psi_i = \frac{A_i^{flex}}{A_i^{rig}}$	
Step 5	Coefficient Ψ_{flex}	$\Psi_{flex} = min[\psi_1, \psi_2, \dots, \psi_i]$	
Step 6	Concrete cone resistance flexible baseplate	$N_{Rk,c}^{flex} = \Psi_{flex} N_{Rk,c}^{rig}$	

Step 2: The method proceeds with the evaluation of baseplate rigidity using the analytical criteria as introduced in Section 5.2. These criteria determine whether the baseplate stiffness is sufficient to justify the rigid-plate assumption or whether flexibility effects must be explicitly considered. If the baseplate is deemed stiff enough to satisfy requirements of the EC2-4, the user can choose to proceed with the calculation of the resistance according to the code prescription. In cases where the analytical criteria indicate a deviation from the close-to-rigid assumption, the user can choose to proceed with the new resistance model. This model considers that critical anchors, which may attract higher tension due to localized deformation, govern the assessment rather than an averaged or idealized distribution.

Step 3: This step applies if the user decides to proceed with the new resistance model that incorporates the effect of flexible baseplate behavior. In this context, the objective is to quantify the difference in resistance between (a) a linear load distribution, corresponding to the conventional rigid-baseplate assumption, and (b) a nonlinear load distribution reflecting realistic anchor force redistribution obtained from flexible assumption. To achieve this, the method uses a **tributary areas approach**, which provides a simplified way of translating the non-linear anchor force pattern into an equivalent reduction factor applied to concrete cone resistance. A visual illustration of this concept is presented in Fig. 15, and a detailed numerical example is provided in Section 6.1.

In the tributary areas method, the projected concrete cone area $A_{c,N}$ of the anchor group is divided among the individual anchors. Each anchor is assigned a share of this area (its tributary area) based on the relative magnitude of the tension force it carries. Consequently, anchors experiencing higher tension forces are associated with larger tributary areas, while those with smaller forces receive proportionally smaller areas. The tributary areas are determined by evaluating the ratios of tension forces between neighboring anchors. By comparing anchor forces in pairs, the method partitions the overall projected area in a way that reflects the nonlinear distribution of tension within the group. This proportional assignment captures the fact that anchors attracting higher loads dominate the formation of the concrete breakout cone and therefore govern the reduced resistance.

Step 4: To account for the deviation from the rigid-baseplate assumption at the anchor level, the ratio of the tributary area attributed to each anchor is calculated separately for both the rigid and flexible baseplate cases as $\Psi_i = A_i^{flex}/A_i^{rig}$. By comparing these two area distributions, the method quantifies how much each anchor's effective contribution to the group resistance changes when realistic, non-linear force distribution is considered.

Step 5: The overall factor Ψ_{flex} is finally obtained by taking the minimum of all Ψ_i values. This governing value represents the anchor most adversely affected by the deviation from the rigid baseplate assumption.

Step 6: The factor Ψ_{flex} is applied to reduce the concrete cone breakout resistance of the anchor group, ensuring that the model reflects realistic force distribution associated to flexible baseplate behavior. The resistance $N_{Rk,c}^{flex}$ is obtained as

$$N_{Rk,c}^{flex} = N_{Rk,c}^{rig} \cdot \psi_{flex} \tag{7}$$

where $N_{Rk,c}^{rig}$ is the concrete cone resistance of the anchorage calculated with rigid baseplate assumption according to EC2-4.

It is worth emphasizing that if the baseplate is in fact rigid, the flexible baseplate modelling approach will deliver the same results as those obtained using the rigid baseplate assumption in EC2-4 design. This means there is effectively no downside to applying the new design strategy when the actual stiffness is uncertain: if the baseplate behaves rigidly, the results converge to the rigid solution; if it does not, the flexible model correctly captures the deviations.

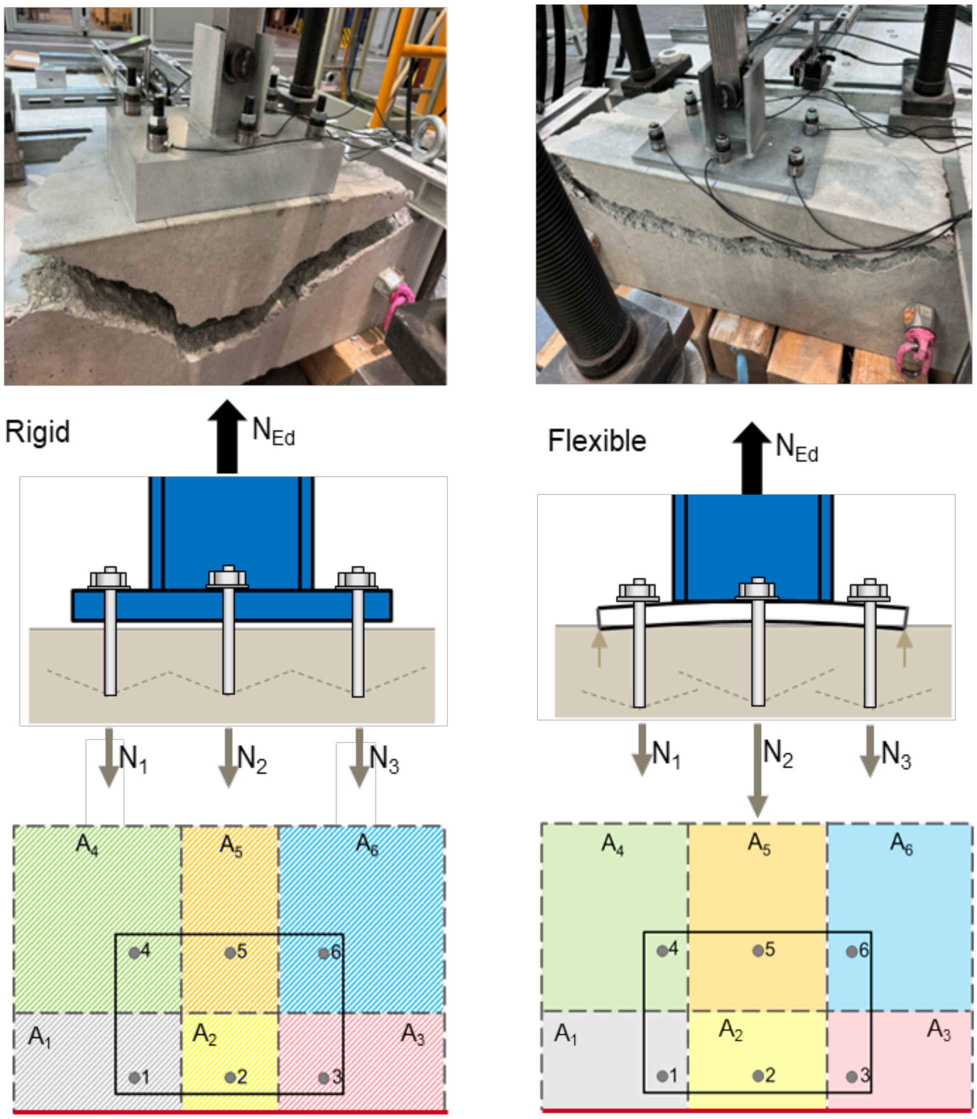


Fig. 15: Graphical representation of tributary areas associated to rigid baseplate behavior (linear forces distribution) and flexible baseplate behavior (non-linear forces distribution)

6. Design with PROFIS Engineering

Hilti's new design method for S2C connection design is now integrated into the cloud-based design software PROFIS Engineering. With this update, PROFIS Engineering includes additional features that help structural engineers model every component of a S2C connection while considering baseplate flexibility and its impact on both anchor forces and resistances. These new capabilities help to enable safer and more reliable S2C connection designs and streamline the workflow, especially when connection flexibility plays a critical role.

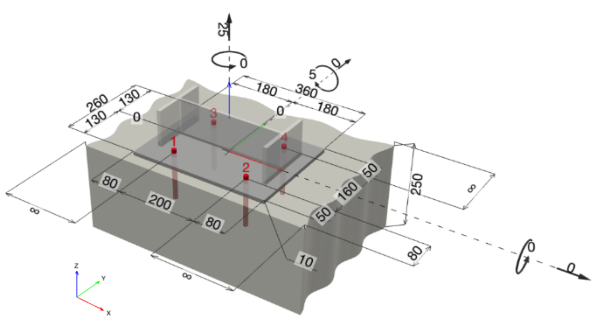
Some key benefits of using the latest enhancements in PROFIS Engineering include:

- **anchor load distribution calculations based on CBFEM** to realistically account for stiffness contribution of all connection components
- **new, research-based guidance for classifying baseplate behavior**, supporting engineers to better distinguish between rigid and flexible conditions
- advanced calculation with **new resistance model** for tension-governed concrete failure modes
- comprehensive verification of the **entire steel-to-concrete connection**, including anchor design for both rigid and flexible baseplate conditions
- a complete and transparent design report containing all required details for documentation and review

6.1 Design example: calculation of concrete cone resistance to account for baseplate flexibility

To illustrate the procedure described in Section 5.3, this section presents a design example created in PROFIS Engineering that examines in detail the new method for calculating the concrete cone resistance of an HST4-R anchor group when baseplate flexibility is taken into account. For clarity, the determination of the tributary area is demonstrated only for one anchor; the same methodology can then be applied to the remaining anchors in the group.

The S2C connection used as example is shown in Fig. 16, along with all design input. The baseplate is subject to tension load $N_{Ed} = 25\,kN$ and bending moment $M_{y,Ed} = 5\,kNm$.



Details of post installed anchors	
Anchors layout	2x2
Anchor type	HST4-R M12
Embedment depth	h_{ef}=125 mm
Details of baseplate and steel profile	
Baseplate dimension	360x260 mm
Baseplate thickness	t=10 mm
Profile	IPE 300
Material (plate and profile)	S235
Base material	
Concrete class	C25/30
Concrete condition	cracked
Base material thickness	250 mm

Fig. 16: Baseplate connection using HST4-R anchor installed at 80 mm distance from a free edge

As an initial step, the forces acting on the anchors are calculated using the CBFEM method for both rigid and flexible baseplate assumptions, and the results for tensile forces are summarized in Table 5.

Table 5: Summary of tensile forces acting on the anchors as calculated in PROFIS Engineering

	CBFEM modelling the plate as rigid	**CBFEM modelling the plate with t =10 mm**
	anchor forces [kN] (+tension, -compression)	anchor forces [kN] (+tension, -compression)
Anchor 1	14.661	17.131
Anchor 2	10.965	5.3150
Anchor 3	14.661	17.130
Anchor 4	10.965	5.3230

In PROFIS Engineering, the classification of the baseplate is evaluated by using the analytical criteria as described in Section 5.1 and results are summarized in a pop-up window as in Fig. 17. At this point the user can review the key information provided and confirm whether the rigid baseplate assumption is valid or whether a flexible baseplate model should be adopted for the analysis.

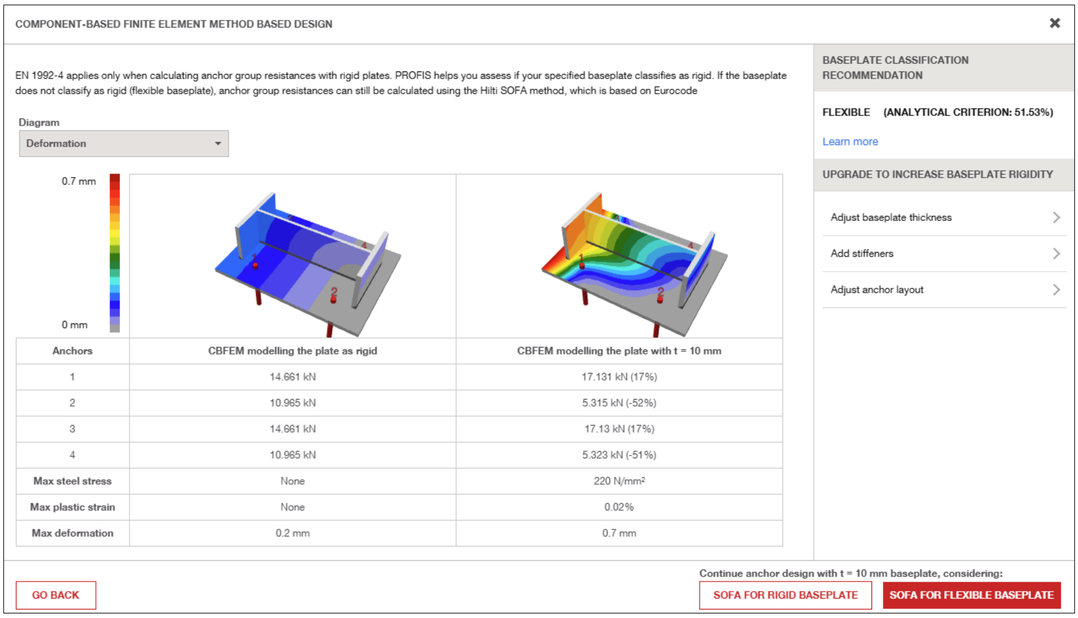


Fig. 17: Pop-up window available in PROFIS Engineering with summary of forces acting on the anchors as calculated with the CBFEM method by assuming the assumption of rigid baseplate and considering the original design thickness t =10 mm, and results of analytical criteria for the classification of the baseplate.

Assuming flexible baseplate behavior, the designer proceeds with the calculation of the resistance of the anchorage with the new resistance model as included in the Hilti method. At this stage, the software performs the required calculations and generates the corresponding design output. For the sake of clarity, the detailed calculation of the influence areas, as described in Section 5.2, is presented in the following. Fig. 18 considers a generic group of 2x2 anchors with the definition of nomenclature. Note that in Fig. 18, c_1, c_2, c_3, c_4 are the distances of the anchors from the free edges $(x+, x-, y+, y-)$ (for the example in Fig. 16 $c_4 = 80\ mm$) and h_{ef} is the effective embedment depth of the anchors.

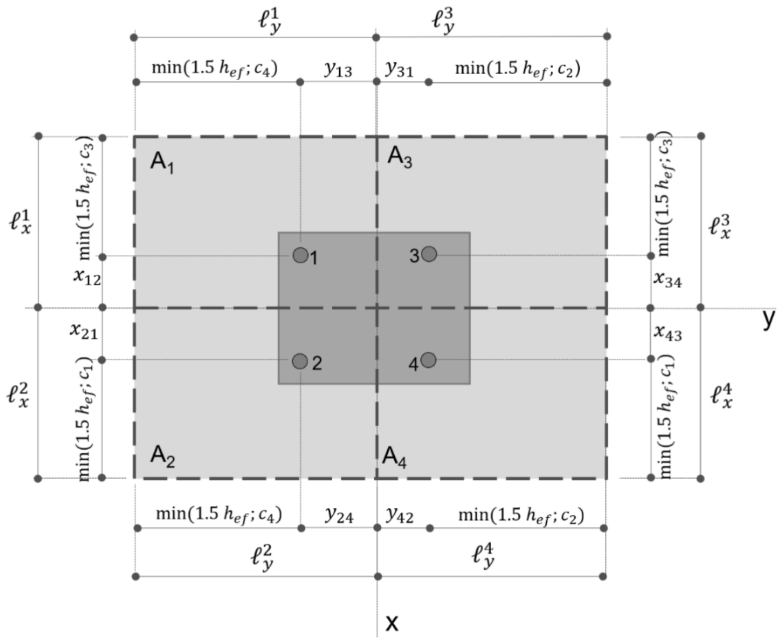


Fig. 18: Example of a 2x2 anchor group to demonstrate the tributary area approach and nomenclature used.

With reference to the anchor configuration in Fig. 18, the influence length $(x_{12}, x_{21}, \dots, y_{13}, y_{31}, \dots)$ are calculated for each anchor in both orthogonal directions x and y for rigid and flexible baseplate according to the formulae below.

The influence length in x direction can be calculated as

$$x_{ij} = 0.5\, s_{ij} + \left[1 - \left(\frac{N_{lower}}{N_{higher}}\right)^{0.75}\right]\left(1.5\, h_{ef} - 0.5\, s_{ij}\right) \leq 1.5\, h_{ef} \qquad \text{if } N_i > N_j \qquad (8a)$$

$$x_{ij} = 0.5\, s_{ij} - \left[1 - \left(\frac{N_{lower}}{N_{higher}}\right)^{0.75}\right]\left(1.5\, h_{ef} - 0.5\, s_{ij}\right) \qquad \text{if } N_i < N_j \qquad (8b)$$

$$x_{ij} = 0.5\, s_{ij} \qquad \text{if } N_i = N_j \qquad (8c)$$

The influence length in y direction can be calculated as

$$y_{ij} = 0.5\, s_{ij} + \left[1 - \left(\frac{N_{lower}}{N_{higher}}\right)^{0.75}\right]\left(1.5\, h_{ef} - 0.5\, s_{ij}\right) \leq 1.5\, h_{ef} \qquad \text{if } N_i > N_j \qquad (9a)$$

$$y_{ij} = 0.5\, s_{ij} - \left[1 - \left(\frac{N_{lower}}{N_{higher}}\right)^{0.75}\right]\left(1.5\, h_{ef} - 0.5\, s_{ij}\right) \qquad \text{if } N_i < N_j \qquad (9b)$$

$$y_{ij} = 0.5\, s_{ij} \qquad \text{if } N_i = N_j \qquad (9c)$$

where s_{ij} is the spacing between neighboring anchors i and j.

The influence lengths depend on the anchor force ratio of the adjacent anchors $r = N_{lower}/N_{higher}$. As the tensile force in an individual anchor increases, a larger effective concrete area is attributed to that anchor, resulting in a greater contribution to the resistance of the anchor group.

In this example, detailed calculations of tributary areas are shown for anchor 2 only and the results for all anchors of the group are summarized in Table 6.

Table 6: Details of calculation of tributary areas for anchor 2.

	rigid	flexible
$x_{12} = 0.5\, s_{12} + \left[1 - \left(\frac{N_1}{N_2}\right)^{0.75}\right]\left(1.5\, h_{ef} - 0.5\, s_{12}\right)$	117.1	151.1
$x_{21} = s_{12} - x_{12}$	82.9	48.9
$y_{24} = 0.5 \cdot s_{24}$	80	80
$y_{42} = s_{42} - y_{24}$	80	80
$A_2 = \ell_x^2 \cdot \ell_y^2 = \left[x_{21} + min\left(c_1; 1.5\, h_{ef}\right)\right] \cdot \left[y_{24} + min\left(c_4; 1.5\, h_{ef}\right)\right]$	43259	37791

Once the tributary area is calculated, the corresponding area ratio is given as

$$\Psi_{2,flex} = \frac{A_2^{flex}}{A_2^{rigid}} = 0.874 \tag{10}$$

Results of calculations for the other anchors are provided in Table 7 below.

Table 7: Summary of tributary area calculation for all anchors in the group.

	$A_i^{rigid} = \ell_x^i \cdot \ell_y^i$	$A_i^{flex} = \ell_x^i \cdot \ell_y^i$	$\Psi_{i,flex} = \frac{A_i^{flex}}{A_i^{rigid}}$
Anchor $i = 1$	48741	54182	1.112
Anchor $i = 2$	43259	37791	0.874
Anchor $i = 3$	81488	90569	1.111
Anchor $i = 4$	72324	63270	0.875

The smallest of the area ratios $\Psi_{i,flex} = A_i^{flex} / A_i^{rigid}$ is then calculated as in Equation (11) and it defines the resistance reduction factor for the flexible baseplate.

$$\Psi_{flex} = min\left[\Psi_{1,flex}; \Psi_{2,flex}; \Psi_{3,flex}; \Psi_{4,flex}\right] = 0.874 \tag{11}$$

At this point the concrete cone resistance for the flexible baseplate $N_{Rk,c}^{flex}$ can be derived as reduction of the concrete cone resistance calculated for the rigid baseplate $N_{Rk,c}^{rig}$

$$N_{Rk,c}^{flex} = N_{Rk,c}^{rig} \cdot \psi_{flex} \tag{12}$$

The results of the design example demonstrate that considering baseplate flexibility may lead to a reduction in the concrete cone resistance of the anchor group compared to the rigid baseplate assumption. From a design perspective, this highlights the importance of accounting for baseplate deformability, particularly for thin baseplates or configurations with uneven anchor force distribution. Neglecting baseplate flexibility may result in an overestimation of anchorage resistance and unconservative design outcomes. Therefore, the use of analysis methods and software tools that incorporate baseplate flexibility, such as the new resistance model implemented in the Hilti method, is recommended to achieve a more realistic assessment of anchor group behavior and to enhance the reliability and safety of the design.

7. Summary

Steel-to-concrete connections are critical components of load-bearing systems, as they govern the transfer of forces between steel and concrete elements and significantly influence the global structural response. Although the Eurocodes provide a comprehensive and well-established framework for their design, differences in underlying assumptions, particularly regarding baseplate behavior, introduce inconsistencies between codified models and actual structural performance.

This paper has shown that the rigid baseplate assumption required by Eurocode 2-4 for anchor group verification using the Concrete Capacity Design method does not always reflect the real flexural behavior of steel baseplates. In practical applications, especially those involving thin plates, large eccentricities, or uneven force distribution, baseplate flexibility can lead to nonlinear distribution of anchor forces and altered load-transfer mechanisms in the concrete. Neglecting this behavior may result in inaccurate force prediction and, in some cases, an overestimation of anchor group resistance. To address these limitations, designers often adopt conservative detailing measures that increase material consumption and construction effort without necessarily improving overall structural performance.

The presented findings highlight the importance of incorporating baseplate flexibility into the assessment of anchorage behavior. Doing so enables a more realistic representation of force distribution within the anchor group and the surrounding concrete, thereby improving the reliability and transparency of the design. **The new design method introduced by Hilti addresses this gap by explicitly accounting for baseplate deformability in the evaluation of tension-governing failure modes**. The method is fully implemented in **PROFIS Engineering**, where it is seamlessly **integrated into the existing design workflow**, as applicable, allowing engineers to directly apply the enhanced resistance model within practical project environments. By reconciling steel and anchorage design philosophies, the proposed approach supports safer, more economical, and optimized steel-to-concrete connections.

In conclusion, while existing standards remain essential for supporting structural safety, enhanced design models that reflect actual structural behavior offer clear benefits. The adoption of methods that consider baseplate flexibility represents a step toward more efficient, and performance-based design of steel-to-concrete connections in modern engineering practice.

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