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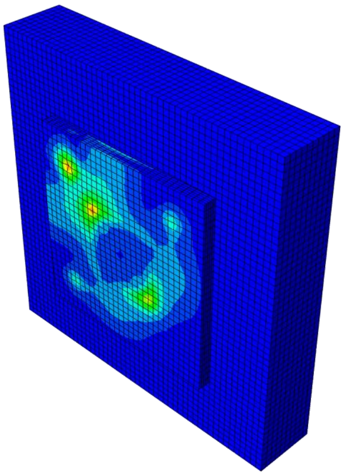

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Avdelningen för Konstruktionsteknik
Lunds Tekniska Högskola
Lunds Universitet, 2011

Rapport TVBK - 5199

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2011

Keywords
Anchor plate, Automated calculations, Numerical, Non-linear calculations, Theoretical

Report TVBK-5199
ISSN 0349-4969
ISRN: LUTVDG/TVBK-11/5199(180p)

Master's Thesis
Supervisor: Johan Kölfors, Scanscot Technology AB
Examiner: Sven Thelandersson, Professor at Lund University
June 2011

Down the rabbit hole.

Preface

This report is written as a master's thesis in Civil Engineering at the Department of Structural Engineering at Lund Institute of Technology. The work has been done on location at Scanscot Technology AB in Lund, Sweden.

We would like to thank our supervisor Johan Kölfors for his expert opinions and helpful supervision.

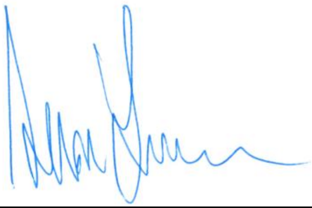
We would also like to thank the rest of the staff at Scanscot Technology AB for their help, but maybe even more important letting us use their facilities, necessary software and making us feel as part of the group while we were there.

Examiner of this master's thesis is Sven Thelandersson at Lund Institute of Technology. Thelandersson is a professor at the Department of Structural Engineering.

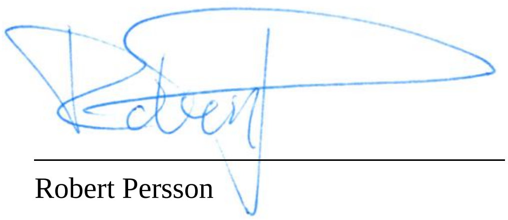
Hopefully this document will give you, the reader, insight in the issues of the safety design process of anchor plates and the possibilities to make the calculations more accurate in an efficient manner.

This work corresponds to 30 credits at the University, it started in September 2010 and was finished in April 2011.

Lund, June 2011



Albin Larsson



Robert Persson

Abstract

In industrial buildings there is often a need to attach processing equipment and other components to the building's structure. If the main structure is made of concrete this is usually done by using anchor plates. An anchor plate is composed of a steel plate, fixed to the concrete with two or more concrete fasteners. There is a large variety of fasteners with different mechanical means of operation. Which system to be used depends on the circumstances and may differ quite much. Based upon the forces and moments which act on the anchor plate one can calculate material stresses and strains in the anchor plate as well in the fasteners and the surrounding concrete. These load effects are then compared against the normative failure type capacities in order to verify the design. Capacities are calculated according to the chosen design code, which in this project is CEN/TS 1992-4 technical specification by the European Committee for Standardization, a design code aimed entirely for safety design of anchor plates. The CEN/TS 1992-4 is a normative document produced by a Technical Committee, and if approved will be adopted into a Eurocode Standard.

The main problem lies within the fact that there are many different load combinations to be analysed, including normal operation load cases to severe accident load cases. Each combination can contain several variable loads and even some loads of dynamic character. This gives a large number of combinations of normal force, shear forces, bending and torsional moment, all of which has to be verified. Add to this that it is common for industrial buildings to have thousands of anchor plates, which is why there is great benefit to automate the safety design process.

The objective of this master's thesis is to develop a method for automatisation of safety design and verification of anchor plates and parameterise information (geometry, type of anchorage, material properties, load effect, and much more) that are of importance in the safety design process. On basis of the above mentioned information a model for FEM-analysis is to be automatically created and the load response in the anchor plate's different parts calculated. Finally this response is compared and verified against CEN/TS 1992-4.

In this project a survey for different possible models of analysis has to be carried out, where parameters such as element type, interaction properties, boundary conditions etcetera are to be varied and the results compared to each other. The objective is to generate a material for decision regarding a final FE-model that gives results very close to considerably more complex models, whilst the time of analysis is kept at a minimum.

The results of this master's thesis show that a very qualified method of automated design is possible for mass-design of anchor plates which leads to accurate results at a fraction of the time it would consume to model, analyse, and normatively verify the anchor plates manually. For a standard-type anchor plate that is fairly simple and common (0.3 x 0.3 meter square steel plate with 4 anchors placed in perfect rows and columns) the time for model generation, analysis of model, and verification of results the total consumed time is just below 30 seconds for a single enveloped set of loads.

Sammanfattning

I industribyggnader finns vanligen behov av att fästa processutrustning och andra komponenter mot den bärande betongstrukturen. Detta görs som regel med hjälp av så kallade ankarplattor. Dessa utgörs av en stålplatta som fästs i betongen med betongförankringar. Det finns ett stort antal olika typer av betongförankringar med olika mekaniska verkningssätt. Utgående från de krafter och moment som verkar på plattan kan krafter och spänningar i ankarplattans olika delar (inkluderat närbetongen) beräknas. Denna lastpåverkan jämförs sedan med kapaciteter för olika brottmoder beräknade enligt normanvisningar. Den normbeskrivning som valdes i detta projekt är en teknisk specifikation utgiven av Europeiska Standardiseringskommittén (CEN/TS 1992-4) och berör just ankarplattor specifikt.

Som regel måste ett stort antal lastkombinationer beaktas. Eftersom dessa vanligen innehåller flera variabla laster, varav vissa är av dynamisk karaktär, blir det ett stort antal uppsättningar med krafter som måste kontrolleras. Utöver detta tillkommer det faktum att det i en byggnad kan finnas ett mycket stort antal ankarplattor. Det finns därför en stor potential i att försöka automatisera denna beräkningsprocess.

Examensarbetets syfte är att utveckla en metod för automatisering av beräkningsprocessen vid dimensionering och strukturell verifiering av ankarplattor. I metoden ingår att systematisera all indata (geometri, förankringstyp, materialegenskaper, belastning etc.) som är av betydelse vid dimensioneringen. Baserat på dessa indata ska en beräkningsmodell automatiskt kunna genereras och lastpåverkan i ankarplattans olika delar beräknas. Slutligen skall denna lastpåverkan kontrolleras mot bärförmågan med avseende på tänkbara brottmoder. Bärförmågan beräknas enligt anvisningarna i Eurocode och Eurocode-relaterade regelverk.

I projektet ingår även att undersöka olika sätt att modellera ankarplattor med avseende på val av elementtyp, kontaktvillkor, randvillkor etcetera med målsättning att minimera beräkningstiden med bibehållen beräkningsnoggrannhet.

Resultatet av examensarbetet påvisar mycket goda möjligheter att effektivisera dimensioneringen av ankarplattor med ett automatiserat program som i en bråkdel av den tid det skulle ta att manuellt modellera, analysera och verifiera en ankarplatta kan göra allt detta på strax under 30 sekunder för ett enda enveloperat lastfall i en lastkombination och som består utav en typplatta som är relativt vanligt förekommande (0,3 x 0,3 meter stålplatta och 4 ankare som är placerade i rader och kolumner).

Table of contents

1 Introduction

1.1 Background

In industrial buildings there is often a need to attach processing equipment and other components to the building's structure. If the main structure is made of concrete this is usually done by using anchor plates. An anchor plate is composed of a steel plate, fixed to the concrete with two or more fasteners, see Figure 1.1. There is a large variety of fasteners with different mechanical means of operation, which system to be used depends on the circumstances and may differ quite much.

Based upon the forces and moments which act on the anchor plate, material stresses and strains in the anchor plate as well in the fasteners and the surrounding concrete can be calculated. These load effects are then compared to the normative failure type capacities in order to verify the construction design.

Load effect analysis can be made with numerical calculation software, such as Abaqus or BRIGADE. With such a solution, taking the non-linear behaviour of the anchor plate components into consideration, one can obtain very accurate results compared to other more traditional methods.

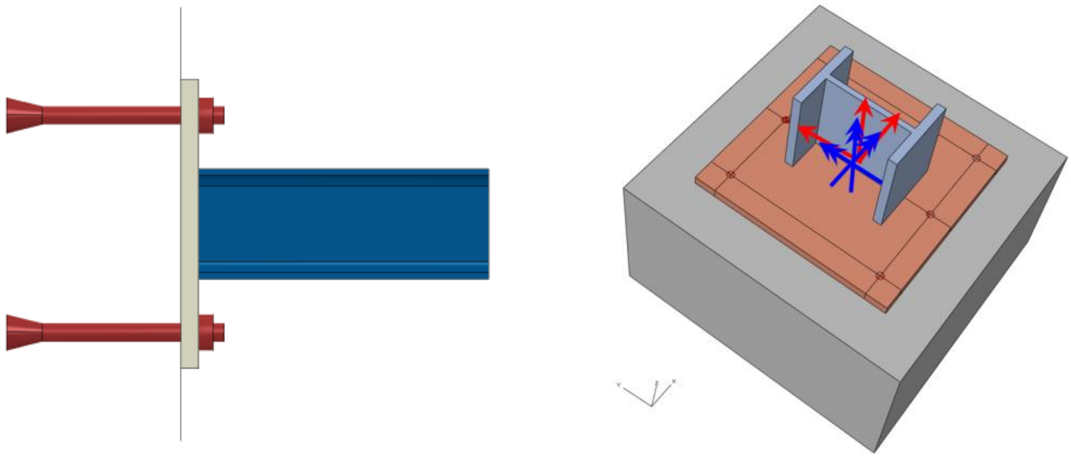


Figure 1.1. Series of images exemplifying the overall geometry of an anchor plate.

1.2 Objectives

The main problem lies within the fact that there are many different load combinations to be analysed, including normal operation load cases to severe accident load cases. Each combination can contain several variable loads and even some loads of dynamic character. This gives a large number of combinations of normal force, shear forces, bending and torsional moment, all of which has to be verified.

Add to this that it is common for industrial buildings to have thousands of anchor plates, which is why there is a great potential to automate the safety design process and create a method of modelling which gives results very close to considerably more complex models, whilst the time of analysis is kept at a minimum.

The focal point of this master's thesis is to outline different possible models and compare these in terms of how forces and stresses differ and also to observe the cost of the analysis, e.g. does the result accuracy decrease with a smaller time footprint?

The purposes are specified as follows:

1. Evaluate possible Finite Element models for the anchor plate analysis.
2. Develop a method for an automated process regarding the safety design of anchor plates and structural verification of anchor plates.
3. Evaluate the method by developing a program code, which auto-generates the Finite Element model and performs appropriate analyses.

1.3 Method

The task of evaluating Finite Element models starts off with a parametric study, where parameters such as element type, element size, material definitions etc. are adjusted individually. Conclusions on reasonable models for an automated process are then made with the results from the parametric study as basis.

Continued development of the method is furthermore dependent on reliable capacities. The normative design documents used throughout this master's thesis are:

- Eurocode documents regarding concrete and steel design.
- Eurocode related documents, such as the *Design code of fastenings for use in concrete* (CEN/TS 1992-4) by the European Committee for Standardization.

BRIGADE is used as the numerical analysis software. The software package is developed by Scanscot Technology AB and uses a Finite Element solver from Abaqus, which is well known to have good capabilities of capturing non-linear behaviours.

The programming language used to develop the automated design script is Python. This choice was limited since the program will become a module accessible through the BRIGADE user interface which is partly built upon this language. Nevertheless, Python is a powerful programming language and meets the requirements quite satisfactory.

Input data, such as loads, information about the anchor plates and its fasteners and materials, are delivered as several Microsoft Excel-documents. These documents will be formatted in a standardised manner enabling homogenous inventory of anchor plates. Without a standardised user input format there can be no automatisation, which disables the massive safety design process at large.

1.4 Scope

The work of this master thesis is limited to the calculation and evaluation of the anchor plate, its fasteners and the nearby surrounding concrete. The given loads are concentrated forces and moments with the point of action in the centre of mass of the corresponding beam attachment.

All loads are calculated as characteristic forces and are delivered separately so they can be load combined correctly. However, load combination will not be included in this project, the reason is that the level of complexity and work burden will quite soon be overwhelming due to the fact that the load combined effects for each attachment may in turn be load combined between attachments if more than one beam joint exist.

With reason of the above statement the load deliverables will in this project already be load combined and only load combinations affecting the first beam attachment will be considered.

1.5 Disposition

Chapter 2 summarizes the geometrical arrangement of an anchor plate, describing the general geometry of the steel plate and the corresponding attachments. Even a detailed summary over the available anchorage systems is given.

Chapter 3 describes the different fastener types available on the market for use as anchorage of anchor plates to concrete structures.

Chapter 4 treats the basic behaviour of how the anchor plate objects behaves in general but also specific behaviours of the components the anchor plate consist of (steel plate, fasteners, and concrete).

Chapter 5 renders the validations of failure modes required in relation to the chosen safety design code.

Chapter 6 serves as decision support for which FE-model is appropriate for use in the automatised method of safety design later to be developed within the frame of this master's thesis.

Chapter 7 presents the input information required for auto generation of the FE-model and normative verification of the analysis results. Implementation regarding loads etc. is also discussed in this chapter.

Chapter 8 gives details on how the application is built and how implementations regarding normative verifications are carried out.

Chapter 9 and chapter 10 presents and discuss the end result of the master's thesis, the developed application for automatised safety design.

2 Background on anchor plates

2.1 Historical overview and purpose of application

There has always existed a necessity to connect components to each other when building houses and other types of structures. Example areas of employment are connection of beams and slabs to the structural core (Figure 2.1), connection of roof parts, joints between the stabilisation system and the structural core (Figure 2.2), and attachments of essential systems required to maintain modern building functions, such as ventilation systems and other installations.

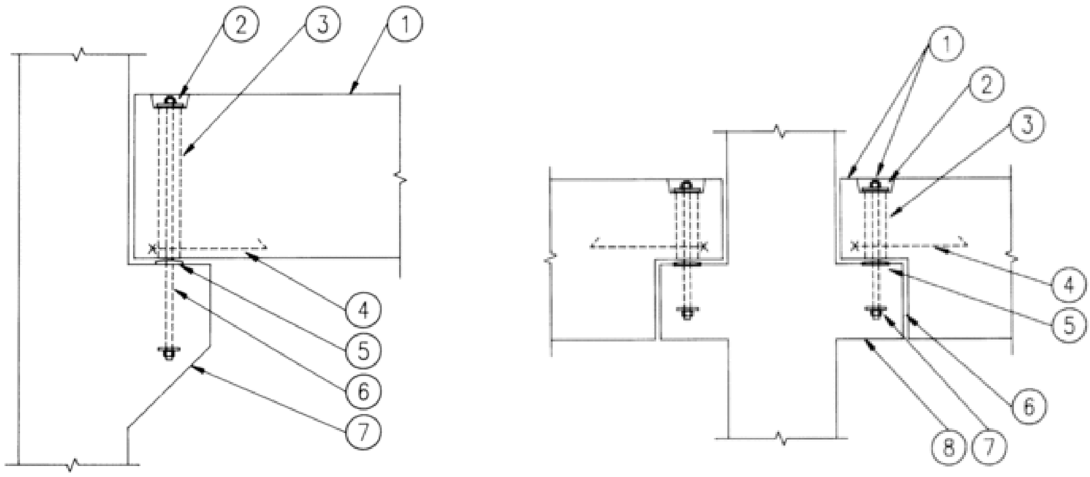


Figure 2.1. Connection joints between slab/beam parts and the concrete core. (Bygga med prefab, chapter 4.1.2)

Figure 2.2. Connection joints between wooden arches and concrete foundations.
(Håkons Hall, Lillehammer, Norway. Håganfang. Architect: Östgaard Arkitekter A/S. Foto: Egil Bjerke.)

Typical methods of joinery vary for different kinds of building materials. For wooden constructions the most used methods are various types of interlocking, screwed, doweled and glued joints, and other embedded materials such as steel plates and ring connectors which are placed into cut slits and locked in place with bolts and screws. In steel construction the joints between components often are bolted or welded together.

Concrete component joints are very much alike previously mentioned joinery methods used by other materials, for example interlocking of concrete beams and slabs etcetera. In addition to these common methods there is the anchor plate attachment method. Anchor plates are commonly used to fasten secondary steel structures to the main concrete structure.

Some typical applications of fastenings with anchor plates are listed below, (S. Hoehler, 2006).

- Facades
- Suspended ceilings
- Heating, ventilation, air conditioning
- Pipelines
- Mechanical equipment
- Structural connections

This master's thesis focuses on the application of anchor plates in industrial buildings, such as processing plants and nuclear facilities. The main purpose of application for anchor plates in industrial buildings is to transfer loads from installation systems such as ventilation, plumbing and miscellaneous equipment to the concrete structure. In nuclear facilities there can be several thousand anchor plates, some of which are designed and installed during the construction phase and others sometime after the building was erected, so called cast-in-place and post-installed fastening systems. This gives several varying design solutions, all of which have different geometry and different fastening systems.

The history of fastening methods in concrete is described in *Anchorage in Concrete Construction* written by Rolf Eligehausen, Rainer Mallée and John F. Silva. Anchorage systems in concrete has evolved from wooden lathes placed in the formwork and anchored with nails, later to be used as attachment points for building systems such as suspended ceilings, threaded sleeves, anchor channels and headed studs welded to steel plates. All of the systems are secured by casting them into the concrete.

Cast-in-place techniques are often sufficient, but if it is desirable to attach a part to the concrete member after it has cured, new methods must be developed. E.g. when building regulations and safety codes are changed over time, resulting in an increase of building requirements, or when new functions are to be added to the present. This is how the next step in the anchor plate evolution was initiated, creating methods of joinery for post-installation in cured concrete enabled by innovations in drilling technology, (Eligehausen et al., 2006).

As the cast-in-place systems, even the post-installed systems have been refined over time, with the development of expanding steel anchors in pre-drilled holes which can withstand higher loads compared to the simpler lead plugs. The details on an expansion anchor are carefully designed and to achieve full bearing these have to be installed according to given specifications from the manufacturer. Similar to the expansion anchors are the newly developed bonded anchors which are fixed to a pre-drilled hole in the concrete through chemical adhesives and the undercut anchors, a hybrid anchor which apply any combination of friction, mechanical interlocking and bonding through complex details and installations, (Eligehausen et al., 2006).

The latest innovations in post-installed anchors are direct installation anchors, a post-installed anchor which does not require a pre-drilled hole to be mounted, but takes advantage of the strength of high-grade steel and is driven into the concrete by an explosion or a device that propel the nail by pneumatic force (also called power-actuated fasteners), (Eligehausen et al., 2006).

Eligehausen et al. (2006) states that “post-installed fastening in concrete and masonry is a relatively young discipline, meaning that the state of the art is generally in a state of flux. Consequently, these systems typically cannot be regulated via prescriptive standards, as is done, say, with high-strength structural bolts.” They continue this statement by concluding that post-installed anchors are therefore designed according to product-specific approvals of utilisation.

2.2 General geometry

The common geometry of an anchor plate used in structural applications is quite simple.

One or several attachments, which consist of any kind of beam profiles, are mounted to a steel plate. The attachments are most often mounted with a welded joint since both the base plate and the attachments are made of steel. The geometrical form of the steel plate is in most cases limited to a right quadrangular plate, see Figure 2.3 below.

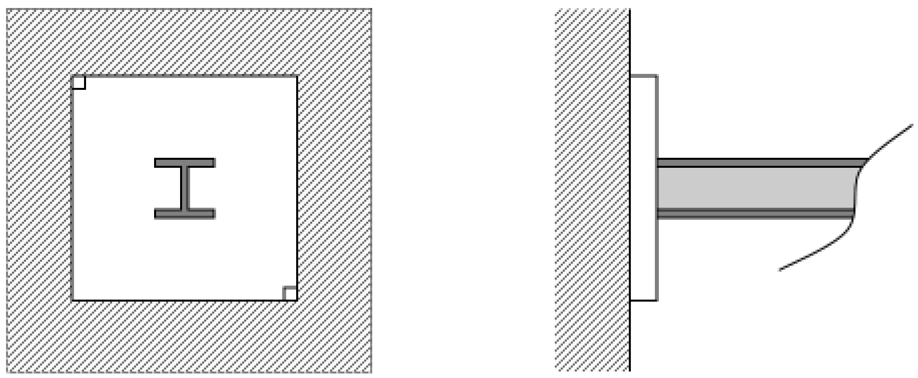


Figure 2.3. Front-view and side-view illustrations showing the general geometry of an anchor plate.

Put into a larger context, the geometry of the anchor plate extends to the concrete member in which it is mounted. Example of important geometrical properties for both the anchor plate and the concrete structure are:

- Distance(s) to nearby anchor plate(s) *can disturb the distribution of stresses*
- Distance(s) to free edge(s) *can disturb the distribution of stresses*
- Presence of additional embedment(s) *additional distribution of shear loads*
- Thickness of the concrete structure
- Spacing distance between the concrete structure and the base plate

The anchors can be attached to the base plate in various ways. A more detailed geometrical definition of the anchors is given in chapter 3.

3 Fastening systems

This chapter will cover the most commonly used systems, but also the lesser utilised systems are mentioned. Since the objective of this master's thesis is to create an automated design process for anchor plates, it is important that this method is general and cover typical anchor plate fasteners, meaning that headed studs and various post-installed systems will be described in detail.

3.1 General

As briefly mentioned in the previous chapters, there are several methods in which the fasteners of an anchor plate can be attached to the concrete member. These are ordered accordingly into two main categories, specifying if the type of anchorage is installed while erecting the concrete structure or post-installed into already cured concrete.

The load-transfer mechanisms are typically identified as mechanical interlock, friction and bond as shown in Figure 3.1. These mechanisms will be explained more thoroughly in chapter 4.

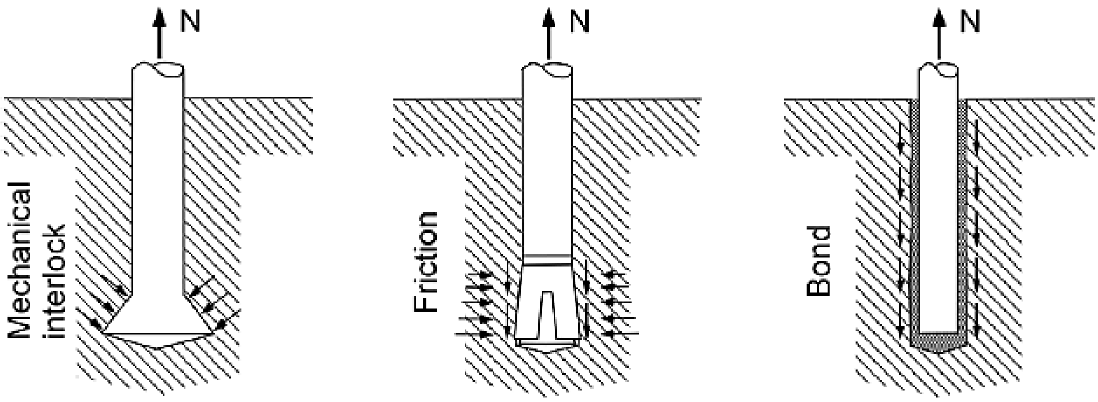


Figure 3.1. Overview of the three main mechanisms in which anchor plates transfer loads to the concrete member, (S. Hoehler, 2006).

3.2 Cast-in-place installation

There is a wide range of cast-in-place fastener solutions on the consumer market, including lifting inserts, anchor channels, embedded plates with headed studs, bent reinforcing bars equipped with internally threaded unions, (Eligehausen et al., 2006). Some of these are shown in Figure 3.2.

The benefits in using cast-in-place systems are that the load action points are known in advance, enabling design of the concrete material and rebar layers with the loads taken into account. Since the fasteners are installed in the erection phase it is even possible to place them in heavily reinforced areas without interference. The disadvantages are that the method requires more planning resources and a denser rebar layout with an increased risk for misplacement of reinforcement, (Eligehausen et al., 2006).

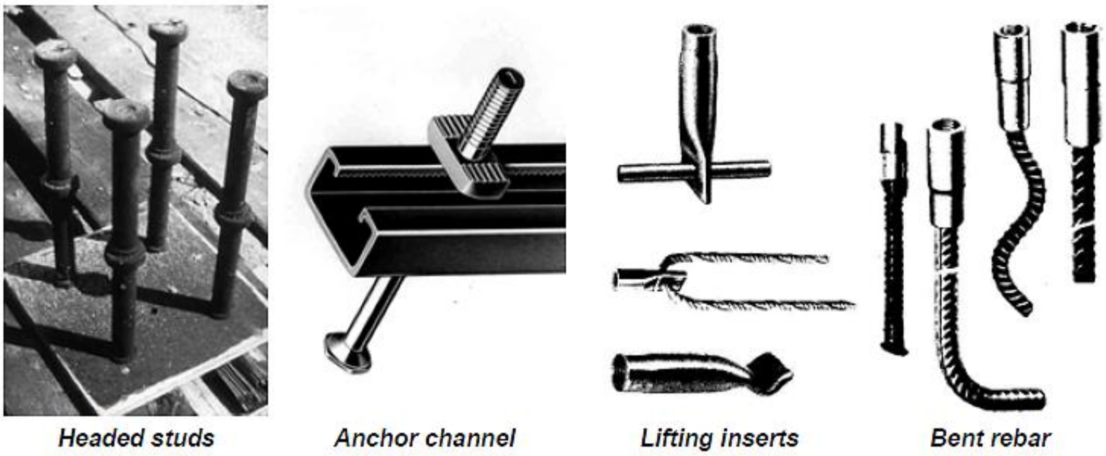


Figure 3.2. Typical cast-in-place fastener systems. (S. Hoehler, 2006)

Cast-in-place systems often employ mechanical interlock to fix the anchors to the concrete.

3.2.1 Lifting inserts

Lifting inserts are primarily used when transporting and installing precast concrete components. By casting in cable loops, internally threaded sleeves, transverse dowels and hairpins, and internally threaded inserts mounted onto the ends of the reinforcement, a crane hook is able to connect to and lift the elements. See Figure 3.3 below.

When installing and using various types of lifting inserts one have to comply with the instructions given by the manufacturer, where details such as required concrete strength, permissible loads, minimum spacing and more are specified, (Eligehausen et al., 2006).

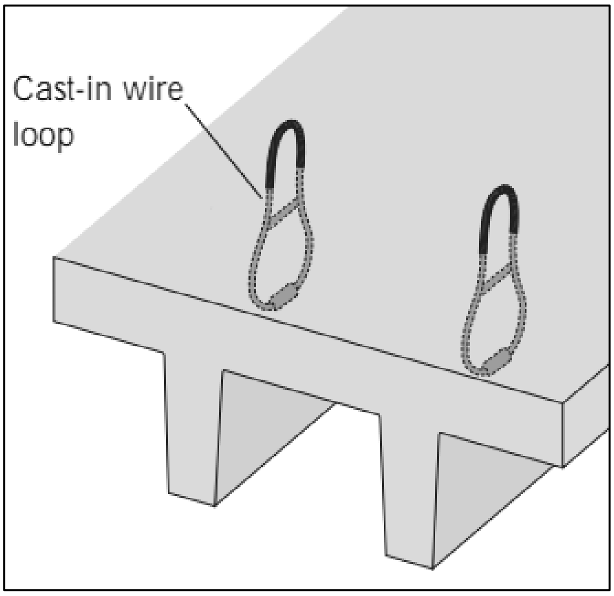
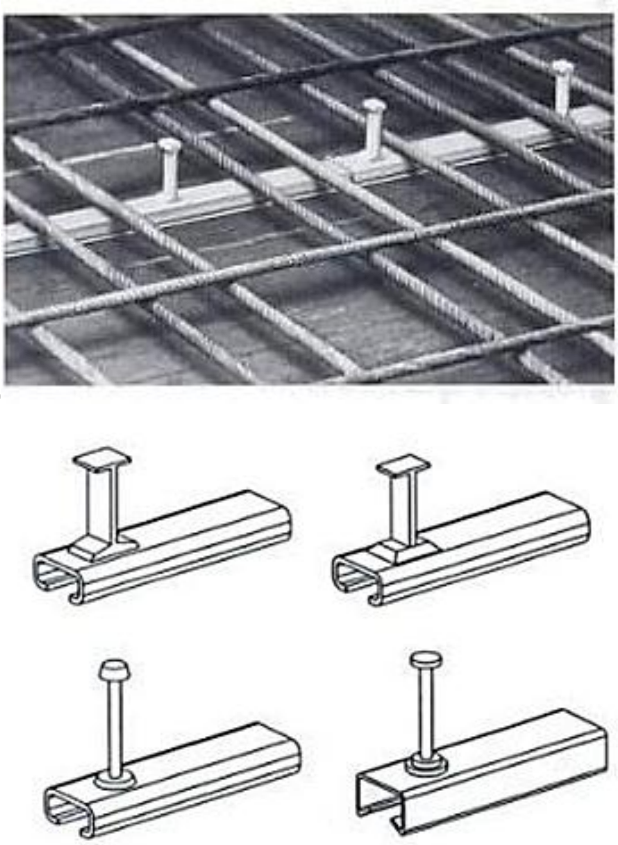


Figure 3.3. Cast-in cable loop for crane hook. (from http://www.halfen.co.uk)

3.2.2 Anchor channels

Anchor channels are cast into the concrete, often embedded with headed studs, I- or T-profiles, see Figure 3.4. With help of certain bolts, objects can then be attached to the channel. These systems are commonly positioned horizontally where the resulting forces will cause tension loads on the anchors and the extending embedment profiles.

There are special kinds of anchors and bolts with serrated edges enabling the anchor systems to resist shear forces in both horizontal and vertical arrangements, (Eligehausen et al., 2006).



a)

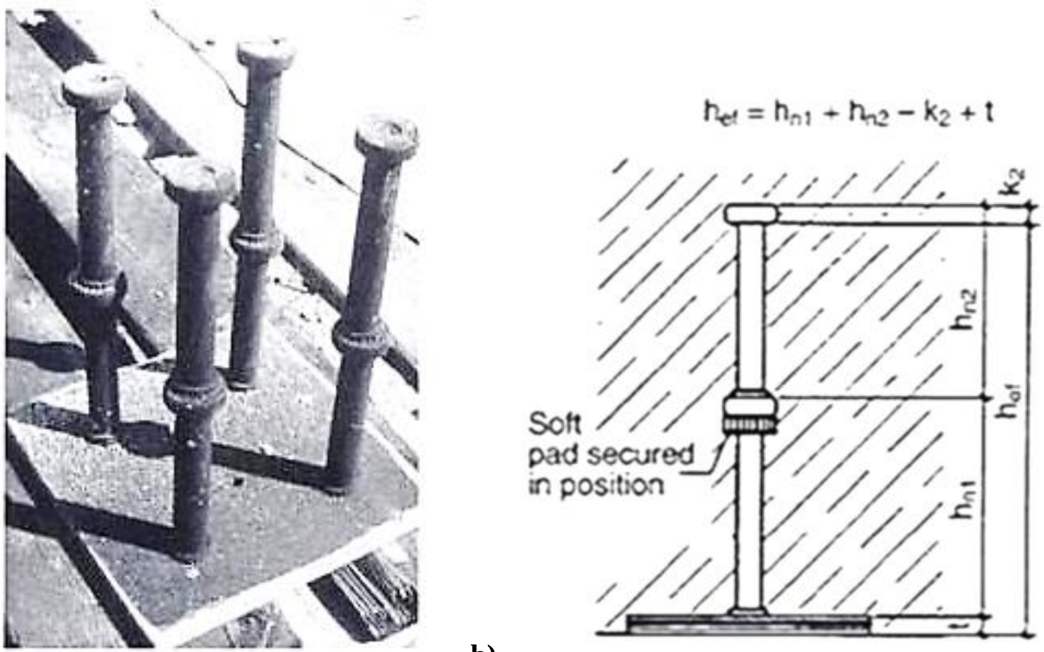
b)

Figure 3.4. Anchor channels, (Eligehausen et al., 2006).
a) Anchor channel being cast in concrete.
b) Variants of cast-in-place anchor channels.

3.2.3 Headed studs

Anchor plates with headed studs as fasteners are a common combination regarding anchor plates for structural use. These consist of iron studs or ribbed reinforcement welded onto a steel plate, where the group is entirely or partly cast into the concrete member.

The production of steel plate anchors with headed studs is usually carried out off-site, in factories, where the production conditions can be monitored, ensuring proper assembly. Longer studs can be made of two or more shorter studs welded to each other, important when doing this is to apply a soft pad beneath all intermediate joints to prevent interlock mechanisms that will shorten the embedment lengths of the anchors, see Figure 3.5, (Eligehausen et al., 2006).



a) b)

Figure 3.5. Headed studs anchors, (Eligehausen et al., 2006).
a) Steel embedded plate with headed studs.
b) Two studs welded together separated with a soft pad.

3.2.4 Threaded sleeves

The areas of use for threaded sleeves where mentioned in section 3.2.1 regarding lifting inserts. Depending on whether the sleeve is used as an attachment point for lifting or as an attachment point for sleeve anchors the layout of the sleeve will vary. In the first case the end of the sleeve is flat with a hole for supporting the cable-loops or transverse dowels. In the latter case the flat end of the sleeve is a hook. One last form of anchorage with sleeves is the curved anchors which comprise a bent ribbed reinforcing bar with a threaded sleeve pressed on, see Figure 3.6, (Eligehausen et al., 2006).

See Figure 3.2 - *Lifting inserts* for different kinds of threaded sleeves.

Figure 3.6. Hooked reinforcing bar with threaded sleeve, (Eligehausen et al., 2006).

3.3 Post-installed installation

The majority of post-installed fasteners require drilling of holes where anchors can be inserted into. Furthermore there are fasteners installed by direct installation as mentioned in section 2.1.

Post-installed systems often employ friction or bond to fix the anchors to the concrete.

The advantages in using post-installed anchors are the flexibility in planning and installations as well as they often are the only option to seismically strengthen and rehabilitate structures, (S. Hoehler, 2006).

3.3.1 Installation configuration

Eligehausen states that post-installed anchors can be mounted in three different configurations as seen in Figure 3.7.

- Pre-positioned
- In-place
- Stand-off

Pre-positioned installation is carried out by drilling a hole with a diameter corresponding to the fastener being inserted. The steel plate which the fasteners are to attach typically has holes smaller than the fastener casing.

In-place installation uses the steel plate element as a template to drill the fastener holes in the concrete. This requires the holes in the steel plate to be at least the same size as the fastener casing.

Stand-off installation fixes the steel plate at a certain distance from the concrete. This requires that the fasteners are capable of transferring both tension and compression loads since the steel plate does not rest against the concrete as in the other cases. Transfer of compression loads can be secured by adding a bearing washer and nut at the surface of the fastener.

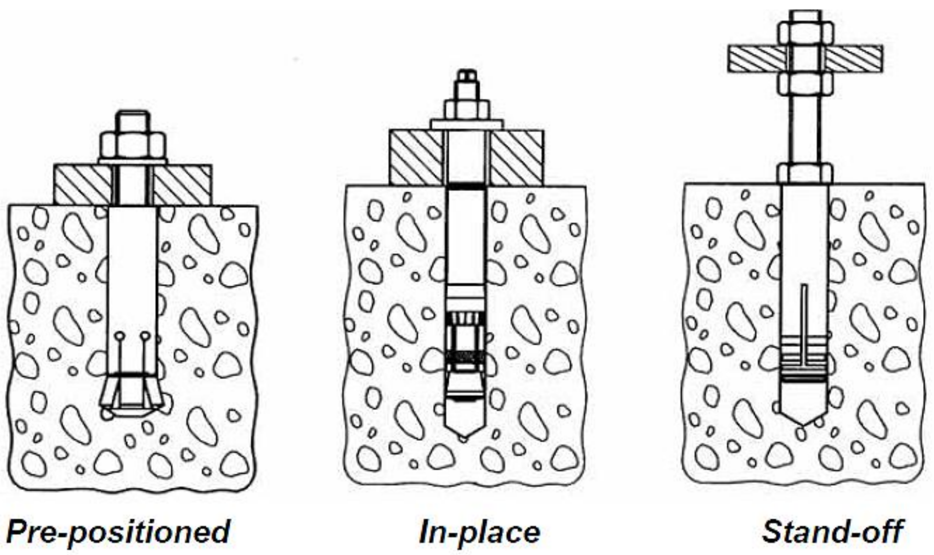


Figure 3.7. Installation configurations of post-installed anchors, (S. Hoehler, 2006).

3.3.2 Drilled-in anchor types

Eligehausen et al. (2006) states that fasteners that require pre-drilling to be installed can be categorised in the following groups and describes them accordingly.

- Mechanical expansion anchors
- Undercut anchors
- Bonded anchors
- Screw anchors
- Ceiling anchors
- Plastic anchors

To maintain focus only mechanical expansion anchors and undercut anchors are described further.

3.3.2.1 Mechanical expansion anchors

Mechanical expansion anchors can be divided into two subgroups according to how the friction force is generated; Torque-controlled and Displacement-controlled.

3.3.2.1.1 Torque-controlled mechanical expansion anchors

Torque-controlled expansion anchor can be further categorised into sleeve-type anchors and Bolt-type anchors.

The sleeve-type anchor consists of a steel bolt or threaded rod which is fixated with a washer, spacer and nut. There are expansion sleeves preventing the anchor from rotating when being set and installed. Bolt-type anchors consist of a steel bolt with a conical head and expansion segments nested to the sleeve carrying the bolt. The bolt is tightened with a washer and a nut.

Installation is carried out by drilling a hole, removing dust and debris and securing the anchor by applying the correct amount of torque to the nut or bolt head specified by the manufacturer. The application of torque will draw the cone at the end of the anchor into the sleeve or expansion segments, forcing the casing to expand against the concrete, see Figure 3.8. The expansion of the anchor generates tension forces in the bolt or rod and compressive forces on the concrete, this will however ensure frictional resistance for the anchorage system. Improper setting of the anchor, e.g. holes drilled too big, can lead to reduced anchor capacity.

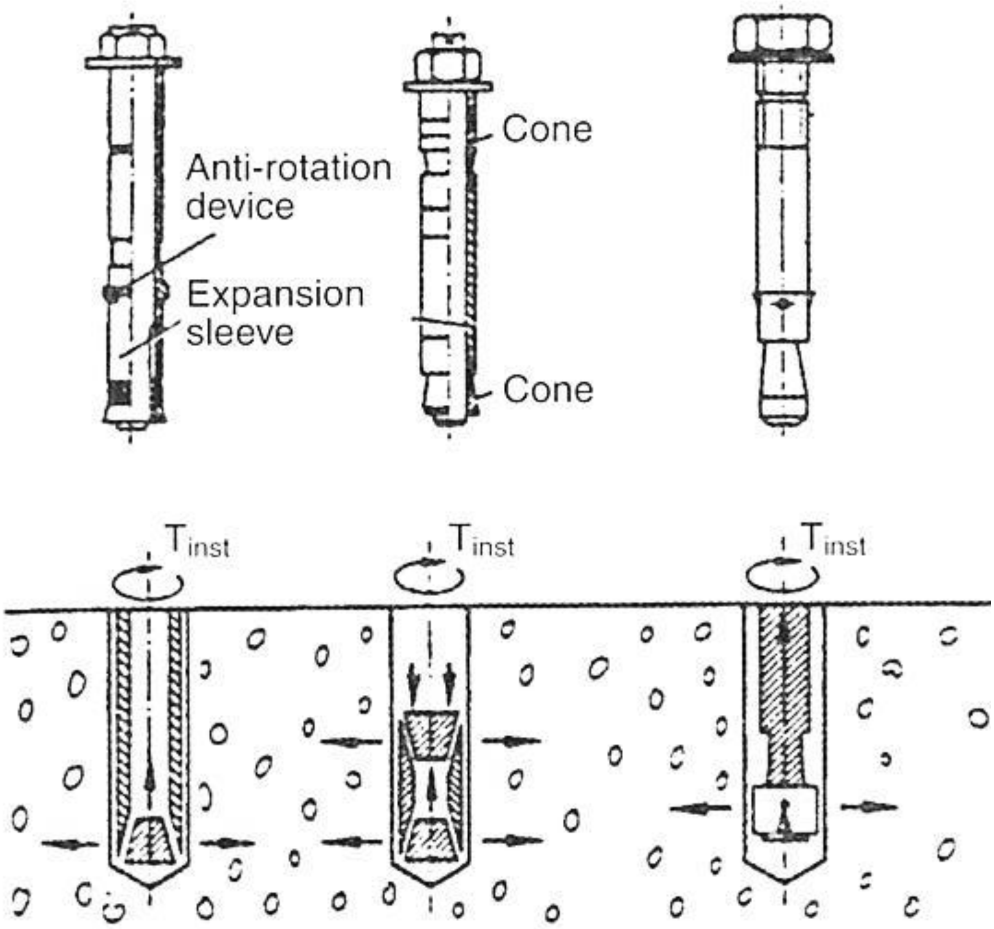


Figure 3.8. Working principles of torque-controlled expansion anchors, (Eligehausen et al., 2006).

The first two anchors from the left in Figure 3.8 illustrate sleeve-type anchors with one and two expansion cones respectively. Systems with two or more expansion cones give a larger load-carrying capacity, but to the cost of a larger required minimum edge distance since the expansion forces are equally larger. The anchor to the right in Figure 3.8 illustrates a bolt-type anchor where the bolt is driven into the sleeve which upon expansion generates friction forces.

Given the nature of installation, torque-controlled anchors generate high stresses and local deformations in the concrete near the expansion zones. This is illustrated in Figure 3.9. The stresses and deformations are directly affected by the magnitude of force with which the cone head is driven into the sleeve. One more factor of importance for how the deformations will behave is the resistance of compression in the concrete.

Figure 3.9. Distribution of stresses at the expansion zone of the anchor, (Seghezzi, 1983).

Torque-controlled anchors transfer external tensile forces primarily through friction but also, to a limited extent, through mechanical interlock at the expanded parts. The friction forces are maintained by pre-stressing the bolt or rod. However, due to the effects of local relaxation in the steel material and concrete material, the pre-stressing force will be reduced and consequently also lowering the load-bearing capacity. Other factors that can lower the capacity include cracking in the anchorage zone which lowers the pre-stressing force further.

When an external tensile load is applied to the anchor it will act to relieve the existing pre-stressing force. At the point when the external load and the pre-stressing force balance each other out, an increase of external load will result in the bolt cone-head to be drawn further into the sleeve or segments thus expanding the anchor. This effect is called follow-up expansion. Follow-up expansion is only possible when the friction between the cone-head and sleeve is lesser than the friction between the sleeve and the concrete material. If this condition is not fulfilled the anchor will partially or fully slip out of the hole as a result of too large external tensile load. This is why manufacturers add ribs, knurling and other friction solutions to increase the friction potential. Worth mentioning is that the follow-up expansion effect is dependent of how accurate the anchor has been installed. In Figure 3.10 below the same anchor type has been installed in two holes of different diameter. As the hole clearance increases so does the required expansion of the sleeve to maintain sufficient surface friction, thus decreasing the follow-up capacity.

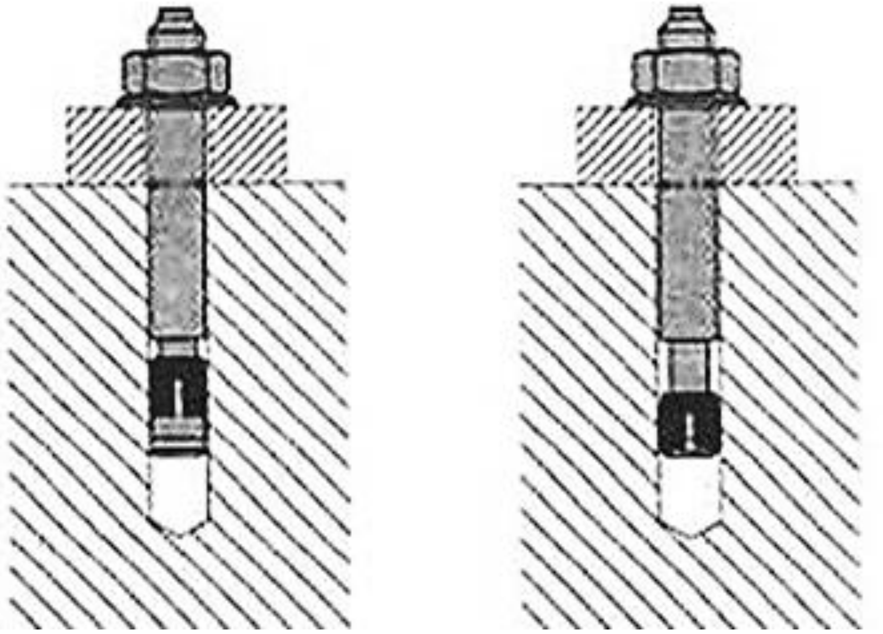


Figure 3.10. Expansion of the sleeve by the cone head in holes with varying clearance, (Eligehausen et al., 2006).

Rolf Eligehausen et al. (2006) states in the end of their description of torque-controlled post-installed anchors that anchors made in Europe "*are typically fabricated to conform to the requirements of Grade 8.8 steel according to ISO 898, Part 1 (1988)*" and that "*for anchors fabricated in the U.S., no single standard is universally specified*".

Typical diameters for torque-controlled expansion anchors range from 6 mm to 24 mm.

3.3.2.1.2 Displacement-controlled mechanical expansion anchors

The displacement-controlled expansion anchor consists of an expansion sleeve and a conical expansion-plug at the end. Friction force is generated by expanding the sleeve against the sides of the hole controlled by the axial displacement of the expansion-plug within the sleeve. A major difference compared to the torque-controlled anchor is that the sleeve is internally threaded to accept any kind of threaded element, this is necessary since the element cannot attach to the conical head risking to eliminate the friction resistance.

Displacement-controlled anchors can be divided into Cone-down and Cone-up categories depending on how the sleeve is expanded, (S. Hoehler, 2006).

Cone-down anchors are set by first drilling a hole in which the expansion sleeve is inserted. The next step is to expand the sleeve to gain friction resistance against the concrete. This is done by driving the expansion-plug into the sleeve using a setting-tool and a hammer. (See Figure 3.11, the utmost anchor to the left)

Cone-up anchors are set in the reverse order compared to the cone-down anchor. Here the expansion sleeve is instead forced upon the conical expansion-plug by hitting the sleeve with a hammer. (See Figure 3.11, the utmost anchor to the right)

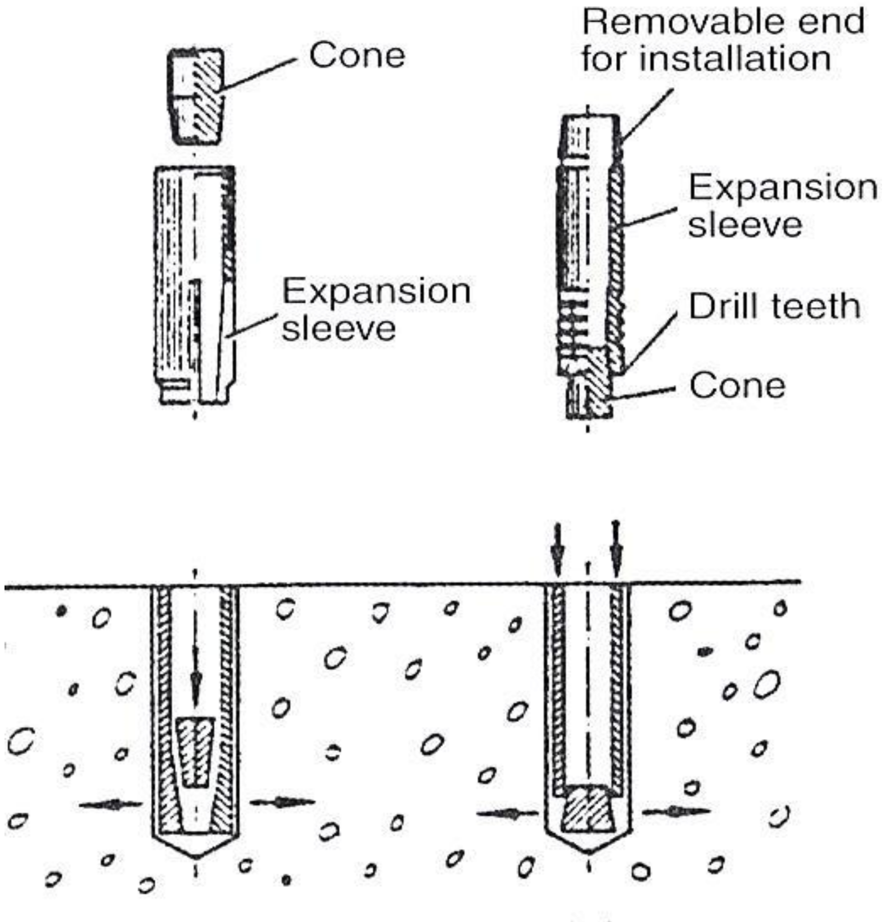


Figure 3.11. Working principles of displacement-controlled expansion anchors, (Eligehausen et al., 2006).

External tension loads are transferred mainly by friction forces and somewhat by mechanical interlock at the deformed parts of the anchor, similar to the torque-controlled expansion anchors. Other similarities are that the expansion force is affected by the degree of expansion, the hole-clearance of the pre-drilled hole and the deformation resistance of the concrete.

A major difference is that the initial expansion force is much higher than that generated by torque-controlled anchors. The expansion force will however decrease over time as a result of concrete relaxation and the only way to increase it again is by re-setting the anchor.

The hole clearance tolerance is much more strict when coming to installation safety. If the pre-drilled hole is too big the expansion force cannot create a sufficient frictional capacity, but if the hole is too small the concrete will crunch as a result of too high compression forces. As a step to increase the safety when installing anchors several visual methods of verification have been adopted. When installed correctly the setting tool must be in contact with the sleeve of the anchor, ensuring that the correct setting energy has been applied. Here lies an advantage in using torque-controlled anchors since there is a reserve capacity when the applied loads forces the cone head to retract, expanding the sleeve furthermore (the so called follow-up expansion). This type of reserve does not exist for displacement-controlled anchors.

Typical diameters for displacement-controlled expansion anchors range from 6 mm to 20 mm.

3.3.2.2 Undercut anchors

Undercut anchors deploy a combination of anchorage methods. As with cast-in-place systems the undercut anchor generates a mechanical interlock but also a bonding effect as in bonded anchorage systems. The primary anchorage is however maintained by mechanical interlock. To install an undercut anchor one needs to utilise a special drilling bit with which the hole is drilled and a cut is made with a special device for the corresponding undercut part. See Figure 3.12 and Figure 3.13 below for example of undercut anchor installations.

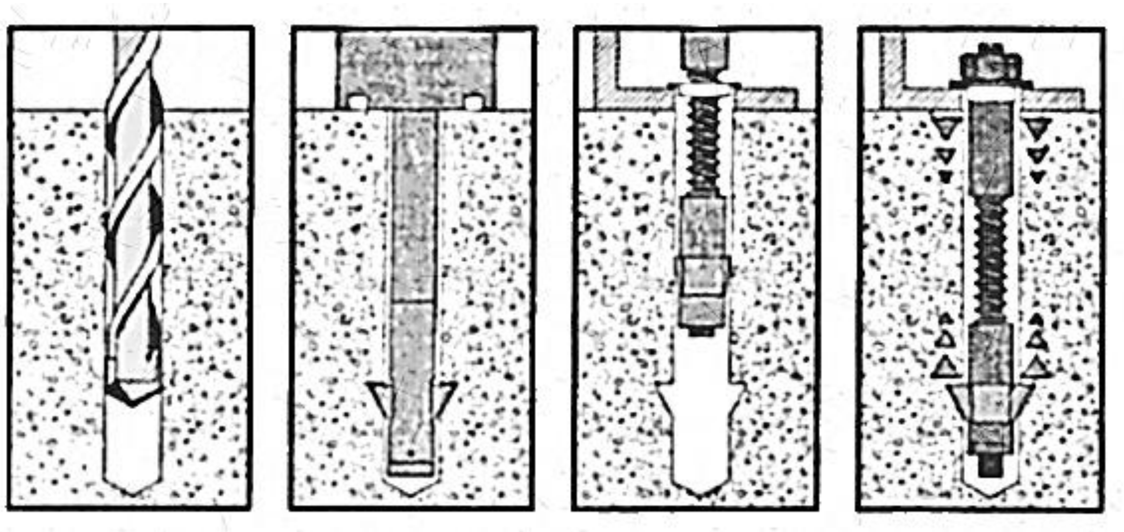


Figure 3.12. Example of an undercut anchor widen towards the surface, (Eligehausen et al., 2006).

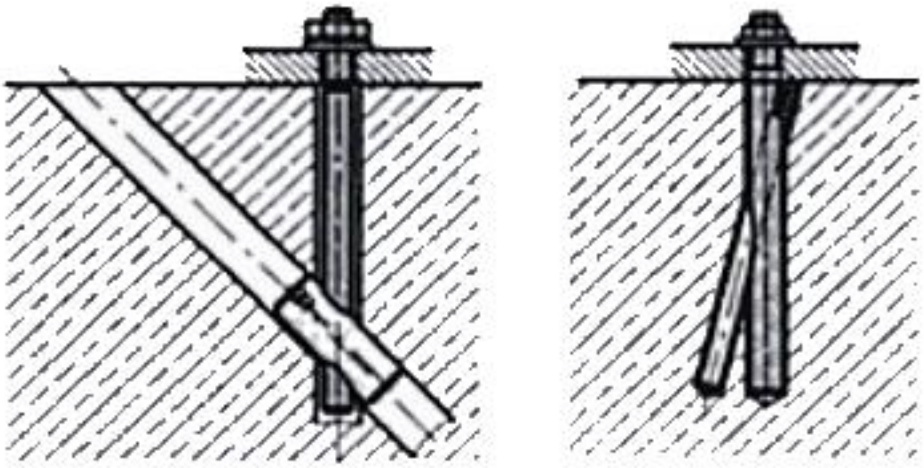


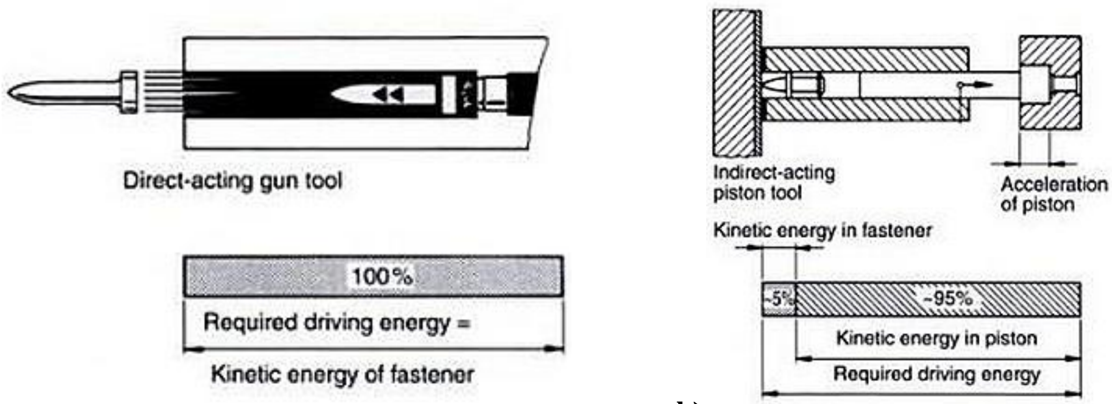
Figure 3.13. Example of undercut anchors utilising other interlocking methods, (Eligehausen et al., 2006).

3.3.3 Direct installation

By using a direct installation method, the fasteners are driven through force into the concrete in which they are supposed to be anchored. This is done by using a power-actuated setting tool powered by either an explosive charge or compressed air.

There are two types of fasteners utilised in this kind of installation, a steel pin type and a threaded stud type. The steel pins are flattened at their head and are driven not only into the concrete member but also through the component which is to be fastened. Stud anchors are threaded at their head enabling the attachment to be secured with a nut to the studs after it has been shot into the concrete.

Power-actuated setting tools exist in two variants. The first option accelerates the fastener through kinetic energy, caused by a black powder explosion or combustible gas, forcing it through the base material upon impact. This technology is similar to the one of a gun and generates high risks since the pin or stud gain large velocities. Therefore a substitute has been developed where a piston, with larger mass and thus lower velocity, forces the fastener to penetrate the base material. With this method the process becomes displacement-controlled reducing the risks for the operator dramatically. (Figure 3.14 below shows a brief distribution of the required energy.)



a) **b)**

Figure 3.14. Direct-installed anchors, (Eligehausen et al., 2006).
a) Anchor installed by direct-acting tool with explosive charge.
b) Anchor installed by direct-acting tool with gas-powered piston.

Directly installed fasteners use friction to transfer tension forces to the concrete member. As a result of the high speed penetration of the concrete, the microstructure is eliminated and the concrete compacted, increasing the compressive strength in the vicinity of the anchor. Add to this that the rise of temperature during installation gives a sort of adhesive effect between the concrete and the steel fastener, which increases the friction capacity for the anchor set.

4 Basic behaviour of anchor plates

Even though the geometry of an anchor plate is quite simple, the theory behind it is more complicated. This chapter will explain the basic behaviour of fasteners and the anchor plate in whole including the concrete member.

4.1 Requirements for anchor plates

As with all types of construction elements, anchor plates must be designed to withstand all effects in the application they are supposed to operate.

> *"Fastenings must be designed in such a way that they do the job for which they are intended, are durable and robust, and exhibit sufficient load-carrying and deformation capacity."* (Eligehausen et al., 2006)

Therefore the methods of design vary with the type of application the anchor plate is intended for. For example installations for lesser critical application such as lightning, wiring and so on are often designed by experience of the engineer. But if the failure of an anchor plate brings forth hazards for human life or can result in significant economic loss, the design process must be executed by a structural engineer according to the current norms taking both the ultimate- and serviceability limit state into consideration, (Eligehausen et al., 2006).

ETAG 001, Edition 1997, Part one by the *European Organisation for Technical Approvals* specifies four consequences of failure that the appropriate design must prevent:

a) collapse of the whole or part of the work.
b) major deformations to an inadmissible degree.
c) damage to other parts of the work or to fittings or installed equipment as a result of major deformation of the load-bearing construction.
d) damage by an event to an extent disproportionate to the original cause.

The norm continues with requirements stating that the installation shall sustain design loads in not only tension and shear but also in a combined effect of the two. Anchors are thus required to sustain tension, shear, and combined load effects during the entire working life and provide:

1) an adequate resistance to failure (ultimate limit state)
2) adequate resistance to displacements (serviceability limit state)

All of these requirements are confirmed in the new *CEN/TS 1992-4-1, Design of fastenings for use in concrete* by the *European Committee for Standardization*.

CEN/TS 1992-4-1, section 4 also add that the material of the fasteners and the corrosion protection shall be chosen with regard to the environmental conditions at the place of installation and that fasteners should be inspectable, maintainable and replaceable. Fasteners are also required to have an adequate fire resistance appropriate to the corresponding setting of operation.

4.2 Anchor plate object

The main purpose of an anchor plate is to relocate loads from a subordinated bearing system to the primary construction. In terms of anchor plates in industrial buildings (particularly nuclear facilities) the primary construction is made out of concrete and the system attached to the anchor plate commonly is a steel beam. As the loads acting on the beam may have any operational direction the anchor plate with its fasteners must be able to transfer moments, shear forces, and normal forces in all directions. The main mode of action is that all pressure forces should be taken by the concrete and all tensional forces by the fasteners. This is the general behaviour that needs to be captured in the final model for automatisation. Figure 4.1 shows static equilibrium achieved for a bending moment around the x-axis according to elastic theory.

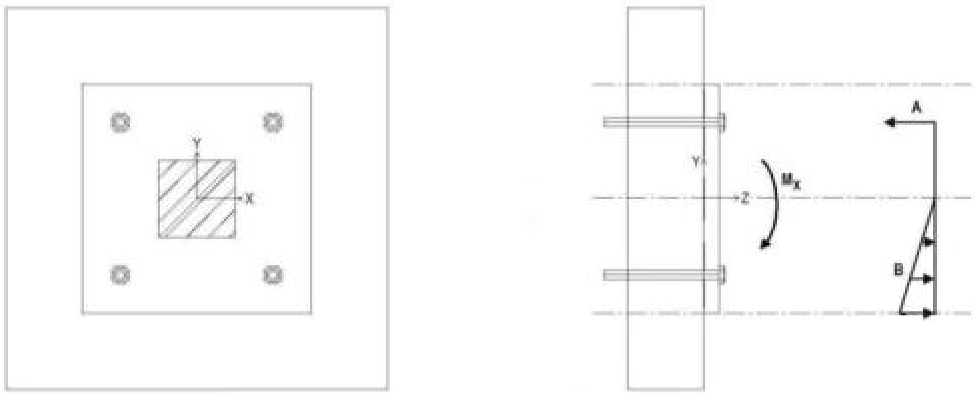


Figure 4.1. Schematic illustration of stress distribution of contact between steel plate and concrete.

The bending moment M_x in Figure 4.1 creates tension forces (A) in the two upper fasteners and a contact pressure (B) on the concrete. The dash-dotted line in the middle indicates the neutral layer, which is where the pressure on the concrete has its starting point.

Another behaviour that is important to consider is the presence of prying forces. When the steel plate deforms as it is attached by fasteners its corners will establish local pressure on the concrete surface as seen in Figure 4.2. This may even lead to concrete material failure but can be avoided by using rigid fixtures, (CEN/TS 1992-4-1, section 5.2.1).

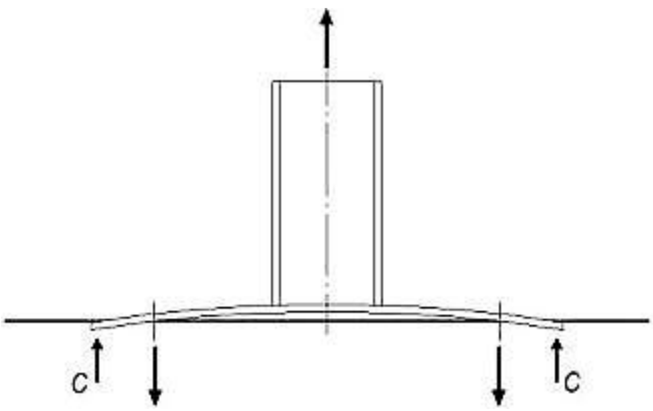


Figure 4.2. Force vectors indexed with C illustrates prying actions, (CEN/TS 1992-4-1).

4.3 Steel plate

The steel plate fastened upon the base material consists of homogenous steel and serves among others the purpose of transmitting compressive loads to the concrete. Shear forces are induced in the fasteners connecting the steel plate to the concrete. However there are some special solutions of design which imply spreading of load effects in the construction. By welding a piece of steel, so called shear lug, to the backside of the steel plate (casted into the concrete) this will unburden the fasteners when excited with transverse loads (ACI 349).

4.4 Fasteners

4.4.1 General

Without going into the specifics of fastener failure behaviours, but instead referring the reader to Chapter 3.3 of *Anchorage in Concrete Construction* by *Rolf Eligehasuen, Rainer Mallée and John F. Silva,* the conclusions of their studies are presented briefly.

Experimental tests and theoretical studies have been made for a headed fastener embedded in concrete and loaded in tension. Circumferential tensile stresses instantly develop in the vicinity of the anchor head. As the load reaches service level the stresses will result in micro crack formations in the concrete that will continue to propagate with increasing load. Depending on geometric design and material strength the failure mode will differ. If the anchor diameter is small relative to its head a steel failure is most likely to occur. If not the concrete fractures will spread and consequently lead to concrete cone failure. This cone has on average an angle 35˚with respect to a plane perpendicular to the fasteners. The different types of failure modes are more elaborately described in section 5.6 and section 5.7.

As already stated tension loads on anchorage systems can be transmitted by mechanical interlock, friction or bond, see Figure 3.1. Cast-in headed- and undercut anchors are perfect examples of operating mechanical interlock since the loads are transferred to the base material by means of a bearing interlock. Friction forces between the anchor and the sides of the hole are another way of securing fasteners. This is above all used by expansion anchors. Bonded anchors uses chemical interlock to establish adhesion to the concrete. However most of the commercial anchors available conform to a combination of these stated mechanisms.

4.4.2 Resistance of tensile forces

Anchors themselves have a tensile capacity depending on design and steel material class. However when they are merged with concrete the tensile forces will spread and affect its surrounding. Hence the local concrete tensile strength will be employed. Commonly concrete only is assumed to deal with compressive forces i.e. the tensile strength is neglected whilst tension forces entirely is carried by the reinforcement. This is not entirely true since tensile stresses in the concrete are induced by restraint of deformations such as creep, thermal movements, and shrinkage. These stresses generally acts parallel to stresses caused by external loads and may exceed the concrete tensile strength, hence lead to failure if not sufficient reinforcement is provided. For anchorages in concrete the geometry is different and the stress distribution will lead to a failure surface inclined with respect to the concrete surface as seen in Figure 4.3 b). This means the component of load stresses acting parallel to the restraint only occurs over a small part of the failure surface. According to Eligehausen for mentioned cases a maximum reduction in the concrete cone failure load of 20 % can be predicted and it is safe to exploit the concrete tensile capacity when designing anchors. Anyhow a conservative safety factor is used in the verification process.

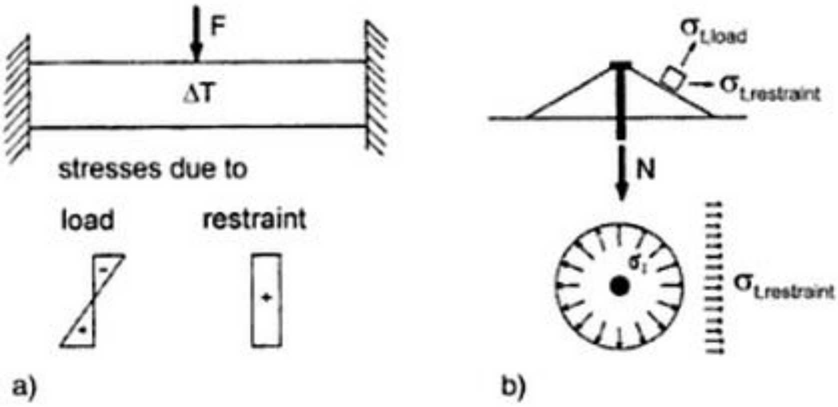


Figure 4.3. Superimposition of stresses due to load and restraint, (Eligehausen et al., 1991).

4.4.3 Pre-stressing of fasteners

Especially post installed anchors develop a pre-stressing force when mounted. When the bolts are tightened a clamping force arise between the steel plate and the base material (concrete) which is balanced by the pre-stressing force in the fastener. For threaded cast in place headed studs the same effect will arise when bolts are fastened since they will push the fixture against the concrete. In order to induce the right amount of pre-stressing force in the fastener a predefined torque is applied. The pre-stressing force $F_{S,V}$ mainly depends on the friction between the nut and washer as well the thread friction and can be described with equation 1.

$$F_{S,V} = \frac{T}{0.5 \cdot d_2 \cdot \tan(\delta_g + \alpha) + 0.5 \cdot d_k \cdot \tan(\delta_k)} \quad \text{(Eq. 1)}$$

Where:

T = torque	α = thread flank angle
d_2 = flank diameter	δ_g = thread friction angle
d = head friction diameter	δ_k = head friction angle

Recommended values for bolt head and thread friction is in range arctan(0.10) to arctan(0.5) depending on the surface of the fastener (Eligehausen et al., 2006, table 3.1). The clamping force, F_K, between the fixture and base material will have the same magnitude as the tension force unless it is impeded. When an external tension force, N, is applied on the anchor plate this clamping force will be reduced as the fastener gets further tensioned. Ultimately when N reaches the magnitude of the clamping force the fixture becomes loose and the anchor force will increase in proportion to N. The pre-stressing force will diminish over time due to relaxation of the steel and creep of the stressed concrete. However by re-torqueing the anchor the level of residual pre-stressing force can be raised, (Eligehausen et al., 2006).

4.5 Concrete

The reader is assumed to possess basic knowledge of how concrete material behaves when excited with external loads.

4.6 Calculations according to elastic theory

4.6.1 Tension loads

When performing elastic theory calculations for anchorage systems loaded in tension and with bending moments, R. Eligehausen states some assumptions that are made.

a) The steel plate remains plane which means the fixture must be sufficiently stiff and initially adjacent to the base material unless the anchors are configured for a stand-off installation (steel plate not in direct contact with the base material). This corresponds to Bernoulli hypothesis that plane cross sections remain plane.
b) Anchors located in the concrete compression zone will not be induced by any force unless they are configured for a stand-off installation.
c) The stiffness of all anchors assumes to be the same and proportional to the stressed cross section area and modulus of elasticity for the steel. As for the concrete the stiffness is dependent on the modulus of elasticity and the stressed area.

In practice the steel plate will deform and hence change the distribution of internal forces. Moreover, the assumptions of a triangular stress distribution in the concrete and neglecting anchor displacements are conservative.

4.6.2 Shear loads

When designing anchor plates according to CEN/TS 1992-4 the shear load shall be distributed on the different anchors in certain manners depending on load direction and distances to nearby concrete edges. How this is done is elaborately described in section 8.4.3.

There are requirements for the hole clearance between the fastener and the base material as well. If the diameters of the holes are larger than certain values these fasteners cannot be accounted for as load bearers according to some regulations. In fact it is quite complicated to determine the capacity for such anchors since their ability to carry load depend on the initial position of the anchor in the hole and its deformation response. This is why the conservative solution of “worst case” is applied meaning the total shear load should be taken by the most unfavourable anchor(s).

4.7 Calculations according to non-linear theory

When performing non-linear analyses for anchor plate constructions realistic assumptions need to be made considering stress – strain relation in the concrete, load displacement behaviour of the anchors and stiffness of the fixture. These types of calculations require both equilibrium and compatibility satisfaction whilst plastic analyses only should fulfil equilibrium. With help of computers using numerical integration techniques, non-linear problem formulations often result in accordance with practical test results. According to R. Eligehausen detailed investigations of tension- and shear loaded anchor plates according to plastic theory will overestimate the bearing capacity of the anchors. Hence elastic theory is preferable in absence of reliable numerical analysis programs, (Eligehausen et al., 2006).

5 Normative resistance calculations

5.1 Available guidelines, codes, and standards for design of anchor plates

Several documentations are available for design of concrete structures, and anchorage in particular, published by different institutes. The major publications regarding anchorage are:

- ETAG 001 guideline by the European Organisation for Technical Approvals (EOTA).
- ACI 318 and ACI 349-code by the American Concrete Institute (ACI).
- CEN/TS 1992-4 technical specification by the European Committee for Standardization.

Research findings published in 1995 by the Comité Euro-International du Béton (CEB) acted as stepping stone for an innovative method of analysis for fasteners set in concrete materials. The method takes into account the directions of working loads as well as different kind of failure modes necessary to optimise the utilisation of the anchors. The design concept was called the CC-method (Concrete Capacity method) and is applicable on cast-in-place headed anchors with or without supplementary reinforcement, post-installed expansion anchors and undercut anchors, (Eligehausen et al., 2006).

Common for all three stated publications listed above is that they have all adopted the CC-method, first the European Organisation for Technical Approvals (EOTA) adopted the CC-method in 1997 which in late 2008 lead to an approval of the newly developed technical specification for fasteners in concrete constructions, also called CEN/TS 1992-4. The approval granted is still on a provisional basis and valid for three years. After this period of time the technical specification will be re-evaluated, and if accepted converted into a European Standard by the European Committee for Standardisation, (Eligehausen et al., 2006).

In 2005 the American Concrete Institute followed their European colleagues and published their report on safe design of fasteners in concrete constructions, ACI 318-05. Later on they also published a code on design in nuclear facilities, ACI 349-01, (Eligehausen et al., 2006).

All of the mentioned publications cover safety design of cast-in-place headed fasteners and post-installed fasteners of various types. However, the major difference between the publications is that CEN/TS and ACI also include safety design of anchor plates with dynamic load effects, while the ETAG publication only is valid for non-dynamic loading, (Eligehausen et al., 2006).

5.2 Safety design concept

As stated by Eligehausen et al. in *Anchorage in Concrete Construction (2006)* a "safe design requires not only detailed planning and design but also anchor systems that function reliably under job-site conditions". This is enforced by several organisations and authorities that validate the material quality of produced anchor systems, but also technical properties in so called technical approvals whereas an anchor undergoes rigorous testing.

Safe design is maintained by collaboration between all parties involved in the installation of anchor systems. It is important that the manufacturer publish necessary information for a system, the user upholds required maintenance levels, the planner designs the system correctly according to the corresponding code, and the installer mounts the system properly, (Eligehausen et al., 2006).

This process is illustrated in Figure 5.1.

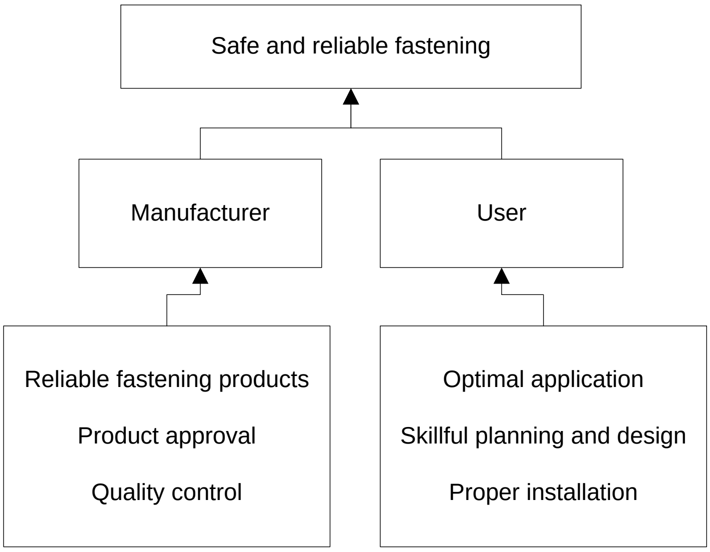


Figure 5.1. Cooperation scheme between actors in the anchor plate safety design process, (Eligehausen et al., 2006).

General concept of design for the CC-method can be explained by the following conditions which have to be fulfilled for all active modes of failure associated with the studied anchor plate system.

For the ultimate limit state and the limit state of fatigue it must be shown that $E_d \leq R_d$.

Where:

E_d = design value of effect of actions

$R_d = \frac{R_k}{\gamma_M}$ = design value of resistance

where:

R_k = characteristic resistance of single fastener or group of fasteners

γ_M = partial factor for resistance

For the serviceability limit state it must be shown that $E_d \leq C_d$.

where:

E_d = design value of fastener displacement

C_d = nominal value, e.g. limiting displacement

Above stated conditions must be met for failure modes of tension- and shear load respectively. Also capacity for simultaneous loading as a result of interaction of maximum tension- and shear loads has to be verified.

Identified anchor plate failure modes include:

Tension loads

- Steel failure of fastener
- Pull-out failure of fastener
- Concrete cone failure
- Concrete splitting failure
- Concrete blow-out failure
- Steel failure of reinforcement

Shear loads

- Steel failure without lever arm
- Steel failure with lever arm
- Concrete pry-out failure
- Concrete edge failure

However, depending on criteria's specified in related design publications one or several modes of failure may be cancelled from the validation process due to improbable occurrence of the specific mode.

5.3 Choice of normative method of verification

The choice of design code to be used in this master thesis fell on the technical specification *CEN/TS 1992-4 - Design of fastenings for use in concrete*. A Technical Specification (TS) is a normative document produced and approved by a Technical Committee.

This is how the European Committee for Standardization describes the status of a CEN/TS publication:

"*Technical specifications can be developed by CEN Technical Committees as a pre-standard which contains technical requirements for innovative technology, or when various alternatives need to coexist in anticipation of future harmonization that would not gather enough as to allow agreement on a European Standard (EN). A TS does not have the status of an European Standard but may be adopted as national standard. Moreover there is no standstill, no public enquiry and no weighted vote. However after the trial period the CEN/TS will be re-evaluated and later on maybe accepted as a European Standard if no conflicts are to find.”*

(CEN official webpage, 2011)

Because of the fact that the CEN/TS 1992-4 publication may become a future European Standard and that national building codes are already implementing the EN-codes, the choice was quite natural. Moreover, the methodology developed by Scanscot Technology AB and partly utilised in this report is also written with the CEN/TS in focus.

See chapter 8 for a comprehensive description over how the implementation of normative verification in the automatised design process is made.

5.4 Limitations of CEN/TS 1992 technical specification

CEN/TS 1992-4-1 state in section 1 the restrictions of the norm.

The design methods of the CEN/TS 1992 are admissible to the following anchor types:

- Cast-in fasteners such as headed anchors and anchor channels with rigid connections.
- Post-installed fasteners such as expansion anchors, undercut anchors, and bonded anchors.

Moreover the anchors must be suitable for installation in concrete members and cover the requirements made by CEN/TS. If the fastener differ from the European Standards and the CEN/TS it may still be admissible if a European Technical Approval (ETA) is provided. This is usually the case for post-installed anchors where the construction of the fasteners provides scattered results.

The installation of the anchor plates are assumed to be carried out by professionals which grant no erroneous systems. All fasteners attached to an anchor plate are also assumed to be of the same dimensions and type.

When tension loads or bending moments are applied on the anchor plate, compressive forces in the construction element will be produced which must be transferred to the base material. One can generally not include compressive forces taken by the anchor, even though the fastener does have this function (e.g. headed studs). Anchors must be specifically designed for this purpose if it is to be utilised. (See Figure 5.2 below where the utmost figure to the right bear all compression loads through the anchor studs. The figure in the middle shows the importance of using washer bearings in the case of stand-off installed anchor plates enabling distribution of compressive stresses throughout the concrete member.)

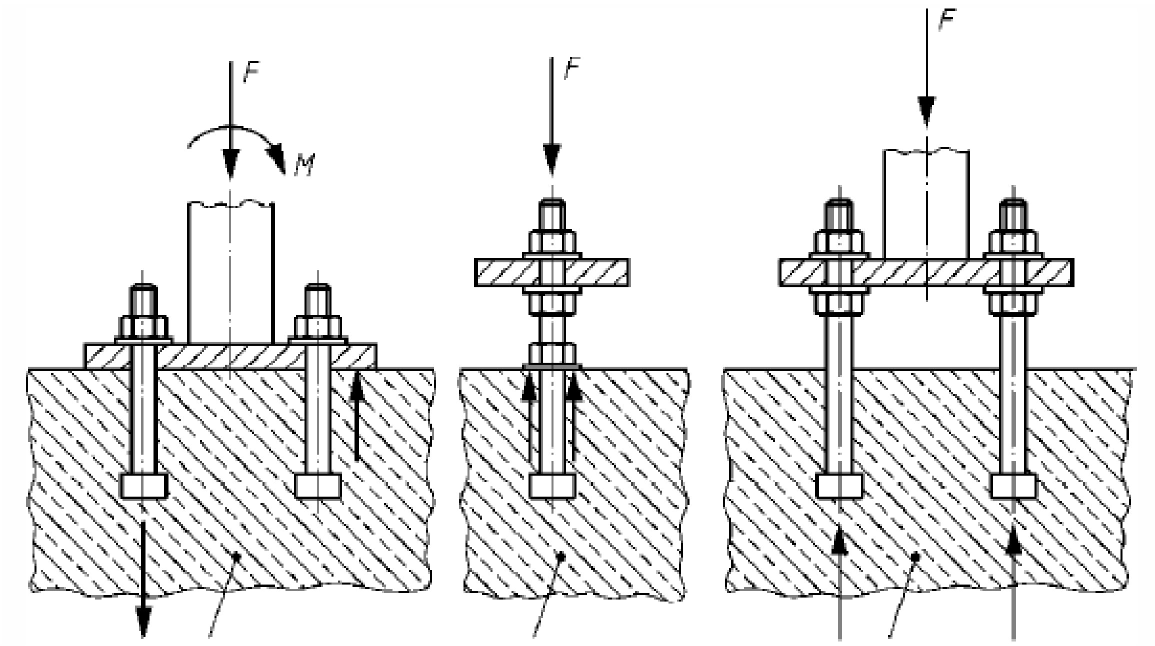


Figure 5.2. Transfer of compression and tension loads to the concrete member, (CEN/TS 1992-4-1, Figure 2, 2009).

Additional limitations regarding force distribution effects such as the above stated is the fact that eventual friction between the steel plate and the concrete surface which can increase the performance of the anchor plate cannot be accounted for. The reason is because it is difficult to quantify with confidence the effect of friction on the resistance against shear forces.

Moreover, limitations concerning general material properties are listed as following.

Fasteners must be made of:

- Carbon steel (ISO 898)
- Stainless steel (EN 10088, ISO 3506)
- Malleable cast iron (ISO 5922)

Fasteners may:

- include other non-bearing materials such as plastics
- have a nominal steel tensile strength up to 1000 MPa (f_{yk})

Choice of concrete base material is limited:

- from European strength class C12/15
- to European strength class C90/105

5.5 Cracked and non-cracked concrete

An important factor that will affect the capacity calculations of the various failure modes is the determination if the anchor and its fasteners are installed in cracked or non-cracked concrete.

The condition of the concrete base material will have impact on the partial safety factor with which the equation of resistance to failure is multiplied by. If the concrete is cracked the partial factor will be set to a lower value than what would be the case if the concrete would be non-cracked. Values which are to be used are given dependent on what failure mode is studied.

Determination of the concrete condition is regulated by the *CEN/TS 1992-4-1* documentation.

In paragraph 5 it is stated that the condition of the concrete should be determined by the designer and that in general, it is always conservative to assume that the concrete is in fact cracked. Non-cracked concrete may be assumed if it is proven that under the service conditions the fastener with its entire embedment depth is located in non-cracked concrete. This can be satisfied by the following equation:

$$\sigma_L + \sigma_R \leq \sigma_{adm}$$

Where:

σ_L = stresses induced by external loads

σ_R = stresses induced by restraints

σ_L = admissible tensile stress for non-cracked concrete (Recommended value = 0 Pa)

The stresses should be calculated assuming that the concrete is non-cracked. For concrete members which transmit loads in two directions, the stress-condition above must be satisfied in both directions.

Moreover, the concrete condition must be assumed to be cracked when designing against seismic load situations.

5.6 Cast-in-place headed anchors

This chapter will describe the normative capacity calculations for headed fasteners according to the technical specification *CEN/TS 1992-4-2 – Headed fasteners*. Calculations are based on the limit state concept used in conjunction with a partial factor method.

Forces on the fasteners are assumed as being calculated using elastic analysis, hence the set of equations presented in Table 5-1 and Table 5-2 shall be used for tension and shear force respectively. If the forces on the fasteners are calculated using plastic analysis instead another set of equations should be used for verification. (See *CEN/TS 1992-4-1, Annex B.*)

5.6.1 Verification of supplementary reinforcement

When designing cast-in-place headed anchors the supplementary reinforcement in the region of the anchor plate and its fasteners must be verified. Supplementary reinforcement is the bending and shear reinforcement placed in the concrete member to extend the concrete tension capacity.

Failure of the supplementary reinforcement is not regarded as an anchor plate failure mode, but must be verified as a design requirement when designing the concrete in which the anchor plate is fastened. Therefore the presence of supplementary reinforcement will not affect the calculations of the failure mode capacities by, for example, introducing a partial safety factor.

Since this master’s thesis work only focuses on the verification of anchor plate failure modes, the verification of supplementary reinforcement will not be implemented in this methodology.

5.6.2 Required verifications for fasteners loaded in tension

In the presence of supplementary reinforcement conditions ①, ② and ④ to ⑦ in Table 5-1 must be verified. The concrete cone failure mode is omitted and replaced with verification of the tension load resistance of supplementary reinforcement regarding the total load effect.

When no supplementary reinforcement exists conditions ① to ⑤ in Table 5-1 applies. If these are verified (with or without actual supplementary reinforcement) the outcome will be conservative.

Table 5-1. Required verification for fasteners loaded in tension. (CEN/TS 1992-4-2, Table 1)

		Single fastener	Fastener group	
			most loaded fastener	*fastener group*
①	**Steel failure of fastener**	$N_{Ed} \leq N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}}$	$N_{Ed}^{h} \leq N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}}$	-
②	**Pull-out failure of fastener**	$N_{Ed} \leq N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}}$	$N_{Ed}^{h} \leq N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}}$	-
③	**Concrete cone failure**	$N_{Ed} \leq N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}}$	-	$N_{Ed}^{g} \leq N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}}$
④	**Splitting failure**	$N_{Ed} \leq N_{Rd,sp} = \frac{N_{Rk,sp}}{\gamma_{Msp}}$	-	$N_{Ed}^{g} \leq N_{Rd,sp} = \frac{N_{Rk,sp}}{\gamma_{Msp}}$
⑤	**Blow-out failure**[a]	$N_{Ed} \leq N_{Rd,cb} = \frac{N_{Rk,cb}}{\gamma_{Mc}}$	-	$N_{Ed}^{g} \leq N_{Rd,cb} = \frac{N_{Rk,cb}}{\gamma_{Ms}}$
⑥	Steel failure of reinforcement	$N_{Ed,re} \leq N_{Rd,re} = \frac{N_{Rk,re}}{\gamma_{Ms,re}}$	$N_{Ed,re}^{h} \leq N_{Rd,re} = \frac{N_{Rk,re}}{\gamma_{Ms,re}}$	-
⑦	Anchorage failure of reinforcement	$N_{Ed,re} \leq N_{Rd,a}$	$N_{Ed,re}^{h} \leq N_{Rd,a}$	-

[a] Not required for fasteners with edge distance, $c > 0{,}5\ h_{ef}$

5.6.3 Failure modes for fasteners loaded in tension

5.6.3.1 Steel failure

The characteristic capacity in case of steel failure (Figure 5.3) is calculated according to the relevant European Technical Specification. Furthermore it is stated that the strength calculation is based on the characteristic ultimate tensile strength.

The calculated capacity is to be compared with the load effect of the most loaded fastener.

Given these pre-requisites the capacity is calculated accordingly:

$$N_{Rk,s} = f_{uk} * A \qquad [N] \qquad \text{(Eq. 2)}$$

With:

f_{uk} = characteristic steel ultimate tensile strength (nominal value)

A = effective tensioned steel area

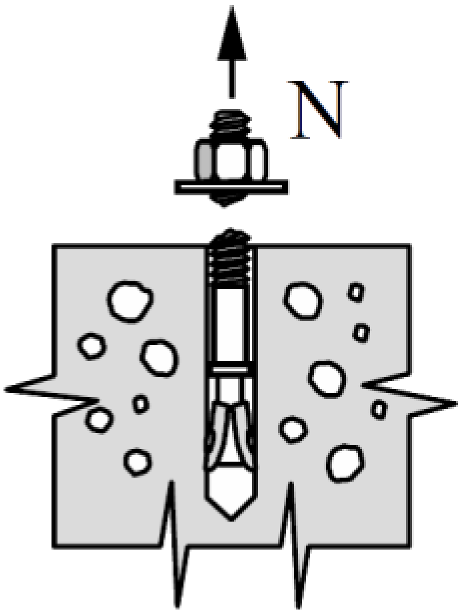


Figure 5.3. Sectional overview of the steel failure mode caused by tension forces. Note that the image shows a post-installed anchor being loaded until its breaking point, but the principle of the failure mode stands the same.

5.6.3.2 Pull-out failure

For headed cast-in-place anchors with studs the resistance is limited by the concrete pressure under the head of the fastener according to equation 3 below. Pull-out failure is illustrated in Figure 5.4 below.

The calculated pull-out capacity $N_{Rk,p}$ is to be compared with the load effect of the most loaded fastener.

$$N_{Rk,p} = 6 * A_h * f_{ck,cube} * \psi_{ucr,N} \quad [N] \qquad \text{(Eq. 3)}$$

with:

A_h = load bearing area of the head of the fastener $= \frac{\pi}{4} * \left(d_h^2 - d^2\right)$ (Eq. 4)

d_h = External diameter of the anchor studs

d = External diameter of the anchor shaft

$f_{ck,cube}$ = characteristic cube strengt of the corresponding concrete strength class

$\psi_{ucr,N}$ = 1.0 for fasteners in cracked concrete

= 1.4 for fasteners in non-cracked concrete

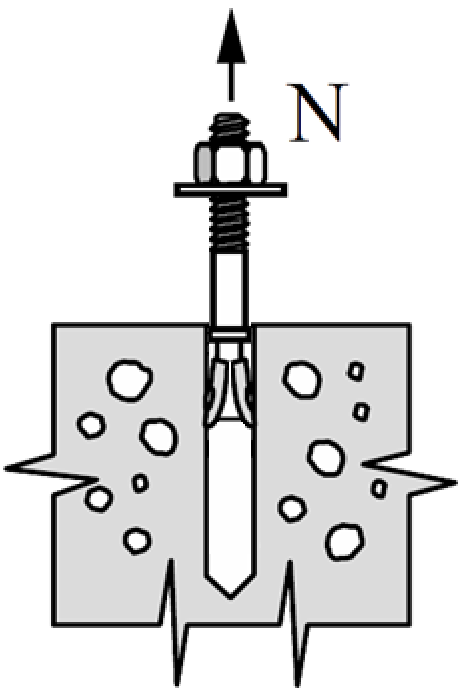


Figure 5.4. Sectional overview of the pull-out failure mode caused by tension forces. Note that the image shows a post-installed anchor being pulled out of its base material, but the principle of the failure mode stands the same.

5.6.3.3 Concrete cone failure

The characteristic resistance of a fastener, a group of fasteners and the tensioned fasteners of a group of fasteners in case of concrete cone failure may be obtained by:

$$N_{Rk,c} = N^0_{Rk,c} \; * \; \frac{A_{c,N}}{A^0_{c,N}} \; * \; \psi_{s,N} \; * \; \psi_{re,N} \; * \; \psi_{ec,N} \qquad \text{[N]} \qquad \text{(Eq. 5)}$$

with:

$N^0_{Rk,c} = k \cdot \sqrt{f_{ck,cube}} \cdot h_{ef}^{1.5}$ [N] Characteristic resistance of a single fastener (Eq. 6)

$k \; = k_{cr}$ Factor to take into account the influence of load transfer mechanisms for application in cracked concrete, the actual value is given in the corresponding European Technical Specification.

For headed fasteners the value of k_{cr} is set to 8.5 according to current experience.

$= k_{ucr}$ Factor to take into account the influence of load transfer mechanisms for application in non-cracked concrete, the actual value is given in the corresponding European Technical Specification.

For headed fasteners the value of k_{ucr} is set to 11.9 according to current experience.

$f_{ck,cube}$ Characteristic cube strength of the concrete strength class.
Specified with the unit [N/mm^2] or [MPa].

h_{ef} Effective length of anchorage (See *CEN/TS 1992-4-1*, *Figure 5*). Specified with the unit [mm].

$\frac{A_{c,N}}{A^0_{c,N}}$ Effect of axial spacing and edge distances

$\psi_{s,N} \; = \; 0.7 + 0.3 \cdot \frac{c}{c_{cr,N}} \leq 1$ Effect of disturbance on stress distribution in the concrete due to edges (Eq. 7)

$\psi_{re,N} \; = \; 0.5 + \frac{h_{ef}}{200} \leq 1$ Effect of shell spalling (Eq. 8)

$\psi_{ec,N} \; = \; \frac{1}{1+2 \cdot \frac{e_N}{s_{cr,N}}} \leq 1$ Effect of the eccentricity of the load (Eq. 9)

$c \;;\; c_{cr,N}$ Edge distance ; Characteristic edge distance

$s \;;\; s_{cr,N}$ Spacing ; Characteristic spacing

According to current experience $s_{cr,N} = 2 * c_{cr,N} = 3 * h_{ef}$.

The capacity value is to be compared against the load effect of the active anchor group. Figure 5.5 shows the concrete cone failure mode.

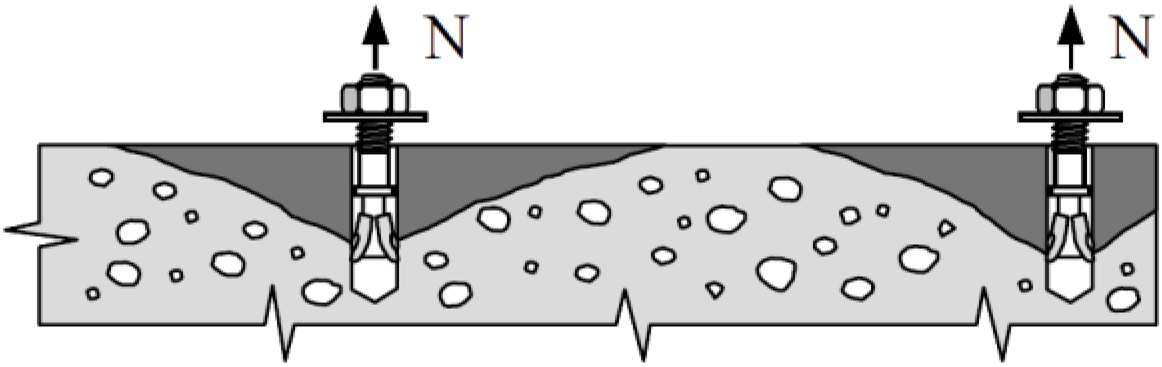


Figure 5.5. Sectional overview. Note that the image shows post-installed anchors creating concrete break out cones, but the principle of the failure mode stands the same.

The trademark of the Concrete Capacity Method (CC-method) is how the actions caused by various effects are managed.

By calculating a reference area for the corresponding effects, according to given normative procedures, where no influence from possible interferences are taken into respect and then divide the actual projected area of effect where interferences as overlapping anchors, edges and more are considered, one can derive a factor which will range in theory from 0 to n. (Where n is the number of active anchors.)

An example:

Consider an anchor plate with 4 anchors fitted to the steel plate all of which are active in the currently studied mode of failure.

Assume that the calculated reference area is 1 m^2, $A_c^0 = 1$.

If there is no interferences present (large spacing distances and edge distances) one will receive an actual projected area of 4 m^2, $A_c = 4$. The modification factor will then be $\frac{4}{1} = 4$.

But if the spacing distance between the active anchors would have been smaller, the actual projected area may have been reduced since overlapping areas cannot be taken into consideration. This would have given a modification factor lesser than 4.

This is the general method of application where the concrete area has a direct impact on the effect.

Effect of axial spacing and edge distances

The geometric effect of axial spacing and edge distance on the characteristic resistance is taken into account by the value $\frac{A_{c,N}}{A_{c,N}^{0}}$.

With:

$A_{c,N}^{0}$ = Reference projected area

= $s_{cr,N} * s_{cr,N}$

$A_{c,N}$ = Actual projected area, limited by overlapping concrete cones of adjacent fasteners $(s < s_{cr,N})$ as well as by edges of the concrete member $(c < c_{cr,N})$.

Calculation of the reference projected area:

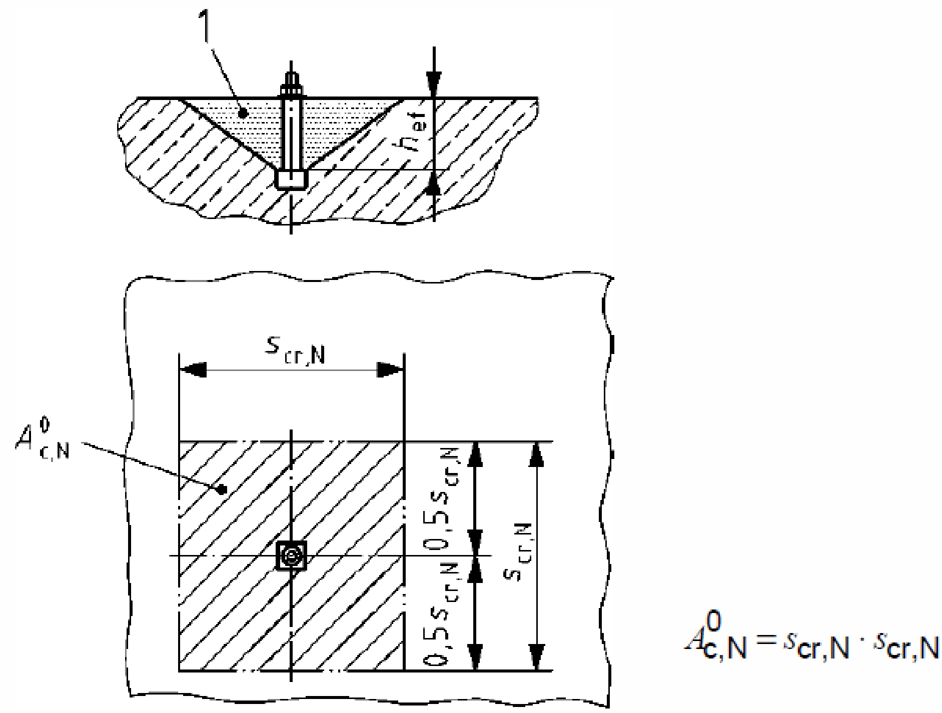
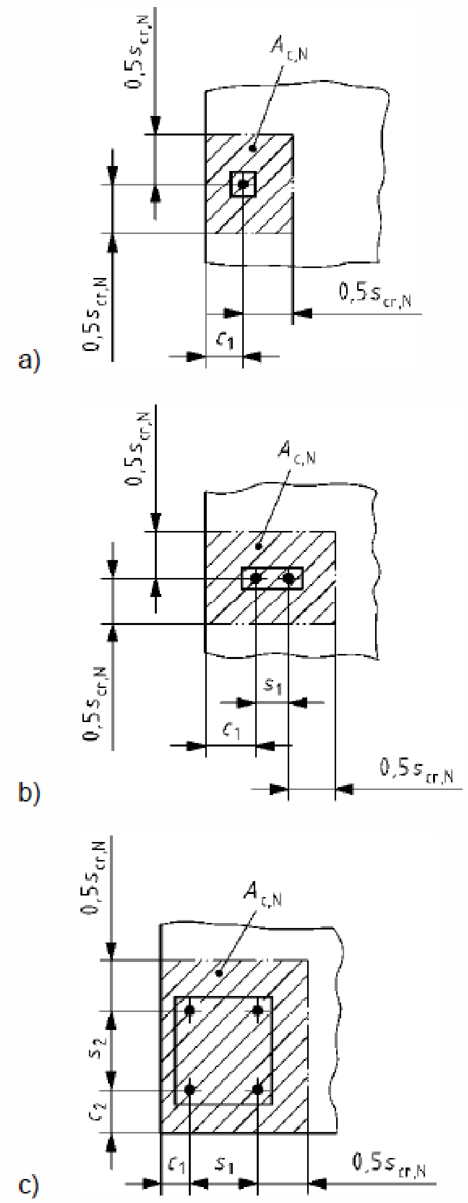


Figure 5.6. Illustration showing the projected area of failure for the concrete cone failure mode. (CEN/TS 1992-4-2, Figure 3)

$$A_{c,N}^{0} = s_{cr,N} * s_{cr,N} = 9 * h_{\text{ef}}^{2} \quad \text{(Eq. 10)}$$

Calculation of the actual projected area:



a) $A_{c,N} = (c_1 + 0{,}5\, s_{cr,N}) \cdot s_{cr,N}$

if: $c_1 \le c_{cr,N}$

b) $A_{c,N} = (c_1 + s_1 + 0{,}5\, s_{cr,N}) \cdot s_{cr,N}$

if: $c_1 \le c_{cr,N}$

$s_1 \le s_{cr,N}$

c) $A_{c,N} = (c_1 + s_1 + 0{,}5\, s_{cr,N}) \cdot (c_2 + s_2 + 0{,}5\, s_{cr,N})$

if: $c_1;\, c_2 \le c_{cr,N}$

$s_1\,;\, s_2 \le s_{cr,N}$

Key

a) Individual fastener at the edge of a concrete member
b) Group of two fasteners at the edge of a concrete member
c) Group of four fasteners at a corner of a concrete member

Figure 5.7. Illustration exemplifying calculations of projected area for fasteners influenced by other anchors and edges, (CEN/TS 1992-4-2, Figure 4).

As shown in the example above the overlapping areas generated in case of nearby anchors and/or edges are neglected in the projected area calculation.

Effect of disturbance on stress distribution in the concrete due to edges:

The effect takes account of the disturbance of the distribution of stresses in the concrete due to edges of the concrete member. For fastenings with several edge distances (close to a corner) the smallest edge distance (c_1 or c_2 according to Figure 5.7), should be used.

Effect of shell spalling:

This effect takes account of the consequence of a dense reinforcement when the embedment depth $h_{\mathrm{ef}} < 100$ mm. Shell spalling, commonly called concrete cancer, is the result when moisture in primarily salty environments initiate a corrosion process where the reinforcement material is forced to expand. In the worst case scenario a shell surface blow-out of the concrete may occur. In practice this will give little effect due to most anchors used in heavy duty applications have embedment lengths equal to or larger than 100 mm.

Effect of eccentricity of the load:

The effect takes account of a group effect when different tension loads are acting on the individual fasteners of a group. This is done by calculating the geometrical concentration point for the weighted tension forces in the anchors and then calculate the distance with signs to the concentration point for the tension anchor group weighted against each anchors tension force.

$$e_N = CP_{geometrical} - CP_{weighted\ against\ force}$$

If there exist eccentricities to the load in two directions, a factor $\psi_{ec,N}$ is calculated for each direction and the product of both factors should be inserted into equation 9.

5.6.3.4 Concrete splitting failure

According to the *CEN/TS 1992-4-2, Section 6.2.6.1* splitting failure during the installation of fasteners can be avoided by complying with minimum values for internal anchor spacing (s_{min}), edge distance (c_{min}), concrete member thickness (h_{min}) and for reinforcement of the concrete member. An illustration of the concrete splitting failure mode can be seen in Figure 5.8.

Verification of splitting failure due to loading is not required if one of the following conditions is satisfied.

1. Edge distance in all directions exceeds $1.0 * c_{cr,sp}$ for one anchor or $1.2 * c_{cr,sp}$ for an anchor group.
2. Concrete cone and pull-out failure modes are calculated with regard to cracked concrete and the reinforcement resists splitting forces and limits the crack width to $w_k \leq 0.3$ mm.

According to the European technical specification $c_{cr,sp}$ is by experience equal to $2.0 * h_{ef}$ and $s_{cr,sp} = 2{,}0 * c_{cr,sp}$

If neither one of the conditions 1 or 2 stated above is fulfilled then the characteristic resistance of one fastener or a group of fasteners is calculated accordingly:

$$N_{Rk,c} = N_{Rk}^0 * \frac{A_{c,N}}{A_{c,N}^0} * \psi_{s,N} * \psi_{ec,N} * \psi_{re,N} * \psi_{h,sp} \quad [\text{N}] \qquad \text{(Eq. 11)}$$

With:

$$N_{Rk}^0 = \min(N_{Rk,p}, N_{Rk,c}^0)$$

$N_{Rk,p}$ = Characteristic pull-out failure resistance, according to section 5.6.3.2.

$N_{Rk,c}^0$, $N_{Rk,p}$, $\psi_{s,N}$, $\psi_{ec,N}$, $\psi_{re,N}$

calculated according to concrete cone failure according to section 5.6.3.3 with the difference that values $c_{cr,N}$ and $s_{cr,N}$ are replaced with $c_{cr,sp}$ and $s_{cr,sp}$.

$$\psi_{h,sp} = \left(\frac{h}{h_{min}}\right)^{\frac{2}{3}} \leq \left(\frac{2*h_{ef}}{h_{min}}\right)^{\frac{2}{3}}$$

Effect of concrete member thickness on splitting resistance (Eq. 12)

Capacity values are to be compared against the load effect of the active anchor group.

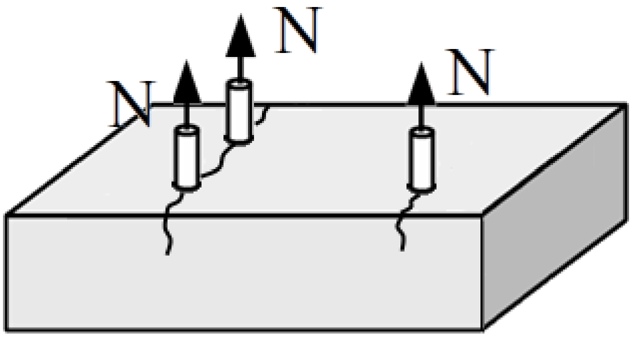


Figure 5.8. Illustration of the splitting failure mode.

Effect of concrete member thickness on splitting resistance:

The effect takes into account the influence of the actual member depth h on the splitting resistance.

5.6.3.5 Concrete blow-out failure

The concrete blow-out failure mode (Figure 5.9) is only valid for cast-in-place headed anchors. Similar to the concrete splitting failure mode, verification can be discarded if the following condition is satisfied.

- Edge distance in all directions exceeds $0.5 * h_{ef}$.

If the condition stated above is not fulfilled then the characteristic resistance in case of blow-out failure is calculated accordingly (for groups of fasteners perpendicular to the edge and loaded uniformly, verification is only required for the fasteners closest to the edge):

$$N_{Rk,cb} = N^0_{Rk,cb} * \frac{A_{c,Nb}}{A^0_{c,Nb}} * \psi_{s,Nb} * \psi_{g,Nb} * \psi_{ec,Nb} * \psi_{ucr,N} \quad [\mathrm{N}] \tag{Eq. 13}$$

With:

$N^0_{Rk,cb} = 8 * c_1 * \sqrt{A_h} * \sqrt{f_{ck,cube}}$ [N] Characteristic resistance of a single fastener (Eq. 14)

c_1	Edge distance for anchor perpendicular to the edge. Specified with the unit [mm].
c_2	Edge distance for anchor parallel to the edge. Specified with the unit [mm].
A_h	Load bearing area. See Equation 4. Specified with the unit [mm].
$f_{ck,cube}$	Characteristic cube strength of the concrete strength class. Specified with the unit [N/mm^2] or [MPa].

$\frac{A_{c,Nb}}{A^0_{c,Nb}}$ Effect of axial spacing and edge distances

$\psi_{s,Nb} = 0.7 + 0.3 \cdot \frac{c_2}{c_1} \leq 1$ Effect of disturbance on stress distribution in the concrete due to edges (Eq. 15)

$\psi_{g,Nb} = \sqrt{n} + (1 - \sqrt{n}) \cdot \frac{s_1}{4c_1} \geq 1$ Effect of bearing area on the behaviour of groups where n is number of tensioned fasteners in a row parallel to the edge (Eq. 16)

$\psi_{ec,Nb} = \frac{1}{1+2 \cdot e_N/(4*c_1)} \leq 1$ Effect of the eccentricity of the load (Eq. 17)

$\psi_{ucr,N} = $ 1.0 for fasteners in cracked concrete

$=$ 1.4 for fasteners in non-cracked concrete

The capacity value is to be compared against the load effect of the active anchor group.

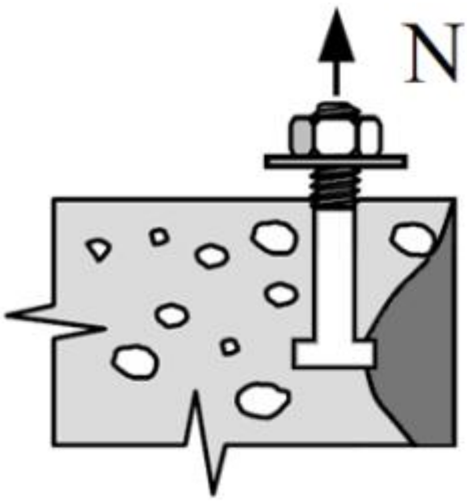


Figure 5.9. Illustration of the concrete blow-out failure mode.

Effect of axial spacing and edge distances

The geometric effect of axial spacing and edge distance on the characteristic resistance is taken into account by the value $\frac{A_{c,Nb}}{A^0_{c,Nb}}$.

With:

$A^0_{c,Nb}$ = Reference projected area, see Figure 5.10.

= $(4 * c_1)^2$ (Eq. 18)

$A_{c,Nb}$ = Actual projected area, limited by overlapping concrete cones of adjacent fasteners ($s < 4 * c_1$) and by edges of the concrete member ($c_2 < 2 * c_1$), see Figure 5.11.

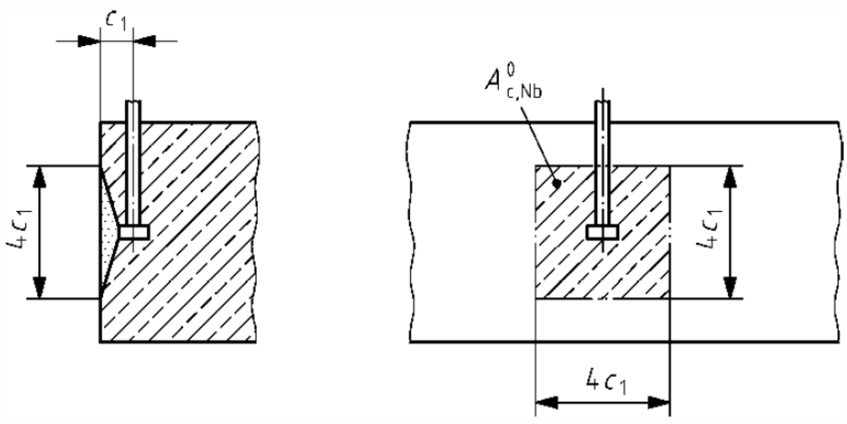


Figure 5.10. Illustration showing the projected area of failure for a single un-interfered anchor for concrete blow-out failure mode, (CEN/TS 1992-4-2, Figure 7).

Calculation of the actual projected area:

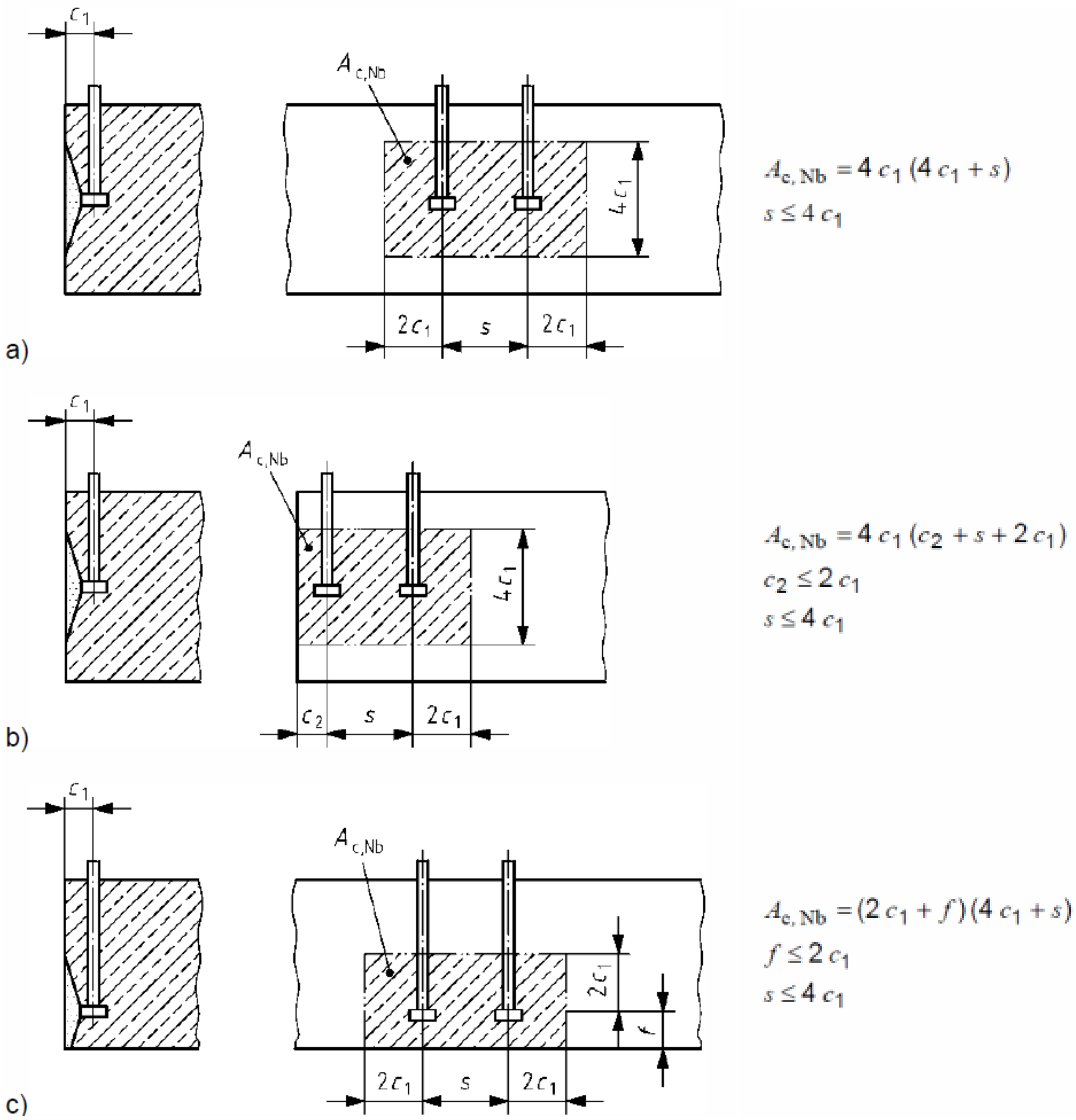


Figure 5.11. Illustration exemplifying calculations of projected area for fasteners influenced by other anchors and edges, (CEN/TS 1992-4-2, Figure 8).

As shown in the example above the overlapping areas generated in case of nearby anchors and/or edges are neglected in the projected area.

Effect of disturbance on stress distribution in the concrete due to edges:

The effect takes account of the disturbance of the distribution of stresses in the concrete due to edges of the concrete member. For fastenings with several edge distances (close to a corner) the smallest edge distance, c_2 , should be used.

Effect of bearing area on the behaviour of groups:

The effect takes account of the group behaviour when verifying the blow-out failure mode. If spacing between the anchors is large the stress distribution of the group can act favourably and increase the characteristic resistance.

Effect of eccentricity of the load:

See section 5.6.3.3.

5.6.4 Required verifications for fasteners loaded in shear

In the presence of supplementary reinforcement conditions ①, ② and ④ to ⑥ in Table 5-2 must be verified. The concrete edge failure mode is omitted. See Section 6.3.2 in *CEN/TS 1992-4-2* for more information regarding detailing of supplementary reinforcement and other requirements.

When no supplementary reinforcement exists conditions ① to ④ in Table 5-2 applies.

Table 5-2. Required verifications for fasteners loaded in shear, (CEN/TS 1992-4-2, Table 2).

		Single fastener	Fastener groups	
			most loaded fastener	*fastener group*
①	**Steel failure of fastener without 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 with lever arm**	$V_{Ed} \leq V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{Ms}}$	$V_{Ed}^{h} \leq V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{Ms}}$	-
③	**Concrete edge failure**	$V_{Ed} \leq V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{Mc}}$	-	$V_{Ed}^{g} \leq V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{Mc}}$
④	**Concrete pry-out failure**	$V_{Ed} \leq V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{Mc}}$	-	$V_{Ed}^{g} \leq V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{Mc}}$
⑤	Steel failure of supplementary reinforcement	$V_{Ed,re} \leq V_{Rd,re} = \frac{V_{Rk,re}}{\gamma_{Ms,re}}$	$V_{Ed,re}^{h} \leq V_{Rd,re} = \frac{V_{Rk,re}}{\gamma_{Ms,re}}$	-
⑥	Anchorage failure of supplementary reinforcement	$V_{Ed,re} \leq N_{Rd,a}$	$V_{Ed,re}^{h} \leq N_{Rd,a}$	-

5.6.5 Failure modes for fasteners loaded in shear

5.6.5.1 Steel failure without lever arm

The characteristic capacity in case of steel failure is calculated according to the relevant European Technical Specification. Furthermore it is stated that the strength calculation is based on the characteristic ultimate tensile strength. See Figure 5.12 for steel failure mode.

The calculated capacity is to be compared with the load effect of the most loaded fastener.

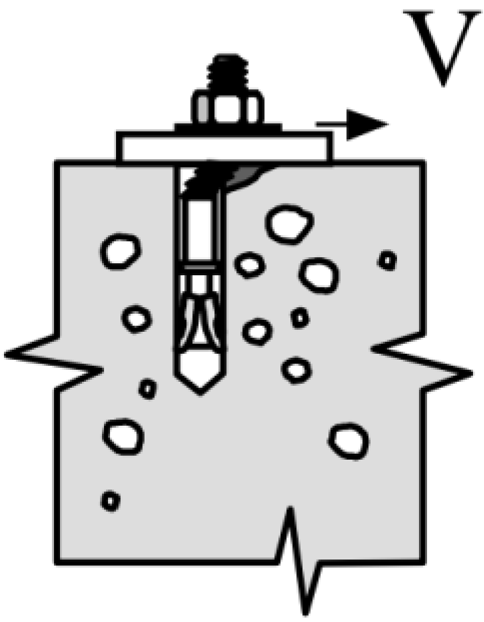


Figure 5.12. Sectional overview of the steel failure mode caused by shear forces. Note that the image shows a post-installed anchor being loaded until its breaking point, but the principle of the failure mode is the same.

5.6.5.2 Steel failure with lever arm

The characteristic capacity in case of steel failure for shear load with lever arm is calculated according to the equation below. Figure 5.13 illustrates the failure mode.

The calculated capacity is to be compared with the load effect of the most loaded fastener.

$$V_{Rk,s} = \frac{\alpha_m * M_{Rk,s}}{l} \quad [N] \qquad \text{(Eq. 19)}$$

With:

α_m , l = See Section 5.2.3.3 in *CEN/TS 1992-4-1*.

$$M_{Rk,s} = M^0_{Rk,s} * \left(1 - \frac{N_{Sd}}{N_{Rd,s}}\right) \quad [N] \qquad \text{(Eq. 20)}$$

$$N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}} \quad [N] \qquad \text{(Eq. 21)}$$

The characteristic resistance under tension load in case of steel failure ($N_{Rk,s}$), the partial safety factor (γ_{Ms}) and the characteristic bending resistance of a single headed fastener ($M^0_{Rk,s}$) are given in the relevant European Technical Specification.

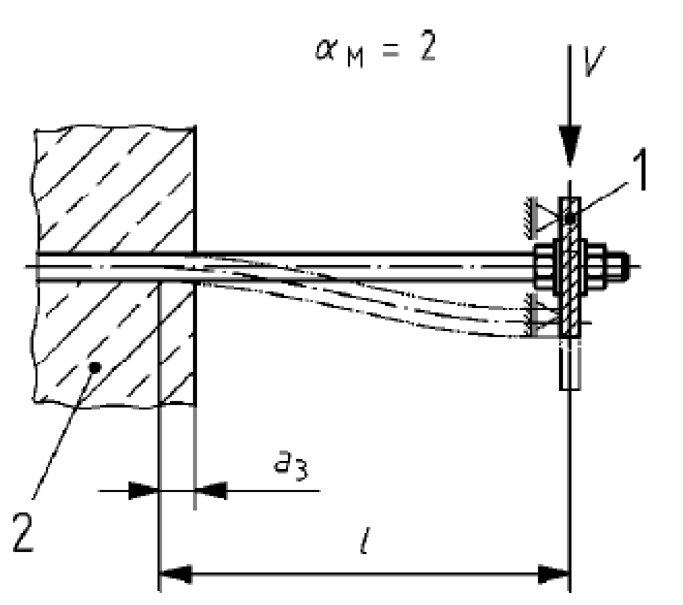


Figure 5.13. Sectional overview of the steel failure mode caused by shear forces.

5.6.5.3 Concrete pry-out failure

Fastenings may fail due to concrete pry-out as illustrated by Figure 5.14 below. The failure mode is characterised by crushing of the concrete near the surface on the same side of the acting load and pry-out of the concrete at the side opposite to the load direction, hence the name concrete pry-out failure.

The characteristic resistance $V_{Rk,cp}$ is calculated according to equation 22 below.

$$V_{Rk,cp} = k_3 * N_{Rk,c} \quad [\mathrm{N}] \qquad \text{(Eq. 22)}$$

With:

k_3 = Factor to be taken from the relevant European Technical Specification, valid for applications without supplementary reinforcement. In case of supplementary reinforcement the factor should be multiplied with 0.75.

$N_{Rk,c}$ = Calculated as for concrete cone failure according to section 5.6.3.3, determined for fasteners loaded in shear instead of tension.

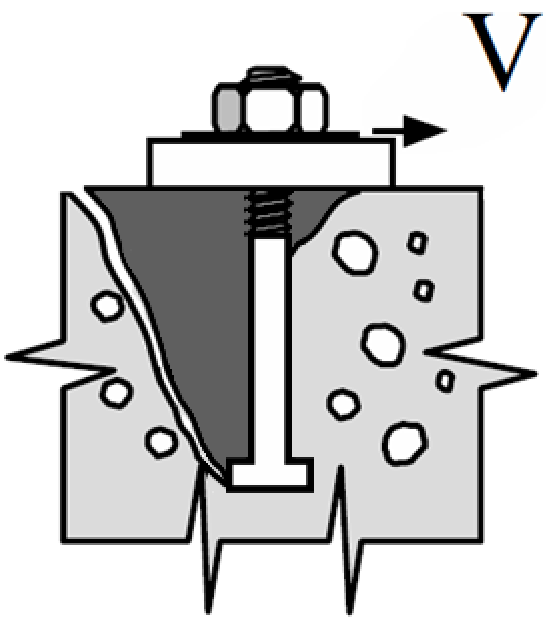


Figure 5.14. Sectional overview of the concrete pry-out failure mode caused by shear forces.

The calculated resistance is to be compared against the load effect of the group of fasteners loaded with shear forces. However when applying external loads to the anchor plate, shear forces and torsion moments may act favourably and eliminate each other. In such cases the anchor loaded by the largest individual shear force is considered decisive.

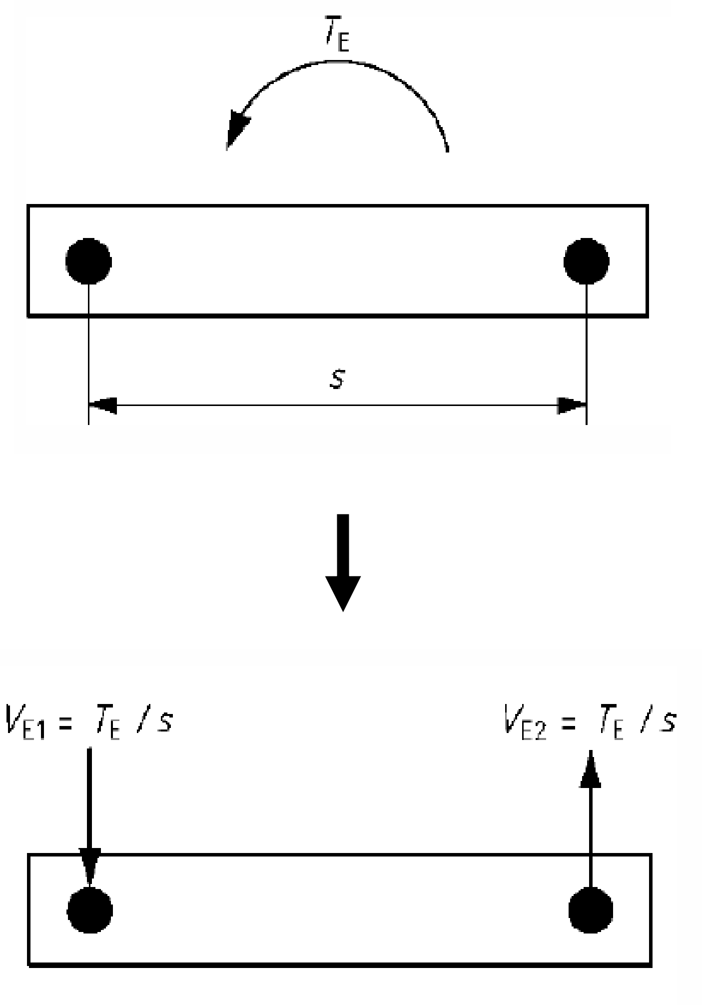


Figure 5.15. External loads acting favourably eliminating each other. The torsion moment is divided into it's equivalent force components.

In the example illustrated by Figure 5.15 the shear components of the torsion moment will result in a total shear force $V_{Ed} = 0$ thus the resistance should be verified against the most sheared individual fastener, in this case $V_{Ed} = \frac{T_\varepsilon}{s}$.

5.6.5.4 Concrete edge failure

As stated in *CEN/TS 1992-4-2, Section 6.3.5.1* the following conditions shall be observed:

- For single fasteners and groups with not more than 4 fasteners and with an edge distance in all directions $c \geq 10 * h_{ef}$ or $c \geq 60 * d$, a check of characteristic concrete edge failure may be omitted. The smaller value is decisive.
- For fastenings with more than one edge, the resistance for all edges shall be calculated. The smaller value is decisive.
- For groups of fasteners arranged perpendicular to the edge and loaded parallel to the edge or by a torsional moment where $s_1 < c_1$ and $c_1 < 150\,mm$ the design method for concrete edge failure may yield unconservative results.

The characteristic resistance in case of concrete edge failure is calculated according to equation 23 below.

$$V_{Rk,c} = V_{Rk,c}^0 * \frac{A_{c,V}}{A_{c,V}^0} * \psi_{s,V} * \psi_{h,V} * \psi_{ec,V} * \psi_{\alpha,V} * \psi_{re,V} \quad [N] \qquad \text{(Eq. 23)}$$

With:

$$V_{Rk,c}^0 = 1.6 * d_{nom}^{\alpha} * l_f^{\beta} * \sqrt{f_{ck,cube}} * c_1^{1.5} \quad [N]$$ Characteristic resistance of a single fastener (Eq. 24)

d_{nom}	Outside diameter of fastener. (≤ 60 mm) Specified with the unit [mm].
l_f	$= h_{ef}$ in case of a uniform diameter of the shank of the headed fastener. ($\leq 8 * d_{nom}$) Specified with the unit [mm].
α	$= 0.1 * \left(\frac{l_f}{c_1}\right)^{0.5}$ (Eq. 25)
β	$= 0.1 * \left(\frac{d_{nom}}{c_1}\right)^{0.2}$ (Eq. 26)
c_1	Edge distance for anchor perpendicular to the edge. Specified with the unit [mm].
$f_{ck,cube}$	Characteristic cube strength of the concrete strength class. Specified with the unit [N/mm^2] or [MPa].

$\frac{A_{c,V}}{A^0_{c,V}}$ Effect of axial spacing and edge distances

$\psi_{s,V} = 0.7 + 0.3 \cdot \frac{c_2}{1.5 * c_1} \leq 1$ Effect of disturbance on stress distribution in the concrete due to edges (Eq. 27)

$\psi_{h,V} = \left(\frac{1.5 * c_1}{h}\right)^{0.5} \geq 1$ Effect of the thickness of the structural component (Eq. 28)

$\psi_{ec,V} = \frac{1}{1 + 2 \cdot e_N / (4 * c_1)} \leq 1$ Effect of the eccentricity of the load (Eq. 29)

$\psi_{a,V} = \sqrt{\frac{1}{(\cos \alpha_V)^2 + (0.4 * \sin \alpha_V)^2}} \leq 1$ Effect of the load direction (Eq. 30)

α_V = angle between design shear load V_{Sd} and a line perpendicular to the edge. ($0° \leq \alpha_V \leq 90°$)

$\psi_{re,V}$ = 1.0 for fasteners in cracked concrete without edge reinforcement

= 1.2 or 1.4 for fasteners in cracked concrete with reinforcement.

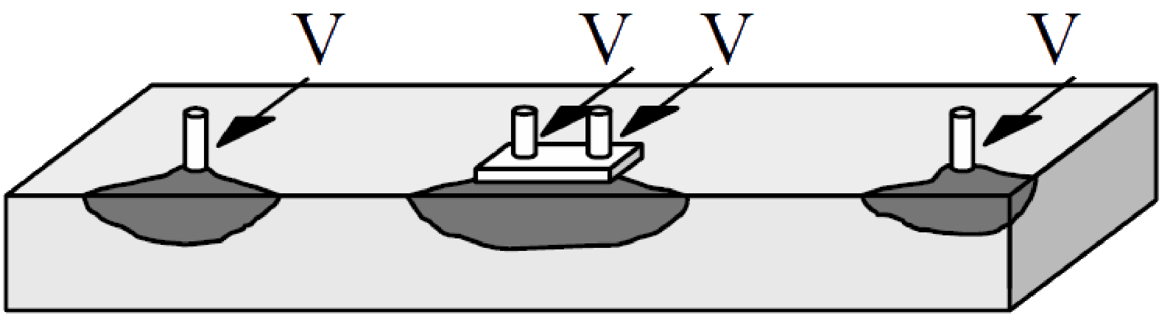


Figure 5.16. The concrete edge failure mode exemplified for single anchor, group of anchors, and near corner.

Effect of axial spacing and edge distances

The geometric effect of axial spacing and edge distance on the characteristic resistance is taken into account by the value $\frac{A_{c,V}}{A^0_{c,V}}$.

With:

$A^0_{c,V}$ = Reference projected area (Figure 5.17)

= $4.5 * c_1^2$ (Eq. 31)

$A_{c,V}$ = Actual projected area, limited by overlapping concrete cones of adjacent fasteners ($s < 3 * c_1$) as well as by edges of the concrete member ($c_2 < 1.5 * c_1$) and concrete member depth ($h < 1.5 * c_1$). See Figure 5.18.

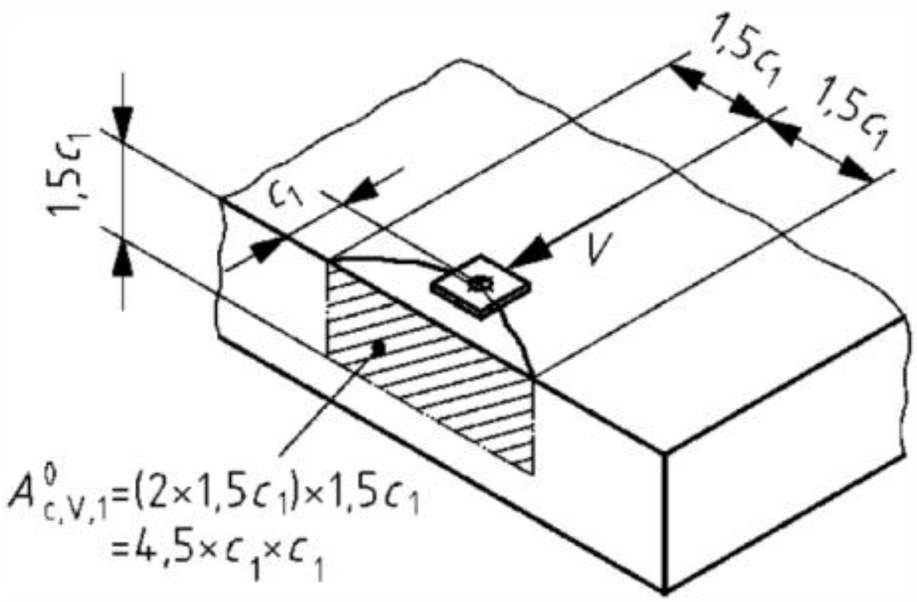


Figure 5.17. Illustration showing the projected area of failure for a single un-interfered anchor for concrete edge failure mode, (CEN/TS 1992-4-2, Figure 16).

Calculation of the actual projected area:

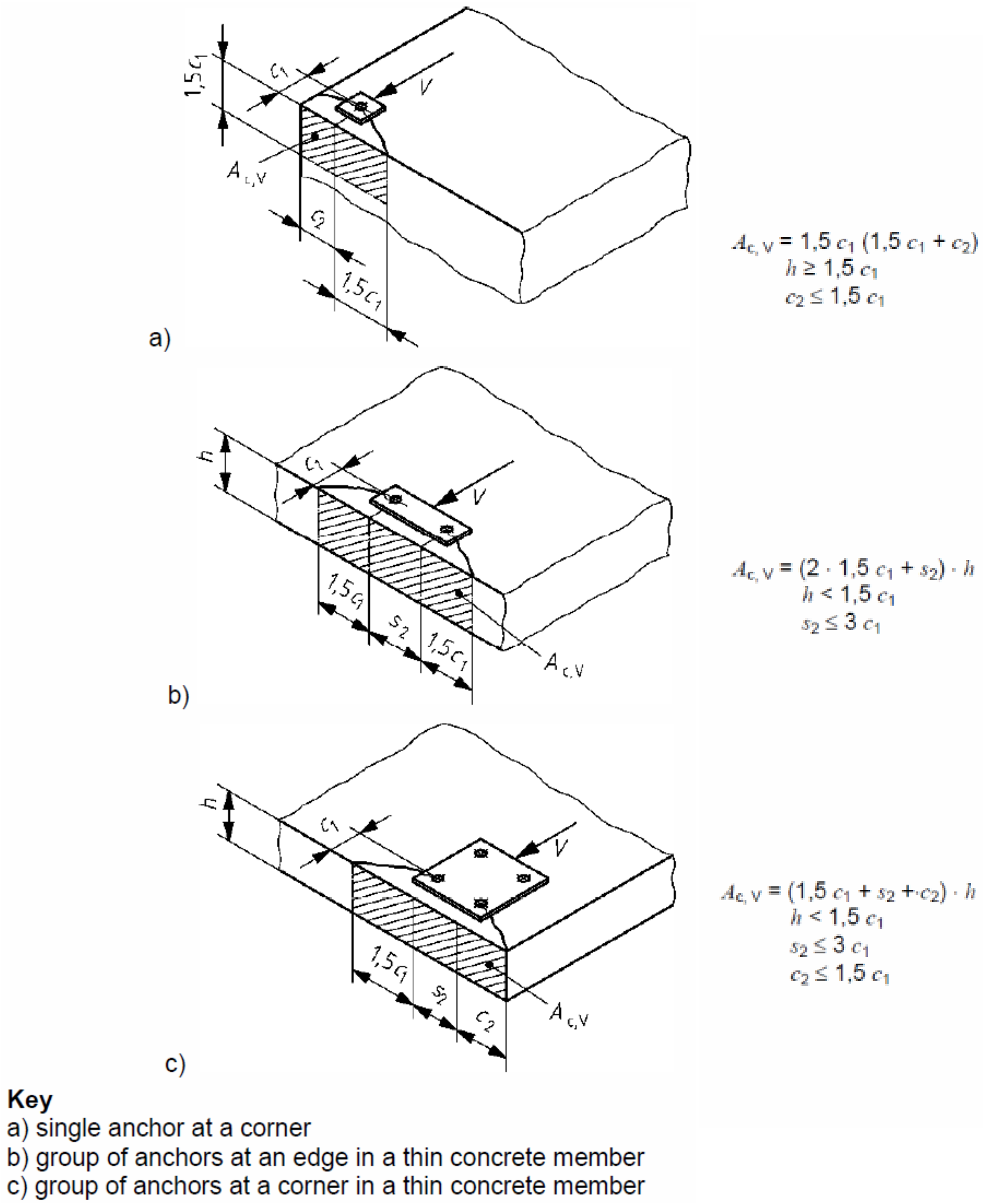


Figure 5.18. Illustration exemplifying calculations of projected area for fasteners influenced by other anchors and edges, (CEN/TS 1992-4-2, Figure 8).

As shown in the example above the overlapping areas generated in case of nearby anchors and/or edges are neglected in the projected area calculation.

Resistance values are to be compared to the shear load of the entire active anchor group.

Effect of disturbance on stress distribution in the concrete due to edges:

The effect takes account of the disturbance of the distribution of stresses in the concrete due to further edges of the concrete member. For fastenings with two edges parallel to the direction of loading (e.g. in a narrow concrete member) the smaller edge distance should be used.

Effect of the thickness of the structural component:

The effect takes account of the fact of the concrete edge resistance does not decrease proportionally to the member thickness as assumed by the area ratio $\frac{A_{c,V}}{A_{c,V}^{0}}$.

Effect of eccentricity of the load:

The effect takes account of a group effect when different shear loads are acting on the individual fasteners of a group, see Figure 5.19. This is done by calculating the geometrical concentration point for the shear loaded anchors and then calculate the distance with signs to the concentration point for the sheared anchor group weighted against each anchor's shear force.

$$e_N = CP_{geometrical} - CP_{weighted\ against\ force}$$

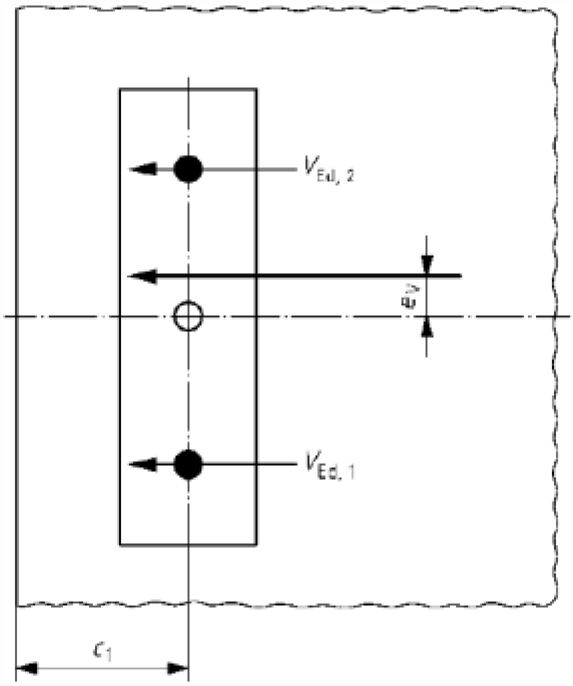


Figure 5.19. Eccentricity of load (Ved,2 > Ved,1).

Effect of the load direction:

The effect takes account of the angle between the applied load (not separated into components) and the direction perpendicular to the free edge under consideration for the calculation of concrete edge resistance.

Effect of the position of the fastening:

Self explanatory. Note that a factor larger than 1.0 for application in cracked concrete is only valid if the embedment depth, h_{ef}, is greater than $2.5 * t_{\text{concrete cover of edge reinforcement}}$.

Effect of a narrow thin member:

Since our finite element model is limited to a maximum of two free edges according to section 7.1.1, the effect of narrow thin members can never occur. The reader is referred to *CEN/TS 1992-4-2, Section 6.3.5.2.8* for more information regarding this effect.

5.7 Post-installed expansion anchors

For the design of post-installed fasteners in the ultimate limit state, there are three different design methods available.

What differentiates the methods is the level of simplicity at the cost of result conservatism. The three methods are:

- Method A: Resistance is established for all load directions and all modes of failure using actual values of edge- and spacing distances. This method is the least conservative.
- Method B: A single value of resistance is used for all load directions and all modes of failure. This resistance is related to characteristic values c_{cr} and s_{cr} . It is permitted to use smaller values than these, however the resistance should then be modified as such.
- Method C: A single value of resistance is used for all load directions and all modes of failure. This resistance is related to characteristic values c_{cr} and s_{cr} . The values for c and s cannot be smaller than c_{cr} and s_{cr} . This method is the most conservative.

Method A is chosen since the value of this entire project is result accuracy when calculating the load effects and thus it is important to maintain accuracy also when calculating the failure mode capacities.

5.7.1 Required verifications for fasteners loaded in tension

For post-installed expansion anchors equations ① through ④ in Table 5-3 are to be verified in case of fasteners loaded in tension.

Table 5-3. Required verification for fasteners loaded in tension. (CEN/TS 1992-4-4, Table 3)

		Single fastener	Fastener group	
			most loaded fastener	*fastener group*
①	**Steel failure of fastener**	$N_{Ed} \leq N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}}$	$N_{Ed}^{h} \leq N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}}$	-
②	**Pull-out failure of fastener**	$N_{Ed} \leq N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}}$	$N_{Ed}^{h} \leq N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}}$	-
③	**Concrete cone failure**	$N_{Ed} \leq N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}}$	-	$N_{Ed}^{g} \leq N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}}$
④	**Splitting failure**	$N_{Ed} \leq N_{Rd,sp} = \frac{N_{Rk,sp}}{\gamma_{Msp}}$	-	$N_{Ed}^{g} \leq N_{Rd,sp} = \frac{N_{Rk,sp}}{\gamma_{Msp}}$

5.7.2 Failure modes for fasteners loaded in tension

5.7.2.1 Steel failure

Steel failure capacity is determined as defined in section 5.6.3.1.

5.7.2.2 Pull-out failure

Pull-out failure capacity for post-installed fasteners is given by the relevant European Technical Specification and cannot be determined by an empirical equation as in the case of cast-in-place headed anchors, otherwise as defined in section 5.6.3.2. (See section 3.3 regarding this issue.)

5.7.2.3 Concrete cone failure

Concrete cone failure capacity is determined as defined in section 5.6.3.3.

5.7.2.4 Concrete splitting failure

According to the *CEN/TS 1992-4-4, section 6.2.1.5.1* splitting failure during the installation of fasteners due to torqueing can be avoided by complying with minimum values for internal anchor spacing (s_{min}), edge distances (c_{min}) and concrete member thickness (h_{min}).

Concrete splitting failure capacity is determined as defined in section 5.6.3.4.

5.7.3 Required verifications for fasteners loaded in shear

For post-installed expansion anchors equations ① through ④ in Table 5-4 are to be verified in case of shear loaded fasteners.

Table 5-4. Required verifications for fasteners loaded in shear. (CEN/TS 1992-4-4, Table 4)

		Single fastener	Fastener groups	
			most loaded fastener	*fastener group*
①	**Steel failure of fastener without 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 with lever arm**	$V_{Ed} \leq V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{Ms}}$	$V_{Ed}^{h} \leq V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{Ms}}$	-
③	**Concrete edge failure**	$V_{Ed} \leq V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{Mc}}$	-	$V_{Ed}^{g} \leq V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{Mc}}$
④	**Concrete pry-out failure**	$V_{Ed} \leq V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{Mc}}$	-	$V_{Ed}^{g} \leq V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{Mc}}$

5.7.4 Failure modes for fasteners loaded in shear

5.7.4.1 Steel failure without lever arm

Steel failure capacity is determined as defined in section 5.6.5.1.

5.7.4.2 Steel failure with lever arm

Steel failure capacity for anchors with lever arms is determined as defined in section 5.6.5.2.

5.7.4.3 Concrete pry-out failure

Concrete pry-out failure capacity is determined as defined in section 5.6.5.3.

5.7.4.4 Concrete edge failure

Concrete edge failure capacity is determined as defined in section 5.6.5.4.

5.8 Interaction between tension- and shear loads

Section 5.6 and section 5.7 have processed the design methods of tension- and shear loads respectively. When an anchor plate is loaded with simultaneous tension- and shear load several interaction conditions must be verified to ensure a safe design. The following conditions are valid for both cast-in-place anchors and post-installed anchors. (See *CEN/TS 1992-4-2, Section 6.4* and *CEN/TS 1992-4-4, Section 6.2.3*)

If steel failure is the decisive failure mode for both tension- and shear load the interaction condition of safe design is met by the following equation:

$$\beta_N^2 + \beta_V^2 \leq 1 \qquad \text{(Eq. 32)}$$

For any other combination of failure modes the interaction condition of safe design is met by the satisfaction of at least one of the following two equations:

$$\beta_N + \beta_V \leq 1{,}2 \qquad \text{(Eq. 33)}$$

$$\beta_N^{1,5} + \beta_V^{1,5} \leq 1 \qquad \text{(Eq. 34)}$$

where:

$$\beta_N = \frac{N_{Ed}}{N_{Rd}} \leq 1 \qquad \text{(Eq. 35)}$$

$$\beta_V = \frac{V_{Ed}}{V_{Rd}} \leq 1 \qquad \text{(Eq. 36)}$$

An interaction diagram is presented in Figure 5.20 below where the boundaries for the different condition equations are presented. For interaction values within these boundaries the criteria of safe design is met.

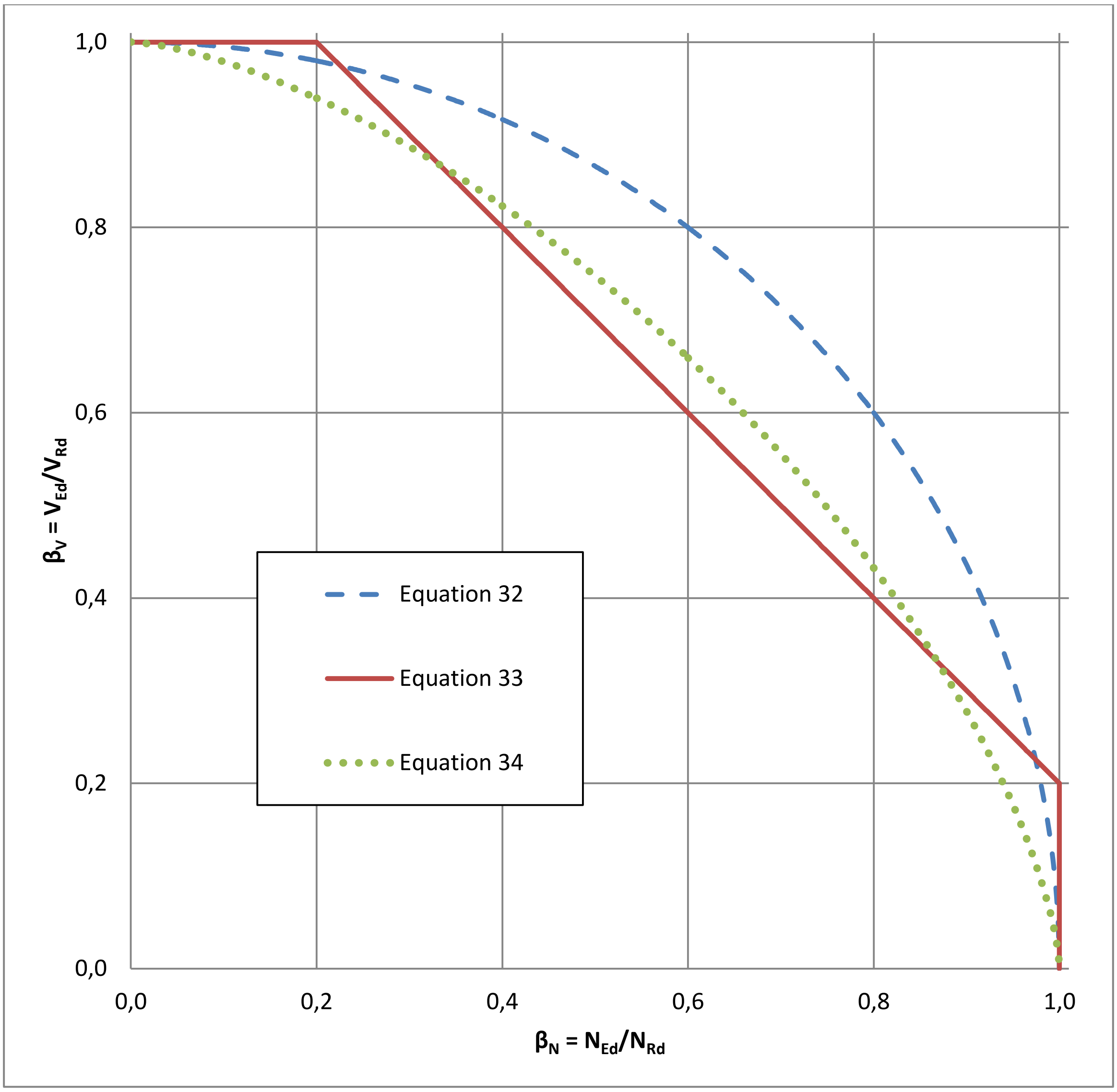


Figure 5.20. Interaction diagram for combined tension- and shear loads.

6 Parametric study of FE-models

In order to establish an optimal model to be used in the final automated analysis process a parametric study was set up containing different models and external loads. Whenever creating models you ought to simulate reality as good as possible. There are a great number of ways to model even the simplest kind of structure using the finite element method which is why different modelling techniques was tried out in different cases. The purpose of the study was to come up with an optimal model considering computational time without expense of realistic results. An overview of the study showing the main prerequisites of each case as well as useful results is presented in Appendix *1 - Parametric study - Tables*.

6.1 Approach

When first starting the parametric study a mind-map was created to get perspective on the different parameters of interest. Figure 6.1 shows a first draft of possible problem areas in need for consideration.

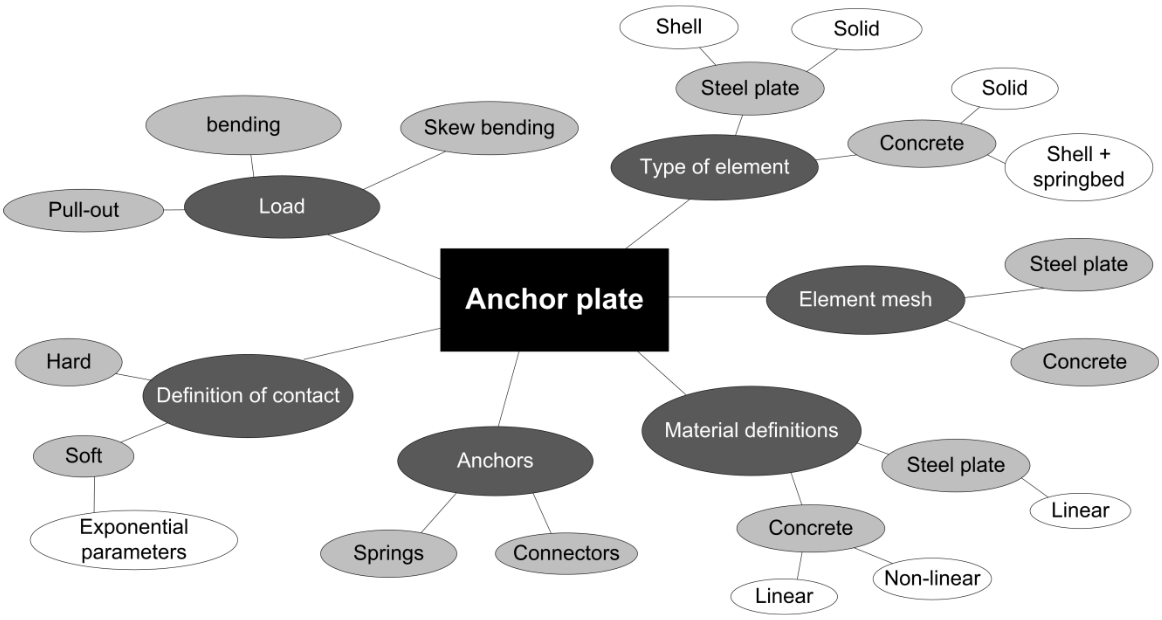


Figure 6.1. Mind map of anchor plate modelling

Obviously one cannot try every combination possible of the stated parameters in the extent of a master thesis. However the methodology used in this study is to change one variable at a time to appreciate the impact on the results for that variable.

Since one specific model can generate accurate results for a specific load case, for example tensional normal force, but rather unrealistic results for another load case like a bending moment the study is divided into three main groups where each group is loaded with a unique external force. This needs to be done in order to compare the different models with each other as well as control the load effects for different load cases. Each group consists of 22 different cases which means a total of 66 cases has been investigated.

The first sets of models (1.A-U) are loaded with a tensional normal force with the magnitude of 100 kN. The second set (2.A-U) are loaded with a single bending moment of 100 kNm around the x-axis and the third set (3.A-U) are loaded with two separate bending moments of 100 kNm around the x-axis and the y-axis respectively. This means that the number preceding the model indicates the type of loading, e.g. case 1.A, case 2.A and case 3.A are all the same model only loaded with different external loads.

For each and one of the three main groups a reference case is created, which serves as the most realistic model. The remaining cases (A-U) in each main group is compared to the corresponding reference case considering total time of analysis as well as the load effects in the construction. Figure 6.2 illustrates the anticipated convergence for models considering computational time of analysis and achieving realistic results. These parameters are to be investigated further in the parametric study

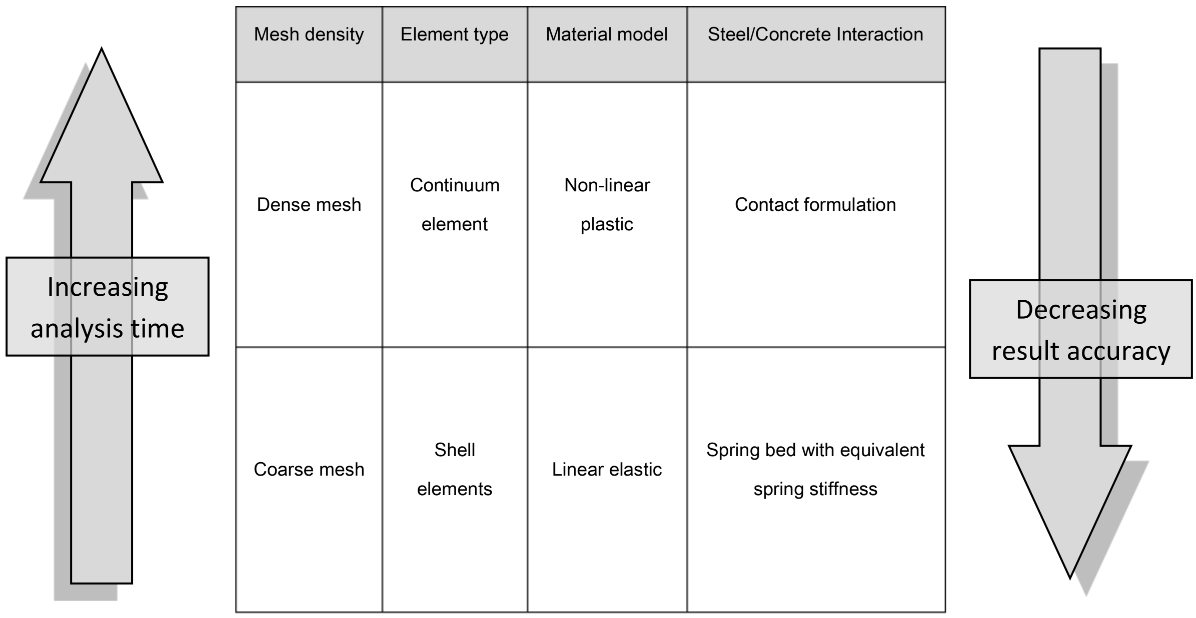


Figure 6.2. Computational time vs. result accuracy.

Since the stress distribution in the construction will differ depending on factors such as linear/non-linear material behaviour and type of elements the results presented in Appendix 1 - *Parametric study - Tables* refers to the maximum Mises stress in the steel plate and the largest contact pressure between the steel plate and the surrounding concrete, no matter occurring position.

6.2 Basis of design

In order to compare different models the geometry of the construction is the same in all cases of the study. It is not randomly chosen but do rather represent a typically dimensioned anchor plate. The width and height of the homogenous steel plate is set to 0.3 m and the thickness to 0.02 m. The surrounding concrete has been modelled with a depth of 0.1 m and in cases where the concrete has been modelled as a separate part the width and height is set to 0.5 m. The chosen geometry has four fasteners placed 0.05 m from each corner side of the steel plate. See Figure 6.3 for dimensioned illustration.

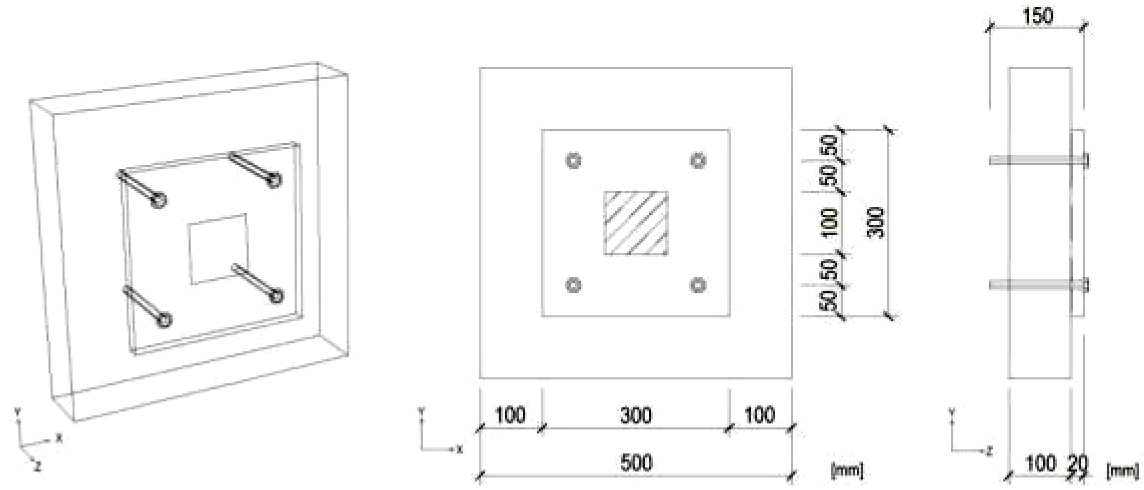


Figure 6.3. Geometric specification of model used in parametric study. (The coordinate axes show the directions, not the position of the local coordinate system.)

The four fasteners in the model are all of the same size which in reality is common. Especially when the construction is double symmetrical as in this study. The length of the anchors is set to 0.15 m and the diameter to 0.012 m. As seen in Figure 6.3 above there is a marked surface in the middle of the steel plate. This is where the attachment rigidly is welded to the steel plate. This beam constraint is FE-modelled by prescribing all degrees of freedom along the edges of the rectangle. The profile of the attachment in reality seldom has the simplicity of a quadratic cross section. It is nevertheless a simplification in this case study, (as well as in the automated calculation process). All loads will act in the reference point positioned on the steel plate surface in the centre of the attachment.

A cartesian coordinate system (CSYS) is positioned between the concrete surface and the steel plate in the direction of the global CSYS. The loads are specified according to this global orientation, thus compressive normal force is defined as positive in the z-direction (parallel with the anchor plate's normal).

6.3 Element types

The BRIGADE/Plus software provides an extensive element library, enabling generation of FE-models with large variations. The ones tried out in the parametric study are continuum-, shell-, spring- and connector- elements. There features are more elaborately described in the forthcoming subchapters.

In order to allow complete generality in material behaviour the Abaqus solver use numerical integration for all elements. The requested output data is calculated in the integration points in each element. Based on the shape functions these values are portioned to the nearby node(s). For both the continuum and shell elements used in the study reduced integration is applied. This means calculations are done in fewer points for each element. The advantage is that strains and stresses are calculated at points that give optimal accuracy. These locations are called Barlow points.

Another advantage is the decreased computational time and storage requirements which speed up the analysis. However by only using one integration point for each element as done in this study, deformation modes may appear that cause no strain in the Barlow point. These are called spurious zero-energy modes and are the reason for a phenomenon called “hourglassing” which means the element deforms but no strains are registered, see Figure 6.4, (Abaqus Theory Manual, chapter 3.1.1).

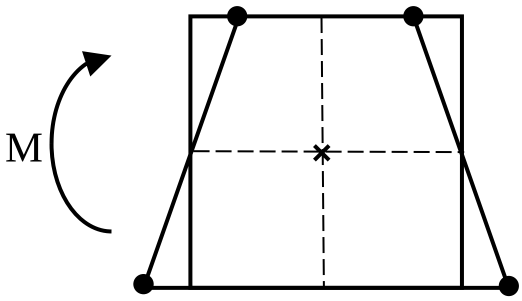


Figure 6.4. Illustration of a spurious zero-energy mode leading to hourglassing.

The dash-dotted line in Figure 6.4 illustrates there is no change in lengths, i.e. that no strains are detected for the only integration point in the middle of the element. The hourglassing phenomenon propagates easily in first-order elements with reduced integration points and causes unreliable results. By modelling the thickness of the structures with at least four elements hourglassing can be avoided. Figure 6.5 show that the integration points detect the strains. In this case the upper two elements capture compressive strains whilst the two below capture tensile strains.

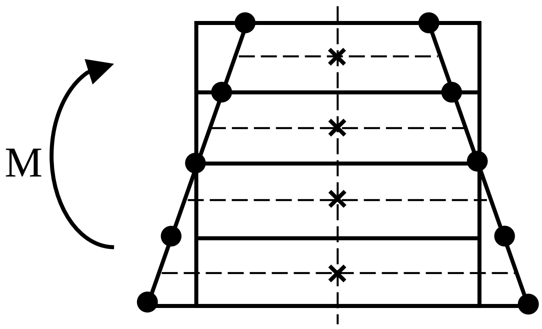


Figure 6.5. Four elements with reduced integration points resulting in a deformed body with strain recognition.

BRIGADE/Plus has built-in hourglass control which limits the problems caused by hourglassing. It verifies that the quota of artificial energy used to control hourglassing and the internal energy is less than one percent (<1%), (Abaqus Theory Manual, chapter 3.1.1).

6.3.1 Continuum elements

The used element in the case study is called C3D8R which is a stress/displacement, eight-node linear brick element with reduced integration points and hourglass control. When this element is fully integrated there are eight integration points whilst this reduced type only has one, which is located in the middle of the element thus small elements are required to capture stress concentrations. For cases where the steel plate is modelled with these solid elements there are five elements through the thickness to prevent hourglassing. The modelled concrete is thick enough (0.1 m) to fit several elements hence there will be no problem. Figure 6.6 shows the elements eight nodes and the centred integration point. Each node has three translation- and three rotation degrees of freedom, (Abaqus Analysis User's Manual, Volume IV 24.6.5).

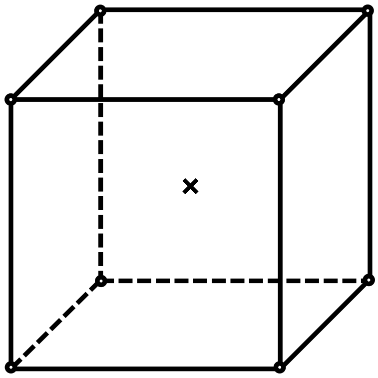


Figure 6.6. Eight-node continuum element with reduced integration points.

6.3.2 Shell elements

Shell elements are used when modelling structures with a significant smaller thickness than the other dimensions. The used element in the case study is called S4R which is a quadrilateral shell element with four nodes and reduced integration points. Instead of four integration points there is only one in the middle of each element, see Figure 6.7.

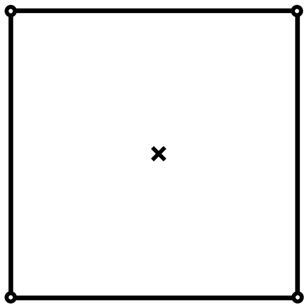


Figure 6.7. Four-node shell element with reduced integration points.

The thickness and number of integration points through the thickness is defined as a homogenous section property which is assigned to the element. The cross-sectional behaviour is calculated using Simpson's rule. The default value is five integration points but in this study nine has been used since some cases are modelled with non-linear material. One can chose to place the reference surface where the nodes will be situated on the bottom, in the middle or on top of the specified element thickness. All shell elements in the study are using middle surface definition which means the thickness will propagate in both negative and positive z-direction. Each node has three displacement and three rotational degrees of freedom, (Abaqus Analysis User's Manual, Volume IV 24.6.5).

6.3.3 Spring elements

Spring elements are often used for modelling physical springs and idealized axial or torsional components. They can also be used as structural damping or eliminate rigid body motion in different analyses. In the finite element analysis program used one can define three kinds of spring elements. The first two are both acting in a fixed direction but differentiates to one another by connecting two nodes in the model as to one node and ground. The third option is a spring connecting two nodes that has its line of action between the nodes whereas the line can rotate in large-displacement analyses, (Abaqus Analysis User's Manual, Volume IV Elements 27.1).

In this parametric study the spring elements used are connected to ground, see Figure 6.8. They are modelled linearly hence only given stiffness. When modelled non-linearly their behaviour is defined by nodal displacements and spring forces.

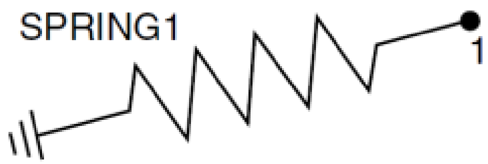


Figure 6.8. Spring element connected to ground, (Abaqus Analysis User's Manual Volume IV, 27.1.2).

6.3.4 Connector elements

There are two types of connectors available in BRIGADE/Plus, CONN2D2 and CONN3D2, which are used in two-dimensional and three-dimensional analysis respectively. Both of them can be defined between two nodes or one node and a ground node. Since all models in this study are three-dimensional the connector element used is CONN3D2.

When the connector element is connected to ground, (i.e. to a fixed point in space) this ground node has the initial position of the other node. As the other node moves during the analysis, displacements and relative positions are calculated. A connector element operating in three dimensions have up to three translation degrees of freedom and three rotational degrees of freedom in each node. The connectors are defined as non-linear by given force and displacement relations, (Abaqus Analysis User's Manual Volume IV, 26.1.2).

6.4 Material modelling

The anchor plate and the fasteners are made of steel and the surrounding structure is made of concrete. All material values in the case study are characteristic and taken from the appropriate Eurocode.

Since the analysis is performed to capture the stress distribution in the steel plate and the forces in respective fastener, not to simulate the failure of the anchor plate system, modelling of reinforcement in the concrete is redundant. Moreover, the content of reinforcement will not affect the contact pressure of the concrete member allowing validation of the concrete material by comparing maximum stresses with compressive material strength.

6.4.1 Steel plate

The steel plate has been modelled with continuum- and shell- elements with a linear elastic material definition in all cases. Since steel is an isotropic material the defined properties operate in all directions. The properties specified for the analysis are density (ρ), modulus of elasticity (E_k), and Poisson's ratio (υ) which is set to 7800 kg/m^3, 210 GPa, and 0.3 respectively. The stress – strain relation in the finite element analysis process for the steel plate is shown in Figure 6.9. The inclination of the graph is determined by the modulus of elasticity.

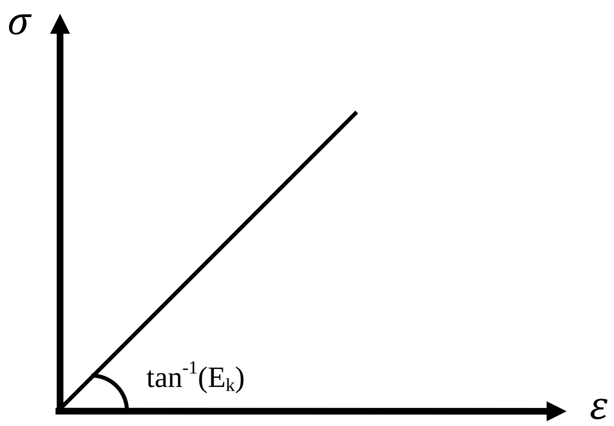


Figure 6.9. Stress - strain relation for the steel plate

Because of this material modelling approach without an upper steel yield limit the stresses may be higher than physically possible. A material failure control will therefore be necessary when studying the capacities.

6.4.2 Anchors

The anchors have been modelled with spring- or connector elements. These two have a lot in common such as both can be modelled linearly and non-linearly, they can connect between two nodes or define a connection between a node and ground. In all cases whether the fasteners are spring- or connector- modelled they are connected to ground. Read more about the element types in section 6.3 - *Element types*. For all cases in the study the anchors have specifically been given properties in the normal direction of the anchor plate. The shear forces on the anchors are in the final program automatically calculated by methods described in CEN/TS 1992-4 and not from the FE-analysis, which is why the modelled fastener behaviour considering the x- and y-directions only serves the purpose to eliminate rigid body motion. Hence the behaviour in the x-, y-plane is linearly defined and set to a high stiffness, i.e. $k = 10^{12}$ N/m for all cases.

Whether the fasteners are modelled with springs or connectors they have been given properties that captivate the specific anchor behaviour. Since the anchors only should carry tensional forces it has to be modelled non-linearly in the z-direction. The equivalent spring stiffness is calculated as for a two dimensional bar accordingly:

$$k = \frac{E \cdot A}{L}\ [N/m] \qquad \text{(Eq. 37)}$$

With the modulus of elasticity E = 210 GPa and dimensions according to section 6.2 - *Basis of design* the anchor stiffness for all models in the study is:

$$k = \frac{E \cdot A}{L} = \frac{210 \cdot 10^9 \cdot \pi \cdot 0{,}006^2}{0.15} = 158\ 336\ 269.7\ [N/m] \qquad \text{(Eq. 38)}$$

The behaviour of a spring can be described by the following equation:

$$F = k \cdot u\ [N] \qquad \text{(Eq. 39)}$$

Where F is the spring force and u is the nodal displacement. On basis of this equation the meaning of operation for the anchors is specified, see Table 6-1. Since the behaviour ought to be linear when tensioned without an upper yield limit it is only the inclination of the graph that is of importance i.e. the stiffness. By setting the displacement to one meter and knowing the stiffness, the anchor force gets the value 158 336 269.7 N.

Table 6-1. Definition of anchor behaviour in the normal direction of the anchor plate.

	F [N]	u [m]
1	0	-1
2	0	0
3	158 336 269.7	1

As the fasteners should not take any compressive forces rows 1 and 2 in Table 6-1 defines the incurvature in the origin of Figure 6.10.

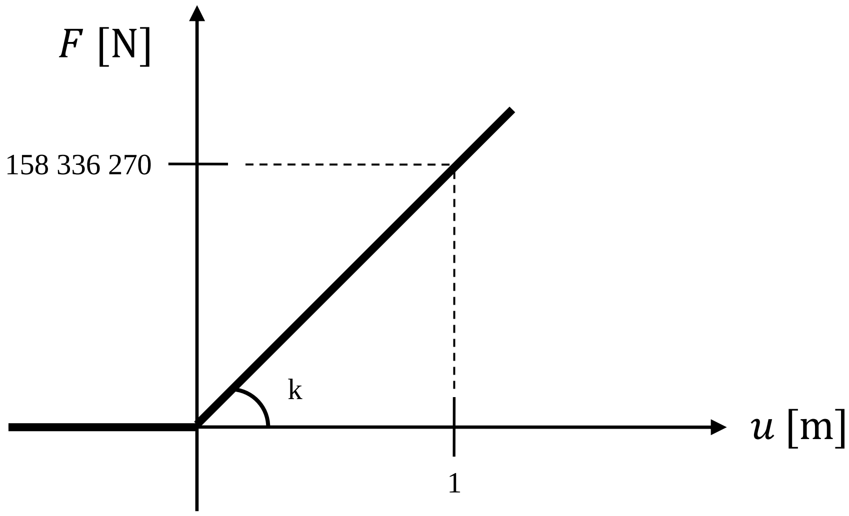


Figure 6.10. Schematic illustration of force – displacement relation for anchors.

6.4.3 Concrete

The chosen concrete strength class in the parametric study is C20/25 with strength values according to Eurocode 1992-1-1 3.1.3. Concrete is an isotropic but yet complex material with high compressive strength in comparison to the tensional strength. Since no tension forces can be transferred in the chosen contact interaction, the material definition of the concrete material is only defined for its compressive behaviour.

Shell-, solid-, connector-, and a combination of shell and spring/connector elements have been used to model the concrete. In some cases the material is linear elastically defined with density, modulus of elasticity, and Poisson's ratio as the only input. The values are then set to ρ = 2500 kg/m^3, E = 30 GPa, and υ = 0.2. As for the non-linear modelled concrete its stress-strain relation is defined as a simplified bilinear curve where the modulus of elasticity defines the inclination of the graph and the characteristic compressive strength its maximum stress value, see Figure 6.11. For concrete class C20/25 the value for f_{ck} is 20 MPa. The described non-linear material behaviour means that the strain is 0.67 ‰ when the stress has reached characteristic compressive strength.

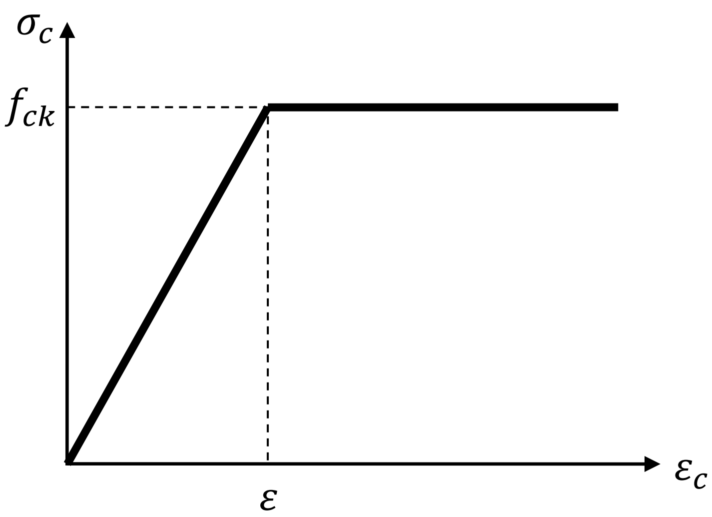


Figure 6.11. Bi-linear stress-strain relation for compressed concrete in parametric study.

Consider a negative bending moment around the x-axis where the effective pressure component F_c is situated at the far end of the steel plate as seen in Figure 6.12. With linear elastic defined concrete the stress will distribute triangularly and the component $F_{c,\,el}$ will be located one third of the distance d from the steel plate edge. With non-linear defined concrete the stress distribution will instead look like the dash-dotted line in Figure 6.12 and the pressure component $F_{c,\,pl}$ will move further in. This means the lever arm may be longer in order to establish moment equilibrium when the concrete is modelled with linear material behaviour and consequently the tension force in the fastener(s) smaller.

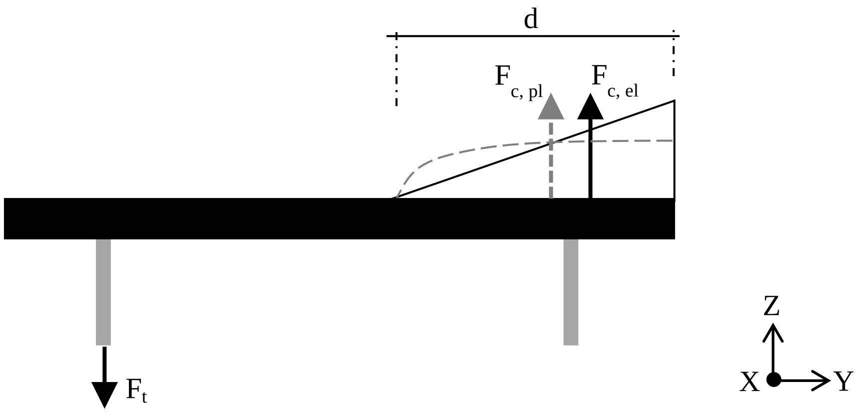


Figure 6.12. Elastic vs. plastic material definition of the concrete member.

For Case U described in section 6.6.5 the concrete material is modelled as a one-dimensional spring bed where connectors with simple spring definitions constitute the concrete member. Every node in the steel plate has a connector element attached with the purpose of transferring compressive normal forces. In order to accurately define the stiffness of each concrete connector element, the area of influence for each node is required.

<u>*An example:*</u>

If the element mesh is quadratic with the side length of 0.01 meter, there will be three different possible areas of influence for the connectors constituting the concrete spring bed.

Elements with an area of influence indexed as 1 in Figure 6.13 represent connectors in the middle region of the steel plate, i.e. 0.010^2 m^2. Areas of influence indexed as 2, which is half the size of area 1, is valid for elements positioned along the boundary of the steel plate except for the utmost elements which are identified as Area 3. For this example the third influence area will be 0.005^2 m^2.

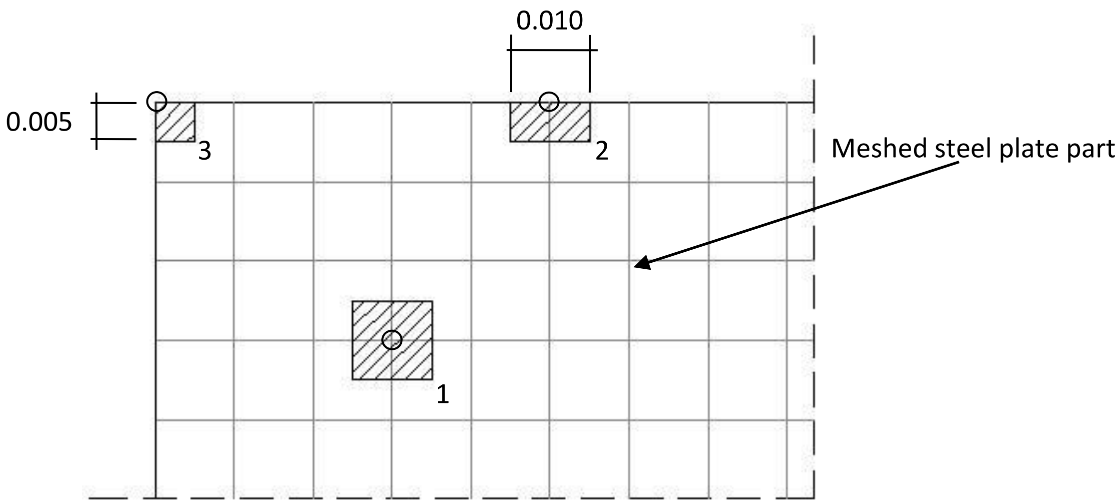


Figure 6.13. Different areas of influence for connector elements used for concrete modelling.

The sum of all areas of influence will cover the area of the steel plate and thereby represent the concrete surface that the anchor plate is fastened upon. The stiffness for every individual connector is assigned as described in section 6.4.2 – *Anchors* based on the influence area, the modulus of elasticity for the concrete, and thickness of the concrete member.

6.5 Interaction – contact modelling

For almost all cases contact pressure between the steel plate and concrete elements will develop. The formulation used for modelling interaction between the two deformable bodies is called finite sliding. When contact between two surfaces is initiated they may deform and rotate relative to each other. In order to accomplish convergence the surfaces need to be stabilised. For all of these models it is quite obvious where there will be contact which makes it easier to model. By defining the backside of the steel plate and the front of the concrete part as surfaces that eventually will interact in the analyses, the FE-program automatically generates the appropriate contact elements. The two surfaces are called “slave” and “master”. When deciding which surface should be assigned as master and which should be assigned as slave the following guidelines should be considered.

The master surface should:

- Have the coarser mesh (if the parts are meshed differently)
- Be larger than the slave surface
- Have stiffer structural response

For most cases the steel plate and the concrete part are modelled with the same element size. As for case E and F the concrete has coarser mesh (see tables in section 6.6). The concrete is always adjacent the steel plate which means the concrete surface is the bigger surface for all cases. Based on this the conclusion is to assign the master surface to the concrete and slave surface to the steel plate.

There will be problem if nodes on the slave surface penetrate the master surface since they will be trapped. In order to prevent this from happen the slave nodes are adjusted to its right positions. Figure 6.14 shows two different approaches for doing this. Illustration A) adjusts the slave nodes in set to accomplish smooth interaction whilst B) adjusts only to remove overclosures. Method A is used in the parametric study.

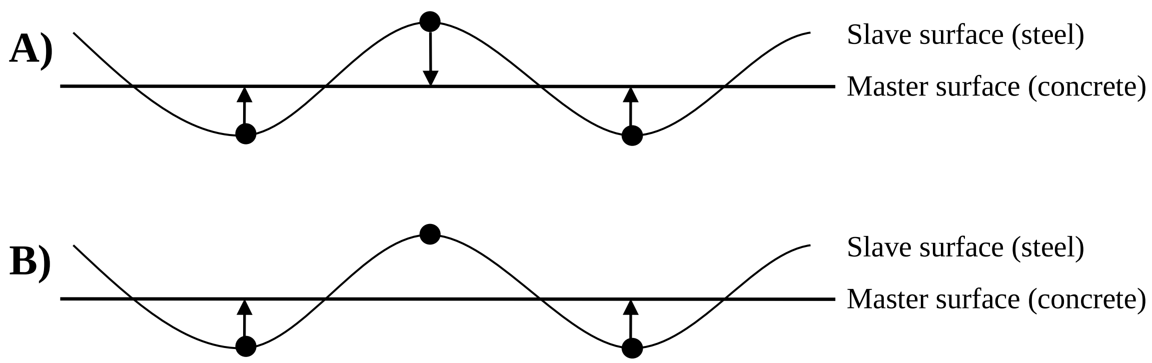


Figure 6.14. Schematic illustration of node adjustments.
A) Adjust slave nodes in set.
B) Adjust only to remove overclosure.

6.5.1 Contact interaction property

It is possible to help the analysis converge by defining the contact properties as "exponential" instead of "hard". Hard contact, which is the default setting, means no forces will be transmitted until actual contact between the two surfaces, see Figure 6.15.

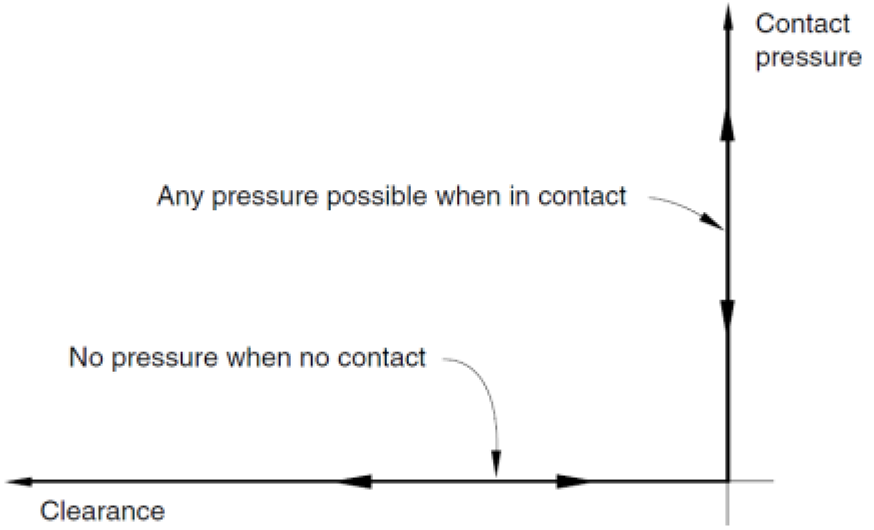


Figure 6.15. Default pressure - overclosure relationship, (Abaqus Analysis User's Manual Volume V, 31.1.2).

With exponential contact properties one can soften the impact by transmitting a fraction of the load before actual contact between the parts has occurred. This is illustrated in Figure 6.16.

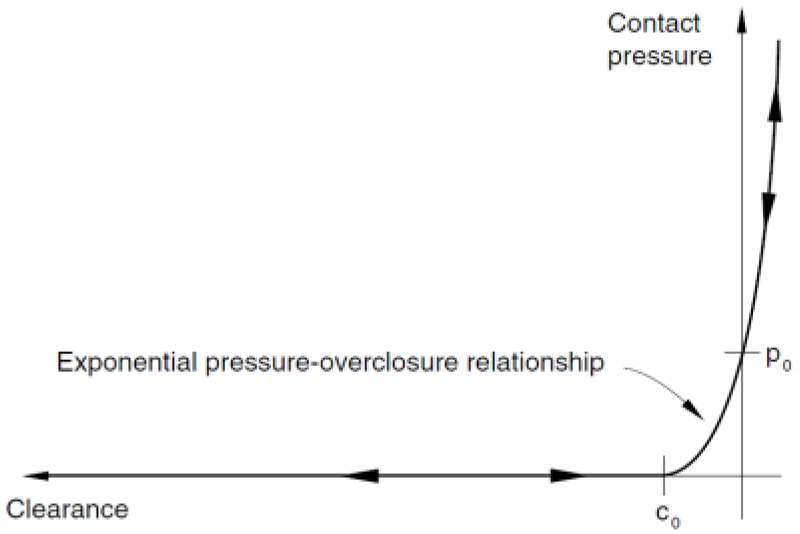


Figure 6.16. Exponential pressure - overclosure relationship, (Abaqus Analysis User's Manual Volume V, 31.1.2).

In the study two different exponential contacts has been defined. As seen in Appendix 1 - *Parametric study - Tables* they are called Hard/Exponential and Soft/Exponential and are defined as shown in Table 6-2.

Table 6-2. Definition of exponential interaction in the parametric study.

Hard/Exponential		Soft/Exponential	
Pressure	Clearance	Pressure	Clearance
$30*10^{6}$	0	$10*10^{6}$	0
0	$1*10^{-5}$	0	$1*10^{-4}$

For all cases in the parametric study the tangential behaviour i.e. friction forces between the steel plate and the concrete are neglected. This simplification ought to be conservative since friction will reduce the shear forces acting on fasteners. Moreover the CEN/TS 1992-4 publication prohibits the consideration of friction forces.

6.6 Building the cases

As earlier mentioned the case study consists of 22 different models excited with three different loads. An overview of all 66 cases is shown below in Table 6-3, Table 6-4 and Table 6-5.

Table 6-3. Parametric properties for cases in load model 1.

	Element type		Element meshing size		Material definition		Anchor plate fasteners	Contact definition	Load definition
	Steel Plate	Concrete	Steel Plate	Concrete	Steel Plate	Concrete			
Ref. Case 1	Solid	Solid	0,010 m	0,010 m	Linear	Non-Linear	Connectors	Hard	Normal force
Case 1.A	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Connectors	Hard/Exponential	Normal force
Case 1.B	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Connectors	Soft/Exponential	Normal force
Case 1.C	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Springs	Hard/Exponential	Normal force
Case 1.D	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Springs	Soft/Exponential	Normal force
Case 1.E	Shell	Solid	0,010 m	0,025 m	Linear	Linear	Connectors	Hard/Exponential	Normal force
Case 1.F	Shell	Solid	0,010 m	0,025 m	Linear	Non-linear	Connectors	Hard/Exponential	Normal force
Case 1.G	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Connectors	Hard/Exponential	Normal force
Case 1.H	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Connectors	Soft/Exponential	Normal force
Case 1.I	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Connectors	Hard/Exponential	Normal force
Case 1.J	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Connectors	Soft/Exponential	Normal force
Case 1.K	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Springs	Hard/Exponential	Normal force
Case 1.L	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Springs	Soft/Exponential	Normal force
Case 1.M	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Springs	Hard/Exponential	Normal force
Case 1.N	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Springs	Soft/Exponential	Normal force
Case 1.O	Solid	Solid	0,025 m	0,025 m	Linear	Linear	Connectors	Soft/Exponential	Normal force
Case 1.P	Solid	Solid	0,025 m	0,025 m	Linear	Non-linear	Connectors	Soft/Exponential	Normal force
Case 1.Q	Solid	Solid	0,025 m	0,025 m	Linear	Linear	Springs	Soft/Exponential	Normal force
Case 1.R	Solid	Solid	0,025 m	0,025 m	Linear	Non-linear	Springs	Soft/Exponential	Normal force
Case 1.S	Solid	Solid	0,010 m	0,010 m	Linear	Linear	Connectors	Soft/Exponential	Normal force
Case 1.T	Solid	Solid	0,010 m	0,010 m	Linear	Non-linear	Connectors	Soft/Exponential	Normal force
Case 1.U	Shell	-	0,010 m	-	Linear	Non-linear spring bed	Connectors	-	Normal force

Table 6-4. Parametric properties for cases in load model 2.

	Element type		Element meshing size		Material definition		Anchor plate fasteners	Contact definition	Load definition
	Steel Plate	Concrete	Steel Plate	Concrete	Steel Plate	Concrete			
Ref. Case 2	Solid	Solid	0,010 m	0,010 m	Linear	Non-Linear	Connectors	Hard	Bending M_x
Case 2.A	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Connectors	Hard/Exponential	Bending M_x
Case 2.B	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Connectors	Soft/Exponential	Bending M_x
Case 2.C	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Springs	Hard/Exponential	Bending M_x
Case 2.D	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Springs	Soft/Exponential	Bending M_x
Case 2.E	Shell	Solid	0,010 m	0,025 m	Linear	Linear	Connectors	Hard/Exponential	Bending M_x
Case 2.F	Shell	Solid	0,010 m	0,025 m	Linear	Non-linear	Connectors	Hard/Exponential	Bending M_x
Case 2.G	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Connectors	Hard/Exponential	Bending M_x
Case 2.H	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Connectors	Soft/Exponential	Bending M_x
Case 2.I	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Connectors	Hard/Exponential	Bending M_x
Case 2.J	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Connectors	Soft/Exponential	Bending M_x
Case 2.K	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Springs	Hard/Exponential	Bending M_x
Case 2.L	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Springs	Soft/Exponential	Bending M_x
Case 2.M	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Springs	Hard/Exponential	Bending M_x
Case 2.N	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Springs	Soft/Exponential	Bending M_x
Case 2.O	Solid	Solid	0,025 m	0,025 m	Linear	Linear	Connectors	Soft/Exponential	Bending M_x
Case 2.P	Solid	Solid	0,025 m	0,025 m	Linear	Non-linear	Connectors	Soft/Exponential	Bending M_x
Case 2.Q	Solid	Solid	0,025 m	0,025 m	Linear	Linear	Springs	Soft/Exponential	Bending M_x
Case 2.R	Solid	Solid	0,025 m	0,025 m	Linear	Non-linear	Springs	Soft/Exponential	Bending M_x
Case 2.S	Solid	Solid	0,010 m	0,010 m	Linear	Linear	Connectors	Soft/Exponential	Bending M_x
Case 2.T	Solid	Solid	0,010 m	0,010 m	Linear	Non-linear	Connectors	Soft/Exponential	Bending M_x
Case 2.U	Shell	-	0,010 m	-	Linear	Non-linear spring bed	Connectors	-	Bending M_x

Table 6-5. Parametric properties for cases in load model 3.

	Element type		Element meshing size		Material definition		Anchor plate fasteners	Contact definition	Load definition
	Steel Plate	Concrete	Steel Plate	Concrete	Steel Plate	Concrete			
Ref. Case 3	Solid	Solid	0,010 m	0,010 m	Linear	Non-Linear	Connectors	Hard	Bending M_x, M_y
Case 3.A	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Connectors	Hard/Exponential	Bending M_x, M_y
Case 3.B	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Connectors	Soft/Exponential	Bending M_x, M_y
Case 3.C	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Springs	Hard/Exponential	Bending M_x, M_y
Case 3.D	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Springs	Soft/Exponential	Bending M_x, M_y
Case 3.E	Shell	Solid	0,010 m	0,025 m	Linear	Linear	Connectors	Hard/Exponential	Bending M_x, M_y
Case 3.F	Shell	Solid	0,010 m	0,025 m	Linear	Non-linear	Connectors	Hard/Exponential	Bending M_x, M_y
Case 3.G	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Connectors	Hard/Exponential	Bending M_x, M_y
Case 3.H	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Connectors	Soft/Exponential	Bending M_x, M_y
Case 3.I	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Connectors	Hard/Exponential	Bending M_x, M_y
Case 3.J	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Connectors	Soft/Exponential	Bending M_x, M_y
Case 3.K	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Springs	Hard/Exponential	Bending M_x, M_y
Case 3.L	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Springs	Soft/Exponential	Bending M_x, M_y
Case 3.M	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Springs	Hard/Exponential	Bending M_x, M_y
Case 3.N	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Springs	Soft/Exponential	Bending M_x, M_y
Case 3.O	Solid	Solid	0,025 m	0,025 m	Linear	Linear	Connectors	Soft/Exponential	Bending M_x, M_y
Case 3.P	Solid	Solid	0,025 m	0,025 m	Linear	Non-linear	Connectors	Soft/Exponential	Bending M_x, M_y
Case 3.Q	Solid	Solid	0,025 m	0,025 m	Linear	Linear	Springs	Soft/Exponential	Bending M_x, M_y
Case 3.R	Solid	Solid	0,025 m	0,025 m	Linear	Non-linear	Springs	Soft/Exponential	Bending M_x, M_y
Case 3.S	Solid	Solid	0,010 m	0,010 m	Linear	Linear	Connectors	Soft/Exponential	Bending M_x, M_y
Case 3.T	Solid	Solid	0,010 m	0,010 m	Linear	Non-linear	Connectors	Soft/Exponential	Bending M_x, M_y
Case 3.U	Shell	-	0,010 m	-	Linear	Non-linear spring bed	Connectors	-	Bending M_x, M_y

6.6.1 The reference case

This case serves the purpose of capturing the anticipated physical behaviour of the anchor plate construction looked upon and thereby work as a key for the rest of the cases considering stress distributions and its magnitude. Both parts (steel plate and nearby concrete) are modelled with continuum elements and the fasteners with connectors. The concrete is modelled non-linearly as explained in section 6.4.3 - *Concrete*. Figure 6.17 shows the reference case model and its 0.010 seeded mesh.

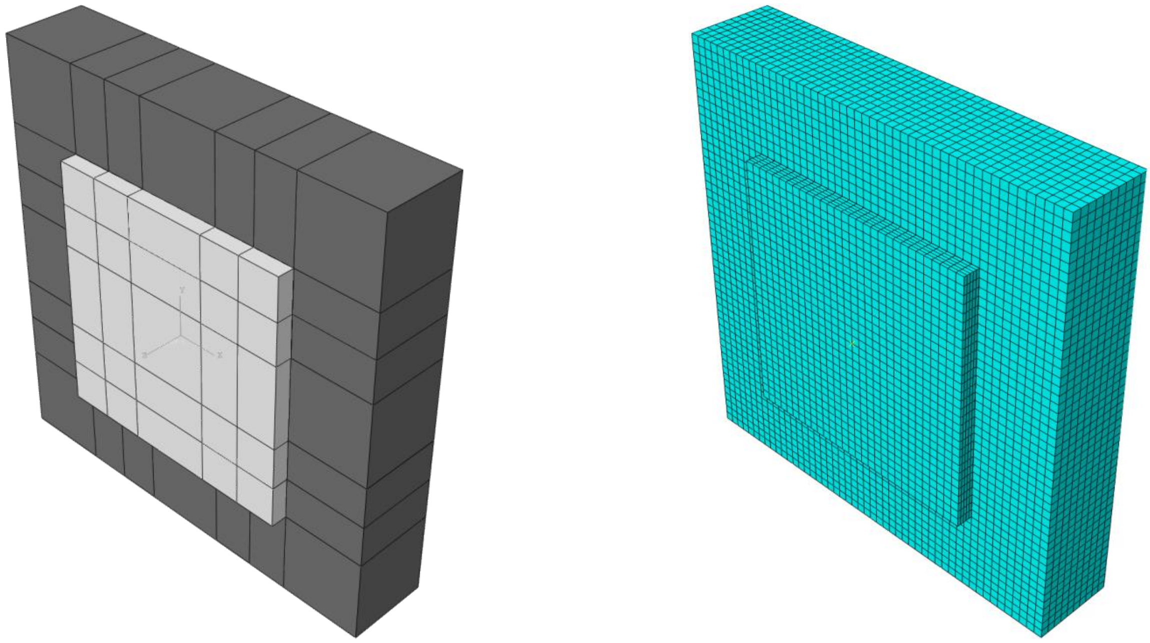


Figure 6.17. Geometric model (left) and finite element model (right) of the reference case.

For all cases where the concrete is modelled with solid elements, including the reference case, there is a pinned boundary condition on the back side of the concrete part. This means the translation degrees of freedom are fixed but it is free to rotate around all axes. The boundary condition is needed in order to prevent rigid body motion.

6.6.2 Case A – D

These cases have in common that the steel plate as well as the nearby concrete is modelled with shell elements. However the properties for the concrete shell elements are set to represent a dummy material with negligible stiffness, i.e. the modulus of elasticity is 1 GPa. A linear concrete behaviour is instead defined for the spring bed attached to this dummy material, see section 6.4.3. The spring bed is of type “node to ground” which means no boundary conditions are necessary. The reason for this modelling technique was to limit the degrees of freedom and consequently the analysis time but hopefully still get accurate results. Figure 6.18 shows a schematic illustration of these models in profile.

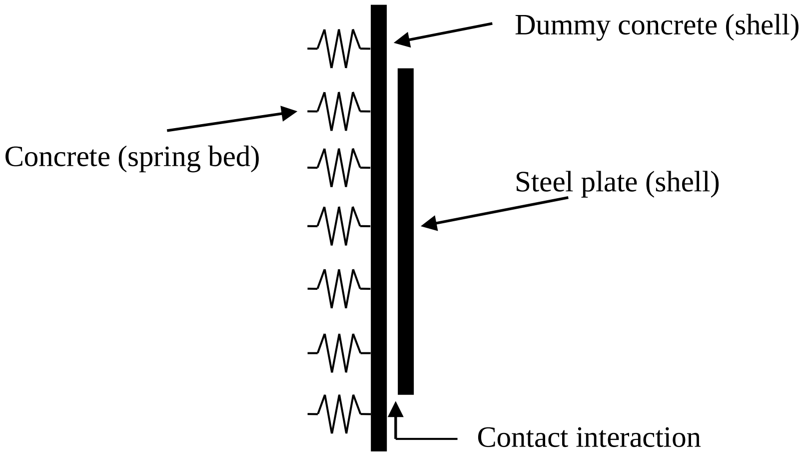


Figure 6.18. Schematic section illustration of cases A – D.

Since all shells are modelled as middle surfaces the defined part thickness will propagate in both negative and positive z-direction. This is why there is a gap between the parts for all models that have shell elements. Figure 6.19 shows the model as well as the chosen element mesh for case A–D.

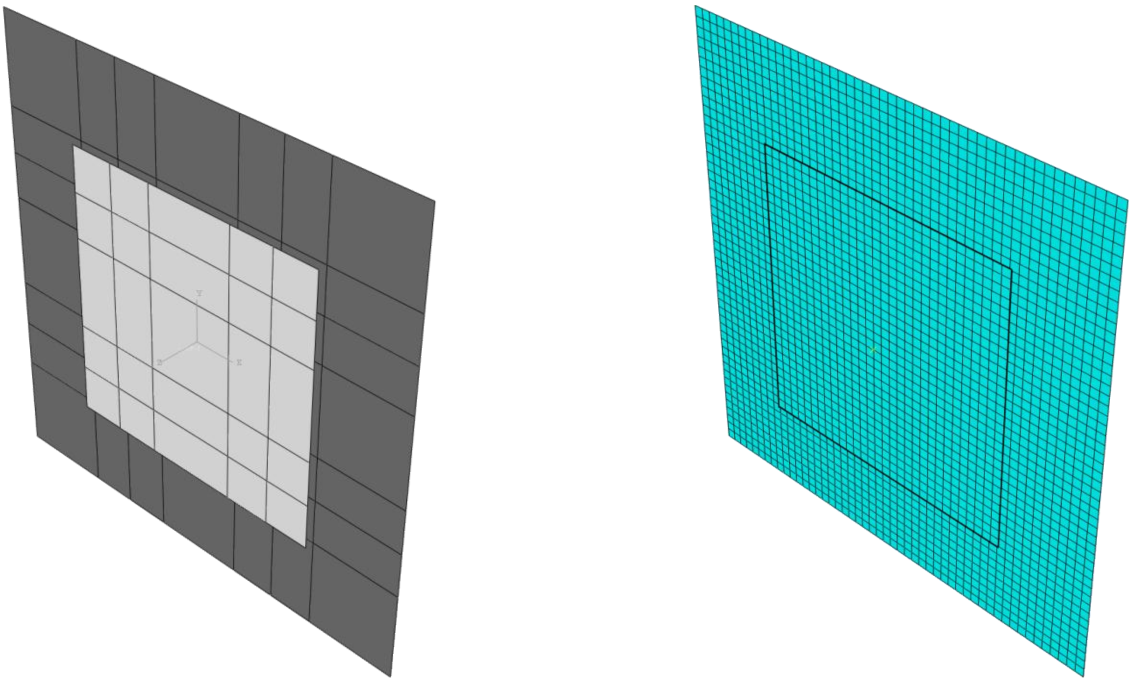


Figure 6.19. Shell modelled steel plate and concrete surface. The right frame shows the mesh generation.

Another parameter of interest that differentiate model A from B as well as C from D is how the contact is defined between the parts. They are both exponentially defined but one of them is harder than the other one, see section 6.5.1 - *Contact interaction properties*.

6.6.3 Case E – N

The concrete is modelled with solid elements and the steel plate with shells in these cases. The premier parameter of interest in this group is the impact of modelling the concrete non-linear instead of linear but also to compare the contact definitions explained in section 6.5.1. The concrete in case E and F is modelled with a coarser mesh as seen to the left in Figure 6.20 whilst the remaining cases of this group are meshed as seen in the right frame. Moreover these models also have a pinned boundary condition on the backside of the concrete.

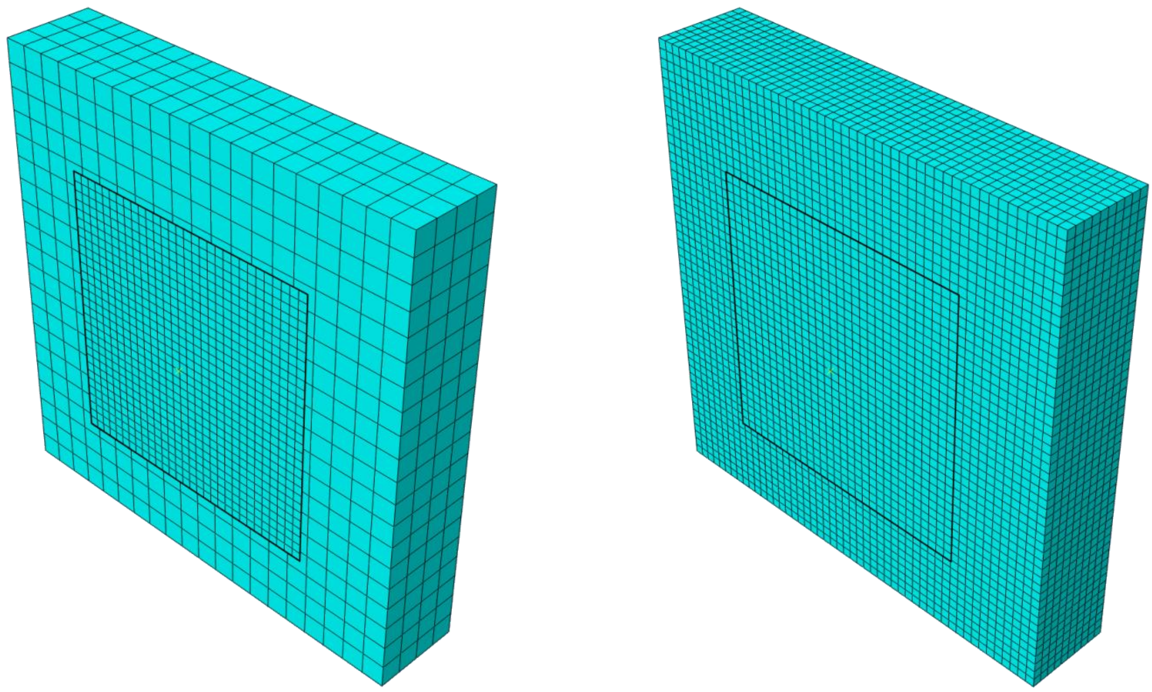


Figure 6.20. The different mesh sizes tried out for the group of cases E to N.

6.6.4 Case O – T

The parts in these cases are modelled with solid elements and are thus much like the reference case. However the coarser mesh with side length 0.025 is used for many of these cases to see the impact of meshing sizes.

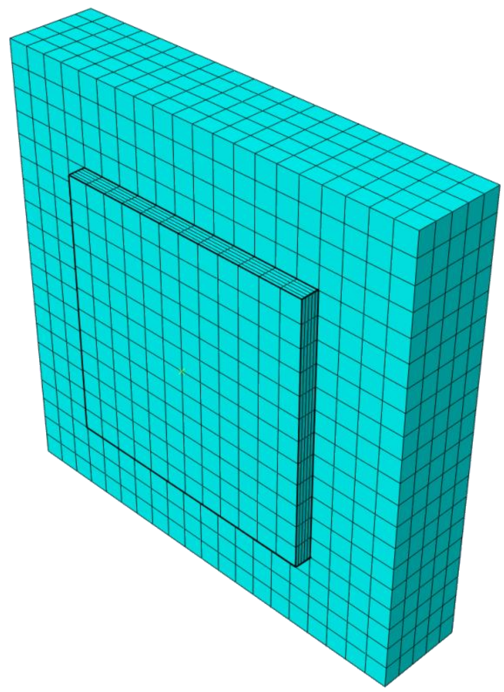


Figure 6.21. Model with coarse mesh.

6.6.5 Case U

Instead of modelling the concrete as a structural part, the concrete is instead modelled with non-linear connectors attached to the nodes on the backside of the steel plate part. With this modelling technique no contact will need to be defined. The left illustration in Figure 6.22 schematically shows the model section and the right illustration show the finite element model.

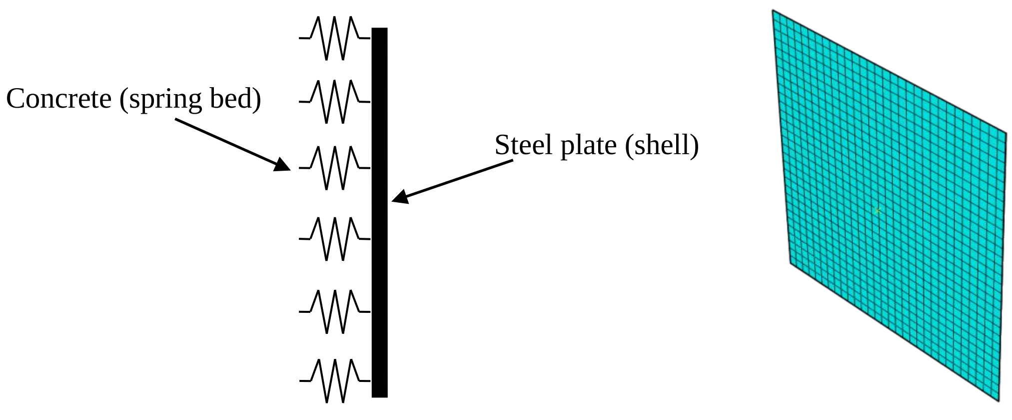


Figure 6.22. To the left the model is seen in profile and to the right a perspective view of the meshed steel plate.

As for case A to D the spring bed in case U is defined with node-to-ground elements, hence the model is not in need of boundary conditions. Both the fasteners and the concrete are modelled with connectors.

6.6.6 Results

The interpreted results from all analysis of the parametric study are presented in Appendix 1 - *Parametric study* - *Tables*. The effects looked upon for comparisons between cases are:

- Pull-out forces in the four fasteners
- Largest Mises stress in steel plate
- Largest contact pressure between steel plate and concrete

Some of the deformation plots showing stress distributions are presented for handpicked models loaded with two bending moments in Appendix 2 - *Parametric study* - *Figures*. The legend in each figure shows the magnitude in Pascal [Pa]. Another important aspect is of course the analysis time for each case which also is stated in Appendix 1 - *Parametric study* - *Tables*.

6.7 Evaluation of parametric study

As mentioned in the beginning of this chapter the purpose of the parametric study is to establish a proper model for the screening process that quickly gives accurate results. Although stresses and forces can be compared directly between the cases the analysis time is an uncertainty. All analysis has been run on the same computer but some have been going simultaneously with other processes running in the background which may have slowed down the computer. For instance, reference case 2 converged faster than case 2T which is a less advanced model. Anyhow the registered times gives a satisfactory estimation.

All models except model type U consists of two different parts, a steel plate- and a concrete part. Because of the absence of another part model type U has the advantage to exclude the somewhat problematic and computational time expensive definition of contact. This is primarily reflected by comparing analysis time between case U and case A – D in main groups two and three (bending and skew bending). These models are quite similar apart from the modelled concrete surface which brings contact formulation. The gain in time compared to the reference case for case A - D is about 200 whilst for case U it is closer to 300. Model O and Q are almost as fast as case U but they are modelled with a coarser mesh which means there are substantially fewer degrees of freedom, hence fewer calculations to perform. Clearly case U has by far the fastest analysis time.

The forces in the fasteners are for all cases pretty much levelled with the reference case results no matter if they are modelled with springs or connectors. A difference though is that the spring modelled fasteners seems to adopt pressure forces (negative values). The reason is they are not non-linearly defined as the connectors. Their behaviour is mirrored whilst the connectors cannot take pressure. Otherwise there is no significant difference between springs and connectors but we have chosen to use connectors to model the fasteners in the final solution.

Considering the largest Mises stress in the steel plate the models with coarse mesh, i.e. case O - R has about 50 % of the reference case magnitude. There are only two different mesh sizes tried out in the study (0.010 vs. 0.025) but by looking at the stress distribution on models with coarser mesh the conclusion is not to use bigger elements, see Appendix 2 - *Parametric study - Figures*. If the mesh were to be refined, that is use smaller elements than 0.010, there is a risk of getting points of singularity. This means one element circles a region where the stress will not spread and consequently be unrealistically high. However the element size parameter will be freely specified by the user in the final program with the default value 0.010.

As seen in Appendix 1 - *Parametric study - Tables* the Mises stresses in the steel plate is very accurate for case S and T compared to the reference cases in each main group. This is not surprising since the models are much alike. As for case T the only difference is how the contact is defined. The same applies for case S but the main difference is rather the linear modelled concrete which seems to reduce the computational time with about 80 %. Even though case S seems to give good results the analysis time is about 10 minutes. When analysing thousands of anchor plates it makes a significant difference if the analysis time is 15 seconds instead of 10 minutes. Based on this reason the interesting models can be narrowed down to case A, B, C, D and U who all delivers adequate results. The results of these cases are similar no matter type of external load but case U is the one converging fastest, despite the fact that the concrete is modelled non-linearly. Hence model type U will be used in the automated calculation process.

7 Parameterisation of important data

As previously mentioned it is of utmost importance that the input data which is to be utilised when generating the analysis model for calculation of load effects and for normative verification of load capacities are delivered in a predefined, structured, and consequent manner to ensure capability of automatisation.

Scanscot Technology AB has already developed a concept methodology for inventory of anchor plates which have been used as a foundation when shaping the data parameterisation method.

7.1 Essential categories for parameterisation

When breaking down the concept of designing anchor plates one can identify the following main categories for which the input data can be divided into:

1. Anchor plate => *Geometrical Specification*
2. Anchor plate => *Load Specification*
3. Anchor => *Anchor Technical Specification*
4. Material => *Material Specification*

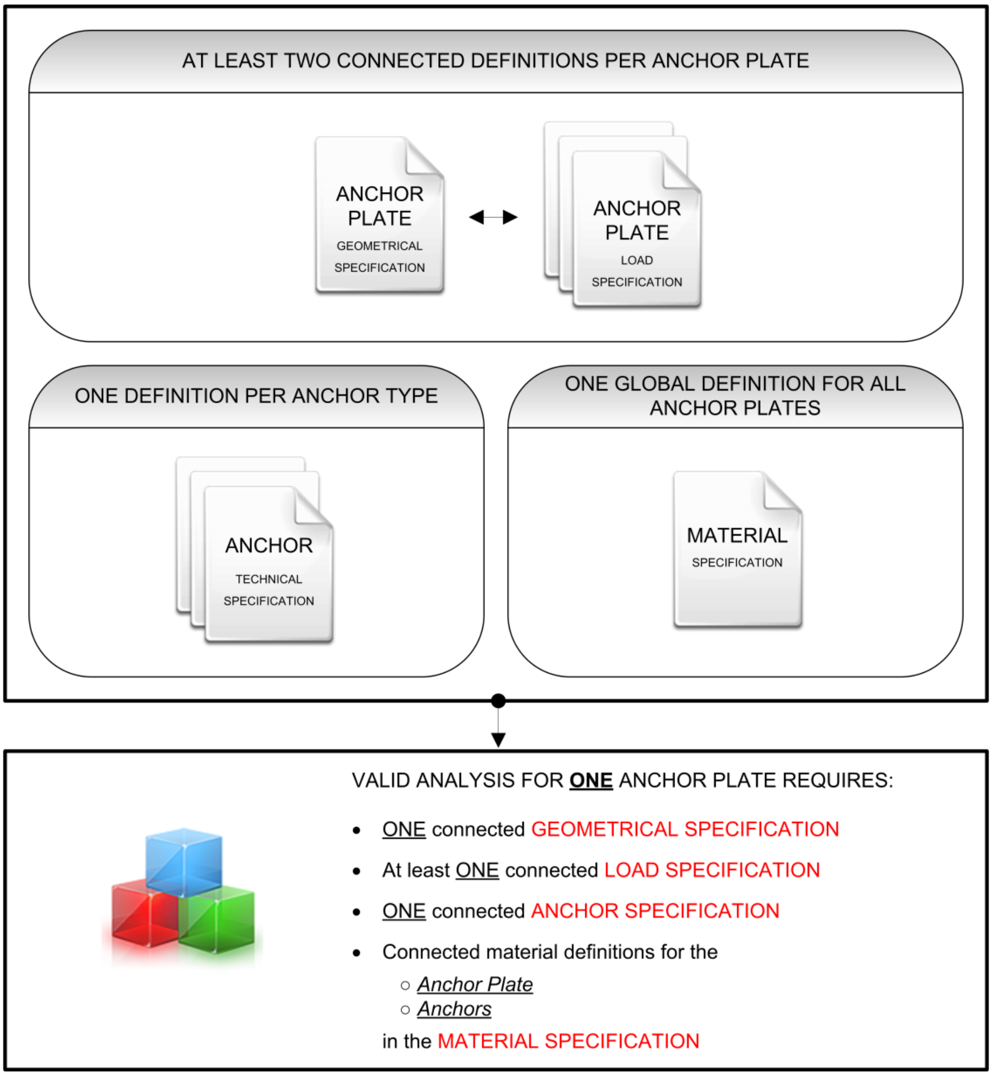


Figure 7.1. Schematic overview for parameterisation of required input data.

7.1.1 Geometrical Specification

Each *Geometrical Specification* document is a geometrical definition of **one** anchor plate containing important data regarding the geometrical properties of the anchor plate and the individual anchors. These documents are to be created during the inventory of anchor plates and must be crafted according to a certain pre-defined template, either by creating it manually following the given guidelines, or by using a pre-existing macro-enabled document as an aiding tool for quicker accessibility.

The pre-defined template is divided into two major parts, *Anchor Plate Description* and *Anchor Plate Type Configuration*, which are described in further detail in section 7.1.1.1 and section 7.1.1.2.

7.1.1.1 Anchor Plate Description

Only required data for the automatised calculation method is presented in summary within this chapter, additional information included in the methodology for anchor plate inventory is not presented due to protection of proprietary intellectual property of Scanscot Technology AB.

- A unique anchor plate ID.
- Orientation of the local anchor plate coordinate system.
- Distance between the anchor plate and the concrete surface.
- Distance to nearby free concrete edges.
- Distance to centre of nearby anchor plates.
- Number of attachments.
- Eccentricity of the attachment(s).
- Dimension of the attachment(s).
- Thickness of the concrete member to which the anchor plate is mounted.
- Concrete structure material.

The anchor plate ID is a key part of the inventory methodology. It is of utmost importance that the identification name (can be either digits or letters) is held unique to enable the ability to reference a specific anchor plate within the different parts of the anchor plate definition documents. The most obvious example is when connecting one or several load specification documents to a specific anchor plate.

The next required parameter is the orientation of the local anchor plate coordinate system. This parameter determines how the anchor plate relates to the global coordinate system defined for the inventory system in which the studied anchor plate belongs. An example of usage is stated in section 7.1.2. The input data shall be specified as the following directional matrix:

$$\begin{bmatrix} x_{anchor\ plate} \\ y_{anchor\ plate} \\ z_{anchor\ plate} \end{bmatrix} = \begin{bmatrix} a_1 & b_1 & c_1 \\ a_2 & b_2 & c_2 \\ a_3 & b_3 & c_3 \end{bmatrix} * \begin{bmatrix} X \\ Y \\ Z \end{bmatrix}$$

Where:

x, y, z = local -SYS

X, Y, Z = gl bal C-SYS

The surface distance parameter specifies the distance between the back side of the anchor plate and the casted concrete surface due to grouting, shims, or stand-off installation type anchors. (See Figure 7.2 for how the distance shall be measured.) Note that there is a difference between this parameter and the one given in section 7.1.1.2 where the specified distance in the *Anchor Plate Description* part is the actual distance and the distance in the *Anchor Plate Type Configuration* part is the theoretical distance according to the design documents.

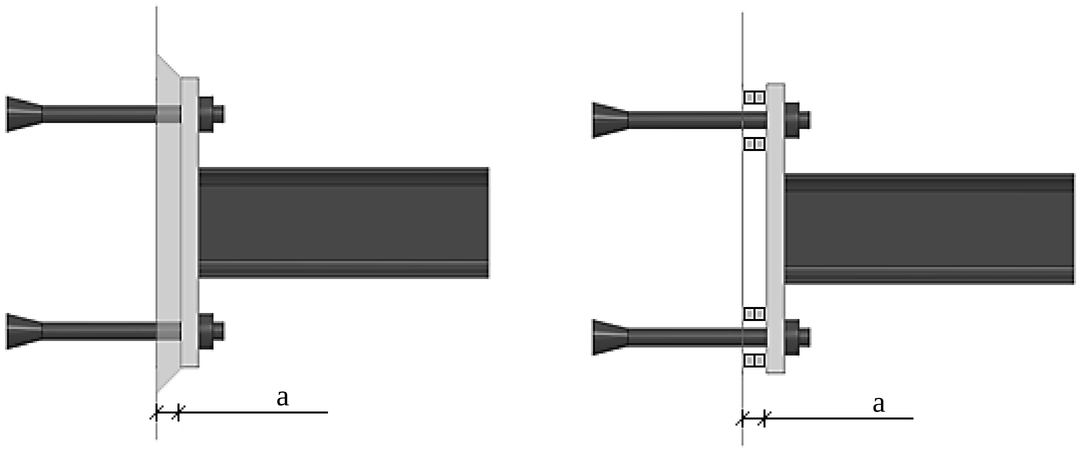


Figure 7.2. Definition of the spacing distance between the anchor plate and the concrete base material. The left image shows an installation with grouting and the right image an installation with shims.

Continuing to next parameter, one has to specify the distance to nearby free edges. The input format is given as a set of two coordinates, one offset coordinate in the x-direction, and one offset coordinate in the y-direction. Hence a maximum of two free edges can be specified for a single anchor plate. The offset coordinates must be given in the anchor plate local coordinate system. (See Figure 7.3 for an illustration of the distance to nearby edges definition.)

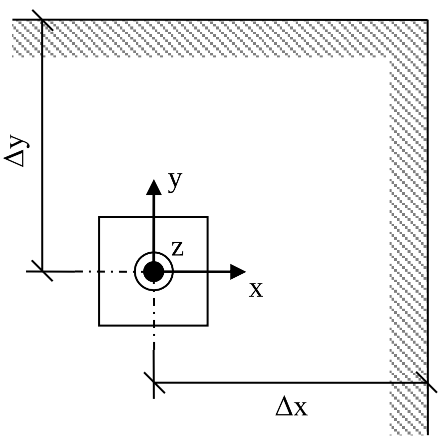


Figure 7.3. Definition of the edge-offset coordinates.

While the next input is as important as the distance to nearby free edges parameter, the distance to nearby anchor plates parameter is not a required input. Instead the definition of this information is implemented in the distance to nearby free edges parameter. The benefit of this method is that geometrical information for the adjacent anchor plates does not have to be specified. However, the disadvantage is that this limits the number of free edges and nearby anchor plates to a maximum of two since the parameter is defined with only one set of offset coordinates in the x-direction and y-direction.

Calculations of distance to nearby anchor plates for all possible scenarios are presented below. The smallest s_1 and s_2 values are decisive.

The calculation method of distance in respect to concrete cone area is done accordingly:

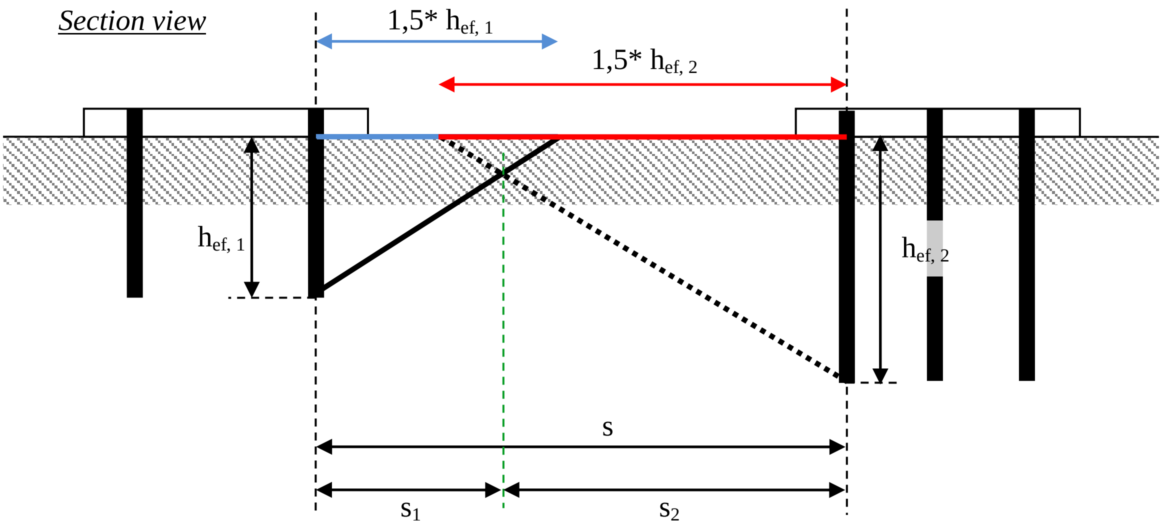


Figure 7.4. Conversion of distance to nearby anchor to distance to free edge in case of concrete cone failure.

The distance to a nearby anchor plate is handled as an artificial distance to a nearby edge which occur where the two anchor plate's concrete cone area intersect each other as in Figure 7.4. This is easily determined with trigonometrically calculations and is presented for both anchor plates in equation 40 and equation 41 respectively.

$$s_1 = 1.5 * h_{ef,1} - \frac{|1.5*h_{ef,1} - 1.5*h_{ef,2}|}{2} \quad \text{(Eq. 40)}$$

$$s_2 = 1.5 * h_{ef,2} - \frac{|1.5*h_{ef,1} - 1.5*h_{ef,2}|}{2} \quad \text{(Eq. 41)}$$

In a similar manner the calculation method in respect of concrete blow-out cones is done:

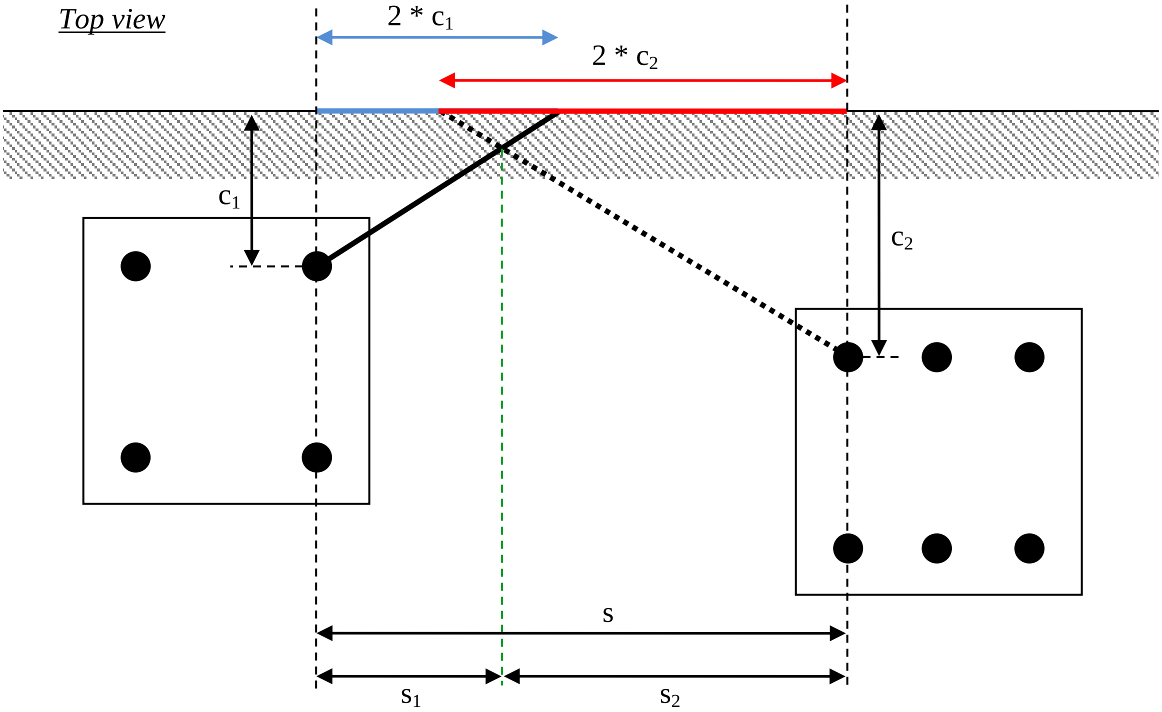


Figure 7.5. Conversion of <u>distance to nearby anchor</u> to <u>distance to free edge</u> in case of concrete blow-out failure.

$$s_1 = 2 * c_1 - \frac{|\,2*c_1 - 2*c_2\,|}{2} \qquad \text{(Eq. 42)}$$

$$s_2 = 2 * c_2 - \frac{|\,2*c_1 - 2*c_2\,|}{2} \qquad \text{(Eq. 43)}$$

And for concrete edge failure area:

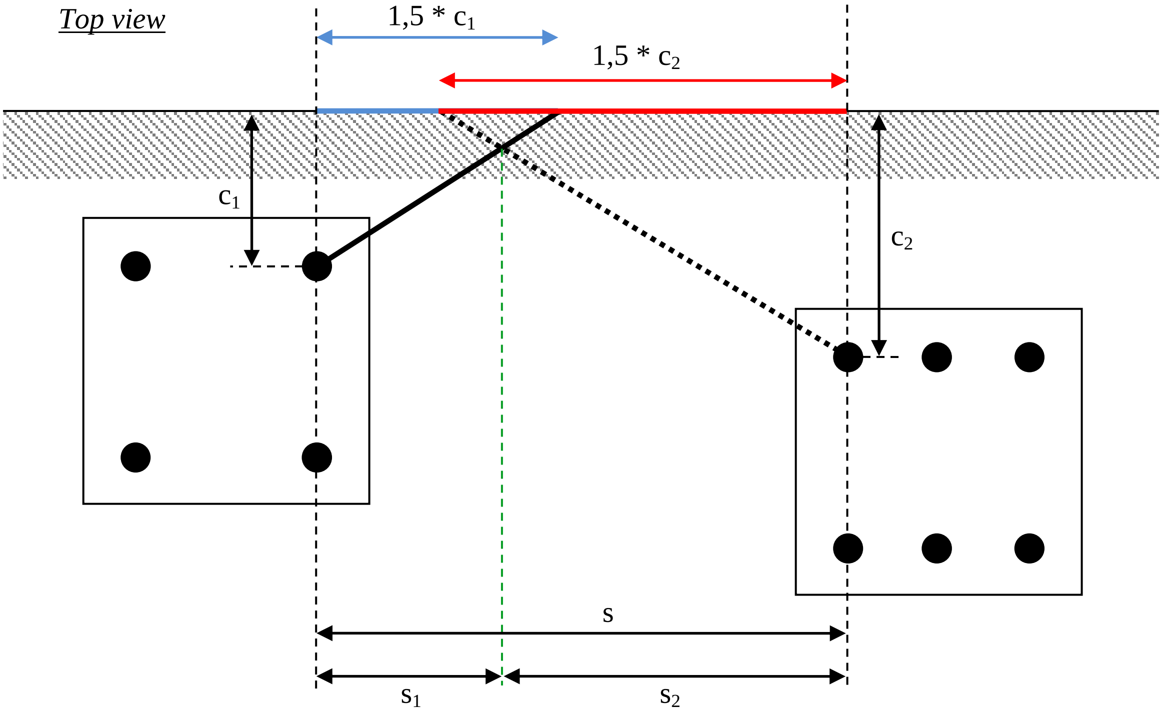


Figure 7.6. Conversion of distance to nearby anchor to distance to free edge in case of concrete edge failure.

$$s_1 = 1.5 * c_1 - \frac{|\,1.5{*}c_1 - 1.5{*}c_2\,|}{2} \qquad \text{(Eq. 44)}$$

$$s_2 = 1.5 * c_2 - \frac{|\,1.5{*}c_1 - 1.5{*}c_2\,|}{2} \qquad \text{(Eq. 45)}$$

Input parameters regarding the beam attachments is divided into three input requirements: number of attachments, eccentricity of the attachment(s) and the dimension of the attachment(s) whereas the last two parameters varies with dependency of the first parameter.

With *n* number of attachments it must follow:

- *n* coordinate sets (x-direction and y-direction in the local C-SYS) defining the concentration point of each attachment.
- *n* dimension sets defining the boundaries of each attachment (x-direction and y-direction in the local C-SYS).

Finally the last two input parameters define:

- The thickness of the concrete member in which the anchor plate is mounted.
- The concrete member material according to the standard Eurocode concrete classes (Must be specified as CXX/XX where the class may range from C12/15 to C90/105 according to the limitations of the CEN/TS norm, see section 5.4)

7.1.1.2 Anchor Plate Type Configuration

Only the required data for the automatised calculation method is presented in the summary below.

- Main geometry of the anchor plate.
- Distance between the anchor plate and the concrete surface.
- Anchor plate material.
- Type of anchor system.
- Manufacturer and product specification.
- Number of anchors.
- Positioning of each anchor.
- Effective anchor diameter and Embedment length.
- Anchor material.

First the geometrical properties of the anchor plate are specified. The input parameter accepts three values: the boundaries of the steel plate (in x-direction and y-direction in the local C-SYS) and the thickness of the steel plate.

The next parameter is defined as specified in section 7.1.1.1 regarding the distance to concrete structure surface. Required input is the theoretical distance from the backside of the anchor plate to the surface of the concrete member.

The anchor plate material parameter links to the global *Material Specification* document. The specified material name shall reference to a unique anchor plate material post containing the material properties for the current anchor plate definition. Several anchor plate definitions may share the same material definition. (See section 7.1.4 for more information.)

In the type of anchor system parameter the user shall specify either 'Cast-in headed anchors' or 'Post-installed anchors'.

This is then followed by a reference to the manufacturer and product specification which is an uniquely specified identification name given in one of the *Anchor Technical Specification* documents containing the behaviour of the specific anchor plate's anchors. It is recommended that the name is chosen so both manufacturer and anchor product type is easily perceived thus simplifying corrections of input information. (See section 7.1.3 for more information.)

Input parameters regarding the anchors is divided into two input requirements: number of anchors and eccentricity of the anchor(s) whereas the last parameter varies with dependency of the first parameter.

With *n* number of anchors it must follow:

- *n* coordinate sets (x-direction and y-direction in the local C-SYS) defining the concentration point of each anchor.

Information concerning the effective anchor diameter and anchor embedment length is also required.

- The effective anchor diameter is used to calculate the equivalent spring stiffness in the automatised analysis method described in chapter 8 and the concrete cone capacity calculation, and should be determined with this in mind when rendering the anchor geometry. *An essential question is whether the threads should be considered or not, if present?*
- The effective embedment length is defined as in Figure 7.7.

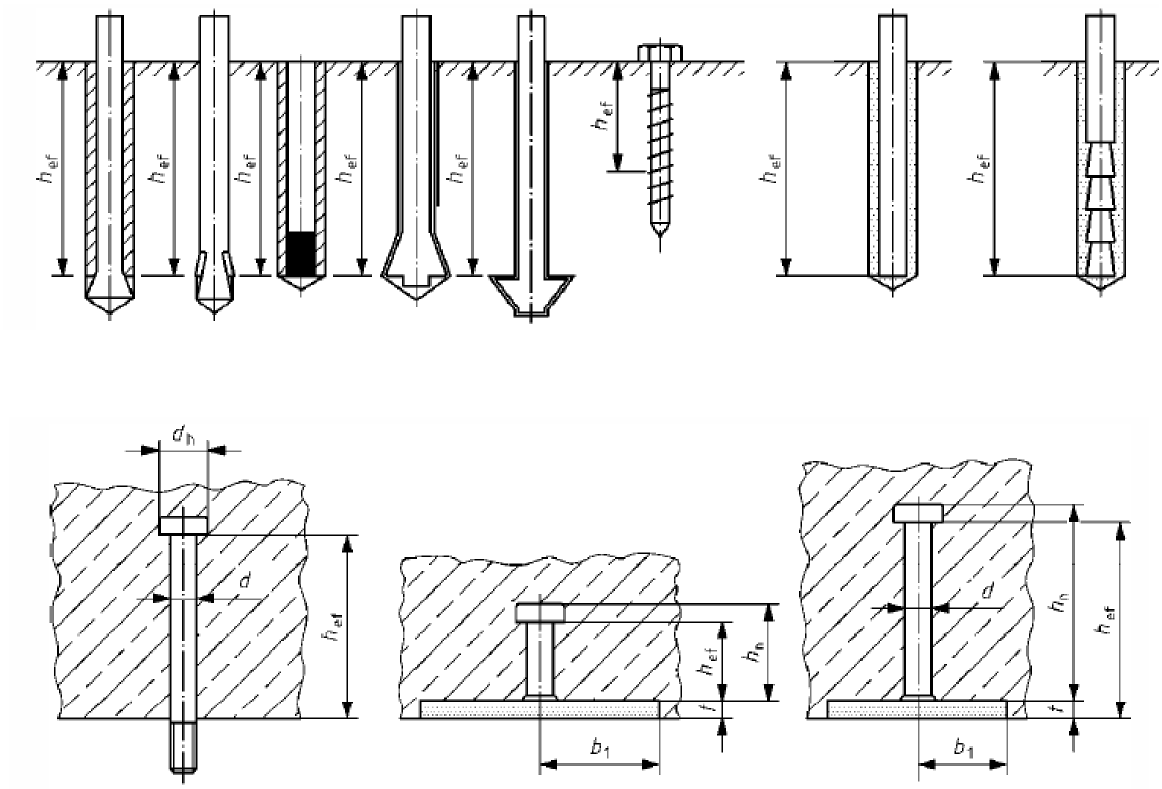


Figure 7.7. Illustration defining the effective embedment length for different types of anchorage. The centre figure in the bottom row define the embedment length for headed fasteners with a large anchor plate in any direction (b_1 > $0.5h_n$ or $t \geq 0.2h_n$) and the right figure for headed fasteners with a small anchor plate in each direction ($b_1 \leq 0.5h_n$ or $t < 0.2h_n$), (CEN/TS 1992-4-1, Figure 3 and Figure 5).

Finally the anchor material parameter links to the global *Material Specification* document. The specified material name shall reference to a unique anchor material post containing the material properties for the current anchor plate definition. Several anchor plate definitions may share the same material definition. (See section 7.1.4 for more information.)

7.1.2 Load Specification

Every *Load Specification* document must be connected to strictly one *Geometrical Specification* document. The main disposition of the *Load Specification* document is as following:

- Reference to the unique anchor plate ID.
- Reference to the unique attachment ID.
- Orientation of load definition coordinate system.
- Definition of permanent loads.
- Definition of variable loads.
- Definition of load combinations by adding permanent and variable loads.

As explained in section 7.1.1.1 the anchor plate ID is used to connect the load definitions to one specific anchor plate.

The attachment ID is a reference to which attachment support the defined load combinations should be applied to. This identification reference is an integer number ranging from 1 to *n* where *n* is total number of attachments. Within an anchor plate object (consisting of *Geometrical*-, *Load*-, *Anchor Technical*-, and *Material Specifications*) an attachment ID may only be defined once.

The next required parameter is the orientation of load definition coordinate system. This parameter determines how loads and moments in three dimensions are oriented in respect to the global coordinate system.

Input data shall be specified according to the following directional matrix:

$$\begin{bmatrix} x_{load} \\ y_{load} \\ z_{load} \end{bmatrix} = \begin{bmatrix} a_1 & b_1 & c_1 \\ a_2 & b_2 & c_2 \\ a_3 & b_3 & c_3 \end{bmatrix} * \begin{bmatrix} X \\ Y \\ Z \end{bmatrix}$$

Where:

x, y, z = loca C-SYS

X, Y, Z = global C-SYS

When orientations of both anchor plate and load definition are known it is possible to transform one coordinate system to the other enabling correct application of loads and moments in the geometrical space. The advantage of implementing such a system is that the anchor plate can be positioned entirely unconfined in wall elements, ceilings, and floors and yet still the loads can be applied.

To transform the load- and moment components to the local anchor plate coordinate system the following directional matrixes are used.

$$\begin{bmatrix} x_{anchor\ plate} \\ y_{anchor\ plate} \\ z_{anchor\ plate} \end{bmatrix} = \begin{bmatrix} & & \\ & A & \\ & & \end{bmatrix} * \begin{bmatrix} X \\ Y \\ Z \end{bmatrix}$$

$$\begin{bmatrix} x_{load} \\ y_{load} \\ z_{load} \end{bmatrix} = \begin{bmatrix} & & \\ & B & \\ & & \end{bmatrix} * \begin{bmatrix} X \\ Y \\ Z \end{bmatrix}$$

This will result in a transformation operation for load effect conversion with loads and moments according to the local orientation of the anchor plate:

$$\begin{bmatrix} E_{x,\ \ anchor\ plate\ def.} \\ E_{y,\ \ anchor\ plate\ def.} \\ E_{z,\ \ anchor\ plate\ def.} \end{bmatrix} = \begin{bmatrix} & & \\ & A & \\ & & \end{bmatrix} * \begin{bmatrix} & & \\ & B^{-1} & \\ & & \end{bmatrix} * \begin{bmatrix} E_{x,\ \ load\ def.} \\ E_{y,\ \ load\ def.} \\ E_{z,\ \ load\ def.} \end{bmatrix}$$

However, due to the limitations explained in section 1.4 regarding the scope of this report the load specification document is somewhat simplified. Input information regarding anchor plate ID and support ID is still required. The main difference lies in the definition of the load combinations.

Instead of defining permanent and variable loads for later load combination, this information is given as manually pre-load-combined effects already transformed to the anchor plate coordinate system. The simplified document will have the following outline:

- Reference to the unique anchor plate ID.
- Reference to the unique attachment ID.
- Definition of load combinations.

The definition of load combinations is composed of one or several load combination blocks. (See Figure 7.8 for an overview of the load combination block template.) These blocks consist of input parameters for:

- Load combination ID. (Which must be unique within the same document.)
- Name of the load combination. (Preferably a descriptive identity.)
- Load type.
- Crack width.
- Enveloped load effects.

The load types define the character of the applied load. Load type A and load type B are loads in the ultimate limit state, and load type C and load type D define loads that are accidental or extraordinary.

Specification of crack width is used to determine which state the concrete is in. The primary interest is whether the concrete is cracked or not (crack width ≥ 0).

Load Comb ID	Load name	Load type	w_k	Envelope DOF	Fx	Fy	Fz	Mx	My	Mz
		A	0.0	Max Fx						
		A	0.0	Min Fx						
		A	0.0	Max Fy						
		A	0.0	Min Fy						
		A	0.0	Max Fz						
		A	0.0	Min Fz						
		A	0.0	Max Mx						
		A	0.0	Min Mx						
		A	0.0	Max My						
		A	0.0	Min My						
		A	0.0	Max Mz						
		A	0.0	Min Mz						

Figure 7.8. Template for one load combination block utilised as input to the application.

When applying a load according to load history one acquires a response in the system that corresponds to the actual loading scenario. Load history gives the load effect components (normal force, shear Force in both direction, bending moments in both directions and torsion) for every time step in the history. The major drawback of such an approach is that this utilises a great deal of computing recourses since the same system has to be verified in an iterative process where every analysis takes substantial time, some more and some less depending on the modelling technique.

One possible alternative is to use extreme values for all load components due to a load combined load case, thus applying a worst case loading scenario. If the system withstands such loading it is regarded as safe. However, this method is extremely conservative and statistically improbable, and will yield unreasonable results where the capacity requirement may never be fulfilled.

Instead of using load history which would generate results that are completely accurate, nor extreme values which will generate unlikely results, enveloped load components are used to define the load combination blocks. An enveloped load case is a set of 6 load combinations response components where one component is of extreme character whereas the remaining components are the associated components that appear simultaneously as the extreme component.

To summarise the approach regarding application of loads, the calculated response from a load combination will result in a load history in time rendering all six response components. From this load history, the extreme values for each component are extracted with its belonging enveloped components of non-extreme nature. This will generate in 12 extreme load combination response components (maximum and minimum), all of which have 5 associated components.

These 72 response components will constitute 12 load cases which will represent the load combination when performing the analysis. This might not be as accurate as in the case of applying load history, but neither as conservative as when using extreme values for all 6 load components from the load combination response history.

In Figure 7.8 the envelope for one load combination block is presented, where maximum and minimum loads for each force and moment component in the three dimensional space is registered with each associated value. A valid load combination block consists of a 12*6 matrix. This method is valid by statistical definitions where the possibility of occurrence of several extreme effect components is deemed unlikely.

7.1.3 Anchor Technical Specification

As described in section 7.1.1, every *Geometrical Specification* must reference to an *Anchor Technical Specification* containing the geometrical-, material-, and characteristic resistance properties for the attached anchors.

The following parameters are specified in the *Anchor Technical Specification*:

- A unique technical specification ID.
- Minimum geometric properties, e.g. spacing and edge distances, concrete member thickness etcetera.
- Characteristic values of resistance to tension loads.
- Characteristic values of resistance to shear loads.

It is of utmost importance that the technical specification ID (can be either digits or letters, recommended that the name is chosen so both manufacturer and anchor product type is easily perceived) is held unique to enable the ability to reference a specific *Anchor Technical Specification* within the different anchor plate *Geometrical Specification* documents. If an identification name is defined more than once, the later instances will be expunged due to avoidance of over-determination.

Parameters for required geometric properties are given for the specific anchor type. The entire anchor plate system has to fulfil these requirements to be valid for normative verification.

The required parameters regarding geometric properties are:

- Minimum thickness of concrete member.
- Cracked concrete.
    - Minimum spacing distance between anchors.
    - Minimum edge distance.
- Non-cracked concrete.
    - Minimum spacing distance between anchors.
    - Minimum edge distance.

For cast-in-place headed anchors the minimum spacing and edge distance can be set to $10*d_{anchor}$ if not otherwise determined. However, for expansion anchors these values must be defined as specified in the approved technical specification.

Characteristic values of resistance to tension loads consider all required parameters used when verifying anchor plates against failure modes associated to tension. This includes:

- Steel failure
  - CAST-IN-PLACE and POST-INSTALLED:
    Characteristic tensile resistance ($N_{Rk,s}$) and belonging partial safety factor.

- Pull-out failure
  - CAST-IN-PLACE:
    Diameter of anchor head (d_h) and partial safety factors for load type A+B and load type C+D.
  - POST-INSTALLED:
    Characteristic resistance in cracked concrete ($N_{Rk,p}* \psi_c$) and partial safety factors for load type A+B and load type C+D.

- Concrete cone failure
  - CAST-IN-PLACE and POST-INSTALLED:
    Characteristic resistance coefficient (k) for cracked and non-cracked concrete, effective embedment depth (h_{ef}), and partial safety factors for load type A+B and load type C+D.

- Splitting failure
  - CAST-IN-PLACE and POST-INSTALLED:
    Characteristic resistance coefficient (k) for cracked and non-cracked concrete, effective embedment depth (h_{ef}), and partial safety factors for load type A+B and load type C+D.

Characteristic values of resistance to shear loads consider all required parameters used when verifying anchor plates against failure modes associated to shear. This includes:

- Steel failure without lever arm
  - CAST-IN-PLACE and POST-INSTALLED:
    Characteristic shear resistance ($V_{Rk,s}$) and belonging partial safety factor.

- Concrete pry-out failure
  - CAST-IN-PLACE and POST-INSTALLED:
    Characteristic resistance coefficient (k) and belonging partial safety factor.

- Concrete edge failure
  - CAST-IN-PLACE and POST-INSTALLED:
    Effective length of anchor in shear loads (l_f), external nominal diameter of anchor (d_{nom}), and partial safety factors for load type A+B and load type C+D.

7.1.4 Material Specification

The *Material Specification* document shall contain information regarding the material properties of the steel plate and the anchors. Concrete material properties are not included since they are implemented in the concrete material library directly in the analysis software. This can be done due to the limitation of the CEN/TS norm only allowing standard Eurocode concrete classes in the design method thus all vital parameters are already known.

7.1.4.1 Steel plate

Input parameters regarding the steel plate requires the following information:

- A unique material name.
- Density of the steel.
- Characteristic modulus of elasticity, E_k.
- Poisson's ratio
- Characteristic yield stress, f_{yk}.
- Characteristic ultimate stress, f_{uk}.

The material name may only occur once in the *Material Specification* document, including definition of anchor material properties. It is used as a reference in the *Geometrical Specification* when connecting the anchor plate to a certain steel plate material definition.

The density, characteristic modulus of elasticity, and Poisson's ratio is used when creating a model for analysis.

The stresses are used to define the material behaviour in the created model for analysis, but also when normatively verifying the anchor plate.

7.1.4.2 Anchors

Input parameters regarding the anchors require the following information:

- A unique material name.
- Characteristic modulus of elasticity, E_k.
- Characteristic yield stress, f_{yk}.
- Characteristic ultimate stress, f_{uk}.

The material name may only occur once in the *Material Specification* document, including definition of steel plate material properties. It is used as a reference in the *Geometrical Specification* when connecting the anchor plate to a certain anchor material definition.

The characteristic modulus of elasticity and Poisson's ratio is used when creating a model for analysis.

The stresses are used to define the material behaviour in the created model for analysis, but also when normatively verifying the anchor plate.

8 Application for automatised computing

8.1 General

With knowledge of all required input information and how to validate an anchor plate the development of an application for automatised safety design can proceed. The main steps of the application can be identified as:

1. Gathering of required input information.
    a. *Geometrical Specifications.*
    b. *Load Specifications.*
    c. *Anchor Technical Specification.*
    d. *Material Specifications.*

2. Automatic generation of FE-models for analysis.

3. Analysis of generated FE-models.

4. Extraction of results from analysis and calculation of shear forces.
    a. *Extraction of normal forces in anchors.*
    b. *Extraction of maximum stress in steel plate.*
    c. *Extraction of maximum compressive stress in concrete.*
    d. *Calculations of shear forces at anchors due to system shear force components and torsion in accordance with CEN/TS.*

5. Validation of the extracted information according to the safety design documents.
    a. *Failure modes associated to normal forces.*
    b. *Failure modes associated to shear forces.*
    c. *Interaction failure.*

The application should be of automatic character and thus it is important to create a natural working flow that is easily interpreted and can be modified when necessary. Therefore the application will follow the above stated steps quite implicitly.

Important when automatising an application is that the user interface is user-friendly and the user-input is held at a minimum, but still enabling the user to have full control over the calculation process and insight in what type of operations are made in application core. As a result the most important user-input when starting the application are:

- Specification of project path (selection of project folder where inventory-files are kept).
- Specification of the element size (aim length of the sides for the element square).
- Concrete Tri-axial stress factor (due to axial elements constituting the FE-modelled concrete the 3D-effects need to be integrated).

In Appendix 3 - *Application flow chart* the entire application method is presented by execution basis.

8.2 Gathering of information

Information provided by the user for inventory of anchor plates has to be delivered as specified in chapter 7 according to certain pre-defined templates.

The application will search for files matching the templates anywhere in the project folder and automatically read them into the computer memory. Any conflicts regarding name uniqueness will result in that any duplicate read to memory after the first definition will be expunged from the anchor plate automatic analysis. The result of this may be that an anchor plate will reference to an incorrect material definition, anchor technical properties definition, and/or load definition. Therefore the user is informed of this error before the application continues.

Gathered information is then stored in a catalogue hierarchy based on the specified anchor plate ID. The reason to why this data is stored and not just directly utilised from the computer memory is to enable the user to re-execute the application without having to read in all the Excel-documents again which is a much more resource-heavy process than storing the data in a structured plain-text file.

If previous data from either the automatic model generation and/or analysis of FE-models exist in the project folder when the application is initiated it will ask to delete this files before proceeding, allowing the user to back-up possible important data.

If there is not enough inventory files in the project folder the application will self-terminate. The graphical user interface is seen in Figure 8.1 below.

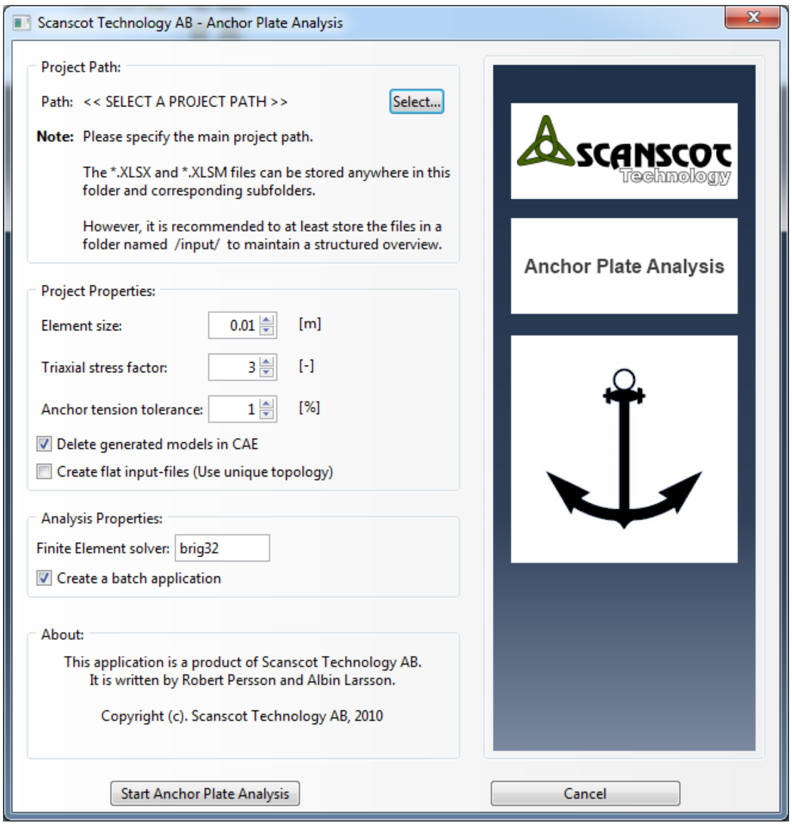


Figure 8.1. The main user-interface which let the user start the application.

8.3 Automatised model generation

8.3.1 Implementation of finite element model

The chosen model implemented in the automatised model generation process is described in detail in chapter 6.

8.3.2 Generation of model files

The generated model files are delivered as INP-files which can be run in the BRIGADE/Plus Software. However, it has been implemented an option that generates input files which are flat, meaning that every element and every node is unique in the FE-model definition, enabling the file being inserted to applications as Hypermesh and can be converted to other file formats from there. The main objective is not to create a general application that can be executed in all environments, but instead focus lies on the description of the automatised method for anchor plate design.

Model files are generated sequentially and the time cost is heavily dependent on the geometry of the anchor plate and the mesh density. As a guideline the model generation takes about 0.25 seconds for an average anchor plate 0.3 x 0.3 m in dimension, with 4 anchors, and a meshing size of 0.01 m. This value is specified as estimated elapsed time per enveloped load case. In one load combination block there are 12 enveloped load cases and a load specification is composed of at least one load combination block. Compared to a manual modelling procedure, even for an experienced user, the saving in time is manifold.

The process of model generation is instantiated by first creating the steel plate with the centre of gravity placed in origo and applying corresponding material definitions to it and later partition the surface element where needed, e.g. in intersections due to anchors and beam constraints.

By placing the steel plate in the global origin, various elements as beam attachments and anchors can be easily appended. This method enables the user to specify the absolute coordinate position of the element without any coordinate transformation operations required by either the user or the application itself, thus minimizing risks of erroneous definitions.

The next step in the model generation process is to create the beam constraints and anchor connector elements in their defined positions with their corresponding physical properties. Now the load combination effects can be added to the related beam attachment's centre of gravity as explained in chapter 6.

Finally the model is meshed into the desired element size. When this is done the application will calculate the actual size of each element and the area of influence for each surrounding node of that element with form functions for a quad-type element, making the process entirely mesh-independent and automatised. The total area of influence for each node is summarised by adding the influence area components for each element a particular node is neighbouring. Now the spring stiffness can be calculated for each connector element that constitutes the concrete spring-bed and is placed in the nodes of the mesh. (See Figure 8.2 for an illustrative explanation.)

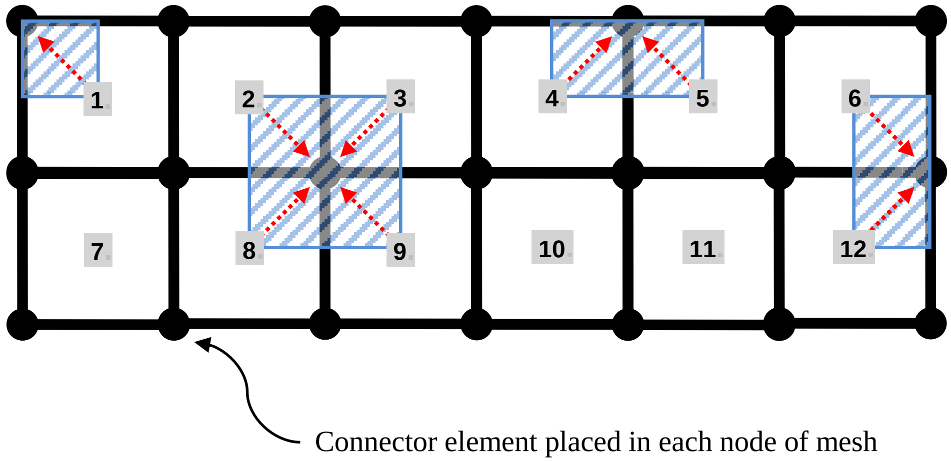


Figure 8.2. Render of how the area of influence from each element is summarised for all neighbouring nodes. (The numbers represent the elements).

One detail of importance in the implementation of the automated model generator is the effect of the tri-axial stress factor. Determination of the actual value for this factor lies outside the scope of this master's thesis and is therefore left to the user to specify in the application. Figure 8.3 shows the von Mises and Tresca yield criterion in case of two dimensional stress effects. In case of three dimensional stress effects which is of interest in this report the criterion will form a surface. The default tri-axial stress factor is set to 2.0 in accordance with the Scanscot Technology Methodology Report.

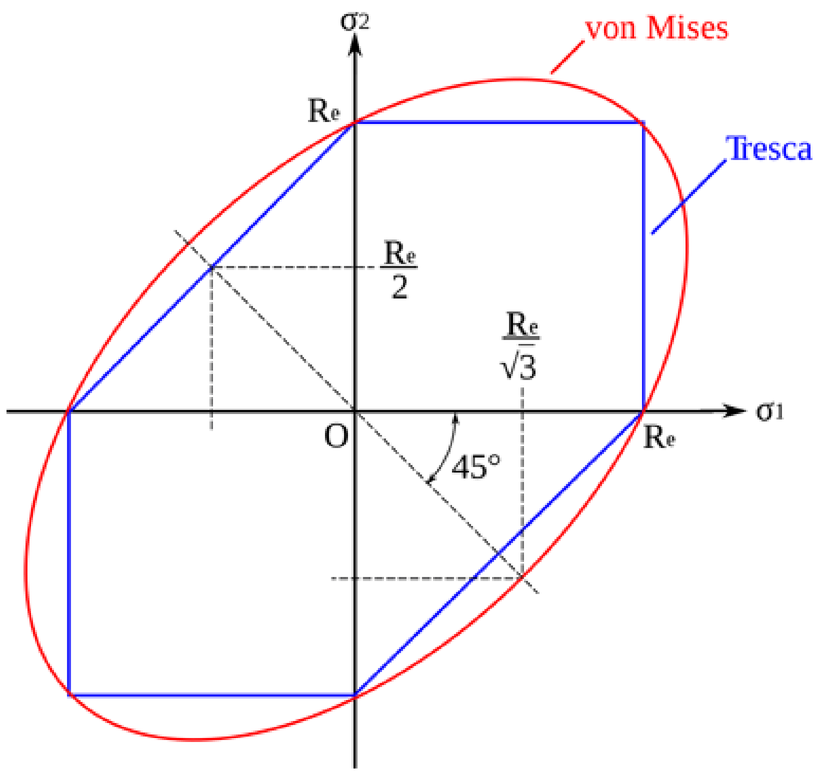


Figure 8.3. Von Mises and Tresca yield criterion.

8.4 Automatised normative validation

When all the FE-model files have been generated they will undergo analysis which will result in one database file for each of the valid defined anchor plate objects containing all analysis information. The result files are placed in the same catalogue hierarchy based on anchor plate ID created in the information gathering process.

The result files are later processed in the normative calculation module of the automatised application. These steps are described in the following sub-sections.

Computing time for normative verification of anchor plates vary with the complexity of the geometry definition of the anchor plate. With a denser mesh and increased number of anchors and beam attachments the time of analysis will greatly increase. As a guideline the result analysis takes about 2 seconds for an average anchor plate 0.3 x 0.3 m in dimension, with 4 anchors, and a meshing size of 0.01 m. This value is specified as estimated elapsed time per enveloped load case. In one load combination block there are 12 enveloped load cases and a load specification is composed of at least one load combination block.

Additional deployment of computing resources is required to perform the FE-analysis, which will add to the total elapsed time.

8.4.1 Extraction of essential normative information

During the automatised model generation process, several data files containing area of influence for each node and general information (such as geometries, load combinations and other anchor plate definitions) are created. These files are necessary for the normative validation process to be successful. (See section 8.2 for an explanation on why this information is stored in separate files.)

8.4.2 Extraction of normal forces and stresses

From the created result databases the normal forces in anchor connectors and in concrete spring-bed connectors are extracted automatically and sorted correspondingly. Moreover maximum Mises-stresses in the steel plate are extracted.

With the information extracted according to section 8.4.1 the concrete stresses can be calculated since both spring force and area of influence is known, $= \frac{F_n}{A}$.

8.4.3 Calculation of shear forces

Next step in the normative validation process is to calculate shear force effects in corresponding anchors. Why these effects are not extracted in the same manner as for normal forces and stresses is because of the formulation of the *CEN/TS 1992-4* norm.

CEN/TS 1992-4 utilises different sets of shear forces depending on active failure mode. In some modes the shear force is to be distributed correctly over all active anchors and other cases the shear force component orthogonal to an edge should be distributed over the anchors closest to the edge whilst the shear force components parallel to the edge should be distributed over all active anchors.

As a result of this complexity the shear forces are calculated manually according to static elastic methods where shear forces working in the steel plate plane and torsion moments are taken into consideration. Figure 8.4 shows how forces influence the shear force components of a single anchor.

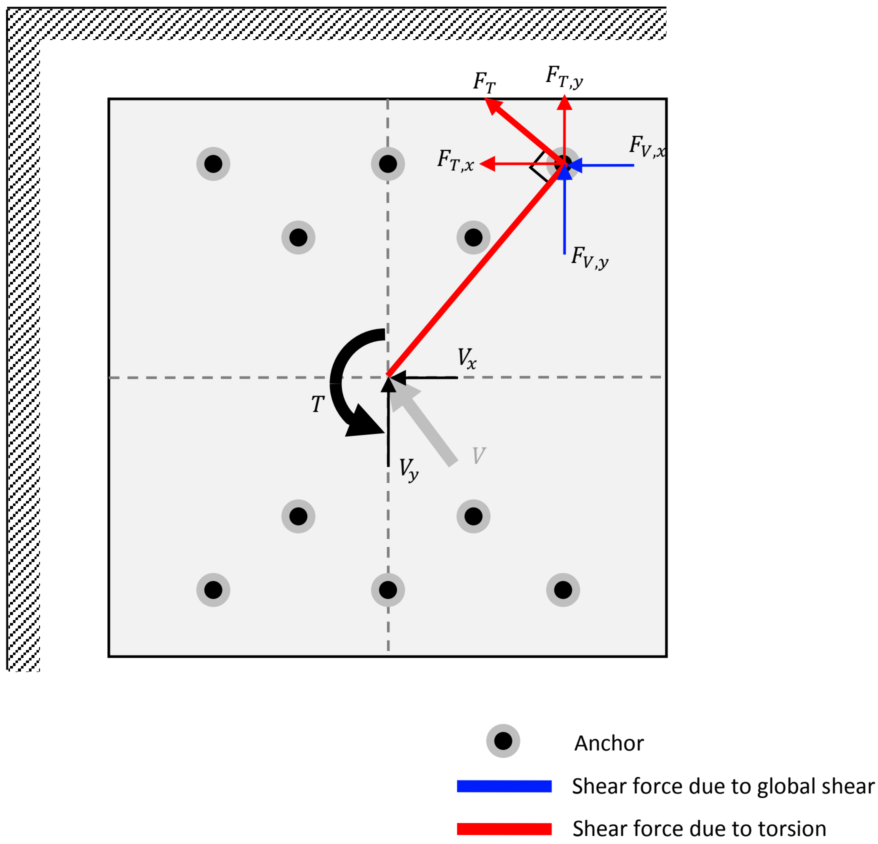


Figure 8.4. Shear force components.

Shear in anchor when forces acting in x-direction:

$F_{V,x} = \frac{V_x}{\text{nr. of active anchors}}$ Shear component due to global shear. (Eq. 46)

$F_{T,x} = -\frac{T*y_i}{I_p}$ Shear component due to torsion. (Eq. 47)

$I_p \quad = \sum_n (x_i^2 + y_i^2)$ Polar moment of inertia for all anchors. (Eq. 48)

$$F_x = F_{V,x} + F_{T,x} = \frac{V_x}{\text{nr. of active anchors}} - \frac{T*y_i}{I_p} \qquad \text{(Eq. 49)}$$

Shear in anchor when forces acting in y-direction:

$F_{V,y} = \frac{V_y}{\text{nr. of active anchors}}$ Shear component due to global shear. (Eq. 50)

$F_{T,y} = \frac{T*x_i}{I_p}$ Shear component due to torsion. (Eq. 51)

$I_p \quad = \sum_n (x_i^2 + y_i^2)$ Polar moment of inertia for all anchors. (Eq. 52)

$$F_y = F_{V,y} + F_{T,y} = \frac{V_y}{\text{nr. of active anchors}} + \frac{T*x_i}{I_p} \qquad \text{(Eq. 53)}$$

8.4.4 Calculation of concrete areas

There are 4 different concrete failure areas that have to be calculated:

1. Concrete cone area & Splitting cone area (same area for both failure modes).
2. Concrete pry-out area.
3. Concrete blow-out area.
4. Concrete edge area.

The algorithms used are quite complex and comply with the methods described in section 8.6 and according to the rules in *CEN/TS 1992-4* norm (See chapter 5). Therefore the calculations will not be presented in this section.

8.4.5 Calculation of capacities and validation of effects

Depending on what type of anchor is attached to the steel plate, cast-in-place headed fasteners or post-installed fasteners, the application will carry out a certain sequence of verification calculations. These verifications comply with section 8.6 and according to the rules in *CEN/TS 1992-4* norm (See chapter 5).

Material stresses for the concrete member and the steel plate are automatically verified (see section 8.4.2.)

Generous error handling has been implemented with call-back functions that will notify and log any errors that may occur during the verification process, including area calculations. If an error occurs the application will continue to the next anchor plate consequently making the program difficult to crash.

8.4.6 Output

The results of the verification are then saved in two separate Excel-files. The first file is a global summary file, where the most severe mode of failure is appended so a quick overview for all anchor plates in the project folder is accessible.

The second file is an anchor plate specific file containing detailed information regarding stresses in the steel plate and concrete, normal and shear loads in each defined anchor for different types of failure modes, concrete areas for all valid failure modes, and finally effects, capacities, and ratio of utilisation for each load case in all load combination blocks.

See Appendix *4 - Comparison example* for a representation of the result files.

8.5 Limitations of the application

Due to the chosen type of FE-model utilised in the automatised process and specifications in section 1.4 regarding project scope several limitations exists within the application which has to be revised before professional deployment.

These limitations are:

- No capability to define stand-off type anchor plates since the model uses spring elements to connect the steel plate to the concrete member. (See Figure 3.7).

- At least two anchors have to be defined even though *CEN/TS 1992-4* specify methods of design even for a single anchor. This is due to the FE-modelling approach which does not have any boundary conditions. In order to prevent rotational body motion at least two anchors have to be defined. Since the program only accepts at least two fasteners the single fastener column in Table 1 of *CEN/TS 1992-4-2* is omitted.

- A total of two definitions of free edges or neighbouring anchor plates. This is a result of the inventory method which can only accept two input parameters.

- All anchors in one anchor plate object must be of the same type and length. This is due to limitations of the *CEN/TS 1992-4* norm.

- No reinforcement can be taken into consideration. E.g. factors connected to reinforcement $\psi_{re,V}$ will always be assigned the value 1 in a conservative manner (on the safe side).

- Only Eurocode standard concrete classes are accepted. If a construction element is defined with any other type of concrete class this has to be converted to a Eurocode class with same properties or the class with inferior properties closest to the actual values. This is also a conservative method and will generate results on the safe side.

- Accepted anchor-types are limited to cast-in-place headed fasteners and post-installed fasteners.

- As a result of the concrete cone area and pry-out area calculation algorithms the anchors has to be defined in perfect rows and columns to ensure correct results. This has to do with how the algorithm calculates delimiting anchors. Improvements can be easily implemented as the application is programmed modular where data is fetched from the different modules. As long as the improved algorithm delivers the newly calculated areas in the same format as earlier there is no need to further re-programming.

- Analysis and normative validation only accept static loads due to definition of the *Load Specification* document.

8.6 Implementation of normative verification

As explained in chapter 5 regarding the normative verification of anchor plates, the adapted norm utilised in the automated design process is the CEN/TS 1992-4 publication.

Section 5.6 and section 5.7 specify how the different modes of failure present at verification of anchor plates are to be validated. In the following chapters the actual implementation of these validations in the specific automatised application are explained in detail.

8.6.1 Cast-in-place headed anchors

8.6.1.1 Failure modes for fasteners loaded in tension

The failure modes for cast-in-place headed fasteners loaded in tension are to be verified according to Table 5-1, presented in full context below.

Table 8-1. Required verification for fasteners loaded in tension, (CEN/TS 1992-4-2, Table 1).

		Single fastener	Fastener group	
			most loaded fastener	*fastener group*
①	**Steel failure of fastener**	$N_{Ed} \leq N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}}$	$N_{Ed}^{h} \leq N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}}$	-
②	**Pull-out failure of fastener**	$N_{Ed} \leq N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}}$	$N_{Ed}^{h} \leq N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}}$	-
③	**Concrete cone failure**	$N_{Ed} \leq N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}}$	-	$N_{Ed}^{g} \leq N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}}$
④	**Splitting failure**	$N_{Ed} \leq N_{Rd,sp} = \frac{N_{Rk,sp}}{\gamma_{Msp}}$	-	$N_{Ed}^{g} \leq N_{Rd,sp} = \frac{N_{Rk,sp}}{\gamma_{Msp}}$
⑤	**Blow-out failure**[a]	$N_{Ed} \leq N_{Rd,cb} = \frac{N_{Rk,cb}}{\gamma_{Mc}}$	-	$N_{Ed}^{g} \leq N_{Rd,cb} = \frac{N_{Rk,cb}}{\gamma_{Ms}}$
⑥	Steel failure of reinforcement	$N_{Ed,re} \leq N_{Rd,re} = \frac{N_{Rk,re}}{\gamma_{Ms,re}}$	$N_{Ed,re}^{h} \leq N_{Rd,re} = \frac{N_{Rk,re}}{\gamma_{Ms,re}}$	-
⑦	Anchorage failure of reinforcement	$N_{Ed,re} \leq N_{Rd,a}$	$N_{Ed,re}^{h} \leq N_{Rd,a}$	-

[a] Not required for fasteners with edge distance, $c > 0{,}5\ h_{ef}$

8.6.1.1.1 Steel failure

Characteristic capacity values are specified by the user in the *Anchor Technical Specification* document connected to each anchor plate which is to be analysed. Required input data is:

1. Characteristic capacity, $N_{Rk,s}$, calculated according to Eurocode Standards above.
2. Partial factor for resistance, $\gamma_{Ms} \quad \rightarrow \quad \{Recommended\ value\} = 1.2 * \frac{f_{yk}}{f_{uk}}$.

8.6.1.1.2 Pull-out failure

In case of cast-in-place headed fasteners the user is asked to specify relevant data in the *Anchor Technical Specification* document connected to each anchor plate which is to be analysed.

Required input data is:

1. External diameter of the anchor head (studs), d_h.
2. Partial factor for resistance, γ_{Mp}, given for load type A+B and load type C+D. (See section 7.1.2 for more information regarding different load types.)

According to *CEN/TS 1992-4-1* the recommended value for γ_{Mp} is γ_{Mc} and is determined accordingly:

$$\gamma_{Mc} = \gamma_c * \gamma_{inst} \qquad \text{(Eq. 54)}$$

The recommended value for γ_c is 1.5 and if the conditions of *CEN/TS 1992-4-1, section 4.5* and *EN 1992-1-2:2004, section 4.5.5* are fulfilled one may assume a high installation safety for all load directions. In the case of high installation safety γ_{inst} is set to 1.0.

Worth mentioning is the fact that the diameter of the anchor shaft is not specified in the *Anchor Technical Specification*, but instead this data is fetched from the *Geometrical Specification* for the corresponding anchor plate. (See section 7.1.3 for more information regarding different diameter definitions.)

Moreover the determination of which state the concrete material should be classified in, i.e. whether it is cracked or not, is made on basis of input data from the *Load Specification* document connected to the relevant anchor plate in which the crack width parameters for each load combination are specified.

8.6.1.1.3 Concrete cone failure

The implementation of concrete cone capacity calculations is made as stated above. No interpretations are made to the code. In the *Technical Specification* document the user is asked to specify the characteristic resistance coefficient for cracked (k_{cr}) and non-cracked concrete (k_{ucr}) as well as the effective anchorage depth (h_{ef}) and partial safety factors (γ_{Mc}) for load type A+B and load type C+D.

The automatised process will calculate and compare concrete cone areas $n * (n - 1)$ times where n is number of active anchors.

Optimisations are done to improve the performance of the application. Two major implementations for performance improvement are:

- studying the influence zone.
- validation of internal anchor distances.

The first validation monitors whether or not the two studied anchors in the comparison loop lies within the minimum distance of influence. Every anchor is geometrically positioned by a set of coordinates. We choose to name the distance between these two coordinate sets the *distance of influence*. By defining a minimum distance of influence where overlapping of areas never can occur it is possible to eliminate redundant calculations.

The definition of the distance of influence is presented in Figure 8.4 below.

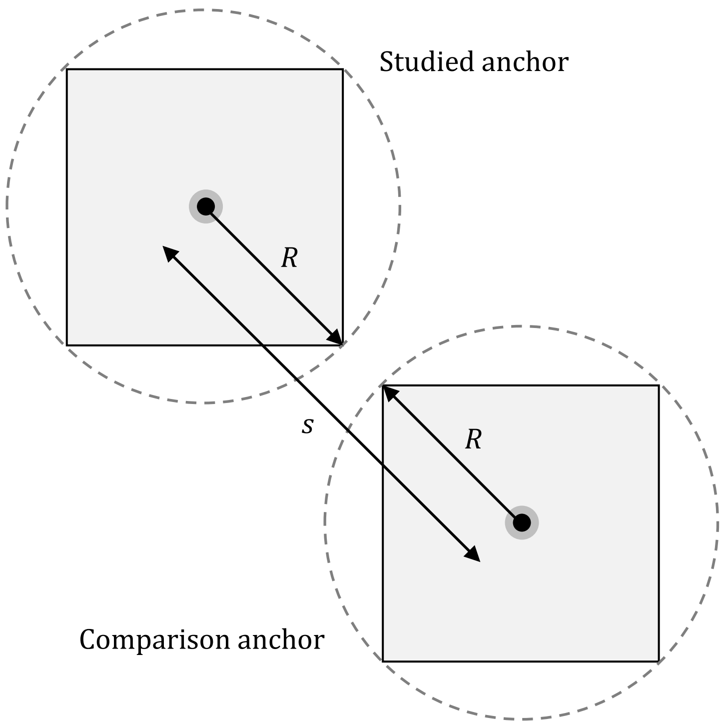


Figure 8.4. Definition of the minimum distance of influence, 2*R.

With the anchor coordinate as a concentration point each anchor will have a quadratic area of influence surrounding it. By setting the most distant point in the quadrate as the radius of a circle one receives an influence zone according to Figure 8.4 which guarantees no interference of the projected area if no objects intrude this zone.

Since an anchor plate is assumed to be built of the same type of anchor according to the limitations in the utilised CEN/TS norm this give us the following equations of validation.

Condition: $s \geq 2R$ No control necessary, the comparison anchor will not intrude the studied anchor's zone of influence. If condition is not met, the studied anchors will interfere and correct concrete area must be calculated.

$$s = \sqrt{(x_2 - x_1)^2 + (y_2 - y_1)^2}$$ Actual distance of influence (Eq. 55)

$$2R = 2 * \sqrt{2 * \left(1.5 * s_{cr,N}\right)^2}$$ Minimum distance of influence (Eq. 56)

$$= 2 * \sqrt{4.5 * h_{ef}^2} = \sqrt{18 * h_{ef}^2}$$

The second validation made to improve performance is through verification of the distance orthogonal to the studied direction. If the distance is larger than $s_{cr,N} = 3 * h_{ef}$ the two anchors will never interfere with each other. Figure 8.5 illustrates the validation principle.

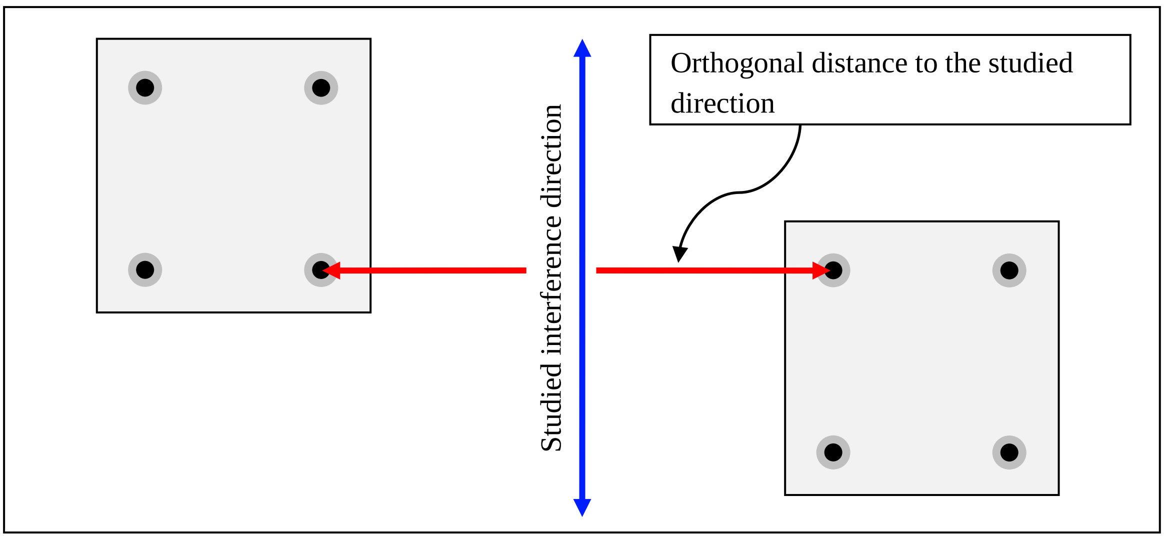


Figure 8.5. Definition of the orthogonal distance compared to the studied direction of interference.

8.6.1.1.4 Concrete splitting failure

Implemented correspondingly to the concrete cone failure mode calculations according to section 5.6.3.4 with addition of *skipping-of-validation* conditions. Because of the simplification regarding the lack of reinforcement, no splitting reinforcement can be accounted for, hence only condition 1 (one) is verified.

Since the projected area and reference area calculations are identical for both failure modes there is no need to process all anchors again which will save computing resources. The effect factors are however re-calculated due to changes in the characteristic edge distance and characteristic spacing distance for ensuring the transmission of the characteristic resistance of a single fastener.

8.6.1.1.5 Concrete blow-out failure

A generalized method of verification is to be developed which regardless of how the different fasteners are positioned can calculate a valid resistance in case of blow-out failure. As it is implied by the normative directives only anchors within the distance of $0.5 * h_{ef}$ from the edge will be considered active in the blow-out failure mode.

Calculations are done in the following steps:

1. Sorting out anchors that lies outside the active blow-out zone of $0.5 * h_{ef}$.
2. Project all active anchors to a common baseline which is extended from the anchor positioned closest to the edge.
3. Calculate all blow-out areas for each separate anchor with regard to interferences of nearby anchors and edges. (Even edges that exist in two directions, e.g. corners.)

This process is done for all eventual edges and the edge with highest ratio of utilisation is presented as the decisive ratio for the blow-out failure mode.

When breaking down this method of verification we get a combination of favourable and unfavourable factors which are to be considered balanced. The effects this calculation method has on the ratio of utilisation are listed below:

- When projecting anchors to a common baseline the edge distance will be set as the shortest possible for each fastener. This will result in a characteristic resistance of a single unobstructed anchor that is smaller than the actual value would have been for each separate anchor (unfavourable). This is however a necessary step since the CC-method manages the group effect as described in the example of Chapter 7.3.2.3.1 and requires a homogenous characteristic resistance of a single anchor not influenced by adjacent fasteners or free structural component edges.
- Opposite to the effect on the characteristic resistance of a single anchor, a smaller edge distance will act favourably on the ratio of utilisation regarding the factor of geometric effect due to axial spacing and edge distance. When the edge distance c_1 decreases, so will the reference projected area ($A^0_{c,Nb}$) resulting in an increase of the quota $\frac{A_{c,Nb}}{A^0_{c,Nb}}$.

- Decrease in edge distance will act favourably with regard to calculations concerning the disturbance of stresses in concrete due to corners and the group behaviour effect of bearing area.
- Decrease in edge distance will act unfavourably on the factor regarding the eccentricity of the load.

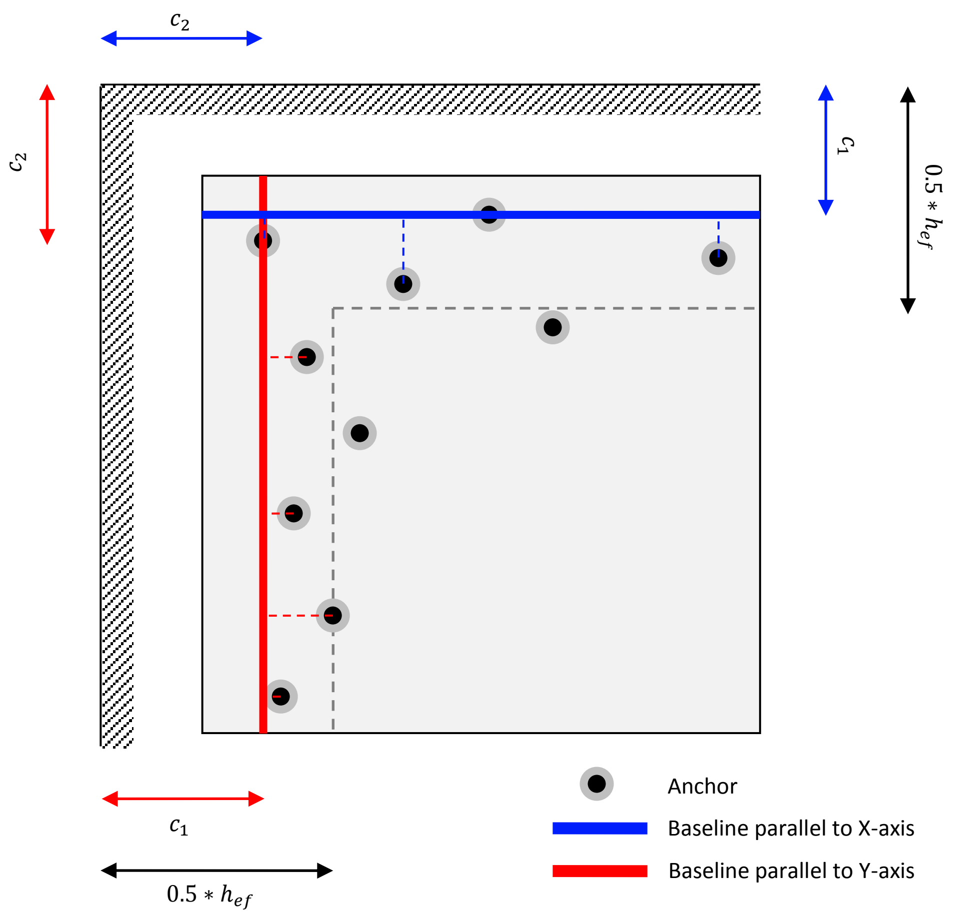


Figure 8.6. Definition of valid edge distances in case of blow-out failure for generalised method of verification. Anchors are to be projected to a common baseline for each present edge (the dashed lines).

The verification whether or not an anchor lays within the blow-out failure zone is processed during the calculation of concrete cone areas. Thus optimising the application and reducing the amount of computer recourses consumed. Calculation of blow-area cones will require:

$$\left(n_{\text{active blow-out anchors parallel x-axis}} * \left(n_{\text{active blow-out anchors parallel x-axis}} - 1\right)\right) +$$
$$\left(n_{\text{active blow-out anchors parallel y-axis}} * \left(n_{\text{active blow-out anchors parallel y-axis}} - 1\right)\right)$$

8.6.1.2 Failure modes for fasteners loaded in shear

The failure modes for cast-in-place headed fasteners loaded in shear are to be verified according to Table 5-2, presented in full context below.

Table 5.2. Required verifications for fasteners loaded in shear. (CEN/TS 1992-4-2, Table 2)

		Single fastener	Fastener groups	
			most loaded fastener	*fastener group*
①	**Steel failure of fastener without 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 with lever arm**	$V_{Ed} \leq V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{Ms}}$	$V_{Ed}^{h} \leq V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{Ms}}$	-
③	**Concrete edge failure**	$V_{Ed} \leq V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{Mc}}$	-	$V_{Ed}^{g} \leq V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{Mc}}$
④	**Concrete pry-out failure**	$V_{Ed} \leq V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{Mc}}$	-	$V_{Ed}^{g} \leq V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{Mc}}$
⑤	Steel failure of supplementary reinforcement	$V_{Ed,re} \leq V_{Rd,re} = \frac{V_{Rk,re}}{\gamma_{Ms,re}}$	$V_{Ed,re}^{h} \leq V_{Rd,re} = \frac{V_{Rk,re}}{\gamma_{Ms,re}}$	-
⑥	Anchorage failure of supplementary reinforcement	$V_{Ed,re} \leq N_{Rd,a}$	$V_{Ed,re}^{h} \leq N_{Rd,a}$	-

8.6.1.2.1 Steel failure without lever arm

Characteristic capacity values are specified by the user in the *Anchor Technical Specification* document connected to each anchor plate which is to be analysed.

Required input data is:

1. Characteristic capacity, $V_{Rk,s}$, calculated according to Eurocode Standards.
2. Partial factor for resistance, $\gamma_{Ms} \quad \rightarrow \quad \{Recommended\ value\} = 1.2 * \frac{f_{yk}}{f_{uk}}$.

8.6.1.2.2 Steel failure with lever arm

Not implemented due to limitations to the scope of this master's thesis. Stand-off capability is not implemented in the automatized calculation.

8.6.1.2.3 Concrete pry-out failure

Implemented as in section 5.6.5.3. The area calculations are needed to be re-calculated since active anchors can differ because of the concrete pry-out failure mode are determined for anchors loaded with shear forces. This will, as in the case of concrete cone area calculations, require $n * (n - 1)$ iterations.

The concrete pry-out verification is done two times, once for the load effect of the entire fastener group and once for the most loaded anchor. Both results are presented in the Excel result-sheet.

Factor, k_3, and partial safety factors are specified by the user in the *Anchor Technical Specification*.

8.6.1.2.4 Concrete edge failure

Verification of concrete edge failure is implemented similar to the concrete blow-out failure mode. The same importance lies within the fact to keep the method of verification generalised enabling calculations on arbitrary geometries. An example is illustrated in Figure 8.7 below whereas the shear load can be divided into two components acting perpendicular towards the different edges. (Note that both shear load components affect the resistance calculations for each edge.)

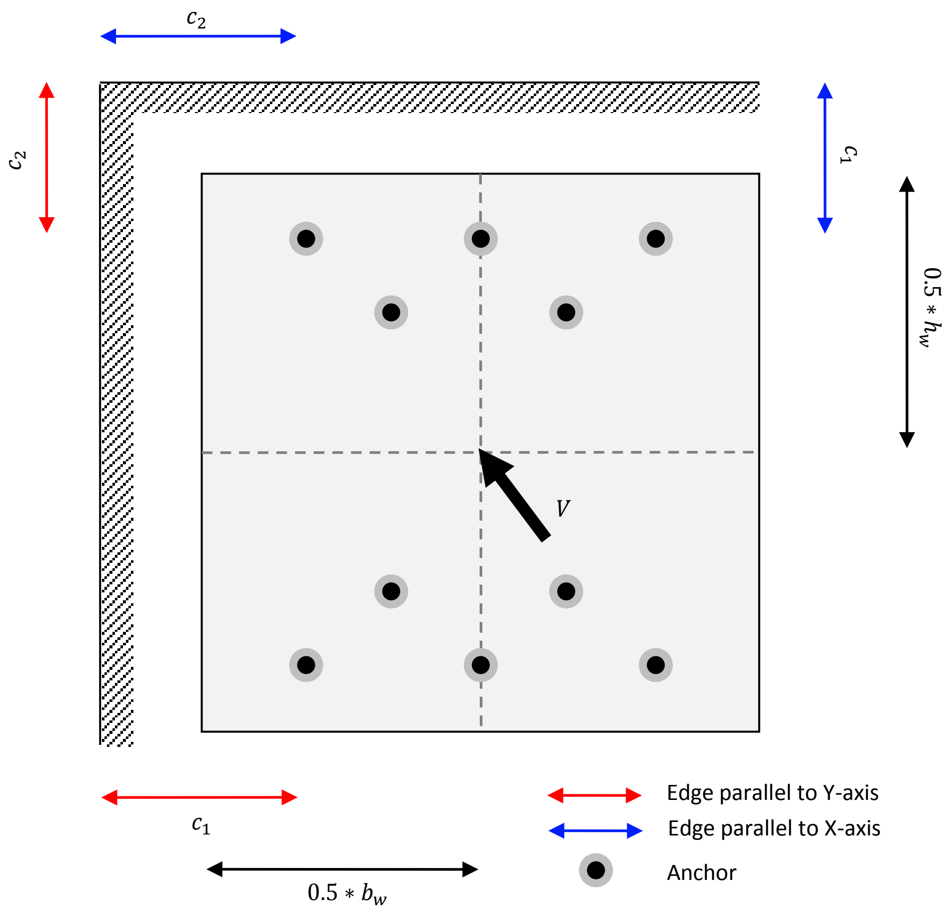


Figure 8.7. Concrete edge failure example. Two free edges present with respective shear load components acting perpendicular towards them.

A major difference compared to the concrete blow-out failure mode is that there is no active anchor zone specified, but instead one needs to verify if the individual anchors are loaded with shear loads and how they can contribute to the concrete edge break-out area.

Verification of the concrete edge failure mode is done accordingly:

- First and foremost the shear loads are calculated for each edge. Shear loads depend on edge direction thus utilising one iteration per present edge and number of active anchors.
- A selection of active anchors is made.
- Active anchors are projected to a common baseline which is extended from the anchor positioned closest to the edge.
- Calculation of all concrete edge areas for each separate anchor with regard to interferences of nearby anchors and edges. (Even edges that exist in two directions, e.g. corners.)
- Verification of the largest ratio of utilisation. The edge with largest ratio is decisive.

Effective shear load on each anchor is calculated manually with contribution from shear components acting on the beam attachment in the direction of the X-axis, the Y-axis, and from eventual torsion moment. In accordance to the definition in *CEN/TS 1992-4-2, Section 6.3.5.2* regarding distribution of shear load on individual anchors; shear load acting perpendicular to the edge shall be divided over the active anchors in the concrete edge failure mode. However, shear load acting parallel to the edge shall be distributed over all anchors enabled to manage shear. This results in the following scenario in Figure 8.8 for the example given in Figure 8.7.

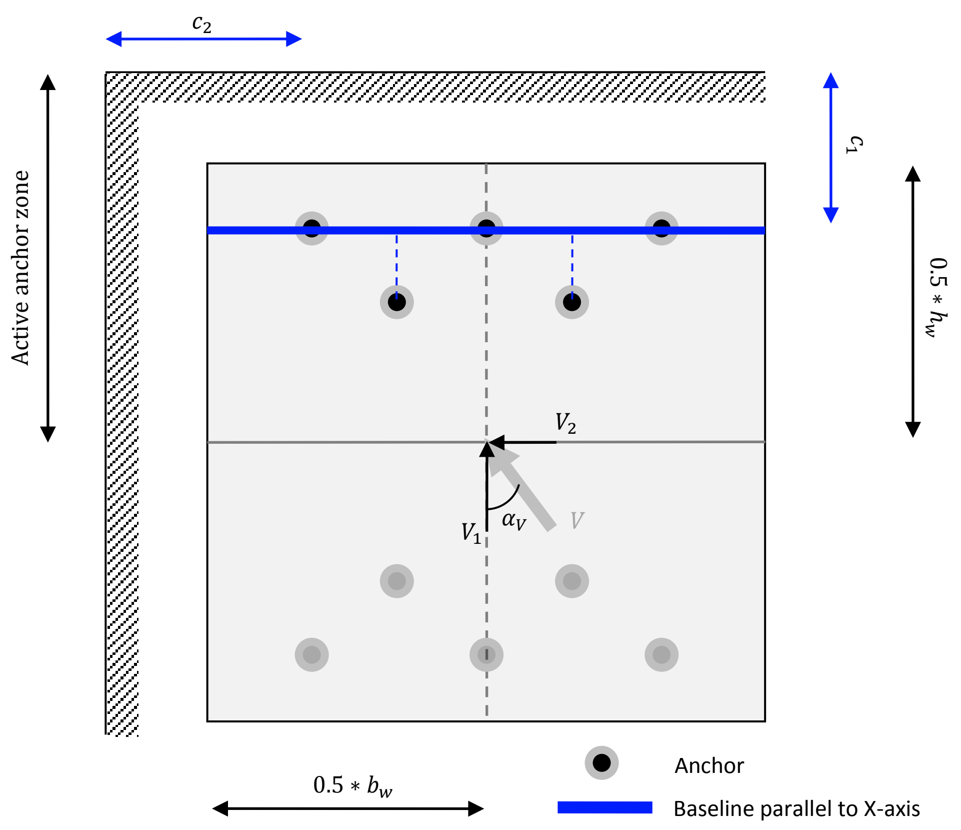


Figure 8.8. Calculation of shear loads with regard to edge parallel to X-axis.

All sheared anchors within the anchorage zone, limited by a parallel line to the studied edge through the geometric concentration point of the steel plate, are considered as active anchors in case of concrete edge failure. (See Figure 8.8 and Figure 8.9 for examples over the active anchor selection zone.)

As with concrete blow-out failure the active anchors are projected to a common baseline (the blue line and the red line in Figure 8.8 respective Figure 8.9) with a constant edge distance, c_1. (See section 5.6.3.5 on why this is done and how this will affect the resistance calculation.)

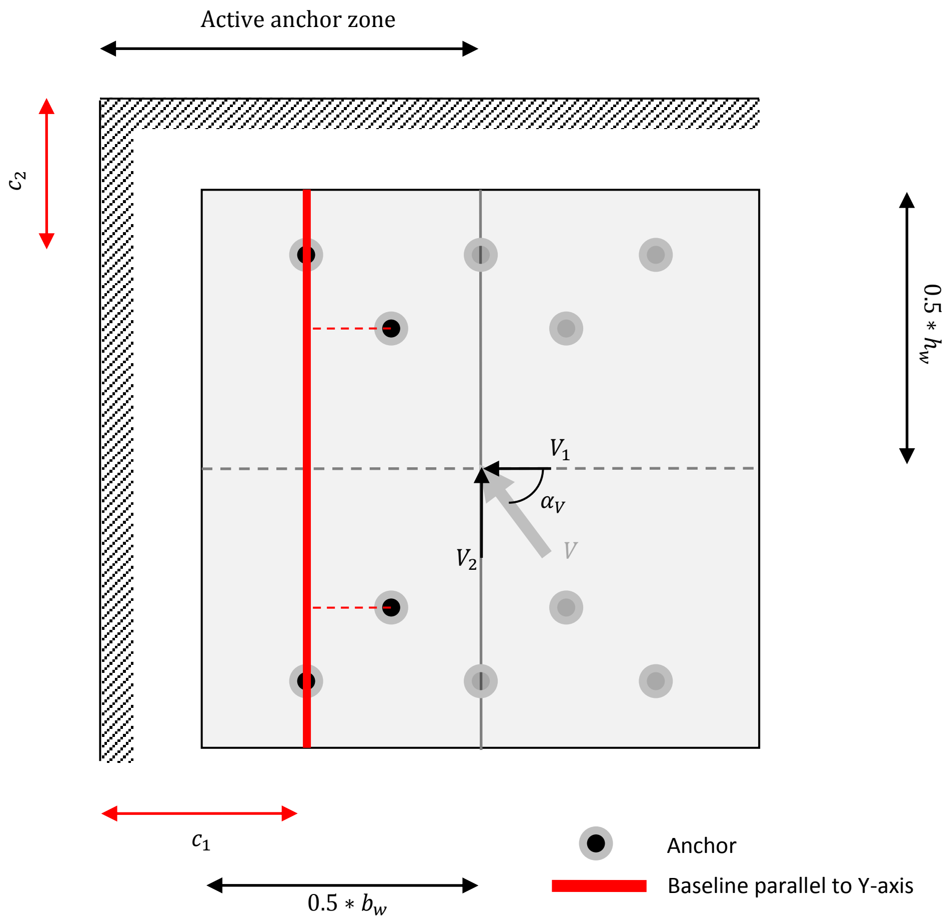


Figure 8.9. Calculation of shear loads with regard to edge parallel to Y-axis.

It is important to identify the shear load acting perpendicular respectively parallel to the studied edge, permitting the correct calculations of the effect factors but also to accurately calculate the correct load effects and utilisation ratios for corresponding edges. The shear load components are calculated according to section 8.4.3.

8.6.2 Post-installed expansion anchors

8.6.2.1 Failure modes for fasteners loaded in tension

The failure modes for expansion anchors loaded in tension are to be verified according to Table 5.3, presented in full context below.

Table 5.3. Required verification for fasteners loaded in tension. (CEN/TS 1992-4-4, Table 3)

		Single fastener	Fastener group	
			most loaded fastener	*fastener group*
①	**Steel failure of fastener**	$N_{Ed} \leq N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}}$	$N_{Ed}^{h} \leq N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}}$	-
②	**Pull-out failure of fastener**	$N_{Ed} \leq N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}}$	$N_{Ed}^{h} \leq N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}}$	-
③	**Concrete cone failure**	$N_{Ed} \leq N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}}$	-	$N_{Ed}^{g} \leq N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}}$
④	**Splitting failure**	$N_{Ed} \leq N_{Rd,sp} = \frac{N_{Rk,sp}}{\gamma_{Msp}}$	-	$N_{Ed}^{g} \leq N_{Rd,sp} = \frac{N_{Rk,sp}}{\gamma_{Msp}}$

8.6.2.1.1 Steel failure

The steel failure mode is implemented as defined in section 8.6.1.1.1.

8.6.2.1.2 Pull-out failure

The user is asked to specify the characteristic resistance stated in the European Technical Specification. When later calculating the normative resistance the application will first search for values that apply for the used concrete strength class. If corresponding values do not exist the application will search for values at closest concrete class with lesser strength than actually used. If this fails, the application will not analyse the anchor plate and continue to the next anchor plate.

Partial safety factors are specified as in section 8.6.1.1.2.

8.6.2.1.3 Concrete cone failure

The concrete cone failure mode is implemented as defined in section 8.6.1.1.3.

8.6.2.1.4 Concrete splitting failure

The concrete cone failure mode is implemented as defined in section 8.6.1.1.4.

8.6.2.1.5 Concrete blow-out failure

The concrete splitting failure mode is implemented as defined in section 8.6.1.1.5.

8.6.2.2 Failure modes for fasteners loaded in shear

The failure modes for expansion anchors loaded in shear are to be verified according to Table 5-4, presented in full context below.

Table 5.4. Required verifications for fasteners loaded in shear. (CEN/TS 1992-4-4, Table 4)

		Single fastener	Fastener groups	
			most loaded fastener	*fastener group*
①	**Steel failure of fastener without 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 with lever arm**	$V_{Ed} \leq V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{Ms}}$	$V_{Ed}^{h} \leq V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{Ms}}$	-
③	**Concrete edge failure**	$V_{Ed} \leq V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{Mc}}$	-	$V_{Ed}^{g} \leq V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{Mc}}$
④	**Concrete pry-out failure**	$V_{Ed} \leq V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{Mc}}$	-	$V_{Ed}^{g} \leq V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{Mc}}$

8.6.2.2.1 Steel failure without lever arm

The steel failure mode is implemented as defined in section 8.6.1.2.1.

8.6.2.2.2 Steel failure with lever arm

Not implemented due to limitations to the scoop of this master's thesis. Stand-off capability is not implemented in the automatized calculation.

8.6.2.2.3 Concrete pry-out failure

The concrete pry-out failure mode is implemented as defined in section 8.6.1.2.3.

8.6.2.2.4 Concrete edge failure

The concrete edge failure mode is implemented as defined in section 8.6.1.2.4

9 Results

This master's thesis has resulted in a methodology and an application for automatised safety verification of anchor plates which comply with normative guidelines of the CEN/TS 1992-4 technical specification. The result is a method defining necessary information which is used when generating a FE-model for advanced analysis and normative verification but also methods of implementation enabling the automatisation of safety design.

A part of the project result that is easily forgotten, but still is an essential part of the methodology for automatised computing is the parametric study, where conclusions from this study lies as base for the entire methodology. It was used to evaluate which type of model was best suited for a high performance analysis with sufficient accuracy where the time of analysis is of utmost importance. The results showed that a shell element for modelling of the steel plate, non-linear springs modelling the anchors, and an attached spring-bed simulating the concrete foundation was an optimal solution to keep the result accuracy satisfactory meanwhile the analysis time was of incomparable scale. Analysis time of the chosen model vary with geometry complexity but as a guideline a fairly simple, but quite common type of anchor plate, takes about 20 seconds to analyse.

Maybe the most important result of this master's thesis is the conclusion that an advanced screening application for mass-verification of a great many anchor plates may be done in a time-efficient and accurate manner. The application developed within the boundaries of this project has shown this, in a somewhat restricted manner even though the application is in many aspects totally general.

In Appendix 4 - *Comparison example* the developed application is compared to a manually verified anchor plate. Firstly the results of the manual verification are presented, after which the results from the application is presented. In the automated report, load effects have only been applied to the first envelope case in a load combination block named '*Temperature*'. This envelope case match the defined load effects from the hand calculated verification. All other envelopes in the same load combination block are not loaded.

10 Discussion

The result of this master's thesis is in no way absolute. Continuation of method development is required to enable a complex screening method for safety design of anchor plates. Example areas of improvement are to implement an automatic method for load combination of defined loads instead of the current operation where pre-load combined definitions have to be entered manually. Moreover it is also important to enable the appliance of load effects on multiple attachments.

As stated in the results and in the report itself, the main purpose of this application is to create a method for screening a large number of anchor plates to receive an overview on basis of which one can point out specific anchor plates in need of special investigation. It is possible to save a lot of time in the construction of anchor plates respective validation of present anchor plates where only the items which does not pass the automated process needs additional manual enquiries.

In Appendix 4 - *Comparison example* the calculated capacities could in a matter of a second be verified against the load effects. As it is shown the automatically calculated capacities concur with the capacities calculated by hand. The differences that do exist lies within the fact that the load effects are calculated according to advance non-linear FE-methods whereas the manually calculated load effects are for a rigid elastic body. Therefore the ratios of utilisation will not coincide and other factors may differ due to the centre of gravity weighted against anchor forces no longer match the geometric centre of gravity because of irregular distribution of normal forces. However, the potential in the application is shown powerful and accurate.

11 References

11.1 Literature

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11.2 Technical documentations

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11.3 Design codes

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European Committee for Standardization, *Technical Specification CEN/TS 1992-4-4*. (2009)

Appendix 1 - Parametric study - Tables

Table 1. Overview of parametric study. Different cases with varying parameters are defined.

	Element type		Element meshing size		Material definition		Anchor plate fasteners	Contact definition	Load definition
	Steel Plate	Concrete	Steel Plate	Concrete	Steel Plate	Concrete			
Ref. Case 1	Solid	Solid	0.010 m	0.010 m	Linear	Non-Linear	Connectors	Hard	Normal force
Case 1.A	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Connectors	Hard/Exponential	Normal force
Case 1.B	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Connectors	Soft/Exponential	Normal force
Case 1.C	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Springs	Hard/Exponential	Normal force
Case 1.D	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Springs	Soft/Exponential	Normal force
Case 1.E	Shell	Solid	0,010 m	0,025 m	Linear	Linear	Connectors	Hard/Exponential	Normal force
Case 1.F	Shell	Solid	0,010 m	0,025 m	Linear	Non-linear	Connectors	Hard/Exponential	Normal force
Case 1.G	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Connectors	Hard/Exponential	Normal force
Case 1.H	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Connectors	Soft/Exponential	Normal force
Case 1.I	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Connectors	Hard/Exponential	Normal force
Case 1.J	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Connectors	Soft/Exponential	Normal force
Case 1.K	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Springs	Hard/Exponential	Normal force
Case 1.L	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Springs	Soft/Exponential	Normal force
Case 1.M	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Springs	Hard/Exponential	Normal force
Case 1.N	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Springs	Soft/Exponential	Normal force
Case 1.O	Solid	Solid	0,025 m	0,025 m	Linear	Linear	Connectors	Soft/Exponential	Normal force
Case 1.P	Solid	Solid	0,025 m	0,025 m	Linear	Non-linear	Connectors	Soft/Exponential	Normal force
Case 1.Q	Solid	Solid	0,025 m	0,025 m	Linear	Linear	Springs	Soft/Exponential	Normal force
Case 1.R	Solid	Solid	0,025 m	0,025 m	Linear	Non-linear	Springs	Soft/Exponential	Normal force
Case 1.S	Solid	Solid	0,010 m	0,010 m	Linear	Linear	Connectors	Soft/Exponential	Normal force
Case 1.T	Solid	Solid	0,010 m	0,010 m	Linear	Non-linear	Connectors	Soft/Exponential	Normal force
Case 1.U	Shell	-	0,010 m	-	Linear	Non-linear spring bed	Connectors	-	Normal force

Table 2. Overview of the results for the different cases. (Positive values = Tension)

	Normal force in fasteners [kN]				Largest Mises stress in steel plate [MPa]	Largest contact pressure [MPa]	Total time of analysis [hh:mm:ss]	Gain in time [-]
	B1	B2	B3	B4				
Ref. Case 1	25	25	25	25	280	2	00:40:30	-
Case 1.A	26	26	26	26	246 (-12%)	17 (750%)	00:00:40	60,75
Case 1.B	27	27	27	27	241 (-14%)	8 (300%)	00:00:25	97,20
Case 1.C	26	26	26	26	246 (-12%)	17 (750%)	00:00:20	121,50
Case 1.D	27	27	27	27	241 (-14%)	8 (300%)	00:00:24	101,25
Case 1.E	26	26	26	26	244 (-13%)	14 (600%)	00:00:29	83,79
Case 1.F	26	26	26	26	246 (-12%)	14 (600%)	00:02:22	17,11
Case 1.G	26	26	26	26	243 (-13%)	31 (1450%)	00:06:54	5,87
Case 1.H	27	27	27	27	240 (-14%)	9 (350%)	00:06:59	5,80
Case 1.I	26	26	26	26	246 (-12%)	31 (1450%)	00:52:32	0,77
Case 1.J	27	27	27	27	242 (-14%)	9 (350%)	00:59:45	0,68
Case 1.K	26	26	26	26	243 (-13%)	31 (1450%)	00:06:58	5,81
Case 1.L	27	27	27	27	240 (-14%)	9 (350%)	00:07:05	5,72
Case 1.M	26	26	26	26	246 (-12%)	31 (1450%)	00:55:44	0,73
Case 1.N	27	27	27	27	242 (-14%)	9 (350%)	00:59:07	0,69
Case 1.O	27	27	27	27	122 (-56%)	6 (200%)	00:00:15	162,00
Case 1.P	27	27	27	27	122 (-56%)	6 (200%)	00:01:56	20,95
Case 1.Q	27	27	27	27	122 (-56%)	6 (200%)	00:00:15	162,00
Case 1.R	27	27	27	27	122 (-56%)	6 (200%)	00:01:55	21,13
Case 1.S	26	26	26	26	282 (1%)	6 (200%)	00:08:11	4,95
Case 1.T	26	26	26	26	283 (1%)	6 (200%)	01:06:13	0,61
Case 1.U	26	26	26	26	247 (-12%)	-	00:00:15	162,00

Table 3. Overview of parametric study. Different cases with varying parameters are defined.

	Element type		Element meshing size		Material definition		Anchor plate fasteners	Contact definition	Load definition
	Steel Plate	Concrete	Steel Plate	Concrete	Steel Plate	Concrete			
Ref. Case 2	Solid	Solid	0.010 m	0.010 m	Linear	Non-Linear	Connectors	Hard	Bending M_x
Case 2.A	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Connectors	Hard/Exponential	Bending M_x
Case 2.B	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Connectors	Soft/Exponential	Bending M_x
Case 2.C	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Springs	Hard/Exponential	Bending M_x
Case 2.D	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Springs	Soft/Exponential	Bending M_x
Case 2.E	Shell	Solid	0,010 m	0,025 m	Linear	Linear	Connectors	Hard/Exponential	Bending M_x
Case 2.F	Shell	Solid	0,010 m	0,025 m	Linear	Non-linear	Connectors	Hard/Exponential	Bending M_x
Case 2.G	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Connectors	Hard/Exponential	Bending M_x
Case 2.H	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Connectors	Soft/Exponential	Bending M_x
Case 2.I	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Connectors	Hard/Exponential	Bending M_x
Case 2.J	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Connectors	Soft/Exponential	Bending M_x
Case 2.K	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Springs	Hard/Exponential	Bending M_x
Case 2.L	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Springs	Soft/Exponential	Bending M_x
Case 2.M	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Springs	Hard/Exponential	Bending M_x
Case 2.N	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Springs	Soft/Exponential	Bending M_x
Case 2.O	Solid	Solid	0,025 m	0,025 m	Linear	Linear	Connectors	Soft/Exponential	Bending M_x
Case 2.P	Solid	Solid	0,025 m	0,025 m	Linear	Non-linear	Connectors	Soft/Exponential	Bending M_x
Case 2.Q	Solid	Solid	0,025 m	0,025 m	Linear	Linear	Springs	Soft/Exponential	Bending M_x
Case 2.R	Solid	Solid	0,025 m	0,025 m	Linear	Non-linear	Springs	Soft/Exponential	Bending M_x
Case 2.S	Solid	Solid	0,010 m	0,010 m	Linear	Linear	Connectors	Soft/Exponential	Bending M_x
Case 2.T	Solid	Solid	0,010 m	0,010 m	Linear	Non-linear	Connectors	Soft/Exponential	Bending M_x
Case 2.U	Shell	-	0,010 m	-	Linear	Non-linear	Connectors	-	Bending M_x

Table 4. Overview of the results for the different cases. (Positive values = Tension)

	Normal force in fasteners [kN]				Largest Mises stress in steel plate [MPa]	Largest contact pressure [MPa]	Total time of analysis [hh:mm:ss]	Gain in time [-]
	B1	B2	B3	B4				
Ref. Case 2	246	0	0	246	2469	50	01:33:59	-
Case 2.A	247	0	0	247	2711 (10%)	60 (20%)	00:00:33	170,88
Case 2.B	247	0	0	247	2711 (10%)	53 (6%)	00:00:27	208,85
Case 2.C	247	-2	-2	247	2711 (10%)	60 (20%)	00:00:30	187,97
Case 2.D	247	-3	-3	247	2712 (10%)	53 (6%)	00:00:27	208,85
Case 2.E	247	0	0	247	2711 (10%)	71 (42%)	00:00:30	187,97
Case 2.F	246	0	0	246	2731 (11%)	47 (-6%)	00:03:54	24,10
Case 2.G	248	0	0	248	2714 (10%)	88 (76%)	00:10:20	9,10
Case 2.H	247	0	0	247	2714 (10%)	67 (34%)	00:09:16	10,14
Case 2.I	247	0	0	247	2737 (11%)	49 (-2%)	01:30:10	1,04
Case 2.J	247	0	0	247	2735 (11%)	48 (-4%)	01:23:54	1,12
Case 2.K	248	0	0	248	2714 (10%)	88 (76%)	00:09:26	9,96
Case 2.L	247	-1	-1	247	2714 (10%)	67 (34%)	00:09:22	10,03
Case 2.M	247	-1	-1	247	2737 (11%)	49 (-2%)	01:31:34	1,03
Case 2.N	247	-2	-2	247	2735 (11%)	48 (-4%)	01:23:14	1,13
Case 2.O	250	0	0	250	1296 (-48%)	58 (16%)	00:00:22	256,32
Case 2.P	250	0	0	250	1300 (-47%)	44 (-12%)	00:03:05	30,48
Case 2.Q	251	-7	-7	251	1296 (-48%)	58 (16%)	00:00:24	234,96
Case 2.R	250	-8	-8	250	1300 (-47%)	44 (-12%)	00:03:19	28,34
Case 2.S	246	0	0	246	2465 (0%)	68 (36%)	00:12:57	7,26
Case 2.T	245	0	0	245	2465 (0%)	49 (-2%)	01:41:10	0,93
Case 2.U	248	2	2	248	2731 (11%)	-	00:00:19	296,79

Table 5. Overview of parametric study. Different cases with varying parameters are defined.

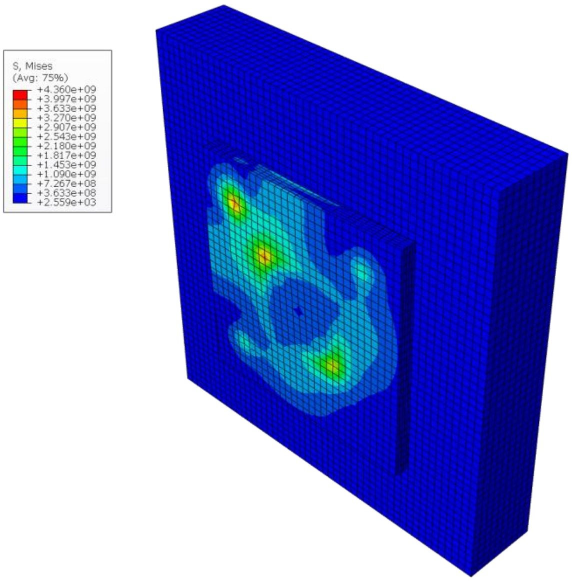
	Element type		Element meshing size		Material definition		Anchor plate fasteners	Contact definition	Load definition
	Steel Plate	Concrete	Steel Plate	Concrete	Steel Plate	Concrete			
Ref. Case 3	Solid	Solid	0.010 m	0.010 m	Linear	Non-Linear	Connectors	Hard	Bending M_x, M_y
Case 3.A	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Connectors	Hard/Exponential	Bending M_x, M_y
Case 3.B	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Connectors	Soft/Exponential	Bending M_x, M_y
Case 3.C	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Springs	Hard/Exponential	Bending M_x, M_y
Case 3.D	Shell	Shell	0,010 m	0,010 m	Linear	Spring bed	Springs	Soft/Exponential	Bending M_x, M_y
Case 3.E	Shell	Solid	0,010 m	0,025 m	Linear	Linear	Connectors	Hard/Exponential	Bending M_x, M_y
Case 3.F	Shell	Solid	0,010 m	0,025 m	Linear	Non-linear	Connectors	Hard/Exponential	Bending M_x, M_y
Case 3.G	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Connectors	Hard/Exponential	Bending M_x, M_y
Case 3.H	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Connectors	Soft/Exponential	Bending M_x, M_y
Case 3.I	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Connectors	Hard/Exponential	Bending M_x, M_y
Case 3.J	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Connectors	Soft/Exponential	Bending M_x, M_y
Case 3.K	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Springs	Hard/Exponential	Bending M_x, M_y
Case 3.L	Shell	Solid	0,010 m	0,010 m	Linear	Linear	Springs	Soft/Exponential	Bending M_x, M_y
Case 3.M	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Springs	Hard/Exponential	Bending M_x, M_y
Case 3.N	Shell	Solid	0,010 m	0,010 m	Linear	Non-linear	Springs	Soft/Exponential	Bending M_x, M_y
Case 3.O	Solid	Solid	0,025 m	0,025 m	Linear	Linear	Connectors	Soft/Exponential	Bending M_x, M_y
Case 3.P	Solid	Solid	0,025 m	0,025 m	Linear	Non-linear	Connectors	Soft/Exponential	Bending M_x, M_y
Case 3.Q	Solid	Solid	0,025 m	0,025 m	Linear	Linear	Springs	Soft/Exponential	Bending M_x, M_y
Case 3.R	Solid	Solid	0,025 m	0,025 m	Linear	Non-linear	Springs	Soft/Exponential	Bending M_x, M_y
Case 3.S	Solid	Solid	0,010 m	0,010 m	Linear	Linear	Connectors	Soft/Exponential	Bending M_x, M_y
Case 3.T	Solid	Solid	0,010 m	0,010 m	Linear	Non-linear	Connectors	Soft/Exponential	Bending M_x, M_y
Case 3.U	Shell	-	0,010 m	-	Linear	Non-linear	Connectors	-	Bending M_x, M_y

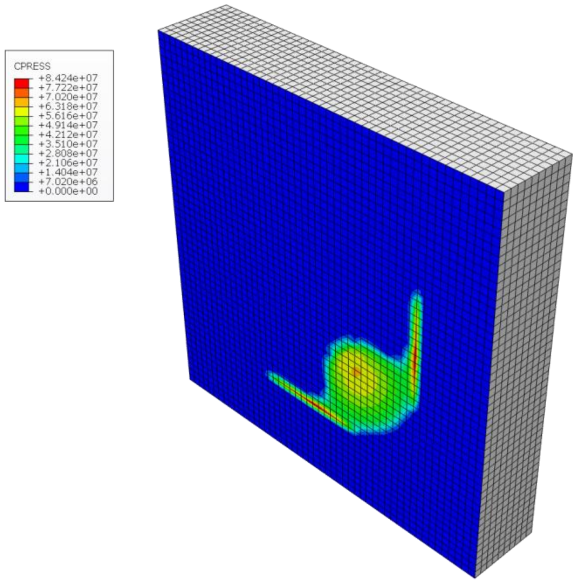
Table 6. Overview of the results for the different cases. (Positive values = Tension)

	Normal force in fasteners [kN]				Largest Mises stress in steel plate [MPa]	Largest contact pressure [MPa]	Total time of analysis [hh:mm:ss]	Gain in time [-]
	B1	B2	B3	B4				
Ref. Case 3	148	0	148	449	4360	84	01:35:05	-
Case 3.A	159	0	159	449	4787 (10%)	127 (51%)	00:00:27	211,30
Case 3.B	156	0	156	450	4801 (10%)	115 (37%)	00:00:34	167,79
Case 3.C	159	-15	159	449	4785 (10%)	127 (51%)	00:00:26	219,42
Case 3.D	156	-21	156	450	4798 (10%)	114 (36%)	00:00:26	219,42
Case 3.E	167	0	167	448	4802 (10%)	173 (106%)	00:01:27	65,57
Case 3.F	158	0	158	448	4840 (11%)	195 (132%)	00:03:36	26,41
Case 3.G	167	0	167	448	4774 (9%)	244 (190%)	00:11:10	8,51
Case 3.H	161	0	161	449	4787 (10%)	190 (126%)	00:11:25	8,33
Case 3.I	158	0	158	449	4846 (11%)	76 (-10%)	01:39:21	0,96
Case 3.J	155	0	155	450	4856 (11%)	64 (-24%)	01:33:57	1,01
Case 3.K	167	-13	167	448	4773 (9%)	244 (190%)	00:11:17	8,43
Case 3.L	161	-18	161	449	4785 (10%)	190 (126%)	00:11:14	8,46
Case 3.M	158	-25	158	448	4841 (11%)	77 (-8%)	01:45:04	0,90
Case 3.N	155	-29	155	449	4849 (11%)	64 (-24%)	01:23:25	1,14
Case 3.O	146	0	146	455	2230 (-49%)	120 (43%)	00:00:27	211,30
Case 3.P	140	0	140	455	2244 (-49%)	60 (-29%)	00:03:41	25,81
Case 3.Q	146	-22	146	455	2228 (-49%)	119 (42%)	00:00:24	237,71
Case 3.R	140	-33	140	454	2240 (-49%)	60 (-29%)	00:02:52	33,17
Case 3.S	152	0	152	450	4382 (1%)	190 (126%)	00:13:22	7,11
Case 3.T	146	0	146	449	4366 (0%)	70 (-17%)	01:53:03	0,84
Case 3.U	157	0	157	449	4820 (11%)	-	00:00:20	285,25

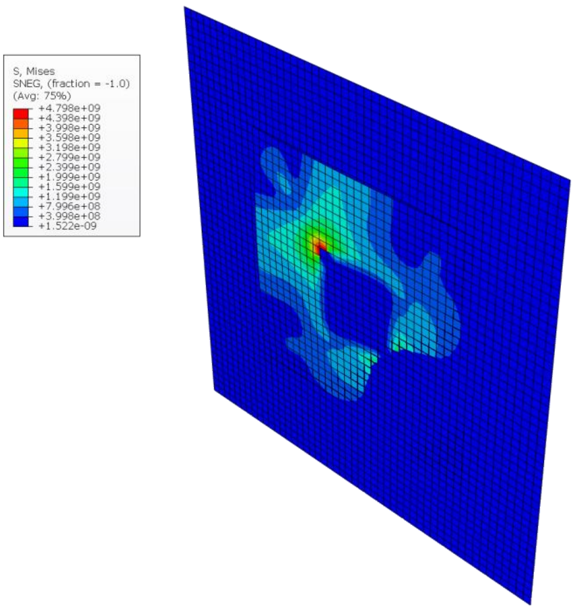
Appendix 2 - Parametric study - Figures

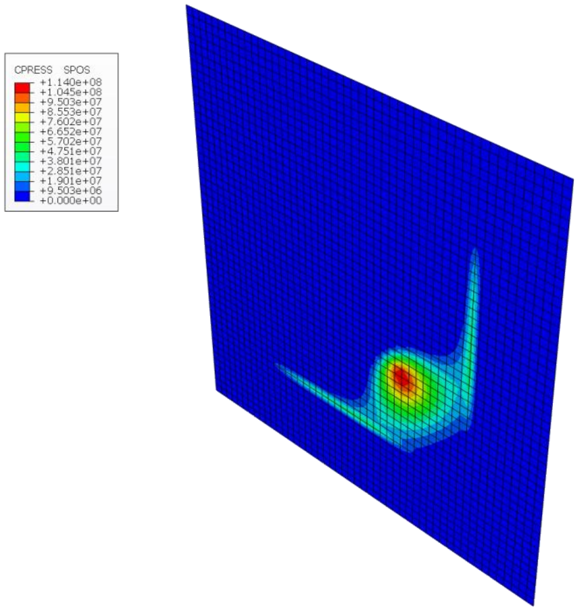
Reference case 3



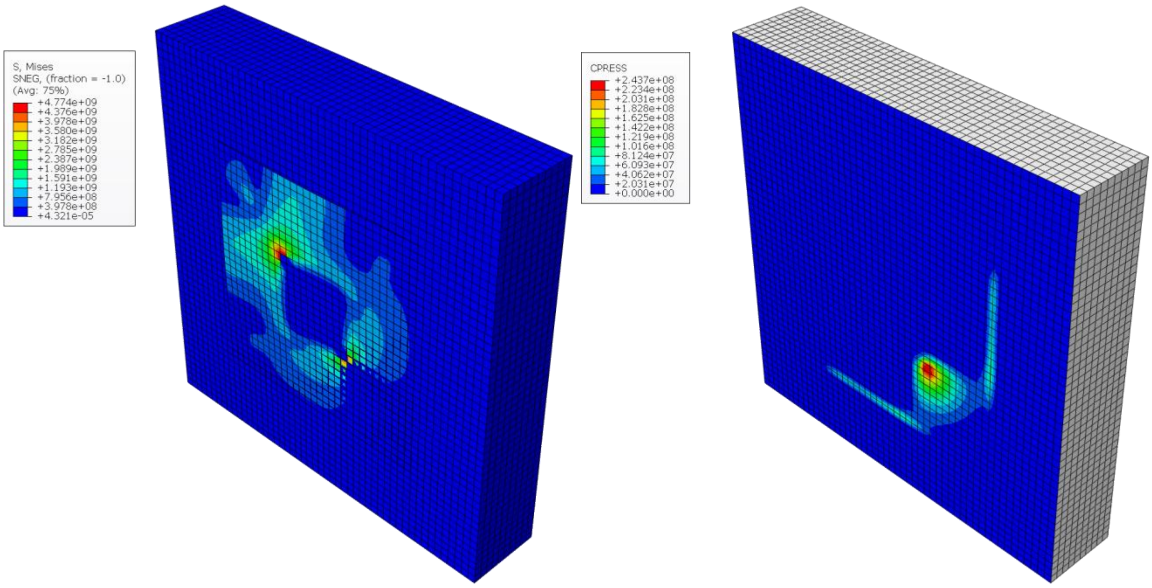


Case 3D

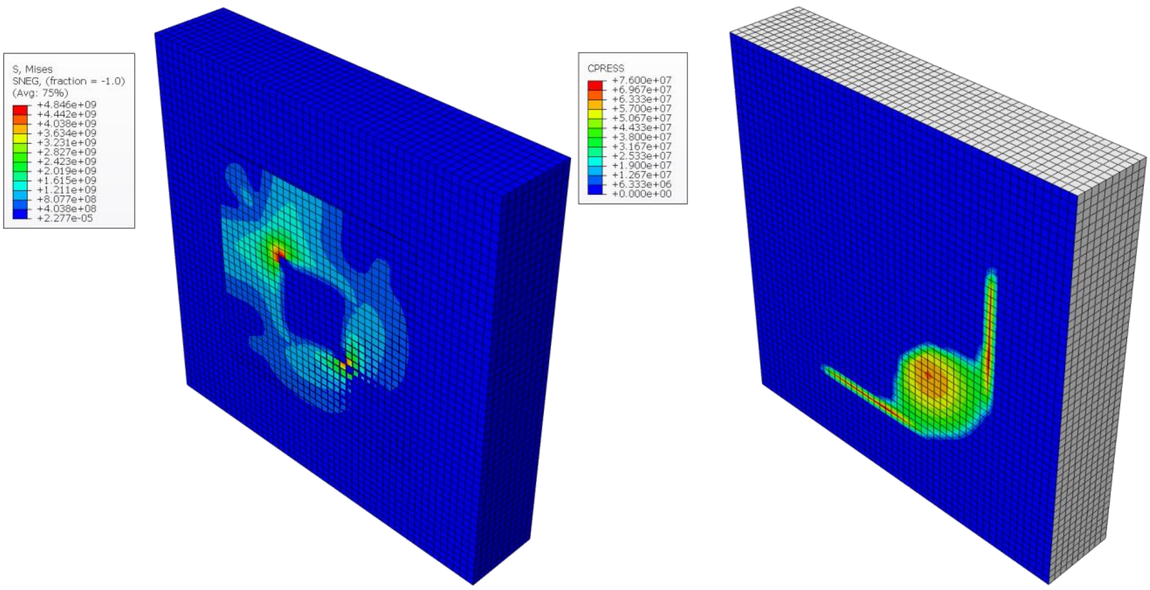




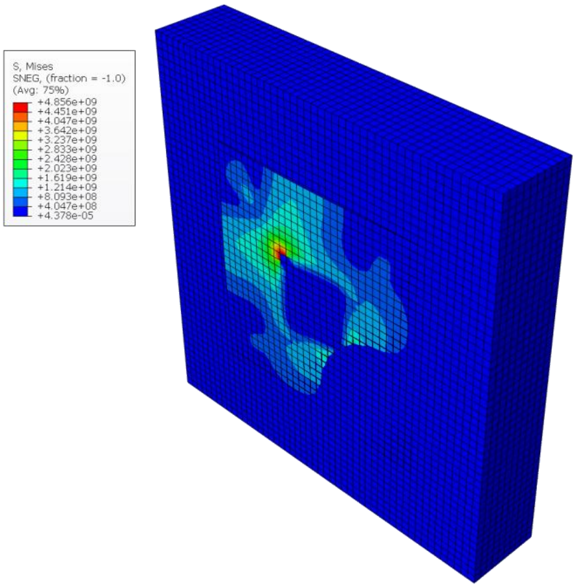
Case 3G

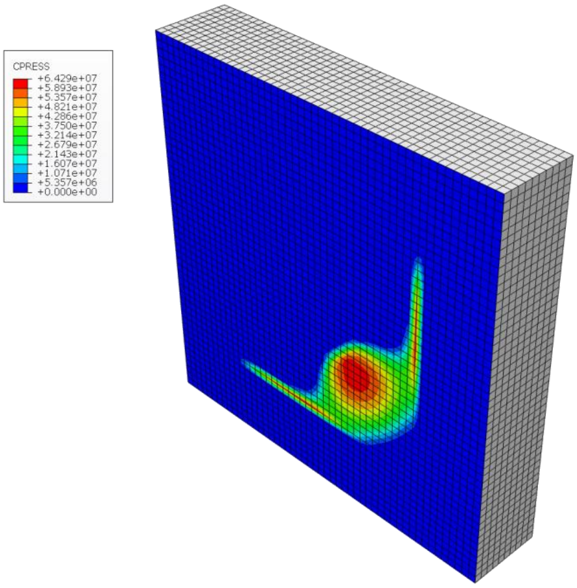


Case 3I

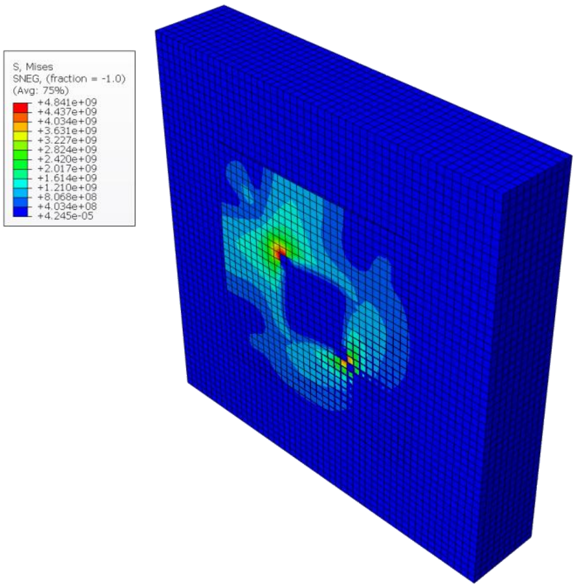


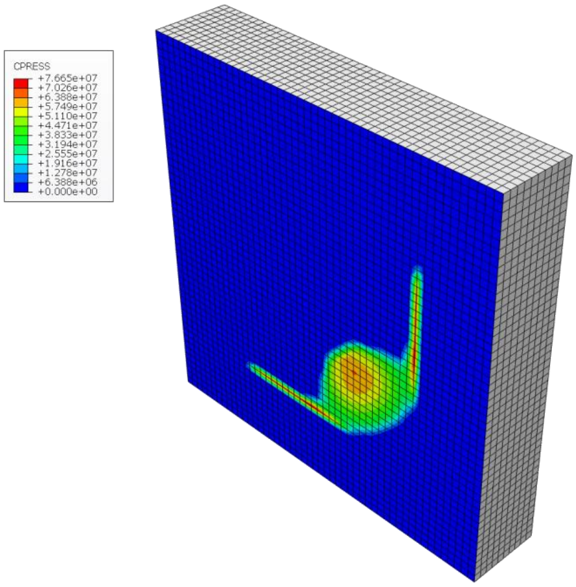
Case 3J



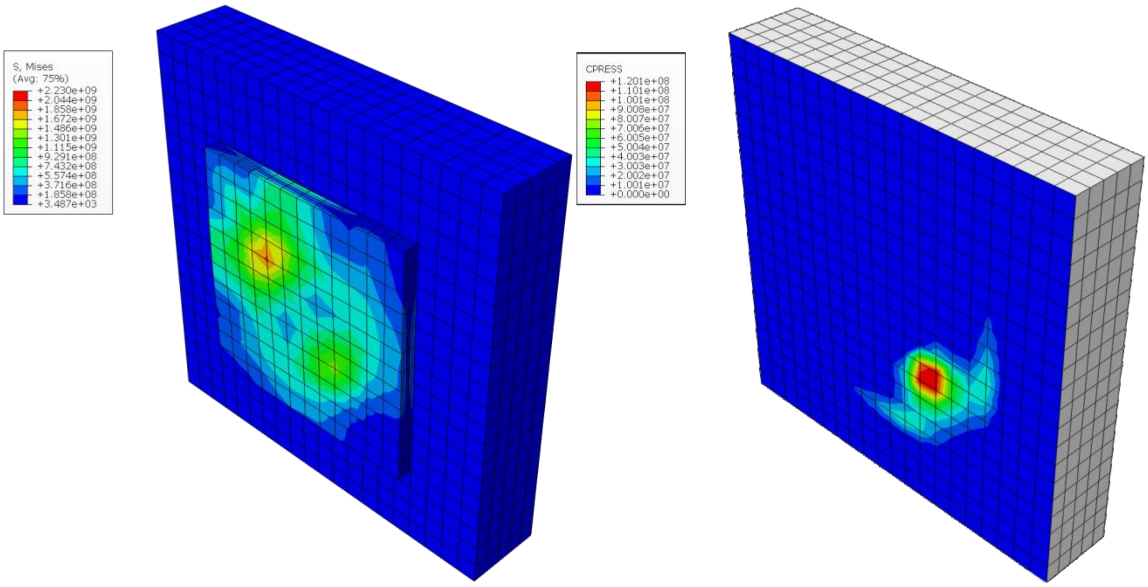


Case 3M

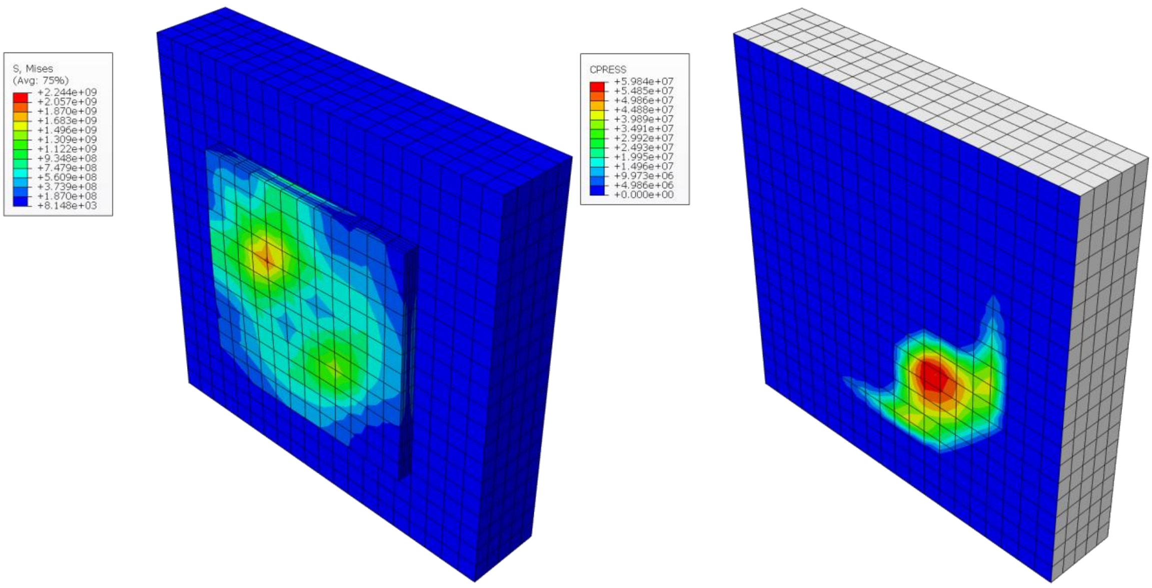




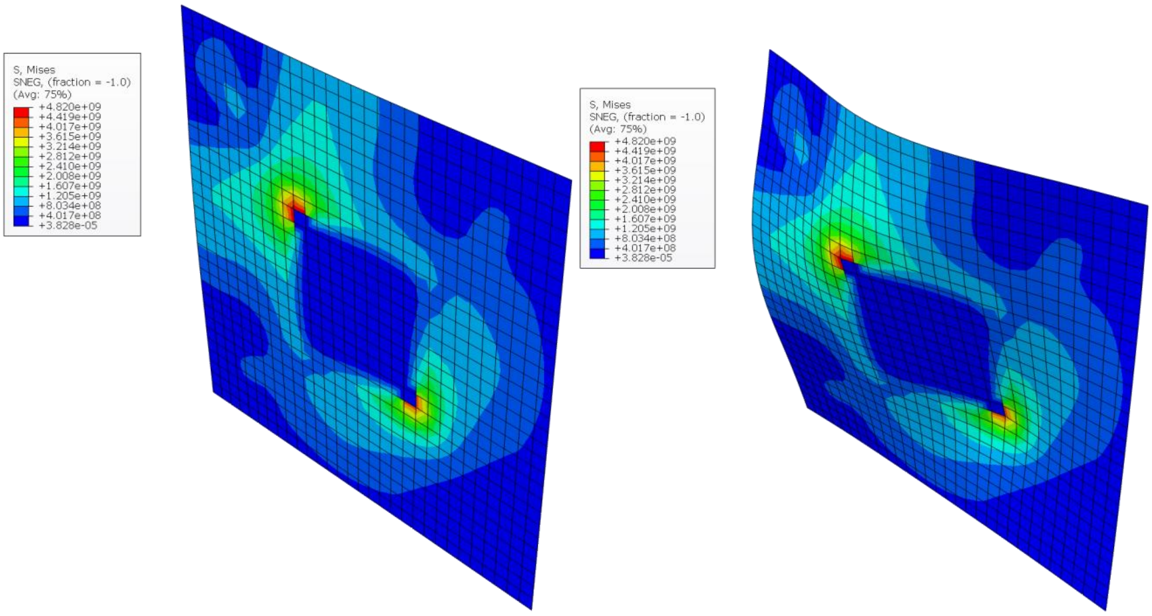
Case 3O



Case 3P



Case 3U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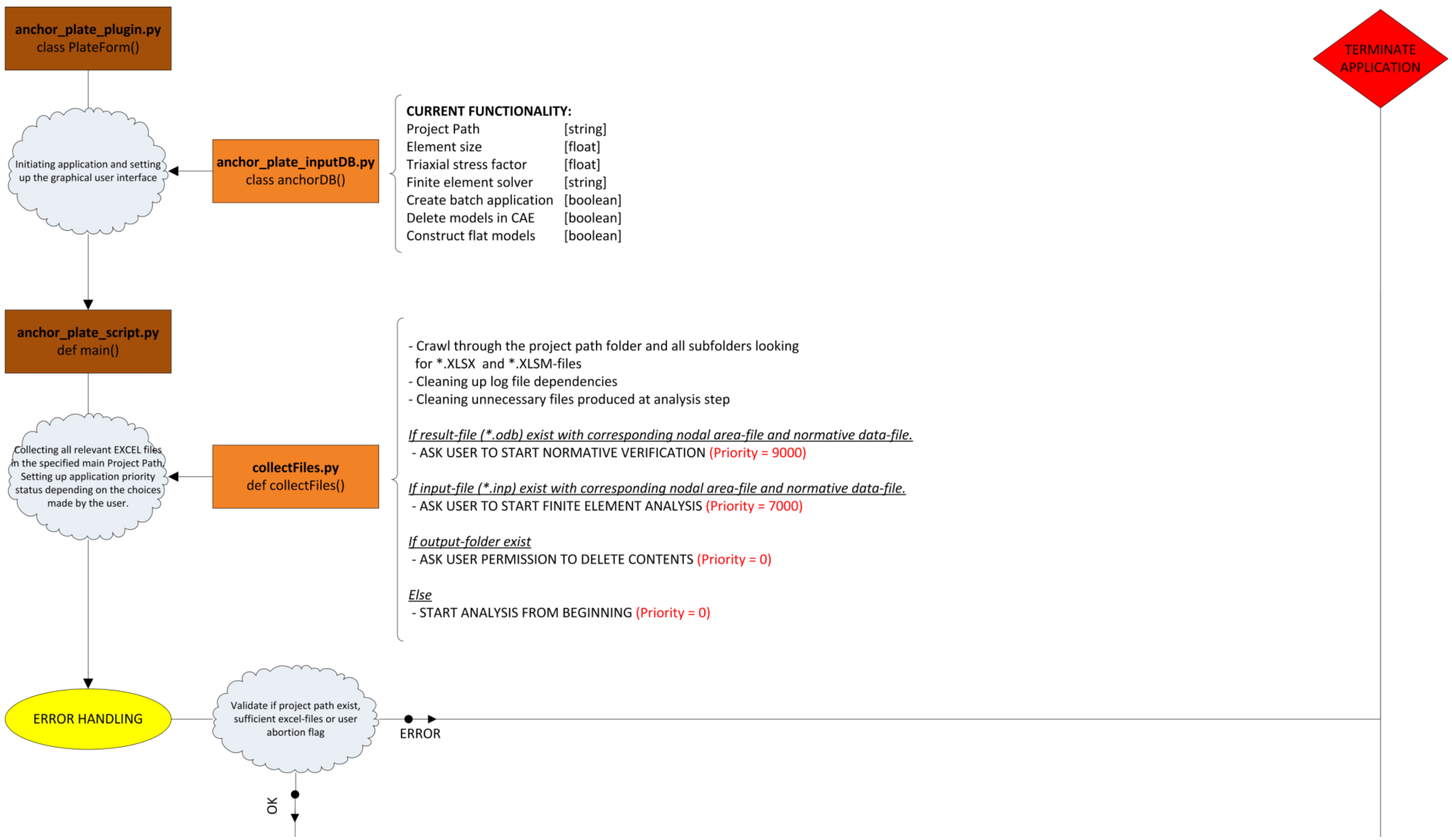
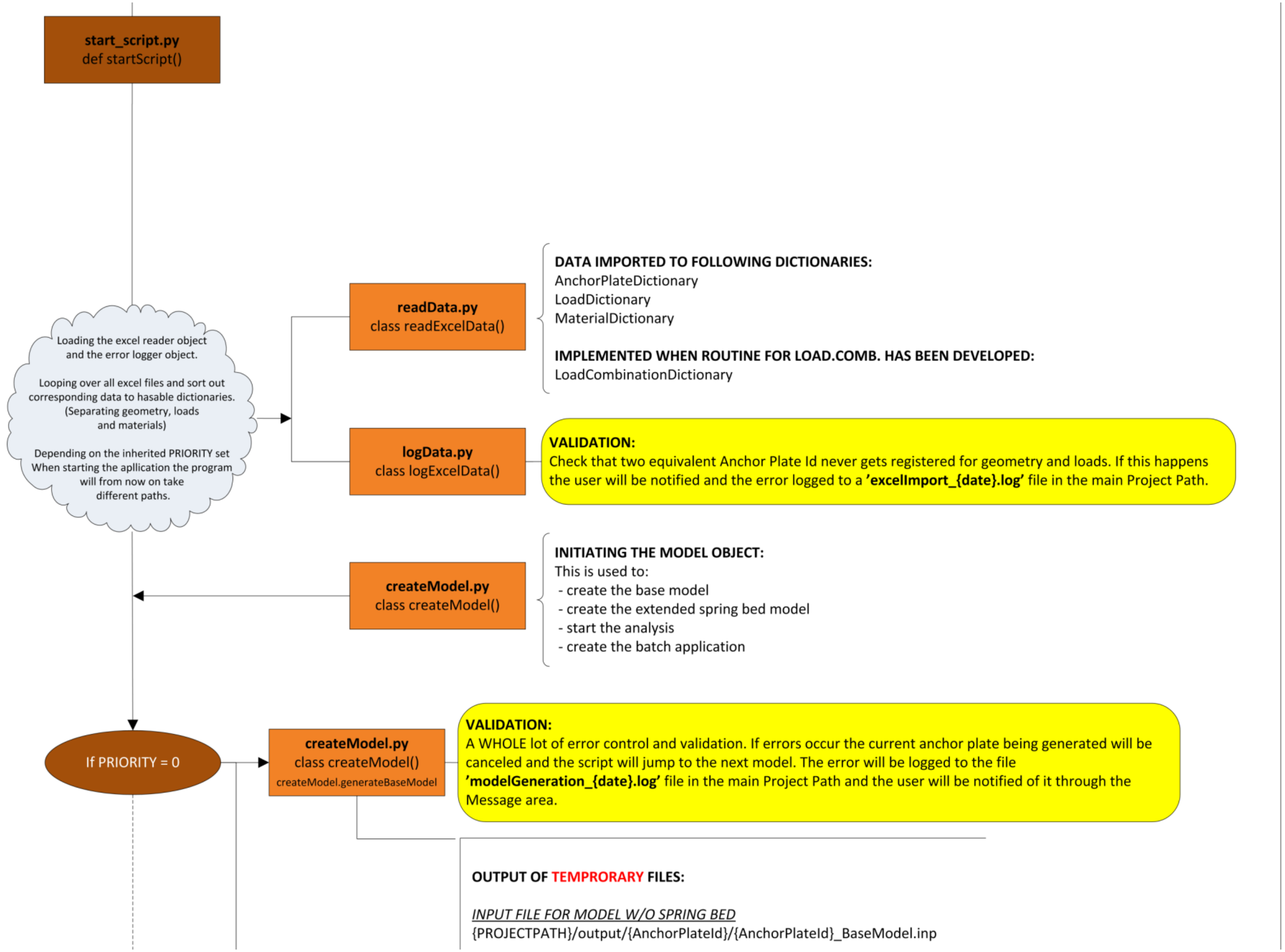
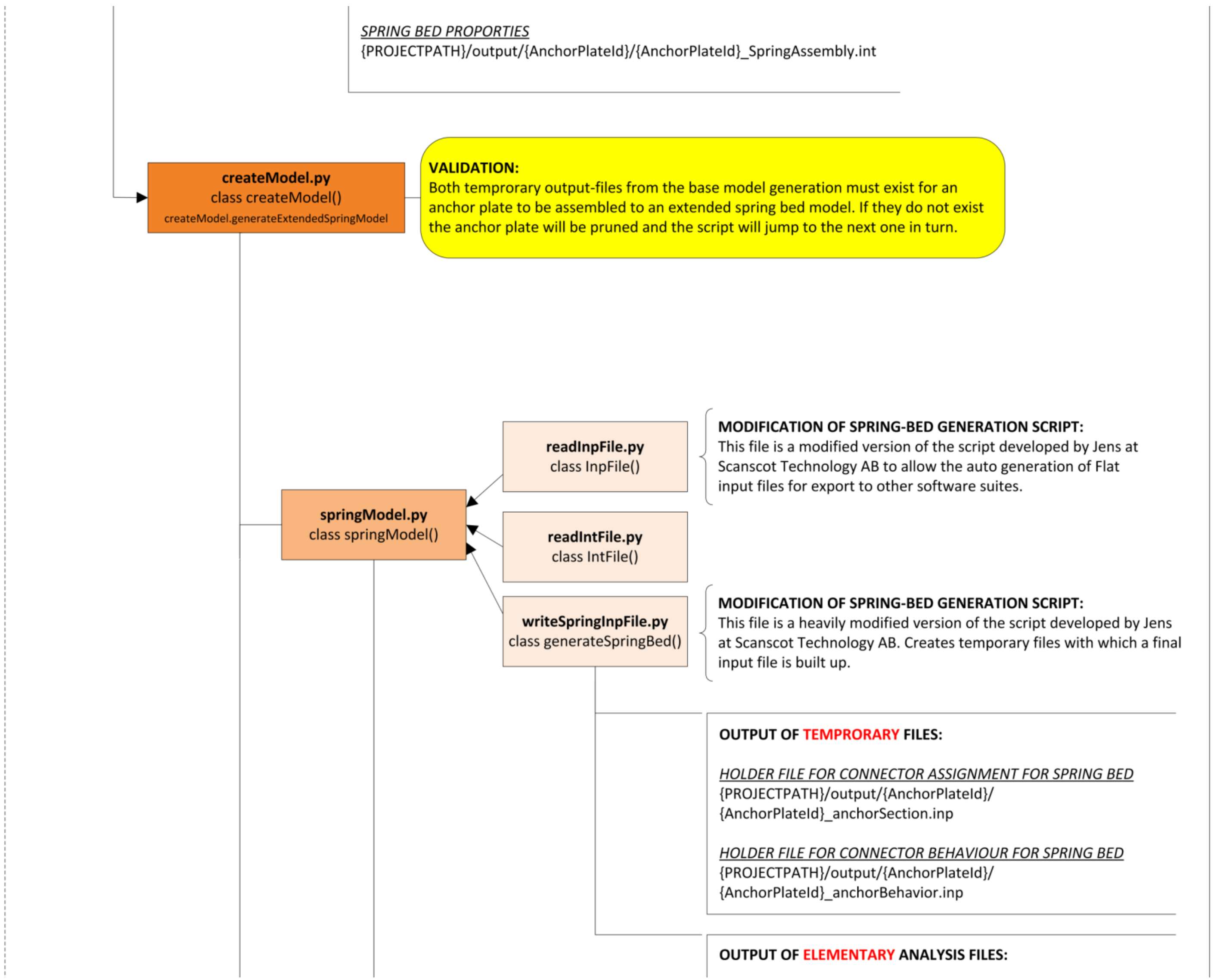
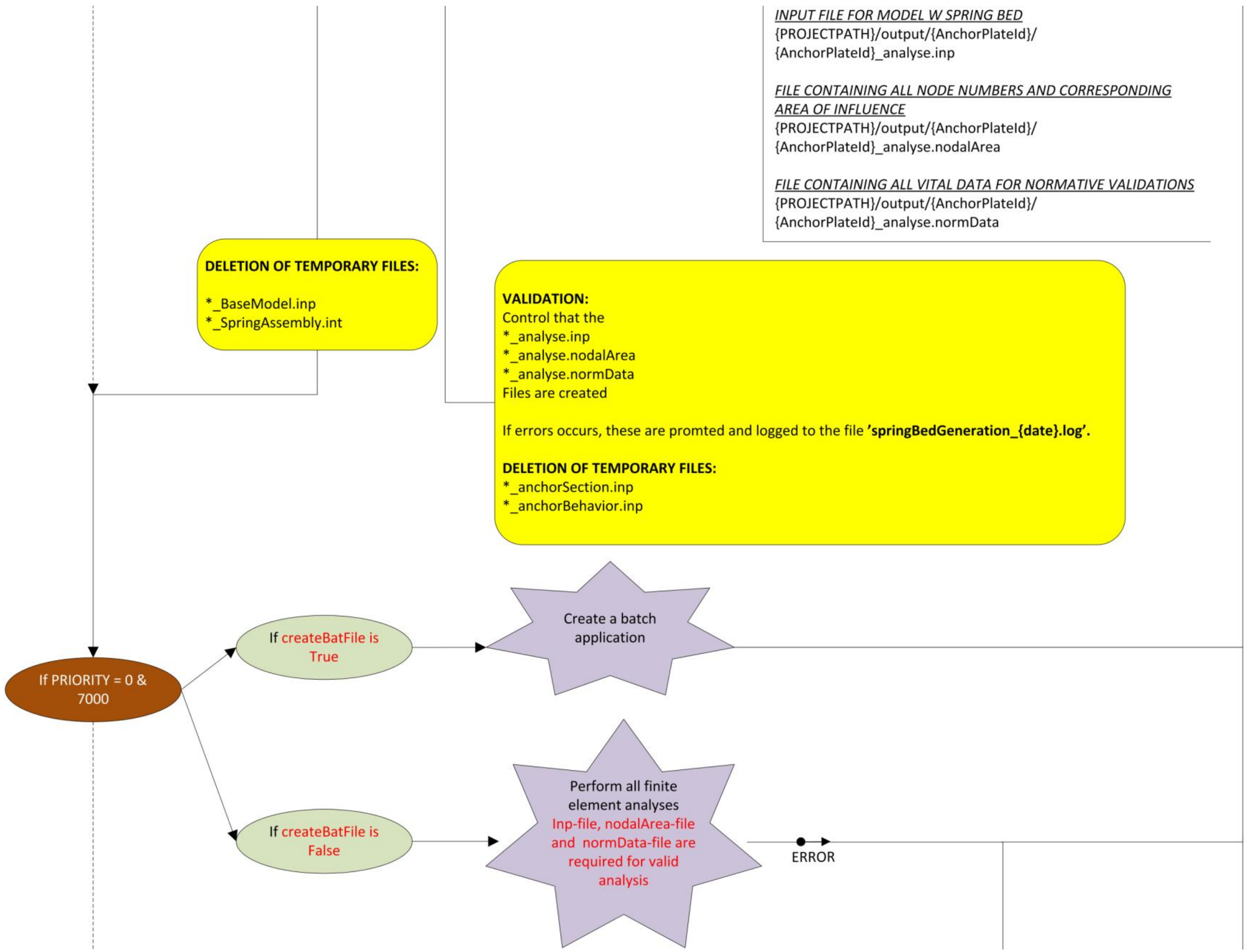
(Avg: 75%)
+4.820e+09
+4.419e+09
+4.017e+09
+3.615e+09
+3.214e+09
+2.812e+09
+2.410e+09
+2.008e+09
+1.607e+09
+1.205e+09
+8.034e+08
+4.017e+08
+3.828e-05
S, Mises
SNEG, (fraction = -1.0)
(Avg: 75%)
+4.820e+09
+4.419e+09
+4.017e+09
+3.615e+09
+3.214e+09
+2.812e+09
+2.410e+09
+2.008e+09
+1.607e+09
+1.205e+09
+8.034e+08
+4.017e+08
+3.828e-05

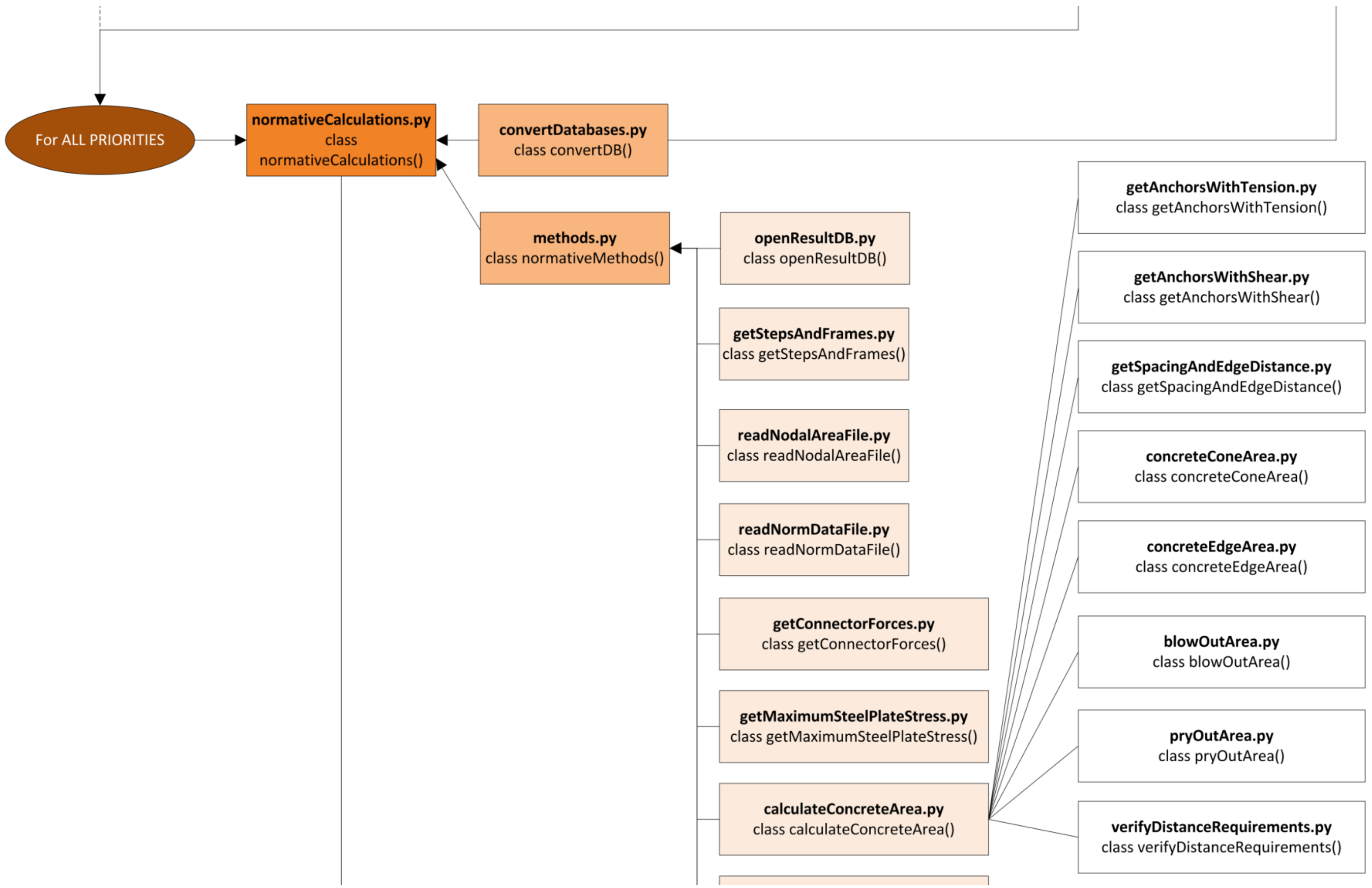
Appendix 3 - Application flow chart

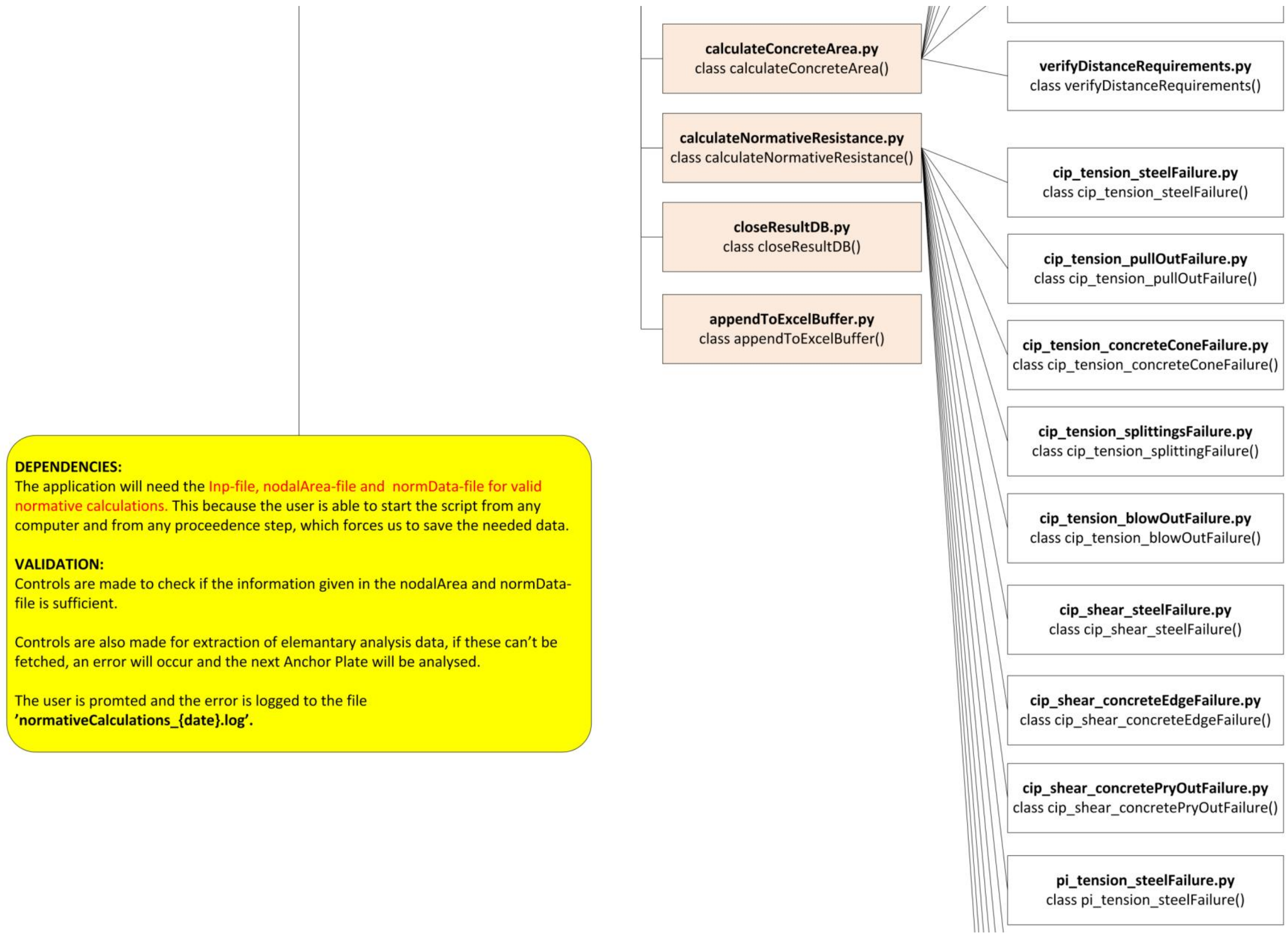



start_script.py
def startScript()
Loading the excel reader object and the error logger object.
Looping over all excel files and sort out corresponding data to hasable dictionaries. (Separating geometry, loads and materials)
Depending on the inherited PRIORITY set When starting the apllication the program will from now on take different paths.
readData.py
class readExcelData()
DATA IMPORTED TO FOLLOWING DICTIONARIES:
AnchorPlateDictionary
LoadDictionary
MaterialDictionary
IMPLEMENTED WHEN ROUTINE FOR LOAD.COMB. HAS BEEN DEVELOPED:
LoadCombinationDictionary
logData.py
class logExcelData()
VALIDATION:
Check that two equivalent Anchor Plate Id never gets registered for geometry and loads. If this happens the user will be notified and the error logged to a 'excelImport_{date}.log' file in the main Project Path.
createModel.py
class createModel()
INITIATING THE MODEL OBJECT:
This is used to:
- create the base model
- create the extended spring bed model
- start the analysis
- create the batch application
If PRIORITY = 0
createModel.py
class createModel()
createModel.generateBaseModel
VALIDATION:
A WHOLE lot of error control and validation. If errors occur the current anchor plate being generated will be canceled and the script will jump to the next model. The error will be logged to the file 'modelGeneration_{date}.log' file in the main Project Path and the user will be notified of it through the Message area.
OUTPUT OF TEMPRORARY FILES:
INPUT FILE FOR MODEL W/O SPRING BED
{PROJECTPATH}/output/{AnchorPlateId}/{AnchorPlateId}_BaseModel.inp

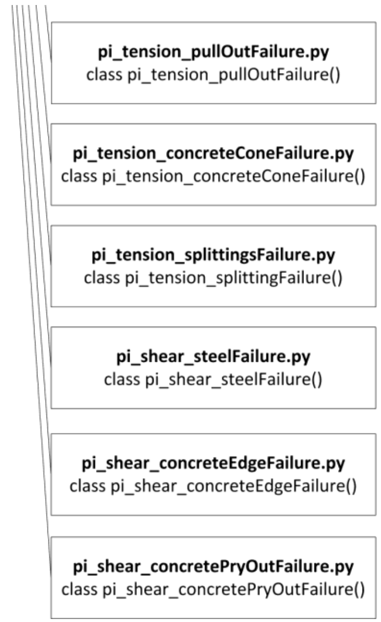

SPRING BED PROPORTIES
{PROJECTPATH}/output/{AnchorPlateId}/{AnchorPlateId}_SpringAssembly.int
createModel.py
class createModel()
createModel.generateExtendedSpringModel
VALIDATION:
Both temprorary output-files from the base model generation must exist for an anchor plate to be assembled to an extended spring bed model. If they do not exist the anchor plate will be pruned and the script will jump to the next one in turn.
springModel.py
class springModel()
readInpFile.py
class InpFile()
readIntFile.py
class IntFile()
writeSpringInpFile.py
class generateSpringBed()
MODIFICATION OF SPRING-BED GENERATION SCRIPT:
This file is a modified version of the script developed by Jens at Scanscot Technology AB to allow the auto generation of Flat input files for export to other software suites.
MODIFICATION OF SPRING-BED GENERATION SCRIPT:
This file is a heavily modified version of the script developed by Jens at Scanscot Technology AB. Creates temporary files with which a final input file is built up.
OUTPUT OF TEMPRORARY FILES:
HOLDER FILE FOR CONNECTOR ASSIGNMENT FOR SPRING BED
{PROJECTPATH}/output/{AnchorPlateId}/
{AnchorPlateId}_anchorSection.inp
HOLDER FILE FOR CONNECTOR BEHAVIOUR FOR SPRING BED
{PROJECTPATH}/output/{AnchorPlateId}/
{AnchorPlateId}_anchorBehavior.inp
OUTPUT OF ELEMENTARY ANALYSIS FILES:


INPUT FILE FOR MODEL W SPRING BED
{PROJECTPATH}/output/{AnchorPlateId}/
{AnchorPlateId}_analyse.inp
FILE CONTAINING ALL NODE NUMBERS AND CORRESPONDING AREA OF INFLUENCE
{PROJECTPATH}/output/{AnchorPlateId}/
{AnchorPlateId}_analyse.nodalArea
FILE CONTAINING ALL VITAL DATA FOR NORMATIVE VALIDATIONS
{PROJECTPATH}/output/{AnchorPlateId}/
{AnchorPlateId}_analyse.normData
DELETION OF TEMPORARY FILES:
*_BaseModel.inp
*_SpringAssembly.int
VALIDATION:
Control that the
*_analyse.inp
*_analyse.nodalArea
*_analyse.normData
Files are created
If errors occurs, these are promted and logged to the file 'springBedGeneration_{date}.log'.
DELETION OF TEMPORARY FILES:
*_anchorSection.inp
*_anchorBehavior.inp
If PRIORITY = 0 & 7000
If createBatFile is True
Create a batch application
If createBatFile is False
Perform all finite element analyses
Inp-file, nodalArea-file and normData-file are required for valid analysis
ERROR


For ALL PRIORITIES
normativeCalculations.py
class
normativeCalculations()
convertDatabases.py
class convertDB()
methods.py
class normativeMethods()
openResultDB.py
class openResultDB()
getStepsAndFrames.py
class getStepsAndFrames()
readNodalAreaFile.py
class readNodalAreaFile()
readNormDataFile.py
class readNormDataFile()
getConnectorForces.py
class getConnectorForces()
getMaximumSteelPlateStress.py
class getMaximumSteelPlateStress()
calculateConcreteArea.py
class calculateConcreteArea()
getAnchorsWithTension.py
class getAnchorsWithTension()
getAnchorsWithShear.py
class getAnchorsWithShear()
getSpacingAndEdgeDistance.py
class getSpacingAndEdgeDistance()
concreteConeArea.py
class concreteConeArea()
concreteEdgeArea.py
class concreteEdgeArea()
blowOutArea.py
class blowOutArea()
pryOutArea.py
class pryOutArea()
verifyDistanceRequirements.py
class verifyDistanceRequirements()



calculateConcreteArea.py
class calculateConcreteArea()
calculateNormativeResistance.py
class calculateNormativeResistance()
closeResultDB.py
class closeResultDB()
appendToExcelBuffer.py
class appendToExcelBuffer()
verifyDistanceRequirements.py
class verifyDistanceRequirements()
cip_tension_steelFailure.py
class cip_tension_steelFailure()
cip_tension_pullOutFailure.py
class cip_tension_pullOutFailure()
cip_tension_concreteConeFailure.py
class cip_tension_concreteConeFailure()
cip_tension_splittingsFailure.py
class cip_tension_splittingFailure()
cip_tension_blowOutFailure.py
class cip_tension_blowOutFailure()
cip_shear_steelFailure.py
class cip_shear_steelFailure()
cip_shear_concreteEdgeFailure.py
class cip_shear_concreteEdgeFailure()
cip_shear_concretePryOutFailure.py
class cip_shear_concretePryOutFailure()
pi_tension_steelFailure.py
class pi_tension_steelFailure()
DEPENDENCIES:
The application will need the Inp-file, nodalArea-file and normData-file for valid normative calculations. This because the user is able to start the script from any computer and from any proceedence step, which forces us to save the needed data.
VALIDATION:
Controls are made to check if the information given in the nodalArea and normData-file is sufficient.
Controls are also made for extraction of elemantary analysis data, if these can't be fetched, an error will occur and the next Anchor Plate will be analysed.
The user is promted and the error is logged to the file 'normativeCalculations_{date}.log'.

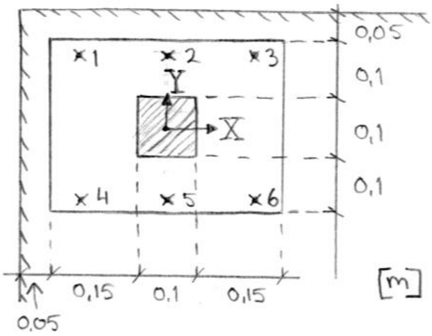

pi_tension_pullOutFailure.py
class pi_tension_pullOutFailure()
pi_tension_concreteConeFailure.py
class pi_tension_concreteConeFailure()
pi_tension_splittingsFailure.py
class pi_tension_splittingFailure()
pi_shear_steelFailure.py
class pi_shear_steelFailure()
pi_shear_concreteEdgeFailure.py
class pi_shear_concreteEdgeFailure()
pi_shear_concretePryOutFailure.py
class pi_shear_concretePryOutFailure()

Appendix 4 - Comparison example

Comparison example

Geometry and loads

$F_z = 30$ kN
$F_y = 10$ kN



Anchor coordinates:

1. (−0,15 ; 0,125)
2. (0 ; 0,125)
3. (0,15 ; 0,125)
4. (−0,15 ; −0,125)
5. (0 ; −0,125)
6. (0,15 ; −0,125)

Steel plate:

width = 0,4 m
height = 0,3 m
thickness = 0,02 m

$E = 210$ GPa
$\rho = 7800$ kg/m^3
$\nu = 0{,}3$
$f_{yk} = 640$ MPa
$f_{uk} = 800$ MPa

Concrete C20/25:

thickness = 0,2 m

$E = 30$ GPa
$\rho = 2500$ kg/m^3
$\nu = 0{,}2$
$f_{ck,cube} = 25$ MPa

Headed fasteners:

$d = 0{,}012$ m
$d_h = 0{,}022$ m
$h_{ef} = 0{,}13$ m

$E = 210$ GPa
$f_{yk} = 640$ MPa
$f_{uk} = 800$ MPa

Verifications for headed fasteners loaded in tension according to CEN/TS 1992-4-2 6.2

Steel failure

$$N_{Ed}^{h} \leq N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}}$$

Most loaded fastener $N_{Ed}^{h} = \frac{F_Z}{6} = \frac{30}{6} = 5 \text{ kN}$

$N_{Rk,s}$ is based on f_{uk} ; $N_{Rk,s} = f_{uk} \cdot A = 800 \cdot 10^6 \cdot \pi \cdot 0{,}006^2 \approx 90{,}5 \text{ kN}$

$\gamma_{Ms} = 1{,}2 \cdot f_{uk} / f_{yk} = 1{,}2 \cdot \frac{800}{640} = 1{,}5$

$\Rightarrow N_{Rd,s} = \frac{90{,}5}{1{,}5} = \underline{\underline{60{,}3 \text{ kN}}} > N_{Ed}^{h} = 5 \text{ kN} \quad \Rightarrow \underline{\underline{\text{OK!}}}$

Pull-out failure

$$N_{Ed}^{h} \leq N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}}$$

$N_{Rk,p} = 6 \cdot A_h \cdot f_{ck,cube} \cdot 1{,}4 \qquad A_h = \frac{\pi}{4}\left(d_h^2 - d^2\right)$

$N_{Rk,p} = 6 \cdot \frac{\pi}{4}\left(0{,}022^2 - 0{,}012^2\right) \cdot 25 \cdot 10^6 \cdot 1{,}4 \approx 56 \text{ kN}$

$\gamma_{Mp} = \gamma_{Mc} = \gamma_c \cdot \gamma_{inst} = 1{,}5 \cdot 1{,}0 = 1{,}5$

$\Rightarrow N_{Rd,p} = \frac{56}{1{,}5} = \underline{\underline{37{,}4 \text{ kN}}} > N_{Ed}^{h} = 5 \text{ kN} \quad \Rightarrow \underline{\underline{\text{OK!}}}$

Concrete cone failure

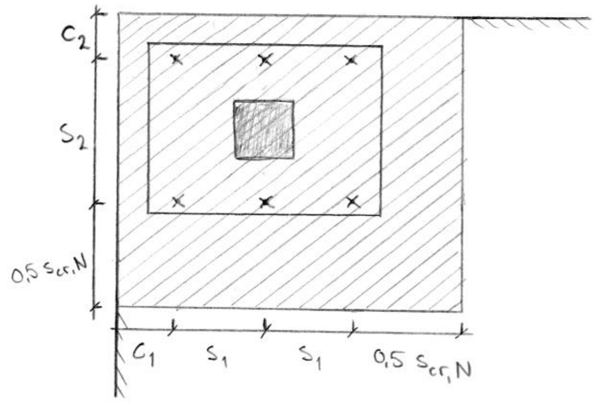
$$N_{Ed}^{g} \leq N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}}$$

Fastener group $N_{Ed}^{g} = F_z = 30\ kN$

$$N_{Rk,c} = N_{Rk,c}^{0} \cdot \frac{A_{c,N}}{A_{c,N}^{0}} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N}$$

$$N_{Rk,c}^{0} = k_{ucr} \cdot \sqrt{f_{ck,cube}} \cdot h_{ef}^{1,5} = 11,9 \cdot \sqrt{25} \cdot 130^{1,5} = 88,2\ kN$$

$$A_{c,N}^{0} = s_{cr,N} \cdot s_{cr,N} = 9 \cdot h_{ef}^{2} = 9 \cdot 0,13^{2} = 0,152\ m^{2}$$



$s_{cr,N} = 3 \cdot h_{ef} = 0,39\ m$

$c_{cr,N} = \frac{3}{2} h_{ef} = 0,195\ m$

$c_1 = 0,1\ m < c_{cr,N}$

$s_1 = 0,15\ m < s_{cr,N}$

$c_2 = 0,075\ m < c_{cr,N}$

$s_2 = 0,25\ m < s_{cr,N}$

$$\Rightarrow A_{c,N} = (c_1 + 2\ s_1 + 0,5\ s_{cr,N}) \cdot (c_2 + s_2 + 0,5\ s_{cr,N}) = 0,31\ m^{2}$$

$$\psi_{s,N} = 0,7 + 0,3 \frac{c_2}{c_{cr,N}} = 0,7 + 0,3\ \frac{0,075}{0,195} = 0,82$$

$$\psi_{re,N} = 0,5 + \frac{h_{ef}}{200} \leq 1 \Rightarrow \psi_{re,N} = 1$$

$\psi_{ec,N} = 1$ (No eccentricity of the load)

$$N_{Rk,c} = 88,2 \cdot \frac{0,31}{0,152} \cdot 0,82 \cdot 1 \cdot 1 = 147,5\ kN \qquad \gamma_{Mc} = 1,5$$

$$\Rightarrow N_{Rd,c} = \frac{147,5}{1,5} = \underline{98,3\ kN} > N_{Ed}^{g} = 30\ kN \Rightarrow \underline{OK!}$$

Splitting failure

$$N_{Ed}^{g} \leq N_{Rd,sp} = \frac{N_{Rk,sp}}{\gamma_{Msp}}$$

$$N_{Rk,sp} = N_{Rk}^{0} \cdot \frac{A_{c,N}}{A_{c,N}^{0}} \cdot \psi_{s,N} \cdot \psi_{ec,N} \cdot \psi_{re,N} \cdot \psi_{h,sp}$$

$$N_{Rk}^{0} = \min\left(N_{Rk,p} \, ; \, N_{Rk,c}^{0}\right) = 56 \text{ kN}$$

$$\psi_{h,sp} = \left(\frac{h}{h_{min}}\right)^{2/3} \leq \left(\frac{2 \cdot h_{ef}}{h_{min}}\right)^{2/3} \Rightarrow \psi_{h,sp} = \left(\frac{0{,}2}{0{,}1}\right)^{2/3} = 1{,}89$$

with $c_{cr,sp} = c_{cr,N}$ and $s_{cr,sp} = s_{cr,N}$ the other factors will not change.

$$N_{Rk,sp} = 56 \cdot \frac{0{,}31}{0{,}152} \cdot 0{,}82 \cdot 1 \cdot 1 \cdot 1{,}89 = 177 \text{ kN}$$

$$\gamma_{Msp} = \gamma_{Mc} = 1{,}5$$

$$\Rightarrow N_{Rd,sp} = \frac{177}{1{,}5} = \underline{\underline{118 \text{ kN}}} > N_{Ed}^{g} = 30 \text{ kN} \Rightarrow \underline{\underline{OK!}}$$

Blow-out failure

Verification is not required if the edge distance in all directions exceeds $c = 0{,}5 \cdot h_{ef} = 0{,}5 \cdot 0{,}13 = 0{,}065\text{m}$

$$c > c_1 , c_2 \Rightarrow \underline{\underline{OK!}}$$

Verifications for headed fasteners loaded in shear according to CEN/TS 1992-4-2 6.3

Steel failure of fastener without lever arm

Hole clearence requirements are fulfilled, (i.e. $k_2 = 1$).

$$V_{Ed}^{h} \le V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{Ms}}$$

Most loaded fastener $V_{Ed}^{h} = \frac{F_y}{6} = \frac{10}{6} = 1,7$ kN

$V_{Rk,s}$ is based on f_{uk} ; $V_{Rk,s} = 0,5 \cdot A \cdot f_{uk} = 0,5 \cdot \pi \cdot 0,006^2 \cdot 800 \cdot 10^6$

$\Rightarrow V_{Rk,s} = 45,2$ kN

$\gamma_{Ms} = 1,0 \cdot f_{uk}/f_{yk} \ge 1,25 \Rightarrow \gamma_{Ms} = \frac{800}{640} = 1,25$

$\Rightarrow V_{Rd,s} = \frac{45,2}{1,25} = 36,2$ kN $> V_{Ed}^{h} = 1,7$ kN $\Rightarrow$ OK!

Concrete edge failure

$$V_{Ed}^{g} \le V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{Mc}}$$

The shear force shall be distributed on the fasteners closest to the free edge, i.e. anchor 1, 2 and 3 since the load works in the y-direction.

Fastener group $V_{Ed}^{g} = F_y = 10$ kN

$$V_{Rk,c} = V_{Rk,c}^{0} \cdot \frac{A_{c,V}}{A_{c,V}^{0}} \cdot \psi_{s,V} \cdot \psi_{h,V} \cdot \psi_{ec,V} \cdot \psi_{\alpha,V} \cdot \psi_{re,V}$$

$V_{Rk,c}^{0} = 1,6 \cdot d_{nom}^{\alpha} \cdot l_f^{\beta} \cdot \sqrt{f_{ck,cube}} \cdot c_1^{1,5}$ with $d_{nom} = d$

$l_f = h_{ef}$

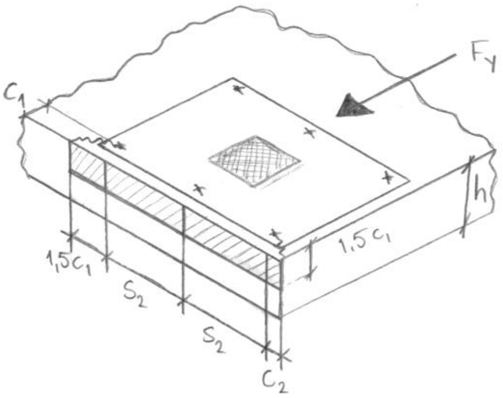
$c_1 = 0,075$ m

$$\alpha = 0,1 \cdot \left(\frac{l_f}{c_1}\right)^{0,5} = 0,1\left(\frac{0,13}{0,075}\right)^{0,5} = 0,13$$

$$\beta = 0,1 \cdot \left(\frac{d_{nom}}{c_1}\right)^{0,2} = 0,1\left(\frac{0,012}{0,075}\right)^{0,2} = 0,07$$

$\Rightarrow V^0_{Rk,c} = 1{,}6 \cdot 12^{0{,}13} \cdot 130^{0{,}07} \cdot \sqrt{25} \cdot 75^{1{,}5} = 10{,}1\ kN$

$A^0_{c,V} = 4{,}5 \cdot c_1^2 = 4{,}5 \cdot 0{,}075^2 = 0{,}025\ m^2$



$c_1 = 0{,}075\ m$
$c_2 = 0{,}1\ m$
$s_2 = 0{,}15\ m$
$h = 0{,}2\ m$

$1{,}5 \cdot c_1 = 0{,}1125\ m$
$s_2 < 3 \cdot c_1$
$c_2 < 1{,}5 \cdot c_1$

$\Rightarrow A_{c,V} = (1{,}5 \cdot c_1 + 2 \cdot s_2 + c_2) \cdot 1{,}5 \cdot c_1 = (1{,}5 \cdot 0{,}075 + 2 \cdot 0{,}15 + 0{,}1) \cdot 1{,}5 \cdot 0{,}075$
$= 0{,}058\ m^2$

$\Psi_{s,V} = 0{,}7 + 0{,}3 \dfrac{c_2}{1{,}5 \cdot c_1} = 0{,}7 + 0{,}3 \dfrac{0{,}1}{1{,}5 \cdot 0{,}075} = 0{,}97 \quad (\leq 1)$

$\Psi_{h,V} = \left(\dfrac{1{,}5 \cdot c_1}{h}\right)^{0{,}5} = \left(\dfrac{1{,}5 \cdot 0{,}075}{0{,}2}\right)^{0{,}5} < 1 \Rightarrow \Psi_{h,V} = 1$

$\Psi_{ec,V} = 1$ (No eccentricity of load)

$\Psi_{\alpha,V} = 1$ $(\alpha_V = 0)$

$\Psi_{re,V} = 1$ (Non-cracked concrete, no reinforcement)

$V_{Rk,c} = 10{,}1 \cdot \dfrac{0{,}058}{0{,}025} \cdot 0{,}97 \cdot 1 \cdot 1 \cdot 1 \cdot 1 = 22{,}7\ kN$

$\gamma_{Mc} = 1{,}5$

$\Rightarrow V_{Rd,c} = \dfrac{22{,}7}{1{,}5} = \underline{15{,}1\ kN} > V^g_{Ed} = 10\ kN \quad \Rightarrow \underline{\underline{OK!}}$

Concrete pry-out failure

$V_{Ed}^{g} \leq V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{Mc}}$ for fastener group and;

$V_{Ed}^{h} \leq V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{Mc}}$ for most loaded fastener.

$V_{Ed}^{g} = F_y = 10\ kN$

$V_{Rk,cp} = k_3 \cdot N_{Rk,c}$ where $k_3 = 1$

$N_{Rk,c}$ according to CEN/TS 1992-4-2 6.2.5, concrete cone failure. This value should be calculated for the group of fasteners loaded in shear. In this case it is valid for all six fasteners which means $N_{Rk,c}$ will have the same value as before.

$V_{Rk,cp} = 1 \cdot 147{,}5 = 147{,}5\ kN$ $\gamma_{Mc} = 1{,}5$

$\Rightarrow V_{Rd,cp} = \frac{147{,}5}{1{,}5} = \underline{\underline{98{,}3\ kN}} > V_{Ed}^{g} = 10\ kN \Rightarrow \underline{\underline{OK!}}$

$V_{Ed}^{h} = \frac{F_y}{6} = \frac{10}{6} = 1{,}7\ kN$

The most unfavourable anchor is number 1 since it has the smallest actual projected area $(A_{c,N})$.
According to the figure in section "concrete cone failure":

$A_{c,N} = \left(c_2 + \frac{s_2}{2}\right) \cdot \left(c_1 + \frac{s_1}{2}\right) = \left(0{,}075 + \frac{0{,}25}{2}\right)\left(0{,}1 + \frac{0{,}15}{2}\right) = 0{,}035\ m^2$

$N_{Rk,c} = N_{Rk,c}^{0} \cdot \frac{A_{c,N}}{A_{c,N}^{0}} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N}$

$N_{Rk,c} = 88{,}2 \cdot \frac{0{,}035}{0{,}152} \cdot 0{,}82 \cdot 1 \cdot 1 = 16{,}7\ kN$

$V_{Rk,cp} = 1 \cdot 16{,}7 = 16{,}7\ kN$

$\Rightarrow V_{Rd,cp} = \frac{16.7}{1{,}5} = \underline{\underline{11{,}1\ kN}} > V_{Ed}^{h} = 1{,}7\ kN \Rightarrow \underline{\underline{OK!}}$

Combined tension and shear load verification for headed fasteners according to CEN/TS 1992-4-2 6.4.1

Utilization ratios $\beta_N = \frac{N_{Ed}}{N_{Rd}}$ and $\beta_V = \frac{V_{Ed}}{V_{Rd}}$:

	N_{Ed}	N_{Rd}	β_N
Steel failure	5	60.3	0,083
Pull-out failure	5	37.4	0,134
Concrete conce failure	30	98.3	**0,305**
Splitting failure	30	118	0,254

	V_{Ed}	V_{Rd}	β_V
Steel failure	1.7	36.2	0,047
Concrete edge failure	10	15.1	**0,662**
Concrete pry-out failure	1.7	11.1	0,153

Since steel failure not is decisive the other modes of failure should be satisfied.

Either $\beta_N + \beta_V \leq 1,2$ or $\beta_N^{1,5} + \beta_V^{1,5} \leq 1$

$0,305 + 0,662 = 0,967 < 1,2 \Rightarrow \underline{\underline{OK!}}$

$\left(0,305^{1,5} + 0,662^{1,5} = 0,707 < 1 \Rightarrow \underline{\underline{OK!}}\right)$

GENERAL INFORMATION

Step name:	Max. Anchor force N: [N]	Max. Anchor force V: [N]	Concrete foundation			Steel Plate		
			util. [%]	σ-Ed	fcd = fck/1.50	util. [%]	σ-Ed	fud = fuk/1.15
Temperature_MaxFx	11111,03906	1666,666667	12%	-1548999,464	13333333,33	6%	39651860	695652173,9
Temperature_MinFx	0	0	0%	0	13333333,33	0%	0	695652173,9
Temperature_MaxFy	0	0	0%	0	13333333,33	0%	0	695652173,9
Temperature_MinFy	0	0	0%	0	13333333,33	0%	0	695652173,9
Temperature_MaxFz	0	0	0%	0	13333333,33	0%	0	695652173,9
Temperature_MinFz	0	0	0%	0	13333333,33	0%	0	695652173,9
Temperature_MaxMx	0	0	0%	0	13333333,33	0%	0	695652173,9
Temperature_MinMx	0	0	0%	0	13333333,33	0%	0	695652173,9
Temperature_MaxMy	0	0	0%	0	13333333,33	0%	0	695652173,9
Temperature_MinMy	0	0	0%	0	13333333,33	0%	0	695652173,9
Temperature_MaxMz	0	0	0%	0	13333333,33	0%	0	695652173,9
Temperature_MinMz	0	0	0%	0	13333333,33	0%	0	695652173,9

FAILURE MODES WHEN TENSIONED ANCHORS:

Step name:	Steel failure			Pull-out failure			Concrete cone failure			Splitting failure			Blow-out failure			
	util. [%]	Ned,h	Nrd,s	util. [%]	Ned,h	Nrd,p	util. [%]	Ned,g	Nrd,c	util. [%]	Ned,g	Nrd,sp	Edge	util. [%]	Ned,g	Nrd,cb
Temperature_MaxFx	18%	11111,04	60318	30%	11111,039	37384,953	33%	31338,417	95944,601	32%	31338,417	96841,9	No validation required			
Temperature_MinFx	0%	0	60318	0%	0	37384,953	0%	0	62411,334	0%	0	96841,9	No validation required			
Temperature_MaxFy	0%	0	60318	0%	0	37384,953	0%	0	62411,334	0%	0	96841,9	No validation required			
Temperature_MinFy	0%	0	60318	0%	0	37384,953	0%	0	62411,334	0%	0	96841,9	No validation required			
Temperature_MaxFz	0%	0	60318	0%	0	37384,953	0%	0	62411,334	0%	0	96841,9	No validation required			
Temperature_MinFz	0%	0	60318	0%	0	37384,953	0%	0	62411,334	0%	0	96841,9	No validation required			
Temperature_MaxMx	0%	0	60318	0%	0	37384,953	0%	0	62411,334	0%	0	96841,9	No validation required			
Temperature_MinMx	0%	0	60318	0%	0	37384,953	0%	0	62411,334	0%	0	96841,9	No validation required			
Temperature_MaxMy	0%	0	60318	0%	0	37384,953	0%	0	62411,334	0%	0	96841,9	No validation required			
Temperature_MinMy	0%	0	60318	0%	0	37384,953	0%	0	62411,334	0%	0	96841,9	No validation required			
Temperature_MaxMz	0%	0	60318	0%	0	37384,953	0%	0	62411,334	0%	0	96841,9	No validation required			
Temperature_MinMz	0%	0	60318	0%	0	37384,953	0%	0	62411,334	0%	0	96841,9	No validation required			

FAILURE MODES WHEN SHEARED ANCHORS:

Step name:	***Steel failure w/o lever arm***			***Concrete pry-out failure (group)***			***Concrete pry-out failure (single)***			***Concrete edge failure***			
	util. [%]	***Ved,h***	***Vrd,s***	***util. [%]***	***Ved,g***	***Vrd,cp***	***util. [%]***	***Ved,h***	***Vrd,cp***	***Edge***	***util. [%]***	***Ved,g***	***Vrd,c***
Temperature_MaxFx	**2%**	1666,6667	72381,6	**10%**	10000	95944,601	**14%**	1666,6667	11741,614	parallel to X-axis	**70%**	10000	14341,597
Temperature_MinFx	*No validation required*			**0%**	0	0	*No validation required*			*No validation required*			
Temperature_MaxFy	*No validation required*			**0%**	0	0	*No validation required*			*No validation required*			
Temperature_MinFy	*No validation required*			**0%**	0	0	*No validation required*			*No validation required*			
Temperature_MaxFz	*No validation required*			**0%**	0	0	*No validation required*			*No validation required*			
Temperature_MinFz	*No validation required*			**0%**	0	0	*No validation required*			*No validation required*			
Temperature_MaxMx	*No validation required*			**0%**	0	0	*No validation required*			*No validation required*			
Temperature_MinMx	*No validation required*			**0%**	0	0	*No validation required*			*No validation required*			
Temperature_MaxMy	*No validation required*			**0%**	0	0	*No validation required*			*No validation required*			
Temperature_MinMy	*No validation required*			**0%**	0	0	*No validation required*			*No validation required*			
Temperature_MaxMz	*No validation required*			**0%**	0	0	*No validation required*			*No validation required*			
Temperature_MinMz	*No validation required*			**0%**	0	0	*No validation required*			*No validation required*			

FAILURE MODES WHEN FOR COMBINED TENSION AND SHEAR FAILURE MODES:

Step name:	Maximum values		Interaction (Steel failure)	Interaction (Other failure modes)	
	ßN	ßV	ßN^2 + ßV^2 <= 1	ßN + ßV <= 1.2	ßN^1.5 + ßV^1.5 <= 1
Temperature_MaxFx	0,3266303	0,6972724	-	1,023902766	0,768916686
Temperature_MinFx	0	0	-	0	0
Temperature_MaxFy	0	0	-	0	0
Temperature_MinFy	0	0	-	0	0
Temperature_MaxFz	0	0	-	0	0
Temperature_MinFz	0	0	-	0	0
Temperature_MaxMx	0	0	-	0	0
Temperature_MinMx	0	0	-	0	0
Temperature_MaxMy	0	0	-	0	0
Temperature_MinMy	0	0	-	0	0
Temperature_MaxMz	0	0	-	0	0
Temperature_MinMz	0	0	-	0	0

OTHER EXPUNGED INFORMATION FROM APPLICATION OUTPUT GENERATION DUE TO EXCESSIVE PRESENTATION:

Several cell-sheets with information regarding anchor forces (tension, shear distributed over all anchors, shear distributed over active anchors near edge parallel to x-axis and shear distributed over active anchors near edge parallel to y-axis), concrete cone area, pry-out area, edge area, blow-out area, input load information and general model information is chosen not to be shown in full context due to unwarranted information presentation.