FASTENING SYSTEMS IN CONCRETE CONSTRUCTION

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Fastening Systems in Concrete Construction

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FOREWORD

It gives me immense pleasure in writing the foreword to this book on Fastening Systems in Concrete Construction. It is first book of its kind on the subject in our country. It aims to bridge the gap in understanding on the subject due to lack of comprehensive reference guide and standard on the subject in our country. This book first introduces the basics of fastening concept and their wide range of applications. It can be overwhelming for someone new to this concept to see how many applications are dependent on use of appropriate fastening systems.

This book then goes on to introduce different types of fastening systems like cast-in anchor channels, post-installed anchors, direct fasteners and post-installed reinforcing bars. Each of these fastening systems have a dedicated chapter to further elaborate the technology, its range of applications, assessment method, design and installation. The book smoothly transitions from cast-in to post-installed technology, giving the readers a chance to draw comparison between the two. Much of this book is devoted to illustrative design examples which makes it easier for the readers to understand the concept and design method. It emphasizes on the need for performance assessment of different types of fastening systems and dependence of design on it. This book also includes sample technical assessment report that has been cross-referred to in the design problems, which makes it easier for the readers to link the assessment to design.

This book aims to draw attention of practicing engineers to establish a standard practice for designing these connections. It is my pleasure to recommend this book to practicing engineers as well as academia or anyone who is interested in the subject of fastening systems.

Hilti India Pvt Ltd. is very excited to be a part of this project. It is just another step in the right direction to raise awareness on fastening systems and to elevate industry standard. I would like to thank and congratulate both authors and our partners for sharing their knowledge and making valuable contributions in promoting right design practices along with innovation, safety and productivity at our construction job sites.

Jayant Kumar

M.D.

Hilti India Pvt. Ltd.

PREFACE

From antiquity to the present-day engineering, the built environment always has included designing connections, some involving the same materials but others different, yet the role that these connections play have only increased in complexity. Thus, mastering their behaviour and specifying their exact performance enables some of the most complex and breath-taking structures ever conceived, which only serves to highlight the need to pay greater attention to them. The modern construction industry is replete with fastenings that connect concrete-to-concrete, concrete-to-steel, steel-to-steel, steel-to-masonry, among others, each having different behaviours with differing influencing parameters and involving additional design complexities. An additional consideration is their installation in the construction sequence, which can be cast-in-place– for instance, concrete-to-concrete connections through rebars and concrete-to-steel connections through anchorages – or post-installed, which alters the behaviour and design significantly. Since the behaviour of steel and reinforced concrete elements and structures is well-documented, engineers can design cast-in-place scenarios efficiently through modern design standards for a specified performance level.

However, whether it is the behavioural understanding or standardized design guidelines, little information is available in most design standards for engineers to reference when post-installed or, in certain cases, cast in-situ fastenings are installed in concrete. Without sufficient information, the plethora of cast in-situ or post-installed fastenings connecting, for instance, a steel baseplate to concrete – the most common application used in numerous construction sites globally – often are designed with poor engineering judgement stemming from a lack of sufficient knowledge on this topic.

The element enabling the connection of these to disparate materials are fasteners, which resist the tension component of the applied force and, therefore, utilize the tensile resistance of the concrete base material, a major element that influences fastening behaviour and fundamental when designing the concrete member as well. The fasteners themselves transfer the applied static or dynamic tension and shear loads to the base material through various mechanisms such as mechanical interlock, friction, bond, or a combination of these three.

Understanding the mechanisms involved enables the right selection – based on environmental conditions, load parameters, and overall suitability – when coupled with right design and installation. This is fundamental to optimize the overall design for the desired performance level of each application and greatly reduces the risk of failure that impacts lives and building assets.

This book is aimed at practicing engineers, students, academics, and anyone in the construction industry involved with selecting, designing, and installing fasteners with the express purpose of introducing the fundamentals of post-installed anchor and rebar connections, cast-in-place anchor channels, and – briefly – concrete-to-steel direct fastening. The book presents, with examples, the basic mechanisms and the ways in which they are captured in the assessment and design criteria currently available in international literature, in absence of equivalent national standards, in order to provide guidance to those involved with the design of such. However, one should refer to the latest standards and approvals for design.

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Er. Narendra Singh, FIE
President

MESSAGE

It is my pleasure to note that Hilti India Pvt Ltd is publishing a technical reference book for "Fastening Systems in Concrete Construction".

In the present construction environment, different types of fastening systems are used to enable construction of simple to complex structures. They form an integral part of any structure and should be given its due share of attention while designing the structure. It is necessary to keep pace with the latest developments in this field to be able to select the appropriate systems and design them properly. It is heartening to see the efforts put in by Hilti India Pvt Ltd to develop a comprehensive technical reference book for "Fastening Systems in Concrete Construction". The contents of this book are based on their technical expertise and experience in the field of innovative fastening technology. I am confident that this technical reference resource will enable students and professionals to obtain an insight in the domain of fastening systems. In this regard, it is important to highlight that the National standards should be established fastening technology in the Indian construction industry from a quality and safety perspective.

The technical book is well researched and has comprehensive reference material which makes this book unique. I wish Hilti India Pvt Ltd. all the best in their endeavour to bring out resource material based on their considerable expertise in Civil Engineering.

I convey my compliments to all associated with this publication.

Er Narendra Singh, FIE
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ISTE

Message

It is the endeavor of Indian Society of Technical Education (ISTE) to assist to incorporate developments in Technical Education System. With this motto in mind, we collaborate with various stakeholders to upgrade the knowledge of our educators and students, on a continuous basis. The collaboration with Hilti India Pvt. Ltd. for publication of this technical book is one such example.

This book on "Fastening Systems in Concrete Construction" has been developed by technical experts from Hilti who have several years of experience in the field of fastening systems. The book reflects their knowledge on the subject and includes a review of the latest international standards. This book will serve as a state-of-art technical reference material for students, teachers, researcher and practicing engineers.

It is important for our educational system to keep pace with advances in the Industry so that we can equip our students with skills relevant for industry and increase their employability. This book draws attention to a subject which needs to be incorporated in the current technical curriculum of our universities. We hope that this book will serve to bridge the gap between theory and practice.

I congratulate Hilti India Pvt. Ltd. for developing this book to benefit of all stakeholders. ISTE is glad to be part of this initiative and will do its part in promoting awareness on this critical topic. I am sanguine that the book will be a big success.

(Dr. Pratapsinh K. Desai)
President, ISTE

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Er. Prashant D. Sathe is currently operating as Technical Expert- Regulations in Codes & Approvals with Hilti India Pvt. Ltd., New Delhi. He post-graduated from Earthquake Engineering Department, Indian Institute of Technology, Roorkee, India with academic degree M. Tech. in Structural Dynamics. He pursued his B. Tech. degree in Civil Engineering from Government College of Engineering, Pune, India. He has been functioning in the field of structural civil engineering for last nine years with enthusiasm. He has contributed significantly in the development of a culture for innovation and execution of new ideas in structural civil engineering by sharing his technical expertise and experience. He has an interest in developing standard codes and is an active member of several technical committees of Bureau of Indian Standards. He is an active member of several civil engineering associations like ISET, The Institution of Engineer (India), ISSE, ISWE, etc. His scientific technical research areas of interest are in structural dynamics & earthquake engineering, performance evaluation of structural & non-structural components and systems, performance-based design & structural rehabilitation, structural health monitoring, structural & material testing, tall building structural design, special structures design, façade design, bridge & tunnel engineering, airport & docks-harbour design, sustainable design & execution with holistic project management etc. He has authored a number of technical papers, presented in conferences and published in journals relating to diverse areas of Structural Civil and Earthquake Engineering.

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CHAPTER 1

Introduction to Fastening Systems for Concrete

1.1 OVERVIEW

A fastening system is an assemblage of components that fastens one material to another in such a way that the connection as a whole can withstand the loads acting on it. For ensuring safety of the structure, the fastening system should be at least as reliable as the members it is used to fasten. The design of fastening system is therefore at least as important as that of the structural members, because its failure can have severe consequences for the connected elements. Fastening systems may be used to connect wood-to-wood, steel-to-steel, steel-to-concrete etc. Each of these applications present unique functional and design challenges. Fastening technology has evolved over the years to address the challenging requirements associated with these applications. For example, headed cast-in-place bolts were used to attach steel column to concrete pedestal but frequent misalignment of bolts during execution delayed the project and increased cost due to rework. Today, alternate technology exists that can overcome these challenges. In this book, such innovative fastening systems are covered for safety relevant concrete connections: steel-to-concrete and concrete-to-concrete.

Concrete fasteners can be used in a wide range of structural and non-structural applications. Structural applications are those in which the members being connected are part of the structural frame that resists the design loads acting on the structure. Examples of structural applications of fastening systems (see Fig.1.1) are mezzanine floor attachment, column to foundation attachment, beam to column attachment, etc. Non-structural applications are those in which the members being connected are an attachment to the primary structure. Examples of non-structural applications of fastening systems (see Fig.1.2) are false ceiling attachment, utility support, equipment fixing, etc. Fastening system applications can also be classified as heavy duty, medium duty and light duty applications based on the intensity of load. Design and safety of fastening system is concerned with identifying the loads that the connection may experience over its expected life, selecting suitable qualified product, determining dimensions of the fixture and fastener as per design, determining a suitable configuration of fasteners, its installation and lastly inspection of the installed fastening system.

(a) Mezzanine floor attachment

(b) Concrete jacketing

(c) Steel jacketing for beam strengthening

Fig.1.1 Examples of structural applications of fastening systems

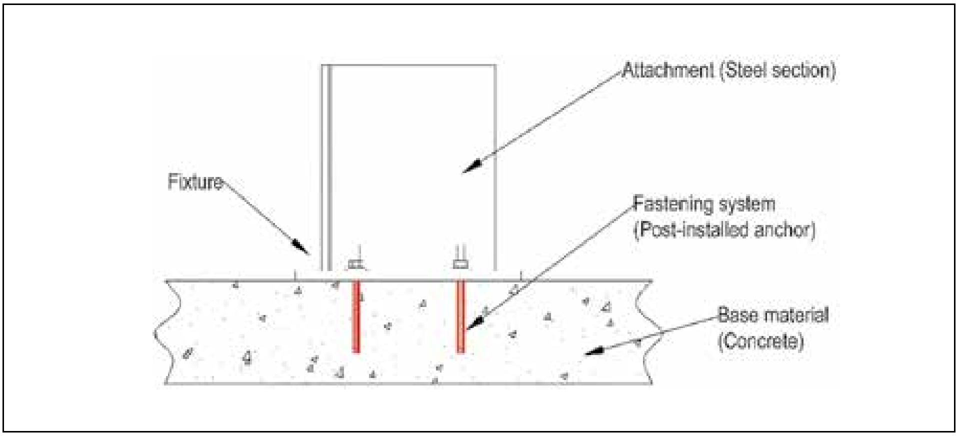
(a) Utility fixing (b) Hand rail attachment

Fig.1.2 Examples of non-structural applications of fastening systems

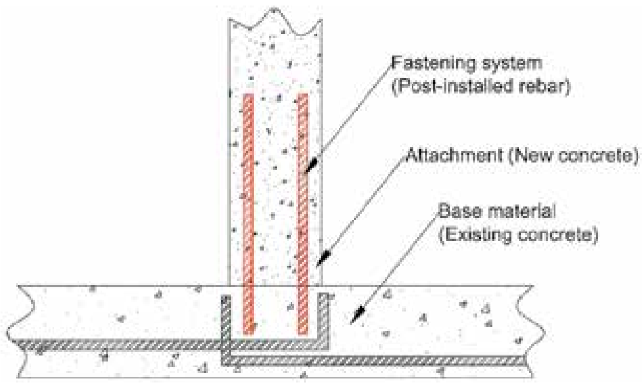
In order to learn to design these fastening systems, it is desirable to begin with an understanding of different types of fastening systems. In this chapter, we first describe the various types of fastening systems and their components. We also present an overview of different load types and establish their importance for the most common applications of fastening systems. We briefly discuss some applications of different types of fastening systems. We then discuss the critical issues that a structural engineer needs to address while designing or assessing the adequacy of the fastening system. As the most important issue is preventing failure, we briefly discuss some failure cases of fastening systems and ponder how these types of failures may be avoided in future.

1.2 TYPES OF FASTENING SYSTEMS AND THEIR APPLICATIONS

Fastening systems for concrete may be broadly categorized into: Steel-to-concrete and concrete-to-concrete. A steel-to-concrete fastening system (see Fig.1.3a) primarily comprises of the fastener, the base material, the fixture and the attachment (i.e. the metal assembly that transmits loads to the fastener). Whereas, a concrete-to-concrete fastening system (see Fig.1.3b) comprises of the existing base material, the attachment (e.g. new concrete member) and the fastening system. The base material could be normal-weight concrete, or special concrete (e.g. aerated concrete, concrete masonry blocks, precast concrete, bricks etc). In this book, discussions have been restricted to fastening systems for normal-weight concrete as the qualification and design principle also varies with the base material type. The attachment to concrete could be steel, concrete, wood, glass etc. For simplicity, only steel and concrete member attachment are discussed.



(a) Steel-to-concrete



(b) Concrete-to-concrete

Fig.1.3 Typical fastening systems for concrete

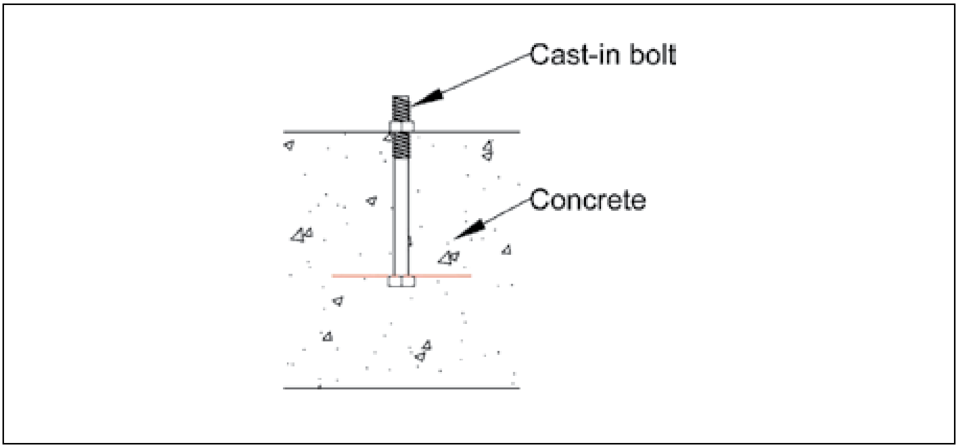
1.2.1 FASTENING SYSTEMS

In this section, steel-to-concrete and concrete-to-concrete connection types have been briefly discussed.

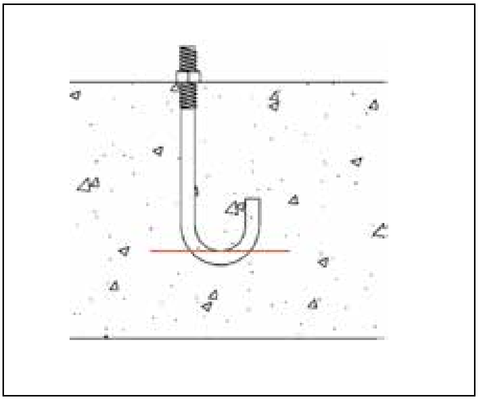
(a) Fastening systems for steel-to-concrete connections:

Most modern structures cannot be constructed today without use of one fastening system or the other. Fastening of steel-to-concrete is one of the most common applications in composite buildings, industrial sheds, retrofitting projects etc. Fastening of steel-to-concrete can be accomplished using any of the following technologies - cast-in-place anchors, cast-in anchor channels, post-installed anchors or direct fastenings.

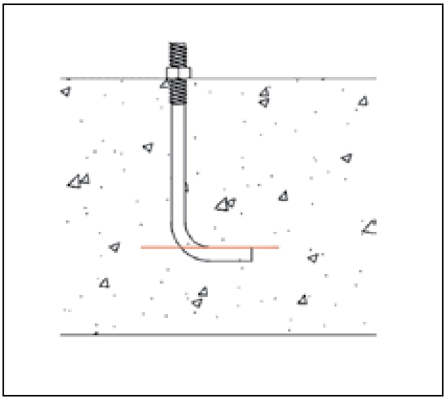
(a) **Cast-in-place anchors**: These are one of the oldest types of anchoring devices, for safely transferring the subjected load onto the base material on which it is fixed. As the name suggests, cast-in-place anchors are cast in position in wet concrete before it sets. They are commonly referred to as cast-in bolts. Bolts with hexagonal head, hooked J bolts, L bolts etc. as shown in Fig.1.4 are some examples of cast-in-place anchor\s. These fastening systems require extensive planning during design and coordination during execution. In case of misalignment or wrong placement, corrective measures are expensive and time consuming. We have not covered cast-in-place anchors in detail in this book as they are already covered in detail in several other literatures.



(a) Headed bolt



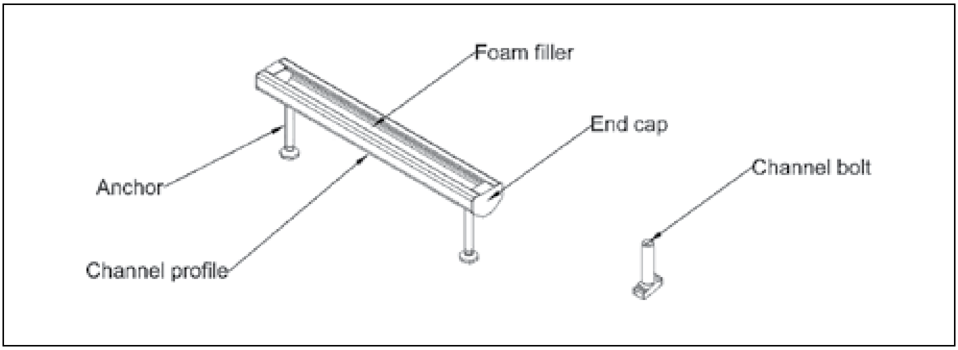
(b) 'J' type bolt



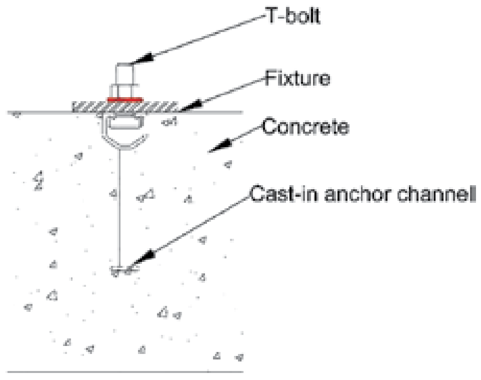
(c) 'L' type bolt

Fig. 1.4 Examples of cast-in-place anchors

(b) Cast-in anchor channels: These are innovative variant of cast-in-place anchors which to an extent overcome the drawbacks of the latter system. In this system, a channel is fitted with multiple headed anchor legs (two or more) as shown in Fig.1.5 The headed anchor legs (for simplicity often termed as "anchor") are either welded or screwed to the anchor channel. The anchor channel is usually square, rectangle or V-shaped. T-bolts swivelled into the anchor channel serves as the fastening point. Cast-in anchor channels are also cast in wet concrete before it sets. This fastening device offers multipoint anchoring system in a single assembly. Cast-in anchor channels can accommodate positioning tolerances in one direction. This fastening solution is commonly used where it is difficult to drill, for example - in high strength concrete, densely reinforced concrete, prestressed concrete etc. Cast-in anchor channels are also considered to be an efficient fastening solution where huge volume of fastening points are required and productivity is a concern, for example utility fixing in tunnels or anchoring of curtain wall elements.



(a) Example of cast-in anchor channel



(b) Cross-sectional view of application

Fig. 1.5.Typical cast-in anchor channel

(c) **Post-installed anchors:** This is an innovative technology which overcomes most of the challenges associated with "cast-in" fastening technology and delivers similar level of performance. Post-installed anchors are installed into hardened concrete and hence the name "post-installed". This fastening technology requires a hole to be drilled in hardened concrete prior to installation of the anchor. The main advantage of this fastening system is the flexibility it offers both at planning and construction stage. Fig.1.6 gives an example of post-installed anchor.

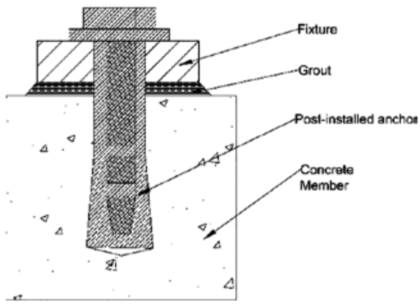


Fig. 1.6 Typical post-installed anchor

(d) **Direct fastenings:** This is another innovative technology in which the fastener is directly driven into the hardened concrete and hence the name "direct" fastenings. Unlike post-installed anchors, this system does not require drilling of hole in concrete prior to installation, in most applications. A nail being hammered into concrete can be considered to be the simplest example of direct fastening. The demand for quick installation and productivity has driven innovation in this segment as well. Tool based direct fastening technology may be broadly classified into - powder-actuated fastening system, gas-actuated fastening system and battery-powered fastening system (see Fig.1.7). In case of powder-actuated systems, the fastener (e.g. nail) is driven in hardened concrete using tool powered by propellant charge. Whereas gas powered tools are used for gas-actuated systems and battery powered tools are used for battery-powered systems. This fastening technology is commonly used for light duty applications like wire mesh fixing, waterproofing membrane fixing etc. In this book, we have briefly covered direct fastening technology without going into design detail.

(a) Powder-actuated fastening systems

(b) Gas-actuated fastening systems

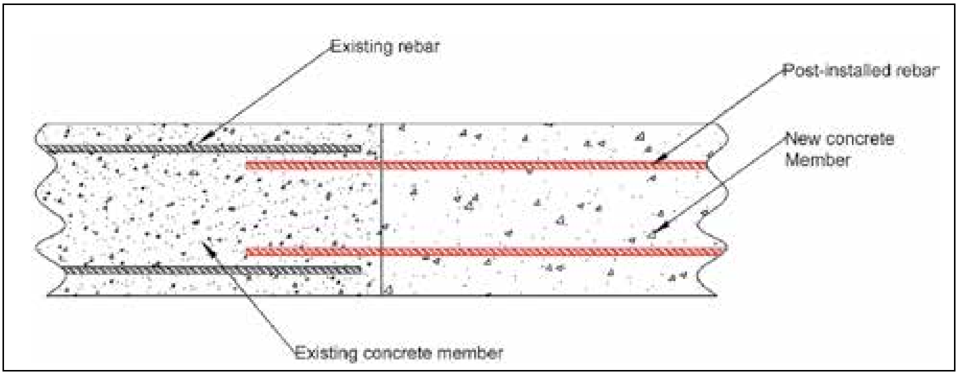
(c) Battery-powered fastening systems

(d) Example of Direct fastener

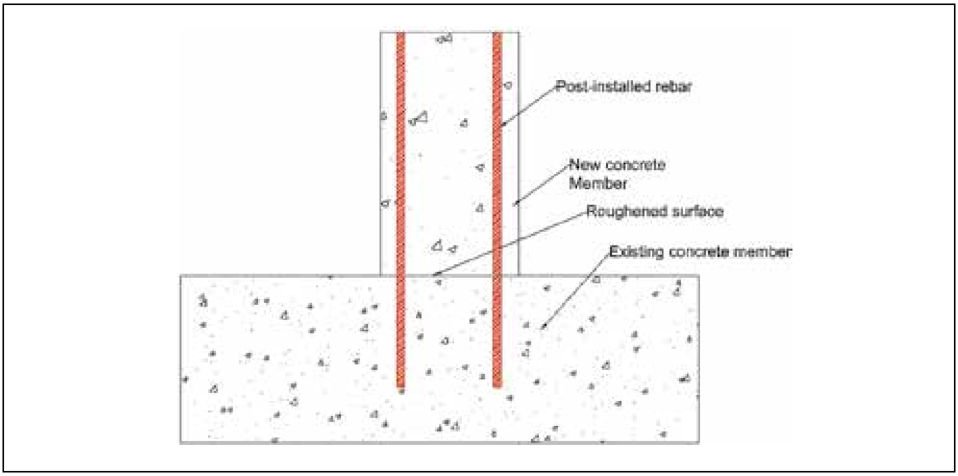
Fig. 1.7 Typical direct fastening systems

(b) **Fastening systems for concrete-to-concrete connections:**

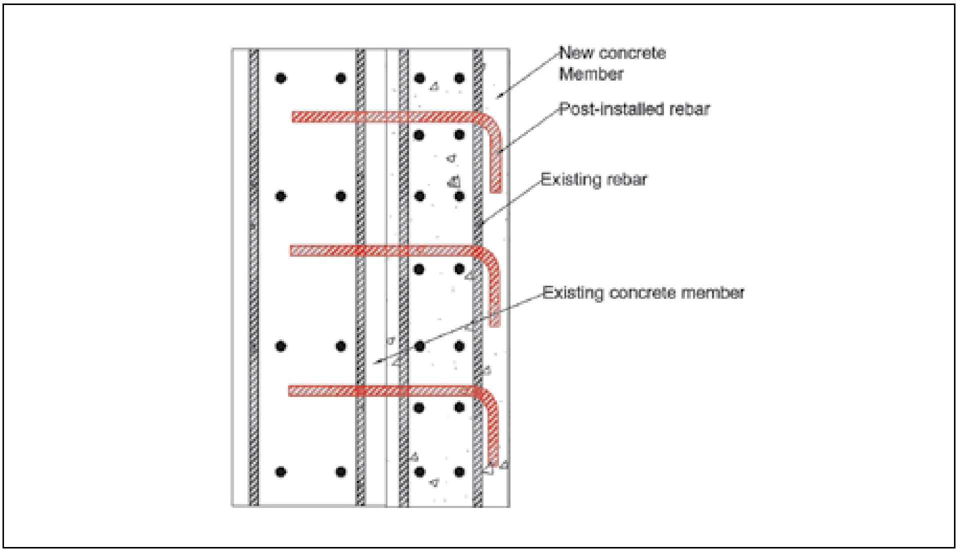
In this section, the fastening technology used for connecting new concrete member (i.e. which is yet to be cast) to existing hardened concrete is discussed. The only available technology for accomplishing this connection is post-installed reinforcing bars, popularly known as post-installed rebar. We have not discussed mechanical couplers, protruding reinforcing bar or similar technologies which require planning and have to be present in hardened concrete to form the new connection. Post-installed reinforcing bar fastening technology requires a rebar which is post-installed, to be spliced with existing rebar or anchored in hardened concrete. In this system, a reinforcing bar is inserted into a drilled hole filled with adhesive in hardened concrete as illustrated in Fig.1.8 These systems are used for extensions as well as strengthening applications in construction industry.



(a) Spliced post-installed rebar application



(b) Starter bar application



(c) Shear dowel application

Fig. 1.8 Typical post-installed rebar

1.2.2 APPLICATIONS

Fastening technology has wide variety of applications in each stage of construction. It is relevant both for new construction as well as for retrofitting projects. During excavation stage, fastening may be used for shore piling, pile cap fixing etc. During erection of superstructure, fastening may be used for formwork support, drywall fixing, curtain wall attachment, strengthening etc. After erection of superstructure, fastening may be used for utility fixing, machine fixing, false floor attachment etc. Some applications of fastening systems for steel-to-concrete connection and concrete-to-concrete connections are shown in Fig.1.9, and Fig.1.10 respectively.

(b) Seismic bracing attachment

(a) Curtain wall attachment

(c) Crash barrier fixing

Fig. 1.9 Application examples of steel-to-concrete fastening systems

(a) Rebar in place of missing coupler

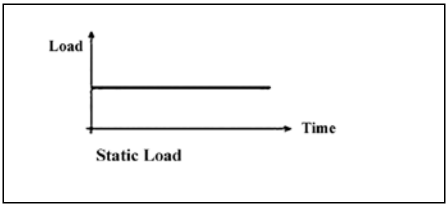
(b) Slab erection from D-Wall

(c) Column strengthening

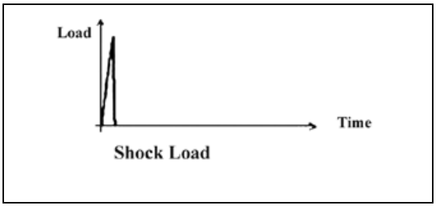
Fig. 1.10 Application examples of concrete-to-concrete fastening systems

1.2.3 LOAD ON FASTENING SYSTEMS

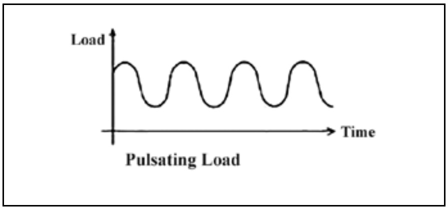
In order to carry out design of these applications using fastening systems, one needs to determine the load acting on the fastener. There is already considerable information about different types of load in technical literature. In this section, we present only an overview of the nature of different loads and highlight which fastening system is suitable for design for a given load type. Fastening systems may be subjected to static, quasi-static, seismic, fatigue or shock loads depending on the application. An illustration of different load types w.r.t design of fastening systems is shown in Fig.1.11.



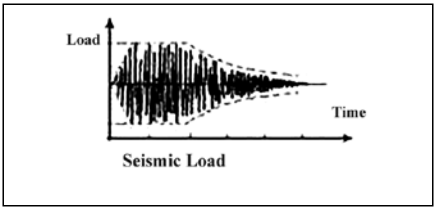
(a) Static or quasi-static load



(b) Shock load



(c) Fatigue load



(d) Seismic load

Fig. 1.11 Types of load w.r.t design of fastening systems

As the fastening technologies discussed in this book are innovative and have been developed over the past few decades, their standards are still evolving in certain parts of the world. There are international standards available for design of post-installed anchors for static, seismic and fatigue loads. Specialized literature is available for shock load design. Whereas, only static design is covered in standards for cast-in anchor channels. The same is the case for post-installed reinforcing bars. However, qualification standards to check suitability of use of post-installed reinforcing bars under seismic conditions have been recently developed. The design loads acting on the connection should be determined as per applicable standards for load and analysis of the structure like “IS 875 - Code of Practice for Design Loads for Buildings and Structures” [1], “IS 1893 - Criteria for Earthquake Resistant Design of Structures” [2], etc.

1.3 CRITICAL CONCERNS

In the previous section, we discussed about innovative fastening technologies and their wide variety of applications. Though these fastening technologies are frequently used, they are not given their due share of attention while designing structures in our country. Due to lack of national standards on the subject there is lack of consistency in technical understanding amongst structural engineers. Thumb rule, load data sheets or design by manufacturer’s engineering team are some of the common practices that are followed for design of connections using technologies discussed in Section 1.2.1. At job site the situation is not very different as the contractor usually subcontracts to specialty contractors who may or may not be trained in product installation. The correct installation tools are often not available. The installation quality is also rarely inspected. To overcome these challenges at jobsite, a small fraction of installed fasteners is sometimes tested to check the load capacity at site.

Such laxity in design and installation may lead to catastrophic failure. There are examples from across the globe on failure of fastening systems due to laxity and their dire consequences. One such infamous example is that of Boston tunnel collapse in 2006. This was one of the busiest commute corridors in Boston. The ceiling of the tunnel collapsed suddenly due to failure of post-installed anchor connection that was supporting it. This accident resulted in the death of one person. As a consequence, the tunnel had to be closed for a very long period of time. Post-accident, extensive investigation was carried out to determine the cause. The investigation team reported incorrect installation and poor creep behaviour of the fasteners used, to be the primary cause of failure. The connection had failed several years after installation. During inspection, several other post-installed anchors were identified to be in critical state and/or were on the verge of failure. The total cost of inspection and repair amounted to staggering value of $54 million [3]. All of this could have been avoided if due attention was given to the product selection, its design and correct installation. A similar accident happened in Japan in 2012. Nine people lost their lives due to collapse of concrete panel of ceiling of Sasago tunnel [4]. In 2014, the rain shelter of a hotel collapsed due to failure of post-installed anchor on a strong rainy day. The issue remains same – either the connection was not designed properly or was not correctly installed.

A structure is only as safe as its weakest link. As structural engineer we need to give this topic its due attention to avoid such failures in future and to minimize risks. Though there are no national standards on the subject, there are several established international standards as well as research papers on the subject. In this book, only European framework is discussed as it is better compatible with our Indian Standards and for sake of simplicity. Some “special literature” for cases or technology that is not directly covered by the European framework is also discussed.

CHAPTER 2

Cast-in Anchor Channels

2.1 OVERVIEW

This chapter is devoted to design of steel-to-concrete connections using cast-in anchor channel fastening technology. From a beginner's perspective who is new to anchoring technology, cast-in anchor channels are ideal for introducing the anchoring concept and transitioning from traditional cast-in bolts to modern post-installed anchoring technology. Over the past few decades, cast-in anchor channel technology's acceptability and popularity has grown in the façade and elevator fixing segment. In the past few years, its use in tunnels as well as applications with large volume of fastening points in densely reinforced concrete has also grown.

Before taking a deep dive into its design philosophy, we begin this chapter by presenting overview of different types of cast-in anchor channels. We then deal with the parameters that may influence performance of these systems, and then move on to talk about different standards available for assessing its performance and design. Design philosophy for determining suitability and its load capacity is presented next. To be able to design, one needs information on load acting on each anchor and anchor group. Therefore, load analysis process is also discussed. Some examples are presented to illustrate how the load acting on the connection is resolved into tension and shear forces acting on each anchor leg. Design of cast-in anchor channels for only static or quasi-static loads is covered. Though this book is intended to be a handy guide for structural design engineers, installation and inspection aspect is also presented for giving the engineers a holistic picture.

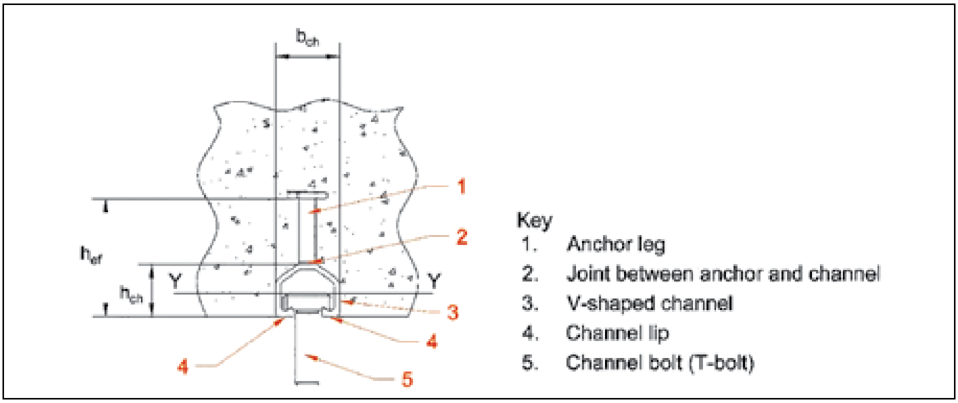
This book focuses on elastic design concept only. Plastic design concept, though available in international literature, is not discussed here as it is not commonly used and is too complex for an engineer who is new to fastening technology.

Much of this chapter is devoted to design than performance evaluation, because the target audience is a structural engineer who can use this as a design guide. However, evaluation is equally important and is prerequisite for design.

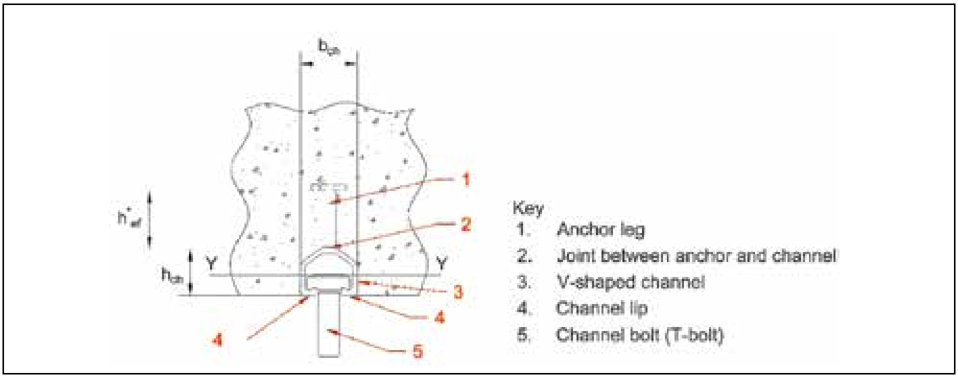
2.2 TYPES OF CAST-IN ANCHOR CHANNELS

Before discussing the types of cast-in anchor channels, lets first talk about cast-in anchors and how cast-in anchor channels are different from them. The primary role of any fastening or anchoring technology is to safely transfer the subjected load onto the base material on which it is fixed. Cast-in anchors (popularly known as cast-in bolts) are anchoring systems which makes use of steel rod bent into different shapes or welded with other steel shapes, to anchor the attachment to the base material. They are placed in position before concreting. The exposed rod protruding from concrete is threaded to allow fixing of nut. Other variations like internal threaded rods are also available. The shape and detailing of cast-in anchors are standardized. However, these anchoring systems require extensive coordination to ensure they are placed and aligned correctly. Any mistake in placement or alignment makes it difficult to fix the attachment. If the deviation is more, the concrete base material has to be demolished and re-casted.

Cast-in anchor channel (see Fig. 2.1) overcomes this issue to an extent by providing a channel fitted with multiple anchoring legs that renders it high load capacity. Unlike cast-in anchors, the dimension and shape of cast-in anchor channels are not standardized as the companies spend huge amount of time and resources in innovating these products and the manufacturing details are privy to them. This necessitates design based on performance of the product in an application under a given set of conditions instead of specifying standard sizes. The main components of cast-in anchor channels are: the channel, the anchor legs, the channel filling material, the channel bolt and the end caps (optional). Different components of cast-in anchor channel along with the notations used are shown in Fig. 2.1 [5]. Commonly, Styrofoam is used as a filler material for the channel to protect it during concreting. However, it is hard to remove after concreting, which makes it difficult to slide in the channel bolt (e.g. T-bolt) which is to be used to fasten the attachment. Nowadays, environment friendly LDPE foam tear out strips are available which overcome the challenges associated with Styrofoam filler material. Some cast-in anchor channels also have end caps to avoid concrete from entering the channel during concreting. Different manufacturers offer different channel bolt types also. Some provide channel bolts on basis of channel cross-section whereas others provide “one size fits all” option, which is beneficial from inventory perspective. Supplementary reinforcement for tying a potential concrete breakout body to the concrete member is also provided sometimes. The detailing requirements for supplementary reinforcement are not discussed in this book. However, the design check for adequacy of provided supplementary reinforcement is briefly discussed in this chapter.



(a) Effective embedment[1] h_{ef} for anchor channels



(b) Effective embedment h^*_{ef} for anchor channels

Fig. 2.1 Components of typical cast-in anchor channel and notations (resketched based on illustration in EN 1992-4 [6])

Cast-in anchor channels can be classified on basis of manufacturing technique, channel shape and method used to fix anchor leg to the channel. Some varieties of cast-in anchor channels are discussed in this section:

[1] The overall effective depth through which the anchor transfers the load to the base material

(a) Classification on basis of the manufacturing technique: The channel can be hot-rolled, cold-rolled or manufactured using specialized techniques like Temperature Controlled Rolled Steel (TCRS). Each method has its pros and cons. Irrespective of the manufacturing method used, it boils down to the performance of the cast-in anchor channel when used as a part of the fastening system in concrete.

(b) Classification on basis of the channel shape: It is common for the anchor channel to be square, rectangle or V-shaped. Other shapes are also possible. Typically, the V-shaped channels allow reduction of edge distance as well as engage larger area of the concrete, thus permitting higher shear loads.

(c) Classification on basis of fixing method of anchor leg: The anchor legs may be threaded-in or welded.

In addition, there are special types of channels like “teethed” channels, “curved” channels, channels specially designed for corners etc., to cater to specific application needs.

2.3 PERFORMANCE INFLUENCING PARAMETERS

There are many varieties of cast-in anchor channels in the market. As there are no set selection criteria in our national standards, a structural engineer may opt for a preferred manufacturing technique or channel shape based on their experience. This approach is not correct as the performance of cast-in anchor channels is dependent on many parameters and not just manufacturing technique. In absence of specification-based standards, the only way to compare two systems is through its performance assessment as per established standards. The performance of such products is evaluated and documented according to a standardized “Performance Assessment Document” (see Section 2.4) which evaluates the fastening system as a whole (i.e. the cast-in anchor channel system in the base material) under a given set of conditions and assesses the influence of different parameters on its performance. The tests are carried out by accredited laboratories and evaluated by third-party approval bodies. This process allows the structural engineer to compare the performance
of two similar looking products, without having to bother about manufacturer’s specifications. Some of these parameters are discussed in this section.

2.3.1 INSTALLATION PARAMETERS

Various installation parameters affect the performance of cast-in anchor channels. The performance is primarily measured in terms of load-displacement behaviour. Any deviation from standard installation procedure recommended by the manufacturer may negatively influence its performance. Some of these installation parameters are discussed:

(a) Concrete grade, type and condition:

The performance of cast-in anchor channels is dependent to a large extent on the quality of concrete it is installed in. If the concrete has not been compacted properly after the cast-in anchor channel is placed in position, then it may lead to voids and gap underneath the channel. This in turn can negatively impact the performance as cast-in anchor channels utilize the tensile capacity of concrete to transfer the loads. If the concrete is not of requisite grade and quality, the cast-in anchor channel again will not perform irrespective of the quality of the product.

(b) Dimension, position and orientation of cast-in anchor channel:

The load carrying capacity changes with the size of anchor channel used, along with number and position of its anchoring legs. The shape of the channel also has an implication on the way load is distributed, which thereby influences its capacity. The location of the channel bolt along the channel also has implication on the way load is transferred. Each product has a different value of minimum spacing and edge distance depending on performance assessment, which needs to be considered during design and installation. All of these aspects are taken into consideration during evaluation as per "Performance Assessment Document".

2.3.2 IN-SERVICE PARAMETERS

The environment in which the cast-in anchor channel is installed and exposed to, and the load type it is subjected to, will all affect its behaviour and capacity.

The expected state of concrete (i.e. uncracked or cracked) during its service life can also affect the performance of cast-in anchor channels. The load capacity of cast-in anchor channels drop when the concrete cracks as illustrated by Fig. 2.2. The effect may vary depending on the orientation and the position of crack. Therefore, it is advisable to assume concrete to be in cracked state for design of anchor channels [7]. Only if the structural engineer can establish that the concrete member in which the cast-in anchor channel is installed will not crack during its service life, then uncracked concrete state may be assumed for design. Clause 4.7 of EN 1992-4 [6] provides guidance on establishing uncracked concrete state.

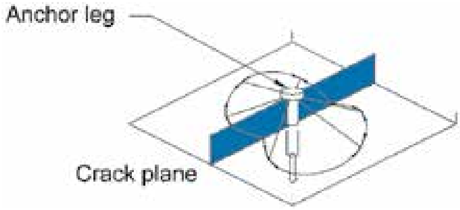


Fig. 2.2 Example of crack passing through installed cast-in anchor channel

2.4 CODES AND STANDARDS

There are no national standards at present for testing, assessment and design of cast-in anchor channels. However, there are several international standards that aid in assessment and design of cast-in anchor channels. For simplicity, the assessment and design provisions based on European standards and regulatory framework only are presented in this book.

The construction products regulation (CPR) lays down harmonized rules for marketing of construction products in Europe. Harmonized specifications include harmonized standards and European Assessment Documents (EAD). When construction product is not covered or not fully covered by a harmonized standard, as is the case for cast-in anchor channels, the manufacturers can request for a “European Technical Assessment” report to be issued by one of the “Technical Assessment Bodies” on the basis of a “European Assessment Document (EAD)” developed by the European Organisation for Technical Assessment (EOTA) [8].

The various parameters discussed in the previous section along with other relevant parameters are considered for testing and assessment as per applicable “European Assessment Document (EAD)”. The performance data in terms of characteristic values and installation parameters determined based on this assessment are documented in “European Technical Assessment (ETA)” report. This performance data is then used by the structural engineer to design a connection using cast-in anchor channel.

2.4.1 PERFORMANCE ASSESSMENT STANDARDS

The "European Assessment Document" EAD 330008-00-0601 [9] for "Anchor Channels" developed by "European Organisation for Technical Assessment (EOTA)" is used to test and assess performance of cast-in anchor channels. The testing and assessment are carried out by a third-party accredited lab and approval body recognized as "Technical Assessment Body". Each "European Assessment Document" defines the product it is applicable for, its intended use, essential performance characteristics required to fulfil the intended use, test and assessment methods to determine essential performance characteristics, and approach for verification for constancy of performance. "European Assessment Document" for cast-in anchor channel covers assessment procedure for static, fatigue and fire. In this book, have not discussed fatigue and fire assessment for cast-in anchor channels. Based on the assessment, "European Technical Assessment (ETA)" report for that fastening system is issued. "European Technical Assessment" report documents at least the product description, its intended use (e.g. cracked or uncracked), recommended design method, installation procedure, and its essential performance characteristics in terms of characteristic load values. A sample ETA is given in Annex I. Though the sample given in Annex I is for post-installed anchor, it will help the readers understand the concept of ETA.

2.4.2 DESIGN STANDARDS

In Europe, cast-in anchor channel design provisions are laid out in "EN 1992-4 (Eurocode 2): Design of Concrete Structures - Part 4: Design of Fastenings for use in Concrete" [6]. This design standard covers design of cast-in anchor channels for only static or quasi-static loads. It is applicable for design of both structural and non-structural connections. It is important to note that it covers the design of cast-in anchor channels only, and not the fixture. The fixture i.e. baseplate has to be designed according to applicable standards like "IS 800 - General Construction in Steel – Code of Practice" [10]. The concrete member, in which the cast-in anchor channel is installed, should be designed and detailed as per applicable standards like "IS 456 - Plain and Reinforced Concrete – Code of Practice" [11]. The design provisions given in EN 1992-4 [6] can be used only if the product has been assessed as per applicable "European Assessment Document" and has an "European Technical Assessment" report. The design utilizes the characteristic values given in “European Technical Assessment” to determine the load carrying capacity.

2.5 DESIGN PROBLEM

Let us first clearly understand the design requirements even before looking at available design standards. To be able to identify the right solution, it is important to identify and list the factors that should be accounted for in design. The design problem statement should clearly identify the application, the environment it will be installed in and exposed to, the value and type of load, serviceability requirement, concrete member details etc.

An example of well-defined problem statement for forming connection using cast-in anchor channels would be: “Attachment of a steel base plate to a concrete floor of thickness 250 mm and Grade M50 for supporting façade structure in a high-rise building. The base plate is to be fixed on the top of 250 mm thick slab. The connection needs to be designed for a static dead load of 4 kN and static shear load of 10 kN due to wind force. Point of load application w.r.t. the top edge of concrete slab is 60 mm horizontally and 50 mm vertically. Due to time constraint, solution that eliminates drilling hole step is preferred.”

2.6 DESIGN PHILOSOPHY

The design of cast-in anchor channel is based on the premise that the tensile capacity of concrete can be utilized to transfer load into the concrete member which is supporting the attachment. An illustration is shown in Fig. 2.3 The design provisions presented in this chapter are based on EN 1992-4 [6]. As stated earlier, prequalification of the fastening system as per “European Assessment Document” is prerequisite for design. EN 1992-4 [6] covers design of cast-in anchor channels for only static or quasi-static loads. It does not address seismic and fatigue design of cast-in anchor channels. The design provisions are applicable only for designing connections with design life of 50 years. It does not impose restriction on no. of anchors for cast-in anchor channels.

It covers design of cast-in anchor channels in normal-weight concrete from M15 to M105[2], provided the anchor is prequalified for use in a specific concrete grade range within these limits. However, for the purpose of design the characteristic concrete strength f_{ck} shall not be taken higher than 60 MPa (cube strength), even if the concrete used in the structure is of higher grade (but in the range less than M105). In the design equations given in EN 1992-4 [6], the *cylinder* strength of concrete is used. Note that the EAD 330008-00-0601 [9] for "Anchor Channels" covers concrete range of M15 to M105.

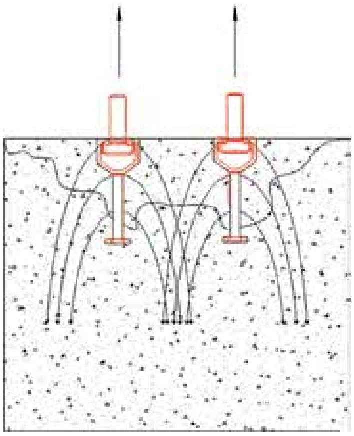


Fig.2.3 Illustration of tensile capacity of concrete being utilized for load transfer by cast-in anchor channel

As per EN 1992-4 [6], the connection being designed should be able to sustain all the design loads, not deform to inadmissible degree and remain fit for use, throughout its service life. Meaning, the ultimate limit state, the serviceability limit state and durability aspect should be addressed.

[2] In Europe the concrete grades are denoted differently than in India. For example, M15 grade would be denoted as C12/15. Basically, the first number is its cylinder strength (in this case 12 MPa) and the second number is its cube strength. One should also be cognizant of the difference in standards as well as method of measurement of compressive strength in the two countries.

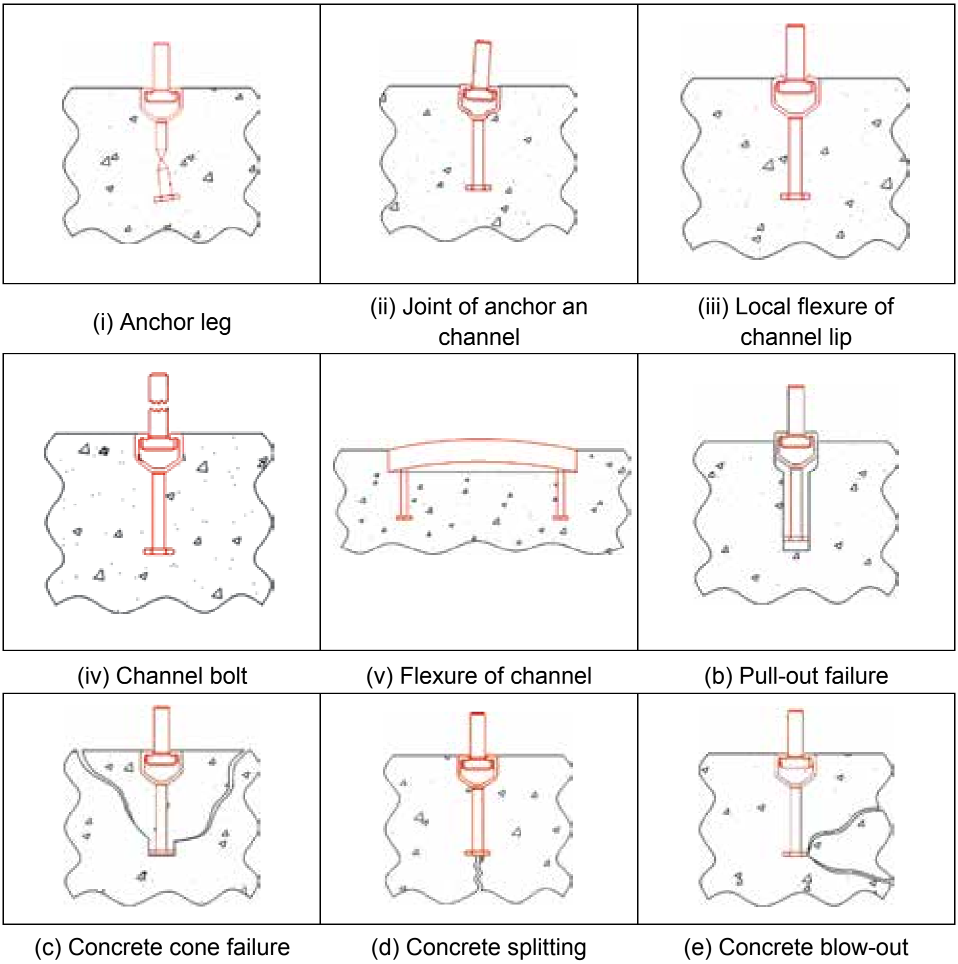
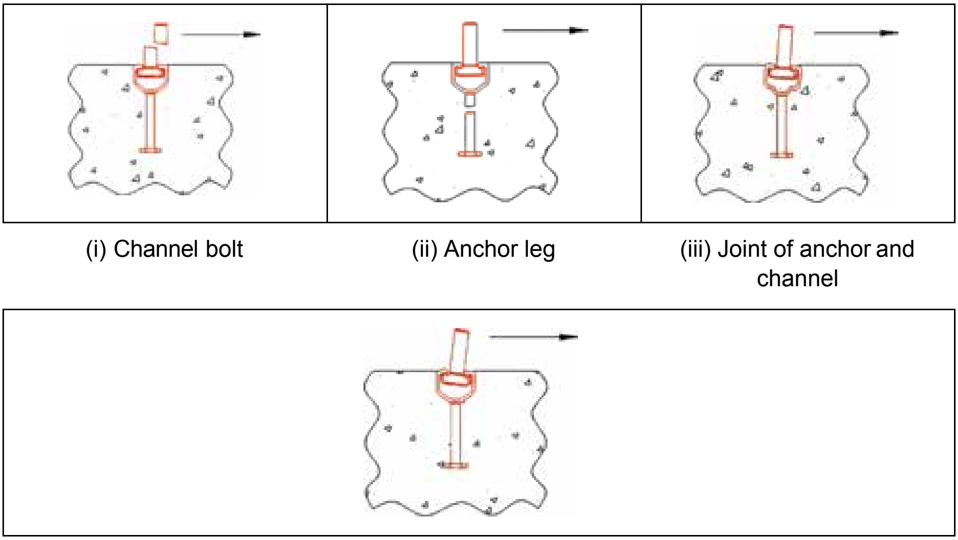


Fig. 2.4 Probable failure modes of cast-in anchor channels in tension
(resketched based on illustration in EN 1992-4 [6])



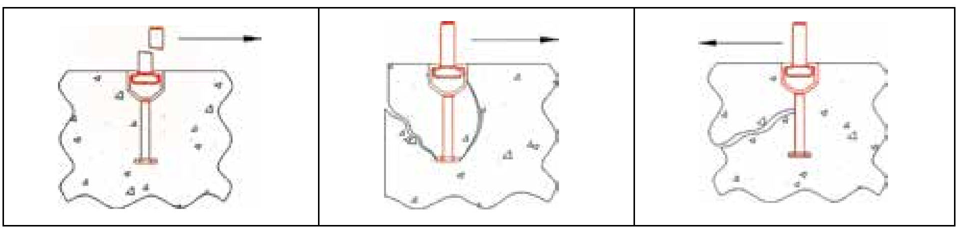
(i) Channel bolt

(ii) Anchor leg

(iii) Joint of anchor and channel

(iv) Local flexure of channel lip

Steel failure (Shear with lever arm)



(b) Steel failure of channel bolt (Shear with lever arm)

(c) Concrete pry-out

(d) Concrete edge failure

Fig. 2.5 Probable failure modes of cast-in anchor channels in shear (resketched based on illustration in EN 1992-4 [6])

To fulfil requirement of ultimate limit state, it shall be established that the design strength (R_d) of the fastening system is higher than or equal to the design load (E_d) i.e. $E_d \leq R_d$. This check needs to be carried out for all applicable load directions (tension, shear and both combined) and all expected failure modes. The probable failure types considered for design of cast-in anchor channels in tension are: - steel failure of fastener, concrete cone failure, pull-out failure of fastener, concrete splitting failure, concrete blow-out failure, steel failure of reinforcement and anchorage failure of reinforcement. The probable failure types considered for design of cast-in anchor channels in shear are: - steel failure of fastener (with or without lever arm), concrete pry-out failure, concrete edge failure, steel failure of supplementary reinforcement, anchorage failure of supplementary reinforcement. An illustration of each failure type is given in Fig. 2.4 and Fig. 2.5 for tension and shear, respectively (with exception of steel and anchorage failure of supplementary reinforcement).

To fulfil requirement of serviceability limit state, it shall be demonstrated that the displacements occurring under load are not larger than the admissible displacement. Aspects like corrosion protection, inspection, maintenance, replacement etc. address durability aspect.

2.7 LOAD ANALYSIS

To be able to carry out the checks for ultimate limit state, it is necessary to determine the force on each fastener. As per EN 1992-4 [6], the forces on fasteners are calculated as per elastic analysis and as per applicable structural standards. In general, the load on the fixture may be calculated ignoring the displacement of the anchor or anchor channels. However, the effect of this displacement should be considered when the fastener is used to fix a statically indeterminate stiff element.

EN 1992-4 [6] covers tension and shear acting perpendicular to its longitudinal axis. It does not address shear along channel's longitudinal axis. EOTA's Technical Report TR 17080 "Design of fastenings for use in concrete — Anchor channels — Supplementary rules" [12] addresses this aspect but is not covered in this book. In case of cast-in anchor channels with two anchor legs, the load on each anchor can be calculated by treating the channel as a simply supported beam and the spacing between anchor legs as its span length. In case of cast-in anchor channels with more than two anchor legs, triangular load distribution method may be used to determine the distribution of tension and shear loads to the anchors. These are simplistic assumptions which assumes partial restraint of the channel ends. The anchor stiffness and the degree of restraint assumed will affect the value of calculated resultant forces on the anchor. Likewise, the pressure distribution in the contact zone between channel and concrete will affect the shear load distribution.

2.7.1 ANALYSIS OF TENSION LOADS

The tension load acting on the anchor channel (N_{Ed}^{cb}) may be resolved into tension force acting on each anchor leg ($N_{Ed,i}^{a}$), by assuming a linear load distribution over its influence length (l_i), as per the following equation:

$$N_{Ed,i}^{a} = k \cdot A_i' \cdot N_{Ed}^{cb} \qquad \text{(Eq. 2.1a)}$$

where,

A_i' - is the ordinate at the position of the anchor “i” of a triangle with the unit height at the position of load N_{Ed}^{cb} and the base length $2l_i$.

$$k = 1/(\textstyle\sum_1^n A_i') \qquad \text{(Eq. 2.1b)}$$

$$l_i = 13 \cdot I_y^{0.05} \cdot s^{0.5} \geq s \qquad \text{(Eq. 2.1c)}$$

n - is the number of anchors on the channel within the influence length l_i on either side of the applied load N_{Ed}^{cb}.

s - is spacing of anchor legs.

I_y - is moment of inertia of the anchor channel w.r.t y-axis of the channel. This value is usually provided in Technical Assessment report like ETA

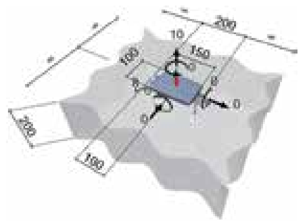
If the anchor channel is subjected to multiple tension loads simultaneously, the force on anchor leg may be determined for each load and the forces thus determined may be superimposed to determine the resultant force due to multiple tension loads. The most unfavourable position of the load for a given failure mode may be used for calculating load on each anchor if the exact position of the load on the anchor channel is not known. For example, the load is assumed to be directly acting over the anchor for anchor steel rupture failure. The tension load N_{Ed}^{cb} may also be resolved into design bending moment M_{Ed}^{ch} acting on the anchor channel by assuming a simply supported beam with a span length equal to the anchor spacing. This is again a simplistic assumption which neglects several influencing parameters like end restraints.

Examples 2.1 to 2.3 illustrate typical calculations necessary for determining tension force on anchor leg due to the applied load.

EXAMPLE 2.1

Determine the tension force acting on each anchor leg for the following connection formed using anchor channel HAC 40, 200 mm long (ETA-11/0006 [13]) with M12 x 40 mm HBC-C 8.8F T-bolt. Note – Anchor leg spacing is 150 mm and 10 kN tension load is acting on channel at 100 mm from one end.

Given:



Anchor leg spacing $s = 150\ \mathrm{mm}$

Point of load application from channel end $= 100\ \mathrm{mm}$

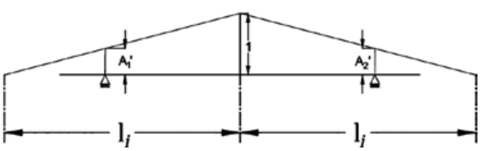
$N_{Ed}^{cb} = 10\ \mathrm{kN}$

Solution:

$I_y = 21463\ \mathrm{mm}^4$ (from Table 1 of ETA 11/0006)

$l_i = 13 \cdot I_y^{0.05} \cdot s^{0.5} \geq s$

$l_i = 13 \cdot (21463)^{0.05} \cdot (150)^{0.5} \approx 262\ \mathrm{mm}$ [$\geq s = 150\ \mathrm{mm}$. OK]



$A'_1 = 0.29$

$A'_2 = 0.29$

$$k = \frac{1}{\sum_1^n A'_i} = \frac{1}{A'_1 + A'_2} = \frac{1}{0.29 + 0.29} = 1.75$$

$$N^a_{Ed,1} = k \cdot A'_1 \cdot N^{cb}_{Ed} = 1.75 * 0.29 * 10 = 5 \text{ kN}$$

$$N^a_{Ed,2} = k \cdot A'_2 \cdot N^{cb}_{Ed} = 1.75 * 0.29 * 10 = 5 \text{ kN}$$

➔ Therefore, tension force of 5 kN is action on each anchor leg.

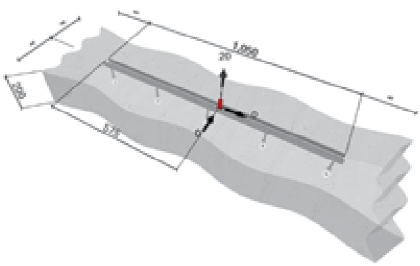
EXAMPLE 2.2

Determine the tension force acting on each anchor leg for the following connection formed using anchor channel HAC 40, 1050 mm long (ETA-11/0006 [13]) with M12 x 40 mm HBC-C 8.8F T-bolt. Note – Anchor leg spacing is 250 mm and 20 kN tension load is acting on channel at 575 mm from one end.

Given:

Anchor leg spacing $s = 250$ mm

Point of load application from channel end $= 575$ mm



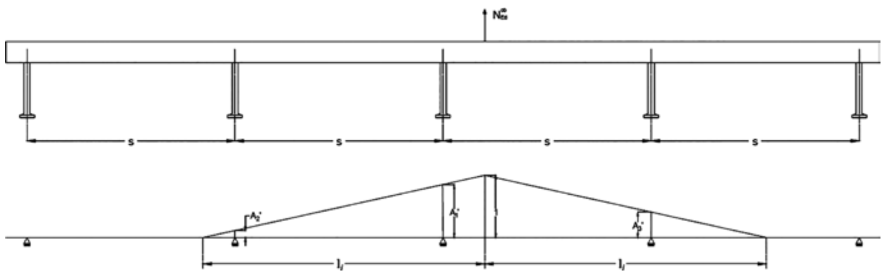
$N^{cb}_{Ed} = 20\ kN$

Solution:

$I_y = 21463 \text{ mm}^4$ (from Table 1 of ETA-11/0006)

$l_i = 13 \cdot I_y^{0.05} \cdot s^{0.5} \geq s$

$l_i = 13 \cdot (21463)^{0.05} \cdot (150)^{0.5} \approx 262 \text{ mm}$ [$\geq s = 150$ mm. OK]



$A_1' = 0.85$

$A_2' = 0.11$

$A_3' = 0.41$

$$k = \frac{1}{\sum_1^n A_i'} = \frac{1}{A_1' + A_2' + A_3'} = \frac{1}{0.85+0.11+0.41} = 0.73$$

$$N_{Ed,1}^{a} = k \cdot A_1' \cdot N_{Ed}^{cb} = 0.73 * 0.85 * 20 = 12.397 \text{ kN}$$

$$N_{Ed,2}^{a} = k \cdot A_2' \cdot N_{Ed}^{cb} = 0.73 * 0.11 * 20 = 1.653 \text{ kN}$$

$$N_{Ed,3}^{a} = k \cdot A_3' \cdot N_{Ed}^{cb} = 0.73 * 0.41 * 20 = 5.950 \text{ kN}$$

➔ Therefore, tension force of approximately 12.4 kN is acting on anchor leg 2, 1.7 kN is acting on anchor leg 3 and 6 kN is acting on anchor leg 4. No tension load is acting on anchor leg 1 and 5.

EXAMPLE 2.3

Determine the tension force acting on each anchor leg for the following connection formed using anchor channel HAC 40, 1050 mm long (ETA-11/0006 [13]) with M12 x 40 mm HBC-C 8.8F T-bolt. Note – Anchor leg spacing is 250 mm and 10 kN tension load is acting at 550 mm from one end and 8 kN tension load at 350 mm.

Given:

Anchor leg spacing $s = 250$ mm

Point of load application $N^{cb}_{Ed,1}$ from channel end $= 550$ mm

$N^{cb}_{Ed,1} = 10$ kN

Point of load application $N^{cb}_{Ed,2}$ from channel end $= 350$ mm

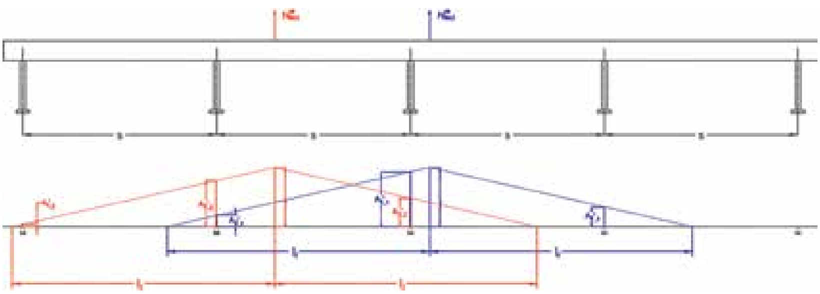
$N^{cb}_{Ed,2} = 8$ kN

Solution:

$I_y = 21463 \text{ mm}^4$ (from Table 1 of ETA-11/0006)

$l_i = 13 \cdot I_y^{0.05} \cdot s^{0.5} \geq s$

$l_i = 13 \cdot (21463)^{0.05} \cdot (150)^{0.5} \approx 262$ mm [$\geq s = 150$ mm. OK]



$A'_{2,1} = 0.19$

$A'_{3,1} = 0.93$

$A'_{4,1} = 0.34$

$$k_1 = \frac{1}{\sum_1^n A'_{i,1}} = \frac{1}{A'_{2,1} + A'_{3,1} + A'_{4,1}} = \frac{1}{0.19+0.93+0.34} = 0.69$$

$A'_{1,2} = 0.04$

$A'_{2,2} = 0.78$

$A'_{3,2} = 0.48$

$$k_2 = \frac{1}{\sum_1^n A'_{i,2}} = \frac{1}{A'_{1,2} + A'_{2,2} + A'_{3,2}} = \frac{1}{0.04+0.78+0.48} = 0.77$$

$$N^a_{Ed,1} = k_2 \cdot A'_{1,2} \cdot N^{cb}_{Ed,2} = 0.244 \text{ kN}$$

$$N^a_{Ed,2} = (k_2 \cdot A'_{2,1} \cdot N^{cb}_{Ed,1}) + (k_2 \cdot A'_{2,2} \cdot N^{cb}_{Ed,2}) = 6.080 \text{ kN}$$

$$N^a_{Ed,3} = (k_1 \cdot A'_{3,1} \cdot N^{cb}_{Ed,1}) + (k_2 \cdot A'_{3,2} \cdot N^{cb}_{Ed,2}) = 9.362 \text{ kN}$$

$$N^a_{Ed,4} = k_1 \cdot A'_{4,1} \cdot N^{cb}_{Ed,1} = 2.314 \text{ kN}$$

➔ Therefore, tension force of approximately 0.24 kN is acting on anchor leg 1, 6.1 kN is acting on anchor leg 2, 9.4 kN is acting on anchor leg 3 and 2.3 kN is acting on anchor leg 4. No tension load is acting on anchor leg 5.

2.7.2 ANALYSIS OF SHEAR LOADS

The shear load acting perpendicular to the channel's longitudinal axis may be resolved into forces acting on each anchor leg using the concept described in Section 2.7.1. This is again a simplified approach which assumes that the forces are transferred to the anchor legs by bending of the channel due to shear load and then from anchors legs into the concrete. The shear loads acting away from the edge are not considered while calculating forces on anchor for checking concrete edge failure.

For design, it is important to determine whether the shear load is acting with or without a lever arm on the channel bolt. The shear load on the channel bolt may be assumed to be acting without a lever arm if the fixture is made out of steel and is in contact with the fastener over a length of at least 0.5 times the fixture thickness (i.e. $0.5t_{fix}$), and is fixed either directly to the concrete without an intermediate layer or is fixed using a levelling mortar with strength equivalent to that of the concrete base material (not less than 30 N/mm^2) and with a thickness less than or equal to 0.5 times the anchor diameter (i.e. $0.5d$), under at least the full dimensions of the fixture on a rough concrete surface as intermediate layer. If this condition is not fulfilled, then shear force is assumed to act with lever arm.

Some exceptions to this requirement are given in EN 1992-4 [6] but are not discussed in this book.

When the shear force (V_{Ed}) is acting with lever arm (l_a) as illustrated by Fig. 2.6, the bending moment (M_{Ed}) acting on the fastener is calculated as follows:

$M_{Ed} = V_{Ed}(l_a/\alpha_M)$ (Eq. 2.2a)

where,

α_M – is a factor that is dependent on the degree of restraint of the anchor at the side of the fixture. For no restraint (i.e. fixture can rotate freely), $\alpha_M = 1.0$. For full restraint (i.e. fixture cannot rotate), $\alpha_M = 2.0$.

$l_a = a_3 + e_1$ (Eq. 2.2b)

a_3 – is equal to zero if the washer and nut are directly clamped to concrete surface or to the surface of anchor channel or if a levelling grout (as per strength and thickness stated above) is used. For all other cases, it is equal to 0.5 times the nominal diameter of the anchor.

e_1 – is the distance between shear load and concrete surface. The grout thickness is ignored for calculating this distance.

Examples 2.4 and 2.5 illustrate typical calculations necessary for determining shear force on the anchor leg due to the applied load.

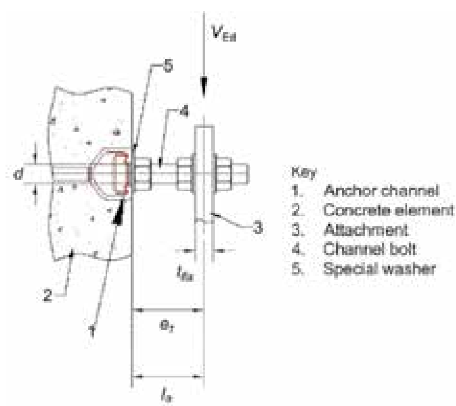
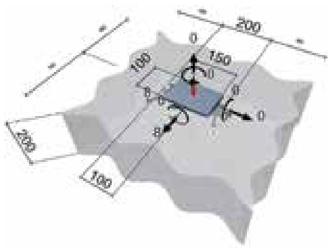


Fig. 2.6 Lever arm for shear – Stand-off installation with anchor channel (resketched based on illustration in EN 1992-4 [6])

EXAMPLE 2.4

Determine the shear force acting on each anchor leg for the following connection formed using anchor channel HAC 40, 200 mm long (ETA-11/0006 [13]) with M12 x 40 mm HBC-C 8.8F T-bolt. Note – Anchor leg spacing is 150 mm and 8 kN shear load is acting at 100 mm from one end.

Given:



Anchor leg spacing $s = 150\ \text{mm}$

Point of load application from channel end $= 100\ \text{mm}$

$V_{Ed}^{cb} = 8\ kN$

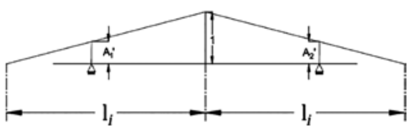
Solution:

$I_y = 21463\ \text{mm}^4$ (from Table 1 of ETA-11/0006)

$l_i = 13 \cdot I_y^{0.05} \cdot s^{0.5} \geq s$

$l_i = 13 \cdot (21463)^{0.05} \cdot (150)^{0.5} \approx 262\ \text{mm}$ [$\geq s = 150\ \text{mm}$. OK]

Load distribution scheme based on influence length, l_i



$A_1' = 0.29$

$A_2' = 0.29$

$$k = \frac{1}{\sum_1^n A_i'} = \frac{1}{A_1' + A_2'} = \frac{1}{0.29+0.29} = 1.75$$

$$V_{Ed,1}^{a} = k \cdot A_1' \cdot V_{Ed}^{cb} = 1.75 * 0.29 * 8 = 4\ \text{kN}$$

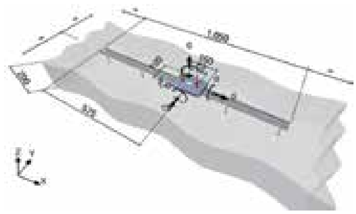
$$V_{Ed,2}^{a} = k \cdot A_2' \cdot V_{Ed}^{cb} = 1.75 * 0.29 * 8 = 4\ \text{kN}$$

➔ Therefore, shear force of 4 kN is acting on each anchor leg.

EXAMPLE 2.5:

Determine the shear force acting on each anchor leg for the following connection formed using anchor channel HAC 40, 1050 mm long (ETA-11/0006 [13]) with M12 x 40 mm HBC-C 8.8F T-bolt. Note – Anchor leg spacing is 250 mm and 15 kN shear load is acting at 575 mm from one end.

Given:



Anchor leg spacing $s = 250\ \text{mm}$

Point of load application
from channel end $\quad = 575\ \text{mm}$

$V_{Ed}^{cb} = 15\ \text{kN}$

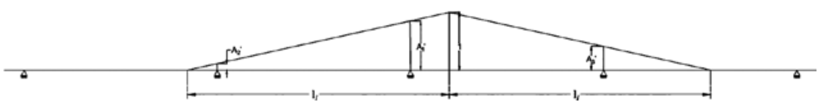
Solution:

$I_y = 21463\ \text{mm}^4$ (from Table 1 of ETA-11/0006)

$I_y = 21463$

$l_i = 13 \cdot I_y^{0.05} \cdot s^{0.5} \geq s$

$l_i = 13 \cdot (21463)^{0.05} \cdot (150)^{0.5} \approx 262\ \text{mm}$ [$\geq s = 150\ \text{mm}$. OK]



Load distribution scheme based on influence length, l_i

$A'_1 = 0.85$

$A'_2 = 0.11$

$A'_3 = 0.41$

$k = \frac{1}{\sum_1^n A'_i} = \frac{1}{A'_1 + A'_2 + A'_3} = \frac{1}{0.85+0.11+0.41} = 0.73$

$V_{Ed,1}^{a} = k \cdot A'_1 \cdot N_{Ed}^{cb} = 0.73 * 0.85 * 15 = 9.298\ \text{kN}$

$V^{a}_{Ed,2} = k \cdot A'_{2} \cdot N^{cb}_{Ed} = 0.73 * 0.11 * 15 = 1.239 \text{ kN}$

$V^{a}_{Ed,3} = k \cdot A'_{3} \cdot N^{cb}_{Ed} = 0.73 * 0.41 * 15 = 4.463 \text{ kN}$

➔ Therefore, shear force of approximately 9.3 kN is acting on anchor leg 2, 1.2 kN is acting on anchor leg 3 and 4.5 kN is acting on anchor leg 4. No shear force is acting on anchor leg 1 and 5.

2.7.3 ANALYSIS OF FORCES ON SUPPLEMENTARY REINFORCEMENT

Strut-and-tie model is used to determine the design tension and the design shear that the supplementary reinforcement will be subjected to.

(a) Analysis of tension force on supplementary reinforcement.

It is possible to design supplementary reinforcement for a single anchor leg as well as anchor leg group. The supplementary reinforcement of all anchor legs has to be designed for the force acting on the most loaded anchor leg (N^{a}_{Ed}).

(b) Analysis of shear force on supplementary reinforcement.

The supplementary reinforcement of all anchor legs of an anchor channel is designed for the greater of the two – shear on the most loaded anchor leg and shear on the most loaded channel bolt. The design shear force V_{Ed} acting perpendicular to the fixture towards the edge may be resolved into design tension force $N_{Ed,re}$ acting on the supplementary reinforcement provided in the direction of shear, as follows:

$$N_{Ed,re} = [(e_s/z) + 1] \cdot V_{Ed} \quad \text{(Eq. 2.3a)}$$

where,

e_s - is the distance between axis of reinforcement and line of shear force acting on the fixture (see Fig. 2.7).

$$z \approx 0.85d \quad \text{with } d < min(2h_{ef}, c_1) \quad \text{(Eq. 2.3b)}$$

In Eq. 2.3b, h_{ef} is the effective embedment and c_1 is the edge distance.

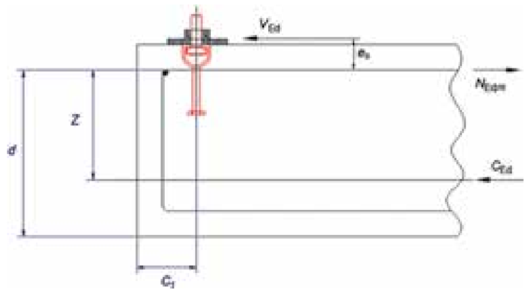


Fig.2.7 Forces in the supplementary (surface) reinforcement due to shear (resketched based on illustration in EN 1992-4 [6])

If the channel bolts of a fixture are subjected to different shear load values, then the design tension force ($N^h_{Ed,re}$) to be considered for design of the supplementary reinforcement of all fasteners shall correspond to shear load of the most loaded channel bolt (V^h_{Ed}). If the supplementary reinforcement is not provided in the direction of the shear force, then this should be accounted for while determining design tension force on the reinforcement to maintain equilibrium in the strut and tie model. Conservatively, the shear may be assumed to be acting perpendicular to the channel and towards the edge for design of supplementary reinforcement, irrespective of whether it is actually inclined or parallel.

2.8 DESIGN FOR STATIC OR QUASI-STATIC LOADS

If a connection formed using cast-in anchor channels supported with supplementary reinforcement is subjected to static loads increasing from zero to the stage it may fail, there are 11 different ways in which it may fail in tension and 8 different ways in which it may fail in shear (as shown in Fig. 2.4 and Fig. 2.5) depending upon which one is the weakest. Design strength is therefore determined by dividing characteristic resistance to each failure, by recommended partial safety factor (see Fig. 2.8). This design strength (R_d) is compared to design load (E_d) to establish $E_d \leq R_d$, for all applicable load directions (tension, shear and both combined) and all expected failure modes.

Failure type related to steel failure – anchor channels	Partial factor for permanent and transient design situations
Tension in anchor legs and channel bolts	$\gamma_{Ms} = 1.2\, f_{uk} / f_{yk} \geq 1.4$
Shear with and without lever arm in channel bolts	$\gamma_{Ms} = \left\{ \begin{matrix} f_{uk} / f_{yk} \geq 1.25 & When\, f_{uk} \leq 800\ N/mm^2 and\, f_{uk} / f_{yk} \leq 1.25 \\ 1.5 & When\, f_{uk} \leq 800\ N/mm^2 and\, f_{uk} / f_{yk} \leq 1.25 \end{matrix} \right\}$
Joint between anchor legs and channel in tension and shear	$\gamma_{Ms,ca} = 1.8$
Local failure of anchor channel by bending of lips in tension and shear	$\gamma_{Ms,l} = 1.8$

Failure type related to concrete related failure	**Partial factor for permanent and transient design situations**
ending of channel	$\gamma_{Ms,flex} = 1.15$
Concrete cone failure, concrete edge failure, concrete blow-out failure and concrete pry-out failure	$\gamma_{Mc} = \gamma_c \gamma_{inst}$ where, $\gamma_c = 1.5$ * $\gamma_{inst} \begin{cases} = 1 & for\ headed\ fasteners\ and\ anchor\ channels\ ** \\ \geq 1 & for\ post-installed\ fasteners\ in\ tension;\ also\ see\ ETA \\ = 1 & for\ post-installed\ fasteners\ in\ shear \end{cases}$ * This value is as per EN 1992-1-1 [14]; for seismic repair & strengthening of existing structures refer Eurocode 8 [15] * *as per EN 1992-4 [6] section 4.6 in tension & shear
Concrete splitting failure	$\gamma_{Msp} = \gamma_{Mc}$
Pull-out and bond failure	$\gamma_{Mp} = \gamma_{Mc}$

Fig. 2.8 Excerpt of Table 4.1 of EN 1992-4 – Partial safety factors EN 1992-4 [6] [Note - Refer EN 1992-4 for complete table]

2.8.1 DESIGN CHECKS FOR TENSION LOAD

(a) Resistance to steel failure

As per EN 1992-4 [6], the design steel strength in tension (N_{Rd}) is determined by dividing characteristic steel resistance by recommended partial material safety factor. The characteristic values for resistance to different forms of steel failure of cast-in anchor channel should be taken from the product's Technical Assessment report like ETA. The design steel strength should be greater than the design load (N_{Ed}). The design tension load on the anchor leg is denoted as N^{a}_{Ed} and on the channel bolt is denoted as N^{cb}_{Ed}.

For steel failure of anchor leg, the design steel strength of anchor leg, $N_{Rd,s,a}$, is calculated by dividing characteristic resistance of anchor leg to steel failure, $N_{Rk,sa}$, by partial safety factor, γ_{Ms}, as follows. This check is carried out for the most unfavourably loaded anchor leg.

$$N_{Rd,s,a} = N_{Rk,s,a} / \gamma_{Ms} \geq N^{a}{}_{Ed} \tag{Eq. 2.4a}$$

For steel failure of joint between the anchor leg and the channel, the design steel strength of joint, $N_{Rd,s,c}$, is calculated by dividing characteristic resistance of joint to steel failure, $N_{Rk,s,c}$, by partial safety factor, $\gamma_{Ms,ca}$, as follows. This check is carried out for the most unfavorably loaded anchor leg.

$$N_{Rd,s,c} = N_{Rk,s,c} / \gamma_{Ms,ca} \geq N^{a}{}_{Ed} \tag{Eq. 2.4b}$$

For local flexure of channel lip, the design steel strength of channel lip, $N_{Rd,s,l}$, is calculated by dividing characteristic resistance of channel lip to flexure, $N_{Rk,s,l}$, by partial safety factor, $\gamma_{Ms,l}$, as follows. This check is carried out for the most unfavourably loaded channel bolt.

$$N_{Rd,s,l} = N_{Rk,s,l} / \gamma_{Ms,l} \geq N^{cb}_{Ed} \tag{Eq. 2.4c}$$

where,

$$N_{Rk,s,l} = N^{0}_{Rk,s,l} \cdot \psi_{l,N} \qquad \text{Eq. 2.4c(i))}$$

$$\psi_{l,N} = 0.5(1 + s_{cbo}/s_{l,N}) \leq 1 \qquad \text{(Eq. 2.4c(ii))}$$

$N^0_{Rk,s,l}$ - is the basic value for local failure by flexure of channel lips and is taken from product's Technical Assessment report like ETA.

s_{cbo} - is the spacing between channel bolts.

$s_{l,N}$ - is the characteristic spacing for channel lip flexure under tension load and is taken from product's Technical Assessment report like ETA. If this value is not provided in the report then conservatively it may be taken as $2b_{ch}$; where b_{ch} is the channel width.

For steel failure of channel bolt, the design steel strength of channel bolt, $N_{Rd,s}$, is calculated by dividing characteristic resistance of channel bolt, $N_{Rk,s}$, by partial safety factor, γ_{Ms}, as follows. This check is carried out for the most unfavourably loaded channel bolt.

$$N_{Rd,s} = N_{Rk,s} / \gamma_{Ms} \geq N^{cb}_{Ed} \quad \text{(Eq. 2.4d)}$$

For flexure of channel, the design steel strength of channel, $M_{Rd,s,flex}$, is calculated by dividing characteristic resistance of channel lip to flexure, $M_{Rk,s,flex}$, by partial safety factor, $\gamma_{Ms,flex}$, as follows. This check is carried out for the moment on channel.

$$M_{Rd,s,flex} = M_{Rk,s,flex} / \gamma_{Ms,flex} \geq M^{cb}_{Ed} \quad \text{Eq. 2.4e)}$$

Examples 2.6 and 2.7 illustrate typical calculations necessary for checking resistance to steel failure in tension.

EXAMPLE 2.6

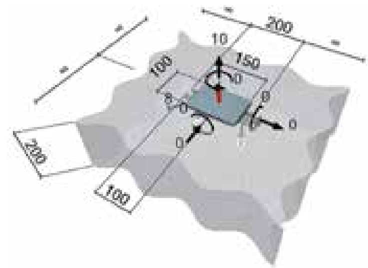
Check resistance to steel failure in tension for the following connection formed using anchor channel HAC 40, 200 mm long (ETA-11/0006 [13]) with M12 x 40 mm HBC-C 8.8F T-bolt. Note – Anchor leg spacing is 150 mm and 10 kN tension load is acting at 100 mm from one end.

Given:

Anchor leg spacing $s = 150\ \mathrm{mm}$

Point of load application from channel end= $100\ \mathrm{mm}$

$N_{Ed}^{cb} = 10\ \mathrm{kN}$



Solution:

$N_{ED,1}^{a} = 5\ \mathrm{kN}$ [from Ex. 2.1]

$N_{ED,2}^{a} = 5\ \mathrm{kN}$ [from Ex. 2.1]

Check design steel resistance in tension

a) Anchor leg:

$N_{Rk,s,a} = 33.1\ \mathrm{kN}$ (from Table 12 of ETA-11/0006)

$\gamma_{Ms} = 1.8$ (from Table 12 of ETA-11/0006)

$N_{Rd,s,a} = N_{Rk,s,a} / \gamma_{Ms} = 33.1/1.8 = 18.39\ \mathrm{kN} \geq N^{a}_{Ed}$

$\beta = N_{Ed}^{a} / N_{Rd,s,a} = 5/18.39 = 0.272$ → 27.2% utilization. OK

b) Joint of anchor leg to channel:

$N_{Rk,s,c} = 25\ \mathrm{kN}$ (from Table 12 of ETA-11/0006)

$\gamma_{Ms,ca} = 1.8$ (from Table 12 of ETA-11/0006)

$N_{Rd,s,c} = N_{Rk,s,c} / \gamma_{Ms,ca} = 25/1.8 = 18.39\ \mathrm{kN} \geq N^{a}_{Ed}$

$\beta = N_{Ed}^{a} / N_{Rd,s,c} = 5/13.9 = 0.360$ → 36% utilization. OK

c) Local flexure of channel lip:

$N_{Rk,s,l} = 25\ \mathrm{kN}$ (from Table 12 of ETA-11/0006)

$\gamma_{Ms,l} = 1.8$ (from Table 12 of ETA-11/0006)

$N_{Rd,s,l} = N_{Rk,s,l} / \gamma_{Ms,l} = 25/1.8 = 13.9\ \mathrm{kN} \geq N^{cb}_{Ed}$

$\beta = N_{Ed}^{cb} / N_{Rd,s,l} = 10/13.9 = 0.720$ → 72% utilization. OK

d) Channel bolt:

$N_{Rk,s} = 67.4$ kN (from Table 22 of ETA-11/0006)

$\gamma_{Ms} = 1.5$ (from Table 22 of ETA-11/0006)

$N_{Rd,s} = N_{Rk,s} / \gamma_{Ms} = 67.4/1.5 = 44.93\ kN \geq N^{cb}{}_{Ed}$

$\beta = N^{cb}_{Ed}/N_{Rd,s} = 10/44.93 = 0.185$ → 22.2% utilization. OK

e) Flexure of channel:

$M_{Rk,s,flex} = 1.136$ kN (from Table 13 of ETA-11/0006)

$\gamma_{Ms,flex} = 1.8$ (from Table 13 of ETA-11/0006)

f) Determination of acting moment based on single supported beam:

$M^{cb}_{Ed} = 5\ \text{kN} * 0.075\ \text{m} = 0.375\ \text{kN} - \text{m}$

$M_{Rd,s,flex} = M_{Rk,s,flex} / \gamma_{Ms,flex} = 1.136/1.15 = 0.987\ \text{kN} \geq M^{cb}{}_{Ed}$

$\beta = M^{cb}_{Ed}/M_{Rd,s,flex} = 0.375/0.987 = 0.380$ → 38% utilization. OK

EXAMPLE 2.7

Check resistance to steel failure in tension for the following connection formed using anchor channel HAC 40, 200 mm long (ETA-11/0006 [13]) with M12 x 40 mm HBC-C 8.8F T-bolt. Note – Anchor leg spacing is 150 mm and 1.2 kN-m moment is acting on 200 mm x 200 mm base plate bolted with channel using channel bolt.

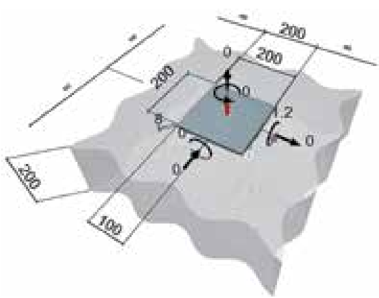
Given:

Anchor leg spacing $s = 150$ mm

Point of load application from channel end $= 100$ mm

$M_{Ed} = 1.2$ kN − m

Length of plate, $L_{plate} = 200$ mm



Solution:

Tension load in channel bolt due to moment, $N_{Ed}^{cb} = M_{Ed}/(L_{plate}/2) = 1.2/0.1 = 12\ \text{kN}$

$I_y = 21463\ \text{mm}^4$ (from Table 1 of ETA-11/0006)

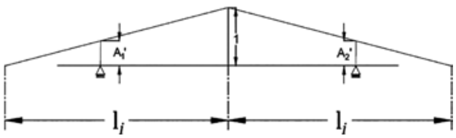
$l_i = 13 \cdot I_y^{0.05} \cdot s^{0.5} \geq s$

$l_i = 13 \cdot (21463)^{0.05} \cdot (150)^{0.5} \approx 262\ \text{mm}$ [$\geq s = 10\ \text{mm}$. OK]

Load distribution scheme based on influence length, l_i

$A'_1 = 0.29$

$A'_2 = 0.29$



Weighting factor k

$k = \frac{1}{\sum_1^n A'_i} = \frac{1}{A'_1 + A'_2} = \frac{1}{0.29+0.29} = 1.75$

$N_{Ed,1}^{a} = k \cdot A'_1 \cdot N_{Ed}^{cb} = 1.75 * 0.29 * 12 = 6\ \text{kN}$

$N_{Ed,2}^{a} = k \cdot A'_2 \cdot N_{Ed}^{cb} = 1.75 * 0.29 * 12 = 6\ \text{kN}$

Check design steel resistance in tension

a) Anchor leg:

$N_{Rk,s,a} = 33.1\ \text{kN}$ (from Table 12 of ETA-11/0006)

$\gamma_{Ms} = 1.8$ (from Table 12 of ETA-11/0006)

$N_{Rd,s,a} = N_{Rk,s,a}/\gamma_{Ms} = 33.1/1.8 = 18.39\ \text{kN} \geq N^a{}_{Ed}$

$\beta = N_{Ed}^{a}/N_{Rd,s,a} = 6/18.39 = 0.326$ → 32.6% utilization. OK

b) Joint of anchor leg to channel:

$N_{Rk,s,c} = 25\ \text{kN}$ (from Table 12 of ETA-11/0006)

$\gamma_{Ms,ca} = 1.8$ (from Table 12 of ETA-11/0006)

$N_{Rd,s,c} = N_{Rk,s,c}/\gamma_{Ms,ca} = 25/1.8 = 18.39\ \text{kN} \geq N^a{}_{Ed}$

$\beta = N_{Ed}^{a}/N_{Rd,s,c} = 6/13.9 = 0.431$ → 43.1% utilization. OK

c) Local flexure of channel lip:

$N_{Rk,s,l} = 25$ kN (from Table 12 of ETA-11/0006)

$\gamma_{Ms,l} = 1.8$ (from Table 12 of ETA-11/0006)

$N_{Rd,s,l} = N_{Rk,s,l} / \gamma_{Ms,l} = 25/1.8 = 13.9 \text{ kN} \geq N^{cb}_{Ed}$

$\beta = N^{cb}_{Ed}/N_{Rd,s,l} = 12/13.9 = 0.863$ → 86.3% utilization. OK

d) Channel bolt:

$N_{Rk,s} = 67.4$ kN (from Table 22 of ETA-11/0006)

$\gamma_{Ms} = 1.5$ (from Table 22 of ETA-11/0006)

$N_{Rd,s} = N_{Rk,s} / \gamma_{Ms} = 67.4/1.5 = 44.93 \text{ kN} \geq N^{cb}_{Ed}$ (from ETA-11/0006)

$\beta = N^{cb}_{Ed}/N_{Rd,s} = 12/44.93 = 0.267$ → 26.7% utilization. OK

e) Flexure of channel:

$M_{Rk,s,flex} = 1.136$ kN (from Table 13 of ETA-11/0006)

$\gamma_{Ms,flex} = 1.8$ (from Table 13 of ETA-11/0006)

Determination of acting moment based on single supported beam:

$M^{cb}_{Ed} = 6 \text{ kN} * 0.075 \text{ m} = 0.450 \text{ kN} - \text{m}$

$M_{Rd,s,flex} = M_{Rk,s,flex} / \gamma_{Ms,flex} = 1.136/1.15 = 0.987 \text{ kN} \geq M^{cb}{}_{Ed}$

$\beta = M^{cb}_{Ed}/M_{Rd,s,flex} = 0.45/0.987 = 0.455$ → 45.5% utilization. OK

(b) Resistance to anchor leg pull-out

As per EN 1992-4 [6], the design pull-out strength of the anchor leg in tension ($N_{Rd,p}$) is determined by dividing characteristic pull-out resistance, $N_{Rk,\mathrm{p}}$, by recommended partial safety factor, γ_{Mp}, as shown in Eq. 2.5. The check for resistance to pull-out failure is carried out for the most unfavourably loaded anchor leg.

$$N_{Rd,\mathrm{p}} = N_{Rk,p} / \gamma_{Mp} \geq N^{a}_{Ed} \qquad \text{(Eq. 2.5)}$$

where,

$N_{Rk,p}$ – is taken from the product's Technical Assessment report like ETA and is limited by the concrete pressure under the head of the anchor leg equal to $k_2 \cdot A_h \cdot f_{ck}$.

A_h - is the load bearing area of the headed anchor leg and is equal to $(d_h^2 - d_a^2)\pi/4$ for circular shaped heads. d_h and d_a are diameter of anchor head and anchor leg, respectively. For determining A_h, d_h should not be taken larger than $(6t_h + d)$.; where t_h is the thickness of diameter head and d is diameter of fastener.

k_2 - is taken as 7.5 and 10.5 for fasteners in cracked and uncracked concrete, respectively.

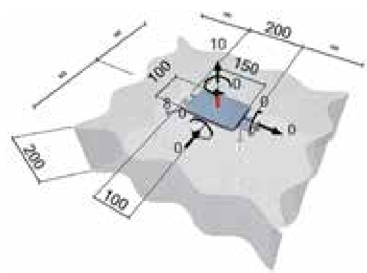
f_{ck} - is nominal characteristic compressive cylinder strength of concrete.

Example 2.8 illustrates typical calculations necessary for checking resistance to pull-out failure in tension.

EXAMPLE 2.8

Check resistance to pull-out failure for the following connection formed using anchor channel HAC 40, 200 mm long (ETA-11/0006 [13]) with M12 x 40 mm HBC-C 8.8F T-bolt. The base material is of M25 grade concrete. Note – Anchor leg spacing is 150 mm and 10 kN tension load is acting at 100 mm from one end. Assume the concrete to be cracked for design.

Given:



Anchor leg spacing $s = 150$ mm

Point of load application from channel end $= 100$ mm

$N_{Ed}^{cb} = 10$ kN

Solution:

$N_{Ed,1}^{a} = 5$ kN (from Ex. 2.1)

$N_{Ed,2}^{a} = 5$ kN (from Ex. 2.1)

Check design concrete pull-out resistance in tension

$N_{Rk,p} = 18.8\ \mathrm{kN}$ (from Table 14 of ETA-11/0006)

$\gamma_{Mc,p} = 1.5$ (from Table 14 of ETA-11/0006)

Amplification factor $\Psi_c = 1.67$ (from Table 14 of ETA-11/0006)

$N_{Rd,\mathrm{p}} = (N_{Rk,p} * \Psi_c)/\gamma_{Mp}$

$N_{Rd,\mathrm{p}} = (18.8 * 1.67)/1.5 = 20.9\mathrm{kN} \geq N^a_{Ed} = 5\ \mathrm{kN}$

$\beta = N^a_{Ed}/N_{Rd,\mathrm{p}} = 5/20.9 = 0.238$ → 24% utilization. OK

(c) Resistance to concrete cone failure

The design concrete cone strength of the anchor leg in tension ($N_{Rd,c}$) is determined by dividing characteristic concrete cone resistance, $N_{Rk,\mathrm{c}}$, by recommended partial safety factor, γ_{Mc}, as shown in Eq. 2.6. The check for resistance to concrete cone failure is carried out for the most unfavourably loaded anchor leg. The load on the anchor leg along with edge and spacing of anchor leg should be taken into consideration for determining the most unfavourably loaded anchor leg.

$$N_{Rd,\mathrm{c}} = N_{Rk,c}/\gamma_{Mc} \geq N^a_{Ed} \qquad \text{(Eq. 2.6)}$$

The effective embedment depth to be used for determining value of characteristic concrete cone resistance to be used in Eq. 2.7 is dependent on the dimensions of the anchor channel. If the ratio[3] $h_{ch}/h_{ef} \leq 0.4$ and $b_{ch}/h_{ef} \leq 0.7$ then the effective embedment depth is considered from the anchor lip to the anchor head as shown in Fig. 2.1(a). If the ratio $h_{ch}/h_{ef} > 0.4$ and/or $b_{ch}/h_{ef} > 0.7$ then either the length of anchor leg may be considered as effective embedment as shown in Fig. 2.1(b) i.e. $h_{ef} = h^*_{ef}$, or the effective embedment depth may be considered as per Fig. 2.1(a) provided the critical spacing. is as per the product's Technical Assessment report is not smaller than the critical spacing corresponding to anchor channels with ratio $h_{ch}/h_{ef} \leq 0.4$ and $b_{ch}/h_{ef} \leq 0.7$.

The characteristic concrete cone strength ($N_{Rk,c}$) of one anchor leg of an anchor channel is determined according to Eq. 2.7a.

[3] h_{ch} and b_{ch} are height and width of the channel as shown in Fig. 2.1

$$N_{Rk,c} = N^0_{Rk,c}\Psi_{ch,s,N}\Psi_{ch,e,N}\Psi_{ch,c,N}\Psi_{re,N} \qquad \text{(Eq. 2.7a)}$$

where,

$$N^0_{Rk,c} = k_1\sqrt{f_{ck}}h_{ef}^{1.5} \qquad \text{(Eq. 2.7b)}$$

$$s_{cr,N} = 2 * (2.8 - 1.3 * (h_{ef}/180)) * h_{ef} \geq 3h_{ef} \qquad \text{(Eq. 2.7c)}$$

$$\Psi_{ch,s,N} = \frac{1}{1+\sum_{i=1}^{n_{ch,N}}\left[\left(1-\frac{s_i}{s_{cr,N}}\right)^{1.5}\cdot\frac{N_i}{N_0}\right]} \qquad \text{(Eq. 2.7d)}$$

$$\Psi_{ch,e,N} = (c_1/c_{cr,N})^{0.5} \leq 1 \qquad \text{(Eq. 2.7e)}$$

$$\Psi_{ch,c,N} = (c_2/c_{cr,N})^{0.5} \leq 1 \qquad \text{(Eq. 2.7f)}$$

$$\Psi_{re,N} = 0.5 + h_{ef}/200 \leq 1 \qquad \text{(Eq. 2.7g)}$$

In Eq. 2.7a, $N^0_{Rk,c}$ is the basic characteristic concrete cone strength of single anchor leg whose strength is not influenced by adjacent anchor legs, edges or corners of concrete member. The factor k_1 is taken from ETA, which is denoted as $k_{ucr,N}$ and $k_{cr,N}$ for uncracked and cracked concrete, respectively. f_{ck} is nominal characteristic compressive strength measured on cylinder of size 150 mm diameter by 300 mm height. h_{ef} is the embedment depth.

The factor $\Psi_{ch,s,N}$ accounts for the influence of neighbouring anchor legs on the characteristic concrete cone strength of anchor leg under consideration. The distance s_i is the spacing between anchor leg under consideration and the neighboring anchor legs limited to the critical spacing value, i.e. $s_i \leq s_{cr,N}$. N_0 and N_i are tension force on anchor leg under consideration and on an influencing anchor, respectively. $n_{ch,N}$ is the no. of anchor legs within the distance $s_{cr,N}$ on both sides of anchor leg under consideration.

The factor $\Psi_{ch,e,N}$ and $\Psi_{ch,c,N}$ account for the effect of edge and corner of concrete member on the characteristic concrete cone strength, respectively. $c_{cr,N}$ is the critical edge distance. c_1 and c_2 are edge distances.

The factor $\Psi_{re,N}$,also known as shell spalling factor, accounts for the effect of dense reinforcement on capacity of anchor legs with effective embedment depth less than or equal to 100 mm (i.e. $h_{ef} \leq 100$ mm). The factor $\Psi_{re,N}$ may be assumed to be 1 provided 10 mm or smaller bar is spaced at distance $\geq$ 100 mm, or the reinforcement (irrespective of the bar diameter) is spaced at a distance ≥ 150 mm. If the reinforcement is provided in two directions, then this reinforcement spacing requirement should be fulfiled in both directions.

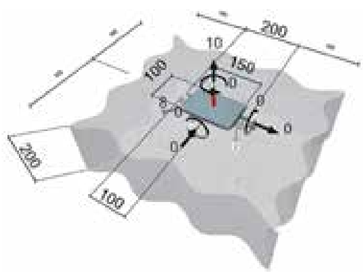
This formula doesn't yield precise results for anchor channels with effective embedment depth greater than 180 mm casted in narrow members and are under influence of neighbouring anchor legs, edge and two corners. EN 1992-4 [6] recommends modification to this formula. These modifications are not discussed in this book for the sake of simplicity.

Examples 2.9 and 2.10 illustrate typical calculations necessary for checking resistance to concrete cone failure in tension.

EXAMPLE 2.9

Check resistance to concrete breakout failure for the following connection formed using anchor channel HAC 40, 200 mm long (ETA-11/0006 [13]) with M12 x 40 mm HBC-C 8.8F T-bolt. The base material is of M25 grade concrete. Note – Anchor leg spacing is 150 mm and 10 kN tension load is acting at 100 mm from one end. Assume concrete to be cracked for design.

Given:



Anchor leg spacing $s = 150 \text{ mm}$

Point of load application from channel end $= 100 \text{ mm}$

$N_{Ed}^{cb} = 10 \text{ kN}$

$f_{ck} = 20 \text{ N/mm}^2$ (cylinder strength corresponding to M25)

Solution:

$N_{Ed,1}^{a} = 5 \text{ kN}$ (from Ex. 2.1)

$N_{Ed,2}^{a} = 5 \text{ kN}$ (from Ex. 2.1)

Check design concrete cone breakout resistance in tension

$h_{ef} = 91$ mm (from Table 6 of ETA-11/0006)

$k_1 = k_{cr,N} = 8$ (from Table 14 of ETA-11/0006)

$N^0_{Rk,c} = k_1\sqrt{f_{ck}}h_{ef}^{1.5} = 8 * \sqrt{20} * (91)^{1.5} = 31.058$ kN

$s_{cr,N} = 2 * (2.8 - 1.3 * (h_{ef}/180)) * h_{ef} = 390$ mm $> 3h_{ef}$ OK

$c_{cr,N} = 0.5s_{cr,N} = 195$ mm

$$\Psi_{ch,s,N} = \frac{1}{1 + \sum_{i=1}^{n_{ch,N}}[(1 - \frac{s_i}{s_{cr,N}})^{1.5} \cdot \frac{N_i}{N_0}]}$$

$$\Psi_{ch,s,N} = \frac{1}{1+\sum_{i=1}^{n_{ch,N}}[(1-\frac{150}{390})^{1.5} \cdot \frac{5}{5}]} = 0.674$$

$\Psi_{ch,e,N} = 1$ (as infinite edge has been assumed)

$\Psi_{ch,c,N,1} = 1$ (as infinite edge has been assumed)

$\Psi_{ch,c,N,2} = 1$ (as infinite edge has been assumed)

$\Psi_{re,N} = 0.5 + h_{ef}/200 = 0.955 \approx 1$

$N_{Rk,c} = N^0_{Rk,c}\Psi_{ch,s,N}\Psi_{ch,e,N}\Psi_{ch,c,N}\Psi_{re,N} = 31.058 * 0.674 * 1 * 1 * 1 * 1 = 20.94$ kN

$N_{Rk,c} = 31.058 * 0.674 * 1 * 1 * 1 * 1 = 20.94$ kN

$\gamma_{Mc} = 1.5$ (from 13 of ETA-11/0006)

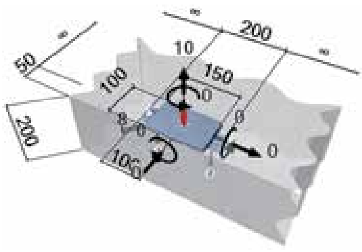
$N_{Rd,c} = N_{Rk,c}/\gamma_{Mc} = 20.94/1.5 = 13.96 \geq N^a_{Ed}$

$\beta = N^a_{Ed}/N_{Rd,c} = 5/13.96 = 0.358$ → 36% utilization. OK

EXAMPLE 2.10

Check resistance to concrete breakout failure for the following connection formed using anchor channel HAC 40, 200 mm long (ETA-11/0006 [13]) with M12 x 40 mm HBC-C 8.8F T-bolt. The base material is of M25 grade concrete. Note – Anchor leg spacing is 150 mm and 10 kN tension load is acting at 100 mm from one end. Assume concrete to be cracked for design.

Given:



Anchor leg spacing $s = 150$ mm

Point of load application from channel end= 100 mm

$N_{Ed}^{cb} = 10$ kN

$f_{ck} = 20$ N/mm² (cylinder strength corresponding to M25)

$c_1 = 50$ mm

Solution:

$N_{Ed,1}^{a} = 5$ kN (load resolution will be similar to Ex. 2.1)

$N_{Ed,2}^{a} = 5$ kN (load resolution will be similar to Ex. 2.1)

Check design concrete cone breakout resistance in tension

$h_{ef} = 91$ mm (from Table 6 of ETA-11/0006)

$k_1 = k_{cr,N} = 8$ (from Table 14 of ETA-11/0006)

$N^0_{Rk,c} = k_1\sqrt{f_{ck}}\,h_{ef}^{1.5} = 8 * \sqrt{20} * (91)^{1.5} = 31.058$ kN

$$\Psi_{ch,s,N} = \frac{1}{1+\sum_{i=1}^{n_{ch,N}}\left[\left(1-\frac{s_i}{s_{cr,N}}\right)^{1.5}\cdot\frac{N_i}{N_0}\right]}$$

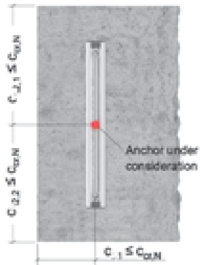
$$\Psi_{ch,s,N} = \frac{1}{1+\sum_{i=1}^{n_{ch,N}}\left[\left(1-\frac{150}{390}\right)^{1.5}\cdot\frac{5}{5}\right]} = 0.674$$

$s_{cr,N} = 2 * (2.8 - 1.3 * (h_{ef}/180)) * h_{ef} = 390 \text{ mm} > 3h_{ef}$

$c_{cr,N} = 0.5 s_{cr,N} = 195$ mm

$\Psi_{ch,e,N} = (c_1/c_{cr,N})^{0.5} = (50/195)^{0.5} = 0.506 \leq 1$

Take $c_{2,1} = c_{2,2} = c_{cr,N}$ (due to infinite edge on either side)



$\Psi_{ch,c,N,1} = (c_{2,1}/c_{cr,N})^{0.5} = 1$

$\Psi_{ch,c,N,2} = (c_{2,2}/c_{cr,N})^{0.5} = 1$

$\Psi_{re,N} = 0.5 + h_{ef}/200 = 0.955 \approx 1$

$N_{Rk,c} = N^0_{Rk,c}\Psi_{ch,s,N}\Psi_{ch,e,N}\Psi_{ch,c,N}\Psi_{re,N}$

$N_{Rk,c} = 31.058 * 0.674 * 1 * 0.506 * 1 * 1 = 10.6 \text{ kN}$

$\gamma_{Mc} = 1.5$ (from 13 of ETA-11/0006)

$N_{Rd,c} = N_{Rk,c} / \gamma_{Mc} = 10.6/1.5 = 7.07 \geq N^a_{Ed}$

$\beta = N^a_{Ed}/N_{Rd,c} = 5/7.07 = 0.707$ → 71% utilization. OK

(d) Resistance to concrete splitting

The concrete can split either during application of installation torque on channel bolt or during application of load on channel. The concrete splitting during installation may be avoided by providing spacing and edge distance greater than the minimum value as well as maintaining minimum member thickness and reinforcement as recommended in product's Technical Assessment report.

The concrete splitting during application of load may be avoided if edge distance greater than 1.2 times the characteristic critical spacing corresponding to splitting failure ($c_{cr,sp}$) is provided in all directions and the minimum member depth corresponding to this edge distance is provided as recommended in product's Technical Assessment report. If it is not possible to do so then separate verification needs to be carried to check resistance to splitting.

The design concrete splitting strength of the anchor leg in tension ($N_{Rd,sp}$) is determined by dividing characteristic concrete splitting resistance, $N_{Rk,sp}$, by recommended partial safety factor, γ_{Msp}, as shown in Eq. 2.8a. The check for concrete splitting failure is carried out for the most unfavourably loaded anchor leg. The load on the anchor leg along with edge and spacing of anchor leg should be taken into consideration for determining the most unfavourably loaded anchor leg.

$$N_{Rd,sp} = N_{Rk,sp} / \gamma_{Msp} \geq N^a_{Ed} \qquad \text{(Eq. 2.8a)}$$

The characteristic concrete splitting strength ($N_{Rk,sp}$) of one anchor leg of an anchor channel should be determined according to Eq. 2.8b. In this equation, the factors $\Psi_{ch,s,N}$, $\Psi_{ch,e,N}$, $\Psi_{ch,c,N}$, and $\Psi_{re,N}$, are calculated as per Eq. 2.7 but using critical edge[4] and critical spacing values corresponding to splitting failure for a given minimum member thickness.

$$N_{Rk,sp} = N_{Rk}^{0} \Psi_{ch,s,N}\Psi_{ch,e,N}\Psi_{ch,c,N}\Psi_{re,N}\Psi_{h,sp} \quad \text{(Eq. 2.8b)}$$

where

$N_{Rk}^{0} = \min(N_{Rk,p}, N_{Rk,c}^{0})$; Here $N_{Rk,p}$ is as per Eq. 2.6 and $N_{Rk,c}^{0}$ as per 2.7 (Eq. 2.8c)

$\Psi_{h,sp} = \left(\frac{h}{h_{min}}\right)^{2/3} \leq \max[1; [\frac{h_{ef}+c_{cr,N}}{h_{min}}]^{2/3}] \leq 2$; Here h_{min} corresponding to $c_{cr,sp}$ is used (Eq. 2.8d)

This verification may be skipped if cracked concrete is assumed for calculation of concrete cone and pull-out strength, and the reinforcement provided to resist the splitting forces limits the crack width to 0.3 mm. The required reinforcement ($\sum A_{s,re}$) to resist splitting may be determined as per Eq. 2.9. This reinforcement should be placed symmetrically and close to each anchor leg of the channel.

$$\sum A_{s,re} = 0.5 \frac{N_{Ed}^{a}}{f_{yk,re}/\gamma_{Ms,re}} \quad \text{(Eq. 2.9)}$$

where,

N_{Ed}^{a} is the design tensile force on the most loaded anchor leg.

$f_{yk,re}$- is the nominal yield strength of the reinforcement; but it should be ≤ 600 N/mm^2.

$\gamma_{Ms,re}$ - is the partial safety factor for the supplementary reinforcement.

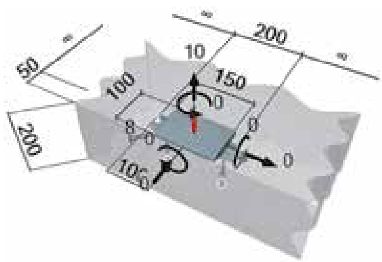
Example 2.11 illustrates typical calculations necessary for checking resistance to concrete splitting failure in tension.

[4] It is also known as characteristic edge. It is the distance from edge to ensure that edge does not influence characteristic resistance of the fastener.

EXAMPLE 2.11

Check resistance to concrete splitting failure for the following connection formed using anchor channel HAC 40, 200 mm long (ETA-11/0006 [13]) with M12 x 40 mm HBC-C 8.8F T-bolt. The base material is of M25 grade concrete. Note – Anchor leg spacing is 150 mm and 10 kN tension load is acting at 100 mm from one end. Assume concrete to be cracked for design.

Given:



Anchor leg spacing $s = 150\ \mathrm{mm}$

Point of load application from channel end $= 100\ \mathrm{mm}$

$N^{cb}_{Ed} = 10\ \mathrm{kN}$

$f_{ck} = 20\ \mathrm{N/mm^2}$ (cylinder strength corresponding to M25)

$c_1 = 50\ \mathrm{mm}$

Solution:

$N^{a}_{Ed,1} = 5\ \mathrm{kN}$ (load resolution will be similar to Ex. 2.1)

$N^{a}_{Ed,2} = 5\ \mathrm{kN}$ (load resolution will be similar to Ex. 2.1)

Check design concrete splitting resistance in tension

$h_{ef} = 91\ \mathrm{mm}$ (from Table 6 of ETA-11/0006)

$k_1 = k_{cr,N} = 8$ (from Table 14 of ETA-11/0006)

$N^0_{Rk,c} = k_1 \sqrt{f_{ck}}\, h_{ef}^{1.5} = 8 * \sqrt{20} * (91)^{1.5} = 31.058\ \mathrm{kN}$

$A_h = 209\ \mathrm{mm^2}$ (from Table 2 of ETA-11/0006)

$N_{Rk,p} = k_2 \cdot A_h \cdot f_{ck} = 7.5 * 209 * 20 = 31.35\ \mathrm{kN}$

$N^0_{Rk} = \min(N_{Rk,p}, N^0_{Rk,c}) = \min(31.35, 31.06) = 31.06\ \mathrm{kN}$

$c_{cr,sp} = 273\ mm$ (from Table 14 of ETA-11/0006)

$s_{cr,sp} = 2 * c_{cr,sp} = 546\ mm > 3h_{ef}$

$$\Psi_{ch,s,N} = \frac{1}{1+\sum_{i=1}^{n_{ch,N}}[(1-\frac{s}{s_{cr,sp}})^{1.5}\cdot\frac{N_i}{N_0}]}$$

$$\Psi_{ch,s,N} = \frac{1}{1+\sum_{i=1}^{n_{ch,N}}[(1-\frac{150}{546})^{1.5}\cdot\frac{5}{5}]} = 0.618$$

$\Psi_{ch,e,N} = (c_1/c_{cr,sp})^{0.5} = (50/273)^{0.5} = 0.428 \leq 1$

Take $c_{2,1} = c_{2,2} = c_{cr,N}$ (due to infinite edge on either side)

$\Psi_{ch,c,N,1} = (c_{2,1}/c_{cr,N})^{0.5} = 1$

$\Psi_{ch,c,N,2} = (c_{2,2}/c_{cr,N})^{0.5} = 1$

$\Psi_{re,N} = 0.5 + h_{ef}/200 = 0.955 \approx 1$

$$\Psi_{h,sp} = (\frac{h}{h_{min}})^{2/3} = (\frac{200}{105})^{2/3} = 1.537 \leq \max[1; [\frac{h_{ef}+c_{cr,N}}{h_{min}}]^{2/3}] \leq 2$$

$N_{Rk,sp} = N^0_{Rk}\Psi_{ch,s,N}\Psi_{ch,e,N}\Psi_{ch,c,N}\Psi_{re,N}\Psi_{h,sp}$

$N_{Rk,sp} = 31.058 * 0.618 * 0.428 * 1 * 1 * 1.537 = 12.625 \text{ kN}$

$\gamma_{Msp} = 1.5$ (from Table 14 of ETA-11/0006)

$N_{Rd,sp} = N_{Rk,sp}/\gamma_{Msp} = 12.625/1.5 = 8.417 \geq N^a_{Ed}$

$\beta = N^a_{Ed}/N_{Rd,sp} = 5/8.417 = 0.594$ → 59% utilization. OK

(e) Resistance to concrete blow-out

This check is not required if the provided edge distance is greater than 0.5 times the effective embedment depth. In all other cases, resistance to concrete blow-out should be checked. However, when anchor channels are located perpendicular to the edge then this check needs to be carried out only for the anchor closest to the edge.

The design concrete blow-out strength of the anchor leg in tension ($N_{Rd,cb}$) is determined by dividing characteristic concrete splitting resistance, $N_{Rk,cb}$, by recommended partial safety factor, γ_{Mc}, as shown in Eq. 2.10a. The check for concrete blow-out failure is carried out for the most unfavourably loaded anchor leg. The load on the anchor leg along with edge and spacing of anchor leg should be taken into consideration for determining the most unfavourably loaded anchor leg.

$$N_{Rd,\mathrm{cb}} = N_{Rk,cb} / \gamma_{Mc} \geq N^{a}_{Ed} \quad \text{(Eq. 2.10a)}$$

The characteristic concrete blow-out strength ($N_{Rk,cb}$) of one anchor leg of an anchor channel with embedment depth h_{ef} is determined according to Eq. 2.10b. In this formula, $N^{0}_{Rk,c}$ is basic characteristic concrete blow-out strength of single anchor leg whose strength is not influenced by adjacent anchor legs or edges of concrete member and is determined according to Eq. 2.10c. In Eq. 2.10c, the factor k_5 is taken as 8.7 and 12.2 for cracked and uncracked concrete, respectively. The factor A_h is the load bearing area of the headed anchor leg and is equal to $(d_h^2 - d_a^2)\pi/4$ for circular shaped heads. d_h and d_a are diameter of anchor head and anchor leg, respectively. For determining A_h, d_h should not be taken larger than $(6t_h + d)$; where t_h is the thickness of anchor head. f_{ck} is nominal characteristic compressive strength measured on cylinder of size 150 mm diameter by 300 mm height. c_2 is the edge distance. h_{ef} is the embedment depth.

The factor $\Psi_{ch,s,Nb}$ accounts for the influence of neighbouring anchor legs on the characteristic concrete blowout strength of anchor leg under consideration. The distance s_i is the spacing between anchor leg under consideration and the neighboring anchor legs limited to the critical spacing value, i.e. $s_i \leq s_{cr,Nb}$. N_0 and N_i are tension force on anchor leg under consideration and on an influencing anchor, respectively. $n_{ch,N}$ is the no. of anchor legs within the distance $s_{cr,Nb}$ on both sides of anchor leg under consideration.

The factor $\Psi_{ch,c,Nb}$ accounts for the effect of concrete corner of anchor leg, with corner distance of c_2, on the characteristic concrete blowout strength. If the anchor leg is influenced by two corners, then the factor $\Psi_{ch,c,Nb}$ is calculated for both corner distances separately and the product of the two is used in Eq. 2.10b. $c_{cr,Nb}$ is the critical edge distance. The factor $\Psi_{ch,h,Nb}$ accounts for the effect of concrete thickness in case the distance $f \leq 2c_1$ (see Fig. 2.9).

$$N_{Rk,cb} = N^{0}_{Rk,cb} \Psi_{ch,s,Nb} \Psi_{ch,c,Nb} \Psi_{ch,h,Nb} \quad \text{(Eq. 2.10b)}$$

where,

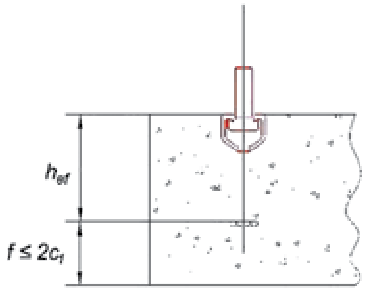
$N^0_{Rk,cb} = k_5 \cdot c_1 \cdot \sqrt{A_h} \cdot \sqrt{f_{ck}}$ (Eq. 2.10c)

$\Psi_{ch,s,N} = \frac{1}{1+\sum_{i=1}^{n_{ch,N}} [(1-\frac{s_i}{s_{cr,Nb}})^{1.5} \cdot \frac{N_i}{N_0}]}$ [where $s_{cr,Nb} = 4c_1$](Eq. 2.10d)

$\Psi_{ch,c,Nb} = (c_2/c_{cr,Nb})^{0.5} \leq 1$ [where $c_{cr,Nb} = 0.5s_{cr,Nb}$] (Eq. 2.10e)

$\Psi_{ch,h,Nb} = \frac{h_{ef}+f}{4c_1} \leq \frac{2c_1+f}{4c_1} \leq 1$ [where f is distance as shown in Fig.2.9]

(Eq. 2.10f)



f is the distance between the anchor head and the lower surface of the concrete member

Fig. 2.9 Anchor channel close to corner in a thin concrete member (resketched based on illustration in [6])

EXAMPLE 2.12

Check resistance to concrete blowout failure for the following connection formed using anchor channel HAC 40, 200 mm long (ETA-11/0006 [13]) with M12 x 40 mm HBC-C 8.8F T-bolt. The base material is of M25 grade concrete. Note – Anchor leg spacing is 150 mm and 10 kN tension load is acting at 100 mm from one end. Assume concrete to be cracked for design.

Given:

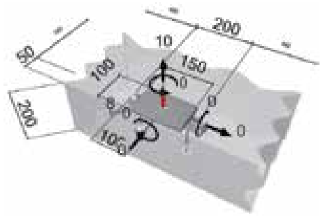
Anchor leg spacing $s = 150\ \mathrm{mm}$

Point of load application from channel end $= 100\ \mathrm{mm}$

$N_{Ed}^{cb} = 10\ \mathrm{kN}$

$f_{ck} = 20\ \mathrm{N/mm^2}$ (cylinder strength corresponding to M25)

$c_1 = 50\ \mathrm{mm}$



Solution:

To avoid blow-out failure – $c_1 > 0.5h_{ef}$

$c_1 = 50\ \mathrm{mm}$

$0.5h_{ef} = 0.5 * 91 = 45.5\ \mathrm{mm}$

$\therefore c_1 > 0.5h_{ef}$ OK

➔ There is no need to check blowout resistance in this case.

(f) Failure of supplementary reinforcement

The steel and anchorage strength are checked to determine adequacy of supplementary reinforcement.

(i) Steel strength of supplementary reinforcement

The design steel strength of the supplementary reinforcement in tension ($N_{Rd,re}$) is determined by dividing characteristic steel resistance, $N_{Rk,\mathrm{re}}$, by recommended partial safety factor, $\gamma_{Ms,re}$, as shown in Eq. 2.11a. The check for steel failure of supplementary reinforcement is carried out for force on reinforcement due to the most unfavourably loaded anchor leg ($N_{Ed,re}^{a}$). The total number of bars of supplementary reinforcement (n_{re}) effective for one anchor leg is used for determining $N_{Rk,\mathrm{re}}$.

$$N_{Rd,\mathrm{re}} = N_{Rk,re} / \gamma_{Ms,re} \geq N^{a}_{Ed,re} \quad \text{(Eq. 2.11a)}$$

Where,

$N_{Rk,re} = \sum_{i=1}^{n_{re}} A_{s,re,i}\, f_{yk,re}$ [with $f_{yk,re} \leq 600$ N/mm^2] (Eq. 2.11b)

In Eq. 2.11b, $A_{s,re,i}$ is area of supplementary reinforcement and $f_{yk,re}$ is nominal characteristic steel yield strength.

(ii) Anchorage strength of supplementary reinforcement

For this check, it should be established that the design anchorage resistance of the supplementary reinforcement ($N_{Rd,a}$) is greater than or equal to the force on reinforcement due to the most unfavourably loaded anchor leg ($N^{a}_{Ed,re}$). The design anchorage strength of the supplementary reinforcement in tension ($N_{Rd,a}$) is determined as per Eq. 2.12b. Again, the total number of bars of supplementary reinforcement (n_{re}) effective for one anchor leg is used for determining $N_{Rk,a}$.

$N_{Rd,a} \geq N^{a}_{Ed,re}$ (Eq. 2.12a)

Where,

$N_{Rd,a} = \sum_{i=1}^{nre} N^{0}_{Rd,a,i}$ (Eq. 2.12b)

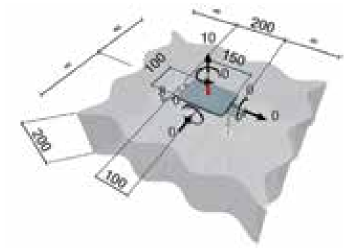
$N^{0}_{Rd,a,i} = \frac{l_1 \cdot \pi \cdot \phi \cdot f_{bd}}{\alpha_1 \cdot \alpha_2} \leq A_{s,re} f_{yk,re} \frac{1}{\gamma_{Ms,re}}$ (Eq. 2.12c)

In Eq. 2.12c, l_1 is the anchorage length of the concrete breakout body and it should be larger than the minimum anchorage length. The minimum anchorage length of supplementary reinforcement is 4ϕ (for anchorage with bends, hooks or loops) and 10ϕ (for anchorage with straight bars with or without welded transverse bars), respectively. ϕ is the bar diameter. The values of design bond strength (f_{bd}) and the influencing factors (α_1, α_2) should be as per Chapter 8 of EN 1992-1-1 [14]. The notations $A_{s,re}$, $f_{yk,re}$ and $\gamma_{Ms,re}$ are explained in Eq. 2.9. The detailing requirements are not covered in this book.

Examples 2.13 and 2.14 illustrate typical calculations necessary for checking adequacy of supplementary reinforcement provided to resist tension loads.

Example 2.13: Check steel strength of supplementary reinforcement for the following connection formed using anchor channel HAC 40, 200 mm long (ETA-11/0006 [13]) with M12 x 40 mm HBC-C 8.8F T-bolt. The base material is of M25 grade concrete. Note – Anchor leg spacing is 150 mm and 10 kN tension load is acting at 100 mm from one end. Assume two 10 mm bars are active as supplementary reinforcement for each anchor leg.

Given:



Anchor leg spacing $s = 150 \text{ mm}$

Point of load application from channel end $= 100 \text{ mm}$

$N_{Ed}^{cb} = 10 \text{ kN}$

$f_{ck} = 20 \text{ N/mm}^2$ (cylinder strength corresponding to M25)

Solution:

$N_{Ed,1}^{a} = 5 \text{ kN}$ (from Ex. 2.1)

$N_{Ed,2}^{a} = 5 \text{ kN}$ (from Ex. 2.1)

⇨ $N_{Ed,re}^{a} = 5 \ kN$

Check steel strength of supplementary reinforcement

Number of bars of supplementary reinforcement effective for one fastener, $n_{re} = 2$

Cross section of a reinforcing bar, $A_{s,re,i} = 79 \text{ mm}^2$

Nominal characteristic steel yield strength of reinforcement, $f_{yk,re} = 500 \text{ N/mm}^2$ (assumption)

$N_{Rk,re} = \sum_{i=1}^{n_{re}} A_{s,re,i} \, f_{yk,re}$ [with $f_{yk,re} \leq 600$ N/mm^2]

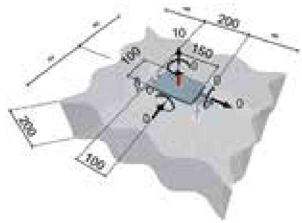
$N_{Rk,re} = 2 * 79 * 500/1000 = 79 \text{ kN}$

$N_{Rd,\text{re}} = N_{Rk,re} / \gamma_{Ms,re} = 79/1.15 = 68.69 \text{ kN} \geq N_{Ed,re}^{a}$ OK

EXAMPLE 2.14

Check anchorage strength of supplementary reinforcement for the following connection formed using anchor channel HAC 40, 200 mm long (ETA-11/0006 [13]) with M12 x 40 mm HBC-C 8.8F T-bolt. The base material is of M25 grade concrete. Note – Anchor leg spacing is 150 mm and 10 kN tension load is acting at 100 mm from one end. Assume two 10 mm bars are active as supplementary reinforcement for each anchor leg.

Given:

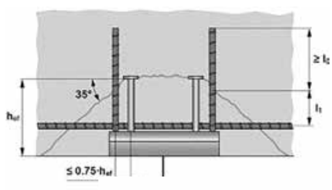


Anchor leg spacing $s = 150\ \mathrm{mm}$

Point of load application from channel end $= 100\ \mathrm{mm}$

$N_{Ed}^{cb} = 10\ \mathrm{kN}$

$f_{ck} = 20\ \mathrm{N/mm^2}$ (cylinder strength corresponding to M25)

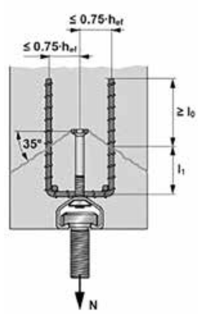


Solution:

$N_{Ed,1}^{g} = 5\ \mathrm{kN}$ (from Ex. 2.1)

$N_{Ed,2}^{g} = 5\ \mathrm{kN}$ (from Ex. 2.1)

$N_{Ed,re}^{a} = 5\ \mathrm{kN}$



Check anchorage strength of supplementary reinforcement

Anchorage length of the supplementary reinforcement in the assumed

Failure cone, $l_1 = 80\ \mathrm{mm}$

Diameter of reinforcement bar, $\phi = 10\ \mathrm{mm}$

Design bond strength according to EN 1992-1-1 [14], $f_{bd} = 2.3\ \mathrm{N/mm^2}$

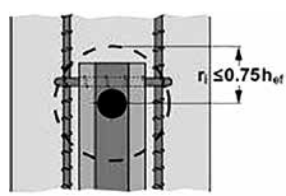
$\alpha_1 = 0.7$

$\alpha_2 = 1.0$

$N_{Rd,a} = \sum_{i=1}^{n_{re}} N_{Rd,a,i}^{0}$

$N_{Rd,a,i}^{0} = \frac{l_1 \cdot \pi \cdot \phi \cdot f_{bd}}{\alpha_1 \cdot \alpha_2} \leq A_{s,re} f_{yk,re} \frac{1}{\gamma_{Ms,re}}$

$N_{Rd,a} = 8.33\ \mathrm{kN} > N_{Ed,re}^{g}$ OK



2.8.2 DESIGN CHECKS FOR SHEAR LOAD

(a) Resistance to steel failure

The check for resistance to steel failure is dependent on whether or not the shear is acting with a lever arm. Therefore, the verification is broadly categorized into – Resistance to steel failure due to shear force "without lever arm" and Resistance to steel failure due to shear force "with lever arm".

(i) Resistance to steel failure due to shear force "without lever arm"

As per EN 1992-4 [6], the design steel strength in shear ($V_{Rd,s}$) is determined by dividing characteristic steel resistance by recommended partial material safety factor. The characteristic values for resistance to different forms of steel failure of cast-in anchor channel should be taken from the product's Technical Assessment report like ETA. The design steel strength should be greater than the design load (V_{Ed}). The design shear load on anchor leg is denoted as V_{Ed}^{a} and on channel bolt is denoted as V_{Ed}^{cb}.

For steel failure of the anchor leg, the design steel strength of anchor leg, $V_{Rd,s,a}$, is calculated by dividing characteristic resistance of anchor leg to steel failure, $V_{Rk,sa}$, by partial safety factor, γ_{Ms}, as follows. This check is carried out for the most unfavourably loaded anchor leg.

$$V_{Rd,s,a} = V_{Rk,s,a} / \gamma_{Ms} \geq V^{a}{}_{Ed} \qquad \text{(Eq. 2.13a)}$$

For steel failure of joint between anchor leg and channel, the design steel strength of joint, $V_{Rd,s,c}$, is calculated by dividing characteristic resistance of joint to steel failure, $V_{Rk,s,c}$, by partial safety factor, $\gamma_{Ms,ca}$, as follows. This check is carried out for the most unfavorably loaded anchor leg.

$$V_{Rd,s,c} = V_{Rk,s,c} / \gamma_{Ms,ca} \geq V^{a}{}_{Ed} \qquad \text{(Eq. 2.13b)}$$

For local flexure of channel lip, the design steel strength of channel lip, $V_{Rd,s,l}$, is calculated by dividing characteristic resistance of channel lip to flexure, $V_{Rk,s,l}$, by partial safety factor, $\gamma_{Ms,l}$, as follows. This check is carried out for the most unfavourably loaded channel bolt.

$$V_{Rd,s,l} = V_{Rk,s,l} / \gamma_{Ms,l} \geq V^{cb}{}_{Ed} \qquad \text{(Eq. 2.13c)}$$

where,

$V_{Rk,s,l} = V^0_{Rk,s,l} \cdot \psi_{l,v}$ [$V^0_{Rk,s,l}$ is taken from product's Technical Assessment report]

(Eq. 2.13c(i))

$\psi_{l,v} = 0.5(1 + s_{cbo}/s_{l,v}) \leq 1$

s_{cbo} - is the spacing between channel bolts.

$s_{l,v}$ - is the characteristic spacing for channel lip flexure under shear load and is taken from product's Technical Assessment report like ETA. If this value is not provided in the report then conservatively it may be taken as $2b_{ch}$.

For steel failure of channel bolt, the design steel strength of channel bolt, $V_{Rd,s}$, is calculated by dividing characteristic resistance of channel bolt, $V_{Rk,s}$, by partial safety factor, γ_{Ms}, as follows. This check is carried out for the most unfavorably loaded channel bolt.

$$V_{Rd,s} = V_{Rk,s} / \gamma_{Ms} \geq V^{cb}_{Ed} \qquad \text{(Eq. 2.13d)}$$

(ii) Resistance to steel failure due to shear force "with lever arm"

For steel failure of channel bolt, the design steel strength of channel bolt, $V_{Rd,s,M}$, is calculated by dividing characteristic resistance of channel bolt, $V_{Rk,s,M}$, by partial safety factor, γ_{Ms}, as follows. This check is carried out for the most unfavourably loaded channel bolt.

$$V_{Rd,s,M} = V_{Rk,s,M} / \gamma_{Ms} \geq V^{cb}_{Ed} \qquad \text{(Eq. 2.13e)}$$

where,

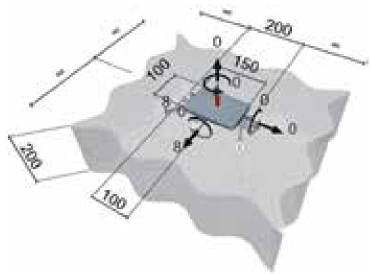
$$V_{Rk,s,M} = \alpha_M M_{Rk,s} / l_a \qquad \text{(Eq. 2.13e(i))}$$

In Eq. 2.13e(i), bending resistance of channel bolt ($M_{Rk,s}$) is equal to $M^0_{Rk,s}$ $(1 - N_{Ed}/N_{Rd,s})$. The value of $M^0_{Rk,s}$ is taken from the product's Technical Assessment report like ETA. There is no separate check required for channel lip failure as it is covered by prequalification on basis of which the ETA is issued. The factor α_M is taken as 1 for no restraint and 2 for full restraint of the fixture as discussed in Section 2.7.2. The length l_a is calculated according to Eq. 2.2b.

Example 2.15 illustrates typical calculations necessary for checking resistance to steel failure in shear.

EXAMPLE 2.15

Check resistance to steel failure in shear for the following connection formed using anchor channel HAC 40, 200 mm long (ETA-11/0006 [13]) with M12 x 40 mm HBC-C 8.8F T-bolt. Note – Anchor leg spacing is 150 mm and 8 kN shear load is acting at 100 mm from one end.



Given:

Anchor leg spacing $s = 150 \text{ mm}$

Point of load application from channel end $= 100 \text{ mm}$

$V_{Ed}^{cb} = 8 \text{ kN}$

Solution:

Shear load $V_{Ed}^{a} = 4 \text{ kN}$ (from Ex. 2.4)

Check design steel resistance in shear

a) Channel bolt (shear perpendicular w/o lever arm)

$V_{Rd,s,a} = 33.7 \text{ kN}$ (from Table 22 of ETA-11/0006)

$\gamma_{Ms} = 1.25$ (from Table 22 of ETA-11/0006)

$V_{Rd,s,a} = V_{Rk,s,a} / \gamma_{Ms}$

$V_{Rd,s,a} = 33.7 / 1.25 = 26.9 \text{ kN} \geq V^{a}{}_{Ed}$

$\beta = V_{Ed}^{a} / V_{Rd,s,a} = 4/26.9 = 0.148$ → 15% utilization. OK

b) Local flexure of channel lip (shear perpendicular w/o lever arm)

$V_{Rk,s,l}^{0} = 34.9 \text{ kN}$ (from Table 16 of ETA-11/0006)

$\psi_{l,v} = 0.5(1 + s_{cbo}/s_{l,v}) = 1$

$V_{Rk,s,l} = V_{Rk,s,l}^{0} \cdot \psi_{l,v}$

$V_{Rk,s,l} = 34.9 * 1 = 34.9 \text{ kN}$

$\gamma_{Ms,l} = 1.8$ (from Table 16 of ETA-11/0006)

$V_{Rd,s,l} = V_{Rk,s,l} / \gamma_{Ms,l} \geq V^{cb}{}_{Ed}$

$V_{Rd,s,l} = 34.9 / 1.8 = 19.4 \text{ kN} \geq V^{cb}_{Ed}$ Ok

c) Anchor leg

$V_{Rk,s,a} = 39.6 \text{ kN}$ (from Table 16 of ETA-11/0006)

$\gamma_{Ms} = 1.5$ (from Table 16 of ETA-11/0006)

$V_{Rd,s,a} = V_{Rk,s,a} / \gamma_{Ms} \geq V^{a}{}_{Ed}$

$V_{Rd,s,a} = 39.6 / 1.5 = 26.4 \text{ kN} \geq V^{a}_{Ed}$ Ok

$\beta = V^{a}_{Ed} / V_{Rd,s,a} = 4/26.4 = 0.152$ → 15% utilization. OK

d) Joint of anchor leg to channel

$V_{Rk,s,c} = 39.6 \text{ kN}$ (from Table 16 of ETA-11/0006)

$\gamma_{Ms,ca} = 1.8$ (from Table 16 of ETA-11/0006)

$V_{Rd,s,c} = V_{Rk,s,c} / \gamma_{Ms,ca} \geq V^{a}{}_{Ed}$

$V_{Rd,s,c} = 39.6 / 1.8 = 22 \text{ kN} \geq V^{a}_{Ed}$ Ok

$\beta = V^{a}_{Ed} / V_{Rd,s,c} = 4/22 = 0.182$ → 18% utilization. OK

(b) Resistance to concrete pry-out

As per EN 1992-4 [6], the design pry-out strength of the anchor leg in shear ($V_{Rd,cp}$) is determined by dividing characteristic pry-out resistance, $V_{Rk,\mathrm{cp}}$, by recommended partial safety factor, γ_{Mc}, as shown in Eq. 2.14a. This check is carried out for the most unfavourably loaded anchor leg. The load on the anchor leg along with edge and spacing of anchor leg should be taken into consideration for determining the most unfavourably loaded anchor leg.

$$V_{Rd,\mathrm{cp}} = V_{Rk,cp} / \gamma_{Mc} \geq V^{a}_{Ed} \quad \text{(Eq. 2.14a)}$$

where,

$V_{Rk,cp} = k_8 \cdot N_{Rk,c}$ [For anchor channels without supplementary reinforcement] (Eq. 2.14b)

$V_{Rk,cp} = 0.75\, k_8 \cdot N_{Rk,c}$ [For anchor channels with supplementary reinforcement] (Eq. 2.14c)

In the formula for determining the characteristic pry-out resistance($V_{Rk,\mathrm{cp}}$), the factor k_8 is taken from the product's Technical Assessment report and the $N_{Rk,c}$ value is determined according to Section 2.8.1(c) for fasteners loaded in shear.

Example 2.16 illustrates typical calculations necessary for checking resistance to concrete pry-out failure in shear.

EXAMPLE 2.16

Check resistance to concrete pry-out failure in shear for the following connection formed using anchor channel HAC 40, 200 mm long (ETA-11/0006 [13]) with M12 x 40 mm HBC-C 8.8F T-bolt. Note – Anchor leg spacing is 150 mm and 8 kN shear load is acting at 100 mm from one end. Assume concrete to be cracked for design and supplementary reinforcement to be present.

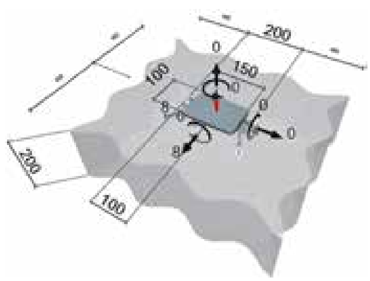
Given:

Anchor leg spacing $s = 150$ mm

Point of load application from channel end= 100 mm

$V_{Ed}^{cb} = 8$ kN

$f_{ck} = 20\ \mathrm{N/mm^2}$ (cylinder strength corresponding to M25)



Solution:

Shear load $V_{Ed}^{a} = 4$ kN (from Ex. 2.4)

Check concrete pry-out resistance in shear

$h_{ef} = 91$ mm (from Table 6 of ETA-11/0006)

$k_1 = k_{cr,N} = 8$ (from Table 14 of ETA-11/0006)

$N^0_{Rk,c} = k_1\sqrt{f_{ck}}\, h_{ef}^{1.5} = 8 * \sqrt{20} * (91)^{1.5} = 31.058$ kN

$$s_{cr,N} = 2 * (2.8 - 1.3 * (h_{ef}/180)) * h_{ef} = 390 \text{ mm} > 3h_{ef}$$

$$c_{cr,N} = 0.5 s_{cr,N} = 195 \text{ mm}$$

$$\Psi_{ch,s,N} = \frac{1}{1 + \sum_{i=1}^{n_{ch,N}} [(1 - \frac{s_i}{s_{cr,N}})^{1.5} \cdot \frac{V_i}{V_0}]}$$

$$\Psi_{ch,s,N} = \frac{1}{1+\sum_{i=1}^{n_{ch,N}}[(1-\frac{150}{390})^{1.5} \cdot \frac{4}{4}]} = 0.674$$

$\Psi_{ch,e,N} = 1$ (as infinite edge has been assumed)

$\Psi_{ch,c,N,1} = 1$ (as infinite edge has been assumed)

$\Psi_{ch,c,N,2} = 1$ (as infinite edge has been assumed)

$$\Psi_{re,N} = 0.5 + h_{ef}/200 = 0.955 \approx 1$$

$$N_{Rk,c} = N^0_{Rk,c} \Psi_{ch,s,N} \Psi_{ch,e,N} \Psi_{ch,c,N} \Psi_{re,N}$$

$$N_{Rk,c} = 31.058 * 0.674 * 1 * 1 * 1 * 1 \approx 21 \text{ kN}$$

$k_8 = 2$ (from Table 18 of ETA-11/0006)

$$V_{Rk,cp} = k_8 \cdot N_{Rk,c} = 2 * 21 = 42 \text{ kN}$$

$\gamma_{Mc} = 1.5$ (from 13 of ETA-11/0006)

$$V_{Rd,\text{cp}} = V_{Rk,cp} / \gamma_{Mc} = 42/1.5 = 28 \text{ kN} \geq V^a_{Ed}$$

$\beta = V^a_{Ed} / V_{Rd,\text{cp}} = 4/28 = 0.143$ → 14% utilization. OK

(c) Resistance to concrete edge breakout

The design concrete edge breakout strength of the anchor leg in shear ($V_{Rd,c}$) is determined by dividing characteristic concrete edge breakout resistance, $V_{Rk,c}$, by recommended partial safety factor, γ_{Mc}, as shown in Eq. 2.15a. The check for concrete cone failure is carried out for the most unfavourably loaded anchor leg. The load on the anchor leg along with edge and spacing of anchor leg should be taken into consideration for determining the most unfavourably loaded anchor leg.

$$V_{Rd,c} = V_{Rk,c} / \gamma_{Mc} \geq V^a_{Ed} \quad \text{(Eq. 2.15a)}$$

The characteristic concrete breakout strength ($V_{Rk,c}$) of one anchor leg of an anchor channel perpendicular to the edge should be determined according to Eq. 2.15b. In Eq. 2.15b, $V^0_{Rk,c}$ is basic characteristic concrete breakout strength of single anchor leg of anchor channel loaded perpendicular to edge, whose strength is not influenced by adjacent anchor legs, edges or corners of concrete member. The factor k_{12} used for determining $V^0_{Rk,c}$ should be taken from product's Technical Assessment report depending on state of concrete i.e. cracked ($k_{cr,V}$) or uncracked ($k_{ucr,V}$). f_{ck} is nominal characteristic compressive strength measured on cylinder of size 150 mm diameter by 300 mm height. c_1 is the edge distance.

The factor $\Psi_{ch,s,N}$ accounts for the influence of neighboring anchor legs on the characteristic concrete cone strength of anchor leg under consideration. The distance s_i is the spacing between anchor leg under consideration and the neighbouring anchor legs limited to the critical spacing value, i.e. $s_i \leq s_{cr,V}$. If the ratio $h_{ch}/h_{ef} \leq 0.4$ and $b_{ch}/h_{ef} \leq 0.7$ then $s_{cr,V} = 4c_1 + 2b_{ch}$. If the ratio $h_{ch}/h_{ef} > 0.4$ and/or $b_{ch}/h_{ef} > 0.7$ then $s_{cr,V}$ should be taken from product's Technical Assessment report but this value should not be smaller than the value $4c_1 + 2b_{ch}$. V_0 and V_i are shear force on anchor leg under consideration and on an influencing anchor, respectively. $n_{ch,V}$ is the no. of anchor legs within the distance $s_{cr,V}$ on both sides of anchor leg under consideration. In Eq. 2.15d, all anchor leg shear forces are assumed to be acting towards edge.

The factor $\Psi_{ch,c,V}$ accounts for the effect of corner (c_2) of concrete member on the characteristic concrete cone strength. For calculating the factor $\Psi_{ch,c,V}$, the critical edge distance ($c_{cr,V}$) is taken as 0.5 times of the critical spacing ($s_{cr,V}$). If the anchor leg is influenced by two corners, then the factor $\Psi_{ch,c,V}$ is calculated for each corner distance and the product of the two is used in Eq. 2.15b.

The factor $\Psi_{ch,h,V}$ accounts for the effect of member thickness. If the ratio $h_{ch}/h_{ef} \leq 0.4$ and $b_{ch}/h_{ef} \leq 0.7$ then $h_{cr,V} = 2c_1 + 2h_{ch}$. If the ratio $h_{ch}/h_{ef} > 0.4$ and/or $b_{ch}/h_{ef} > 0.7$ then $h_{cr,V}$ should be taken from product's Technical Assessment report but this value should not be smaller than the value $2c_1 + 2h_{ch}$. The factor $\Psi_{ch,90,V}$ accounts for influence of shear loads acing parallel to the edge and is taken as 2.5.

The factor $\Psi_{re,N}$ accounts for the effect of reinforcement on capacity. For uncracked concrete and for cracked concrete application without edge reinforcement/stirrups, the factor $\Psi_{re,N}$ is taken as 1. For cracked concrete application with edge reinforcement and closely spaced stirrups/wire mesh ($a \leq \min(100mm,\ 2c_1)$), the factor $\Psi_{re,N}$ is taken as 1.4, provided $h_{ch} \leq 40$ mm and h_{ef} is at least 2.5 times the concrete cover of edge reinforcement.

$$V_{Rk,c} = V^0_{Rk,c}\ \Psi_{ch,s,V}\Psi_{ch,c,V}\Psi_{ch,h,V}\Psi_{ch,90^\circ,V}\Psi_{re,V} \tag{Eq. 2.15b}$$

where,

$$V^0_{Rk,c} = k_{12}\sqrt{f_{ck}}\,c_1^{4/3} \tag{Eq. 2.15c}$$

$$\Psi_{ch,s,V} = \frac{1}{1+\sum_{i=1}^{n_{ch,V}}\left[\left(1-\frac{s_i}{s_{cr,V}}\right)^{1.5}\cdot\frac{V_i}{V_0}\right]} \tag{Eq. 2.15d}$$

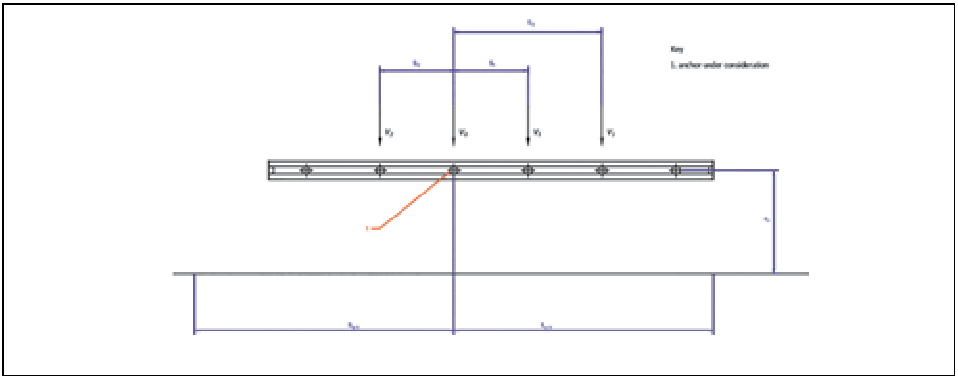
$$\Psi_{ch,c,V} = (c_2/c_{cr,V})^{0.5} \leq 1 \tag{Eq. 2.15e}$$

$$\Psi_{ch,h,V} = (h/h_{cr,V})^{0.5} \leq 1 \tag{Eq. 2.15f}$$

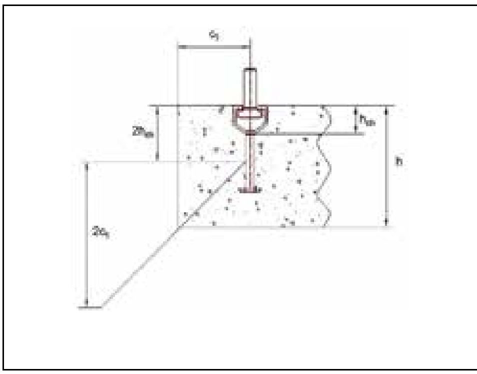
$$\Psi_{ch,90^\circ,V} = 2.5 \text{ (applicable for shear parallel to edge)} \tag{Eq. 2.15g}$$

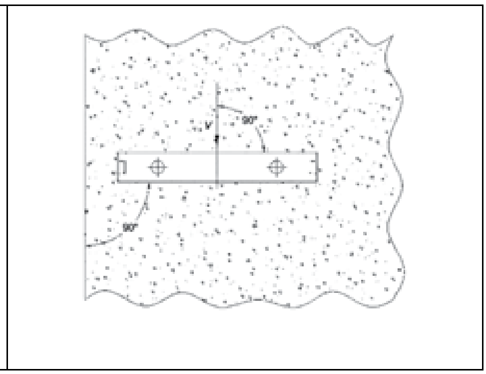
$$\Psi_{re,N} = 0.5 + h_{ef}/200 \leq 1 \tag{Eq. 2.15h}$$

This formula doesn't yield precise results for anchor channels in narrow members and are under influence of neighbouring anchor legs, edge and two corners. EN 1992-4 [6] recommends modification to this formula. These modifications are not discussed in this book for the sake of simplicity.



(a) Different shear forces on anchor channel





(b) Influence of member thickness (c) Shear acting parallel to edge

Fig. 2.10 Illustration of different influencing parameters in shear
(resketched based on illustration in EN 1992-4 [6])

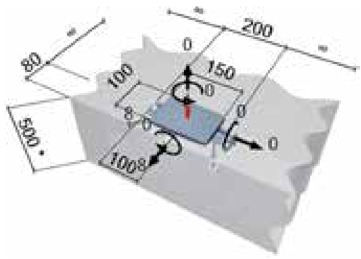
Example 2.16: illustrates typical calculations necessary for checking resistance to concrete edge breakout failure in shear.

EXAMPLE 2.16

Check resistance to concrete edge breakout failure in shear for the following connection formed using anchor channel HAC 40, 200 mm long (ETA-11/0006 [13]) with M12 x 40 mm HBC-C 8.8F T-bolt. Note – Anchor leg spacing is 150 mm and 8 kN shear load as well as 10 kN tension is acting at 100 mm from one end.

Assume concrete to be cracked for design. Also, assume reinforcement of any diameter to be present at spacing ≥ 150 mm.

Given:



Anchor leg spacing $s = 150\ \mathrm{mm}$

Point of load application from channel end $= 100\ \mathrm{mm}$

$c_1 = 80\ \mathrm{mm}$

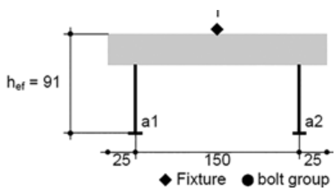
$f_{ck} = 20\ \mathrm{N/mm^2}$ (cylinder strength corresponding to M25)

$h = 500\ \mathrm{mm}$

$V_{Ed}^{cb} = 8\ \mathrm{kN}$

Solution:

$V_{Ed}^{a} = 4\ \mathrm{kN}$ (load resolution will be similar to Ex. 2.4, though concrete member dimension is different)



Check concrete pry-out resistance in shear

$k_{12} = 7.5$ (from Table 18 of ETA-11/0006)

$$V_{Rk,c}^{0} = k_{12}\sqrt{f_{ck}}\,c_1^{4/3}$$

$$V_{Rk,c}^{0} = 7.5 * \sqrt{20} * 80^{\frac{4}{3}} = 11.6\ \mathrm{kN}$$

$b_{ch} = 40.9\ \mathrm{mm}$ (from Table 1 of ETA-11/0006)

$$s_{cr,V} = 4\,.\,c_1 + 2\,.\,b_{ch} = 4 * 80 + 2 * 40.9 \approx 402\ \mathrm{mm}$$

$$c_{cr,V} = 0.5 s_{cr,V} = 0.5 * 402 = 201\ \mathrm{mm}$$

$$\Psi_{ch,s,V} = \frac{1}{1+\sum_{i=1}^{n_{ch,V}}\left[\left(1-\frac{s_i}{s_{cr,V}}\right)^{1.5}\cdot\frac{V_i}{V_0}\right]}$$

$$\Psi_{ch,s,V} = \frac{1}{1+\sum_{i=1}^{n_{ch,V}}\left[\left(1-\frac{150}{402}\right)^{1.5}\cdot\frac{4}{4}\right]} = 0.668$$

$\Psi_{ch,c,V} = (c_2/c_{cr,V})^{0.5} = 1$ (as c_2 is infinite, for calculation its value is limited to $c_{cr,V}$)

$h_{cr,V} = 2 \,.\, c_1 + 2 \,.\, h_{ch} = 2 * 80 + 2 * 28 = 216\,\mathrm{mm}$

$\Psi_{ch,h,V} = (h/h_{cr,V})^{0.5}$

$\Psi_{ch,h,V} = (500/216)^{0.5} = 1.5 > 1$

⇨ Use $\Psi_{ch,h,V} = 1$

$\Psi_{ch,90,V} = 1$ (as shear is acting perpendicular to edge)

$h_{ef} = 91\ \mathrm{mm}$ (from Table 6 of ETA-11/0006)

$\Psi_{re,N} = 1$ (as reinforcement is assumed to be present)

$V_{Rk,c} = V^0_{Rk,c}\Psi_{ch,s,V}\Psi_{ch,c,V}\Psi_{ch,h,V}\Psi_{ch,90,V}\Psi_{re,V}$

$V_{Rk,c} = 11.6 * 0.668 * 1 * 1 * 1 * 1 = 7.72\ \mathrm{kN}$

$V_{Rd,c} = V_{Rk,c}/\gamma_{Mc} = 7.72/1.5 = 5.15\ \mathrm{kN} \geq V^a_{Ed}$

$\beta = V^a_{Ed}/V_{Rd,c} = 4/5.15 = 0.777$ → 78% utilization. OK

(d) Failure of supplementary reinforcement

The steel and anchorage strength are checked to determine adequacy of supplementary reinforcement. The tension force acting on the reinforcement shall be calculated from design shear force according to Eq. 2.3a for the most loaded anchor leg.

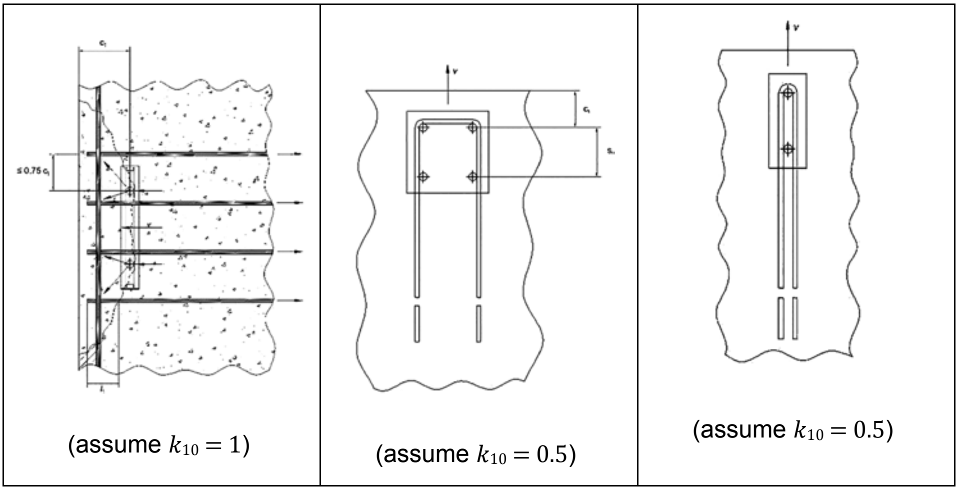
(i) Steel strength of supplementary reinforcement

The design steel strength of the supplementary reinforcement in tension ($N_{Rd,re}$) is determined by dividing characteristic steel resistance, $N_{Rk,re}$, by recommended partial safety factor, $\gamma_{Ms,re}$, as shown in Eq. 2.16a. The check for steel failure of supplementary reinforcement should be carried out for force on reinforcement due to the most unfavourably loaded anchor leg ($N^a_{Ed,re}$). The total number of bars of supplementary reinforcement (n_{re}) effective for one anchor leg should be used for determining $N_{Rk,re}$ as shown in Eq. 2.16b. The factor k_{10} used in Eq. 2.16b is known as efficiency factor. Its value depends on the type of supplementary reinforcement provided as shown in Fig. 2.11. The notations $A_{s,re,i}$ and $f_{yk,re}$ are explained in Eq. 2.9.

$N_{Rd,re} = N_{Rk,re}/\gamma_{Ms,re} \geq N^a_{Ed,re}$ (Eq. 2.16a)

where,

$N_{Rk,re} = k_{10} \sum_{i=1}^{n_{re}} A_{s,re,i}\, f_{yk,re}$ [with $f_{yk,re} \leq 600$ N/mm^2] (Eq. 2.16b)



(a) surface reinforcement to take up shear forces; (b) supplementary reinforcement in the shape of stirrups; (c) supplementary reinforcement in the shape of loops

Fig. 2.11. Reinforcement to take up shear forces on the cast-in anchor channel (resketched based on illustration in EN 1992-4 [6])

(ii) Anchorage strength of supplementary reinforcement

For this check, it should be established that the design anchorage resistance of the supplementary reinforcement ($N_{Rd,a}$) is greater than or equal to the force on reinforcement due to the most unfavorably loaded anchor leg ($N^{a}_{Ed,re}$). The design anchorage strength of the supplementary reinforcement in tension ($N_{Rd,a}$) is determined as per Eq. 2.17b. Again, the total number of bars of supplementary reinforcement (n_{re}) effective for one anchor leg should be used for determining $N_{Rk,a}$.

$N_{Rd,a} \geq N^{a}_{Ed,re}$ (Eq. 2.17a)

where,

$$N_{Rd,a} = \sum_{i=1}^{n_{re}} N_{Rd,a}^{0} \quad \text{(Eq. 2.17b)}$$

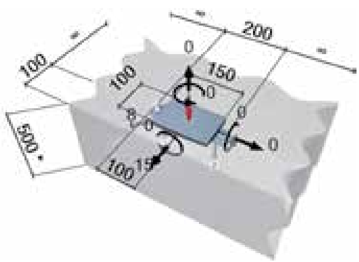
$$N_{Rd,a}^{0} = \frac{l_1 \cdot \pi \cdot \phi \cdot f_{bd}}{\alpha_1 \cdot \alpha_2} \leq A_{s,re} f_{yk,re} \frac{1}{\gamma_{Ms,re}} \quad \text{(Eq. 2.17c)}$$

In Eq. 2.17c, l_1 is the anchorage length of the concrete breakout body (see Fig. 2.11a) and it should be larger than the minimum anchorage length. The minimum anchorage length of supplementary reinforcement is 4ϕ (for anchorage with bends, hooks or loops) and 10ϕ (for anchorage with straight bars with or without welded transverse bars), respectively. ϕ is the bar diameter. The values of design bond strength (f_{bd}) and the influencing factors (α_1, α_2) should be as per Chapter 8 of EN 1992-1-1 [14]. The exceptions and detailing requirements are not covered in this book.

Examples 2.18 and 2.19 illustrate typical calculations necessary for checking adequacy of supplementary reinforcement provided to resist shear loads.

EXAMPLE 2.18

Check steel strength of supplementary reinforcement for the following connection formed using anchor channel HAC 40, 200 mm long (ETA-11/0006 [13]) with M12 x 40 mm T-bolt HBC-C 8.8F. The base material is M25 grade concrete. Note – Anchor leg spacing is 150 mm and 16 kN shear load is acting at 100 mm from one end. Assume reinforcement of any diameter to be present at spacing ≥ 150 mm along with supplementary reinforcement of 10 mm diameter. Also, assume concrete to be cracked for design.



Given:

Anchor leg spacing $s = 150$ mm

Point of load application from channel end $= 100$ mm

$c_1 = 80$ mm

$f_{ck} = 20\ \text{N/mm}^2$ (cylinder strength corresponding to M25)

$h = 500$ mm

$d_{s,re} = 10$ mm

$f_{yk,re} = 500 \text{ N/mm}^2$

$V_{Ed}^{cb} = 15 \text{ kN}$

Solution:

Determination of shear force on anchor leg

$I_y = 21463 \text{ mm}^4$ (from Table 1 of ETA-11/0006)

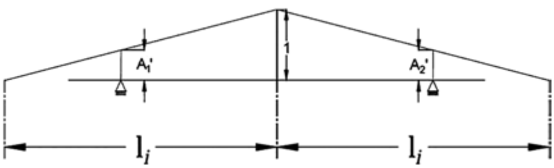
$l_i = 13 \cdot I_y^{0.05} \cdot s^{0.5} \geq s$

$l_i = 13 \cdot (21463)^{0.05} \cdot (150)^{0.5} \approx 262 \text{ mm}$ [$\geq s = 150$ mm. OK]

Load distribution scheme based on influence length, l_i

$A_1' = 0.29$

$A_2' = 0.29$



$k = \frac{1}{\sum_1^n A_i'} = \frac{1}{A_1' + A_2'} = \frac{1}{0.29+0.29} = 1.75$

$V_{Ed,1}^{a} = k \cdot A_1' \cdot V_{Ed}^{cb} = 1.75 * 0.29 * 15 = 7.5 \text{ kN}$

$V_{Ed,2}^{a} = k \cdot A_2' \cdot V_{Ed}^{cb} = 1.75 * 0.29 * 15 = 7.5 \text{ kN}$

Therefore, shear load of 7.5 kN is action on each anchor leg.

Check design steel strength of supplementary reinforcement

a) Determination of the design tension force in the supplementary reinforcement

$e_c = 0$

$t = 0$

$c = 25 \text{ mm}$

$h_{ch} = 28 \text{ mm}$ (from Table 1 of ETA-11/0006)

$e_s = e_c + \frac{t}{2} + \max(c, h_{ch}) + \frac{d_{s,re}}{2}$

$e_s = 0 + 0 + \max(25, 28) + \frac{10}{2} = 33 \text{ mm}$

$d = h - \max(c, h_{ch}) - \frac{d_{s,re}}{2} \leq \min(2 \cdot h_{ef}, 2 \cdot c_1)$

$d = 500 - \max(25{,}28) - \frac{10}{2} = 467 \text{ mm}$

But, $\min\left(2 \cdot h_{ef}, 2 \cdot c_1\right) = 182 \text{ mm}$

Use $d = 182 \text{ mm}$

$z = 0.85d = 0.85 * 182 = 154.7 \text{ mm}$

$N^a_{Ed,re} = V^a_{Ed} \cdot \left(\frac{e_s}{z} + 1\right)$

$N^a_{Ed,re} = 15 \cdot \left(\frac{33}{154.7} + 1\right) = 18.2 \text{ kN}$

b) Check for resistance to steel failure of supplementary reinforcement

$k_{10} = 1$

$A_{s,re,i} = \pi r^2 = 3.14 * (10/2)^2 = 78.5 \text{ mm}^2$

$$N_{Rk,re} = k_{10} \sum_{i=1}^{n_{re}} A_{s,re,i}\, f_{yk,re}$$

$N_{Rk,re} = 1 * (78.5 * 0.5 + 78.5 * 0.5 + 78.5 * 0.5 + 78.5 * 0.5) = 157 \text{ kN}$

$\gamma_{Ms,re} = 1.15$

$N_{Rd,\mathrm{re}} = N_{Rk,re} / \gamma_{Ms,re} = 157/1.15 = 136.5\, kN \geq N^a_{Ed,re}$ Ok

EXAMPLE 2.19

Check the anchorage strength of supplementary reinforcement for the connection in Ex. 2.18

Solution:

$N^a_{Ed,re} = 18.2 \text{ kN}$ (as determined in Ex. 2.18)

$n_{re} = 4$

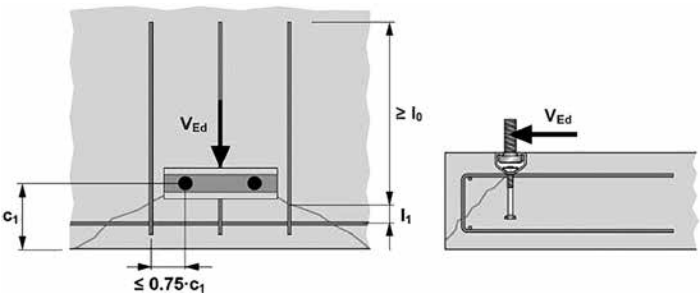
$\sum l_1 = 250$ mm

$\alpha_1 = 1$

$\alpha_2 = 1$

$N_{Rd,a} = \sum_{i=1}^{nre} \frac{l_1 \cdot \pi \cdot \phi \cdot f_{bd}}{\alpha_1 \cdot \alpha_2} = 18.242$ kN

$N_{Rd,a} \geq N_{Ed,re}^{a}$ Ok



2.8.3 DESIGN CHECKS FOR COMBINED TENSION AND SHEAR

(a) For anchor channels without supplementary reinforcement

The design check is based on the type of failure mode. For steel failure, the combined tension and shear check needs to be carried out for the following:

Check for steel failure of channel bolt:

$$\left[\frac{N_{Ed}^{cb}}{N_{Rd,s}}\right]^2 + \left[\frac{V_{Ed}^{cb}}{V_{Rd,s}}\right]^2 \leq 1 \qquad \text{(Eq. 2.18)}$$

Note – This check is not required when shear load is acting with lever arm.

Check for steel failure of channel lip/ flexure failure of channel:

$$\left[N_{Ed}^{ch} \big/ N_{Rd,s,l} ; M_{Ed}^{ch} \big/ M_{Rd,s,flex} \right]^{k_{13}} + \left[V_{Ed}^{cb} \big/ V_{Rd,s} \right]^{k_{13}} \leq 1 \; where \begin{Bmatrix} if\ V_{Rd,s,l} \leq N_{Rd,s,l}, k_{13} = 2 \\ Else\ to\ be\ taken\ from\ ETA \end{Bmatrix} \qquad \text{(Eq. 2.19)}$$

Note – The factor k_{13} may be taken as 1 for simplification if $V_{Rd,s,l} > N_{Rd,s,l}$

Check for steel failure of anchor leg / connection between anchor leg and channels:

$$\left[N_{Ed}^{a} \big/ N_{Rd,s,a} ; N_{Ed}^{a} \big/ N_{Rd,s,c} \right]^{k_{14}} + \left[V_{Ed}^{a} \big/ V_{Rd,s,a} \right]^{k_{14}} \leq 1 \; where \begin{Bmatrix} if\ V_{Rd,s,a} \leq \min(N_{Rd,s,a}, N_{Rd,s,c}), k_{14} = 2 \\ Else\ to\ be\ taken\ from\ ETA \end{Bmatrix} \qquad \text{Eq. 2.20)}$$

Note – The factor k_{14} may be taken as 1 for simplification if $V_{Rd,s,a} > \min(N_{Rd,s,a}, N_{Rd,s,c})$

For all other failure types, the combined tension and shear check needs to be carried out as follows:

$$\left[N_{Ed}^{a} \big/ N_{Rd} \right]^{1.5} + \left[V_{Ed}^{a} \big/ V_{Rd} \right]^{1.5} \leq 1 \qquad \text{(Eq. 2.21a)}$$

(or)

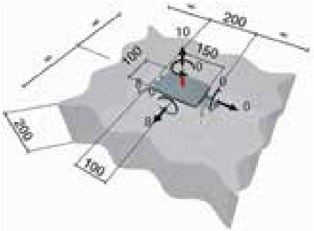
$$N_{Ed}^{a} \big/ N_{Rd} + V_{Ed}^{a} \big/ V_{Rd} \leq 1.2 \qquad \text{(Eq. 2.21b)}$$

The largest value of tension ratio and shear ratio for different failure modes should be used in the above equation. In Eq. 2.21a and 2.21b, the tension and shear ratios should be less than or equal to 1, i.e. $N_{Ed}^{a} \big/ N_{Rd} \leq 1$ and $V_{Ed}^{a} \big/ V_{Rd} \leq 1$.

Example 2.20 illustrates typical calculations necessary for checking combined tension and shear.

EXAMPLE 2.20

Check combined tension and shear for the following connection without supplementary reinforcement [Note – Use resistance data determined in previous examples]



Solution:

Check for steel failure of channel bolt:

$N_{Ed}^{cb} = 10\ \text{kN}$

$N_{Rd,s} = 44.93\ \text{kN}$

$V_{Ed}^{cb} = 8\ \text{kN}$

$V_{Rd,s} = 26.96\ \text{kN}$

$$\left[N_{Ed}^{cb} / N_{Rd,s} \right]^2 = (10/44.93)^2 = 0.049$$

$$\left[V_{Ed}^{cb} / V_{Rd,s} \right]^2 = (8/26.96)^2 = 0.088$$

$$\left[N_{Ed}^{cb} / N_{Rd,s} \right]^2 + \left[V_{Ed}^{cb} / V_{Rd,s} \right]^2 = 0.049 + 0.088 = 0.137 \leq 1 \quad \text{OK}$$

Check for steel failure of channel lip/ flexure failure of channel:

$N_{Ed}^{ch} = 10\ \text{kN}$

$N_{Rd,s,l} = 13.9\ \text{kN}$

$V_{Ed}^{cb} = 8\ \text{kN}$

$V_{Rd,s,l} = 19.4\ \text{kN}$

$M_{Ed}^{ch} = N/A$

$M_{Rd,s,flex} = N/A$

$k_{13} = 2$

$$max\left[{N_{Ed}^{ch}}/{N_{Rd,s,l}}; {M_{Ed}^{ch}}/{M_{Rd,s,flex}}\right]^{k_{13}} = 0.518$$

$$\left[{V_{Ed}^{cb}}/{V_{Rd,s}}\right]^{k_{13}} = 0.170$$

$$max\left[{N_{Ed}^{ch}}/{N_{Rd,s,l}}; {M_{Ed}^{ch}}/{M_{Rd,s,flex}}\right]^{k_{13}} + \left[{V_{Ed}^{cb}}/{V_{Rd,s,l}}\right]^{k_{13}} = 0.688 \leq 1 \quad \text{OK}$$

Check for steel failure of anchor leg / connection between anchor leg and channels:

$N_{Ed}^{a} = 5\ \text{kN}$

$N_{Rd,s,a} = 18.39\ \text{kN}$

$N_{Rd,s,c} = 13.9\ \text{kN}$

$V_{Ed}^{a} = 4\ \text{kN}$

$V_{Rd,s,a} = 26.4\ \text{kN}$

$k_{14} = 2$

$$max\left[{N_{Ed}^{a}}/{N_{Rd,s,a}}; {N_{Ed}^{a}}/{N_{Rd,s,c}}\right]^{k_{14}} = 0.129$$

$$\left[{V_{Ed}^{a}}/{V_{Rd,s,a}}\right]^{k_{14}} = 0.022$$

$$max\left[{N_{Ed}^{a}}/{N_{Rd,s,a}}; {N_{Ed}^{a}}/{N_{Rd,s,c}}\right]^{k_{14}} + \left[{V_{Ed}^{a}}/{V_{Rd,s,a}}\right]^{k_{14}} = 0.152 \leq 1 \qquad \text{OK}$$

Check for all other failure

$N_{Ed}^{a} = 5\ \text{kN}$

$N_{Rd,c} = 13.96$ kN

$V_{Ed}^{a} = 4$ kN

$V_{Rd,cp} = 27.92$ kN

$$\left[{N_{Ed}^{a}}\middle/{N_{Rd}}\right]^{1.5} = 0.214$$

$$\left[{V_{Ed}^{a}}\middle/{V_{Rd}}\right]^{1.5} = 0.054$$

$$\left[{N_{Ed}^{a}}\middle/{N_{Rd}}\right]^{1.5} + \left[{V_{Ed}^{a}}\middle/{V_{Rd}}\right]^{1.5} 0.268 \leq 1$$

(b) For anchor channels with supplementary reinforcement

The combined tension and shear checks required for steel failure for anchor channel without supplementary reinforcement should be carried out for this case as well. However, the tension and shear ratio corresponding to concrete cone and concrete edge failure should be replaced with corresponding values for failure of supplementary reinforcement. For anchor channels located at edge with supplementary reinforcement, the requirement of Section 2.8.3(A) applies but the following equation should be used instead of Eq. 2.21a and 2.21b.

$$ {N_{Ed}^{a}}\big/{N_{Rd}} + {V_{Ed}^{a}}\big/{V_{Rd}} \leq 1.2 \qquad \text{(Eq. 2.22)}$$

2.8.4 DESIGN CHECKS FOR SERVICEABILITY LIMIT STATE

To fulfil requirement of serviceability limit state, it shall be established that the admissible displacement (C_d) of the fastening system is higher than or equal to the expected displacement due to design load (E_d) i.e. $E_d \leq C_d$.

The admissible displacement (C_d) shall be evaluated by the Structural Engineer while designing, depending on the application. The displacements may be assumed to vary as a linear function of the applied load. The displacements for the shear and tension components of the resultant load should be added vectorially for combined check. The product’s Technical Assessment report should be referred for characteristic displacement of the anchor depending on load (tension or shear) and state of concrete (cracked or uncracked) concrete.

2.9 DESIGN SPECIFICATION

The design details like the position of the attachment on fixture, position of fasteners, fixture details, special installation instructions etc. should be clearly marked in the specification and construction drawings. Some typical specification details are shown in Fig. 2.12.

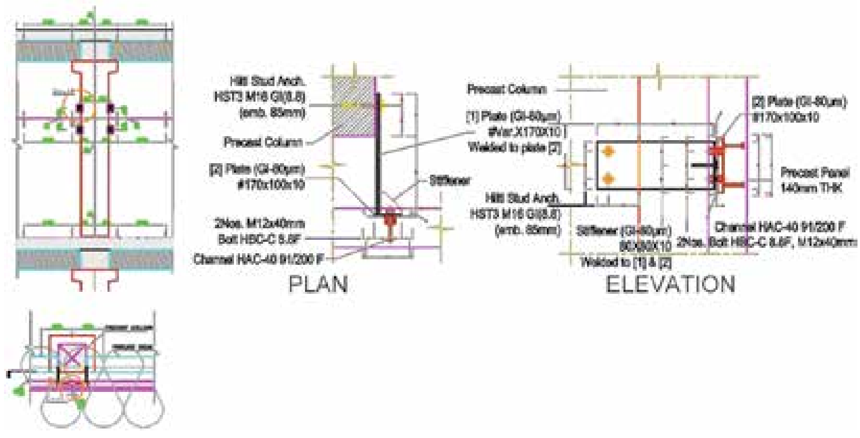
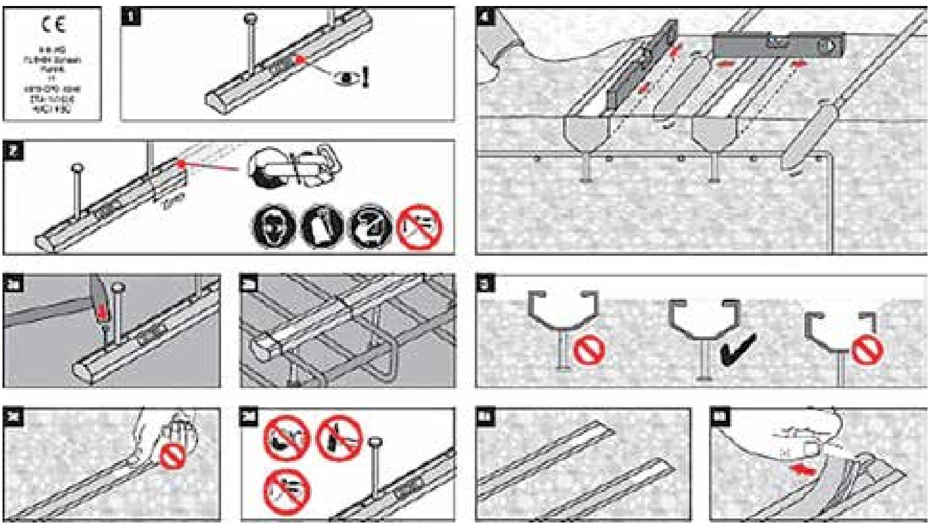


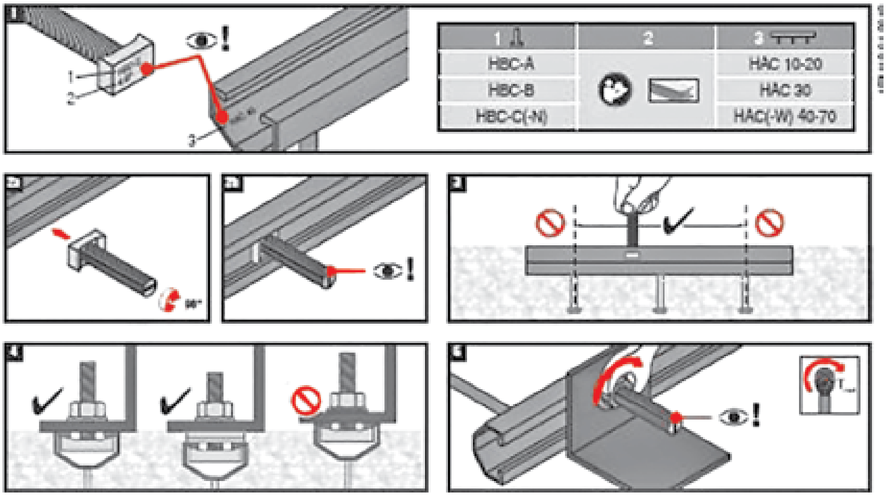
Fig. 2.12 Typical specification details for connections formed using cast-in anchor channels

2.10 INSTALLATION AND INSPECTION

The performance of connection formed using cast-in anchor channels is dependent on the correct installation. The installation steps vary from manufacturer to manufacturer. The installation steps recommended in the product's Technical Assessment report should be adhered to. The installation should be carried out under supervision by a skilled labourer who has been trained on product installation. Some general installation steps are discussed in this section (see Fig. 2.13). The very first step is identification of the location and marking the coordinates. The channel is then secured in position either by nailing it to a wooden board or by tying it to the reinforcement. The levelling is then checked, and it is ensured that the lip of channel is in line with the concrete surface. After concreting, the channel filler material (e.g. LDPE foam) is removed. The channel bolt is slid into position and secured. The installation details like installer name, supervisor name, installation date, product details etc. is maintained for inspection. Sometimes proof load test or onsite test is also carried out to check quality of workmanship. In onsite test, if one product fails at a higher load than other, it doesn't imply that the product is better. It is only a means of checking installation and should not be used as a tool for comparing two products. For each onsite test, details like test equipment, applied load type and value, result interpretation etc. are documented and maintained.



(a) Installation of anchor channel



(b) Installation of channel bolt

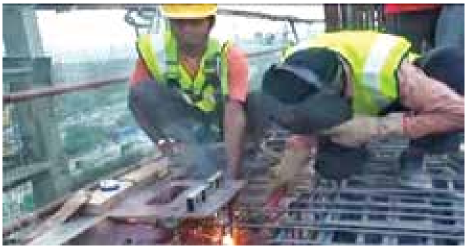
Fig. 2.13 Typical steps for installing cast-in anchor channels [13]

Step 1 – Identification of coordinates using total station

Step 2 – Marking of coordinates

Step 3 – Nailing of cast-in anchor channel to wooden plank

Step 4 – Securing the plank along with channel in position



Step 5 – Welding of rebar support to existing reinforcement cage

Step 6 – Concreting and levelling

Step 7 – After checking level of channel in hardened concrete, removal of filler material

Step 8 – Sliding channel bolt into position

Step 9 – Securing fixture to the channel bolt

Fig. 2.14 Example of installation of cast-in anchor channels at jobsite

2.11 PRACTISE PROBLEMS

Case 1: Design connection for fixing of façade bracket on top of slab of M25 grade concrete with thickness 180 mm preferable with cast-in channels. Bracket size is 200 mm width and 260 mm in length. The wind pressure on the facade bracket is 29.14 kN and DL at the bracket end is 4.3 kN.

Case 2: An air circulating unit needs to be fixed on a beam of M27 grade concrete and size 350 mm x 600 mm. The anchor channel connection to resist a shear load perpendicular to beam edge of 25 kN and moment of 3.75 kN-m on the axis parallel to beam length. Advised to use 2 anchor channels parallel to each other.

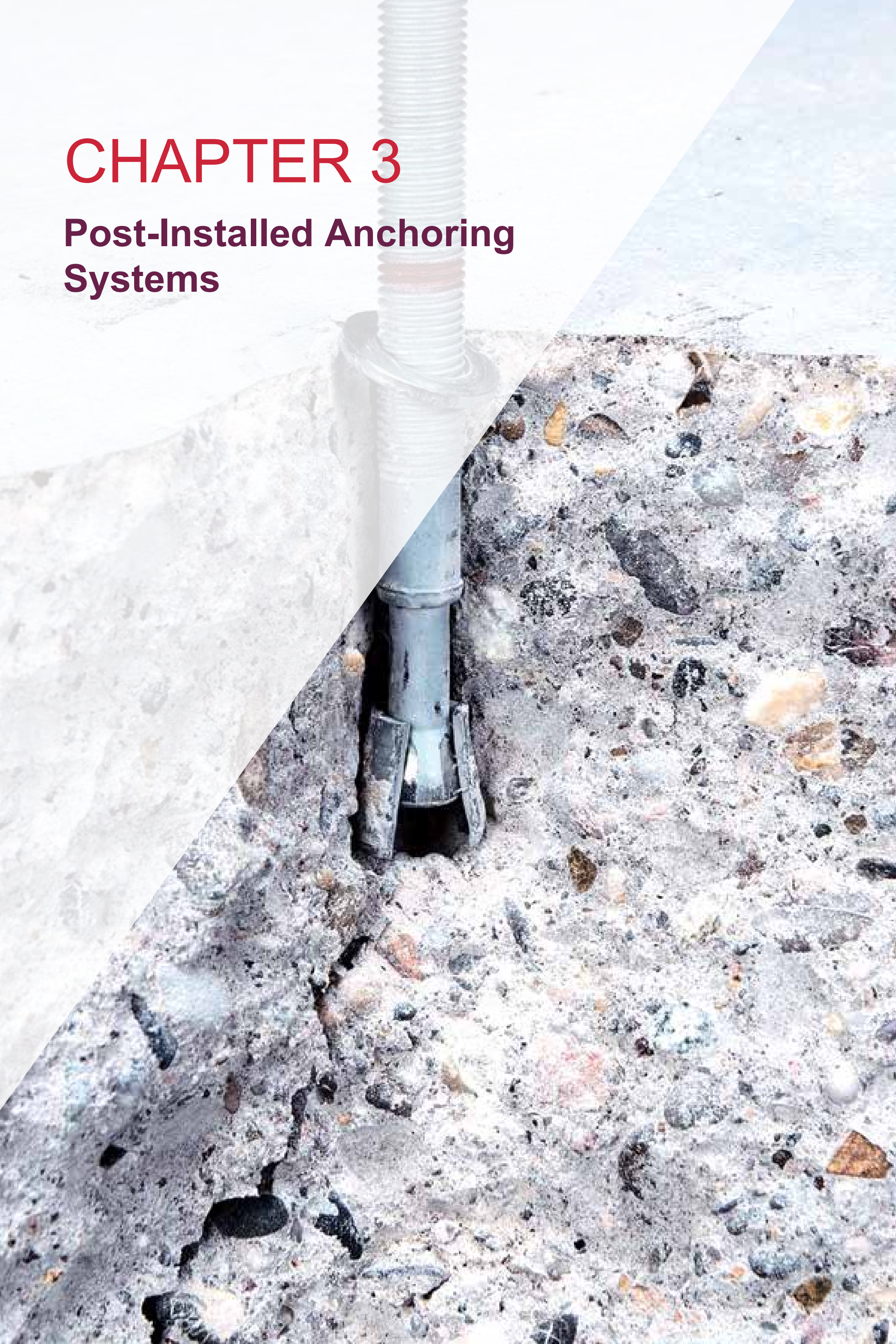
Case 3: Design connection for fixing of façade bracket on top of slab of M30 grade concrete with thickness 200 mm. Façade bracket size is 250 mm width and 380 mm in length. The wind suction on the facade bracket is 22.35 kN and DL at the bracket end is 2.75 kN. Calculate the size and type of cast-in channels required for the connection.

Case 4: Chiller piping for industrial plant needs to be attached from bottom of PT slab. It is favoured that the connection for fixing needs to be in-situ to save time and avoid drilling. The thickness of slab is 250 mm and grade is M55. The T-bolts in the cast-in channels should be 3 in numbers, with each bolt resisting tensile load of 33.98 kN. Design and select the suitable channel type, size and spacing of T-bolts to cost optimize the connection.

Case 5: Design anchor channels for fixing of façade bracket on top of slab with thickness 400 mm and grade of M40. The bracket is designed to take wind pressure of 11.25 kN and dead load and end of bracket is 5.079 kN, with size 250 wide and 210 mm in length. The maximum spacing of the channel is to be at 100 mm from the edge of the slab.

CHAPTER 3

Post-Installed Anchoring Systems



3.1 OVERVIEW

In the previous chapter, we dealt with design of cast-in anchor channels. In this chapter, we focus on design of connections using post-installed anchoring technology. The design concept of the two fastening technologies is very similar and based on the same principles of mechanics. In fact, its design is covered in the same standard EN 1992-4 [6] as that of cast-in anchors in European regulatory framework. Though both technologies are used to fasten steel to concrete and are based on same design philosophy, they cater to different application needs. One leverages the familiarity of cast-in concept whereas the other builds its credibility by overcoming the drawbacks of cast-in systems and offers more flexibility. The need for flexibility during construction phase has helped post-installed anchors create a niche market for itself. Its use has grown manifold over the past two decades. Though there are still some who shy away from switching to a newer technology, most have wholeheartedly accepted it as now there are systems in the market that closely match the load-displacement performance of traditional cast-in bolts, and yet overcome most of the challenges associated with it.

We begin this chapter by presenting different types of post-installed anchors. We then discuss different parameters that may influence performance of these systems. We then move on to talk about different standards available for assessing performance and for design of post-installed anchors. Design philosophy is presented next. Load analysis concept to determine forces on post-installed anchors due to applied load is also discussed. Some illustrative examples are also presented. Design for static/ quasi-static, seismic and fatigue loads is described in detail. Lastly, installation and inspection aspect are also presented. Only elastic design concept is covered in this book.

This chapter also devotes more time on design than performance evaluation as this book is meant to be used as a design guide. As stated in previous chapter, performance evaluation is equally important and is prerequisite for design.

3.2 TYPES OF POST-INSTALLED ANCHORING SYSTEMS

As stated in Section 1.2.(A), post-installed anchors are installed into hardened concrete and hence the name “post-installed”. In the market different varieties of post-installed anchors are available. This makes it difficult for structural engineer to differentiate and choose an anchor, let alone design one. In this section, we describe in detail the different types of post-installed anchoring systems available in our market. Post-installed anchors can be categorized on basis of technology, fixing material and fixing process. Let's look at each category in detail.

3.2.1 CLASSIFICATION ON BASIS OF TECHNOLOGY

On basis of working technology, post-installed anchors may be broadly classified into: Mechanical anchors and Bonded anchors. Post-installed mechanical anchors derive their strength from mechanical principles like friction and keying. The load transferring component could be metal or plastic. Whereas, post-installed bonded anchors derive their strength from bond between mortar/adhesive-fastener interface and mortar/adhesive-concrete interface. Note- In this book, the terms mortarand adhesive have been interchangeably used and refers to the chemical filled in the drilled hole prior to inserting the fastening element to form the connection.

(A) Post-installed mechanical anchors:

These fastening systems rely purely on mechanical principles like friction, keying or combination of both, for transferring the load to the base material. Based on the mechanism on basis of which the anchor develops its load carrying capacity, post-installed mechanical anchors may be further subdivided into: expansion anchors, undercut anchors and concrete screws.

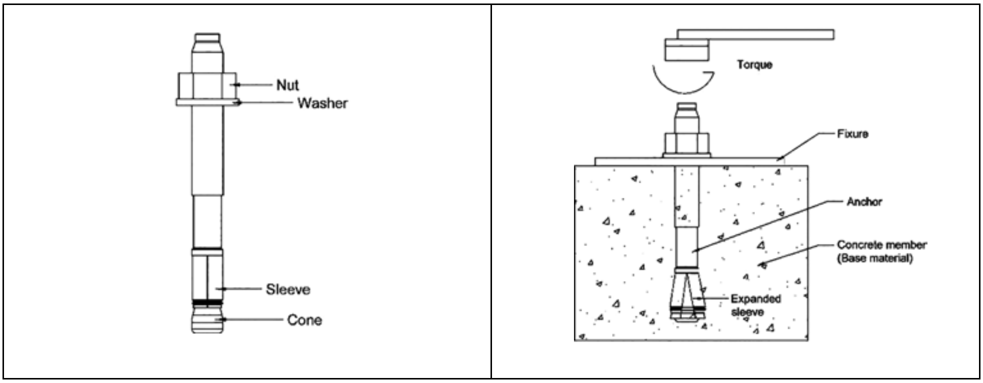
(a) Expansion anchors: These mechanical anchors derive their load carrying capacity by expansion of sleeve against the sides of the drilled hole. Based on how the expansion of sleeve is induced, expansion anchors may be further classified into the following two types –

- **(i) Torque-controlled expansion anchors:** This anchor type induces expansion of sleeve through the application of torque. As the "predefined" torque is applied on the nut, the cone is pulled into the sleeve thereby causing it to expand and press against the wall of the drilled hole. An illustration of the anchor is shown in Fig.3.1(a). Torque-controlled expansion anchors are typically used for light and medium duty applications. Their applications range from simple pipe support to beam-to-column attachment.

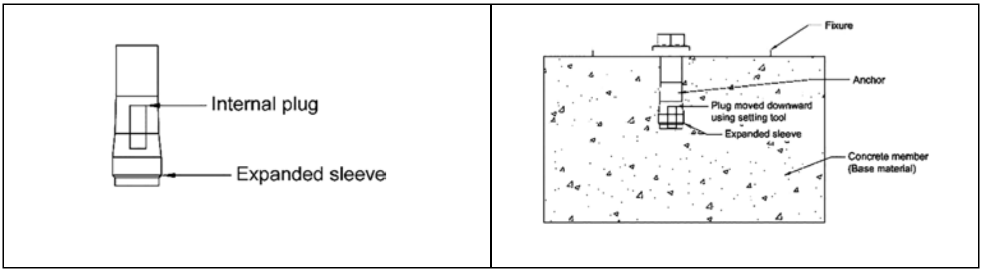
(ii) Displacement-controlled expansion anchors: This anchor type induces expansion of sleeve through displacement of embedded plug. As the plug is pushed downward by a "predefined" displacement value, the sleeve expands as illustrated in Fig.3.1(b). Displacement-controlled expansion anchors are often used for light duty pipe fixing.

(b) Undercut anchors: These mechanical anchors derive their load carrying capacity from the mechanical interlock provided by undercutting of the concrete at the embedded end of the fastener. A special drill is used to create the undercut prior to installation of the fastener. Alternatively, the undercut may be created by the fastener itself during its installation as shown in Fig.3.1(c). Undercut anchors are popular choice for heavy duty applications e.g. heavy machine support in refineries. Their load-displacement behaviour is very similar to cast-in bolts. However, the performance may vary from product to product.

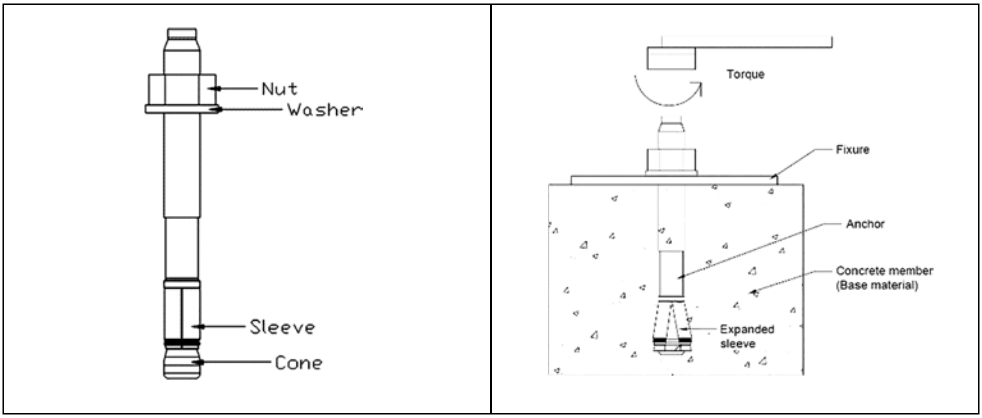
(c) Concrete screws: These mechanical anchors derive their load carrying capacity from the mechanical interlock provided by undercutting of concrete along the length of the fastener. An illustration of the anchor is shown in Fig.3.1 (d). These anchors are screwed into a pre-drilled cylindrical hole, and in the process the threads of the concrete screw cuts into the concrete thereby creating the mechanical interlock.



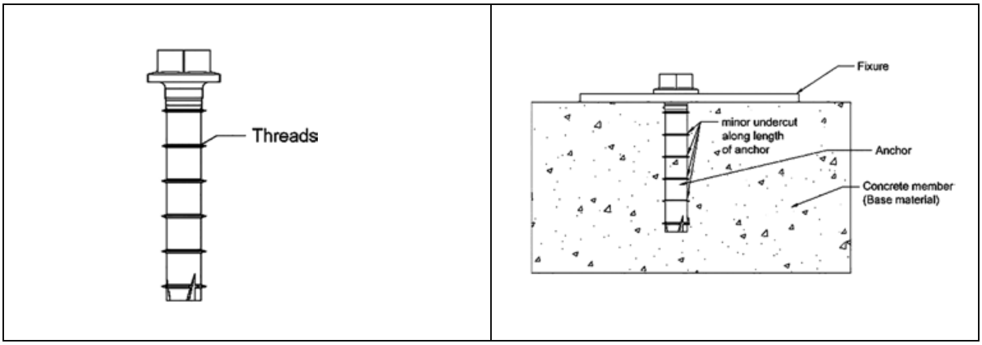
(a) Torque-controlled expansion anchor



(b) Displacement-controlled expansion anchor



(c) Undercut anchor



(d) Concrete screw

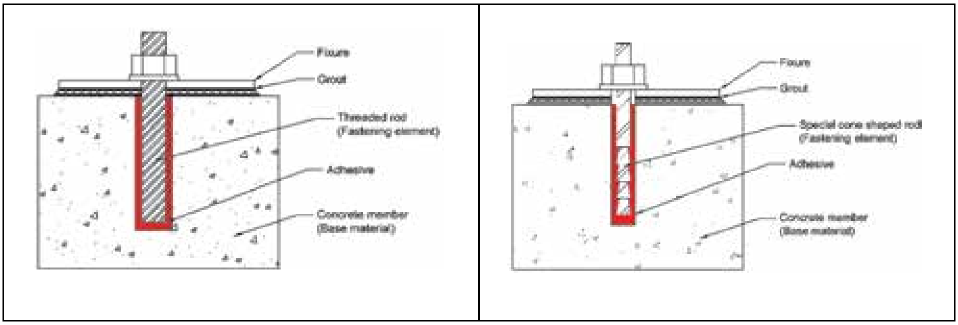
Fig.3.1.Different types of mechanical anchors

(B) Post-installed bonded anchors:

Post-installed bonded anchors utilize the adhesive property of the mortar/adhesive to form bond between adhesive-concrete interface and adhesive-fastener interface, thereby developing its load carrying capacity. Mortar/adhesive may be made of epoxy, polyester, vinyl-ester, hybrid etc. Adhesive usually has a resin and hardener component, which when mixed together gives the adhesive its strength. The adhesive is placed in pre-drilled cleaned hole and the fastening element (e.g. threaded rod, internally threaded rod etc.) is then inserted. These systems can be loaded only after the adhesive has cured and hardened. The curing time may differ from product to product and is specified by the manufacturer. Some adhesive injection systems have 100 years working life. These anchors can be further classified into: bonded anchors and bonded expansion anchors.

(a) **Bonded anchors:** The bonded anchors rely only on the adhesive property of the mortar/adhesive injected in the drilled hole. The micro interlock at concrete-adhesive interface and mechanical interlock at adhesive-fastener (e.g. threaded rod) interface help the anchor develop its load carrying capacity. An illustration of the anchor is shown in Fig.3.2(a). These anchors are preferred in applications with high tension loads.

(b) **Bonded expansion anchors:** The fastening element used is a special threaded rod with multiple steel cones in the bottom portion of the rod (see Fig.3.2(b)). The fastening element is coated with a coating that allows bond to break between concrete-adhesive interface with application of "predefined" torque after the adhesive has cured and hardened. The working mechanism is activated by application of torque, and hence the name bonded expansion anchors. On application of predefined torque moment, the specialized threaded rod moves relative to the hardened bonding compound resulting in expansion forces. This fastener essentially combines the working principle of torque-controlled expansion anchors and bonded anchors. These anchors are preferred in applications with high tension loads and for seismic applications.



(a) Bonded anchors (b) Bonded expansion anchors

Fig.3.2 Different types of bonded anchors

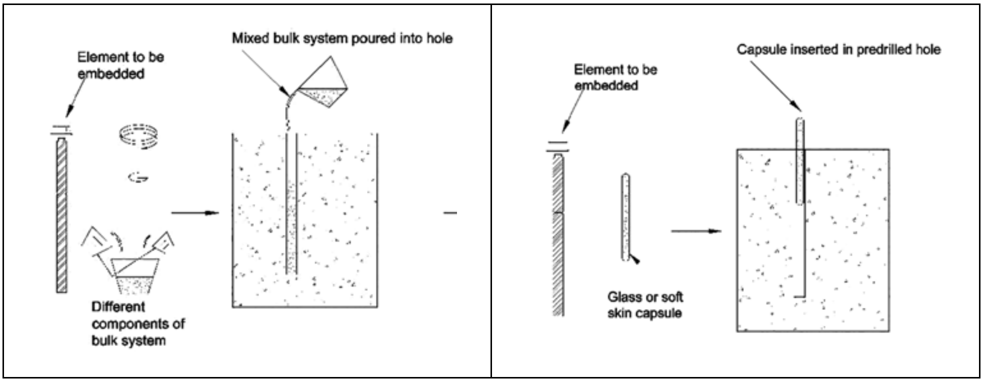
For sake of simplicity, post-installed bonded anchors are here after referred as bonded anchors in this book.

3.2.2 CLASSIFICATION ON BASIS OF ADHESIVE INSTALLATION TECHNIQUE (APPLICABLE FOR BONDED ANCHORS ONLY)

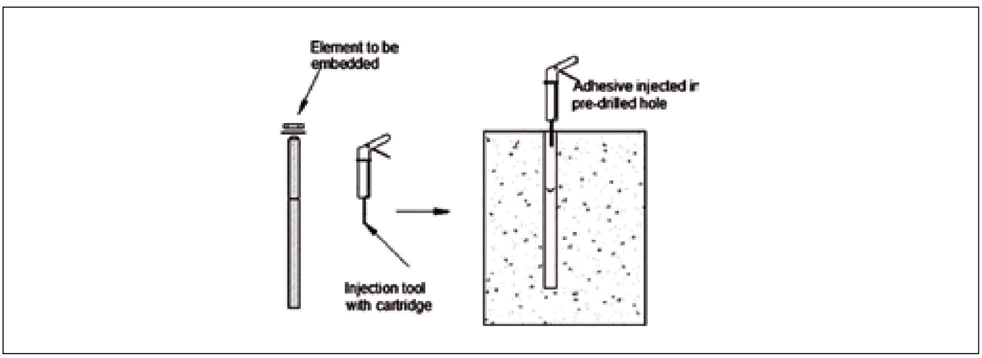
Bonded anchors can be further classified based on method of introducing the adhesive in the drilled hole: Bulk type, capsule type and injection type (see Fig.3.3)

- The **bulk systems** are traditional method of installation of adhesive. In this system, the resin and hardener component of the adhesive are fed into a bulk mixing machine in the ratio specified by the manufacturer. The mixed adhesive is then immediately piped into the drilled and cleaned hole via a hose, and the fastening element is then embedded in it. This installation method is suitable only for fasteners in vertically downward orientation.
- The **capsule systems** are relatively modern method of installation in which the resin and hardner components are packaged in a capsule made of glass, plastic etc. In this system, the capsule is placed in position in predrilled/cleaned hole and the fastening element is hammered or drilled into the capsule to rupture it and mix the two components. Multiple capsules are used in case of deeper embedment.

- The **injection systems** utilize a dispensing tool to inject the adhesive, which is packaged in a cartridge. This dispensing tool compresses the cartridge to release the two components through a metering manifold into the injecting nozzle. The fastening element is then embedded in it.



(a) Bulk system (b) Capsule system

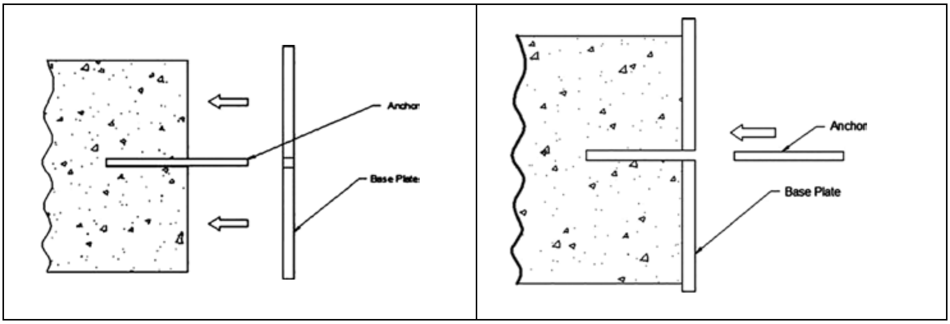


(c) Injection system

Fig. 3.3 Different types of bonded anchors based on installation method

3.2.3 CLASSIFICATION ON BASIS OF FIXING PROCESS

On basis of fixing process, post-installed anchors may be broadly classified into: Pre-fix and Through-fix type. In case of pre-fix type, the fastener is installed first and then the fixture is placed in position as shown in Fig.3.4(a). The holes on the fixture have to exactly match the fastener location in the base material. In case of through-fix type, the fixture is held in position and then the fastener is installed through it as shown in Fig.3.4(b). Depending on the application, the structural engineer may prefer to use either of the two types.



(a) Pre-fix type anchors (b) Through-fix type anchors

Fig. 3.4 Different types of post-installed anchors on basis of fixing process

3.3 PERFORMANCE INFLUENCING PARAMETERS

Just like we discussed in the previous chapter, the question arises how do we select the right solution for a given application. Which parameters should be considered? In this section, we try to address this aspect to some extent. Just like cast-in anchor channels, there are no specification or dimension-based standards for post-installed anchors the engineers are used to in our country. There are standards that spell out manufacturing tolerances, material grades etc. for fasteners like screws for steel-to-steel connections and for traditional cast-in bolts. However, the same format cannot be adopted for post-installed anchors as they are also innovative products like cast-in anchor channels. The manufacturers spend huge sum of money as well as other resources to develop and patent the innovative technology and are not willing to share this confidential data. Therefore, it is not possible to standardize manufacturing processes, tolerances etc. to standardize the product. This implies that the same type of product from two manufacturers may exhibit different load-displacement behaviour, though they may appear to be identical. The performance of post-installed anchors is also evaluated and documented according to a standardized "Performance Assessment Document" by accredited laboratories that allows the structural engineer to compare the performance of two similar looking products or two different types of anchors, without having to bother about manufacturer's specifications.

This "Performance Assessment Document" evaluates the fastening system as a whole (i.e. anchor and base material) under a given set of conditions and assess the influence of different installation and in-service parameters. Some of these parameters are discussed in this section.

3.3.1 INSTALLATION PARAMETERS

The way the anchor is installed, in what material it is installed and where it is positioned can all influence the performance and load-displacement behaviour of the post-installed anchors. Any variation from standard installation procedure recommended by the manufacturer may also negatively influence its performance. The effect of these parameters may vary depending on the anchor type and from product to product.

(A) Concrete grade, type and condition:

As anchors utilize the tensile capacity of concrete to transfer the loads, it can be considered one of the most important installation parameters [7]. If the concrete is honeycombed or is not compacted properly, it will definitely have a negative impact on load carrying capacity irrespective of the type of post-installed anchor. The concrete at site should match the specifications considered at the time of design. For example, if M30 grade concrete is assumed for design then the concrete strength at site should at least correspond to that of M30 or higher. If the construction quality is found to be poor and the concrete strength is not as assumed, then the connection should be redesigned based on the actual strength. The impact of concrete strength on performance of post-installed anchors is also product dependent and does not demonstrate any consistent trend. The type of aggregate used for preparing concrete may also influence the performance of anchor.

The condition of concrete – dry, wet or submerged, also influences the load carrying capacity of the anchor depending on the type. Post-installed bonded anchors are especially sensitive to the condition of concrete. If the surface of the drilled hole is wet, it will hamper bonding at adhesive-concrete interface and thereby reducing its capacity. Not all products are suitable for use in wet, underwater or submerged applications.

(B) Condition of drilled hole surface:

The method used to drill and clean the hole can change the behaviour and load capacity of the post-installed anchors. Mostly manufacturers recommend hammer drill for installing anchors as the drilled hole has rough surface which aids in friction and bonding. At times the application may require the holes to be diamond cored. The holes drilled using diamond core technique are smoother compared to hammer drill and may negatively impact performance of anchors. The diamond cored holes are also coated with a fine layer of drilling dust which further hampers the working mechanism of anchors. Again, not all products are suitable for use in diamond cored holes. There are tools available in the market that may be used to roughen the hole surface. The load capacity may also differ depending on the type of drilling method and accessories used and should be considered in design.

The way and extent to which the drilled hole is cleaned also bears implication on performance. The cleaning procedure is specified by the manufacturer for a given product. If the hole is not cleaned as per recommendation, it can negatively impact load carrying capacity. The loose concrete particles deposited on the inside surface of the hole during drilling process create a partial bond-breaker/ influences surface friction. The effect of extent of cleaning on behaviour is assessed and is considered as one of the parameters for determining “installation safety factor” for the fastening system, which should be considered in design. There are systems available in market that eliminate cleaning or are not significantly impacted by the cleaning process. These systems definitely offer an edge over the regular variants as they reduce dependency on accuracy of installer w.r.t cleaning. Also, appropriate cleaning tools are often not available at construction sites.

(C) Dimension, position and orientation of the anchor:

The load carrying capacity may change with anchor diameter and embedment depth. There is no specific trend as it is anchor type and product dependent. The extent of influence of the embedment depth also depends on the location of the anchor. Each product has a different value of minimum spacingand edge distancedepending on performance assessment. This value needs to be considered for design and installation. For certain anchor types like post-installed bonded anchors, the orientation of anchor i.e. vertically upward, horizontal, inclined or vertically downward, may affect the performance. Especially in overhead applications, it may be difficult to inject the adhesive as it may have a tendency to run out of hole and drip, resulting in voids or air pockets. The presence of voids in turn hampers the performance and load carrying capacity. As structural engineer, we should check suitability of the product for use in a specific orientation before selecting it for design.

(D) Hole drilling diameter/ annular gap:

"Annular gapis the gap between the fastening element and concrete surface of the drilled hole, which is filled with adhesive. The load-displacement behaviour of post-installed bonded anchor is affected by the size of the annular gap. If this gap is bigger in size, then the bond between the concrete and the adhesive may be disrupted due to shrinkage of adhesive during curing. Whereas, smaller annular gap may not allow the adhesive to be uniformly applied on the surface of fastening element. The hole drilling diameter is also important for post-installed mechanical anchors. It has to be of the size recommended by the manufacturer. The effect varies depending on the type of anchor.

(E) Installation temperature (applicable only for bonded anchors):

The temperature that the anchor is installed in can affect the performance of post-installed bonded anchors. For example, it becomes harder to inject adhesive at low temperatures due to increase in viscosity. The product should be designed for and installed at the temperature range specified in its "Technical Assessment" report. The curing time i.e. the time from beginning of the chemical reaction to hardening of the adhesive, is also dependent on the installation temperature and varies from product-to-product. The minimum curing time specified by the manufacture should be observed to achieve the full specified load carrying capacity.

(F) Additional installation parameters (applicable only for mechanical anchors):

For post-installed mechanical anchors, the recommended installation procedure should be followed w.r.t to torque, no. of hammer blows etc., depending on the anchor type. If a torque-controlled anchor is under-torqued, it may not develop the desired load capacity. If it is over-torqued, it may fail. If a displacement-controlled anchor is under-hammered, the sleeve may not expand to the desired extent thereby reducing its load carrying capacity.

3.3.2 IN-SERVICE PARAMETERS

The environment in which the anchor is installed, the temperature it is subjected to in-service and the load duration it is subjected to, will affect its behaviour and capacity.

(A) Environment:

The environment encompasses the direct exposure condition that the post-installed anchor is subjected to. In outdoor conditions, the anchor may be subjected to rain. In indoor application, the anchor will not be subjected to such harsh conditions in a regular structure. However, the anchor may be subjected to chemicals in structures like factory buildings. Each of these exposure conditions will affect the behaviour (e.g. it may cause corrosion) and anchor suitable for use in such condition should only be used for that application. For example, stainless steel grade anchor is usually used for outdoor applications and galvanized anchors are used for indoor applications. Whereas, high corrosion resistance material anchors are used for marine and toxic exposure conditions. The performance of post-installed bonded anchors may also degrade when exposed to freeze-thaw cycle and chemicals.

(B) Temperature:

The temperature that the anchor is installed in (as stated earlier) as well as the temperature it is subjected to during its service life, can both effect the anchor performance. The effect of temperature is more pronounced in case of post-installed bonded anchors compared to post-installed mechanical anchors. For example, it becomes harder to inject adhesive at low temperatures due to increase in viscosity. The product should be designed for and installed at the temperature range specified in “Technical Assessment” report.

(C) Load duration:

This parameter is critical for post-installed bonded anchors. If the anchor is not suitable for use under sustained load, the anchor may slowly pull-out of the hole over a period of time and fail.

(D) Expected state of concrete:

The expected state of concrete during the service life of the connection (i.e. uncracked or cracked) also bears implication on performance of post-installed anchors. As per limit state theory, the concrete is assumed to be cracked for most structures. The structural designer may argue that the crack assumption is built into concrete structure design as per limit state theory and need not be reconsidered for post-installed anchor design. However, this is not correct. If the crack develops and passes through the post-installed anchor at any time during its service life, it can reduce its load carrying capacity significantly (30% or more depending on product) [7]. Some anchors may also fail as they are not suitable for use in cracked concrete. Therefore, this can be considered to be one of the most important parameters for selection. If the structural engineer can establish that the concrete member in which the post-installed anchor is installed will not crack, then uncracked concrete state may be assumed for design. Clause 4.7 of EN 1992-4[6] provides guidance on establishing uncracked concrete state. In all other cases, the concrete should be assumed to be in cracked state for design.

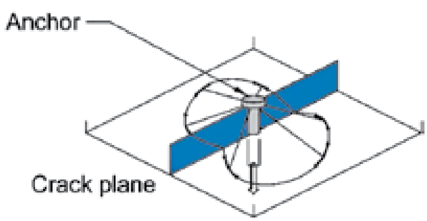


Fig. 3.5 Example of crack passing through installed post-installed anchor

3.4 CODES AND STANDARDS

We have discussed some of the parameters that influence load-displacement behaviour and load carrying capacity of post-installed anchors in the previous section. It is essential to consider these parameters in design. The only way to do it, is by using performance data of the fastening system for design. As stated in Chapter 1, there are no national standards at present for testing, assessment and design of post-installed anchors. However, there are several international standards that provide guidance on this topic. The provisions presented in this book are based on European standards and regulatory framework.

As stated in Chapter2, the construction products regulation (CPR) lays down harmonized rules for marketing of construction products in Europe. Harmonized specifications include harmonized standards and European Assessment Documents (EAD). When construction product is not covered or not fully covered by a harmonized standard, as is the case for post-installed anchors, the manufacturers can request for a "European Technical Assessment" report to be issued by one of the "Technical Assessment Bodies" on the basis of a "European Assessment Document" developed by the European Organisation for Technical Assessment (EOTA) [8].

The various installation and in-service parameters along with other relevant parameters are considered for testing and assessment as per applicable "European Assessment Document (EAD)". The performance data in terms of characteristic values and installation parameters determined based on this assessment are documented in "European Technical Assessment (ETA)" report. This performance data can be used by a structural engineer to design a connection using post-installed anchors.

3.4.1 PERFORMANCE ASSESSMENT STANDARDS

The "European Assessment Document" or Technical Report (TR) developed by "European Organisation for Technical Assessment" is used to test and assess performance of a fastening system by a third-party accredited lab and approval body recognized as "Technical Assessment Bodies" in Europe. Some of the European Assessment Documents developed for the post-installed anchor types discussed in this book are listed below. They cover both static and seismic assessment:

- EAD 330232-00-0601 [16] – European Assessment Document for "Mechanical Anchors for use in Concrete": applicable to torque-controlled expansion anchors, displacement-controlled anchors, undercut anchors and concrete screws.
- EAD 330499-00-0601 [17] – European Assessment Document for "Bonded Anchors for use in Concrete": applicable to bonded anchors and bonded expansion anchors.
- EAD 330011-00-0601 [18] – European Assessment Document for "Adjustable Concrete Screws": applicable to concrete screws involving adjustment step during installation.
- TR049 [19] – Technical Report for "Post-installed Fasteners under Seismic Action"

Each "European Assessment Document" defines the product it is applicable for, its intended use, essential performance characteristics required to fulfil the intended use, test and assessment methods to determine essential performance characteristics, and approach for verification of constancy of performance. "European Assessment Document" covers assessment procedure for static, seismic and fire. In this book, fire assessment and fire design of post-installed anchors is not covered.

As an example, Table 3.1 lists the essential performance characteristics for "Mechanical Resistance and Stability", required for post-installed mechanical anchors as per EAD 330232-00-0601 [16]. "European Assessment Document" offers 12 different assessment options, out of which Option 1 is the most comprehensive one for "Cracked" concrete application and Option 7 for "Uncracked" concrete applications. Seismic qualification is selected as an additional option. The manufacturer can choose out of these assessment options. Based on the selected option, test and assessment is carried out by third-party accredited body and an "European Technical Assessment (ETA)" for that product is issued. The post-installed anchor in concrete is tested under ideal conditions as well as non-ideal conditions in tension as well as shear. Non-ideal conditions include variation in temperature, poor installation conditions, variation in drill bit tolerance, cracked concrete etc. The no. of samples tested can range from few hundreds to thousands depending on the scope of assessment.

Table 3-1 List of essential characteristics as per EAD 330232-00-0601 [16]

Essential characteristics for “mechanical resistance and stability” for post-installed mechanical anchors		
Characteristic resistance to tension load (static or quasi-static load):	**Characteristic resistance to shear load (static or quasi-static load):**	**Characteristic resistance and displacements for seismic:**
– Resistance to steel failure	– Resistance to steel failure	– Resistance to steel failure
– Resistance to pull-out failure	– Resistance to pry-out failure	– Resistance to pull-out failure
– Resistance to concrete cone failure	– Resistance to concrete edge failure	– Fracture elongation
– Robustness	– Displacements under static and quasi-static loading	– Factor for annular gap
– Minimum edge distance and spacing	– Durability	– Displacements
– Edge distance to prevent splitting under load		

“European Technical Assessment” report documents at least the product description, its intended use (e.g. cracked or uncracked, dry or wet condition etc.), recommended design method, installation procedure, and its essential performance characteristics in terms of characteristic load values and displacements. Annex I provides a sample "European Technical Assessment" report of a post-installed anchor as an example.

3.4.2 DESIGN STANDARDS

In Europe, post-installed anchor design provisions are laid out in "EN 1992-4 (Eurocode 2): Design of Concrete Structures - Part 4: Design of Fastenings for use in Concrete" [6]. It is applicable to design of both structural and non-structural connections. It is important to note that it covers the design of anchor or anchor group only, and not the fixture. The fixture i.e. baseplate has to be designed according to applicable standards like "IS 800 - General Construction in Steel – Code of Practice" [10]. The concrete member in which the post-installed anchor is installed should be designed and detailed as per applicable standards like "IS 456 - Plain and Reinforced Concrete – Code of Practice" [11]. The design provisions given in EN 1992-4 [6] can be used only if the product has been assessed as per applicable "European Assessment Document" and has an "European Technical Assessment" report. The design utilizes the product dependent characteristic values given in "European Technical Assessment" to determine the load carrying capacity.

3.5 DESIGN PROBLEM

Before taking a deep dive into the design concept of post-installed anchors, let us first formulate the problem. As structural engineers, it is important to identify and list the factors that should be accounted for in design. This step aids in identifying the right solution. If one states that they need an anchor that can take 10 kN of load, it is wrong and incomplete problem statement. The design problem statement should clearly identify the application, the environment it will be installed in and exposed to, the value and type of load, serviceability requirement, any restriction related to construction site, preference for any technology, concrete member details as well as any member dimension restrictions.

An example of well-defined problem statement would be: attachment of a steel beam ISMB 250 to a rectangular concrete column of size 300 mm x 450 mm and of M25 concrete grade for supporting mezzanine floor of an office building. The beam is to be fixed on the column face of length 450 mm. The connection needs to be designed for a static shear load of 8 ton (DL+LL). Due to time constraint and safety concerns, solution that eliminates hole cleaning step is preferred.

This problem statement clearly highlights the important parameters that may influence the selection of the most suitable post-installed anchor for the application.

3.6 DESIGN PHILOSOPHY

The design provisions presented in this chapter are based on EN 1992-4 [6]. These provisions can be used to design only those post-installed anchors that have been assessed as per "European Assessment Document", i.e. prequalification is prerequisite for design. The design of post-installed anchors is based on the premise that the tensile capacity of concrete can be utilized to transfer load into the concrete member as illustrated in Fig. 3.6

The fixture is assumed to be rigid and the axial stiffness of all anchors as equal. This allows for assumption of linear strain distribution. The fixture may be assumed to be rigid only if it remains elastic under design forces and its deformation remains negligible in comparison to the axial displacement of the anchors. In the zone of compression under the fixture, it is assumed that the anchors do not take up normal forces and the compression forces are transmitted to the concrete by the fixture.

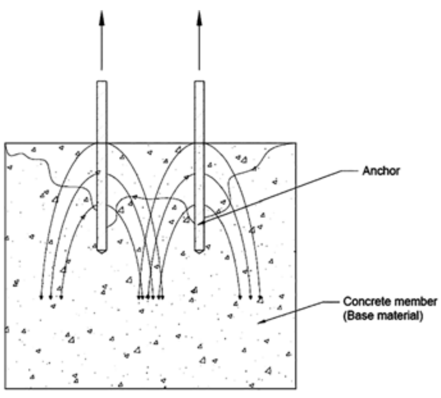
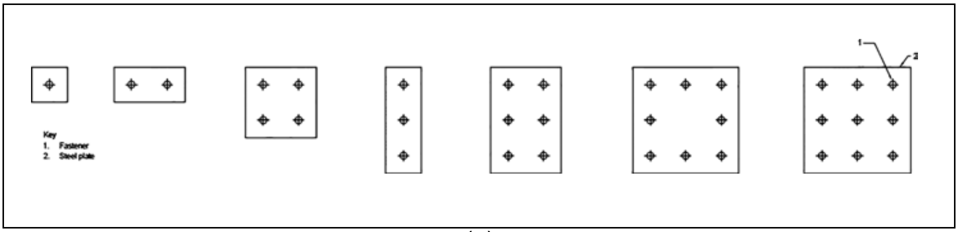
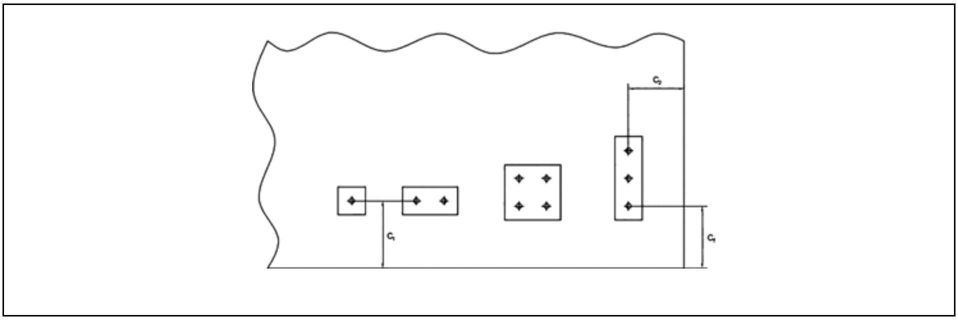


Fig. 3.6 Illustration of tensile capacity of concrete being utilized for load transfer by post-installed anchor

EN 1992-4 [6] covers design of post-installed anchors for static or quasi-static load, fatigue and seismic. The provisions given in EN 1992-4 [6] are applicable only for designing connections with design life of 50 years. It can be used for designing connections with single anchor or anchor group, with anchor diameter greater than 6 mm and embedment depth greater than 40 mm for statistically determinate systems. Some exceptions w.r.t anchor diameter and embedment depths for statistically indeterminate systems are given in EN 1992-4 [6] and are not addressed in this book. Anchors are considered as an anchor group only if anchors of the same type and size are used and the loads are applied to the individual anchors of the group by means of a common fixture. EN 1992-4[6] covers design of anchors in normal-weight concrete from M15 to M105, provided the anchor is prequalified for use in a specific concrete grade range within these limits. However, for the purpose of design the characteristic concrete strength f_{ck} shall not be taken higher than 60 MPa (cube strength), even if the concrete used in the structure is of higher grade. The cylindrical concrete strength is used in design calculations. The anchor configurations permitted are shown inFig.3.7. Anchors located at an edge distance ≥ $\max(10h_{ef};\ 60d_{nom})$ are considered to be "far" from edge else it is considered to be situated "near" to edge. Fig.3.7(a). shows permitted anchor configurations for fastenings without hole clearance for all edge distances and for all load directions, and fastenings with permitted hole clearance - situated far from edges for all load directions and situated near to edge loaded in tension only. Fig.3.7(b). shows permitted anchor configurations for fastening with hole clearance situated near to edge for all load directions.



(a)



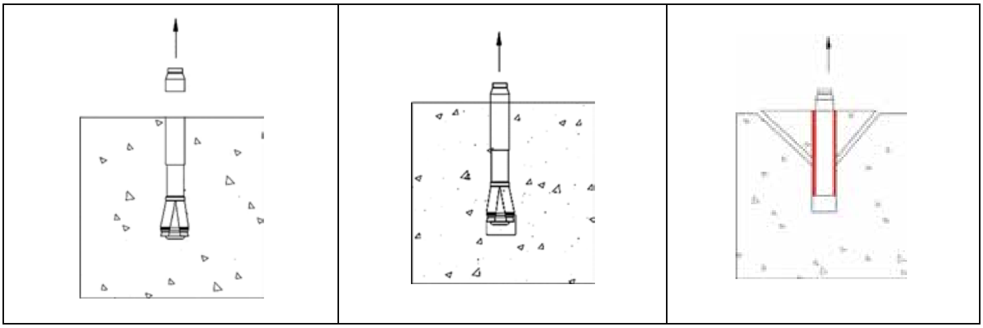
(b)

Fig.3.7 Permitted anchor configurations as per EN 1992-4 [6]

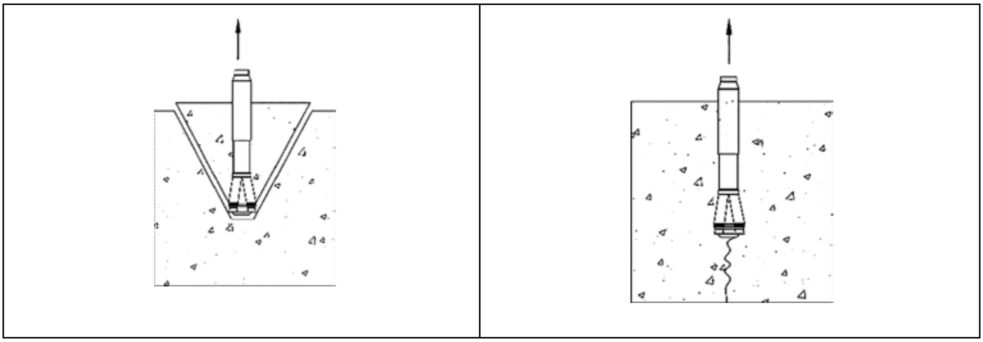
As per EN 1992-4 [6], the connection designed should be able to sustain all the design loads, not deform to inadmissible degree and remain fit for use, throughout its service life. Meaning, the ultimate limit state, the serviceability limit state and durability aspect should be addressed.

To fulfil requirement of ultimate limit state, it shall be established that the design strength (R_d) of the fastening system is higher than or equal to the design load (E_d) i.e. $E_d \leq R_d$. This check needs to be carried out for all applicable load directions (tension, shear and both combined) and all expected failure modes.

Only the anchor or anchor group effective for a specific failure mode for loads resulting from applied forces on anchor should be checked for that failure mode. The probable failure types considered for static design of post-installed anchors in tension are: - steel failure of fastener, concrete cone failure, pull-out/ bond failure of fastener and concrete splitting failure. The probable failure types considered for static design of post-installed anchors in shear are: - steel failure of fastener (with or without lever arm), concrete pry-out failure and concrete edge failure. An illustration of each failure type is given in Fig. 3.8 and Fig. 3.9

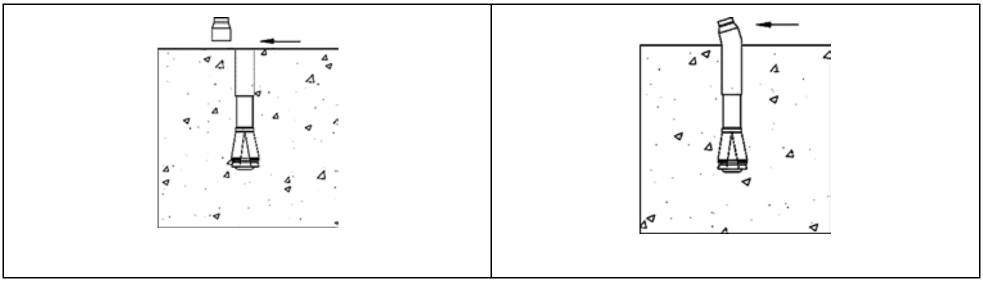


(a) Steel failure (b) Pull-out failure (c) Bond failure

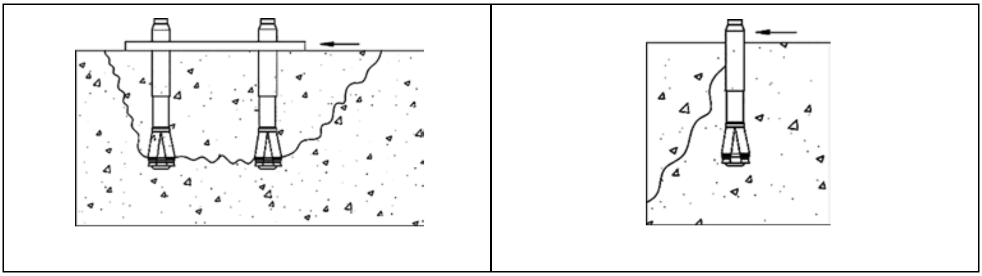


(d) Concrete cone failure (e) Concrete splitting

Fig.3.8 Probable failure modes of post-installed anchors in tension [6]



(a) Steel failure with lever arm (b) Steel failure without lever arm



(c) Concrete pry-out | (d) Concrete edge failure

Fig.3.9 Probable failure modes of post-installed anchors in shear [6]

To fulfil requirement of serviceability limit state, it shall be demonstrated that the displacements occurring under load are not larger than the admissible displacement. Aspects like corrosion protection, inspection, maintenance, replacement etc. address durability aspect. The additional requirements for seismic and fatigue design are discussed in the relevant sections.

3.7 LOAD ANALYSIS

To be able to carry out the checks for ultimate limit state, it is necessary to determine the force on each fastener. As per EN 1992-4 [6], the forces on fasteners are calculated as per elastic analysis. The connection may be subjected to tension, shear, bending or torsion. The force[1] on the connection is determined as per applicable standards e.g. IS 1893 [2]. In general, the load on the fixture may be calculated ignoring the anchor displacements. However, the effect of this displacement should be considered when the anchor is used to fix a statically indeterminate stiff element.

These forces are resolved into axial tension and/or shear acting on the anchor. If the shear forces act at a lever arm, then the fastener will be subjected to

[1] Additional requirements are given in Annex C of EN 1992-4 [6] for determination of seismic forces on connection based on type of connection but is not covered in this book as the approach is different to Indian Standard and may lead to confusion. The readers of this book are urged to refer to the standard for guidance. This book mostly focuses on determining strength of the fasteners and compares it to loads that are assumed to have been determined by Structural Engineers as per applicable standards.

bending moment as well. The effect of friction arising due to bending moment and/or compression is ignored. Only the axial compression on the fixture which is transmitted either directly to concrete or via anchor suitable for use in compression is considered in EN 1992-4 [6]. These rules for load analysis are applicable for static load case and can be applied for seismic case as well provided the fixture remains elastic in the seismic design situation. The additional requirements to be considered for load analysis for fatigue is described in the respective design section.

3.7.1 ANALYSIS OF TENSION LOADS

The normal force and bending moment acting on the rigid fixture may be resolved into tension force acting on each anchor by assuming a linear distribution of strains as illustrated in Fig.3.10A linear relation between stress and strain is assumed. As a simplification, the modulus of elasticity of concrete (E_c) and steel (E_s) may be taken as 30000 MPa and 210000 MPa, respectively in absence of relevant data.

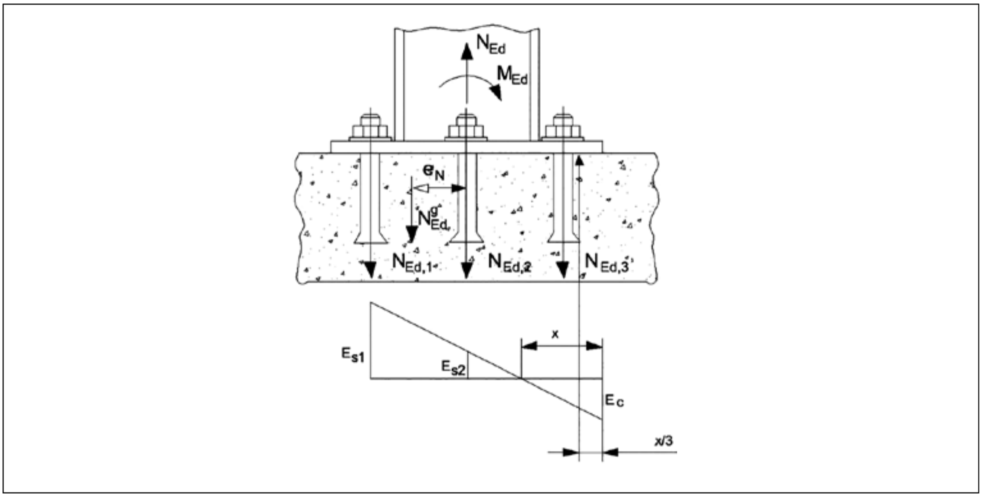


Fig. 3.10. Example of linear distribution of strain for a fastening system subjected to bending moment and normal forces (resketched based on illustration in [6])

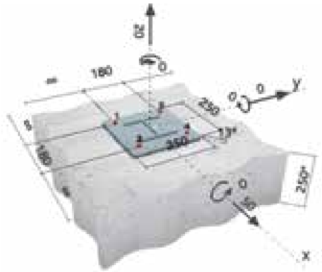
Examples 3.1 and 3.2 illustrate typical calculations necessary for determining tension force on each anchor due to applied load.

EXAMPLE 3.1

Determine the tension force acting on each post-installed mechanical anchor for the following connection formed using HST3 M12 with 60 mm nominal embedment (ETA-98/0001 [20]).

Given:

Tension load on anchor group = $20\ \mathrm{kN}$



Solution:

In this case the load will be equally distributed on each anchor.

Force on each anchor =

$$N_{Ed,1} = N_{Ed,2} = N_{Ed,3} = N_{Ed,4} = {}^{20}/_{4} = 5\ \mathrm{kN}$$

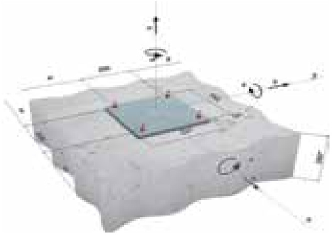
➔ Therefore, tension force of 5 kN is acting on each anchor.

EXAMPLE 3.2

Determine the tension force acting on each post-installed bonded anchor for the following connection formed using HY 200 with Hit-V 8.8 grade rods of size M12 with 109 mm effective embedment (ETA-12/0084 [21]). The anchors are spaced at 200 mm c/c. [Given – $E_s = 200000$ MPa and $E_c = 30000$ MPa; Neutral axis (NA) is located at 40.71 mm from the compression edge; Note - NA is calculated by equating tension to compression forces]

Given:

Moment on anchor group $M_{Ed} = 8\ \mathrm{kN-m}$



Anchor 2 and 4 are in tension

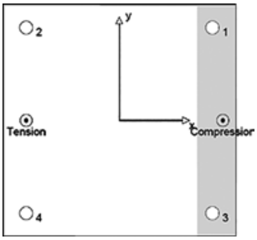
Anchor 1 and 3 in compression (due to NA location)

Solution:

$$z = 250 - 25 = 225 \text{ mm}$$

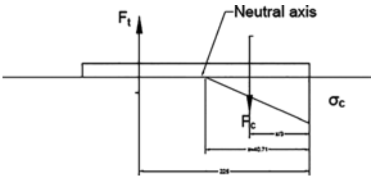
$$F_t(z - x/3) = M_{Ed}$$

$$F_t = \frac{8000}{225 - \frac{40.71}{3}} = 37.83 \text{ kN}$$



Force on each anchor $= 37.83/2 = 18.9$ kN

➔ Therefore, tension force on anchor 2 and 4 is 18.9 kN.



If the anchors in an anchor group are subjected to different values of tension forces as illustrated in Fig. 3.11, then the eccentricity e_N of the tension force of the group (N^g_{Ed}) with respect to the centre of gravity of the tensioned anchors should be calculated and accounted for in the concrete related resistances of the post-installed anchor.

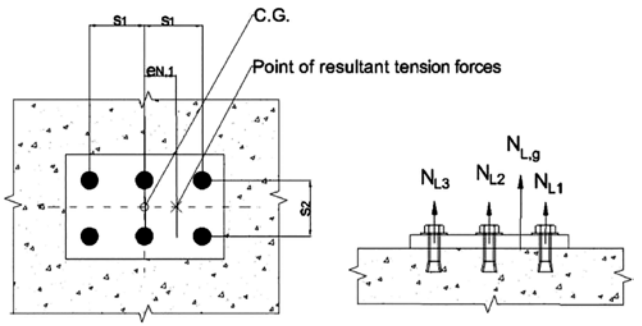
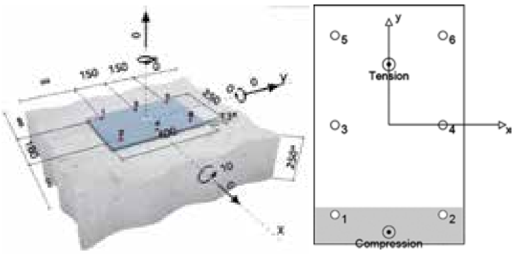


Fig.3.11. Example of anchor group subjected to eccentric tensile force in one direction (resketched based on illustration in [6])

Example 3.3 illustrates typical calculations necessary for eccentricity of applied tension load.

EXAMPLE 3.3

Determine the eccentricity of tension load acting on the anchor group shown below if the resultant tension force of 35.617 kN is acting at y = 101 mm.



Given:

Resultant tension force $= 35.617\ \mathrm{kN}$

Resultant is acting at (0,101 mm) from (0,0)

Solution:

As anchors 3, 4, 5 and 6 are acting in tension, their C.G. will be located at 0, 150/2, i.e. (x=0, y= 75 mm).

Therefore, eccentricity in y direction will be 101 – 75 = 26 mm. Eccentricity in x direction will be zero.

3.7.2 ANALYSIS OF SHEAR LOADS

The design shear force is distributed to the anchors based on its effectiveness to resist shear load, which in turn is dependent on the hole clearance (nil[2] or as

[2] If the fastener is screwed or welded to the fixture, or the gap between the fastener and the fixture is filled with mortar of compressive strength ≥ 40 N/mm^2 then it may be considered to have no hole clearance.

per Table3-2) and the edge distance. If the hole is slotted in the direction of the shear force, then the anchor doesn't take up the shear loads. All anchors are considered to take up shear load if the shear is acting parallel to the edge or are subjected to torsion or are located far from the edge ($c_i \geq \max\{10h_{ef};\ 60d_{nom}\}$). For steel and pry-out checks also all anchors of an anchor group are considered effective. For concrete edge failure check, only the anchors close to the edge ($c_i < max\{10h_{ef};\ 60d_{nom}\}$) are considered to be effective in resisting shear acting perpendicular to the edge. Components of shear loads acting away from the edge are neglected in concrete edge resistance check for anchors close to the edge.

Table 3-2 Required hole clearance in fixture [6]

External diameter of anchor (d^aor d^b_{nom}	**6 mm to 8 mm**	**10 mm to 24 mm**	**27 mm and above**
Diameter of clearance hole in fixture (d_f)	$d + 1$ or $d_{nom} + 1$	$d + 2$ or $d_{nom} + 2$	$d + 3$ or $d_{nom} + 3$

[a]If bolt bears against the fixture

[b]If sleeve bears against the fixture

After determining which anchors will resist shear, it is necessary to determine whether the shear load is acting with or without a lever arm on the post-installed anchor. The shear load on the anchor may be assumed to be acting without a lever arm if the fixture is made out of steel and is in contact with the fastener over a length of at least 0.5 times the thickness of the fixture (i.e.$0.5t_{fix}$) , and is fixed either directly to the concrete without an intermediate layer or is fixed using a levelling mortar with strength equivalent to that of concrete base material (not less than 30 N/mm^2) and with a thickness less than or equal to 0.5 times the anchor diameter (i.e. $0.5d$), under at least the full dimensions of the fixture on a rough concrete surface as intermediate layer. If this condition is not fulfilled then shear force is assumed to act with lever arm. Some exceptions to this requirement are given in EN 1992-4 [6] but are not discussed in this book.

When the shear force (V_{Ed}) is acting with lever arm (l_a), the bending moment (M_{Ed}) acting on the fastener is calculated as follows:

$$M_{Ed} = V_{Ed}(l_a/\alpha_M) \quad \text{(Eq. 3.1a)}$$

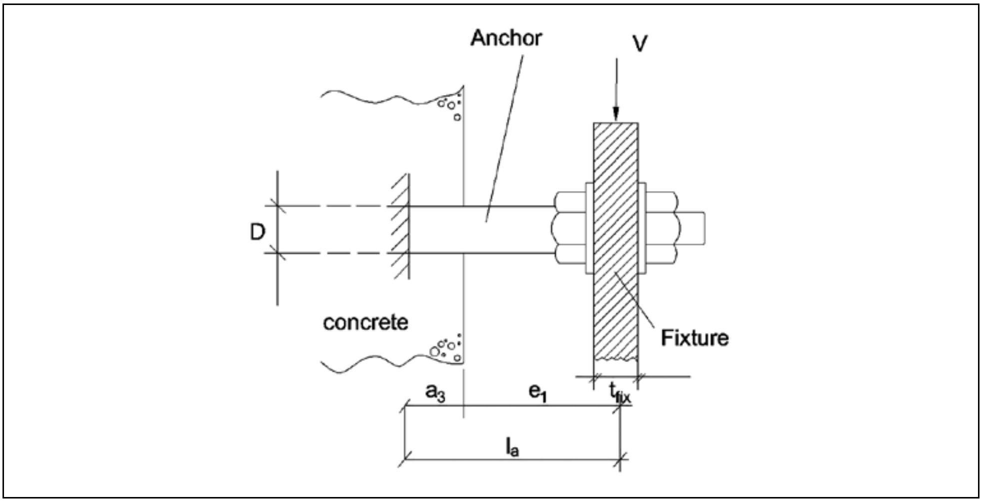
where,

α_M – is a factor that is dependent on the degree of restraint of the anchor at the side of the fixture. For no restraint (i.e. fixture can rotate freely), $\alpha_M = 1.0$. For full restraint (i.e. fixture cannot rotate), $\alpha_M = 2.0$.

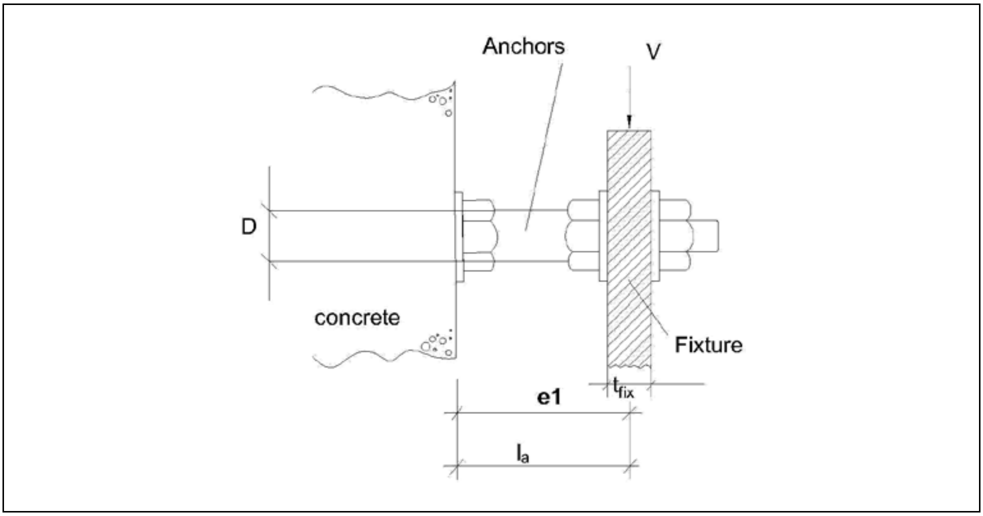
$$l_a = a_3 + e_1 \quad \text{(Eq. 3.1b)}$$

a_3 – is equal to zero if the washer and nut are directly clamped to concrete surface or to the surface of anchor channel or if a levelling grout (as per strength and thickness stated above) is used. For all other cases, it is equal to 0.5 times the nominal diameter of the anchor.

e_1 – is the distance between shear load and concrete surface. The grout thickness is ignored for calculating this distance.



(a) Stand-off installation



(b) Stand-off installation with nut and washer

Fig. 3.12 Lever arm for shear – Stand-off installation [6]

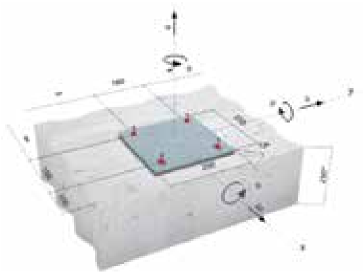
Examples 3.5 to 3.6 illustrate typical calculations necessary for determining shear force or bending force on each anchor due to applied load.

EXAMPLE 3.4

Determine the shear force acting on each post-installed mechanical anchor for the following connection formed using HST3 M12 with 60 mm nominal embedment (ETA-98/0001 [20]).

Given:

Shear load on anchor group $= 20\ \text{kN}$



Solution:

In this case the load will be equally distributed on each anchor.

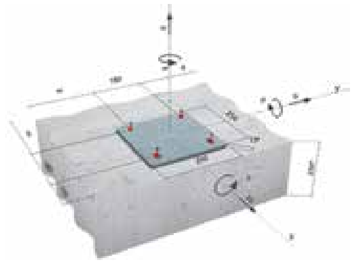
Force on each anchor =

$$V_{Ed,1} = V_{Ed,2} = V_{Ed,3} = V_{Ed,4} = {}^{20}/_{4} = 5\ \text{kN}$$

➔ Therefore, shear force of 5 kN is acting on each anchor.

EXAMPLE 3.5

Determine the shear force acting on each post-installed mechanical anchor for the following connection formed using HST3 M12 with 60 mm nominal embedment (ETA-98/0001 [20]).



Given:

Shear load on anchor group $= 20\ \mathrm{kN}$

Solution:

In this case the load will be equally distributed to anchor 2 and 4

Force on each anchor =

$$V_{Ed,2} = V_{Ed,4} = {}^{20}/_{2} = 10\ \mathrm{kN}$$

$V_{Ed,1} = V_{Ed,3} = 0\ \mathrm{kN}$ (due to slotted holes)

➔ Therefore, shear force of 10 kN is acting on anchor 2 and 4. No shear force is acting on anchor 1 and 3.

EXAMPLE 3.6

Determine the bending moment acting on anchor due to shear load for the following connection formed using HST3 M12 with 60 mm nominal embedment (ETA-98/0001 [20]).

Given:

Shear load on anchor group $= 10\ \mathrm{kN}$

Solution:

$V_{Ed} = \frac{10}{4} = 2.5\ \mathrm{kN}$ (on each anchor)

$\alpha_M = 2.0$ (full restraint case)

$e_1 = 30 + \frac{13}{2} = 36.5\ \mathrm{mm}$

$l_a = a_3 + e_1 = 36.5\ \mathrm{mm}$

$M_{Ed} = V_{Ed}\,(l_a/\alpha_M) = 2.5 * (36.5/2) = 45.6\ \mathrm{kN-mm}$

➔ $M_{Ed} = 0.045\ \mathrm{kN-m}$

3.8 DESIGN FOR STATIC OR QUASI-STATIC LOADS

A connection formed using post-installed anchors when subjected to static loads increasing from zero to the stage it may fail, there are 5 different ways in which it may fail in tension and 3 different ways in which it may fail in shear (as shown in Fig.3.9 and Fig.3.9) depending upon which one is the weakest. Design strength is therefore determined by dividing characteristic resistance to each failure, by recommended partial safety factor (see Fig.3.13). This design strength (R_d) is compared to design load (E_d) to establish $E_d \leq R_d$, for all applicable load directions (tension, shear and both combined) and all expected failure modes.

Failure type related to steel failure – anchor channels	**Partial factor for permanent and transient design situations**
Tension	$\gamma_{Ms} = 1.2\, f_{uk}/f_{yk} \geq 1.4$
Shear with and without lever arm	$\gamma_{Ms} = \begin{cases} f_{uk}/f_{yk} \geq 1.25 & When\ f_{uk} \leq 800\ N/mm^2 and\ f_{uk}/f_{yk} \leq 0.8 \\ 1.5 & When\ f_{uk} > 800\ N/mm^2 and\ f_{uk}/f_{yk} > 0.8 \end{cases}$
Concrete cone failure, concrete edge failure, concrete blow-out failure and concrete pry-out failure	$\gamma_{Mc} = \gamma_c \gamma_{inst}$ where, $\gamma_c = 1.5$ * $\gamma_{inst} \begin{cases} = 1 & for\ headed\ fasteners\ and\ anchor\ channels\ ** \\ \geq 1 & for\ post-installed\ fasteners\ in\ tension;\ also\ see\ ETA \\ = 1 & for\ post-installed\ fasteners\ in\ shear \end{cases}$ * This value is as per EN 1992-1-1 [14]; for seismic repair & strengthening of existing structures refer Eurocode 8 [15] * *as per EN 1992-4 [6] section 4.6 in tension & shear
Concrete splitting failure	$\gamma_{Msp} = \gamma_{Mc}$
Pull-out and bond failure	$\gamma_{Mp} = \gamma_{Mc}$

Fig. 3.13 Excerpt of Table 4.1 of EN 1992-4 – partial safety factors [6]
(Note – refer the standard for complete table)

3.8.1 DESIGN CHECKS FOR TENSION LOAD

(A) Resistance to steel failure

As per EN 1992-4 [6], the design steel strength in tension ($N_{Rd,s}$) is determined by dividing characteristic steel resistance ($N_{Rk,s}$) by recommended partial material safety factor (γ_{Ms}) as shown in Eq. 3.2a and Eq. 3.2b. The characteristic values for resistance to steel failure ($N_{Rk,s}$) should be taken from the product's Technical Assessment report like ETA. The design steel strength should be greater than the design load (N_{Ed}). The design tension load on most stressed anchor in an anchor group is denoted as N^h_{Ed} and on entire anchor group is denoted as N^g_{Ed}. The check for steel failure is carried out for the most unfavourably loaded anchor in an anchor group.

$N_{Rd,s} = N_{Rk,s}/\gamma_{Ms} \geq N_{Ed}$ [for single anchor] (Eq. 3.2a)

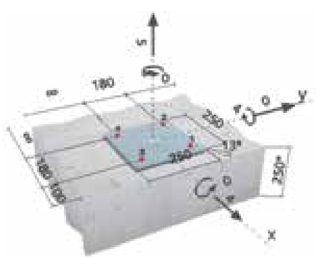
$N_{Rd,s} = N_{Rk,s}/\gamma_{Ms} \geq N^h_{Ed}$ [for anchor group] (Eq. 3.2b)

Examples 3.7 and 3.8 illustrate typical calculations necessary for checking resistance to steel failure in tension.

EXAMPLE 3.7

Check resistance to steel failure in tension for the following connection formed using HST3 M12 with 80 mm nominal embedment (ETA-98/0001 [20]). [Given - tension force on anchor 1 & 3 is 0.283 kN and on anchor 2 & 4 is 11.128 kN, obtained after resolving the load on the connection.]

Given:



$N^h_{Ed} = 11.128$ kN (on most stressed anchors 2&4)

Solution:

$N_{Rk,s} = 45.1$ kN (from Table C2 of ETA-98/0001)

$\gamma_{Ms} = 1.4$ (from Table C2 of ETA-98/0001)

$N_{Rd,s} = N_{Rk,s}/\gamma_{Ms} = 45.1/1.4 = 32.214 \text{ kN} \geq N^h_{Ed}$

$\beta = N^h_{Ed}/N_{Rd,s} = 11.128/32.214 = 0.345$ → 35% utilization. OK

EXAMPLE 3.8

Check resistance to steel failure in tension for the following connection formed using HY 200 with Hit-V 8.8 grade rods of size M12 with 109 mm effective embedment (ETA-12/0084 [21]). [Given - tension force on anchor 2 & 4 is 18.9 kN. Anchor 1 & 3 are in compression as determined in Ex. 3.2]

Given:

$N^h_{Ed} = 18.9$ kN (on most stressed anchors 2 & 4)

Solution:

$N_{Rk,s} = A_s f_{uk}$ (from Table C1 of ETA-12/0084)

$\gamma_{Ms} = 1.5$ (from Table C1 of ETA-12/0084)

$f_{uk} = 800\ \text{N/mm}^2$ (from Table A1 of ETA-12/0084)

$A_s = 84.3\ \text{mm}^2$ (from Hilti FTM 2018)

$$N_{Rd,s} = (A_s f_{uk})/\gamma_{Ms} = (800 * 84.3)\ /1.5 = 44960\ \text{N} = 44.9\ \text{kN} \geq N^h{}_{Ed}$$

$\beta = N^h_{Ed}/N_{Rd,s} = 18.9/44.9 = 0.42$ → 42% utilization. OK

(B) Resistance to anchor pull-out

This check is applicable only for post-installed mechanical anchors. As per EN 1992-4 [6], the design pull-out strength of the anchor in tension ($N_{Rd,p}$) is determined by dividing characteristic pull-out resistance,$N_{Rk,\text{p}}$, by recommended partial safety factor, γ_{Mp}, as shown in Eq. 3.3a and Eq. 3.3b. The characteristic value for resistance to pull-out failure ($N_{Rk,p}$) should be taken from the product's Technical Assessment report like ETA. The check for pull-out failure should be carried out for the most unfavourably loaded anchor in an anchor group.

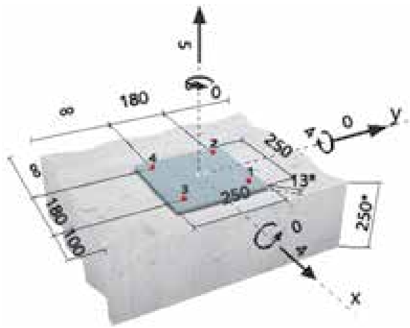
$N_{Rd,p} = N_{Rk,p}/\gamma_{Mp} \geq N_{Ed}$ [for single anchor] (Eq. 3.3a)

$N_{Rd,\text{p}} = N_{Rk,p}/\gamma_{Mp} \geq N^h_{Ed}$ [for anchor group] (Eq. 3.3b)

Example 3.9 illustrates typical calculations necessary for checking resistance to pull-out failure in tension.

EXAMPLE 3.9

Check resistance to pull-out failure for the following connection formed using HST3 M12 with 80 mm nominal embedment (ETA-98/0001 [20]). The base material is of M25 grade concrete. Given - tension force on anchor 2 & 4 is 0.283 kN and on anchor 2 & 4 is 11.128 kN, obtained after resolving the load on the connection. Assume concrete to be cracked for design.



Given:

N^{h}_{Ed} = 11.128 kN (on most stressed anchors 2&4)

Solution:

$N_{Rk,p}$ = 20 kN (from Table C2 of ETA-98/0001)

$\gamma_{inst} = 1$ (from Table C2 of ETA-98/0001)

$\gamma_{Mp} = \gamma_c * \gamma_{inst} = 1.5 * 1 = 1.5$

$N_{Rd,p} = N_{Rk,p}/\gamma_{Mp} = 20\ /1.5 = 13.33 \text{ kN} \geq N^{h}_{Ed}$

$\beta\ = N^{h}_{Ed}\ /N_{Rd,\text{p}} = 11.128/13.33 = 0.833$ → 83% utilization. OK

(C) Resistance to bond failure

This check is applicable only for post-installed bonded anchors. In EN 1992-4 [6], bond failure of post-installed bonded anchors is referred to as "Combined pull-out and concrete failure". The design bond strength of the anchor in tension ($N_{Rd,p}$) is determined by dividing characteristic bond resistance, $N_{Rk,\text{p}}$, by recommended partial safety factor, γ_{Mp}, as shown in Eq. 3.4a and 3.4b. The check for bond failure is carried out for group as a whole.

$N_{Rd,p} = N_{Rk,p}/\gamma_{Mp} \geq N_{Ed}$ [for single anchor] (Eq. 3.4a)

$N_{Rd,\text{p}} = N_{Rk,p}\ \ /\gamma_{Mp} \geq N^{g}_{Ed}$ [for anchor group] (Eq. 3.4b)

The characteristic bond strength ($N_{Rk,p}$) of an anchor or anchor group with embedment depth of h_{ef} and diameter d, should be determined according to Eq. 3.4c. In Eq. 3.4c, $N_{Rk,p}^{0}$ is characteristic bond strength of single anchor whose strength is not influenced by adjacent anchor, or edges of concrete member and is calculated according to Eq. 3.4d. This factor is dependent on the characteristic bond strength (τ_{Rk}) and should be taken from ETA depending on the state of concrete i.e. $\tau_{Rk,ucr}$ for uncracked and $\tau_{Rk,cr}$ for cracked. The factor $N_{Rk,p}^{0}$ also takes into account the effect of sustained load through the Ψ_{sus} factor, which is dependent on Ψ_{sus}^{0} and α_{sus}. The factor Ψ_{sus}^{0} is taken from ETA. If the value of Ψ_{sus}^{0} is not given in ETA then it should be taken as 0.6. The factor α_{sus} in Eq. 3.4d is the ratio of the sustained load to the total load. f_{ck} is nominal characteristic compressive strength measured on cylinder of size 150 mm diameter by 300 mm height.

The area ratio $A_{p,N}/A_{p,N}^{0}$ in Eq. 3.4c accounts for the effect of axial spacing and edge distance. In this ratio, $A_{p,N}^{0}$ is calculated according to Eq. 3.4e and $A_{p,N}$ is the actual bond influence area (see Fig. 3.14) limited by overlapping areas of adjacent fasteners ($s \leq s_{cr,Np}$) as well as by edges of the concrete member ($c \leq c_{cr,Np}$), where $c_{cr,Np}$ and $s_{cr,Np}$ are critical edge and spacing respectively.

In Eq. 3.4c, the factor $\Psi_{g,Np}$ accounts for the group effect of closely spaced n anchors on the characteristic bond strength and is calculated according to Eq. 3.4f. In Eq. 3.4g, the factor k_3 is taken as 11 for uncracked concrete and 7.7 for cracked concrete. If the anchors are spaced at unequal distances, then the mean spacing should be used in Eq. 3.4g.

The factor $\Psi_{s,Np}$ accounts for the effect of edge of concrete member on the characteristic bond strength and is calculated on basis of the smallest edge distance, c,as per Eq. 3.4h.

The factor $\Psi_{re,N}$,also known as shell spalling factor, accounts for the effect of dense reinforcement on capacity of anchors with effective embedment depth less than or equal to 100 mm (i.e. $h_{ef} \leq 100$ mm) and is calculated according to Eq. 3.4i. The factor $\Psi_{re,N}$ may be assumed to be 1 provided 10 mm or smaller bar is spaced at distance $\geq$ 100 mm, or the reinforcement (irrespective of the bar diameter) is spaced at a distance ≥ 150 mm. If the reinforcement is provided in two directions, then this reinforcement spacing requirement should be fulfilled in both directions.

The factor $\Psi_{ec,Np}$ accounts for the effect of eccentricity of tension load e_N which arises due to different loads acting on individual anchors and is calculated according to Eq. 3.4j. If there is eccentricity in two directions, then the factor $\Psi_{ec,Np}$ should be determined for each direction and the product of the two should be used in Eq. 3.4c.

$$N_{Rk,p} = N^0_{Rk,p} \, \frac{A_{p,N}}{A^0_{p,N}} \, \Psi_{g,Np} \, \Psi_{s,Np} \, \Psi_{re,Np} \, \Psi_{ec,Np} \qquad \text{(Eq. 3.4c)}$$

where,

$$N^0_{Rk,p} = \Psi_{sus}\tau_{Rk}\pi d h_{ef}, \text{where } \begin{cases} For\,\alpha_{sus} \le \Psi^0_{sus}, & \Psi_{sus} = 1 \\ For\,\alpha_{sus} > \Psi^0_{sus}, & \Psi_{sus} = \Psi^0_{sus} + 1 - \alpha_{sus} \end{cases} \qquad \text{(Eq. 3.4d)}$$

$$A^0_{p,N} = s_{cr,Np} \cdot s_{cr,Np}, \qquad \text{where } s_{cr,Np} = 7.3d(\Psi_{sus}\tau_{Rk})^{0.5} \le 3h_{ef}$$ [3]

(Eq. 3.4e)

$$\Psi_{g,Np} = \Psi^0_{g,Np} - (s/s_{cr,Np})^{0.5} * (\Psi^0_{g,Np} - 1) \ge 1 \qquad \text{(Eq. 3.4f)}$$

$$\Psi^0_{g,Np} = \sqrt{n} - (\sqrt{n} - 1) \cdot \left[\frac{\tau_{Rk}}{\tau_{Rk,c}}\right]^{1.5} \ge 1, \quad \text{where } \tau_{Rk,c} = (k_3/\pi d)\sqrt{h_{ef} f_{ck}} \qquad \text{(Eq. 3.4g)}$$

$$\Psi_{s,Np} = 0.7 + 0.3(c/c_{cr,Np}) \le 1, \text{ where } c_{cr,Np} = 0.5 s_{cr,Np} \qquad \text{(Eq. 3.4h)}$$

$$\Psi_{re,N} = 0.5 + h_{ef}/200 \le 1 \qquad \text{(Eq. 3.4i)}$$

$$\Psi_{ec,Np} = \frac{1}{1+2(e_N/s_{cr,Np})} \le 1 \qquad \text{(Eq. 3.4j)}$$

The Eq. 3.4c doesn't yield precise results for anchor installed in narrow members (3 or more edges). EN 1992-4 [6] recommends modification to this formula. These modifications are not discussed in this book for the sake of simplicity.

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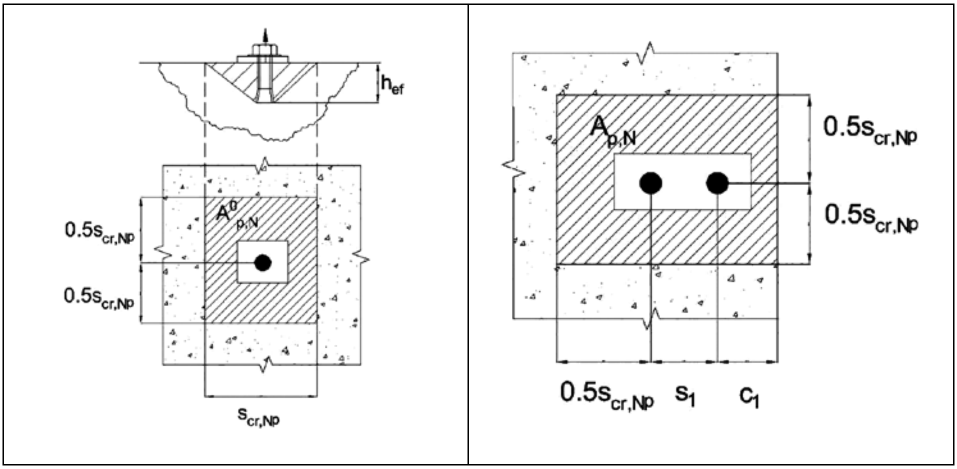
[3] Here τ_{Rk} is the characteristic bond strength corresponding to uncracked M25 grade concrete ($\tau_{Rk,ucr}$)



(a) Idealized cone (b) Projected bond influence area

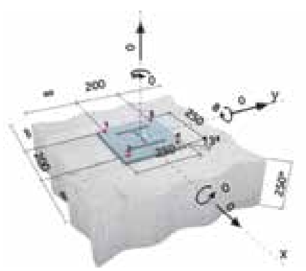
Fig. 3.14Example of idealized and project pull-out influence area (resketched based on illustration in [6])

Example 3.10 illustrates typical calculations necessary for checking resistance to bond failure in tension.

EXAMPLE 3.10

Check resistance to bond failure for the following connection formed using HY 200 with Hit-V 8.8 grade rods of size M12 with 109 mm effective embedment (ETA-12/0084 [21]). The base material is of M25 grade concrete. Assume concrete to be cracked for design. [Given - tension force on anchor 2 & 4 is 18.9 kN. Anchor 1 & 3 are in compression as determined in Ex. 3.2].

Given:



$h_{ef} = 109\ \text{mm}$

$d = 12\ \text{mm}$

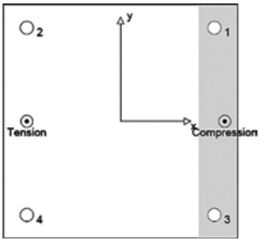
$f_{ck} = 20\ \text{N/mm}^2$ (cylinder strength corresponding to M25)

$k_3 = 7.7$ (for cracked concrete)

Solution:

$N_{Ed}^{g} = 2 * 18.9 = 37.8$ kN (only anchors 2 & 4 will act as group

to resist pull-out))



$\tau_{Rk,cr} = 8.5\ \mathrm{N/mm^2}$(from Table C1 of ETA-12/0084)

$\Psi_{sus}^{0} = 0.6$ (default, as no value is provided in ETA)

$\alpha_{sus} = 0$(as there is no sustained load)

$\Psi_{sus} = 1$(For $\alpha_{sus} \le \Psi^{0}_{sus}$)

$N_{Rk,p}^{0} = \Psi_{sus}\tau_{Rk,cr}\pi d h_{ef}$

$N_{Rk,p}^{0} = 1 * (8.5/1000) * 3.14 * 12 * 109 = 34.9\ \mathrm{kN}$

$s_{cr,Np} = 7.3d(\Psi_{sus}\tau_{Rk,ucr})^{0.5} \le 3h_{ef}$

$s_{cr,Np} = 7.3 * 12 * (1 * 18)^{0.5} = 371.6\ \mathrm{mm} > 3h_{ef} = 327$ Not OK

Use $s_{cr,Np} = 327\ \mathrm{mm}$

$c_{cr,Np} = 0.5 s_{cr,Np} = 163.5\ \mathrm{mm}$

$A_{p,N}^{0} = s_{cr,Np} \cdot s_{cr,Np} = 327 * 327 = 106929\ \mathrm{mm^2}$

$A_{p,N} = (163.5 + 200 + 163.5) * (163.5 + 163.5) = 172329\ \mathrm{mm^2}$

$A_{p,N} / A_{p,N}^{0} = 172329/106929 = 1.61$

$\tau_{Rk,c} = (k_3/\pi d)\sqrt{h_{ef} f_{ck}} = (7.7/(3.14 * 12))\sqrt{109 * 20} = 9.54\ \mathrm{N/mm^2}$

$$\Psi_{g,Np} = \sqrt{n} - (\sqrt{n} - 1) \cdot \left[\frac{\tau_{Rk}}{\tau_{Rk,c}}\right]^{1.5}$$

$$\Psi_{g,Np}^{0} = \sqrt{2} - (\sqrt{2} - 1) \cdot \left[\frac{8.5}{9.54}\right]^{1.5} = 1.066 \ge 1 \text{ OK}$$

$\Psi_{g,Np} = \Psi_{g,Np}^{0} - (s/s_{cr,Np})^{0.5} * (\Psi_{g,Np}^{0} - 1) = 1.066 - (200/327)^{0.5} * 0.066 = 1.014 \ge 1$ OK

$\Psi_{s,Np} = 0.7 + 0.3(c/c_{cr,Np}) = 1$ (due to assumption of infinite edge)

$\Psi_{re,N} = 0.5 + h_{ef}/200 = 0.5 + 109/200 = 1.045 > 1$

Use $\Psi_{re,N} = 1$

$\Psi_{ec,Np} = 1$ (as there is no eccentricity of load)

$$N_{Rk,p} = N^0_{Rk,p} \, A_{p,N}/A^0_{p,N} \, \Psi_{g,Np} \, \Psi_{s,Np} \, \Psi_{re,Np} \, \Psi_{ec,Np}$$

$N_{Rk,p} = 34.9 * 1.61 * 1.014 * 1 * 1 * 1 \approx 57 \text{ kN}$

$\gamma_{Mp} = 1.5$ (from Table C1 of ETA-12/0084)

$N_{Rd,\mathrm{p}} = N_{Rk,p} / \gamma_{Mp} = 57 / 1.5 = 38 \text{ kN} \geq N^g_{Ed}$

$\beta = N^g_{Ed} / N_{Rd,\mathrm{p}} = 37.8/38 = 0.994$ → 99% utilization. OK

(D) Resistance to concrete cone failure

The design concrete cone strength of the anchor in tension ($N_{Rd,c}$) is determined by dividing characteristic bond resistance, $N_{Rk,\mathrm{c}}$, by recommended partial safety factor, γ_{Mc}, as shown in Eq. 3.5a and 3.5b. The check for concrete cone failure is carried out for group as a whole.

$N_{Rd,c} = N_{Rk,c}/\gamma_{Mc} \geq N_{Ed}$ [for single anchor] (Eq. 3.5a)

$N_{Rd,\mathrm{c}} = N_{Rk,c}/\gamma_{Mc} \geq N^g_{Ed}$ [for anchor group] (Eq. 3.5b)

The characteristic concrete cone strength ($N_{Rk,c}$) of an anchor or anchor group with embedment depth of h_{ef} and diameter d, should be determined according to Eq. 3.5c. In Eq. 3.5c, $N^0_{Rk,c}$ is characteristic concrete cone strength of single anchor whose strength is not influenced by adjacent anchor or edges of concrete member and is calculated according to Eq. 3.5d using the factor k_1 depending on state of concrete i.e. $k_{ucr,N}$ for uncracked and $k_{cr,N}$ for cracked as per ETA. f_{ck} is nominal characteristic compressive strength measured on cylinder of size 150 mm diameter by 300 mm height.

The area ratio $A_{c,N}/A^0{}_{c,N}$ in Eq. 3.5c accounts for the effect of axial spacing and edge distance. In this ratio, $A^0_{c,N}$ is calculated according to Eq. 3.5e and $A_{c,N}$ is the actual projected cone area (see Fig.3.15) limited by overlapping areas of adjacent fasteners ($s \leq s_{cr,N}$) as well as by edges of the concrete member ($c \leq c_{cr,N}$).

The factor $\Psi_{s,Np}$accounts for the effect of edge of concrete member on the characteristic bond strength and is calculated on basis of the smallest edge distance, c,as per Eq. 3.5f.

The factor $\Psi_{re,N}$,also known as shell spalling factor, accounts for the effect of dense reinforcement on capacity of anchors with effective embedment depth less than or equal to 100 mm (i.e. $h_{ef} \leq 100$ mm) and is calculated according to Eq. 3.5g. The factor $\Psi_{re,N}$ may be assumed to be 1 provided 10 mm or smaller bar is spaced at distance ≥ 100 mm, or the reinforcement (irrespective of the bar diameter) is spaced at a distance ≥ 150 mm. If the reinforcement is provided in two directions, then this reinforcement spacing requirement should be fulfilled in both directions.

The factor $\Psi_{ec,N}$accounts for the effect of eccentricity of tension load e_N which arises due to different loads acting on individual anchors and is calculated according to Eq. 3.5h. If there is eccentricity in two directions, then the factor $\Psi_{ec,N}$ should be determined for each direction and the product of the two should be used in Eq. 3.5c.

The factor $\Psi_{M,N}$ accounts for the effect of compression force between fixture and concrete surface in case the connection is subjected to bending moments with or without axial force and is calculated according to Eq. 3.5i. If the bending moment is acting in two directions, then the value to be used in Eq. 3.5i should be determined for combined effect of the moments in two directions along with axial force.

$$N_{Rk,c} = N^0_{Rk,c} \, {A_{c,N}}/{A^0_{c,N}} \, \Psi_{s,N} \, \Psi_{re,N} \, \Psi_{ec,N} \, \Psi_{M,N} \quad \text{(Eq. 3.5c)}$$

where

$$N^0_{Rk,c} = k_1 \sqrt{f_{ck}} h_{ef}^{1.5}, \quad \text{(Eq. 3.5d)}$$

$A^0_{c,N} = s_{cr,N} \cdot s_{cr,N},$ where $s_{cr,N}$ and as $c_{cr,N}$ are as per ETA (Eq. 3.5e)

$$\Psi_{s,N} = 0.7 + 0.3(c/c_{cr,N}) \leq 1 \quad \text{(Eq. 3.5f)}$$

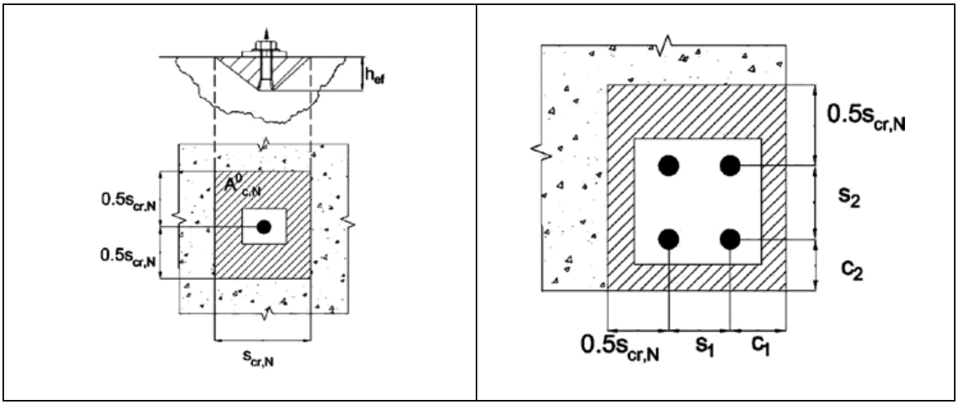
$$\Psi_{re,N} = 0.5 + h_{ef}/200 \leq 1 \quad \text{(Eq. 3.5g)}$$

$$\Psi_{ec,N} = \frac{1}{1+2(e_N/s_{cr,N})} \leq 1 \quad \text{(Eq. 3.5h)}$$

$\Psi_{M,N} = 1$,for $c < 1.5h_{ef}$ or $c \geq 1.5h_{ef}$ with C_{Ed}/N_{Ed}[4] < 0.8 or $z/h_{ef} \geq 1.5$; else

$$\Psi_{M,N} = 2 - \frac{z}{1.5h_{ef}} \geq 1, \text{ all other cases} \qquad \text{(Eq. 3.5i)}$$

The Eq. 3.5c doesn't yield precise results for anchor installed in narrow members (3 or more edges). EN 1992-4 [6] recommends modification to this formula. These modifications are not discussed in this book for the sake of simplicity.



(a) Idealized cone (b) Projected cone

Fig. 3.15 Example of idealized and project cone area (resketched based on illustration in [6])

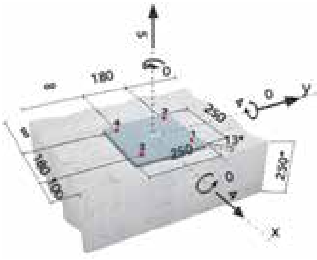
Examples 3.11 and 3.12 illustrate typical calculations necessary for checking resistance to concrete cone failure in tension.

[4] C_{Ed} is the absolute value of the resultant compression force between fixture and concrete surface and N_{Ed} is the resultant tension force of the anchors in tension; z is internal lever arm of anchor calculated according to theory of elasticity.

EXAMPLE 3.11

Check the resistance to concrete cone failure for the following connection formed using HST3 M12 with 80 mm nominal embedment (ETA-98/0001 [20]). The base material is of M25 grade concrete. Assume concrete to be cracked for design. [Given - tension force on anchor 1 & 3 is 0.283 kN and on anchor 2 & 4 is 11.128 kN, obtained after resolving the load on the connection. The load eccentricity $e_{c1,N} = 86\ \text{mm}$].

Given:



$h_{ef} = 70\ \text{mm}$ (from Table B9 of ETA-98/0001)

$$d = 12\ \text{mm}$$

$f_{ck} = 20\ \text{N/mm}^2$ (cylindrical strength corresponding to M25)

$k_1 = 7.7$ (for cracked concrete)

$c = 100\ \text{mm}$

Solution:

$N^g_{Ed} = 2 * 11.128 + 2 * 0.283 = 22.822\ \text{kN}$ (In this case all 4 anchors will act as group to resist concrete cone failure as all of them are in tension)

$N^0_{Rk,c} = k_1\sqrt{f_{ck}}h_{ef}^{1.5} = 7.7 * \sqrt{20}(70)^{1.5} = 20167\ N = 20.2\ \text{kN}$

$s_{cr,N} = 3h_{ef}$ (from Table C2 of ETA-98/0001)

$s_{cr,N} = 3 * 70 = 210\ \text{mm}$

$c_{cr,N} = 0.5 s_{cr,N} = 0.5 * 210 = 105\ \text{mm}$

$A^0_{c,N} = s_{cr,N} \cdot s_{cr,N} = 210 * 210 = 44100\ \text{mm}^2$

$A_{c,N} = (105 + 180 + 105) * (105 + 180 + 100) = 150150\ \text{mm}^2$

$A_{c,N} / A^0_{c,N} = 150150/44100 = 3.4$

$\Psi_{s,N} = 0.7 + 0.3(c/c_{cr,N})$

$\Psi_{s,N} = 0.7 + 0.3(100/105) = 0.986 \ < 1$ OK

$\Psi_{re,N} = 0.5 + h_{ef}/200 \leq 1$

$\Psi_{re,N} = 0.5 + 70/200 = 0.85 \ \leq 1$ OK

$\Psi_{ec,N} = \frac{1}{1+2(e_N/s_{cr,N})}$

$\Psi_{ec,N} = \frac{1}{1+2(86/210)} = 0.55 \leq$ OK

$\Psi_{M,N} = 1$

$N_{Rk,c} = N^0_{Rk,c} \, A_{c,N}/A^0_{c,N} \, \Psi_{s,N} \, \Psi_{re,N} \, \Psi_{ec,N} \, \Psi_{M,N}$

$N_{Rk,c} = 20.2 * 3.4 * 0.986 * 1 * 0.55 * 1 = 37.25 \text{ kN}$

$\gamma_{inst} = 1$ (from Table C2 of ETA-98/0001)

$\gamma_{Mc} = \gamma_c * \gamma_{inst} = 1.5 * 1 = 1.5$

$N_{Rd,c} = N_{Rk,c}/\gamma_{Mc} = 37.25 \ /1.5 = 24.83 \geq N^g_{Ed}$

$\beta \ = N^g_{Ed} \ /N_{Rd,c} = 22.8/24.83 = 0.918$ → 92% utilization. OK

EXAMPLE 3.12

Check the resistance to concrete cone failure for the following connection formed using HY 200 with Hit-V 8.8 grade rods of size M12 with 109 mm effective embedment (ETA-12/0084 [21]). The base material is of M25 grade concrete. Assume concrete to be cracked for design. [Given - tension force on anchor 2 & 4 is 18.9 kN. Anchor 1 & 3 are in compression as determined in Ex. 3.2].

Given:

$h_{ef} = 109 \text{ mm}$

$d = 12 \text{ mm}$

$f_{ck} = 20 \text{ N/mm}^2$ (cylinder strength corresponding to M25)

$k_1 = 7.7$ (for cracked concrete)

Solution:

$N^g_{Ed} = 2 * 18.9 = 37.8$ kN (only anchors 2 & 4 will act as group to resist concrete cone)

$$N^0_{Rk,c} = k_1\sqrt{f_{ck}}h^{1.5}_{ef} = 7.7 * \sqrt{20}(109)^{1.5} = 39187 \text{ N} = 39.2 \text{ kN}$$

$s_{cr,N} = 3h_{ef}$

$s_{cr,N} = 3 * 109 = 327$ mm

$c_{cr,N} = 0.5s_{cr,N} = 0.5 * 327 = 163.5$ mm

$A^0_{c,N} = s_{cr,N} \cdot s_{cr,N} = 327 * 327 = 106929 \text{ mm}^2$

$A_{c,N} = (163.5 + 200 + 163.5) * (163.5 + 163.5) = 172329 \text{ mm}^2$

$A_{c,N}/A^0_{c,N} = 172329/106929 = 1.61$

$\Psi_{s,N} = 0.7 + 0.3(c/c_{cr,N}) = 1$ (due to infinite edge assumption)

$\Psi_{re,N} = 0.5 + h_{ef}/200 \leq 1$

$\Psi_{re,N} = 1$ (as $h_{ef} > 100$ mm)

$\Psi_{ec,N} = 1$ (as there is no eccentricity of load)

$\Psi_{M,N} = 1$

$$N_{Rk,c} = N^0_{Rk,c} \, A_{c,N}/A^0_{c,N} \, \Psi_{s,N} \, \Psi_{re,N} \, \Psi_{ec,N} \, \Psi_{M,N}$$

$N_{Rk,c} = 39.2 * 1.61 * 1 * 1 * 1 * 1 = 63.112$ kN

$\gamma_{inst} = 1$ (from Table C1 of ETA-12/0084)

$\gamma_{Mc} = \gamma_c * \gamma_{inst} = 1.5 * 1 = 1.5$

$N_{Rd,c} = N_{Rk,c}/\gamma_{Mc} = 63.112/1.5 = 42.07 \geq N^g_{Ed}$

$\beta = N^g_{Ed}/N_{Rd,c} = 37.8/42.07 = 0.898$ → 90% utilization. OK

(E) Resistance to concrete splitting

The concrete can split either during application of installation torque on anchor or during application of load on channel. The concrete splitting during installation may be avoided by providing spacing and edge distance greater than the minimum value as well as maintaining minimum member thickness and reinforcement as recommended in product's Technical Assessment report.

The concrete splitting during application of load may be avoided if edge distance greater than 1.2 times the characteristic critical spacing corresponding to splitting failure ($s_{cr,sp}$) is provided in all directions and the minimum member depth corresponding to this edge distance is provided as recommended in product's Technical Assessment report. If it is not possible to do so then separate verification needs to be carried to check resistance to splitting.

The design concrete splitting strength of the anchor in tension ($N_{Rd,sp}$) is determined by dividing characteristic concrete splitting resistance, $N_{Rk,\mathrm{sp}}$, by recommended partial safety factor, γ_{Msp}, as shown in Eq. 3.6a. The check for concrete splitting failure is carried out for the most unfavourably loaded anchor. The load on the anchor along with edge and spacing of anchor should be taken into consideration for determining the most unfavourably loaded anchor.

$N_{Rd,\mathrm{sp}} = N_{Rk,sp}/\gamma_{Msp} \geq N_{Ed}$ [for single anchor] (Eq. 3.6a)

$N_{Rd,\mathrm{sp}} = N_{Rk,sp}/\gamma_{Msp} \geq N_{Ed}^{g}$ [for anchor group] (Eq. 3.6b)

The characteristic concrete splitting strength ($N_{Rk,sp}$) of an anchor or an anchor group should be determined according to Eq. 3.6c. In this equation, the factors $A_{c,N}/A_{c,N}^{0}$, $\Psi_{s,N}$, $\Psi_{re,N}$ and $\Psi_{ec,N}$, is calculated as per Eq. 3.5 but using critical edge and spacing values corresponding to splitting failure for a given minimum member thickness. The factor $N_{Rk,sp}^{0}$ should be taken from ETA. The factor $\Psi_{h,sp}$ accounts for the influence of actual member thickness on concrete splitting resistance and is calculated according to Eq. 3.6d. In this equation, h_{min} is permitted minimum member thickness and c_1 is the edge distance.

$N_{Rk,sp} = N_{Rk,sp}^{0} \, A_{c,N}/A_{c,N}^{0} \, \Psi_{s,N} \, \Psi_{re,N} \, \Psi_{ec,N} \, \Psi_{h,sp}$ (Eq. 3.6c)

where,

$\Psi_{h,sp} = \left(\frac{h}{h_{min}}\right)^{2/3} \leq \max[1; [\frac{h_{ef}+1.5c_1}{h_{min}}]^{2/3}] \leq 2$ (Eq. 3.6d)

This verification may be skipped if cracked concrete is assumed for calculation of concrete cone and pull-out/bond strength and the reinforcement provided to resist the splitting forces limits the crack width to 0.3 mm. The required reinforcement ($\sum A_{s,re}$) to resist splitting may be determined as per Eq. 3.7. This reinforcement should be placed symmetrically and close to each anchor leg of the channel.

$$\sum A_{s,re} = k_4 \frac{\sum N_{Ed}}{f_{yk,re}/\gamma_{Ms,re}} \quad \text{(Eq. 3.7)}$$

where,

N_{Ed}^{g}- is the sum of design tensile force on anchors in tension under design load.

$f_{yk,re}$- is the nominal yield strength of the reinforcement; but it should be ≤ 600 N/mm^2.

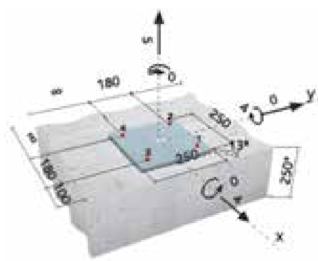
k_4 - is 2 for deformation-controlled expansion anchors, 1.5 for torque-controlled & bonded expansion anchors, 1 for undercut anchors & concrete screws, 0.5 for bonded anchors.

$\gamma_{Ms,re}$- is the partial safety factor for supplementary reinforcement.

Example 3.13 illustrates typical calculations necessary for checking resistance to concrete splitting failure in tension.

EXAMPLE 3.13

Check resistance to concrete splitting for the following connection formed using HST3 M12 with 80 mm nominal embedment (ETA-98/0001 [20]). The base material is of M25 grade concrete. Assume concrete to be cracked for design. [Given - tension force on anchor 1 & 3 is 0.283 kN and on anchor 2 & 4 is 11.128 kN, obtained after resolving the load on the connection].



Given:

$h_{ef} = 70$ mm (from Table B9 of ETA-98/0001)

$d = 12$ mm

$f_{ck} = 20$ N/mm^2 (cylindrical strength corresponding to M25 grade)

$k_1 = 7.7$ (for cracked concrete)

$$c = 100 \text{ mm}$$

$$h = 250\ mm$$

Solution:

$N_{Ed}^{g} = 2 * 11.128 + 2 * 0.283 = 22.822 \text{ kN}$ (In this case all 4 anchors will act as group to resist concrete splitting failure as all of them are in tension)

$$N_{Rk,c}^{0} = k_1\sqrt{f_{ck}}h_{ef}^{1.5} = 7.7 * \sqrt{20}(70)^{1.5} = 20167 \text{ N} = 20.2\ \text{kN}$$

$s_{cr,sp} = 3h_{ef}$ (from Table C2 of ETA-98/0001)

$s_{cr,sp} = 3 * 70 = 210 \text{ mm}$

$c_{cr,sp} = 0.5s_{cr,N} = 0.5 * 210 = 105 \text{ mm}$

$A_{c,N}^{0} = s_{cr,sp} \cdot s_{cr,sp} = 210 * 210 = 44100\ \text{mm}^2$

$A_{c,N} = (105 + 180 + 105) * (105 + 180 + 100) = 150150 \text{ mm}^2$

$A_{c,N}/A_{c,N}^{0} = 150150/44100 = 3.4$

$\Psi_{s,N} = 0.7 + 0.3(c/c_{cr,sp})$

$\Psi_{s,N} = 0.7 + 0.3(100/105) = 0.986 < 1$ OK

$\Psi_{re,N} = 0.5 + h_{ef}/200 \leq 1$

$\Psi_{re,N} = 0.5 + 70/200 = 0.85 \leq 1$ OK

$\Psi_{ec,N} = \frac{1}{1+2(e_N/s_{cr,N})}$

$\Psi_{ec,N} = \frac{1}{1+2(86/210)} = 0.55 \leq$ OK

$h_{min} = 140 \text{ mm}$ (from Table B11 of ETA-98/0001)

$$\Psi_{h,sp} = \left(\frac{h}{h_{min}}\right)^{2/3} \leq \max[1; [\frac{h_{ef} + 1.5c_1}{h_{min}}]^{2/3}] \leq 2$$

$$\Psi_{h,sp} = \left(\frac{250}{140}\right)^{2/3} = 1.471$$

$$\max\left[1; \left[\frac{h_{ef} + 1.5c_1}{h_{min}}\right]^{\frac{2}{3}}\right] = 1.35$$

➔ Use $\Psi_{h,sp} = 1.35$

$$N_{Rk,sp} = N^0_{Rk,sp} \; A_{c,N}/A^0_{c,N} \; \Psi_{s,N} \; \Psi_{re,N} \; \Psi_{ec,N} \; \Psi_{h,sp}$$

$$N_{Rk,sp} = 20.2 * 3.4 * 0.986 * 1 * 0.55 * 1.35 = 50.28 \text{ kN}$$

$\gamma_{inst} = 1$ (from Table C2 of ETA-98/0001)

$\gamma_{Mc} = \gamma_c * \gamma_{inst} = 1.5 * 1 = 1.5$

$N_{Rd,sp} = N_{Rk,sp} / \gamma_{Mc} = 50.28 / 1.5 = 33.52 \geq N^g_{Ed}$

$\beta = N^g_{Ed} / N_{Rd,sp} = 22.8/33.52 = 0.68$ → 68% utilization. OK

(F) Resistance to concrete blow-out

For undercut mechanical anchors acting as headed anchors and located at edge distance greater than $0.5h_{ef}$, an additional check for concrete blow-out strength is required as per EN 1992-4 [6] but is not covered in this book.

3.8.2 DESIGN CHECKS FOR SHEAR LOAD

(A) Resistance to steel failure

The check for resistance to steel failure is dependent on whether or not the shear is acting with a lever arm. Therefore, the verification is broadly categorized into – Resistance to steel failure due to shear force “without lever arm” and Resistance to steel failure due to shear force “with lever arm”.

(i) Resistance to steel failure due to shear force "without lever arm"

As per EN 1992-4 [6], the design steel strength in shear ($V_{Rd,s}$) without lever arm is determined by dividing characteristic steel resistance ($V_{Rk,s}$) by recommended partial material safety factor (γ_{Ms}). The design steel strength should be greater

than the design load (V_{Ed}). The design shear load on most stressed anchor in an anchor group is denoted as V_{Ed}^{h} and on entire anchor group is denoted as V_{Ed}^{g}. The check for steel failure is carried out for the most unfavourably loaded anchor in an anchor group.

$V_{Rd,s} = V_{Rk,s}/\gamma_{Ms} \geq V_{Ed}$ [for single anchor] (Eq. 3.8a)

$V_{Rd,s} = V_{Rk,s}/\gamma_{Ms} \geq V_{Ed}^{h}$ [for anchor group] (Eq. 3.8b)

In general, the characteristic steel resistance value should be taken from the product's Technical Assessment report like ETA. However, the characteristic resistance of anchors accounting for ductility of anchor in an anchor group and including grout layer (with thickness $t_{grout} < d/2$) should be calculated according to Eq. 3.8c. The factor k_7 in Eq. 3.8c should be taken as 1 for single anchors and ETA should be referred for anchors in a group[5].

$V_{Rk,s} = k_7 V_{Rk,s}^{0}$ [for anchor group] (Eq. 3.8c)

Example 3.14 illustrates typical calculations necessary for checking resistance to steel failure in shear (without lever arm).

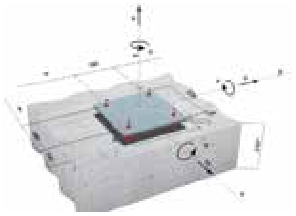
EXAMPLE 3.14

Determine resistance to steel failure in shear for the following connection formed using HST3 M12 with 60 mm nominal embedment (ETA-98/0001 [20]).

---

[5] For anchors with steel rupture $A_5 \leq 8\%$ elongation $k_7 = 0.8$ may be used. For anchors with ductile steel in an anchor group $k_7 = 1$ may be used.

Given:



Shear load on most stressed anchor
$V^h_{Ed} = 5 \text{ kN}$ (from Ex 3.4)

$h_{ef} = 50 \text{ mm}$ (from Table B9 of ETA-98/0001)

Solution:

$k_7 = 1$ (from Table C4 of ETA-98/0001)

$V_{Rk,s} = 34 \text{ kN}$ (from Table C4 of ETA-98/0001)

$\gamma_{Ms} = 1.25$

$V_{Rd,s} = V_{Rk,s}/\gamma_{Ms} = 34\ /1.25 = 27.2 \text{ kN} \geq V^h{}_{Ed}$

$\beta = V^h_{Ed}\ /V_{Rd,s} = 5/27.2 = 0.1834$ → 18% utilization. OK

(ii) Resistance to steel failure due to shear force "with lever arm"

The design steel strength in shear ($V_{Rd,s}$) with lever arm is determined by dividing characteristic steel resistance ($V_{Rk,s,M}$) by recommended partial material safety factor (γ_{Ms}). The check for steel failure is carried out for the most unfavourably loaded anchor in an anchor group.

$V_{Rd,s} = V_{RkM,,s}/\gamma_{Ms} \geq V_{Ed}$ [for single anchor] (Eq. 3.9a)

$V_{Rd,s} = V_{Rk,M,s}/\gamma_{Ms} \geq V^h_{Ed}$ [for anchor group] (Eq. 3.9b)

Where

$V_{Rk,s,M} = \alpha_M M_{Rk,s}/l_a$ (Eq. 3.9c)

In Eq. 3.9c, bending resistance of channel bolt ($M_{Rk,s}$) is equal to $M^0{}_{Rk,s}(1 - N_{Ed}/N_{Rd,s})$. The value of $M^0{}_{Rk,s}$ is taken from the product's Technical Assessment report like ETA. The factor α_M is taken as 1 for no restraint and 2 for full restraint of the fixture as discussed in Section 0. The length l_a is calculated according to Eq. 3.1b. The value of N_{Ed} can only be tension for calculation of bending resistance.

Example 3.15 illustrates typical calculations necessary for checking resistance to steel failure in shear (with lever arm).

EXAMPLE 3.15

Check resistance to steel failure in shear for the following connection formed using HST3 M12 with 60 mm nominal embedment (ETA-98/0001 [20]).

Given:

$$V_{Ed}^{h} = 2.5 \text{ kN}$$

Solution:

$h_{ef} = 50$ mm (from Table B9 of ETA-98/0001)

$\alpha_M = 2.0$ (full restraint case)

$$e_1 = 30 + \frac{13}{2} = 36.5 \text{ mm}$$

$l_a = a_3 + e_1 = 36.5$ mm

$M_{Rk,s} = 105$ N – m (from Table C4 of ETA-98/0001)

$V_{Rk,s,M} = \alpha_M M_{Rk,s} / l_a$

$V_{Rk,s,M} = 2 * 105 * 1000/36.5 = 5750\ N = 5.75$ kN

$V_{Rd,s} = V_{Rk,M,s}/\gamma_{Ms} = 5.75\ /1.25 = 4.6 \text{ kN} \geq V^h{}_{Ed}$

$\beta\ = V_{Ed}^{h}\ /V_{Rd,s}\ = 2.5/4.6 = 0.543$ → 54% utilization. OK

(B) Resistance to concrete pry-out

As per EN 1992-4 [6], the design pry-out strength of the anchor in shear ($V_{Rd,cp}$) is determined by dividing characteristic pry-out resistance, $V_{Rk,\text{cp}}$, by recommended partial safety factor, γ_{Mc}, as shown in Eq. 3.10. This check is carried out for the anchor group.

$V_{Rd,\text{cp}} = V_{Rk,cp}/\gamma_{Mc} \geq V_{Ed}$[for single anchor] (Eq. 3.10a)

$V_{Rd,\text{cp}} = V_{Rk,cp}\ /\gamma_{Mc} \geq V_{Ed}^{g}$[for anchor group] (Eq. 3.10b)

where,

$V_{Rk,cp} = k_8 \cdot N_{Rk,c}$ [For mechanical anchors without supplementary reinforcement]
(Eq. 3.10c)

$V_{Rk,cp} = k_8 \cdot \min(N_{Rk,c}, N_{Rk,p})$ [For bonded anchors without supplementary reinforcement] (Eq. 3.10d)

In the formula for determining the characteristic pry-out resistance($V_{Rk,\mathrm{cp}}$), the factor k_8 is taken from the product's Technical Assessment report, and the $N_{Rk,c}$ and $N_{Rk,p}$ value is determined according to Section 0(C) and 0(D), respectively for single anchor or all anchors in a group loaded in shear.

For anchor groups in which the anchors are subjected to shear forces (or components thereof) in opposing directions, the check for concrete pry-out is carried out for the most unfavourably loaded anchor. One example of such situation is connection subjected to torsion moment. In such cases, virtual edge (with $c = 0.5s$) in the direction of neighbouring anchor is assumed for calculating the areas $A_{c,N}$ and $A_{p,N}$(see Fig.3.16).

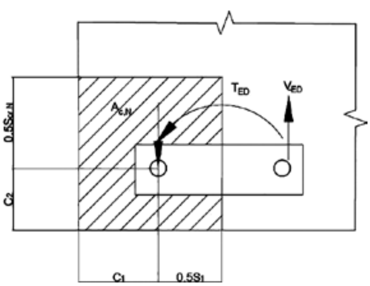


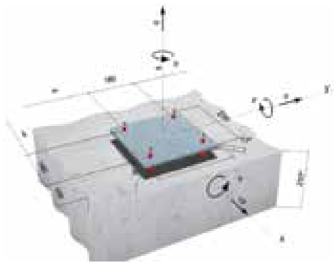
Fig. 3.16 Example of virtual edge for calculation of $\boldsymbol{A_{c,N}}$ for pry-out (resketched based on illustration in [6])

Examples 3.16 and 3.17 illustrate typical calculations necessary for checking resistance to concrete pry-out failure in shear.

EXAMPLE 3.16

Check resistance to concrete pry-out failure for the following connection formed using HST3 M12 with 60 mm nominal embedment (ETA-98/0001 [20]). The base material is of M25 concrete grade. Assume concrete to be cracked for design.

Given:



$V_{Ed}^{g} = 10\ \mathrm{kN}$

$d = 12\ \mathrm{mm}$

$f_{ck} = 20\ \mathrm{N/mm^2}$ (cylinder strength corresponding to M25)

Solution:

$h_{ef} = 50\ \mathrm{mm}$ (from Table B9 of ETA-98/0001)

$k_3 = 7.7$ (for cracked concrete)

$N_{Rk,c}^{0} = k_1\sqrt{f_{ck}}h_{ef}^{1.5} = 7.7 * \sqrt{20}(50)^{1.5} = 12174\ \mathrm{N} = 12.2\ \mathrm{kN}$

$s_{cr,N} = 3h_{ef}$

$s_{cr,N} = 3 * 50 = 150\ \mathrm{mm}$

$c_{cr,N} = 0.5s_{cr,N} = 0.5 * 150 = 75\ \mathrm{mm}$

$A_{c,N}^{0} = s_{cr,N} \cdot s_{cr,N} = 150 * 150 = 22500\ \mathrm{mm^2}$

$A_{c,N} = 4 * 22500 = 90000\ \mathrm{mm^2}$

$A_{c,N}/A_{c,N}^{0} = 90000/22500 = 4$

$\Psi_{s,N} = 0.7 + 0.3(c/c_{cr,N}) = 1$ (due to infinite edge assumption)

$\Psi_{s,N} = 0.7 + 0.3(100/75) = 1.1 > 1$

Use $\Psi_{s,N} = 1$

$\Psi_{re,N} = 0.5 + h_{ef}/200 \leq 1$

$\Psi_{re,N} = 0.5 + 50/200 = 0.75$

$\Psi_{ec,N} = 1$ (as there is no eccentricity of load)

$\Psi_{M,N} = 1$

$$N_{Rk,c} = N^0_{Rk,c} \, {A_{c,N}}/{A^0_{c,N}} \, \Psi_{s,N}\Psi_{re,N}\Psi_{ec,N}\Psi_{M,N}$$

$N_{Rk,c} = 12.2 * 4 * 0.75 * 1 * 1 * 1 = 36.6 \text{ kN}$

$k_8 = 2.78$ (from Table C4 of ETA-98/0001)

$$V_{Rk,cp} = k_8 \cdot N_{Rk,c} = 2.78 * 36.6 = 1101.8 \text{ kN}$$

$\gamma_{inst} = 1$ (from Table C2 of ETA-98/0001)

$\gamma_{Mc} = \gamma_c * \gamma_{inst} = 1.5 * 1 = 1.5$

$V_{Rd,\text{cp}} = V_{Rk,cp}/\gamma_{Mc} \geq V^g_{Ed}$

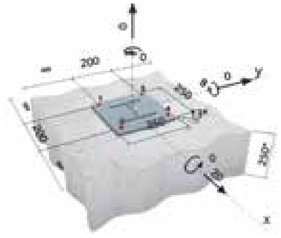
$V_{Rd,\text{cp}} = V_{Rk,cp}/\gamma_{Mc} = 1101.8 /1.5 = 67.8 \text{ kN} \geq V^g_{Ed}$

$\beta = V^g_{Ed}/V_{Rd,\text{cp}} = 10/67.8 = 0.147$ → 14% utilization. OK

EXAMPLE 3.17

Check resistance to concrete pry-out failure for the following connection formed using HY 200 with Hit-V 8.8 grade rods of size M12 with 109 mm effective embedment (ETA-12/0084 [21]). The base material is of M25 grade concrete. Assume concrete to be cracked for design. [Given - tension force on anchor 2 & 4 is 18.9 kN. Anchor 1 & 3 are in compression as determined in Ex. 3.2]

Given:



$h_{ef} = 109 \text{ mm}$

$d = 12 \text{ mm}$

$f_{ck} = 20 \text{ N/mm}^2$ (cylinder strength corresponding to M25)

$k_1 = 7.7$ (for cracked concrete)

$V^{g}_{Ed} = 20$ kN

Solution:

$N_{Rk,c} = 63.1$ kN (from Ex. 3.12)

$N_{Rk,p} = 57$ kN (from Ex. 3.10)

$\tau_{Rk,cr} = 8.5$ N/mm² (from Table C1 of ETA-12/0084)

$\Psi^{0}_{sus} = 0.6$ (default, as no value is provided in ETA)

$\alpha_{sus} = 0$ (as there is no sustained load)

$\Psi_{sus} = 1$ (For $\alpha_{sus} \leq \Psi^{0}{}_{sus}$)

$N^{0}_{Rk,p} = \Psi_{sus} \tau_{Rk,cr} \pi d h_{ef}$

$N^{0}_{Rk,p} = 1 * (8.5/1000) * 3.14 * 12 * 109 = 34.9$ kN

$s_{cr,Np} = 7.3d(\Psi_{sus}\tau_{Rk,ucr})^{0.5} \leq 3h_{ef}$

$s_{cr,Np} = 7.3 * 12 * (1 * 18)^{0.5} = 371.6 \text{ mm} > 3h_{ef} = 327$ Not OK

Use $s_{cr,Np} = 327$ mm

$c_{cr,Np} = 0.5 s_{cr,Np} = 163.5$ mm

$A^{0}{}_{p,N} = s_{cr,Np} \cdot s_{cr,Np} = 327 * 327 = 106929 \text{ mm}^2$

$A_{p,N} = (163.5 + 200 + 163.5) * (163.5 + 200 + 163.5) = 277729 \text{ mm}^2$

$A_{p,N} / A^{0}_{p,N} = 277729/106929 = 2.59$

$\tau_{Rk,c} = (k_3/\pi d)\sqrt{h_{ef} f_{ck}} = (7.7/(3.14 * 12))\sqrt{109 * 20} = 9.54 \text{ N/mm}^2$

$\Psi_{g,Np} = \sqrt{n} - (\sqrt{n} - 1) \cdot \left[\frac{\tau_{Rk}}{\tau_{Rk,c}}\right]^{1.5}$

$\Psi^{0}_{g,Np} = \sqrt{4} - (\sqrt{4} - 1) \cdot \left[\frac{8.5}{9.54}\right]^{1.5} = 1.159 \geq 1$ OK

$\Psi_{g,Np} = \Psi^0_{g,Np} - (s/s_{cr,Np})^{0.5} * (\Psi^0_{g,Np} - 1) = 1.159 - (200/327)^{0.5} * 0.159 =$ $1.035 \geq 1$ OK

$\Psi_{s,Np} = 0.7 + 0.3(c/c_{cr,Np}) = 1$ (due to assumption of infinite edge)

$\Psi_{re,N} = 0.5 + h_{ef}/200 = 0.5 + 109/200 = 1.045 > 1$

Use $\Psi_{re,N} = 1$

$\Psi_{ec,Np} = 1$ (as there is no eccentricity of load)

$N_{Rk,p} = N^0_{Rk,p} \, A_{p,N}/A^0_{p,N} \, \Psi_{g,Np} \, \Psi_{s,Np} \, \Psi_{re,Np} \, \Psi_{ec,Np}$

$N_{Rk,p} = 34.9 * 2.59 * 1.035 * 1 * 1 * 1 \approx 93.6 \text{ kN}$

$\gamma_{Mp} = 1.5$ (from Table C1 of ETA-12/0084)

$N_{Rd,\text{p}} = N_{Rk,p}/\gamma_{Mp} = 93.6 /1.5 = 62.4 \text{ kN}$

$k_8 = 2$

$V_{Rk,cp} = k_8 \cdot \min(N_{Rk,c}, N_{Rk,p}) = 2 * 62.4 = 125 \text{ kN}$

$\beta = V^g_{Ed} / V_{Rd,\text{cp}} = 20/125 = 0.16$ → 16% utilization. OK

(C) Resistance to concrete edge breakout

The design concrete edge breakout strength of the anchor in shear ($V_{Rd,c}$) is determined by dividing characteristic concrete edge breakout resistance, $V_{Rk,\text{c}}$, by recommended partial safety factor, γ_{Mc}, as shown in Eq. 3.11a and 3.11b. The check for concrete cone failure should be carried out for the anchor group. Only anchors located close to edge (refer Section 0) are checked for this failure. Minimum spacing of $4d_{nom}$ should be maintained between the anchors of a group. This check should be carried out for each edge in case of multiple edges.

$V_{Rd,\text{c}} = V_{Rk,c}/\gamma_{Mc} \geq V_{Ed}$ [for single anchor] (Eq. 3.11a)

$V_{Rd,\text{c}} = V_{Rk,c}/\gamma_{Mc} \geq V^g_{Ed}$ [for anchor group] (Eq. 3.11b)

The characteristic concrete breakout strength ($V_{Rk,c}$) of anchor or anchor group loaded in shear towards edge is calculated according to Eq. 3.11c. In Eq. 3.11c,

$V^0_{Rk,c}$ is basic characteristic concrete breakout strength of single anchor loaded perpendicular to edge, whose strength is not influenced by adjacent anchors or edges of concrete member. The factor k_9 used for determining $V^0{}_{Rk,c}$ should be taken 2.4 for uncracked concrete and 1.7 for cracked concrete. The length l_f is taken from ETA. The value of l_f is limited to $12d_{nom}$ for anchors with $d_{nom} \leq 24$ mm and to $\max(8d_{nom}, 300\text{ mm})$ in case of anchors with $d_{nom} > 24$ mm. $f_{ck}$is nominal characteristic compressive strength measured on cylinder of size 150 mm diameter by 300 mm height.

The area ratio $A_{c,V}/A^0{}_{c,V}$ in accounts for the effect of spacing and edge distance. In this ratio, $A^0_{c,V}$ is calculated according to Eq. 3.11e and $A_{c,V}$ is the actual influence area (see Fig3.17) limited by overlapping areas of adjacent fasteners ($s \leq 3c_1$) as well as by edges ($c_2 \leq 1.5c_1$) and thickness of the concrete member ($h \leq 1.5c_1$).

The factor $\Psi_{s,V}$ accounts for the influence of edge on the characteristic concrete edge breakout strength of anchor subjected to shear and is calculated according to Eq. 3.11f. In case of multiple edge, the smallest edge distance is used to calculate the factor $\Psi_{s,V}$.

The factor $\Psi_{h,V}$accounts for the effect of concrete member thickness and is calculated according to Eq. 3.11g. The factor $\Psi_{ec,V}$ accounts for effect of eccentricity of shear load (e_V) w.r.t to the centre of gravity of fasteners loaded in shear, which arises due to different shear loads acting on individual anchors in an anchor group as illustrated in.Fig.3.18. It is determined according to Eq. 3.11h. The factor $\Psi_{\alpha V}$ accounts for design shear load acting (V_{Ed} or V^g_{Ed}) at an angle to the edge under consideration. It is calculated according to Eq. 3.11i,$\alpha_V$is the angle between design shear load and a line perpendicular to the edge under consideration.

The factor $\Psi_{re,N}$ accounts for the effect of edge reinforcement on the strength. For uncracked concrete and for cracked concrete application without edge reinforcement/stirrups, the factor $\Psi_{re,N}$ is taken as 1. For cracked concrete application with edge reinforcement and closely spaced stirrups/wire mesh ($a \leq \min(100mm,\ 2c_1)$), the factor $\Psi_{re,N}$ is taken as 1.4, provided the effective embedment depth h_{ef}.is at least 2.5 times the concrete cover of edge reinforcement.

$$V_{Rk,c} = V_{Rk,c}^{0} \, {}^{A_{c,V}}\!\Big/_{A_{c,V}^{0}} \, \Psi_{s,V} \Psi_{h,V} \Psi_{ec,V} \Psi_{\alpha,V} \Psi_{re,V} \qquad \text{(Eq. 3.11c)}$$

where

$V_{Rk,c}^{0} = k_9 d_{nom}^{\alpha} l_f^{\beta} \sqrt{f_{ck}} \mathrm{c}_1^{1.5}$ where $\alpha = 0.1(l_f/c_1)^{0.5}$ and $\beta = 0.1(d_{nom}/c_1)^{0.2}$

(Eq. 3.11d)

$$A_{c,V}^{0} = 4.5c_1^2 \qquad \text{(Eq. 3.11e)}$$

$$\Psi_{s,V} = 0.7 + 0.3(c_2/1.5c_1) \leq 1 \qquad \text{(Eq. 3.11f)}$$

$$\Psi_{h,V} = (1.5c_1/h)^{0.5} \geq 1 \qquad \text{(Eq. 3.11g)}$$

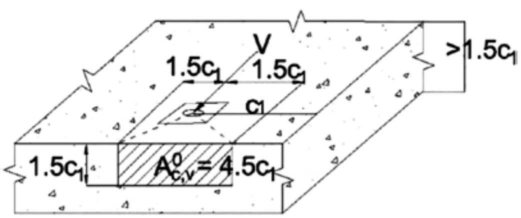
$$\Psi_{ec,V} = \frac{1}{1+2e_V/3c_1} \leq 1 \qquad \text{(Eq. 3.11h)}$$

$\Psi_{\alpha,V} = \sqrt{\frac{1}{(\cos\alpha_V)^2 + (0.5\sin\alpha_V)^2}} \geq 1$, where $0^{\circ} \leq \alpha_V \leq 90^{\circ}$ (Eq. 3.11i)

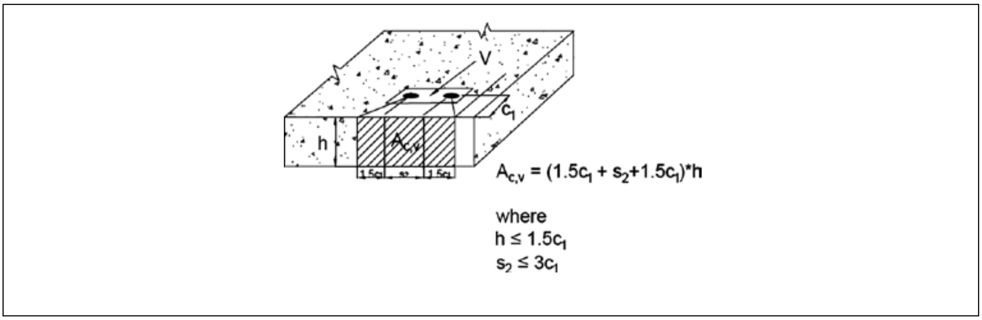
$\Psi_{re,V} = 1 \; or \; 1.4$, depending on state of concrete and edge reinforcement layout

(Eq. 3.11j)

This formula doesn't yield precise results for anchor in narrow and thin members. Also, the same is applicable for group of two anchors under influence of torsion. EN 1992-4 [6] recommends modification to this formula. These modifications are not discussed in this book for the sake of simplicity.



(a)Single anchor subjected to shear



(b) Anchor group subjected to shear

Fig. 3.17 Example of calculation of ideal and actual edge breakout area in case of shear load towards edge (resketched based on illustration in [6])

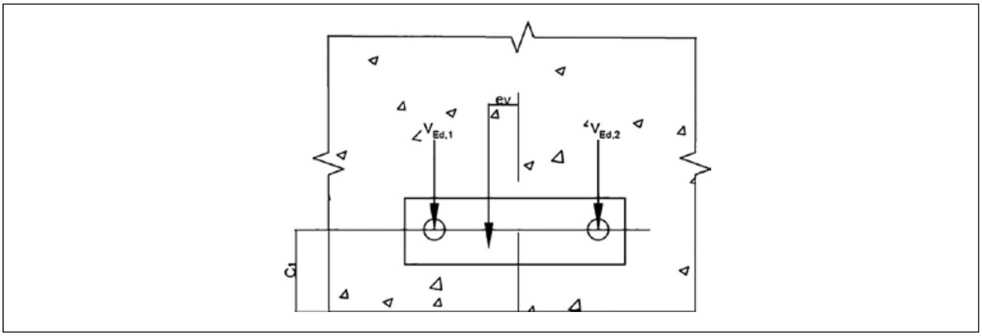


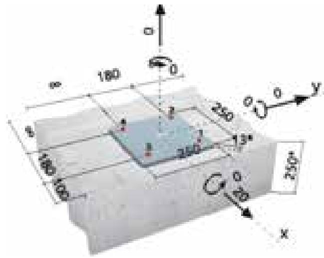
Fig. 3.18. Illustration of different shear load on individual anchors resulting in eccentricity of resultant load (resketched based on illustration in [6])

Examples 3.18 and 3.19 illustrate typical calculations necessary for checking resistance to concrete edge breakout failure in shear.

EXAMPLE 3.18

Check resistance to concrete edge breakout failure for the following connection formed using HST3 M12 with 60 mm nominal embedment (ETA-98/0001 [20]). The base material is of M25 concrete grade. Assume concrete to be cracked for design.

Given:



$V^g_{Ed} = 20$ kN

$d_{nom} = 12$ mm

$f_{ck} = 20$ N/mm^2 (cylindrical strength corresponding to M25 grade)

$c_1 = 100$ mm

Solution:

$h_{ef} = 50$ mm (from Table B9 of ETA-98/0001)

$k_9 = 1.7$ (for cracked concrete)

$l_f = 50$ mm (from Table C4 of ETA-98/0001)

$\alpha = 0.1(l_f/c_1)^{0.5} = 0.1(50/100)^{0.5} = 0.07$

$\beta = 0.1(d_{nom}/c_1)^{0.2} = 0.1(12/100)^{0.2} = 0.065$

$V^0_{Rk,c} = k_9\, d_{nom}^{\alpha}\, l_f^{\beta} \sqrt{f_{ck}}\, c_1^{1.5}$

$$V^0_{Rk,c} = 1.7 * (12)^{0.07} * (50)^{0.065} * \sqrt{20} * (100)^{1.5} = 11666\,N = 11.7\text{kN}$$

$A^0_{c,V} = 4.5c_1^2$

$A^0_{c,V} = 4.5 * (100)^2 = 45000$ mm^2

$A_{c,V} = (1.5 * 100) * (150 + 180 + 150) = 72000$ mm^2

$A_{c,V}/A^0_{c,V} = 72000/45000 = 1.6$

$\Psi_{s,V} = 1$

$\Psi_{h,V} = (1.5c_1/h)^{0.5} \geq 1$

$\Psi_{h,V} = (1.5 * 100/250)^{0.5} = 0.774$

Use $\Psi_{h,V} = 1$

$\Psi_{ec,V} = 1$ (no eccentricity of load)

$\Psi_{\alpha,V} = 1$ (as shear is perpendicular to edge)

$\Psi_{re,V} = 1$ (cracked concrete but without edge reinforcement)

$$V_{Rk,c} = V^0_{Rk,c} \, A_{c,V} / A^0_{c,V} \, \Psi_{s,V} \, \Psi_{h,V} \, \Psi_{ec,V} \, \Psi_{\alpha,V} \, \Psi_{re,V}$$

$V_{Rk,c} = 11.7 * 1.6 * 1 * 1 * 1 * 1 * 1 = 18.72 \text{ kN}$

$\gamma_{Mc} = 1.5$

$V_{Rd,c} = V_{Rk,c} / \gamma_{Mc} = 18.72 / 1.5 = 12.48 \text{ kN} < V^g_{Ed}$

$\beta = V^g_{Ed} / V_{Rd,cp} = 20/12.48 = 1.60$ → 160% utilization. Not OK

EXAMPLE 3.19

Check resistance to concrete edge breakout failure for the following connection formed using HST3 M12 with 60 mm nominal embedment (ETA-98/0001 [20]). The base material is of M25 grade concrete. Assume concrete to be cracked for design.

Given:

$V^g_{Ed} = 20 \text{ kN}$

$d_{nom} = 12 \text{ mm}$

$f_{ck} = 20 \text{ N/mm}^2$ (cylinder strength corresponding to M25)

$c_1 = 280 \text{ mm}$ (due to slotted holes in front)

Solution:

$h_{ef} = 50 \text{ mm}$ (from Table B9 of ETA-98/0001)

$k_9 = 1.7$ (for cracked concrete)

$l_f = 50 \text{ mm}$ (from Table C4 of ETA-98/0001)

$\alpha = 0.1(l_f/c_1)^{0.5} = 0.1(50/280)^{0.5} = 0.042$

$\beta = 0.1(d_{nom}/c_1)^{0.2} = 0.1(12/280)^{0.2} = 0.053$

$$V^0_{Rk,c} = k_9 \, d^{\alpha}_{nom} \, l^{\beta}_f \sqrt{f_{ck}} \, c_1^{1.5}$$

$V^0_{Rk,c} = 1.7 * (12)^{0.042} * (50)^{0.053} * \sqrt{20} * (280)^{1.5} = 48648 \text{ N} = 48.7 \text{ kN}$

$A^0_{c,V} = 4.5 c_1^2$

$A^0_{c,V} = 4.5 * (280)^2 = 352800 \text{ mm}^2$

$A_{c,V} = (250) * (1.5 * 280 + 180 + 1.5 * 280) = 255000 \text{ mm}^2$

$A_{c,V} / A^0_{c,V} = 255000/352800 = 0.72$

$\Psi_{s,V} = 1$

$\Psi_{h,V} = (1.5 c_1/h)^{0.5} \geq 1$

$\Psi_{h,V} = (1.5 * 280/250)^{0.5} = 1.29$

$\Psi_{ec,V} = 1$ (no eccentricity of load)

$\Psi_{\alpha,V} = 1$ (as shear is perpendicular to edge)

$\Psi_{re,V} = 1$ (cracked concrete but without edge reinforcement)

$V_{Rk,c} = V^0_{Rk,c} \, A_{c,V} / A^0_{c,V} \, \Psi_{s,V} \, \Psi_{h,V} \, \Psi_{ec,V} \, \Psi_{\alpha,V} \, \Psi_{re,V}$

$V_{Rk,c} = 48.7 * 0.72 * 1 * 1.29 * 1 * 1 * 1 = 45.23 \text{ kN}$

$\gamma_{Mc} = 1.5$

$V_{Rd,c} = V_{Rk,c} / \gamma_{Mc} = 45.23 / 1.5 = 30.2 \text{ kN} > V^g_{Ed}$

Utilization % $= 20/30.2 = 0.662 \approx 66\%$ OK

$\beta = V^g_{Ed} / V_{Rd,cp} = 20/30.2 = 0.662$ → 66% utilization. OK

This example illustrates one way of overcoming design challenge (see previous example). In this case, slotted hole was introduced for anchors closer to edge as a solution.

3.8.3. DESIGN CHECKS FOR COMBINED TENSION AND SHEAR

The design check is based on the type of failure mode and needs to be carried out as follows:

Check for steel failure of anchor:

$$\left[{}^{N_{Ed}}/_{N_{Rd,s}}\right]^2 + \left[{}^{V_{Ed}}/_{V_{Rd,s}}\right]^2 \leq 1 \qquad \text{(Eq. 3.12)}$$

Note – If the design tension and shear loads are different for the individual fasteners of the anchor group then the interaction shall be checked for all fasteners. This verification is not required for shear loads with lever arm.

Check for all other failure modes of anchor:

$$\left[{}^{N_{Ed}}/_{N_{Rd,i}}\right]^{1.5} + \left[{}^{V_{Ed}}/_{V_{Rd,i}}\right]^{1.5} \leq 1 \qquad \text{(Eq. 3.13a)}$$

(or)

$$ {}^{N_{Ed}}/_{N_{Rd}} + {}^{V_{Ed}}/_{V_{Rd}} \leq 1.2 \qquad \text{(Eq. 3.13b)}$$

The largest value of tension ratio and shear ratio for different failure modes should be used in the above equation. In Eq. 3.13 and 3.13b, the tension and shear ratios should be less than or equal to 1, i.e. ${}^{N^a_{Ed}}/_{N_{Rd}} \leq 1$and ${}^{V^a_{Ed}}/_{V_{Rd}} \leq 1$.

Example 3.20 illustrates typical calculations necessary for checking combined tension and shear.

EXAMPLE 3.20

Check the combined tension and shear for the following data:

Tension			**Shear**		
Failure type	Load N_{Ed} [kN]	Strength $N_{Rd,i}$ [kN]	Failure type	Load V_{Ed} [kN]	Strength $V_{Rd,i}$ [kN]
Steel strength	11.128	32.214	Steel strength (without lever arm)	1	28.320
Pull-out strength	11.128	13.333	Concrete pry-out strength	4	125.442
Concrete breakout strength	22.823	24.867	Concrete edge breakout strength	4	13.183
Splitting strength	22.823	33.332			

Solution

Tension

Failure type	Load N_{Ed} [kN]	Strength $N_{Rd,i}$ [kN]	Utilization %
Steel strength	11.128	32.214	35
Pull-out strength	11.128	13.333	84
Concrete breakout strength	22.823	24.867	92
Splitting strength	22.823	33.332	69

Shear

Failure type	Load N_{Ed} [kN]	Strength $N_{Rd,i}$ [kN]	Utilization %
Steel strength (without lever arm)	1	28.320	4
Concrete pry-out strength	4	125.442	4
Concrete edge breakout strength	4	13.183	31

Check for steel failure of anchor:

$$\left[{}^{N_{Ed}}/_{N_{Rd,s}}\right]^2 + \left[{}^{V_{Ed}}/_{V_{Rd,s}}\right]^2 = [0.35]^2 + [0.04]^2 = 0.124 \leq 1 \quad \text{OK}$$

Check for all other failure modes of anchor:

$$ {}^{N_{Ed}}/_{N_{Rd}} + {}^{V_{Ed}}/_{V_{Rd}} = 0.92 + 0.31 = 1.22 > 1.2 \text{ Not OK} $$

3.8.4 DESIGN CHECKS FOR SERVICEABILITY LIMIT STATE

To fulfil requirement of serviceability limit state, it shall be established that the admissible displacement (C_d) of the fastening system is higher than or equal to the expected displacement due to design load (E_d) i.e. $E_d \leq C_d$.

The admissible displacement (C_d) shall be evaluated by the Structural Engineer while designing, depending on the application. The displacements may be assumed to vary as a linear function of the applied load. The displacements for the shear and tension components of the resultant load should be added vectorially for combined check. The product's Technical Assessment report should be referred for characteristic displacement of the anchor depending on load (tension or shear) and state of concrete (cracked or uncracked).

3.9 DESIGN FOR SEISMIC

EN 1992-4 [6] covers seismic design of structural (denoted as Type A connections) and non-structural attachments (denoted as Type B connections) in concrete using post-installed anchor, except in critical regions where concrete spalling or yielding of the reinforcement might occur during earthquake e.g. in plastic hinge zones. It is important to design non-structural connections for seismic if the primary structure is seismic resistant. Unfortunately, in practice it seldom happens [22]. According to EN 1992-4 [6] , the connection may be subjected to tension, shear or combination of the two. The seismic design forces (R_{Ed}) should be determined as per applicable code of practice depending on the connection type.

The concrete is assumed to be cracked[6] for seismic design. Also, anchor displacement[7] shall be accounted for in the design, with exception of non-critical

[6] Unless it can be demonstrated that the concrete will remain uncracked even in seismic event.

non-structural applications. This displacement shall be limited when a rigid connection is assumed or when immediate occupancy of building post seismic event is desired.

Seismic design of connections formed using post-installed anchors with stand-off installation or with a grout layer ($\geq 0.5d$) and fasteners in redundant systems are not covered by EN 1992-4 [6]. Additionally, it is not ideal to have an annular gap between the anchor and its fixture in seismically designed connections and should be avoided. If it is unavoidable, it should be accounted for in design as discussed in the following sections. In minor non-structural applications that are not critical, an annular gap of less than or equal to $d_{f,1}$ is permitted. In seismically designed connections, appropriate measures should be taken to avoid loosening of the nut or screw.

In case of very low seismicity[8] as defined in the applicable standards, the connection may be designed according to static or quasi static load design provisions as discussed in the previous sections.

3.9.1 PREQUALIFICATION FOR SEISMIC DESIGN

EN 1992-4 [6] provisions for seismic design can only be used for those post-installed anchors that have been assessed for use under seismic conditions as per applicable EAD in addition to the assessment for static application in cracked concrete. The product's Technical Assessment report clearly states whether or not it is suitable for seismic applications.

Based on the assessment, seismic performance categories C1 and/or C2 is assigned to the product in its Technical Assessment report like ETA. The category C2 is more stringent out of the two. Performance in terms of only resistance at ULS is available in case of category C1. Whereas, performance in terms of both resistances at ULS and displacements at ULS as well as DLS is

[7] Anchor displacements under seismic load at both damage limitation state (DLS) and ultimate limit state (ULS) are provided in the relevant ETA for C2 category as defined in Section 3.9.1.

[8] The low seismicity level is defined in EN 1998-1:2004[15], 3.2.1 (5). As the definition is slightly different in Indian standards, engineering judgement should be used to correlate the two provisions.

available in case of category C2. To ease the selection process, these performance categories are linked to seismicity level and importance class of the building as shown in Fig. 3.19 For example, anchor with C2 seismic performance category should be used in hospitals in high seismic zone.

	Seismicity level[a]		Importance Class acc. to EN 1998-1:2004, 4.2.5			
1	Class	$a_g \cdot S^c$	I	II	III	IV
2	Very Low[b]	$a_g \cdot S \le 0{,}05\,g$	No seismic performance category required			
3	Low[b]	$0{,}05\,g < a_g \cdot S \le 0{,}1\,g$	C1	C1[d] or C2[e]		C2
4	> low	$a_g \cdot S > 0{,}1\,g$	C1	C2		

[a] The values defining the seismicity levels are subject to a National Annex. The recommended values are given here.

[b] Definition according to EN 1998-1:2004, 3.2.1.

[c] a_g = design ground acceleration on type A ground (see EN 1998-1:2004, 3.2.1),

S = soil factor (see EN 1998-1:2004, 3.2.2).

[d] C1 for fixing non-structural elements to structures (Type 'B' connections)

[e] C2 for fixing structural elements to structures (Type 'A' connections)

Fig. 3.19 Snapshot of Table C.1 from EN 1992-4 [6] for recommended seismic performance categories

EXAMPLE 3.21

For designing a connection of steel beam to concrete column in an hospital in Delhi, fastener qualified for which seismic performance category should be used?

Solution:

Though direct comparison to Table C.1 of Eurocode cannot be made. We can make comparisons.

Hospital is designated as high important class building as per Indian Standards.

The connection in this example appears to be a structural connection.

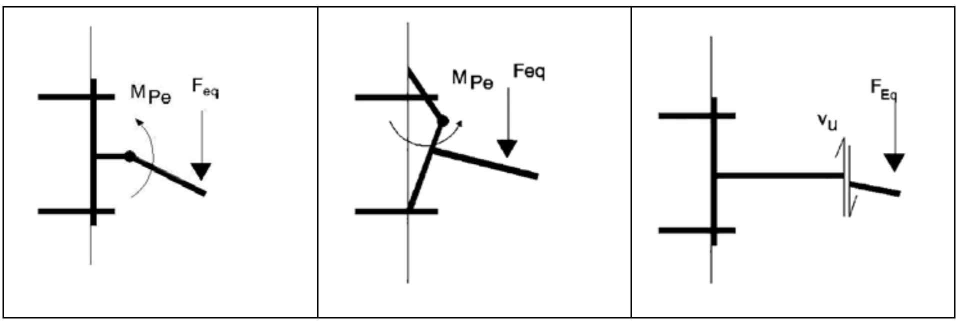
Delhi is designated as seismic zone 4, which corresponds to high seismic risk.

➔ Based on these correlations, it may be advisable to opt for fastener with seismic performance category C2 based on minimal information provided in this example.

3.9.2 SEISMIC DESIGN OPTIONS

EN 1992-4 [6] recommends the following three design options that the structural engineer may choose from for seismically designing their connections:-

The first one is known as capacity design (denoted as method a1). This option does not rely on ductility of fasteners to dissipate energy or contribute in ductile behaviour of structure during seismic event. As per capacity design, the anchor or anchor group is designed for the maximum tension and/or shear load that the fixture or attached element can transmit to the anchor based on either the development of a ductile yield mechanism in the fixture (see Fig. 3.20a) or the attached element (see Fig. 3.20b)). The mechanism should take into account strain hardening and material over-strength or the capacity of a non-yielding attachment (see Fig. 3.20c). For example, if only 5 kN tensile load can be transmitted to the anchor group by the attachment and fixture, then the anchor group should be designed for only 5 kN though its strength may be much higher, say 10 kN.



(a) Yielding in attachment (b) Yielding in fixture (c) Capacity of attachment

Fig. 3.20 Seismic design according to method a1 [6]

The second one is known as elastic design (denoted as method a2). This option also does not rely on ductility of fasteners. As per this method, the anchor or the anchor group is designed for the maximum load obtained from analysis which includes seismic in load combination (corresponding to ULS) assuming elastic behaviour of the anchor and the structure. The uncertainties in the analysis model to derive seismic forces on the fastening shall be accounted for. This is the only method that allows brittle failure in seismic design[9].

The third option is denoted as method b and it is applicable only for the tension component of the load acting on the anchor/anchor group. This method relies on the ductile behaviour of the fastener[10] i.e. sufficient elongation capacity is required. The fastening system is designed for the design load including the seismic component (corresponding to ULS). As per this method, the steel capacity of the anchor in tension shall be smaller than the capacity dependent on concrete failure modes. This method should not be used for anchoring of primary seismic members due to the expected large non-recoverable displacements of the anchor. EN 1992-4 [6] lays down some additional requirements specifically for this design method. As this option is seldom used, it is not discussed further in this book.

For the seismic design cases at the ULS where the seismic design tension or shear load applied to a single anchor or a group is equal to or less than 20 % of the total design tensile or shear load, respectively, for the same load combination then the acting tension/shear component may be verified omitting these design options.

3.9.3 DESIGN CHECK FOR SEISMIC

As per EN 1992-4 [6], the design seismic strength of the anchor ($R_{d,eq}$) is determined by dividing characteristic seismic resistance, $R_{k,\mathrm{eq}}$, by recommended partial safety factor, $\gamma_{M,eq}$, as shown in Eq. 3.14a. The characteristic value for seismic resistance ($R_{k,eq}$) should be determined according to Eq. 3.14b. The

[9] The seismic forces are derived on basis of the behaviour factor in Eurocode 8 [15]. Behaviour factor is denoted as q for structural elements and q_a for non-structural elements as per Eurocode 8[15]. Though it is not covered in Indian Standards, with exception of tall buildings, this factor w.r.t seismic design of anchors is briefly discussed here. For Type A connections, $q = 1$ is used for deriving seismic loads in order to assume non-energy dissipation for the whole structural system. For non-structural applications when $q_a = 1$ is assumed, the load used for designing anchors must still be multiplied by 1.5 amplification factor.

[10] Anchor's contribution should not be accounted for energy dissipation in the global structural analysis or in the analysis of a non-structural attachment. Exceptions are permitted in case of method b if justification is provided e.g. in form of hysteretic behaviour of anchor as per ETA.

partial safety factor for anchor subjected to seismic loads are identical to corresponding quasi static load conditions.

$$R_{d,eq} = R_{k,eq}/\gamma_{M,eq} \geq R_{Ed} \quad \text{(Eq. 3.14a)}$$

$$R_{k,\mathrm{eq}} = \alpha_{gap} \cdot \alpha_{eq} \cdot R^{0}_{k,eq} \quad \text{(Eq. 3.14b)}$$

In the above equation, the factor α_{gap} accounts for effects of annular gap between fastener-fixture in case of shear loading and is taken from ETA. The factor α_{eq} in Eq. 3.14b accounts for the influence of seismic forces along with associated cracking and is determined as per Table shown in Fig. 3.21

The factor $R^{0}_{k,eq}$is the basic characteristic resistance and is determined depending on the failure mode as follows:-

- The characteristic seismic resistance for steel failure in tension ($N_{Rk,\mathrm{s,eq}}$) and shear ($V_{Rk,\mathrm{s,eq}}$) as well as pull-out failure in tension ($N_{Rk,\mathrm{p,eq}}$) should be taken from ETA.
- In case of bonded anchors, the characteristic seismic resistance for bond failure ($N_{Rk,\mathrm{p,eq}}$) should be determined using the procedure defined for calculating ($N_{Rk,\mathrm{p}}$) but using the seismic bond value $\tau_{Rk.eq}$.
- For all other failure modes, the characteristic seismic resistance for that failure type is calculated using the corresponding procedure for static.

Failure type related to tension	**Single fastener[1]**	**Fastener group**
Steel failure	1	1
Concrete cone failure - Headed fastener & undercut fastener with factor same as headed fastener - All other fasteners	 1 0.85	 0.85 0.75
Pull-out failure	1	0.85
Steel failure of reinforcement	1	1

Concrete splitting failure	1	0.85
Concrete blow-out failure	1	0.85
Steel failure of reinforcement	1	0.85
Failure type related to shear	**Single fastener**[1]	**Fastener group**
Steel failure	1	0.85
Concrete pry-out failure - Headed fastener & undercut fastener with factor same as headed fastener - All other fasteners	 1 0.85	 0.85 0.75
Concrete edge failure	1	0.85
Steel failure of reinforcement	1	0.85
Anchorage failure of reinforcement	0.85	0.75

[1]These values are also applicable when only one fastener in a fastener group is subjected to tension.

Fig.3.21 Resistance factor $\boldsymbol{\alpha_{eq}}$ as per Table C.3 of EN 1992-4 [6]

The following combined tension and shear check should be carried out for steel failure and all other failure modes, when subjected to design tension (N_{Ed}) and shear (V_{Ed}) including seismic effects for corresponding failure modes. The factor k_{15} should be taken as one in both cases. The product's Technical Assessment report may provide more precise value for this factor.y

$$\left[{}^{N_{Ed}}/_{N_{Rd,i,eq}}\right]^{k_{15}} + \left[{}^{V_{Ed}}/_{V_{Rd,i,eq}}\right]^{k_{15}} \leq 1 \qquad \text{(Eq. 3.12)}$$

In case of steel failure, the value $N_{Rd,i,eq}$and $V_{Rd,i,eq}$are taken equal to $N_{Rd,s,eq}$and $V_{Rd,s,eq}$,respectively in Eq. 3.12. In case of all other failure modes except steel, the largest ratio of ${}^{N_{Ed}}/_{N_{Rd,i,eq}}$ and ${}^{N_{Ed}}/_{N_{Rd,i,eq}}$ should be used in Eq. 3.12.

3.9.4 DISPLACEMENT OF ANCHORS

The anchor displacement should be limited to $\delta_{N,req(DLS)}$ under tensile load and to $\delta_{V,req(DLS)}$ under shear loads at damage limit state (DLS). This is required to meet functional and support requirements, depending on the application. If a rigid support is assumed in design, then it shall be established that the limiting displacement is compatible to the structural behaviour/ requirement[11]. Anchors should also be able to accommodate expected deformations (displacements or rotations) associated with the connection, e.g. as in case of secondary seismic members. The rotation of the connection θ_p can be taken as equal to $\delta_{N,eq}/s_{max}$, where $\delta_{N,eq}$ is the displacement of the anchor under seismic load and s_{max} is the distance between the outermost row of the anchors and the opposite edge of the fixture. If immediate occupancy of structure is desired (i.e. attached element should be operational post-earthquake) then the relevant displacements should be accounted for.

If the anchor displacements under tension (i.e. $\delta_{N,eq(DLS)}$) and/or under shear (i.e. $\delta_{V,eq(DLS)}$) as per product's Technical Assessment report (e.g. ETA) are higher than the required displacement values ($\delta_{N,req(DLS)}$and/or $\delta_{V,req(DLS)}$) then the design resistance should be reduced as follows.:

$$N_{Rd,eq,red} = N_{Rd,eq} \cdot \delta_{N,req(DLS)}/\delta_{N,eq(DLS)} \quad \text{(Eq. 3.13)}$$

$$V_{Rd,eq,red} = V_{Rd,eq} \cdot \delta_{V,req(DLS)}/\delta_{V,eq(DLS)} \quad \text{(Eq. 3.14)}$$

Example 3.22 illustrates typical calculations necessary for checking seismic resistance.

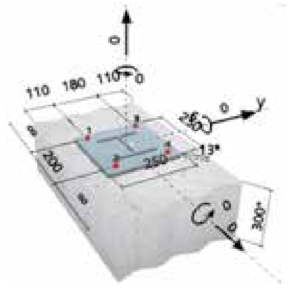
EXAMPLE 3.22

Check if a fixture of size 250 mm by 250 mm supporting a bracket (as shown below) subjected to 6 kN-m moment along y axis can be connected to the column of size 400 mm by 300 mm using HY 200 with M16 Hit-Z rods with 96 mm embedment (ETA-12/0028 [21], C2 seismic performance category). The

[11] In most cases, the acceptable displacement associated with a rigid support is in the range of 3 mm.

base material is of M50 grade concrete. The connection has to be seismically designed as per design option a2. Assume a reinforcement mesh of 10 mm bar spaced at 100 mm and reinforcement to control splitting is also present. Note that the design load is based on seismic analysis. The neutral axis is located at 54.3 mm from compression side.

Given:



$d_{nom} = 16$ mm

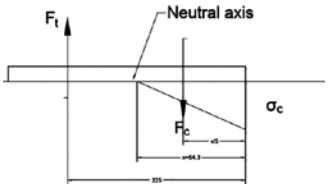
$h_{ef} = 96$ mm

$f_{ck} = 40$ N/mm^2 (cylinder strength corresponding to M50)

Moment on anchor group $M_{Ed} = 6$ kN − m

Solution:

Determination of force on each anchor



$z = 250 - 25 = 225$ mm

$F_t(z - x/3) = M_{Ed}$

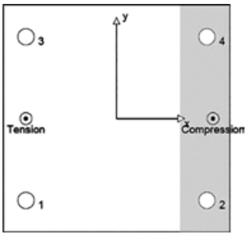
$$F_t = \frac{6000}{225 - \frac{54.3}{3}} = 29 \text{ kN}$$

Force on each anchor $= 29/2 = 14.5$ kN

Tension on anchor 1 and 3 each = 14.5 kN

Anchor 2 and 4 will be in compression

Check design steel resistance in tension



$N^h_{Ed,eq} = 14.5$ kN

$N^0_{Rk,s,eq} = 96$kN (from Table C9 of ETA-12/0028)

$\gamma_{inst} = 1$ (from Table C9 of ETA-12/0028)

$\gamma_{Ms,eq} = \gamma_c * \gamma_{inst} = 1.5 * 1 = 1.5$

$\alpha_{gap} = 1$

$\alpha_{eq} = 1$

$N_{Rk,s,eq} = \alpha_{gap} \cdot \alpha_{eq} \cdot N^0_{Rk,s,eq} = 96$ kN

$N_{Rd,eq} = N_{Rk,s,eq}/\gamma_{Ms,eq} = 96/1.5 = 64$ kN $\geq N^h_{Ed,eq}$

$\beta = N^h_{Ed,eq}/N_{Rd,eq} = 14.5/64 = 0.226$ → 23% utilization. OK

Check design bond failure resistance in tension (i.e. combined pull-out and concrete cone failure)

$N^g_{Ed,eq} = 2 * 14.5 = 29$ kN (only anchors 1 & 3 will act as group to resist pull-out))

$\tau_{Rk,eq} = 19$ N/mm² (from Table C9 of ETA-12/0028)

$\Psi^0_{sus} = 0.6$ (as no value is provided in ETA)

$\alpha_{sus} = 0$ (as there is no sustained load)

$\Psi_{sus} = 1$ (For $\alpha_{sus} \leq \Psi^0{}_{sus}$)

$N^0_{Rk,p,eq} = \Psi_{sus}\tau_{Rk,eq}\pi d h_{ef}$

$N^0_{Rk,p} = 1 * (19/1000) * 3.14 * 16 * 96 = 91.6$ kN

$s_{cr,Np} = 7.3d(\Psi_{sus}\tau_{Rk,ucr})^{0.5} \leq 3h_{ef}$

$s_{cr,Np} = 7.3 * 16 * (1 * 24)^{0.5} = 572.2$ mm $> 3h_{ef} = 288$ mm Not OK

Use $s_{cr,Np} = 288$ mm

$c_{cr,Np} = 0.5 s_{cr,Np} = 144$ mm

$A^0_{p,N} = s_{cr,Np} \cdot s_{cr,Np} = 288 * 288 = 82944$ mm²

$A_{p,N} = (110 + 180 + 110) * (144 + 144) = 115200$ mm²

$A_{p,N}/A^0_{p,N} = 115200/82944 = 1.38$

$k_3 = 7.7$ (for cracked concrete)

$\tau_{Rk,c} = (k_3/\pi d)\sqrt{h_{ef} f_{ck}} = (7.7/(3.14 * 16))\sqrt{96 * 40} = 9.49$ N/mm²

$\Psi_{g,Np} = \sqrt{n} - (\sqrt{n} - 1) \cdot \left[\frac{\tau_{Rk,eq}}{\tau_{Rk,c}}\right]^{1.5}$

$\Psi^0_{g,Np} = \sqrt{2} - (\sqrt{2} - 1) \cdot \left[\frac{19}{9.49}\right]^{1.5} = 0.241 < 1$ Not OK

Use $\Psi^0_{g,Np} = 1$

$\Psi_{g,Np} = \Psi^0_{g,Np} - (s/s_{cr,Np})^{0.5} * (\Psi^0_{g,Np} - 1) = 1 - (180/288)^{0.5} * 0 = 1$ OK

$\Psi_{c,eq} = 1$

$\Psi_{s,Np} = 0.7 + 0.3(c/c_{cr,Np})$

$\Psi_{s,Np} = 0.7 + 0.3(110/144) = 0.929$

$\Psi_{re,N} = 1$ (due to reinforcement assumption)

$\Psi_{ec,Np} = 1$ (as there is no eccentricity of load)

$\alpha_{gap} = 1$

$\alpha_{eq} = 0.85$

$N_{Rk,p} = N^0_{Rk,p,eq} \, A_{p,N}/A^0_{p,N} \, \Psi_{g,Np} \, \Psi_{s,Np} \, \Psi_{re,Np} \, \Psi_{ec,Np} \, \alpha_{gap} \cdot \alpha_{eq}$

$N_{Rk,p,eq} = 91.6 * 1.38 * 1 * 0.929 * 1 * 1 * 1 * 0.85 \approx 99.8 \text{ kN}$

$\gamma_{Mp,eq} = 1.5$ (same as static, from ETA-12/0028)

$N_{Rd,\text{p,eq}} = N_{Rk,p}/\gamma_{Mp} = 99.8\ /1.5 = 66.5 \text{ kN} \geq N^g_{Ed,eq}$

$\beta = N^g_{Ed,eq}/N_{Rd,\text{p,eq}} = 29/66.5\ = 0.436$ → 44% utilization. OK

Check concrete breakout failure resistance in tension

$k_1 = 7.7$ (for cracked concrete)

$N^g_{Ed,eq} = 2 * 14.5 = 29 \text{ kN}$ (only anchors 1 & 3 will act as group to resist concrete cone)

$N^0_{Rk,c,eq} = k_1\sqrt{f_{ck}}h^{1.5}_{ef} = 7.7 * \sqrt{40}(96)^{1.5} = 45807 \text{ N} = 45.807 \text{ kN}$

$s_{cr,N} = 3h_{ef}$

$s_{cr,N} = 3 * 96 = 288\ \mathrm{mm}$

$c_{cr,N} = 0.5 s_{cr,N} = 0.5 * 288 = 144\ \mathrm{mm}$

$A^0_{c,N} = s_{cr,N} \cdot s_{cr,N} = 288 * 288 = 82944\ \mathrm{mm}^2$

$A_{c,N} = (110 + 180 + 110) * (144 + 144) = 115200\ \mathrm{mm}^2$

$A_{c,N} / A^0_{c,N} = 115200/82944 = 1.38$

$\Psi_{s,N} = 0.7 + 0.3(c/c_{cr,N}) = 1$ (due to infinite edge assumption)

$\Psi_{s,N} = 0.7 + 0.3(110/144) = 0.929$

$\Psi_{re,N} = 1$ (due to reinforcement assumption)

$\Psi_{ec,N} = 1$ (as there is no eccentricity of load)

$\Psi_{M,N} = 1$

$\alpha_{gap} = 1$

$\alpha_{eq} = 0.85$

$N_{Rk,c,eq} = N^0_{Rk,c,eq} \; A_{c,N} / A^0_{c,N} \; \Psi_{s,N} \; \Psi_{re,N} \; \Psi_{ec,N} \; \Psi_{M,N} \; \alpha_{gap} \; \alpha_{eq}$

$N_{Rk,c,eq} = 45.807 * 1.38 * 0.929 * 1 * 1 * 1 * 1 * 0.75 = 44.044\ \mathrm{kN}$

$\gamma_{Mc,eq} = \gamma_c * \gamma_{inst} = 1.5 * 1 = 1.5$ (same as static, from ETA-12/0028)

$N_{Rd,\mathrm{c,eq}} = N_{Rk,c,eq} / \gamma_{Mc,eq} = 44.044 / 1.5 = 29.36 \geq N^g_{Ed,eq}$

$\beta = N^g_{Ed,eq} / N_{Rd,\mathrm{c,eq}} = 29/29.36 = 0.987$ → 99% utilization. OK

Design check for splitting failure in tension is not required due to reinforcement assumption

Combined tension and shear check are not applicable as connection is not subjected to any shear load

➔ All the design checks are fulfilled when the connection is formed using HY 200 with M16 Hit-Z rod (C2 seismic performance category) as per seismic design option a2.

3.10 DESIGN FOR FATIGUE

EN 1992-4 [6] covers design of connections using post-installed anchors, subjected to pulsating tension or pulsating shear and alternating shear and/or their combinations. Only connections without lever arm are permitted under shear. Connections with annular gaps are also not permitted under fatigue. It is important to avoid loosening of the nut or screw by adopting appropriate measures. A permanent prestressing force on the anchor shall be present during its service life. For fatigue design, checks are carried out for both static and fatigue. Checks under static load conditions have already been discussed in the previous sections of this chapter. In this section, the checks for fatigue load are discussed. Illustrative examples are not covered in this section.

3.10.1 PREQUALIFICATION FOR FATIGUE DESIGN

The provisionsfor fatigue designin EN 1992-4 [6] can only be used for those post-installed anchors that have been assessed for use under fatigue conditions in addition to the assessment for static application, with exception of redundant non-structural fastening systems. Typically, the product's Technical Assessment report clearly states whether or not the post-installed anchor is suitable for this application. This pre-qualification is very important when the post-installed anchors are expected to be subjected to frequent repeated load cycles.

3.10.2 CHECKS FOR FATIGUE LOAD - TENSION

In general, the fatigue strength should be greater than fatigue load in tension. The fatigue load in tension is determined by multiplying the partial safety factor $\gamma_{F,fat}$ with ΔN_{Ek}, which is peak to peak amplitude of the fatigue tensile action blow-out for 2*10^6 load cycles. i.e. $N_{Ek,max} - N_{Ek,min}$. The ΔN_{Ek} load on most stressed anchor in an anchor group is denoted as ΔN_{Ek}^{h} and on entire anchor group is denoted as ΔN_{Ek}^{g} . The partial safety factor $\gamma_{F,fat}$ may be taken as 1.

The fatigue strength of most stressed anchor of an anchor group is multiplied with a reduction factor denoted as $\Psi_{F,N}$, for steel and pull-out failure. This reduction factor accounts for unequal distribution of the tension load acting on the fixture to the individual anchors of an anchor group. This value should be taken from Product's Technical Assessment report but should be ≤ 1.

(A) Resistance to steel failure

As per EN 1992-4 [6], the check for steel failure should be carried out either for single anchor or for the most unfavourably loaded anchor in an anchor group. The fatigue steel strength of anchor in tension ($\Delta N_{Rk,s}$) should be taken from product's Technical Assessment report. The partial safety factor[12] $\gamma_{Ms,N,fat}$ may be taken as 1.35.

$\gamma_{F,fat} \cdot \Delta N_{Ek} \leq \Delta N_{Rk,s} / \gamma_{Ms,N,fat}$ [for single anchor] (Eq. 3.15a)

$\gamma_{F,fat} \cdot \Delta N_{Ek}^{h} \leq \Psi_{F,N} \cdot \Delta N_{Rk,s} / \gamma_{Ms,N,fat}$ [for anchor group] (Eq. 3.15b)

(B) Resistance to concrete cone failure

As per EN 1992-4 [6], the check for concrete cone failure should be carried out either for single anchor or for the anchor group. The fatigue concrete cone strength of anchor in tension for 2*10^6 load cycles ($\Delta N_{Rk,c}$) is taken as $0.5N_{Rk,c}$, where $N_{Rk,c}$ is calculated using method discussed in static design section. The partial safety factor $\gamma_{Mc,fat}$ may be taken as $1.5\gamma_{inst}$.

$\gamma_{F,fat} \cdot \Delta N_{Ek} \leq \Delta N_{Rk,c} / \gamma_{Mc,fat}$ [for single anchor] (Eq. 3.16a)

$\gamma_{F,fat} \cdot \Delta N_{Ek}^{g} \leq \Delta N_{Rk,c} / \gamma_{Mc,fat}$ [for anchor group] (Eq. 3.16b)

(C) Resistance to pull-out failure

As per EN 1992-4 [6], the check for concrete cone failure should be carried out either for single anchor or for the most unfavourably loaded anchor in an anchor group. This check is applicable for post-installed mechanical anchors and post-installed bonded expansion anchors. The fatigue pull-out strength of anchor in tension ($\Delta N_{Rk,p}$) should be taken from product's Technical Assessment report. The partial safety factor $\gamma_{Mp,fat}$ may be taken as $1.5\gamma_{inst}$.

$\gamma_{F,fat} \cdot \Delta N_{Ek} \leq \Delta N_{Rk,p} / \gamma_{Mp,fat}$ [for single anchor] (Eq. 3.17a)

$\gamma_{F,fat} \cdot \Delta N_{Ek}^{h} \leq \Psi_{F,N} \cdot \Delta N_{Rk,p} / \gamma_{Mp,fat}$ [for anchor group] (Eq. 3.17b)

[12] As per EN 1992-4[6], the partial safety factors may be found in the Country's National Annex

(D) Resistance to concrete splitting failure

As per EN 1992-4 [6], the check for concrete splitting failure should be carried out either for single anchor or for the anchor group. The fatigue concrete splitting strength of anchor in tension for 2*10^6 load cycles ($\Delta N_{Rk,sp}$) is taken as $0.5N_{Rk,sp}$, where $N_{Rk,sp}$ is calculated using method discussed in static design section. The partial safety factor $\gamma_{Mc,fat}$ may be taken as $1.5\gamma_{inst}$.

$\gamma_{F,fat} \cdot \Delta N_{Ek} \leq \Delta N_{Rk,sp}/\gamma_{Mc,fat}$ [for single anchor] (Eq. 3.18a)

$\gamma_{F,fat} \cdot \Delta N_{Ek}^{g} \leq \Delta N_{Rk,sp}/\gamma_{Mc,fat}$ [for anchor group] (Eq. 3.18b)

3.10.3 CHECKS FOR FATIGUE LOAD - SHEAR

The fatigue strength should be greater than the fatigue load in shear. The fatigue load in shear is determined by multiplying the partial safety factor $\gamma_{F,fat}$with ΔV_{Ek}, which is peak to peak amplitude of the fatigue shear load for 2*10^6 load cycles. i.e. $V_{Ek,max} - V_{Ek,min}$. The ΔV_{Ek} load on most stressed anchor in an anchor group is denoted as ΔV_{Ek}^{h} and on entire anchor group is denoted as ΔV_{Ek}^{g} . The partial safety factor $\gamma_{F,fat}$ may be taken as 1.

(A) Resistance to steel failure

As per EN 1992-4 [6], the check for steel failure should be carried out either for single anchor or for the most unfavourably loaded anchor in an anchor group. The fatigue steel strength of anchor in shear ($\Delta V_{Rk,s}$) should be taken from product's Technical Assessment report. The partial safety factor[13]$\gamma_{Ms,V,fat}$ may be taken as 1.35. The fatigue strength of most stressed anchor of an anchor group is multiplied with a reduction factor denoted as $\Psi_{F,V}$. This reduction factor accounts for unequal distribution of the shear load acting on the fixture to the individual anchors of an anchor group. This value should be taken from Product's Technical Assessment report but should be ≤ 1. This reduction factor may be taken as 1 for group of two anchors subjected to shear perpendicular to

[13] As per EN 1992-4[6], the partial safety factors may be found in the Country's National Annex

axis of the anchors, provided the fixture is not restrained against in-plane rotation.

$\gamma_{F,fat} \cdot \Delta V_{Ek} \leq \Delta V_{Rk,s}/\gamma_{Ms,V,fat}$ [for single anchor] (Eq. 3.19a)

$\gamma_{F,fat} \cdot \Delta V_{Ek}^{h} \leq \Psi_{F,N} \cdot \Delta V_{Rk,s}/\gamma_{Ms,V,fat}$ [for anchor group] (Eq. 3.19b)

(B) Resistance to concrete pry-out failure

As per EN 1992-4 [6], the check for concrete cone failure should be carried out either for single anchor or for the anchor group. The fatigue concrete pry-out strength of anchor in shear for 2*10^6 load cycles ($\Delta V_{Rk,cp}$) is taken as $0.5V_{Rk,cp}$, where $V_{Rk,cp}$ is calculated using method discussed in static design section. The partial safety factor $\gamma_{Mc,fat}$ may be taken as $1.5\gamma_{inst}$.

$\gamma_{F,fat} \cdot \Delta V_{Ek} \leq \Delta V_{Rk,cp}/\gamma_{Mc,fat}$ [for single anchor] (Eq. 3.20a)

$\gamma_{F,fat} \cdot \Delta V_{Ek}^{g} \leq \Delta V_{Rk,cp}/\gamma_{Mc,fat}$ [for anchor group] (Eq. 3.20b)

(C) Resistance to concrete edge failure

As per EN 1992-4 [6], the check for concrete edge failure should be carried out either for single anchor or for the anchor group. The fatigue concrete edge strength of anchor in shear for 2*10^6 load cycles ($\Delta V_{Rk,c}$) is taken as $0.5V_{Rk,c}$, where $V_{Rk,cp}$ is calculated using method discussed in static design section. The partial safety factor $\gamma_{Mc,fat}$ may be taken as $1.5\gamma_{inst}$.

$\gamma_{F,fat} \cdot \Delta V_{Ek} \leq \Delta V_{Rk,c}/\gamma_{Mc,fat}$ [for single anchor] (Eq. 3.21a)

$\gamma_{F,fat} \cdot \Delta V_{Ek}^{g} \leq \Delta V_{Rk,c}/\gamma_{Mc,fat}$ [for anchor group] (Eq. 3.21b)

3.10.4 CHECKS FOR FATIGUE LOAD - COMBINED TENSION AND SHEAR

The following check should be satisfied for combined tension and shear. The largest value of $\beta_{N,fat}$ and $\beta_{V,fat}$ out of the different failure modes under consideration should be used in the following equation.

$(\beta_{N,fat})^{\alpha} + (\beta_{V,fat})^{\alpha} \leq 1$ (Eq. 3.22a)

Where

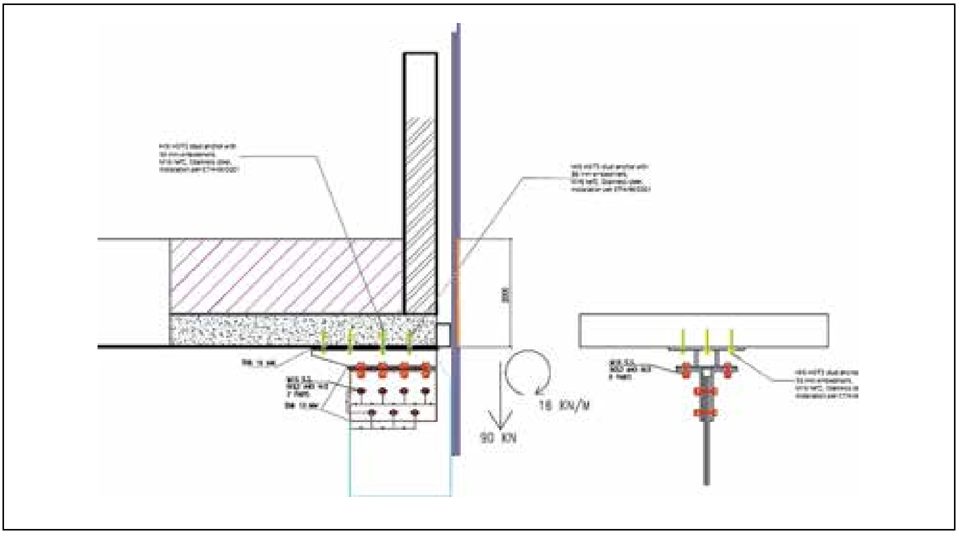
$$\beta_{N,fat} = \frac{\gamma_{F,fat} \cdot \Delta N_{Ek}}{\Psi_{F,N} \cdot \Delta N_{Rk} / \gamma_{M,fat}} \leq 1 \qquad \text{(Eq. 3.21b)}$$

$$\beta_{V,fat} = \frac{\gamma_{F,fat} \cdot \Delta V_{Ek}}{\Psi_{F,V} \cdot \Delta V_{Rk} / \gamma_{M,fat}} \leq 1 \qquad \text{(Eq. 3.21c)}$$

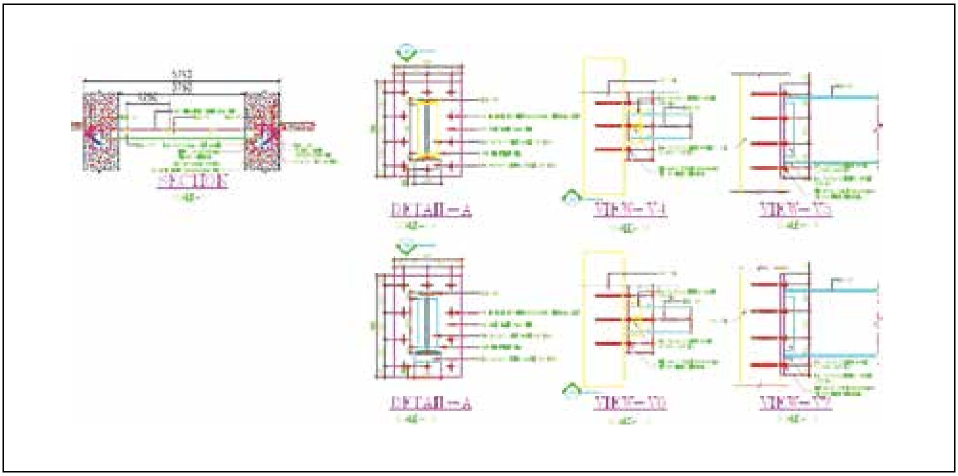
αshould be taken as α_s for steel failure check and α_c for other failure mode checks. Both α_s and α_c should be taken from product's Technical Assessment report.

3.11 DESIGN SPECIFICATION

The design details like - the position of the attachment on fixture, the position of fasteners, fixture details, special installation instructions etc. should be clearly marked in the specification and construction drawings. Some typical specification details are shown inFig.3.22.



(a) Typical specification details of connection formed using post-installed mechanical anchors



(b) Typical specification details of connection formed using post-installed bonded anchors

Fig. 3.22 Typical specification details for connections formed using post-installed anchors

3.12 INSTALLATION AND INSPECTION

Correct installation of post-installed anchors is very important to ensure performance of connection as per design. The installation step varies from manufacturer to manufacturer. The installation steps recommended in the product's Technical Assessment report should be adhered to. The installation should be carried out under supervision by a skilled labourer who has been trained on product installation. Some general installation steps are discussed in this section (see Fig.3.23). The very first step is identification of the location and marking the coordinates. The concrete member is scanned to determine safe drilling location. If scanning is not possible, then construction drawings should be referred to. The hole is then drilled using drilling tools of type and size recommended by the manufacturer. Aborted drill holes (if any) should be filled with non-shrinkage mortar with strength at least equal to that of the base material concrete (but ≥ 40 N/mm^2). The drilled hole is then cleaned. In some systems, like SAFEset, the cleaning step may be eliminated. The anchor is then installed. In case of bonded post-installed anchors, the adhesive is injected and then the fastening element is inserted. The installation details like installer name, supervisor name, installation date, product details etc. is maintained for inspection. Sometimes proof load test/ or onsite test is also carried out to check quality of workmanship. Onsite tests are only a mean of checking installation and not a tool for comparing two products. Details like test equipment, applied load type and value, result interpretation etc. is documented and maintained for each onsite test

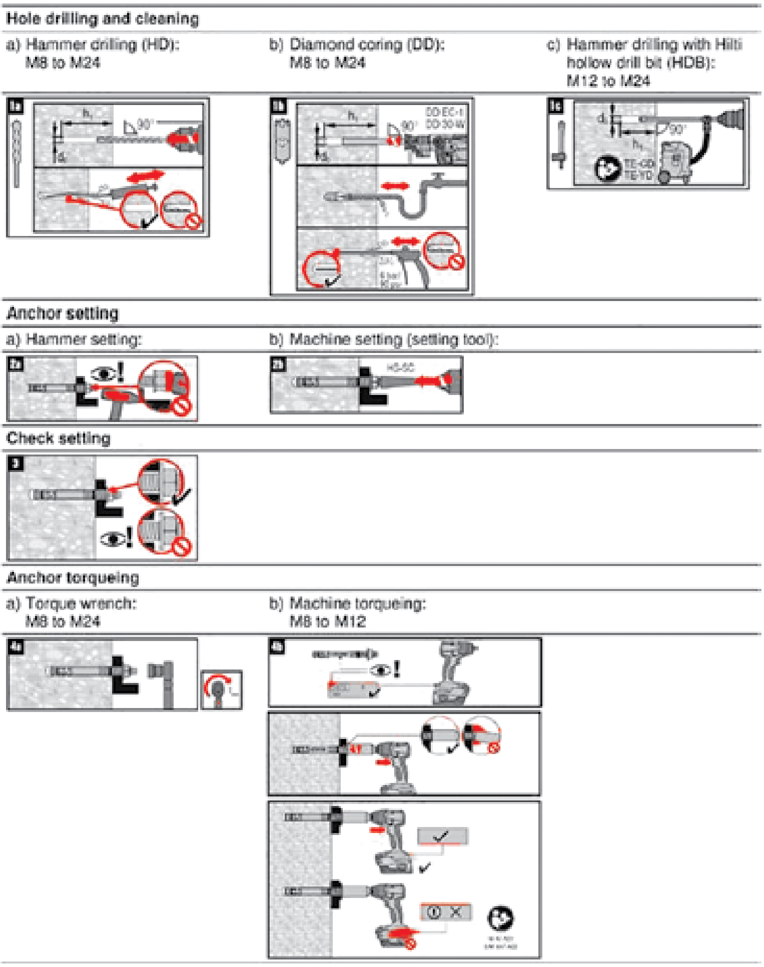


Fig. 3.23 Typical steps for installing torque-controlled post-installed anchors [20]

3.13 PRACTISE PROBLEMS

Case 1: Design anchor system for fixing of I-section on a column size 1200 x 1200 mm with grade of concrete M30 (cracked concrete) for purpose of connecting steel structure to concrete building. The I-section is to transfer tensile load of 268 tons and shear load of 0.43 tons vertically downward (factored) through the given column.

Case 2: A steel column is to be fixed on a slab of thickness 200 mm of concrete grade M30 (infinite edge). The load coming on the base plate are as given below:

	Z (Vertical)	X	Y
Load (kN)	42.56	6.886	0.043
Moment, kN-m	0.108	0.0205	97.755

Case 3: A slab cut-out is to be closed with steel plate. This plate needs to be rested and welded to C-channels fixed on the top side of the slab of the cut-out. Thickness of slab is 450 mm and grade of concrete is M35. Load on each anchor is 6 kN (DL). Design suitable anchor to fix the channel to the slab.

Case 4: Fixing of base plate to be done on a column of 1000 mm x 1200 mm (Face 1000mm). Grade of concrete is M30 and loads on the base plate are 10 kN in tension and 50 kN shear in horizontal direction. Base plate to be fixed is 500 x 750 mm with the profile of ISMB 500. Design the anchor system so that they are located inside the flanges of the I-profile.

Case 5: A grid of steel I-section needs to be fixed to support a water tank. The base plate needs to be fixed on the 220 mm side of the column with depth 480 mm. Grade of column is M16 with DL on base plate is 28.7 kN. Design the anchor system required to support the base plate for the grid.

CHAPTER 4

Direct Fastening

4.1 OVERVIEW

After having understood the design basics of cast-in anchor channels and post-installed anchoring technology, we now deep dive into direct fastening technology. It is also a form of post-installed fastening technology as it is installed into hardened concrete. However, in most cases, it does not require drilling prior to installation. A nail or threaded stud being hammered into the concrete is the simplest and oldest example of direct fastening. In this chapter, the direct fastening systems that are tool based are discussed. Direct fastening systems can be used in a variety of base materials like masonry, concrete, steel, wood etc., but in this chapter, we restrict our discussion to fastening systems for concrete only.

Unlike cast-in anchor channels and post-installed anchoring technology, design of direct fastening systems for concrete is generally not covered by any European standards. The nails are selected based on performance and assessment data provided by the manufacturer as per established guidelines/ process. Hence, design guidelines are not discussed in this chapter. Instead, different parameters that should be considered by the design engineer while selecting this system are discussed.

We begin this chapter by discussing about different types of direct fastening systems and their applications. We then discuss different parameters that may influence performance of these systems. We then move on to talk about parameters to keep in mind while selecting the direct fastening system along with the fastener. We conclude this chapter by discussing about installation and safety aspect.

4.2 TYPES OF DIRECT FASTENING SYSTEMS

Direct fastening technology has evolved over the years and branched out into three based on the primary source of energy that triggers the fastening mechanism, namely - Powder-actuated technology, Gas-actuated technology and Battery-powered technology. Let us discuss in detail about each of these technologies: -

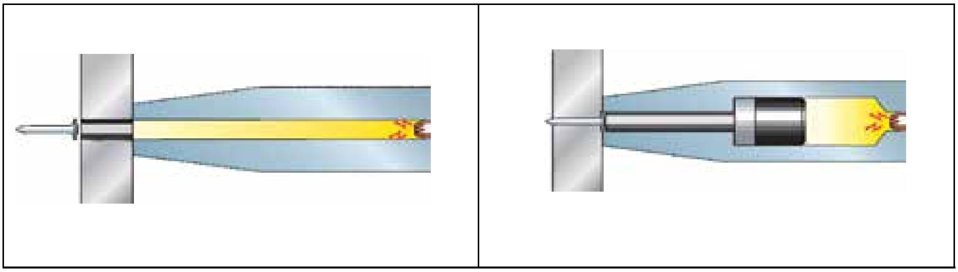
4.2.1 POWDER-ACTUATED TECHNOLOGY

This is the oldest and traditional type of direct fastening system. Powder-actuated fastening system is a direct fastening solution that drives the fastener in the hardened concrete (e.g. nail or threaded stud) using tool powered by propellant charge. The propellant charge is usually available in the form of cartridges of different power. Powder-actuated fastening tool can be categorized into either high-velocity or low velocity tools. In high-velocity powder-actuated fastening tool, also known as “direct-acting tool”, the propellant charge acts directly on the fastener as shown in Fig. 4.1(a). Whereas, in the low-velocity variant, also known as “indirect-acting tool” the propellant acts on a piston which in turn drives the fastener into the base material as shown in Fig. 4.1(b). The high-velocity variant can reach speed of 400 to 500 m/sec, therefore this category of tool is subjected to stringent rules for usage or even prohibited in some countries. The speed of low-velocity variant is much lower, usually below 100 m/sec.

Its more safe to use the low-velocity variant due to reduced injury risk and hence this category of tool is subjected to less stringent rules for usage compared to the high-velocity variant [23] [24]. This classification is based on ANSI A10.3-2006 [25]. One of the requirements to meet European CE marking[1] for Direct fastening tools is that it has to be indirect-acting.

The “indirect-acting tools” are based on either of the following operating principles:- Co-acting, impact and contact operation. Basically, these are the three different ways in which the piston can come into contact with the fastener upon being triggered (see Fig. 4.2) [24]. In India, however, there are restrictive regulations which makes it difficult to procure and use powder-actuated tools. Some contractors have special licenses to procure such tools and execute the application in India.

[1] CE marking is a certification/ conformity mark for products in European economic area



(a) Direct-acting tool (b) Indirect-acting tool

Fig.4.1 Types of powder-actuated tools [24]

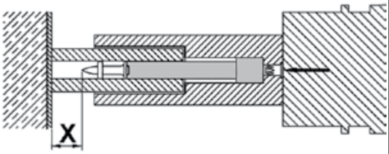
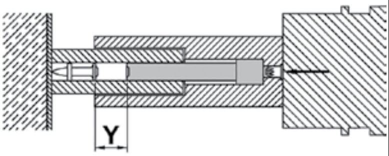
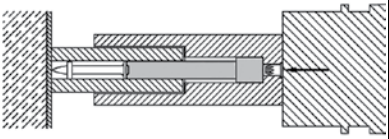
Operating principle	Characteristics	
Co-acting operation	• **X** > 0 ; **Y** = 0 • Highest application limit • Lowest recoil	
Impact operation	• **X** = 0 ; **Y** > 0 • Lower application limit • Higher recoil	
Contact operation	• **X** = 0 ; **Y** = 0 • Lowest application limit • Highest recoil	

Fig.4.2 Different types of operating principles (excerpt from DFTM [24])

4.2.2 GAS-ACTUATED TECHNOLOGY

This direct fastening system uses gas as actuating source. The fuel source is available in form of gas canisters. Upon ignition, the expanding gas transfers the energy to the piston which in turn drives the fastener into the base material. Unlike powder-actuated tools, Gas-actuated tools can be freely used in India, with no restrictive regulations [24].

4.2.3 BATTERY-POWERED TECHNOLOGY

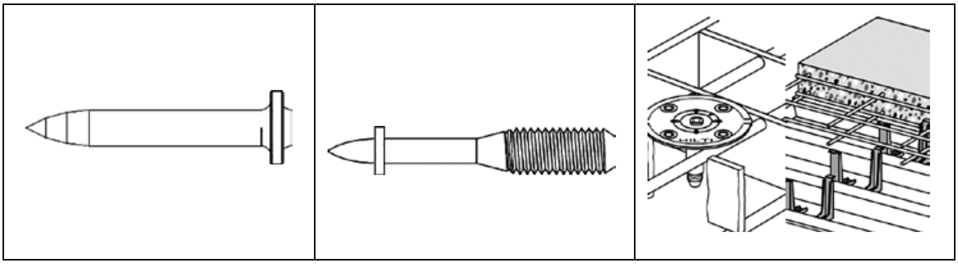
This "combustion free" direct fastening system uses battery as power source. This tool does not require any propellant. The piston in this case is driven by an electric motor (see Fig. 4.3) powered by the battery [24].

Fig. 4.3 Cut-off of a battery-powered tool

4.3 TYPES OF FASTENERS COMPATIBLE WITH DIRECT FASTENING SYSTEMS

The fasteners for concrete applications can be broadly categorized as nails, threaded studs and composite fasteners (see Fig. 4.4). For these fasteners to be able to penetrate the fastened and/or the base material, they should be harder than it. The first variety, i.e. nails, are specially manufactured. Some of them are equipped with plastic washers that aid in holding the nail in fastener guide to prevent it from dropping out and guidance during driving.

Threaded studs are a variety of nails in which the traditional head is replaced with a threaded upper section. The third variety of direct fasteners, known as composite fasteners, are essentially an assembly of nail along with an application specific fastening attachment such as a clip, plate, disk, etc. The direct fasteners for concrete applications are typically longer than their steel counterparts [24].



(a) Nails (b) Threaded studs (c) Composite fasteners

Fig. 4.4 Different types of direct fasteners [24]

These fasteners compatible with "powder-actuated", "gas-actuated" or "battery-powered" tools derive their holding capacity[2] in concrete through combination of the following two mechanisms: - Sintering and keying. Minor clamping action also comes into play but does not contribute significantly to holding capacity of the fastener in concrete and hence is not discussed here. Let's understand how each of these principles work.

4.3.1 SINTERING

When the fastener is pushed into the concrete using the direct fastening tool, it creates space for itself by pushing the concrete on the side and thereby compacting it in the process. Intense heat generated during the process causes concrete to be sintered onto the surface of the fastener. When a fastener that has formed sintered bond with concrete is pulled-out to its maximum capacity, it usually fails due to breakage of this bond. In fact, the most common failure mode observed while performing pull-out tests is breakage of the sintered bond between the concrete and the fastener, especially at and near the point. The presence of

[2] It is a measure of fastener's (e.g. nail) load bearing capacity.

sintered bond can be confirmed by examining the surface of the fastener, which is marked by a characteristic rough surface closer to the point region due to sintered concrete (see Fig.4.5). This sintered concrete can be removed only using a grinding tool and not just by rubbing the surface. The strength of this sintered bond is actually greater than that of the clamping effect due to reactive forces of the concrete on the fastener.

Fig. 4.5 Example of Sintering [24]

4.3.2 KEYING

The sintered concrete in the region of the fastening point forms ridges on the surface of fastener which helps in micro-interlocking between the fastener and the concrete.

4.4 APPLICATIONS

Applications of direct fasteners were not explicitly covered in Chapter 1. of this book. Therefore, we spend some time in this section exploring the wide range of applications that the direct fasteners can be used in. Direct fasteners for concrete can be used for applications like wire mesh fixing, system formwork, light-weight conduit/pipe support, etc. Some if these applications are illustrated in Fig. 4.6.

(a) Conduit support

External kicker

Internal kicker (Form support)

(b) System formwork

(c) Pipe support (Hangers with threaded connectors)

(d) Membrane fixing

(e) Fastening of wooden batten to concrete

(f) Dry wall track fastening to concrete

Fig. 4.6 Application examples of direct fastening systems

4.5 PERFORMANCE INFLUENCING PARAMETERS

Just like other fastening technologies discussed in this book, the performance of direct fasteners is also dependent on installation and in-service parameters, in addition to its nail design features (e.g. tip shape, geometry, hardness etc.) as well as material properties. Some of these parameters are discussed in this section [24].

4.5.1 INSTALLATION PARAMETERS

The depth of penetration of the fastener, the quality and composition of the concrete it is installed in, its position w.r.t the concrete edge and each other, can all affect its pull-out strength.

(A) Concrete grade, type and condition

The concrete grade, the type of aggregate used, the type of cement etc. can all affect the “successful” installation of direct fastener, which in turn dictates the penetration depth and pull-out strength that can be achieved. Typically, the fasteners installed in higher strength concrete is likely to have higher pull-out strength than the one installed in lower strength concrete, provided all other parameters are same. However, it is difficult to achieve higher penetration depth in high strength concrete. Higher driving failure rate can be expected for long fasteners in high strength concrete. In short, the average depth of penetration at which the driving failure rate is minimum, decreases with increase in concrete strength.

Higher penetrability and compactibility of concrete ensure higher stick rate[3] i.e. successful installation. If the aggregate or the concrete itself is hard, it also makes it difficult to drive in the fastener i.e. increase driving failure rate. For example, it is easier to install a direct fastener in low strength concrete with limestone aggregate than granite aggregate near the surface, provided all other parameters are fixed. The fastener may be easily able to penetrate soft aggregate types like

[3] Stick rate indicates the percentage of fasteners that were driven in “correctly” to carry load.

limestone or sandstone. Whereas, presence of hard aggregates like granite near the surface of the concrete may deflect, or worse may bend the fastener. Excessive bending of fastener may in turn lead to fracture of concrete resulting in no holding strength of the fastener. Minor bending may lead to concrete spalling, but in this case the fastener may not lose its holding capacity as strength is usually tied to sintering bond at the point region of the fastener.

Whether the fastener is installed on the top or bottom surface of the casted concrete slab may also affect stick rate. For example, the rate of fastener stick rate is usually lower for the overhead application than floor. This is due to distribution of aggregate and its variation within concrete from top to bottom. Heavier and large aggregates tend to settle towards the bottom layer which makes it difficult to drive in the fastener. The concrete member thickness is also important. The concrete can spall off on the rear surface if minimum thickness recommended by the manufacturer is not observed. It is usually linked to the shank diameter of the fastener.

(B) Position of the fastener

The edge distance and spacing at which the direct fasteners are installed, has an influence on its strength as is the case with other fastening technologies. For example, when fasteners are installed in pairs or in rows along the concrete edge, its strength is heavily dependent on the spacing between them. The fastener that is placed too close to the edge is likely to have lower strength compared to the one installed far off. Whether or not the fastener is placed too close to each other or to the concrete edge can be inferred from the minimum values recommended by the manufacturer. The minimum edge and spacing values are recommended with the objective to maximize strength of a single fastener and prevent cracking or spalling due to installation of the fastener.

(C) Depth of penetration of the fastener

The pull-out strength of each fastener changes with its penetration depth, which is in turn limited by the driving failure rate. Typically, the fastener with higher depth of penetration is expected to have higher pull-out strength, provided it is not being influenced by other parameters. The pull-out strength and the fastener driving failure rate are both expected to increase with increase in depth of penetration of the fastener.

(D) Tool features and energy level

The tool type, quality and features play an important role in achieving high stick rate i.e. “successful” installation. The tools should be able to ensure perpendicularity as well as should be able to guide the fastener at the intended angle. If the driving energy is too high or too low, then also correct installation may not be possible. Most tools by reputed manufacturers offer the option to moderate the energy level based on the application and base material requirement.

4.5.2 IN-SERVICE PARAMETERS

The environment in which the fastener is installed, the temperature it is exposed to and the aging of concrete, may affect performance of direct fasteners.

(A) Environment

The environment encompasses the direct exposure condition that the direct fastener is subjected to. To protect against corrosion, the direct fastener needs to have adequate degree of protection. The fasteners are usually zinc-plated for protection against corrosion during transport, storage and construction (when not-exposed to weather or corrosive environment). The zinc-plating can hamper holding strength and therefore strict controls are placed during galvanization process to prevent excess zinc thickness. Exposure to corrosive environments necessitates use of direct fasteners with sturdier coating material. Corrosion and further damage caused due to it may reduce holding capacity of fasteners and should be given due attention. Usually manufacturers provide information on suitability of fasteners and recommended coating type based on corrosion resistance requirement depending on the application.

(B) Aging of concrete

For direct fasteners that heavily rely on friction as the primary holding mechanism, the effect of aging of concrete may be more pronounced as the concrete may relax in the area around the fastener. But, for the direct fasteners that are driven through powder or gas-actuated or battery-powered tools and for which the main holding mechanism is sintering and keying, the aging of concrete does not seem to affect the holding strength of such fasteners. Test results indicate very strongly that relaxation of the concrete has no detrimental effect on the pull-out resistance of such fasteners.

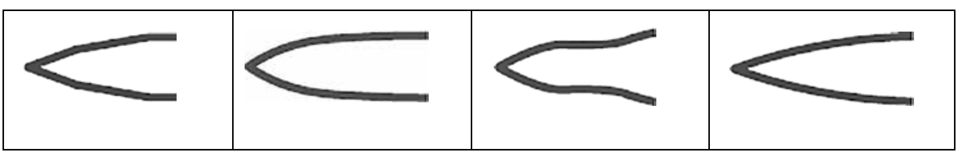
(C) Temperature

It is a known fact that steel tends to become more brittle with decrease in temperature. This applies to direct fasteners as well. In most of the indoor applications, direct fasteners are usually not subjected to very low temperatures during their service life. However, applications like fastening of liner sheets in a cold-storage warehouse, single skin cladding applications etc., expose the direct fasteners to low temperatures and is one of the critical parameters for product selection. The manufacturers provide a range of temperatures in which their products remain ductile and can be used.

When exposed to fire, the direct fasteners like nail usually fracture. Though, the concrete member is also weakened by fire exposure, the rate of deterioration of steel (in this case fastener) is much higher.

4.5.3 FASTENER FEATURES AND MANUFACTURING ASPECT

The nail design features, geometry and material can all affect the performance. The nail design features and geometry cover aspects like length, diameter, tip shape (see Fig 4.5), etc. Each of these parameters dictates how easy or difficult it is for the nail to penetrate the concrete base material. For example, a nail that has long conical shaped tip is expected to have the better ability to penetrate concrete that the other types shown in Fig 4.5. The hardness of nail is also another important parameter. If the nail is too hard it may break while driving in, upon hitting a hard aggregate.



(a) Cut type (b) Ballistic type (c) Stepped shank (d) Long conical

Fig.4.7 Common types of nail tip shape [24]

4.6 PERFORMANCE ASSESSMENT

In the Eurocode framework, there are no standards to test and evaluate performance of direct fasteners in applications, let alone design. Rather, it rests on the extended framework of European Assessment Documents (EADs) to evaluate the performance characteristics of the complete application. One such is EAD 200033 -00-0602 [26] for "Nailed Shear Connectors" that provide an alternative direct fastening solution to welded studs when fastening a composite metal deck to a steel beam, which is beyond the scope of this book. For other nails and threaded studs typically used in the concrete base material, much of the resulting performance data presently stems from the manufacturers' tests, which are tested within a framework of specific applications. For certain applications, failure in driving the fastener can be detected and these fasteners – threaded studs – can be replaced. For other applications such as fastening of wooden battens, detection is almost impossible. EADs relating to these applications do not currently exist [24].

Therefore, the design resistance is determined for: (a) failure loads that do not reflect failure in driving the fastener; and (b) failure loads considering a 20% fastener driving failure rate. The corresponding failure loads in tension and shear tend to display a large scatter with a coefficient of variation of less than 60%, which makes evaluating the characteristic resistances and safety factors unfeasible to use in design of these fasteners as single point supports. However, by adding multiple fastening points and making the system redundant – where the probability of failure is not based on a single but multiple fastening points – a design resistance value can be used to create a feasible solution that allows load transfer between the fasteners and mitigates the risk of failure caused by slip or failure of more than one. To plot the test data, behaviour of the fasteners is examined in two static systems: (1) A suspended beam permitting unrestrained beam flexure; and (2) a beam attached directly to the concrete surface showing restrained flexure.

The method to calculate the recommended tension and shear resistance loads based on the tests stems from the Monte Carlo method, where stochastic values taken from the overall "master" load distribution are attributed to individual fasteners in the system. The system is then checked to determine if the imposed line load can be supported or not. A sufficiently large number of simulations, which can include hidden setting failures, are performed to increase understanding of the system's failure probability. This allows design parameters to be established, where the failure probability is 10^{-6} in a system using five fasteners subjected to a uniformly distributed load with a failure criterion of either two edge or three central fastenings. The resulting load is expressed as a "recommended load per fastening". An example of such a fastening system is suspending a plumbing pipe (see Fig.4.8) using five ceiling hangers: due to the stiffness (EI) characteristics of the pipe, the permanent load (including its self-weight) must be redistributed to the remaining hangers in case two neighbouring ones fail. Here, fixing one hanger with one nail is sufficient. Alternatively, the pipe may not be stiff enough to redistribute the permanent load to the neighbouring hangers in the case where one fastener fails; in this situation, each hanger must be fastened with five nails. Ultimately, much rests on the level of redundancy built into the system.

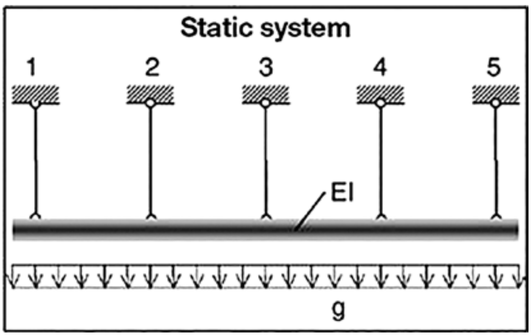


Fig.4.8 Example of suspended pipe fastened to concrete using direct fasteners [24]

4.7 SELECTION CRITERIA

It is important to select the right tool along with the right fastener for a given application. The tool selection is dictated by regulations as well as restrictions at job site. For example, powder-actuated tools cannot be readily procured by any contractor due to the restrictive regulations. There may be restriction on use of gas canisters in some job sites as well, in such cases battery-powered tools could be an option. Tool selection is also dependent on the application and compatibility with nail suitable for a given application. The selection of fastener itself depends on the application (e.g. attachment thickness, material to be fastened, base material parameters, environmental conditions etc.), the load requirement, the product performance data (e.g. holding capacity, stick rate etc.) as well as the parameters discussed in Section 4.5. The manufacturer supplies information on the application range and load capacity of a specific product [24].

4.8 INSTALLATION AND INSPECTION

In case of direct fastening systems, ensuring operator or installer safety is paramount as chances of injury are higher while operating direct fastening tool if not trained properly. The installation should be carried out only by trained and certified professionals to ensure safety as well as correct installation. The operator should be equipped with all necessary safety gears like googles, hear protection etc. Some manufacturers also provide additional safety features like “drop-firing safety”, “trigger safety”, “contact pressure safety”, “unintentional fire safety”, etc. to ensure that the tool doesn’t misfire without intent or on drop. In case of powder-actuated tools, the cartridges are subjected to additional scrutiny as per EN 16264 on “Pyrotechnic articles – Cartridges for Powder Actuated Tools”. Similarly, gas cans for gas-actuated tools are also subjected to additional scrutiny as per various standards. Some direct fastening tools are also equipped with features that minimize noise or vibration to ensure operator safety and comfort.

To ensure correct installation, the steps recommended by manufacturer should be adhered to. The installation steps may vary from manufacturer to manufacturer. As most types of direct-fasteners don’t require pre-drilling, the installation procedure is usually simple. Some general installation steps are illustrated in Fig. 4.10. The very first step would be to identify safe fastening point. The direct fastener should not hit or penetrate embed reinforcement bars, pretensioned cables etc. After a safe fastening point is identified, the tool is then loaded with propellant or powering source and the fastener. The tool is then pressed against the base material surface and the trigger is released to drive in the fastener.

As discussed earlier in this chapter, it may not be possible to drive in the fastener “successfully” at times depending on concrete aggregate hardness, penetration depth etc. There are several tips and tricks to reduce driving failure rate. Most of the methods revolve around the following two – reduction of concrete tensile stresses near the surface or delay the effect of these stresses. For example, reducing the fastener length (i.e. using shorter nails) may reduce the magnitude of the stresses. Other options like increasing the energy level or switching to a higher performing nail (e.g. thicker or with better performing geometry), can also help increase the stick rate.

The installation details like installer name, supervisor name, installation date, product details etc. should be recorded and maintained for inspection. Some manufacturers provide plastic guides to check whether or not the nail's stand-off on concrete is within acceptable range (see Fig. 4.9). If necessary, some types of direct fasteners can also be tested onsite to check the installation quality.

Fig. 4.9 Example of nail stand-off measurement check using a guide

Direct fastening technologies are one of the innovative technologies in the market that cater to the pressing construction needs for productivity and flexibility. Features like cordless, dust-free, vibration free, etc make it an installer friendly tool. Features like speed of installation and ease of use help boost productivity and reduce installation time.

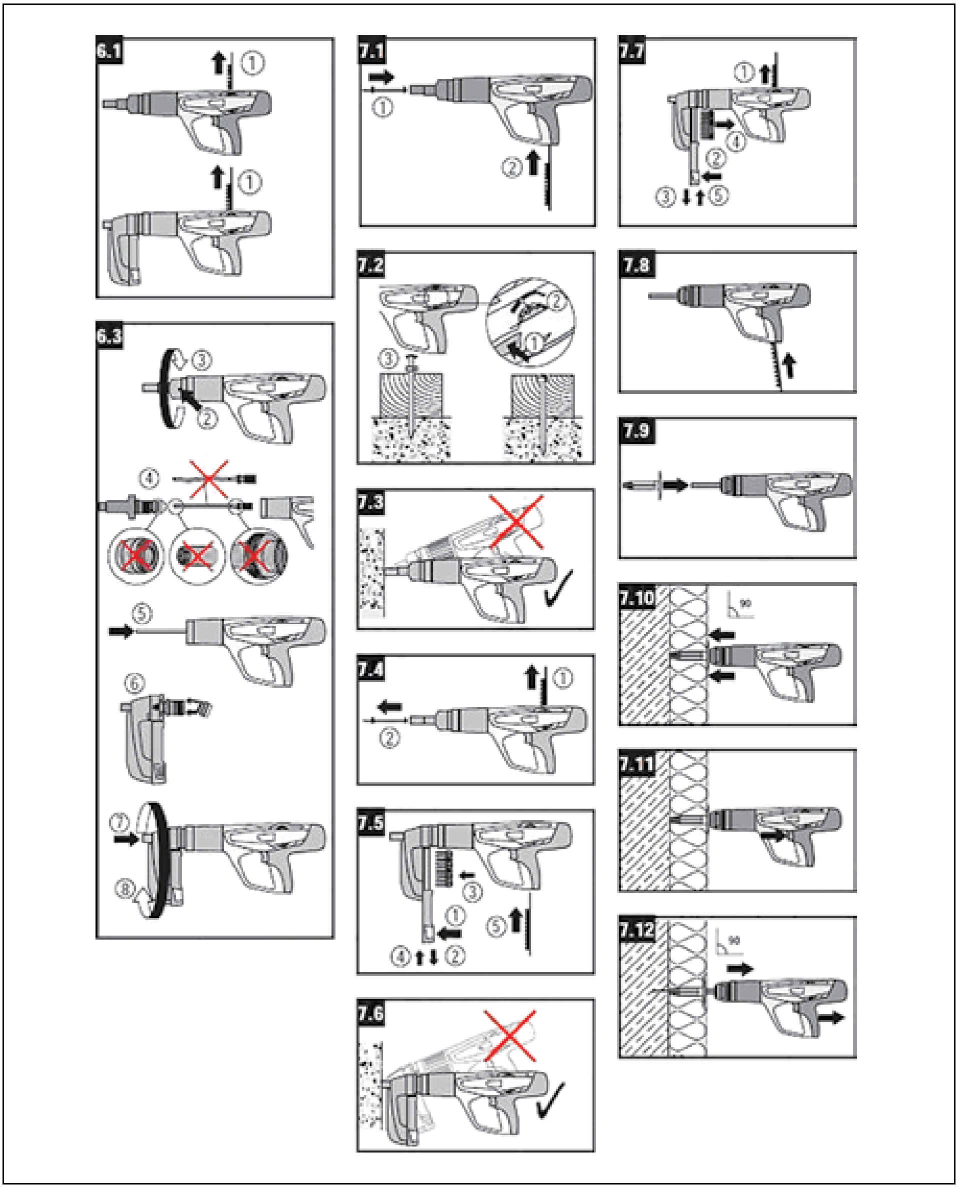


Fig.4.10 Typical steps for installing direct fasteners [27]

CHAPTER 5

Post-Installed Reinforcing Bar Systems

5.1 OVERVIEW

The goal of this chapter is to provide a well-rounded overview of post-installed reinforcing bars (rebars) with adhesive/mortar systems to monolithically connect reinforced concrete members, i.e. to form concrete-to-concrete connections (see Section 1.2.1(B)). This has fundamental differences to connecting a steel member to an existing concrete one, which is what we have discussed in the previous chapters.

Post-installed concrete-to-concrete connections offer greater construction flexibility as the need to chip away or demolish the existing structure does not arise and the design is very similar to cast-in concrete-to-concrete connections. These connections appear in all sectors of construction industry and in the form of three distinct applications:

- Non-contact lap splices (see Fig. 1.8(a)),
- Starter bar (or structural joint) applications (see Fig. 1.8(b)), and
- Shear dowel applications (see Fig. 1.8(c)).

Although these connections are not new to the construction industry, the solutions used today generally rely on combining inorganic (e.g., cementious grouts) or organic (e.g. epoxy based) two component mortars together with a reinforcing bar.

We begin this chapter by discussing the main differences between connections formed using post-installed anchors and post-installed reinforcing bars. As both post-installed bonded anchors and post-installed reinforcing bars make use of adhesive property of a mortar to form the connection, it is normal for a beginner to mix the two. Therefore, it is important to understand the difference between them before going into further detail of post-installed reinforcing bar systems.

We then discuss different parameters that may influence performance of post-installed reinforcing bar systems before reviewing different standards available for assessing performance and design of post-installed reinforcing bars. The link between the two is clearly defined and the importance of performance evaluation is emphasized. We then define the problem statement and discuss the design philosophy. Conventional design approach is discussed next in context of Indian

Standards and explained using illustrative design examples. Special literature for design of post-installed reinforcing bar connections not addressed by any standards is discussed next. The design approach that may be adopted to address fire design requirements is also briefly discussed. Lastly, installation and inspection aspect to ensure safety of connections formed using post-installed reinforcement bar systems is also presented.

5.2 BASICS OF POST-INSTALLED REINFORCING BAR CONNECTIONS

Post-installed reinforcing bar connections are formed by inserting a rebar in a pre-drilled hole injected with adhesive/mortar after proper cleaning of the drill hole based on the manufacturer's instructions. The load carrying mechanism is via bond between the rebar, mortar and lateral surface of the drill hole in the existing concrete member. These systems have a wide variety of applications. The main categories, namely, Non-contact lap splices, Starter bar (i.e. structural joints) applications and the Shear dowel applications (i.e. concrete overlays), are illustrated in Fig. 1.8. Each of these applications have similar influencing parameters but differ in design methods. [Note - Classification on basis of adhesive installation technique has already been discussed in Section 3.2.2.]

Lap splices enable the extension of existing structural members such as beams, columns, slabs, walls, and foundations, while transferring the tension loads between adjacent bars through local compressive struts and hoop stresses in the concrete directly surrounding the spliced reinforcement. An example of non-contact lap splice connection formed using post-installed rebar is shown in Fig.5.1(a).

Starter bars enable connection of new members perpendicular to existing ones, such as a new column or shear wall from foundations, a new beam from existing column, etc. as illustrated in Fig. 5.1(b). The starter bars are straight and perpendicular to the primary reinforcement of the existing concrete member in which they are installed. Typically, they resist tension forces through the global strut-and-tie model while ensuring that there is no direct tension induced in the concrete. These provisions of EN 1992-1-1 [14] rely on this principle. In cases, where the global strut-and-tie model is not applicable (i.e. confined support conditions are not ensured), potential formation of concrete cone breakout should be considered. For such cases the provisions of the EN 1992-4 [6] can be followed for the design of the connection. EN 1992-1-1 [14] has a special provision for end anchorage rebars under compression, where it is assumed that the rebar will share the compressive load with concrete under similar strains ($\varepsilon_{concrete} = \varepsilon_{steel}$). An additional detailing recommendation is provided in Section 5.6.

Shear dowel applications are commonly used to increase the thickness or cross-sectional area of the existing concrete member by adding a new concrete layer (or jacket) using shear dowels as anchors or to ensure monolithic shear dominated connections between reinforced concrete members (e.g. new corbel in existing column or wall). In this application, the shear-friction resistance relies mainly on the friction and dowel action and depends mainly on the roughness at the interface as well as on the density and embedment length of the connectors. It is a very common application in bridge retrofitting where the unique requirements for enhanced corrosion, extended design life, additional shear-friction, fatigue, seismic loads etc. are catered to using post-installed shear connectors.

a. Wall-to-wall connection

b. Column extension

c. Slab-to-column strengthening

Fig.5.1. Typical connections formed using post-installed reinforcement bar systems.

5.2.2 COMPARSION OF CAST-IN AND POST-INSTALLED REINFORCEMENT BAR CONNECTIONS

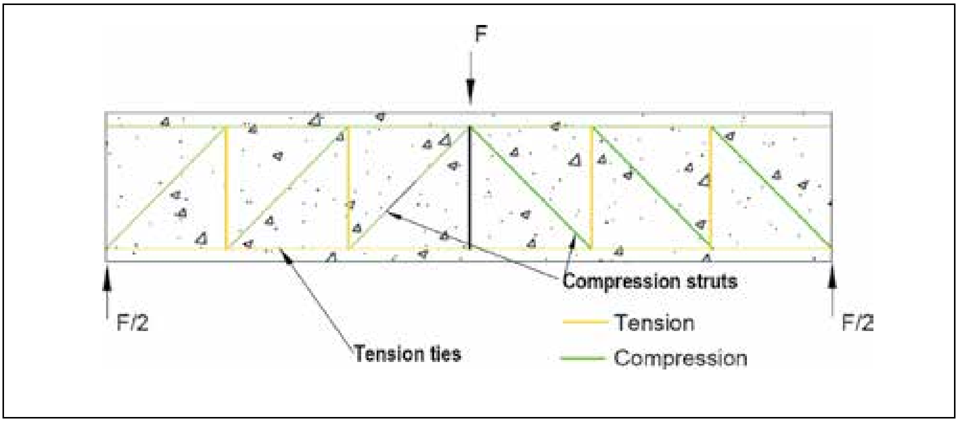
Whether cast-in-place or post-installed, all reinforced concrete connections have comparable behaviour, which means that both connection types contain numerous bars placed close to each other and follow the provisions of a design code such as EN 1992-1-1 [14]. Consequently, they can be designed with the same design logic provided certain prerequisite conditions are met. In short, there are significant overlaps between cast-in-place and post-installed reinforcement connections but there are some differences as well between these two.

It is important to note that the adhesive/mortar used in post-installed concrete-to-concrete connections must uniformly transfer the tension between the rebar and the concrete to ensure monolithic behaviour Any mortar system used to embed in post-installed rebar into existing concrete should exhibit stress redistribution in a manner similar to cast-in-place reinforcement. If confined support conditions are ensured, the potential causes of failure can occur through: (a) concrete splitting; (b) yielding of the steel; or (c) pull-out or loss of bond between the rebar and concrete (for cast-in-place) and between the rebar, adhesive mortar, and/or concrete (for post-installed). The “splitting” failure is characterized by extraction of the rebar and formation of radial cracks, while in the case of “pull-out” the concrete around the bar remains undamaged.

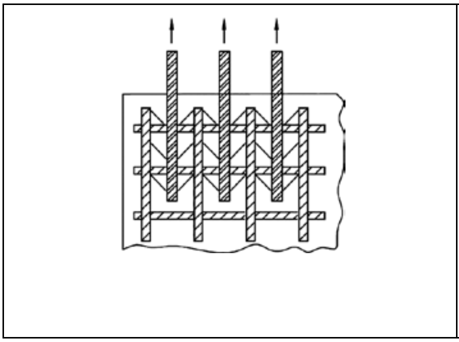
Setting aside the failure through steel yielding, bond and splitting failures of laps or anchorages in joints are governed by the term “confinement”, which is the ratio between the concrete cover around the rebar and the rebar diameter. When this ratio is sufficiently high, typically larger than 3-4 times the rebar diameter, failure arises due to bond-slip or shearing of the concrete on the surface along the tops of the rebar ribs for cast-in-place and either between the adhesive-concrete or adhesive-rebar interface for post-installed. This confinement ratio is low when the rebars are positioned very close to the concrete surface or are closely spaced, potentially causing splitting cracks and, in the worst case, spalling of the concrete if the stress in the reinforcement is too high. Since splitting failure is typically very brittle, provisions in design standards are formulated specifically to mitigate this risk [27].

In case of cast-in-place reinforcement, the concrete’s internal local – member-level – struts resist applied compression forces and tension is resisted by longitudinal reinforcement, the lack

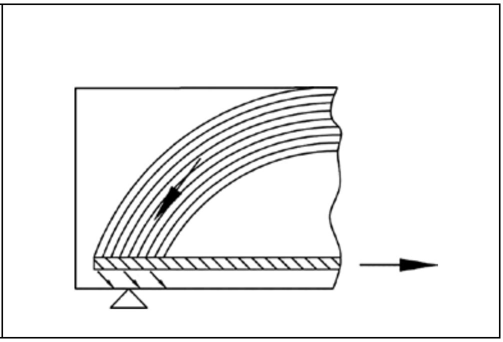
of which causes flexural cracking in the concrete and displacement beyond acceptable limits. Naturally, post-installed reinforcing bar also follows this logic. Fig.5.2 illustrates how EN 1992-1-1 [14] uses the truss analogy to describe a structural joint, where compression in a member is transferred through global diagonal struts and tension through longitudinal reinforcement and ties. For splices, however, this strut-and-tie model is instead on a local level, where monolithic behaviour is ensured by local compressive struts originating from the post-installed spliced reinforcement as illustrated by Fig.5.2(b).



(a) Truss analogy of a reinforced concrete member;
balancing of bond stresses transferred by (b) and (c)



(b) Local Struts for splices



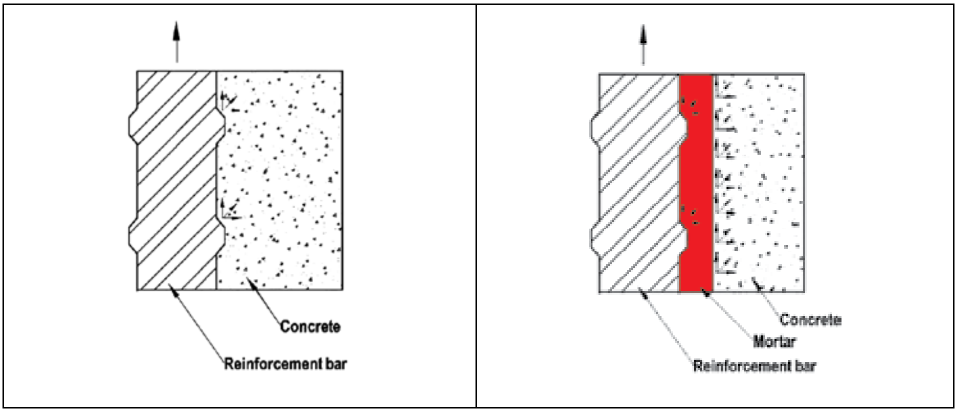
(c) Global struts for joints

Fig.5.2. Load transfer mechanism of reinforcement bar
(concrete-to-concrete connections) [28]

The term “bond” in post-installed reinforcement denotes the force transfer and interaction between the concrete, adhesive, and reinforcement; for cast-in-place reinforcement, there is no intermediary adhesive layer between the concrete and reinforcing bar, but the load-transfer principle remains the same as illustrated by Fig.5.3. In both cases, bond influences the performance of reinforced concrete structures in many ways:

(a) at the serviceability limit state, bond affects the curvature, stiffening caused by tension, as well as both the width and spacing of transverse (or flexural) cracks; and

(b) at the ultimate limit state, it is responsible for the resistance of end anchorages and lapped joints of reinforcement as well as influencing the rotational resistance of plastic hinge regions [27].



(a) (b)

Fig.5.3. Schematic representation of the load transfer mechanism of (a) cast-in-place and (b) post-installed reinforcement bar [29]

5.2.2 DIFFERENCE BETWEEN POST-INSTALLED REINFORCEMENT BAR AND BONDED ANCHOR CONNECTIONS

Post-installed bonded anchors are meant for steel-to-concrete connections, whereas post-installed reinforcement bar systems are used to form concrete-to-concrete connections. The confusion arises because both make use of similar technology, i.e. a fastening element embedded in adhesive in a hole drilled in hardened concrete to transfer/ carry load. In this section, the differences between the two systems is clearly illustrated.

(A) Load transfer mechanism:

As explained in Section 3.6, post-installed anchors rely on the concrete's tensile resistance in the anchorage region to transfer the applied tension and shear loads from the baseplate through the anchors and into the concrete substrate through bonding (see Fig.3.6). The post-installed anchor model assumes no transfer of shear due to friction between the baseplate and concrete substrate. Thus, bonded anchors can be subjected to tension, shear, or a combination of both.

On the other hand, post-installed reinforcement bar does *not* rely on the concrete's tensile resistance such that only tensile stresses are transferred into the concrete substrate, if the design provisions of EN 1992-1-1 [14] are followed. As such the underlying assumption is that they are loaded in pure tension, with the forces transferred through local or global struts just like their cast-in counterparts as explained in Section 5.2.1. Correct design and detailing of post-installed reinforcement ensure that the reinforcement can yield to mitigate the risk of brittle failure. In case of post-installed reinforcement bar connections, shear is resisted by the interface between the existing and new concrete and not through "shear resistance" in the manner of an anchor.

(B) Concrete cover and pertinent failure modes

As discussed in Chapter 3, minimum edge distance and spacing of post-installed anchor is product dependant. The values are spelled out in product assessment document like European Technical Assessment (ETA) should be observed. Whereas, post-installed reinforcement can follow the same edge distances, or concrete cover, as cast-in-place rebar based on exposure conditions such as those provided in reinforced concrete standard like EN 1992-1-1 [14] for instance, but also must adhere to the requirements given in product qualification or "Performance Assessment Document".

Thus, the resistance of post-installed reinforcement bar systems under tension loading is limited by the splitting resistance of the concrete, characterised by splitting cracks that form along the concrete surface parallel to the rebar in a manner illustrated by Fig. 5.4. The splitting cracks arise due to hoop stresses that develop along the length of the rebar. Consequently, the potential causes of failure of post-installed reinforcement bar are restricted to: (a) concrete splitting; (b) yielding of the steel; or (c) pull-out or loss of bond between the rebar, adhesive, and/or concrete. Whereas, concrete cone breakout is also considered for post-installed bonded anchors subjected to tension, in addition to other failure causes as discussed in Section 3.6

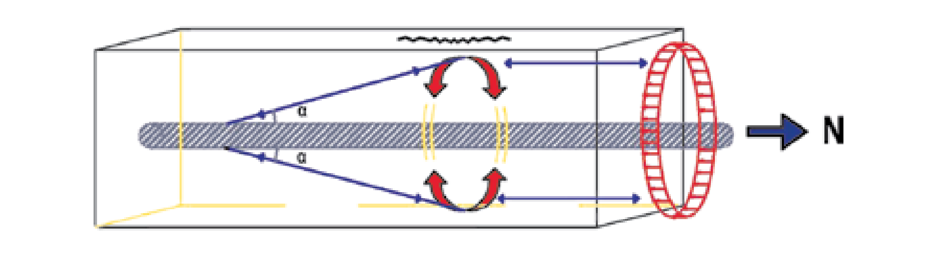


Fig.5.4. Rebar under tension showing splitting cracks [27]

(C) Expected state of concrete in-service

The concrete is assumed to be "cracked" for anchor design as discussed in Section 3.3.2(D), unless it can be proven that the fastener with its entire embedment depth is positioned in a region of uncracked concrete under characteristic loading at the serviceability limit state; in essence, no cracks are formed parallel to the anchor.

On contrary, as mentioned in Section 5.2.1, post-installed reinforcement bar is designed to prevent flexural cracks. As such, the cracks in a post-installed reinforcement system will be perpendicular to the rebar and any formation of splitting cracks parallel to the reinforcement can be avoided by providing sufficient cover and percentage of steel reinforcement. Therefore, the concrete can be considered as "uncracked" for static loading for post-installed reinforcement bar systems. However, for seismic situations, the concrete may be considered as "cracked" for post-installed reinforcement bar systems also – in line with the cracked cross-section concept of cast-in rebar – where the tension forces are transferred across the cracks by the reinforcement.

(D) Design philosophy

The fundamental difference in post-installed bonded anchors and reinforcement is the assumption of rebar development. Typically, reinforcement is embedded to a depth that enables the yield strength to be developed using the development and splice length provisions of the design standard. This means that the design engineer will calculate the area of reinforcement and the *development length* (in *mm*), which is usually between 20 to 60 bar diameters. This is markedly different from post-installed bonded anchors that are limited to 20 bar diameters in embedment and the design engineer checks the *resistances* (in *kN*) of the anchor or anchor group against the loading.

5.3 PERFORMANCE INFLUENCING PARAMETERS

Several parameters influence behaviour of post-installed reinforcing bars. The performance of post-installed reinforcing bars is also evaluated and documented according to a standardized "Performance Assessment Document" issued by Assessment Bodies on the basis of experimental evidences obtained in independent and accredited laboratories. This document provides guidance for evaluating the complete post-installed rebar system – the reinforcing bar, the mortar together with the drilling, cleaning, injection systems – under a prescribed set of conditions to allow assessment of different installation and in-service parameters. The assessment typically revolves around the concept of equivalency of performance between cast-in and post-installed reinforcing bars. Again, product standardisation is not possible as the mortar's chemistry is unique to each manufacturer and their manufacturing processes and tolerances are typically patented. This means that no two mortars can be considered the same unless they are tested in the same manner and exhibit same performance within an acceptable scatter range. This evaluation is critical as post-installed concrete-to-concrete connections depend on the mortar's ability to transfer the stress into the concrete and its robustness to withstand environmental effects that might occur during its service life, in precisely the same manner as if the connection were cast-in. For the structural engineer, selecting the right approved solution for post-installed rebar application will not prove cumbersome if the key influencing parameters are well-understood. In broad terms, this section addresses these parameters.

5.3.1 INSTALLATION PARAMETERS

The post-installed rebar's installation technique, the type of base material, and the tolerances in the positioning of rebar can influence the performance and load-displacement behaviour of the post-installed rebars. Any variation from standard installation procedure recommended by manufacturer may also negatively influence its performance. The effect of these parameters may vary depending on the anchor type and from product to product.

(A) Concrete grade, type and condition:

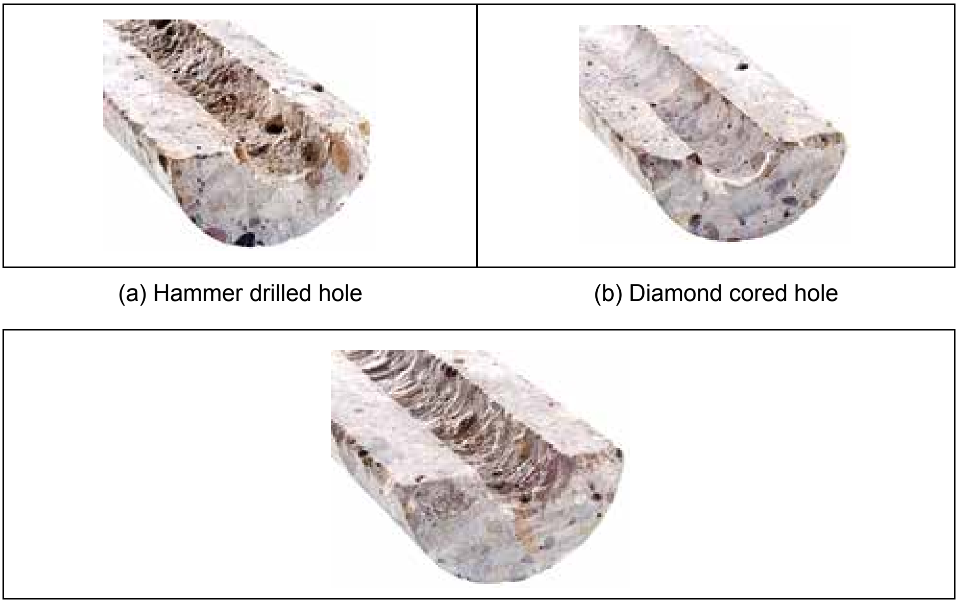
Grade of concrete influences the bond, with higher grades resulting in a higher tensile bond stress of not just the concrete, but also the adhesive used. This results in shorter anchorage lengths required to effectively transfer the loads to the base material. However, it shall be noted that the beneficial influence of increase of concrete compressive strength on the bond resistance of post-installed rebars is usually lower than for cast-in bars. The equivalency in performance between a post-installed rebar system with cast-in bars also in high strength concrete might not be given. More information can be found in the relevant “Performance Assessment document”. As anchors, post-installed rebars too are impacted negatively if the existing concrete is compacted poorly, contains honeycombs, or contains ill-suited aggregates, all of which must be considered during the design phase. As an example, an assumed M25 grade of well-compacted concrete with suitable aggregates and without honeycombs considered during the design stage must correspond to that found at the site.

The performance of post-installed rebars is dependent on the adhesive which in turn is dependent on the condition of concrete – dry, wet or submerged. If the surface of the drilled hole is wet, it will hamper bonding at mortar-concrete interface and thereby reducing its capacity. Only systems assessed as per “Performance Assessment Document” and found to be suitable for use under such conditions should be used.

Dry and wet conditions – Wet concrete implies either water-saturated or water-filled holes. As discussed in previous chapter, cementitious and hybrid vinyl-ester mortars are unsuitable for water-filled holes and only resin epoxies suffice here. Bond performance of the suitable mortars might reduce due to presence of water as well as their curing times may increase, which are typically twice as long as when compared to dry concrete.

(B) Condition of drilled hole surface:

Similar to post-installed anchors, the drilling method employed can influence the behaviour and resistance of post-installed rebars. Starting with the most commonly used, these are: (a) hammer drilling (see Fig.5.5(a)) – producing a rough surface within the drilled hole that aides in creating better adhesion and mechanical interlock; (b) Diamond coring (see Fig.5.5(b)) - commonly used when the reinforcement in the existing member can be cut. Diamond cored holes produce smoother drilled holes that lack the necessary roughness essential for creating the mechanical interlock, thereby negatively impacting the performance of the post-installed rebar system. Diamond cored holes also are coated with a fine dust layer resulting from drilling, further hampering the effectiveness of the bond, as illustrated in Fig.5.6. While most adhesives/mortars are tested only with hammer-drilling, certain others use methods and accessories that provide a performance equivalent to hammer-drills (e.g., roughening tool, see Fig.5.5(c).



(a) Hammer drilled hole

(b) Diamond cored hole

(c) Diamond cored hole roughened using proprietary systems (e.g. roughening tool)

Fig.5.5 Different types of drilling techniques used for installing post-installed rebar

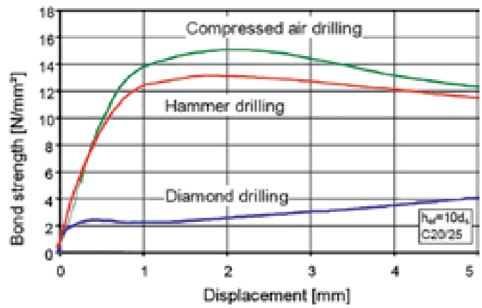
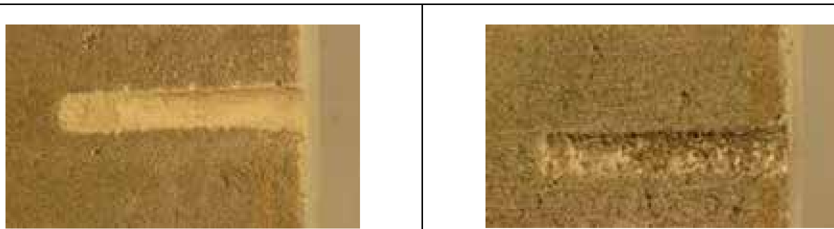


Fig.5.6 Example of the influence of drilling method on the bond-displacement behaviour of a post-installed reinforcing bar (with embedment of 10*d*) installed in cleaned hole in low-strength concrete [29]

Another key component relates to the condition within the drilled hole: the debris resulting from the selected drilling method sticks to the walls and consequently creates a barrier to the adhesive/mortar bond with the existing, hardened concrete surface (see Fig. 5.7). An insufficient drill hole cleaning can result in significant bond reduction and consequent potential failure. Allowed methods of cleaning are included in Performance Assessment Document, such as hollow drill bits attached to vacuum cleaners, blowing out the debris using compressed air, or a manual blow-out pump combined with a metallic wire brush. For diamond drilled holes, roughening the surface and flushing out the debris with water is recommended. Currently, proprietary systems exist in the market that entirely remove the need to clean the hole or, alternatively, are not impacted by the cleaning process and provide a significant edge over their “normal” counterparts by reducing installer error with respect to cleaning.



(a) Uncleaned hole with debris (b) Cleaned hole

Fig.5.7. Illustration of uncleaned and cleaned hammer drilled hole

(C) Dimension, position and orientation of the anchor:

As with their cast-in counterparts, load-carrying capacity of post-installed reinforcing bars is influenced significantly with bar diameter and anchorage length, with larger bars and longer anchorage lengths transferring larger loads to the base material. Other factors, such as concrete cover and spacing, play an equally important role, with concrete cover discussed earlier in Section 5.2.2(B). Spacing is vital to the performance of post-installed reinforcement and a significant difference to cast-in-place reinforcement is the centre-to-centre bar spacing, which cannot be less than either 5 times the rebar diameter, d_s or 50 mm for post-installed rebars. The aforementioned minima for concrete cover and spacing avoids spalling of the concrete during drilling and often will exceed the minima recommended by provisions in the design standards for cast-in-place reinforcement. Potential deviations from drilling perpendicular to the concrete base material are permitted depending on the drilling method and overall drill length and must be considered in the design of post-installed concrete-to-concrete connections. An additional consideration is the orientation of the rebar based on the orientation of the connecting member, which can be installed downwards, sideways, and upwards / overhead. As with post-installed anchor installation, overhead installations are fraught with difficulties in injecting the adhesive, which tends to drip out of the hole and causes air pockets or voids that hamper the system performance. The structural engineer is advised to consider methods that ease the installation process in such scenarios.

(D) Hole drilling diameter/ annular gap:

As mentioned in Chapter 3 dealing with post-installed anchors, this annular gap if filled with adhesive mortar, with larger or smaller drill diameters than those recommended in the manufacturer's printed install instructions (MPII), adversely impact the connection performance. These drill diameters result from those tested in the assessment procedure. Larger-than-recommended diameters potentially disrupt the bond between adhesive and the existing concrete due to shrinkage as the mortar cures, while smaller-than-recommended diameters may not allow the adhesive to uniformly coat the rebar surface. The structural engineer can further reinforce the correct installation procedures in his or her detailed drawings.

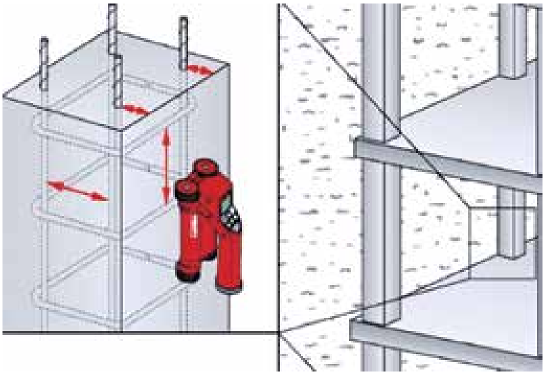
(E) Installation temperature, curing time and working time:

Ambient base material temperatures have a significant effect on the curing and working time. The curing time refers to the time from the beginning of the chemical reaction to the its hardening, thereby making it ready for loading to the desired capacity. On the other hand, working time refers to the time between injection of the adhesive/mortar and start of the hardening cycle. Longer working times are better suited to large diameter bars (25 mm and above) in combination with deep holes (anchorage lengths greater than 350 mm). Inversely, systems with accelerated curing times which increase installation efficiency are better suited for small to medium bar diameters having shorter anchorage lengths. Lower ambient temperatures increase the difficulty in injecting the adhesive due to higher viscosity and, although result in longer working times, also increase the curing times, which can set back the construction schedule.

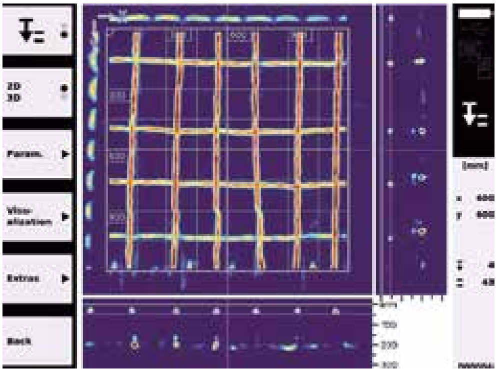
The curing and working times are a result of adhesive chemistry tailored by the manufacturer – making them unique to the adhesive – along with performance in specified temperature ranges outlined in the “Performance Assessment Document”; ultimately, both times are directly proportional to higher ambient and base material temperatures.

(F) Additional installation parameters:

For the sake of installation efficiency and avoid decreasing the life of the drill bit, systems that enable the rebar contained in the existing concrete member to be detected (as illustrated in Fig. 5.8) are highly recommended to avoid the hitting the existing rebar and drill unnecessarily into the concrete, which creates cavities that may negatively impact the stress distribution in the existing concrete member.



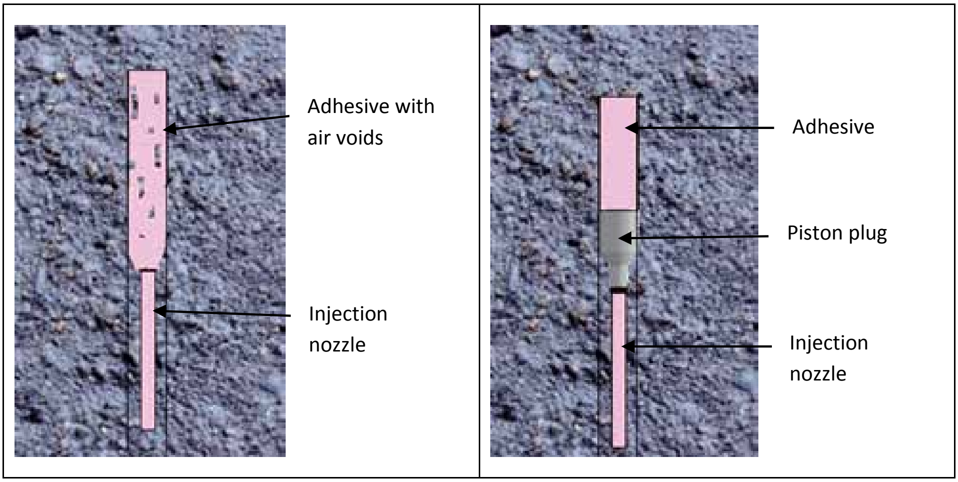
(a) Detection of existing reinforcement
in concrete prior to drilling



(b) Example of report highlighting rebars detected during scanning of concrete

Fig.5.8 Illustration of scanning of concrete using detection systems prior to drilling

To achieve an installation as close as possible to the ideal design conditions, the adhesive must be injected effectively into the drilled and cleaned hole while ensuring that no air voids or pockets are formed as illustrated in Fig. 5.9(b). This is not an easy task and one which cannot be undone if executed incorrectly as illustrated in Fig. 5.9(a). The maximum installation length depends on bar diameter, dispenser type, and availability of extension tubes from the manufacturer. Tested proprietary systems exist that enable adhesive injection without the formation of undesirable air voids, while also offering savings in adhesive/mortar consumption, and increasing installer productivity (e.g., piston plug).



(a) Adhesive injection in deep hole with undesirable air voids

(b) Adhesive injection in deep hole with tested proprietary systems (e.g. piston plug)

Fig.5.9 Illustration of injected adhesive with and without air voids

5.3.2 IN-SERVICE PARAMETERS

In-service parameters relate to the conditions to which the post-installed reinforcing bar is subjected during its lifecycle, which are: aggressive or corrosive environments, long-and-short-term elevated temperatures, sustained loads, freeze-thaw cycles and the expected state of concrete during this cycle. All of these parameters can affect the performance of post-installed reinforcing bar systems.

(A) Environment:

Typically, the expected exposure conditions for cast-in and post-installed reinforcement directly influence the concrete cover so that no part of the reinforcement is exposed. Particularly aggressive environments, such as industrial or marine structures, and exposure to freeze-thaw cycles require additional precautions as they degrade the performance of the post-installed adhesive and reinforcement and must be accounted for during the design phase.

Corrosion of the reinforcement negatively impacts the residual capacity of reinforced concrete, with the effects of corrosion in cured concrete differing from the effects under curing. For normal conditions, passivation of the rebar surface prevents corrosion of cast-in-place reinforcement and, for post-installed variants, the adhesive can provide an additional layer of corrosion resistance (typically true for epoxies) or a passivation layer (usually cement based systems). However, when concrete undergoes carbonation, the decreased pH can lead to early-stage corrosion, with faster corrosion rates (pitting corrosion) observed if the concrete contains chlorides. Surface corrosion does not negatively impact the bond and at higher corrosion levels, confinement from shear ties positively influences the residual bond strength [30]. Overall, post-installed reinforcement must exhibit similar or better corrosion rates to cast-in-place bars in the same concrete, provided the adhesive surrounding the rebar is void-free.

(B) Temperature:

The short- and long-term "in-service" temperature of the concrete base during the structure's service life can alter the adhesive's performance. Typically, the bond performance of the adhesive reduces at higher temperatures. To address this aspect, different temperature ranges are considered during the assessment of post-installed reinforcing bar systems. A temperature range is characterized by a maximum short-term and a maximum long-term temperature. The structural engineer is advised to pay due attention to this temperature range specified in "Performance Assessment Document" and corresponding performance during design.

In the case of fire exposure, most adhesive mortars are adversely affected and suffer a drastic and rapid loss of bond resistance under direct fire exposure. This is especially true to organic mortars. Mitigating the effects of fire can be achieved through increasing the concrete cover and/or the increasing the anchorage length of the rebar after a structural fire design is undertaken.

(C) Load duration:

As for post-installed anchors, this parameter is critical for adhesives used with post-installed reinforcing bars. Calculated as the sum of static and quasi-static loads divided by the sum of the overall loads, sustained loading of the adhesive leads to excess creep for post-installed reinforcing bar and can lead to premature failure. To mitigate this, assessment provisions in “Performance Assessment Document” must evaluate creep behaviour of the loaded rebar at normal ambient temperature and at the maximum long-term temperature, where the performance of the post-installed reinforcing bar must not be affected adversely by long-term temperatures within the service temperature range or by long-term temperatures up to the maximum long-term temperature. If the adhesive is not suitable for use under sustained load, the post-installed reinforcing bar may slowly lose performance over a certain period and fail.

(D) Expected state of concrete:

Although uncracked concrete can be safely assumed for most post-installed concrete-to-concrete connections subject to static loading, there are also situations (under static loading) where cracks may form parallel to the anchorage of post-installed reinforcement, as illustrated in Fig. 5.10. Consequently, and similar to bonded anchors, the design bond strength value for uncracked concrete is higher than that in cracked concrete for same concrete grade, diameter of reinforcing bars. This stems from investigations demonstrating that the bond resistance of cast-in-place reinforcing bars in cracked concrete is about 70% of the uncracked concrete value when the crack width is 1/60th of the rebar diameter[[31] and [32]]. However, if reinforcement is provided to resist transverse tension in the existing concrete member and the post-installed reinforcement is embedded sufficiently deep, the reduction in bond stress due these potential cracks is insignificant. For most assessed post-installed reinforcing bar systems, the bond resistance in cracked concrete is approximately 50% of the value in uncracked concrete. For certain post-installed reinforcing bars systems, however, the influence of cracks on the bond resistance may be smaller.

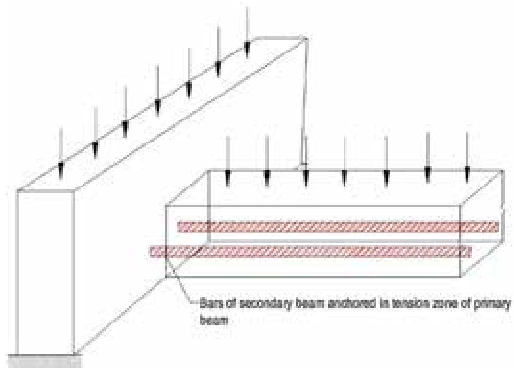


Fig.5.10 Transverse tension in the anchorage zone
causing potential cracks perpendicular to the post-installed reinforcement [27]

5.4 CODES AND STANDARDS

In the previous section, we discussed the main parameters that influence performance and load-displacement behaviour of post-installed reinforcing bars. It is essential to consider these influencing parameters in design of post-installed reinforcing bars also, as was the case for other fastening systems. This is addressed by using performance data of post-installed reinforcing bars for selection and design. As there are no national standards at present for testing, assessment and design of post-installed reinforcing bars, guidance has been sought from European standards and regulatory framework. However, comparisons have been made to Indian Standard IS 456 - “Plain and Reinforced Concrete – Code of Practice” [11] where applicable, for ease of understanding.

The concept of “construction products regulation (CPR)”, “European Assessment Documents (EAD)” and “European Technical Assessment (ETA)” explained in previous chapters apply to post-installed reinforcing bars as well. The various installation and in-service parameters are considered for testing and assessment of post-installed reinforcing bars as per applicable “European Assessment Document (EAD)”, which is essentially the “Performance Assessment Document”. The performance data in terms of characteristic values and installation parameters (e.g., reinforcing bar, adhesive, hole cleaning tool, printed manufacturer instructions etc.) determined based on this performance assessment is documented in “European Technical Assessment (ETA)” report. This performance data can be used by a structural engineer to design a concrete-to-concrete connection using post-installed reinforcing bars.

5.4.1 PERFORMANCE ASSESSMENT STANDARDS

The “European Assessment Document (EAD)” developed by “European Organisation for Technical Assessment (EOTA)” is used to test and assess performance of a post-installed reinforcing bars by a third-party accredited lab and approval body in Europe. Some of the European Assessment Documents developed for the post-installed reinforcing bars are listed below:

- EAD 330087-00-0601 – European Assessment Document on “Systems for Post-installed Rebar Connections with Mortar” [33]: applicable to post-installed rebar for use under static load conditions. This document has superseded TR 023 [34] on “Assessment of Post-installed Rebar Connections” which was used earlier.
- EAD 331522-00-0601 [Publication pending] – European Assessment Document for “Post-installed Rebar with Mortar under Seismic Action” [35]: applicable to post-installed rebar for use under seismic load conditions. The technical concept behind development of this EAD is briefly discussed in Genesio et. al [36].
- EAD 332347-00-0601 [Publication pending] – European Assessment Document for “Connector for strengthening of existing concrete structures by concrete overlay” [37].

Each EAD defines the product it is applicable for, its intended use, essential performance characteristics required to fulfil the intended use, test and assessment methods to determine essential performance characteristics, and approach for verification of constancy of performance. The post-installed reinforcing bars embedded with cementitious and/or polymer adhesives must exhibit equivalent or superior behaviour with comparable stiffness to cast-in-place reinforcement of equivalent diameter, embedment depth, edge distance, spacing, etc. The adhesive also must develop a uniform stress along the length of the rebar, as adhesives with higher stiffnesses may lead to zipper-like failures.

Tests such as those prescribed in EAD 330087-00-0601 [33] are conducted in configurations similar to cast-in-place reinforcement, in which pull-out control the behaviour. Table 5-1 lists the essential performance characteristics for “Mechanical Resistance and Stability” and “Safety in Case of Fire”, required for post-installed rebars as per EAD 330087-00-0601 [33].

Table 5-1 List of essential characteristics as per EAD 330087-00-0601 [33]

Essential characteristics for “mechanical resistance and stability” and “safety in case of fire” for post-installed rebar	
Mechanical Resistance and Stability -Characteristic resistance under static or quasi-static load:	Safety in Case of Fire”
- Bond strength of post-installed rebar	- Reaction to fire
- Reduction factor	- Resistance to fire
- Amplification factor for minimum anchorage length	- Bond strength at increased temperature

Table A.1 of EAD 330087-00-0601 [33] lists 18 different test series that the post-installed reinforcing bars shall be subjected to for assessing performance and to determine the essential characteristics under static load conditions. Reference tests in crack/ uncracked in different concrete grades, robustness in dry/wet concrete, correct injection etc. are examples of some tests that the post-installed rebar may be subjected to. Seismic qualification is selected as an additional option and is evaluated as per a different EAD as listed above. The key requirements verified from the EAD prequalification testing are the ability of the:

- Post-installed reinforcement to develop tension resistance equivalent to or greater than cast-in reinforcement when installed with normal concrete cover when accounting for all applicable influencing factors such as loading type (short-term, long-term, fatigue, or seismic), temperature range, and concrete cracking;

- Drilling method to achieve accurate and perpendicular holes for the maximum embedment length expected for the system connection;
- Injection system to inject the adhesive over the full length of the maximum hole depth without the introduction of air pockets and its suitability for overhead installation; and
- Post-installed reinforcement to exhibit corrosion resistance equivalent to or greater than cast-in reinforcement for the relevant exposure category.

The exhaustive test and assessment regime are carried out by third-party accredited body and an “European Technical Assessment (ETA)” for that product is issued. This assessment report verifies that the system will work for the intended purpose if installed correctly and using the specified tools. Systems that may be otherwise appropriate for anchoring applications do not necessarily fulfil the requirements for safe and reliable post-installed rebar installations.

5.4.2 DESIGN STANDARDS

In Europe, after performance of post-installed concrete-to-concrete connections is established to be on par with its cast-in counterpart via European Technical Assessment (ETA) under static and/or seismic loading, its design is permitted using the limit state philosophy that complies to EN 1992-1-1 [14] (for static), EN 1992-1-2 [14] (for fire), and/or EN1998-1 [15] (for seismic), which both have overlaps with IS 456 [11] & IS 13920 [38]. Irrespective of the design standard used, the underlying philosophy of reinforced concrete according to limit state design is usually intended to prevent brittle failure. Table 5-2 below summarises the earlier sections, illustrating the regulatory framework used in Europe to *qualify, assess,* and *design* post-installed concrete-to-concrete connections, while intending for the system to *behave* like cast-in-place reinforced connections.

Table 5-2 The regulatory framework used for qualifying, assessing, and designing adhesives used for post-installed concrete-to-concrete connections.

Post-installed concrete-to-concrete connections (post-installed rebar = cast-in-place rebar)			
	Static	**Fire**	**Seismic**
Performance assessment Document	EAD 330087-00-0601 ⬇		EAD 331522-00-0601 ⬇
Assessment report (pre-requisite to design)	"European Technical Assessment" for assessed adhesives ⬇		⬇
Design method	EN 1992 (i.e. Eurocode 2)		EN 1998 (i.e. Eurocode 8)

5.5 DESIGN PROBLEM

Typically, the design challenge comes with the structural engineer evaluating the anchorage length (in *mm*) needed to safely transmit the stress from the new member to the existing one. For a post-installed scenario, simply requiring yield is not a sufficient design statement as it does not consider the challenges of being able to fit a straight rebar with, for example, 20 mm diameter rebar in M25 grade concrete with 1000mm anchorage in a 600mm thick base material, nor does it consider higher minimum spacing requirements (≥5*d* or ≥ 50mm) for post-installed reinforcement that may require a detailing revision. Furthermore, a design scenario must identify at least the following information necessary for the design:

- **Connection type:** concrete overlay, lap splice, or structural joint;
- **Environmental, site, and exposure conditions:** for instance, a site located next to a marine or highly aggressive industrial plant requiring a two-hour fire rating;

- **The loading requirement and type of load:** such as a 150 kN-m design moment stemming from the 1.5(DL+LL) combination;
- **Serviceability requirements such as crack width and deflection criteria:** for instance 0.3 mm crack width and L/350 deflection limit for a building with 50 years' service life;
- **Strength assessment and dimensions of the existing concrete:** such as an M30 450x600 mm column;
- **Cover to the post-installed reinforcement, along with the spacing, diameter, and anchorage depth of the post-installed rebars:** for instance 75 mm horizontal and 50 mm vertical cover for a 16 mm diameter bar spaced at 135 mm centres and having an anchorage length of 350 mm;
- **Preference for any specific technology, such as a hollow:** drill and vacuum system; and
- **Roughening requirements for joint interface:** for instance, 3-6 mm amplitude.

5.6 DESIGN PHILOSOPHY

Rarely does an economic and feasible design of a post-installed concrete-to-concrete connection depend on complex theoretical analysis but is achieved instead by practical detailing considerate of the execution at site. The underlying principles of reinforced concrete do not change based on the applied load on the member or cross-sectional shape; these are:

(1) stresses and strains are related through material properties, including the stress-strain curves for concrete and steel;

(2) strain distribution must have compatibility with the distorted shape of the cross-section; and

(3) the resultant forces developed by the section must balance the applied loads to achieve static equilibrium.

The design procedure for most structures starts by designing for conditions at the ultimate limit state – that is, the resistance of the concrete member and the overall structure must exceed the applied loading – which then is followed by checks against disproportionate cracking or deflection of the member at the serviceability limit state.

Furthermore, at the ultimate limit state, a concrete member subject to bending must maintain a sufficient degree of ductility by gradually yielding the steel to avoid brittle failure, thereby enabling plastic hinge formation that permits moment redistribution, leading to a safe and economical structure. Linked to ductility is the potential inability for post-installed connections to *fully* develop straight bars to their required anchorage length in an existing concrete member of limited thickness. However, to overcome this, the design stress in the rebar caused by the imposed loads can be used instead of the bar's yield stress, making the design possible but does not guarantee ductility.

Another consideration relates to service life: with most structures designed to EN 1992-1-1 [14] for a life of 50 years, the adhesive too must not show signs of excessive creep deformation prior to this. EAD 330087-00-0601 [33] includes tests for service life up to 50 years. Similar service life requirements are required as per provisions in Indian Standards.

5.7 DESIGN

This section explains the design procedure for connecting two post-installed reinforcement connections: namely, joints and lap splices. To a considerable extent, the calculations in the design are not affected when compared to cast-in-place reinforcement – that is, the required cross-section area of steel to resist the design bending moment is not impacted. However, design engineers must pay special attention to detailing the reinforcement (concrete cover, spacing, substrate thickness, and so on) and specify in drawings the degree of roughening necessary in the existing concrete. To ensure a monolithic connection between the existing and new concrete members, post-installed rebar connections will develop partial fixity and will be required to resist bending. Consequently, it is advisable to detail the entire connection accordingly.

During the design phase, it is crucial to keep in mind that post-installed reinforcement only can be installed straight, unlike its cast-in-place counterpart that can be bent at any angle, and often leads to the requirement of a thicker substrate to accommodate the reinforcement to allow a feasible connection. As illustrated by Fig. 5.11 the design engineer must appreciate certain spacing requirements that are valid for both joints and splices:

- *Minimum* centre-to-centre spacing must be greater than either 50 mm and 5 times the rebar diameter *between two post-installed rebars*, which is higher than that mentioned in cast-in-place design standards such as EN 1992-1-1 [14] and IS 456 [11];
- *Maximum* centre-to-centre spacing *between two post-installed rebars* follows the same governing principles as cast-in-place rebar;
- *No minimum* requirement between *a cast-in-place and post-installed rebar* (limited by drilling); and
- *Maximum* spacing requirement *between a cast-in-place and post-installed rebar* cannot exceed 4 times the bar diameter.

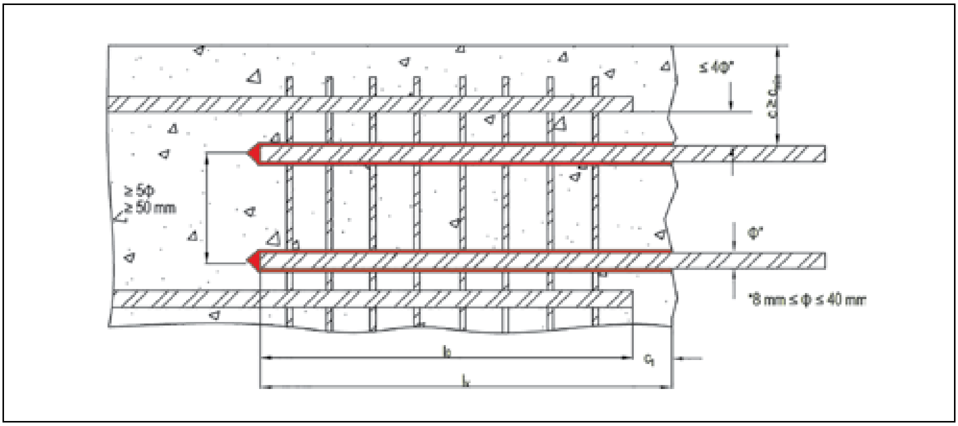


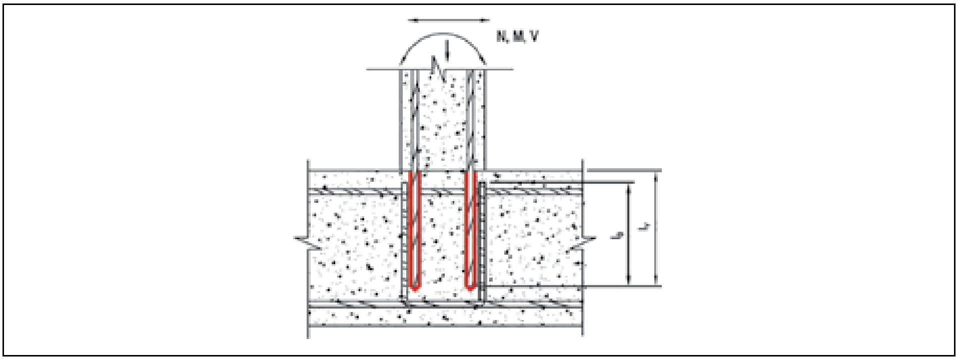
Fig. 5.11 Concrete cover & spacing requirements for post-installed rebar connections (φ = rebar diameter) [39]

The grade of concrete of the existing substrate, together with the confinement ratio, has a significant effect on the resulting installation depth: the bond stress of the adhesive is highly dependent on the bond stress of the existing concrete substrate. While it is not always possible to increase the strength of the existing substrate, the confinement ratio can be maximised by positioning the reinforcement in an appropriate manner and by using smaller diameter rebars where possible: here, it is highly recommended that the post-installed reinforcement is placed inside the existing reinforcement cage to minimise spalling during drilling and to provide additional concrete cover. There is a trade-off from increasing the confinement that stems from higher concrete cover, it will result in higher stresses in the bar generated from a smaller lever arm closer to the member's neutral axis. Higher tolerances must be considered for post-installed reinforcement due to the possible variations from the nominal rebar position due to imperfect drilling during the installation phase; which can negatively affect the performance of the intended system.

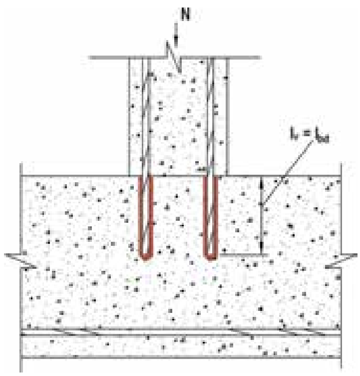
Since IS 456 [11] presently contains no provisions for confinement and does not have minimum anchorage length requirements, the design engineer must use his/her own judgement when justifying anchorage length to EN 1992-1-1 [14] provisions. In general, however, the underlying philosophy does not alter and in many instances of post-installed anchorages, it may be beneficial to use EN 1992-1-1 [14] to make the post-installed connections feasible. Of course, the adhesive used must be prequalified to the aforementioned EAD.

5.7.1 LOAD ANALYSIS

In line with the regulatory framework presented in Section 5.4.2, post-installed rebar connections that have been assessed and found suitable for use as per the EAD 330087-00-0601 [33] can be designed according to EN 1992-1-1 [14] (i.e. Eurocode 2). Some examples are illustrated in Fig.5.12.



(a)



(b)

Fig.5.12. (a) Overlap joint where the rebar is stressed in tension; (b) rebar connection for members stressed in compression, causing the rebar to be stressed in compression [21]

The structural engineer may design the connection as simply supported if the condition for partial fixity are not exceeded. As noted previously, the rebar must be embedded deep enough to develop the desired levels of stress and the objective of calculating and prescribing an anchorage length suffices this purpose. The design force in the rebar may be calculated using Eq. 5.1.

$$F_s = \frac{M_{Ed}}{z} + \Delta F_{td} + N_{Ed} \qquad \text{(Eq. 5.1)}$$

In Eq. 5.1, F_s is the acting tensile force on the rebar which is to be determined. M_{Ed}/z is the ratio of design bending moment (M_{Ed}) from the moment envelope (hogging or sagging) to the lever arm (z). ΔF_{td} is the tensile force to be anchored according to the shift rule in EN 1992-1-1 [14]. N_{Ed} relates to the axial or compression that must be added to or subtracted from the overall tensile force (F_s).

This expression stems from the curtailment rules from Section 9 of EN 1992-1-1 [14] and is known as the Variable Strut Inclination Method. Engineers not familiar with this method are advised to refer to the “Strut-and-tie” model for the design of concrete members for a detailed explanation [14]. In short, the truss-like model (for example see Fig. 5.13 relies on ties to resist the applied shear and assumes no direct contribution from the concrete's shear capacity, where the compression transferred through the concrete acts on the top chord and the diagonal compressive strut is allowed to rotate at an angle from 45° to 22° to the horizontal. This angle is denoted as θ and is inclination angle of the diagonal compressive strut (between 22° and 45° per [14], shortest at a beam's end support and largest at the midspan, see Fig. 5.13). The bottom reinforcement and vertical ties/stirrups represents tension. Although for most situations with uniformly distributed loads, the angle θ is 22°, it is proportional to the applied shear force and thus the compression forces in the diagonal concrete members, with heavier loads requiring a larger angle. To achieve economies with the shallower sections, the structural engineer may alternatively provide higher amounts of shear reinforcement to increase the shear resistance in the section while still maintaining a smaller strut inclination as although doing this increases the stress in the struts, activating a larger percentage of ties to extend the failure zone offsets this.

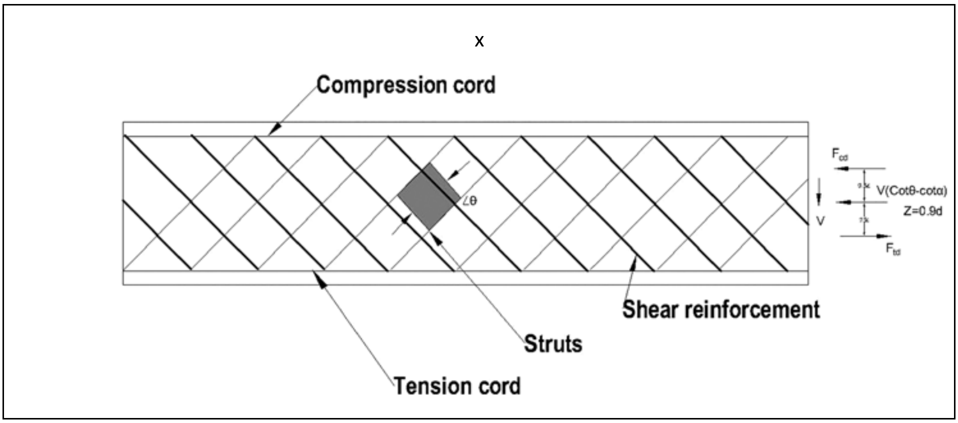


Fig. 5.13 Truss model and notations for shear reinforced members [14]

It is evident that the force in the diagonal concrete compressive strut, $\frac{VEd}{2 \cdot \sin\theta}$, must be balanced by a horizontal tensile force, ΔF_{td}, which generates additional tensile stress in the reinforcement. This value is highest at sections of zero bending moment as it is directly proportional to the applied shear. Moreover, the force is halved when transferred to the reinforcement in the beam's tension zone if a beam has two supports.

Horizontal Force in the bottom reinforcement,

$$\Delta F_{td} = \left(\frac{V_{Ed}}{2 \cdot \sin\theta}\right) \times \cos\theta = \frac{V_{Ed}\cot\theta}{2} \qquad \text{(Eq. 5.2)}$$

Eq. 5.1 and 5.2 form the basis of evaluating the design force, and therefore stress (σ_{sd}), to be resisted by the reinforcement.

According to Section 9.2.1.2 of EN 1992-1-1 [14], partial fixity or simple supported assumptions must also be designed to carry at least 15% of the maximum span bending moment in the top

bars. This means the area of reinforcement of the top bars and their anchorage length also must be calculated.

5.7.2 DETERMINATION OF ANCHORAGE LENGTH

The anchorage length is evaluated in accordance to Section 8.4 of EN 1992-1-1 [14] and is explained in detail below. The anchorage length closely relates to the design bond strength, f_{bd}, defined by Clause 8.4.2 of EN 1992-1-1 [14] as:

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} \qquad \text{(Eq. 5.3)}$$

here f_{ctd} is the design concrete tensile strength (Clause 3.1.6 & Table 3.1 from EN 1992-1-1 [14]):

- η_1 considers the rebar's position during casting – typically in members such as beams and slabs exceeding 250 mm in depth and where concrete is poured vertically, both top and bottom bars are well compacted but in sections exceeding 600 mm in depth, the top bars generally are not well compacted unless all bars have an inclination between 45-90° ($\eta_1 = 1.0$ when good bond conditions are assumed and $\eta_1 = 0.7$ for other conditions[1]);

1 Note that although this coefficient's value of 0.7 is conservative, reductions in bond are observed in deeper pours as unhardened concrete accumulates under the bar. In such positions, additional measures can be undertaken to minimise plastic settlement cracking [30].

- η_2 relates to the diameter of the rebar with bars larger than 32 mm in diameter adversely impacting bond conditions ($\eta_2 = 1$ for $\emptyset \leq 32\ mm$, else $\eta_2 = [(132 - \emptyset)/100]$)

Table 5.3 below illustrates the differences in design bond stress between IS 456 [11] & EN 1992-1-1 [14] for commonly used concrete grades. For post-installed reinforcement, the design bond stress used in the design must not exceed the value from Table-5.3.

Table 5.3 Bond stress for cast-in reinforcing bars assuming good bond conditions and bar diameters ≤ 32mm

Grade of Concrete ($f_{ck,cube}$)	**M20**	**M25**	**M30**	**M35**	**M40**	**M45**	**M50**	**M55**	**M60**
Bond stress τ_{bd}, N/mm^2 for deformed bars in tension per IS 456 [11]	1.92	2.24	2.4	2.72	3.04	3.04	3.04	3.04	3.04
Bond stress f_{bd},N/mm^2 for ribbed bars in tension per EN 1992-1-1 [14]	1.95	2.25	2.70	2.91	3.11	3.30	3.75	4.05	4.35

Similar to Section 26.2.1 in IS 456 [11], design provisions of Section 8.4.3 in EN 1992-1-1 [14] allow computation of the basic anchorage length per Eq. 5.4. In this equation, f_{bd} is the ultimate bond design stress value of cast-in reinforcing bars (values available in EN 1992-1-1 [14]) in N/mm^2 and of post-installed reinforcing bars (values available in the an adhesive's rebar ETA), σ_{sd} is the design stress of rebar considered at the section of design load, in N/mm^2; *d* is the reinforcing bars diameter in mm, and $l_{b,rqd}$ is the required anchorage length in

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} \quad \text{(Eq. 5.4)}$$

The design anchorage length, l_{bd}, is computed from the required anchorage ($l_{b,rqd}$) using Eq. 5.5.

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 l_{b,rqd} \geq l_{b,min} \quad \text{(Eq. 5.5a)}$$

In Eq. 5.5a, the factor α_1 accounts for the effect of the shape of bars assuming adequate cover. $\alpha_1 = 1$ is assumed for post-installed straight reinforcing bars satisfying Clause 8.4.4 in EN 1992-1-1 [14]. Since bends and hooks are feasible in cast-in connections, the design engineer may use factors of $\alpha_1 = 0.7$ if the confinement ratio $(\frac{c_d}{\phi})$ exceeds a value of 3.

The factor α_2 accounts for the effect of confinement due to concrete cover, i.e. the ratio between the minimum of half clear spacing a, clear cover c and c_1, and bar diameter, $\emptyset$, detailed in Fig.5.14.

$$\alpha_2 \begin{cases} = 1 - 0.15\,(c_d - \emptyset)/\emptyset \\ \geq 0.7 \\ \leq 1.0 \end{cases}$$

$$0.7 \leq \alpha_2 \leq 1$$

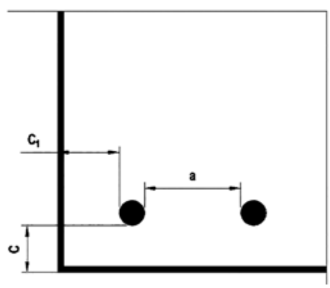


Fig. 5.14 Value of c_d for Straight bars $c_d = min(a/2\,, c_1, c)$ [14]

For post-installed reinforcement, the *centre-to-centre* spacing must either equal or exceed 5Ø and the clear cover must equal or exceed the minimum requirements from Section 4.4.1.2 of EN 1992-1-1 [14] to account for bond and exposure conditions and Table 1.1 of the EAD 330087-00-0601 [33], with the higher value governing:

- For hammer or diamond drilled holes[2]:
  - $c_{min} = 30 + 0.06 l_v \geq 2\emptyset\ (mm)$ for $(\emptyset < 25\ mm)$ (Eq. 5.5b)
  - $c_{min} = 40 + 0.06 l_v \geq 2\emptyset\ (mm)$ for $(\emptyset \geq 25\ mm)$ (Eq. 5.5c)
- For compressed air drilled holes:
  - $c_{min} = 50 + 0.08 l_v (mm)$ for $(\emptyset < 25\ mm)$ (Eq. 5.5d)
  - $c_{min} = 60 + 0.08 l_v \geq 2\emptyset\ (mm)$ for $(\emptyset \geq 25\ mm)$ (Eq. 5.5e)

The factor α_3 accounts for the effect of confinement due to non-welded transverse reinforcement. In case post-installed reinforcement, if no confinement by transverse reinforcement considered, then $\alpha_3 = 1$ is assumed. Transverse reinforcement is lateral steel in the form of ties or stirrups and increases the concrete's ductility thereby providing a beneficial effect to confined members reacting to Poisson-type lateral expansion.

The factor α_4 accounts for the effect of confinement due to welded transverse reinforcement along the anchorage length. For post-installed reinforcement, welded transverse reinforcement is not generally feasible or recommended. Thus, $\alpha_4 = 1$ is assumed.

The factor α_5 accounts for the effect of confinement due to pressure transverse to the plane of splitting along the anchorage length. Transverse pressure (p) will be present at the ultimate limit state along design anchorage length since simply supported and moment connections are always subject to a certain amount of transverse pressure in the anchorage area. This pressure is beneficial for anchorage of deep beams and corbels as there it restrains the splitting force [31]; however, it can be neglected for simplification, thus causing $\alpha_5 = 1$.

[2] Factors of 0.06 and 0.08 account for possible deviations in trajectory during drilling and may be reduced to 0.02 if using drilling aids.

$$\alpha_5 \left\{ \begin{array}{l} = 1 - 0.04p \\ \geq 0.7 \\ \leq 1.0 \end{array} \right. \quad \text{and} \quad 0.7 \leq \alpha_5 \leq 1 \qquad \text{(Eq. 5.5f)}$$

Confinement from cover, transverse reinforcement, and compressive transverse pressure has a beneficial effect on the bond strength; however, EN 1992-1-1 [14] limits the product of $(\alpha_2\alpha_3\alpha_5) \geq 0.7$, which in turn limits the potential reduction of the basic anchorage length.

The design anchorage length must be equal or exceed to the minimum anchorage length ($l_{b,min}$), defined by Eq. 5.6 and Eq. 5.7 as:

$l_{b,min} \geq \max\,(0.3l_{b,rqd,fyd}, 10d, 100mm)$ → for anchorages in tension (Eq. 5.6)

$l_{b,min} \geq \max\,(0.6l_{b,rqd,fyd}, 10d, 100mm)$ → for anchorages in compression (Eq. 5.7)

Here, note that the minimum anchorage lengths considered for both compression and tension are based on the anchorage length at yield as detailed in the German National Annex of EN 1992-1-1 [14]as well as Section 6.1.3.4 of the fib Model Code 2010 [30], which enables a static length for each bar independent on the design stress. Transfer of shear-friction between the old and new concrete is designed to Section 6.2.5 of EN 1992-1-1 [14].

Example 5.1 and 5.2 illustrates typical calculations necessary for calculating required anchorage length for post-installed rebar for forming concrete-to-concrete connections.

EXAMPLE 5.1

A new simply supported 300 x 600 mm beam spanning 6m is to be connected to an existing 300 x 600 mm column on its 300 mm face. On the grid layout, the columns are spaced 6 metres apart in both direction and have a grade of M30, resulting in a permanent load (G_k) of 36 kN/m and variable load (Q_k) of 18 kN/m applied on the beam, excluding its self-weight. Assuming a vertical and horizontal clear cover of 30 mm and 50 mm, respectively, calculate the area of reinforcement required and its anchorage depth into the existing column using a reinforcement grade, f_{yk}, of 500 MPa in the new section. [Application - Connecting a new RC beam between two existing RC columns].

Given:

$f_{ck} = 25\ \mathrm{N/mm^2}$ [Cylinder strength corresponding to M30 concrete grade]

$f_{yk} = 500\ \mathrm{N/mm^2}$

Solution:

We begin this example with an analysis of the section, in which the resulting load on the beam provided by:

$w = 1.35G_k + 1.5Q_k = 1.35[(0.3 * 0.6 * 25) + 36] + 1.5(18) = 81.675\ \mathrm{kN/m}$

Design Moment at midspan, $M_{Ed} = \frac{wl^2}{8} = \frac{81.675*6^2}{8} = 367.5\ \mathrm{kN-m}$

Design Shear Force at the support, $V_{Ed} = \frac{wl}{4} = 122.5\ \mathrm{kN}$

Depth to rebar centre, $d = h - cover - \frac{\phi}{2} = 600 - 30 - \frac{20}{2} = 560\ \mathrm{mm}$

$$K = \frac{M_{Ed}}{bd^2 f_{ck}} = \frac{367.5 * 10^6}{300 * 560^2 * 25} = 0.156 < 0.167$$

$\therefore$ Section designed as singly reinforced

Lever arm[3] , $z = d\left[0.5 + \sqrt{0.25 - \frac{K}{1.134}}\right] = 560\left[0.5 + \sqrt{0.25 - \frac{0.156}{1.134}}\right] = 468\ \mathrm{mm}$

Required area of steel reinforcement at midspan, $A_{s,req} = \frac{M_{Ed}}{0.87 f_{yk} z} = \frac{367.5*10^6}{0.87*500*468} =$ $1806\ \mathrm{mm}^2$

Provide 6∅20mm bars,3 each in the first and second layers, therefore $A_{s,prov} =$ $1890\ \mathrm{mm}^2$

Check provided reinforcement is within minimum limits from Section 7.3.2(2), Section 9.2.1.1(1) & (3) of EN 1992-1-1 [14]:

- $A_{s,min} = \max\left((k_c \cdot k \cdot f_{ct,eff} \cdot A_s)/(\sigma_s)\,;\ 0.26\frac{f_{ctm}}{f_{yk}} b_t d;\ 0.0013 b_t d\right)$

  $* 39600)/(500)\,;\ 0.26\frac{2.6}{500} 300 * 560;\ 0.0013 *$ $300 * 560) = 227\ \mathrm{mm}^2 < 1890\ \mathrm{mm}^2 \therefore \mathrm{OK}$

- $A_{s,max} = 0.04 A_c = 0.04 * 300 * 600 = 7200\ \mathrm{mm}^2 > 1890\ \mathrm{mm}^2 \therefore \mathrm{OK}$

According to the limits set by Section 9.2.1.4(1) of EN 1992-1-1 [14], 25% of the bottom reinforcement must extend to the end supports where little or no end fixity is assumed. Therefore, the three bars in the second layer can be curtailed, leaving only 3∅20 mm bars to be anchored.

Using the shift rule in 9.2.1.4(2) of EN 1992-1-1 [14], Eq. 5.2 is used to convert the applied shear into a tensile force:

$$\Delta F_{td} = \frac{V_{Ed} \cot\theta}{2} = \frac{122.5 * 10^3 \cdot \cot 22°}{2} = 152\ \mathrm{kN}$$

Resulting stress per bar, $\sigma_{sd} = \frac{152*10^2}{3\left(\frac{\pi*20^2}{4}\right)} = 161.3\ \mathrm{N/mm}^2 < 435\ \mathrm{N/mm}^2\ (f_{yd}) \therefore \mathrm{OK}$

3 https://www.concretecentre.com/Codes/Eurocode-2/Flexure.aspx

Using Table 5-3 & Eq. 5.4, the basic anchorage length, $l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} = \frac{20}{4} * \frac{161.3}{2.7} = 299 \text{ mm}$

Anchorage length required at yield, $l_{b,rqd,fyd} = \frac{\phi}{4} \cdot \frac{f_{yd}}{f_{bd}} = \frac{20}{4} * \frac{435}{2.7} = 806 \text{ mm}$

From Eq. 5.6, the minimum anchorage length in tension, $l_{b,min} \geq \max(0.3 l_{b,rqd,fyd}, 10\phi, 100 \text{mm})$

$\therefore l_{b,min} \geq \max(0.3 * 806; 10 * 20; 100 \text{ mm}) = 242 \text{ mm}$

Using Eq. 5.5, the design anchorage length, $l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 l_{b,rqd} \geq l_{b,min}$

Confining cover,

$$c_d = \min\left(c; c_1; \frac{a}{2}\right) = \min\left(50 \text{ mm}; 1000 \text{ mm}; \frac{70}{2}\right) = 35 \text{ mm}$$

$$\therefore \alpha_2 = 1 - 0.15 \frac{(c_d - \phi)}{\phi} = 1 - 0.15 \frac{(35 - 20)}{20} = 0.8875$$

Assuming $\alpha_1 = \alpha_3 = \alpha_4 = \alpha_5 = 1$, $l_{bd} = 1 * 0.8875 * 1 * 1 * 1 * 299 = 265 \text{ mm} \geq 242 \text{ mm } for\ l_{b,min} \therefore \text{OK}$

The three bottom 20 mm diameter rebars, spaced at 90 mm centres, must be anchored at least 265 mm into the existing concrete column.

Although the detailing arrangement and anchorage length for the top bar may also follow same details simplification, however, the same three 20 mm diameter rebars also may be used as top bars. According to Section 9.2.1.2 of EN 1992-1-1 [14], simple supports also must be designed for partial fixity of at least 15% of the maximum bending moment in the span. This means the area of reinforcement of the top bars and their anchorage length also must be calculated.

At end supports, $M_{Ed} = 0.15 * 367.5 = 55.125 \text{ kN} - \text{m}$

$$K = \frac{M_{Ed}}{bd^2 f_{ck}} = \frac{55.125 * 10^6}{300 * 560^2 * 25} = 0.023 < 0.167$$

Lever arm, $z = d\left[0.5 + \sqrt{0.25 - \frac{K}{1.134}}\right] = 560\left[0.5 + \sqrt{0.25 - \frac{0.023}{1.134}}\right] = 548 \text{ mm}$

Required area of steel reinforcement at midspan, $A_{s,req} = \frac{M_{Ed}}{0.87 f_{yk} z} = \frac{55.125*10^6}{0.87*500*548} = 231 \text{ mm}^2$

Provide 2Ø16 mm bars, with $A_{s,prov} = 402 \text{ mm}^2 > A_{s,min} \therefore \text{OK}$

Since there will be little to no stress in the bar caused by Eq. 5.2, $l_{b,min} = l_{bd}$

Anchorage length required at yield , $l_{b,rqd,fyd} = \frac{\phi}{4} \cdot \frac{f_{yd}}{f_{bd}} = \frac{16}{4} * \frac{435}{2.7} = 644 \text{ mm}$

From Eq. 5.6, the minimum anchorage length in tension, $l_{b,min} \geq \max(0.3 l_{b,rqd,fyd}, 10\phi, 100 \text{ mm})$

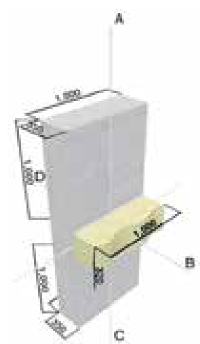
$\therefore l_{b,min} \geq \max(0.3 * 644; 10 * 16; 100 \text{ mm}) = 193 \text{ mm}$

The two top 16 mm diameter rebars, spaced at 184 mm centres, must be anchored at least 193 mm into the existing concrete column.

EXAMPLE 5.2

A new simply supported slab spanning 5 m is to be connected to an existing $350 \ X \ 1000$ mm wall on its 1000 mm face. Concrete strength class is M30 and it is dry concrete. Characteristic variable action Q_k is $25 \ \text{kN/m}^2$, excluding its self-weight. The slab thickness is 320 mm and assume clear cover of 40 mm. Calculate the area of reinforcement required and its anchorage length using a reinforcement grade, $f_{yk} = 500 \ \text{N/mm}^2$ and $\gamma_s = 1.15$. [Application- Connecting a new Slab between two existing RC walls].

Given:



$f_{ck} = 25 \text{ N/mm}^2$ [Cylinder strength corresponding to M30 concrete grade]

$f_{yk} = 500 \text{ N/mm}^2$

Solution:

We begin this example with an analysis, in which the resulting load on the slab provided by:

Characteristic permanent action $G_k = (25 \text{ kN/m}^3) h = 25 * 0.32 = 8 \text{ kN/m}^2$

$S_d = (1.5Q_k + 1.35G_k) = (1.5 * 25 + 1.35 * 8) = 48.3\ \mathrm{kN/m^2}$

Design value of applied internal bending moment, $M_{Ed} = (S_d \cdot l_n^2)/8 = (48.3 * 5^2)/8 = 150.94\ \mathrm{kNm/m}$

Design value of applied shear force, $V_{Ed} = (S_d \cdot l_n)/2 = (48.3 * 5)/2 = 120.75\ \mathrm{kN/m}$

Bottom reinforcement required at mid span,

$$A_{s,rqd,m} = (M_{sd} \cdot \gamma_s)/(0.9 \cdot d \cdot f_{yk}) = (150.94 * 10^6 * 1.15)/(0.9 * 280 * 500) = 1377.63\ \mathrm{mm^2/m}$$

Provide Ø20, spacing $s = 200$ mm reinforcement at mid span, therefore $A_{s,prov,m} = \frac{\pi}{4} * 20^2 * \frac{1000}{200} = 1570.8\ \mathrm{mm^2/m}$

Bottom reinforcement at support,

Tension force to be anchored as per Clause 9.2.1.4(2) of EN 1992-1-1 [14],

$$F_E = |V_{Ed}| \cdot a_l/(0.9d) = 120.75 * 280/(0.9 * 280) = 134.17\ \mathrm{kN/m}$$

Therefore, steel area required is $A_{s,rqd} = (F_E \cdot \gamma_s)/(f_{yk}) = (134.17 * 10^3 * 1.15)/(500) = 308.59\ \mathrm{mm^2/m}$

Minimum reinforcement to be anchored at support as per Clause 7.3.2(2), Clause 9.3.1.2(1), Clause 9.2.1.4(1) of EN 1992-1-1 [14] is

- $A_{s,min} = \max[(k_c \cdot k \cdot f_{ct,eff} \cdot A_s)/(\sigma_s), (0.50 \cdot A_{s,rqd,m}), (0.25 \cdot A_{s,prov,m})]$
- $A_{s,min} = \max[(0.4 * 0.986 * 2.6 * 160 * 1000)/(500), (0.50 * 1377.63), (0.25 * 1570.8)]$
- $A_{s,min} = \max[328.14\ \mathrm{mm^2/m}, 688.82\ \mathrm{mm^2/m}, 392.7\ \mathrm{mm^2/m}] = 688.82\ \mathrm{mm^2/m}$

Provide Ø16 and spacing $s = 200$ mm, so reinforcement provided is $A_{s,prov} = \frac{\pi}{4} * 16^2 * \frac{1000}{200} = 1005.31\ \mathrm{mm^2/m}$

Calculated design stress of the reinforcing bars is $\sigma_{sd} = (A_{s,rqd}/A_{s,prov}) \cdot (f_{yk}/\gamma_s) = (308.59/1005.31) * (500/1.15) = 133.46\ \mathrm{N/mm^2}$

Design value of bond strength according to ETA-16/0142 [39] is $f_{bd} = 2.7\ \mathrm{N/mm^2}$ for M30

So (Required) basic anchorage length as per Section 8.4.3 of EN 1992-1-1 [14] is $l_{b,rqd} = (\emptyset/4) \cdot (\sigma_{sd}/f_{bd}) = (16/4) * (133.46/2.7) = 197.72 \text{ mm}$

As per Section 8.4.4 of EC2: EN 1992-1-1 [14], design anchorage length $l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} \geq l_{b,min}$

For straight bars, $\alpha_1 = 1$

Confining cover, $c_d = min(a/2, c_1, c)$ and $c_d = (200 - 16)/2 = 92 \text{ mm}$; $d_s = 16 \text{ mm}$

$\alpha_2 = 1 - 0.15(c_d - \emptyset)/\emptyset = 0.96$ and $(0.7 \leq \alpha_2 \leq 1.0)$

For no transverse reinforcement, $\alpha_3 = 1$

For no welded transverse reinforcement, $\alpha_4 = 1$

If influence of transverse pressure is neglected, then $\alpha_5 = 1$

Therefore, $l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} = 1 * 0.96 * 1 * 1 * 1 * 197.72 = 190 \text{ mm}$

Minimum anchorage length as per Clause 8.4.4(1) of EN 1992-1-1 [14],

$$l_{b,min} = \max\{0.3 l_{b,rqd};\, 10\emptyset;\, 100 \text{ mm}\} = \max\{0.3 \cdot 197.72;\, 10 \cdot 16;\, 100 \text{ mm}\}$$
$$= 160 \text{ mm} \leq [l_{bd} = 190 \text{ mm}]$$

Therefore, l_{bd} controls the anchorage length and l_{bd} is equal to the drill hole length $l_{inst} = 190 \text{ mm}$. Note that the value of drilled hole length may differ from anchorage length.

Minimum top reinforcement at support is maximum of the

- 25% of bottom steel required at mid-span as per Clause 9.3.1.2(2) of EN 1992-1-1 [14] $A_{s,req} = (0.25 \cdot 1377.63) = 344.41 \text{ mm}^2/\text{m}$
- Requirement for crack limitation as per Clause 7.3.2(2) of EN 1992-1-1 [14] $A_{s,min} = (0.4 * 0.986 * 2.6 * 160 * 1000/435) = 377.17 \text{ mm}^2/\text{m}\ (decisive)$

So, provide $\emptyset 10$ and spacing $s = 200$ mm; therefore $A_{s,prov} = \frac{\pi}{4} * 10^2 * \frac{1000}{200} = 393 \text{ mm}^2/\text{m}$

Now, calculated design stress of the top reinforcing bar is

$\sigma_{sd} = f_{yd} \cdot (A_{s,min}/A_{s,prov}) = (500/1.15) * (377.17/393) = 417.27 \text{ N/mm}^2$

So (Required) basic anchorage length for top reinforcing bar is

$l_{b,rqd} = (\emptyset/4) \cdot (\sigma_{sd}/f_{bd}) = (10/4) * (417.27/2.7) = 386.36 \text{ mm}$

For top reinforcing bar the design anchorage length $l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} \geq l_{b,min}$

$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} = 1 * 0.7 * 1 * 1 * 1 * 386.36 = 270.45 \text{ mm}$

Minimum anchorage length for top reinforcing bar is

$l_{b,min} = \max\{0.3 l_{b,rqd}; 10\emptyset; 100 \text{ mm}\} = 115.91 \text{ mm} \leq [l_{bd} = 270.45 \text{ mm}]$

Therefore, drill hole length for top reinforcing bar is $l_v = l_{bd} \approx 271 \text{ mm}$.

5.7.3 DESIGN RULES FOR LAP SPLICES

While the previous section dealt with post-installed concrete-to-concrete structural joints, the other common application using post-installed reinforcing bar involves extending existing concrete members through lap. This sub-section elaborates the additions to the design procedure detailed in the previous section.

The underlying principle of EN 1992-1-1 [14] provisions relates to the ultimate bond stress being sufficient to preventing bond failure and there is a good reason to prevent lapped joints from bond failure. If the concrete cover or rebar spacing is low, bond action at the interface between the ribs and concrete will create bursting forces that in turn lead to longitudinal cracks along the lap length, causing a brittle, splitting failure where the concrete cover spalls. Laps typically are detailed to prevent this and instead ensure adequate ductility in the member that would result in yielding prior to bond failure [40]. Structural Engineers position these laps in areas where the stress induced on the reinforcement is low – i.e. in areas where plastic hinge formation is *not* expected – and much of the lapped reinforcement follows the same cross-sectional area of the previous reinforcement and is usually of an equivalent diameter. Laps are expected to transmit the full stress on one bar to another, avoid concrete spalling near joints, and avoid large cracking negatively affecting structural robustness. Apart from minor variations, the design of post-installed lap splice connections follows a similar logic to that of cast-in-place reinforcement and that of post-installed structural joints.

With $l_{b,rqd}$ and all other α factors remaining the same, the design lap length, l_0, is computed using Eq. 5.8:

$$l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_{b,rqd} \geq l_{b,min} \qquad \text{(Eq. 5.8)}$$

The minimum length required for lapped bars, $l_{0,min}$, is similar to Eq. 5.6 but with some modifications:

$$l_{b,min} \geq \max(0.3\alpha_6 l_{b,rqd,fyd}, 15d, 200 \text{ mm}) \qquad \text{(Eq. 5.9)}$$

The factor α_6 considers the percentage of longitudinal reinforcement lapped within a given section and its value can be directly referenced from Table 8.3 of EN 1992-1-1 [14], replicated below.

Percentage of lapped bars relative to the total cross-section area	$\boldsymbol{\alpha_6}$
<25%	1
33%	1.15
50%	1.4
>50%	1.5

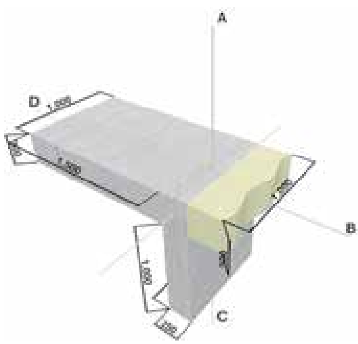
Fig.5.15 Excerpt Table 8.3 from EN 1992-1-1 [14]

The factor α_6 will equal 1.5 for most scenarios as it is customary to extend all the bars. For post-installed reinforcement, however, the Structural Engineer may choose to reduce any unnecessary lapped reinforcement if possible to avoid incurring a penalty on the required lap length. As an example, extending columns to incorporate one additional floor level for a building may not induce high moments based on the loading and therefore may require lesser reinforcement – say the provided reinforcement of 2000 mm^2 reduces to 1000 mm^2 in the section of lapping – allowing a factor 1.4 to be used instead of 1.5. The logic behind this lap length formulation is not too different to Section 26.2.5.1 of IS 456 [11].

EXAMPLE 5.3

A new balcony extension on an existing concrete structure having thickness 300 mm supported by wall of thickness 250 mm. The actions at support are design moment $M_{Ed} = 80\ \mathrm{kNm/m}$ and design shear $V_{Ed} = 50\ \mathrm{kN/m}$. Concrete strength class is M30. The reinforcement grade is $f_{yk} = 500\ \mathrm{N/mm^2}$. The top and bottom cover for cast-in reinforcing bars in existing slab is $c_c = 30$ mm. The cover for new reinforcing bars is $c_n = 50$ mm. $\emptyset 16$ mm at $s = 200$ mm is provided as top reinforcement in new and existing slab having cover to face $c_1 = 30$ mm. $\emptyset 10$ mm at $s = 200$mm is provided as bottom reinforcement. Determine the length for post-installed top reinforcing bars carrying tensile action. Don't assume fire action.

$\gamma_M = 1.5$. [Application-Splice on support].



Given:

$f_{ck} = 25\ \mathrm{N/mm^2}$ [Cylinder strength corresponding to M30 concrete grade]

$f_{yk} = 500\ \mathrm{N/mm^2}$

Solution:

The required design lap length $l_{0,1}$ for top cast-in reinforcing bar is calculated below as per Section 8.7.3 of EN 1992-1-1 [14]

$$l_{0,1} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b,rqd} \geq l_{b,min}$$

Here $d = h - c_c - (\emptyset/2) = 300 - 30 - (16/2) = 262 \text{ mm}$

So, $d - (\emptyset/2) = 262 - (16/2) = 254 > 250 \text{ mm}$ and therefore, for poor bond condition $\eta_1 = 0.7$

Note that the lever arm $z_1 = 239 \text{ mm}$ is from static calculations.

Steel area required: $A_{s,rqd} = (M_{Ed}/z_1) \cdot (\gamma_s/f_{yk}) = (80/0.239) * (1.15/0.5) = 770 \text{ mm}^2/\text{m}$

Calculated design stress of the reinforcing bars is

$$\sigma_{sd} = (A_{s,rqd}/A_{s,prov}) \cdot (f_{yk}/\gamma_s) = (770/1005) * (500/1.15) = 333 \text{ N/mm}^2$$

Design value of bond strength according EN 1992-1-1 [14];

$$f_{bd} = 2.25 \cdot \eta_1 \cdot 0.7 \cdot 0.3 \cdot f_{ck}{}^{2/3}/\gamma_c = 2.25 * 0.7 * 0.7 * 0.3 * 25^{2/3}/1.5$$
$$= 1.89 \text{ N/mm}^2$$

$$l_{b,rqd} = (\emptyset/4) \cdot (\sigma_{sd}/f_{bd}) = (16/4) * (333/1.89) = 705 \text{ mm}$$

For hooked end of cast-in bars, $\alpha_1 = 0.7$

$$\alpha_2 = 1 - 0.15(c_d - \emptyset)/\emptyset \geq 0.7 = 1 - 0.15 * (30 - 16)/16 = 0.87$$

For no transverse reinforcement, $\alpha = 1$

For no transverse pressure, $\alpha_5 = 1$

The splice factor $\alpha_6 = 1.5$

$$l_{b,min1} = \max(0.3\alpha_6 l_{b,rqd,fyd}, 15d, 200mm)$$
$$= \max(0.3 * 1.5 * 705, 15 * 16, 200 \text{ mm}) = 317 \text{ mm}$$

Therefore, $l_{01} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b,rqd} = 0.7 * 0.87 * 1 * 1 * 1.5 * 705 = 644 \text{ mm}$

The required design lap length $l_{0,2}$ for top post-installed reinforcing bar is calculated below as per Section 8.7.3 of EN 1992-1-1 [14]

$$l_{0,2} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} \geq l_{b,min}$$

Here $d = h - c_n - (\emptyset/2) = 300 - 50 - (16/2) = 242\ \mathrm{mm}$

So, $d - (\emptyset/2) = 242 - (16/2) = 234 < 250\ \mathrm{mm}$ and therefore for good bond condition $\eta_1 = 1$

Note that the lever arm $z = 228\ \mathrm{mm}$ is from static calculations.

Steel area required: $A_{s,rqd} = (M_{Ed}/z) \cdot (\gamma_s/f_{yk}) = (80/0.228) * (1.15/0.5) = 807\ \mathrm{mm}^2/\mathrm{m}$

Calculated design stress of the reinforcing bars is

$\sigma_{sd} = (A_{s,rqd}/A_{s,prov}) \cdot (f_{yk}/\gamma_s) = (807/1005) * (500/1.15) = 349\ \mathrm{N/mm}^2$

Design value of bond strength according to ETA-16/0142 [39]; $f_{bd} = 2.7\ \mathrm{N/mm}^2$ for M30

$l_{b,rqd} = (\emptyset/4) \cdot (\sigma_{sd}/f_{bd}) = (16/4) * (349/2.7) = 517\ \mathrm{mm}$

For straight bars, $\alpha_1 = 1$

Confining cover, $c_d = min(a/2\ , c_1, c)$ and $c_d = (200 - 16)/2 = 92\ \mathrm{mm}$; $d_s = 16\ \mathrm{mm}$

$\alpha_2 = 1 - 0.15(c_d - \emptyset)/\emptyset = 1 - 0.15 * (50 - 16)/16 = 0.7$ and $(0.7 \leq \alpha_2 \leq 1.0)$

For no transverse reinforcement, $\alpha_3 = 1$

For no transverse pressure, $\alpha_5 = 1$

The splice factor $\alpha_6 = 1.5$

$$\begin{aligned} l_{b,min,2} &= \max(0.3\alpha_6 l_{b,rqd,fyd}, 15d, 200\ \mathrm{mm}) \\ &= \max(0.3 * 1.5 * 517, 15 * 16, 200\ \mathrm{mm}) = 240\ \mathrm{mm} \end{aligned}$$

Therefore, $l_{0,2} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} = 1 * 0.7 * 1 * 1 * 1.5 * 517 = 542\ \mathrm{mm}$

Now, embedment depth for top post-installed reinforcing bars is $l_0 = \max(l_{0,1}, l_{0,2}, l_{0,3}, l_{0,min}) + \Delta l_0$

Here, $l_{0,1}$ is design overlap length of cast-in bar and $l_{0,1} = 644\ \mathrm{mm}$.

$l_{0,2}$ is design overlap length of post-installed reinforcing bar and $l_{0,2} = 542$ mm.

$l_{0,3}$ is design overlap length of post-installed reinforcing bar under fire and no fire in this case.

The minimum overlap length is $l_{0,min} = 317$ mm

Δl_0 is the increase of lap length due to clear distance. $\Delta l_0 = e - 4\emptyset$.

e is the clear distance between lapped bar.

$e = [(s/2)^2 + (c_n - c_c)^2]^{0.5} - \emptyset = [(100)^2 + (50 - 30)^2]^{0.5} - 16 = 86$ mm

So, $\Delta l_0 = e - 4\emptyset = 86 - 4 \cdot 16 = 22$ mm.

Therefore, $l_0 = \max(l_{0,1}, l_{0,2}, l_{0,3}, l_{0,min}) + \Delta l_0 = 644 + 22 = 666$ mm

Drilling length $l_{inst} = l_0 + \max(w/2\, , c_f) = 666 + \max(250/2\, , 30) = 791$ mm

5.7.4 EXTENSION OF STANDARD DESIGN METHOD

As stated in Section 5.7.1, the EAD 330087-00-0601 [33] currently does not assess rigid connections – those subject to a bending moment – without the presence of parallel, overlap bars illustrated by Fig.5.12. Furthermore, the embedment depths obtained using the standard design method, in some cases, may not be practical or the connection may not be feasible.

In former times, it was generally assumed that the failure mode was splitting, i.e. intended pull-out of the bar accompanied by formation of radial cracks and spalling of the concrete cover is not dependent from the bonding agent between the reinforcing bar and concrete. Therefore, no difference in performance related to splitting failure was assumed between post-installed reinforcing bars and cast-in bars. In the past years, tests have demonstrated that, even in the case of failure, the post-installed reinforcing bars with adhesives can reach to higher failure loads than cast-in anchorages in some conditions. Some technical research work has been carried out to extend the design method based on these findings. Such a specialized design method also allows taking advantage of high bond strength value, resulting in shorter embedment depths. However, these research works are product specific and cannot be applied to other products. Note that the design method presented in this section have been developed by one of the fastener manufacturer based on testing carried out on their products, and cannot be extended for design of products by other manufacturers as it is product specific.

The anchorage length provisions of the EN 1992-1-1 [14] assume that cast-in rebars may be closely spaced and located far from the concrete surface, which implies that the provisions take into consideration, steel, splitting and pull-out failure. Reduction in anchorage length is not given for reinforcing bars with a concrete cover equal to or greater than three times diameter of reinforcing bar, because it is assumed that pull-out could control for these reinforcing bars (as opposed to splitting). However, increases in concrete cover do allow further reductions in anchorage length, which is caused by increase to the design bond strength for post-installed reinforcing bars with a tested mortar. If the reinforcing bar is not loaded against a local or global compression-strut then the reinforcing bars carrying bond stresses could lead to concrete cone failure. So, when post-installed reinforcing bars used in a moment resisting connection without laps to existing reinforcing and installed sufficiently far from edges with a high value of concrete cover, then it may be suitable to refer specialized literature design concepts like HIT Rebar Method (HRM) [29]. It is discussed in this section.

In the EN 1992-1-1 [14] design method for post-installed reinforcing bars, the connection of simply supported slabs to columns or walls is only possible if the wall is thick enough to install the anchorage length. As reductions of the anchorage length with hooks or welded transverse reinforcement cannot be made with post-installed reinforcement, it often occurs that the wall is too small. However, if the confinement of the concrete is large enough, it is possible to use the full pull-out bond strength of the adhesive mortar rather than the equivalent bond strength given by EN 1992-1-1 [14]. This approach of extending the splitting domain makes it possible to design and construct of a higher number of structural joints that previously would be unfeasible under EN 1992-1-1 provisions. Under the EAD 330087-00-0601 [33] recommendations, moment-resisting frame node connections require bent-up bars as per traditional reinforced concrete principles and may not be feasible with straight post-installed rebar connections. The frame node strut and tie model are presented in literature [41] to design moment resisting frame node connections with straight connection bars.

In case of splices, the load is transferred through compression struts from one cast-in reinforcing bar to the adjacent post-installed reinforcing bar. Therefore, the ultimate load carrying behaviour of the splice is given by weaker element within this interaction represented by the cast-in reinforcing bar. Therefore, a utilization of the bond strength values of the used mortar type as followed in the specialized literature design method is limited by the spliced cast-in reinforcing bar embedded using the bond strength values given in EN 1992-1-1 [14].

However, there is typical case where the specialized literature design method may be beneficial for the overall splice length. It is common that reinforcement at the end of slabs/beams is hooked, and therefore for cast-in, the factor $\alpha_1 = 0.7$ and for post-installed reinforcing bar $\alpha_1 = 1$. The result is that the anchorage length of the post-installed reinforcing bar governs the overall anchorage length of the system connection. By using the specialized literature design method, the anchorage length of the post-installed reinforcing bar can be reduced up to the anchorage length of cast-in reinforcement.

The factor α_2 of EN 1992-1-1 [14] gives an explicit consideration for splitting and spalling as a function of related concrete cover $(c_d/\emptyset)$ i.e. passive confinement. The α_2 factor results in a maximum reduction of the anchorage length of approximately 43% within its limitation, if the concrete cover is larger than three times reinforcement diameter. The reduction of anchorage length can also be interpreted by means of an increase of the bond strength of max. 43% $(\alpha_2 = 0.7)$ while $0.7 \leq \alpha_2 \leq 1.0$. If α_2 exceeds 0.7, spalling of the concrete cover or splitting between bars will be the controlling mode of failure. If α_2 is less than 0.7, corresponding to cover dimensions of $(c_d/\emptyset > 3)$, the cover is large enough so that splitting cannot occur anymore, and pull-out will control.

$f_{bd} = f_{bd,EC2}/\alpha_2$ with $0.7 \leq \alpha_2 \leq 1.0$ (Eq. 5.10(a))

$$f_{bd} = f_{bd,EC2}/[1.0 - 0.15(c_d - \emptyset)/\emptyset] \quad \text{(Eq. 5.10(b))}$$

while for

$$(c_d/\emptyset) = 1.0,\ \alpha_2 = 1.0 \quad \text{(Eq. 5.10(c))}$$

$$(c_d/\emptyset) = 2.0,\ \alpha_2 = 0.85 \quad \text{(Eq. 5.10(d))}$$

$$(c_d/\emptyset) = 3.0,\ \alpha_2 = 0.7 \quad \text{(Eq. 5.10(e))}$$

This philosophy is referred with extension to post-installed reinforcing bars via the specialized literature design concept. In the following the footnote “SLD” is added to the denominations of the equations to indicate that the design is done according to the “splitting specialized literature design method”. The basic design anchorage length can be given as follow:

$$f_{bd,SLD} = f_{bd,EC2}/\alpha_2 \quad \text{(Eq. 5.11)}$$

For a related concrete cover of $1 \leq (c_d/\emptyset) \leq 3$; the “splitting” controls and the obtained design bond strength from the “splitting method” for post-installed reinforcing bar becomes the same as for cast-in reinforcing bar obtained from EN 1992-1-1 [14] by means of:

$$f_{bd,SLD,1} = f_{bd,EC2}/\alpha_2 \quad \text{(Eq. 5.12)}$$

For cover dimensions exceeding the range of EN 1992-1-1 [14], $(c_d/\emptyset) \geq 3$ (post-installed reinforcing bar with tested mortar, only)), an adapted factor α_2' is used to create a linear extension of the bond strength function related to the basic bond strength value f_{bd} while the linear approach was derived from a large number of tests to describe the increase of bond strength as a function of the related concrete cover. The increase in the design bond strength is limited by the maximum pull-out bond stress $f_{bd,p}$, which is a value taken from the relevant anchor approval. Thus, the limitation for bond failure in the code has been replaced by the specific design bond stress of the bonding agent for the specific application conditions and the splitting function given by:

$$f_{bd,SLD,2} = f_{bd,EC2}/\alpha'_2 \leq f_{bd,p} \quad \text{(Eq. 5.13(a))}$$

while

$$\alpha'_2 = \frac{1}{\frac{1}{0.7} + \delta \cdot \frac{c_d - 3\emptyset}{\emptyset}} \geq 0.25 \quad \text{(Eq. 5.13(b))}$$

where,

δ – describes the increase in design bond strength with increasing related concrete over taking account of the different mortar types.

$f_{bd,p}$ – maximum bond strength in case of pull-out taken from the relevant anchor approval.

0.25 – is the factor to avoid unreasonably low values of α'_2.

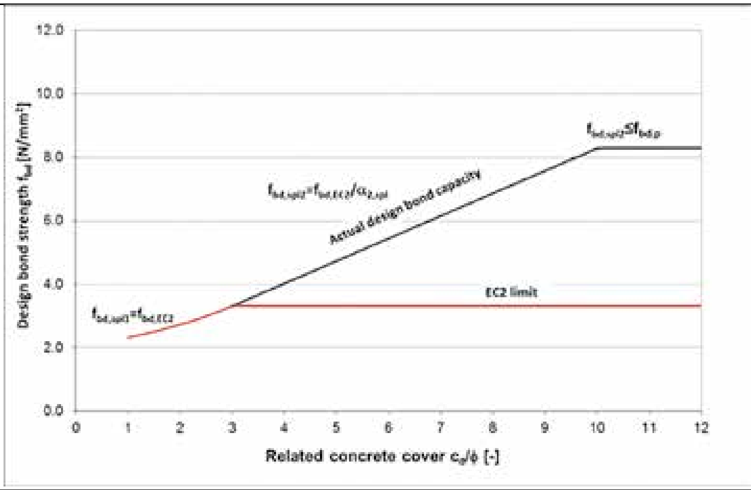


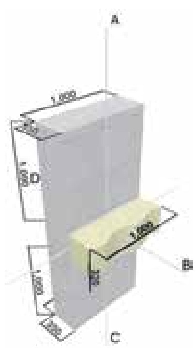
Fig. 5.16 Illustrative representation of effective limit on bond stress for post-installed reinforcing bar using tested mortar systems [29]

Fig. 5.16 shows the effective limit on bond stress for post-installed reinforcing bar using tested mortar systems represented by the “actual design bond capacity” and design bond strength values as provided by EN 1992-1-1 [14].

It shows an illustrative design bond strength f_{bd} curve as a function of the related concrete cover, shown for a concrete class M25 and for a reinforcing bar with a diameter not greater than 32 mm. In this, the equivalent design bond stresses according to EN 1992-1-1 [14] (red line) and the ones resulting from the above described definition of α_2 and α_2', are represented as a function of the related concrete cover (actual design bond capacity). The design bond strength is defined by an inclined line and it increases with larger values of c_d. In the following sample design example is given following as per this design concept.

EXAMPLE 5.4

A new simply supported slab spanning 5 m is to be connected to an existing 350 X 1000 mm wall on its 1000 mm face. Concrete strength class is M30 and it is dry concrete. Characteristic variable action Q_k is $25 \ \mathrm{kN/m^2}$, excluding its self-weight. The slab thickness is 320 mm and assume clear cover of 40 mm. Calculate the area of reinforcement required and its anchorage length according to the splitting specialized literature design method. Use a reinforcement grade, $f_{yk} = \ 500 \ \mathrm{N/mm^2}$ and $\gamma_s = \ 1.15$. [Application- Connecting a new Slab between two existing RC walls].



Given:

$f_{ck} = 25 \ \mathrm{N/mm^2}$ [Cylinder strength corresponding to M30 concrete grade]

$f_{yk} = 500 \text{ N/mm}^2$

Solution:

We begin this example with an analysis, in which the resulting load on the slab provided by:

Characteristic permanent action $G_k = (25 \text{ kN/m}^3) * h = 25 * 0.32 = 8 \text{ kN/m}^2$

$S_d = (1.5Q_k + 1.35G_k) = (1.5 * 25 + 1.35 * 8) = 48.3 \text{ kN/m}^2$

Design value of applied internal bending moment, $M_{Ed} = (S_d \cdot l^2_n)/8 = (48.3 * 5^2)/8 = 150.94 \text{ kNm/m}$

Design value of applied shear force, $V_{Ed} = (S_d \cdot l_n)/2 = (48.3 * 5)/2 = 120.75 \text{ kN/m}$

Bottom reinforcement required at mid span,

$A_{s,rqd,m} = (M_{sd} \cdot \gamma_s)/(0.9 \cdot d \cdot f_{yk}) = (150.94 * 10^6 * 1.15)/(0.9 * 280 * 500) = 1377.63 \text{ mm}^2/\text{m}$

Provide Ø20 mm, spacing $s = 200$ mm reinforcement at mid span, therefore $A_{s,prov,m} = \frac{\pi}{4} * 20^2 * \frac{1000}{200} = 1570.8 \text{ mm}^2/\text{m}$

Bottom reinforcement at support,

Tension force to be anchored as per Clause 9.2.1.4(2) of EN 1992-1-1 [14],

$F_E = |V_{Ed}| \cdot a_l /(0.9d) = 120.75 * 280 /(0.9 * 280) = 134.17 \text{ kN/m}$

Therefore, steel area required is $A_{s,rqd} = (F_E \cdot \gamma_s)/(f_{yk}) = (134.17 * 10^3 * 1.15)/(500) = 308.59 \text{ mm}^2/\text{m}$

Minimum reinforcement to be anchored at support as per Clause 7.3.2(2), Clause 9.3.1.2(1), Clause 9.2.1.4(1) of EN 1992-1-1 [14] is

- $A_{s,min} = \max[(k_c \cdot k \cdot f_{ct,eff} \cdot A_s)/(\sigma_s)\ , (0.50 \cdot A_{s,rqd,m}), (0.25 \cdot A_{s,prov,m})]$
- $A_{s,min} = \max[(0.4 * 0.986 * 2.6 * 160 * 1000)/(500)\ , (0.50 * 1377.63), (0.25 * 1570.8)]$
- $A_{s,min} = \max[328.14 \text{ mm}^2/\text{m}\ , 688.82 \text{ mm}^2/\text{m}\ , 392.7 \text{ mm}^2/\text{m}] = 688.82 \text{ mm}^2/\text{m}$

Provide $\emptyset 16$ and spacing $s = 200$mm, so reinforcement provided is $A_{s,prov} = \frac{\pi}{4} * 16^2 * \frac{1000}{200} = 1005.31 \text{ mm}^2/\text{m}$

Calculated design stress of the reinforcing bars is $\sigma_{sd} = (A_{s,rqd}/A_{s,prov}) \cdot (f_{yk}/\gamma_s) = (308.59/1005.31) * (500/1.15) = 133.46 \text{ N/mm}^2$

Design value of bond strength according to ETA-16/0142 [39] is $f_{bd,EC2} = 2.7 \text{ N/mm}^2$ for M 30

So (Required) basic anchorage length as per Section 8.4.3 of EN 1992-1-1 [14] is $l_{b,rqd} = (\emptyset/4) \cdot (\sigma_{sd}/f_{bd}) = (16/4) * (133.46/2.7) = 197.72 \text{ mm}$

Minimum anchorage length as per Clause 8.4.4(1) of EN 1992-1-1 [14],

$$l_{b,min} = \max\{0.3 l_{b,rqd}; 10\emptyset; 100 \text{ mm}\} = \max\{0.3 * 197.72; 10 * 16; 100 \text{ mm}\} = 160 \text{ mm}$$

Here, cover dimension is $c_d = (s - \emptyset)/2 = (200 - 16)/2 = 92 \text{ mm}$

Confinement is $(c_d/\emptyset) = 92/16 = 5.75 > 3$

Splitting bond strength for $(c_d/\emptyset) > 3$ is $f_{bd,spl,2} = f_{bd,EC2}/max(\alpha_2', 0.25)$

where, $\alpha_2' = \frac{1}{\frac{1}{0.7} + \delta \cdot \frac{c_d - 3\emptyset}{\emptyset}} \geq 0.25$ and $\alpha_2' = \frac{1}{\frac{1}{0.7} + 0.306 * \frac{92 - 3*16}{16}} = 0.4405 \geq 0.25$

So, $f_{bd,spl,2} = 2.3/0.4405 = 5.22 \text{ N/mm}^2$

Now, Pull-out bond strength is $f_{bd,p} = 10 \text{ N/mm}^2$ as per ETA -16/0142 [39]

Applicable design bond strength is $f_{bd} = min(f_{bd,spl}, f_{bd,p}) = 5.22 \text{ N/mm}^2$.
Design development length is $l_{bd} = \frac{\emptyset}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} = \frac{16}{4} * \frac{133.46}{5.22} = 102.27 \text{ mm}$.

So, minimum length $l_{b,min}$ controls the drill hole length $l_{inst} = 160 \text{ mm}$.

Note: Using the splitting specialized literature design approach the minimum anchorage length controls with $l_{b,min} = 160 \text{ mm} = l_{inst}$. In the same design example, following the EN 1992-1-1 [14] the design anchorage length controlled with $l_{bd} = 190 \text{ mm} = l_{inst}$. Using the splitting specialized literature design approach reduced the anchorage length of around 15.79%.

Minimum top reinforcement at support is maximum of the

- 25% of bottom steel required at mid-span as per Clause 9.3.1.2(2) of EN 1992-1-1 [14] $A_{s,req} = (0.25 \cdot 1377.63) = 344.41 \text{ mm}^2/\text{m}$
- Requirement for crack limitation as per Clause 7.3.2(2) of EC2: EN 1992-1-1 [14], $A_{s,min} = (0.4 * 0.986 * 2.6 * 160 * 1000/435) = 377.17 \text{ mm}^2/\text{m}\ (decisive)$

So, provide $\emptyset 10$ mm and spacing $s = 200$ mm; therefore $A_{s,prov} = \frac{\pi}{4} * 10^2 * \frac{1000}{200} = 393 \text{ mm}^2/\text{m}$

Now, design steel stress of the top reinforcing bar is

$$\sigma_{sd} = f_{yd} \cdot (A_{s,min}/A_{s,prov}) = (500/1.15) * (377.17/393) = 417.27 \text{ N/mm}^2$$

Here, design value of bond strength according to ETA-16/0142 [39] is $f_{bd,EC2} = 2.7 \text{ N/mm}^2$ for M30

So (Required) basic anchorage length for top reinforcing bar is

$$l_{b,rqd} = (\emptyset/4) \cdot (\sigma_{sd}/f_{bd}) = (10/4) * (417.27/2.7) = 386.36 \text{ mm}$$

Minimum anchorage length for top reinforcing bar is

$$l_{b,min} = \max\{0.3 l_{b,rqd};\ 10\emptyset;\ 100 \text{ mm}\} = \max\{0.3 * 386.36;\ 10 * 10;\ 100\}$$
$$= 116 \text{ mm} \leq [l_{bd} = 270.45 \text{ mm}]$$

Here, cover dimension is $c_d = (s - \emptyset)/2 = (200 - 10)/2 = 95 \text{ mm}$

Confinement is $(c_d/\emptyset) = 95/10 = 9.5 > 3$

Splitting bond strength for $(c_d/\emptyset) > 3$ is $f_{bd,spl,2} = f_{bd,EC2}/max(\alpha_2', 0.25)$

where, $\alpha_2' = \frac{1}{\frac{1}{0.7} + \delta \cdot \frac{c_d - 3\emptyset}{\emptyset}} \geq 0.25$ and $\alpha_2' = \frac{1}{\frac{1}{0.7} + 0.306 * \frac{95 - 3*10}{10}} = 0.293 \geq 0.25$

So, $f_{bd,spl,2} = 2.7/0.293 = 9.22\ \mathrm{N/mm^2}$

Now, Pull-out bond strength is $f_{bd,p} = 8.5\ \mathrm{N/mm^2}$ as per ETA -16/0142 [39]

Applicable design bond strength is $f_{bd} = min(f_{bd,spl}, f_{bd,p}) = 8.5\ \mathrm{N/mm^2}$.
Design development length is $l_{bd} = \frac{\emptyset}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} = \frac{10}{4} * \frac{417.27}{8.5} = 123\ \mathrm{mm}$.

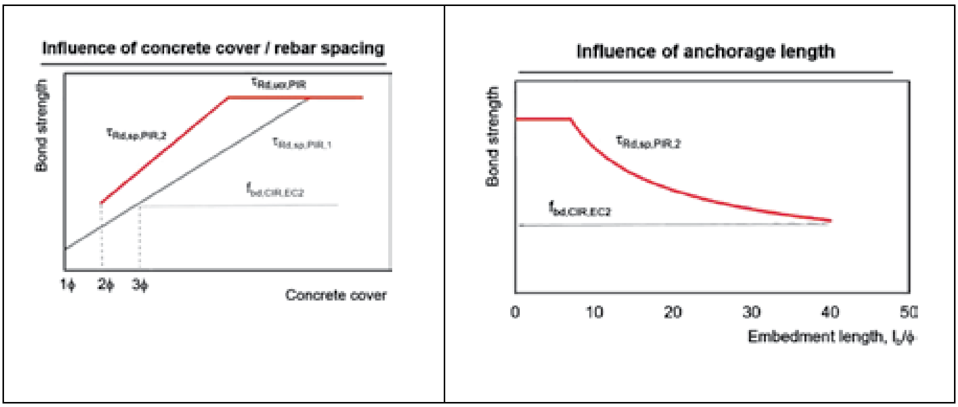
Development length l_{bd} controls the drill hole length $l_{inst} = 123\ \mathrm{mm}$. Therefore, drill hole length $l_{ef} = 123\ \mathrm{mm}$

Note that using the splitting specialized literature design approach the minimum anchorage length controls with $l_{bd} = 123\ \mathrm{mm} = l_{inst}$. In the same design example, following the EN 1992-1-1 [14] the design anchorage length controlled with $l_{bd} = 271\ \mathrm{mm} = l_{inst}$. Using the splitting specialized literature design approach reduced the anchorage length of around 54.61 %.

In continuation as per some testing, the level of performance appears to be linked to the adhesive used. Examples of the utilization of such improved performance have been shown at the beginning of this section. Further, some manufacturers had done detailed research to validate these indications for selected adhesive products and an experimental testing has been conducted to investigate the phenomenon of bond-splitting failure of post-installed versus cast-in reinforcement situated close to an edge or at the corner of concrete substrates. The main varied parameters were the distance of the bars to the edge, i.e. the concrete cover, the embedment depth (i.e. anchorage length) of the bars, the bar diameter, the strength of the concrete and the position of the reinforcing bars during concreting [42]. Numerous tests have been performed in independent laboratories for identifying impact of various influencing parameters like edge distance, concrete strength, anchorage length, rebar diameter, etc. and impact of robustness [42]. The observed results have been evaluated by researchers, comparing the behaviour of cast-in and post-installed reinforcement with each other. The experimental studies confirmed the high performance of the tested mortar for $c_d < 3\emptyset$ [42]. The advance method allows the use a higher bond strength of tested mortar for $c_d < 3\emptyset$. The splitting bond strength increases with

increasing concrete cover (or bar spacing) till the uncracked bond strength is reached and then it is constant for tested adhesive systems (see Fig. 5.17(a)). Furthermore, the splitting bond strength decreases with increasing embedment length (see Fig. 5.17(a)) The maximum splitting resistances were also assessed based on the *fib* model code MC2010 [30] approach for calculating the bond strength of cast-in reinforcement.

Based on the bond-splitting model detailed research, more advanced method can be developed for mortar systems which are properly tested according. This bond-splitting model is applicable for selected adhesive products only which were considered in the investigation. This advanced method allows further optimization of the anchorage length in different cases like rebar close to and far from the concrete edge. However, the applicability in design must also be validated, since bond strengths higher than the values given in EN 1992-1-1 [14].



(a) (b)

Fig.5.17 Schematic representation of bond strength for post-installed reinforcing bar using tested mortar systems considering an advanced bond-splitting model with respect to (a) concrete cover and (b) embedment depth [Hilti Reference]

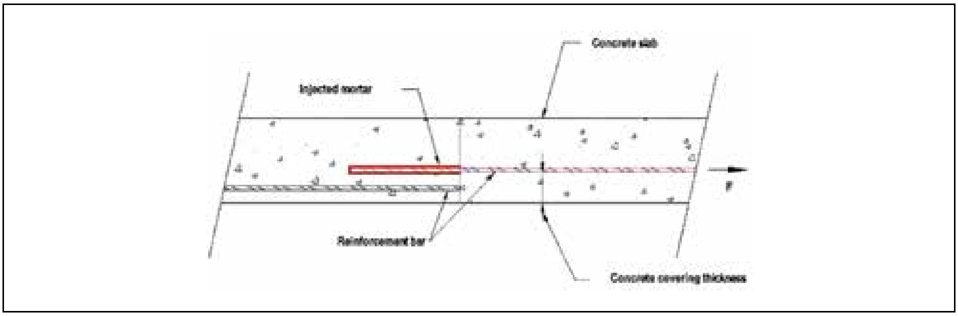
5.7.5 DESIGN FOR EXPOSURE TO FIRE

As an accidental load case applicable to all structures, fires cause reduction in the flexure and shear resistances of reinforced concrete members, with the behaviour of these members exposed to fire typically described in terms of "fire resistance", which is the period under exposure to the standard ISO 834 [43] time-temperature curve. The effect of fire on a reinforced concrete structure is two-fold: direct and indirect. In the case of the former, the temperature in the structure – or more realistically a specific compartment where the fire originates – first increases and then gradually reduces in line with the fire's extinction. Due to the large dimensions of the structural elements, the temperature in the concrete member is not uniform under a fire scenario and the material properties will vary at different locations across the cross-section, thus influencing the behaviour. The indirect effects relate primarily to restraints on thermal expansion, giving rise to internal stresses and deformations. As an example, a beam unable to freely expand longitudinally will be subjected to increased axial forces in the first few minutes of the fire [30]. Building occupants must have sufficient time to evacuate and certain critical connections that may threaten the evacuation or human life itself must not fail and therefore be designed to resist fire exposure.

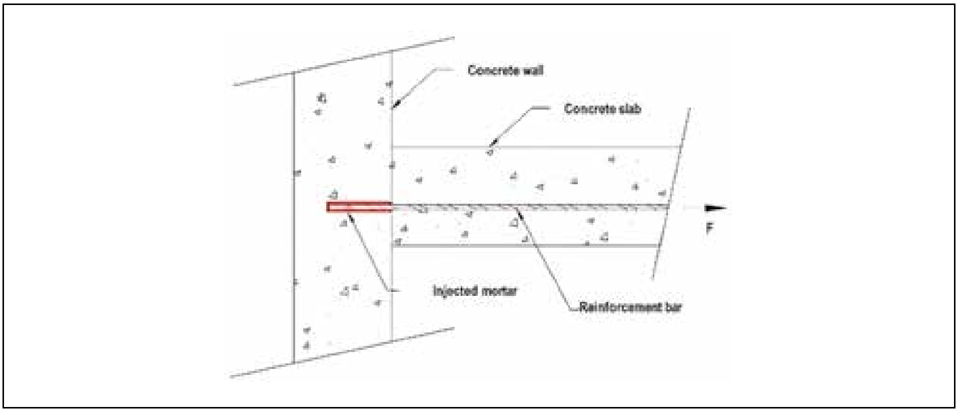
Performance of reinforced concrete under fire typically is defined as: resistance to structural collapse, failure of structural integrity allowing spread of fire, and deflection limit – the latter being the most common approach adapted by current design standards under the "fire limit state". This limit state considers that due to the relative improbability of fire scenarios, lower partial safety factors are used as compared to the ultimate limit states. Additionally, due to partial protection offered in a real scenario where the region affected by fire is restrained by the cooler surrounding areas, the heating will be non-uniform: this stems from concrete having good thermal resistance and insulating the steel reinforcement. For cast-in-place reinforcement, the loss in bond is similar to the loss in the concrete's tensile strength and to guard against pre-mature failure through bond or splitting.

Post-installed reinforcement follows a similar logic and is not too different, but the loss in performance of the adhesive used must be calculated when structural fire design requirements govern the design as the organic compounds forming most adhesives degrade drastically and permanently through carbonisation under *direct* exposure and less drastically under *indirect* exposure, which is the usual scenario. As such, it is vital that the fire resistance of a post-installed structural joint or lap splice connection be evaluated through an assessment such as EAD 330087-00-0601 [33] that provides, in the ETA, the time-dependant reductions to bond stress at elevated temperatures and therefore the resistance of a connection. For lap splices, the temperature along the anchorage length is based

on the concrete cover and time exposure, whereas, for structural joints, this is not constant and based on both the anchorage length and the time exposure, as illustrated by Fig.5.18



(a)



(b)

Fig.5.18 Sketches of (a) a splice and of (b) a structural joint [29]

To improve the performance of a post-installed RC connection in fire, the Structural Engineer is advised to increase the concrete cover to the post-installed rebars to further insulate the adhesive or, alternatively, increase the bond length of the post-installed rebars to compensate for the reduction in bond stress at elevated temperatures. Specifically, at the fire limit state, fire actions in a member must not exceed its fire resistance, outlined by Eq. 5.10:

$$E_{d,fi} \leq R_{d,fi} \qquad \text{(Eq. 5.14)}$$

where,

$$E_{d,fi} = \eta_{fi} \cdot E_d \qquad \text{(Eq. 5.15)}$$

Here, E_d accounts for the design actions under ambient conditions and is modified by an EN 1992-1-2 [44] reduction factor, η_{fi}, of 0.7, resulting in $E_{d,fi}$, which is the design effect of actions in fire situations and this must be less than the design resistance of the member under fire, $R_{d,fi}$. The anchorage length itself can be evaluated using the same design provisions detailed in Eq. 5.1 through Eq. 5.9 but uses $f_{bd,fi}$ instead of f_{bd}. The design bond stress under fire, $f_{bd,fi}$, is calculated from Eq. 5.12.

$$f_{bd,fi}(\theta) = k_{fi}(\theta) \cdot f_{bdPIR} \cdot \left(\frac{\gamma_C}{\gamma_{M,fi}}\right) \quad \text{(Eq. 5.16)}$$

where,

$k_{fi}(\theta)$ – Reduction factor as a function of temperature (see Fig5.19)

$f_{bd,PIR}$ – Design value of the bond strength of a post-installed rebar at ambient temperature (from Technical Assessment report like ETA for rebar)

γ_C – Partial safety factor according to EN 1992-1-1 [14] (usually 1.5)

$\gamma_{M,fi}$ – Partial safety factor according to EN 1992-1-2 [44] (deals with fire design of concrete structures, usually 1.0)

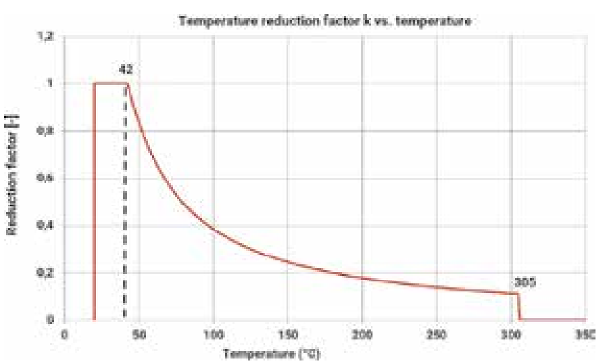


Fig.5.19 Example of the reduction factor $\boldsymbol{k_{fi}(\theta)}$ for M25 grade concrete as a function of temperature for good bond conditions [33]

The design also considers the residual stress in the steel reinforcement in the event of a fire and a strength reduction of $k_s(\theta)$ is applied to the characteristic stress, where these reductions to the steel stress are taken from Section 4.2.4.3 of EN 1992-1-2 [44]. The design stress in the rebar, σ_{sd}, must not exceed the maximum steel stress in fire for a given temperature, $\sigma_{Rd,fi}$.

5.8 DESIGN SPECIFICATION

The design details like - the position of the post-installed reinforcing bar, its diameter, its embedment depth, special installation instructions etc. should be clearly marked in the specification and construction drawings. Some typical specification details are shown in Fig.5.20

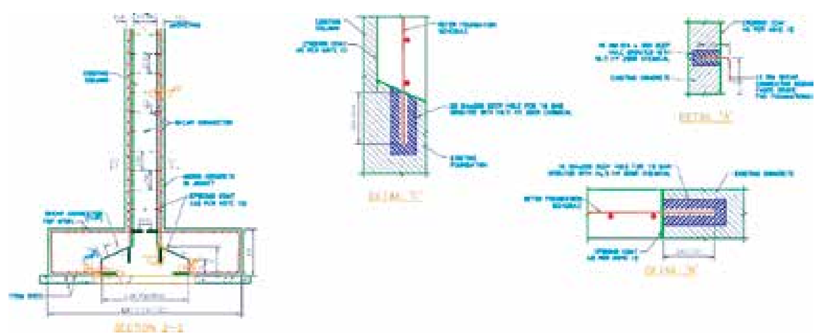


Fig. 5.20 Typical specification details for connections formed using post-installed reinforcing bars

5.9 INSTALLATION AND INSPECTION

Although each application has unique characteristics and parameters that influence the assessment and design criteria as elaborated in this chapter, the execution part stays the same. The very first step is design, i.e. the connection design should be carried out by the responsible engineer. The next step is execution on site. Correct installation of post-installed rebar is paramount to ensure performance as per design. The exact installation steps are manufacturer and product dependent and are available for reference in the product's Technical

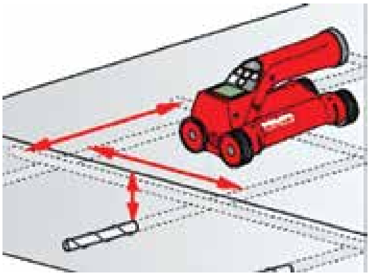
Assessment report (e.g. ETA), which should be adhered to. The installation should be carried out under supervision by a skilled labourer who has been trained on product installation.

Some general installation steps are discussed in this section. At site, all installation parameters (e.g. Installation temperature, condition of base material etc.) are checked prior to installation, to ensure that they are in accordance with the design assumptions. First, the installer uses a concrete detection system to allow the installer to identify the position of the rebar or other pipes in the existing concrete structure; this avoids unnecessary hits to rebar and wasted material and time. The base material is also inspected. Holes are then drilled into the existing concrete member using either a hammer drill or – if the existing reinforcement can

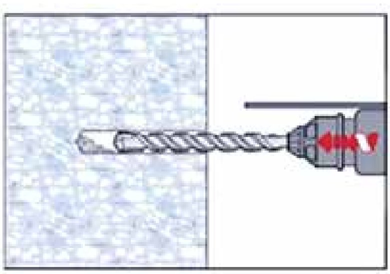
be cut – a dry- or wet-coring machine. A roughening tool is usually used in case of diamond cored holes to make the surface rough and ensure better bond strength. Hole if drilled unnecessarily or due to mistake are filled with a non-shrink grout of suitable compressive strength (but ≥ 40 N/mm^2).

The existing concrete member's surface is roughed where contact is expected with the new member, which enables shear transfer (higher the surface roughness, greater the shear capacity). The drilled hole is then cleaned to remove the debris attached to the walls of the drilled / bore hole; this dictates the efficacy of the bond between the grout or adhesive mortar and the existing concrete. If proprietary systems like hollow drill bit are used then, cleaning step may be skipped if recommended by manufacturer depending on application conditions. Following cleaning the of the hole, the adhesive is injected uniformly to ensure no air pockets or voids are formed; the formation of these voids means that the stress from the reinforcing bar cannot be transferred evenly to the existing concrete structure and can be the cause of potential failures. The new post-installed rebar is then inserted in the hole filled with adhesive, connecting it to the rest of the new rebars. The post-installed reinforcing bars should be loaded only after adhesive is completely cured. In case of deep embedment with large diameter (especially in horizontal or overhead orientation), there should be a mechanism to hold reinforcing bars in place during curing. Last step is to cast the new concrete to complete the connection. Typical installation steps are illustrated in Fig.5.21.

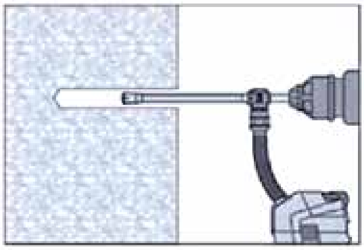
As stated in previous chapters, the installation details should be maintained for inspection. Sometimes proof load test/ or onsite test is also carried out to check quality of workmanship. As stated earlier, onsite tests are only a mean of checking installation and not a tool for comparing two products, because some important characteristic of a post-installed rebar system cannot be checked and quantified via on-site testing (e.g., behaviour under elevated short-and long-term temperature, corrosion protection of the rebar, resistance to freeze-thaw cycles, etc). Details like test equipment, applied load type and value, result interpretation etc. is documented and maintained for each onsite test.



(a) Detection of existing rebar prior to drilling

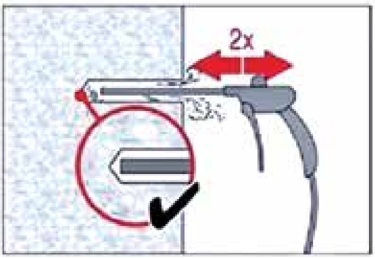


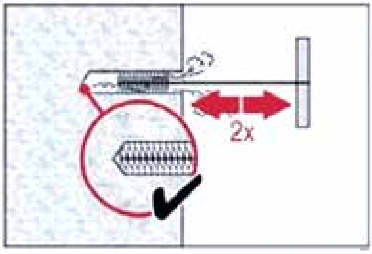
Drilling using hammer drill

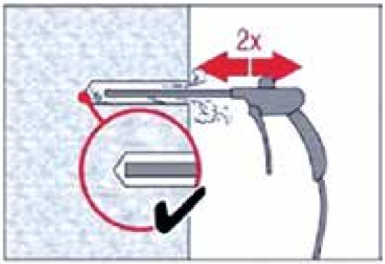


Or, alternate drilling method like hollow drill b
it with vacuum (Safeset system) which eliminates cleaning

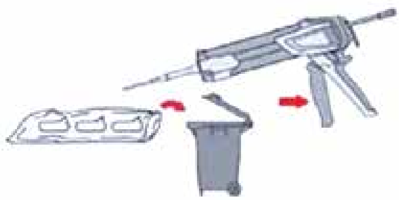
(b) Drilling of hole for rebar installation



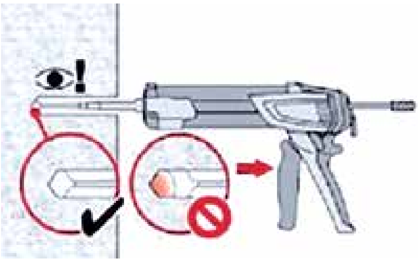




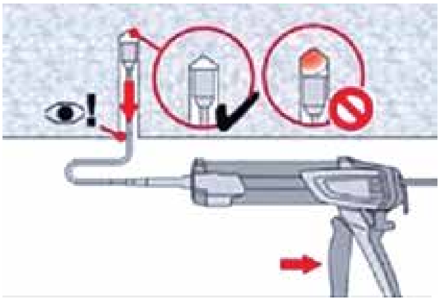
(c) Cleaning as recommended by Manufacturer
(depends on drilling method, application condition etc.)



Discard intial mortar

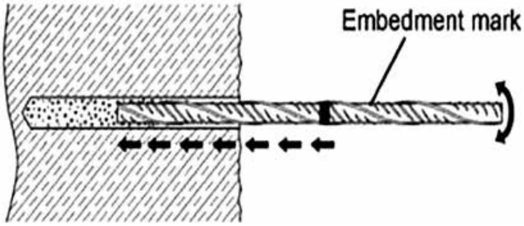


Inject adhesive without air voids

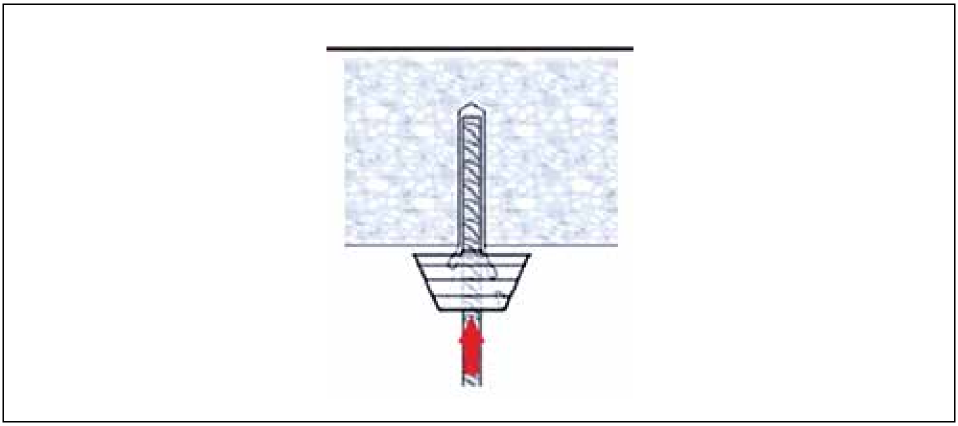


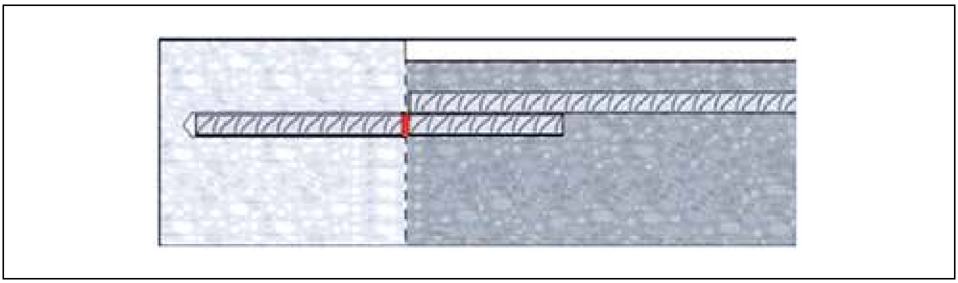
Take suitable measures for proper installation for overhead applications

(d) Injection of adhesive in drilled hole



(e) Insert rebar





(f) Wait for rebar to gain strength (i.e. cure time);
take special measures for overhead instalaltion

Fig. 5.21 Typical steps for installing post-installed rebar [39]

5.10 PRACTISE PROBLEMS

Case1: A new 300 x 650 mm beam spanning 6.5 m is to be connected to an existing 300 x 650 mm column on its 300 mm face. On the grid layout, the columns are spaced 6.5 m apart in both direction and have a grade of M30, resulting in a permanent load (G_k) of 36 kN/m and variable load (Q_k) of 20 kN/m applied on the beam, excluding its self-weight. Assuming a vertical and horizontal clear cover of 30 mm and 50 mm, respectively, calculate the area of reinforcement required and its anchorage depth into the existing column using a reinforcement grade, f_{yk}, of 500 $\mathrm{N/mm^2}$ in the new section.

Case 2: A new simply supported slab spanning 6 m is to be connected to an existing 450 X 1000 mm wall on its 1000 mm face. Concrete strength class is M30 and it is dry concrete. Characteristic variable action Q_k is $30\ \mathrm{kN/m^2}$, excluding its self-weight. The slab thickness is 350 mm and assume clear cover of 40 mm. Calculate the area of reinforcement required and its anchorage length using a reinforcement grade, $f_{yk} = 500\ \mathrm{N/mm^2}$ and $\gamma_s = 1.15$.

Case 3: A new simply supported slab spanning 6 m is to be connected to an existing 450 X 1000 mm wall on its 1000 mm face. Concrete strength class is M30 and it is dry concrete. Characteristic variable action Q_k is $30\ kN/m^2$, excluding its self-weight. The slab thickness is 350 mm and assume clear cover of 40 mm. Calculate the area of reinforcement required and its anchorage length according to the splitting specialized literature design method. Use a reinforcement grade, $f_{yk} = 500\ \mathrm{N/mm^2}$ and $\gamma_s = 1.15$.

CHAPTER 6

Case Studies

6.1 OVERVIEW

In this chapter, some case studies are presented. The objective is to highlight the practical applications and challenges that can be solved using the technologies discussed in this book.

6.2 CASE STUDY 1

Title: Retrofitting Work of Existing Commercial Building to Cater Change in Occupancy Requirement by Client

Author: Er. Umesh Joshi, Partner, JW consultants LLP

6.2.1 INTRODUCTION

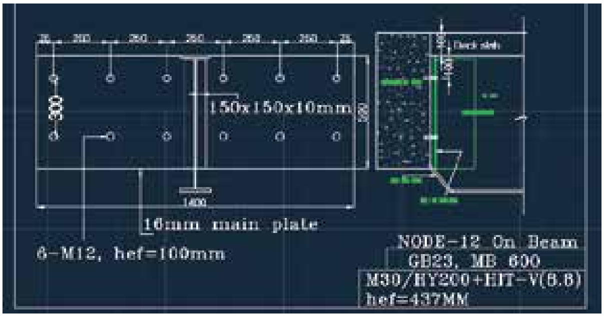
This case study documents the retrofitting approach adopted for a commercial building. The retrofitting work was necessitated due to change in occupancy and/or functionality as per the brief provided by the client. To fulfil the requirements due to change in occupancy, there were many alteration and additions called in existing structure as per architectural space planning to satisfy the revised functional requirement.

6.2.2 RETROFITTING APPROACH

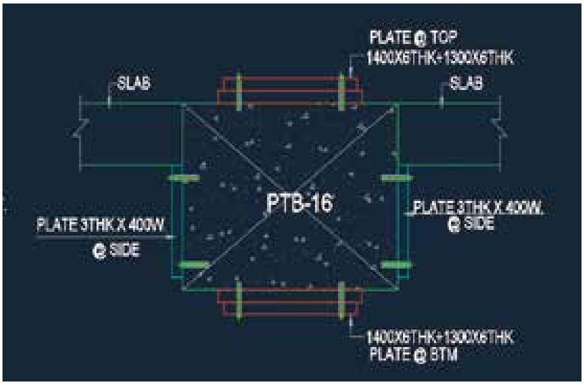
To meet this functionality requirement as described in Section 6.2.1, many additional structural elements were provided, and many existing structural elements were also strengthened to satisfy the strength and serviceability short fall. As per architectural planning, existing large openings were also closed with structural steel deck slabs and small cut-outs were closed with post-installed reinforcing bars. As per the drawing, following retrofitting work was carried out.

- Existing beams were strengthened with MS plates/underneath steel girders i.e. steel jacketing, for lagging moments and shear,
- The large floor cut-outs with closed with deck slabs supported on structural steel girders spanning between the column/beams.
- Steel column base connections

Typical schematic connection details are shown in Fig.6.1. For better understanding of the applications, some site pictures are also presented in Fig.6.2. Alternate retrofitting methods like carbon wrapping and concrete jacketing technics for existing beam was envisaged to be more complicated and difficult as per the requirement and existing site condition.



(a) ISMB fixing connection details



(b) PT beam strengthening details

Fig. 6.1 Retrofitting connection details

(a) ISMB fixing on column

(b) Girder connections

Fig. 6.2 Retrofitting site photographs

6.2.3 SUMMARY OF SOLUTION USED

In this strengthening work approximately 50 beams were strengthened and approximately 500 steel girder connections were carried out. Each connection was unique due to varying site conditions. Because of this each connection was designed by taking actual measurements on site and taking into consideration feasibility of location / installation of anchors. The major challenge was to avoid cutting / damaging the existing reinforcement and PT tendons while carrying out the connections. This could be achieved with the use of Ferro-scanners like Hilti PS 1000.

This helped to detect existing reinforcement layout to decide the anchor locations before execution, which saved lot of rework and time to carry out the connections with very few alterations.

Hilti “HIT-HY 200R” hybrid-based adhesive with HILTI HIT-V (5.8 grade) and AM (8.8 grade) rods was widely used in this project. The Hilti “Profis” software was used to design the anchoring system. With above-foresaid equipment’s, technics and support from all the stakeholders, the said retrofitting execution could be completed well in time (around 11 months) with all the challenges.

6.3 CASE STUDY 2

Title: Strengthening of Existing RCC Frame to Support Bagging House in a Fertilizer Plant

Author: Er. Partha Chakraborty, Technical Director, CDC Consulting Design Engineering Centre Pvt. Ltd.

6.3.1 INTRODUCTION

This is a case study from Offsite and Utilities for CFG3 Plants of Chambal Fertilizers & Chemicals Ltd. at Gadepan, Kota. There was an existing platform shed consisting of RCC columns and tie beams with a steel structure on top (see Fig.6.13). The height of the shed was approximately 9 meters. On top of this RCC Platform, the bagging house (height approximately 20 meters) RCC frame system had to be constructed.

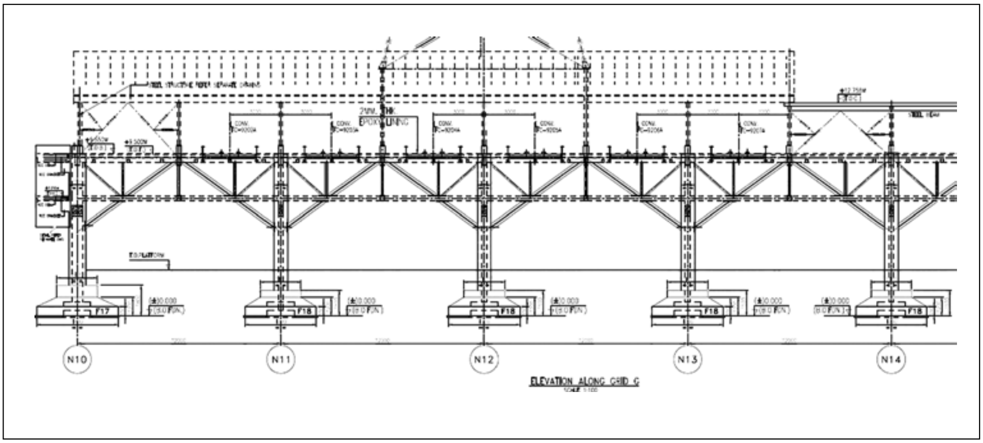


Fig. 6.3 Schematic diagram of existing platform shed

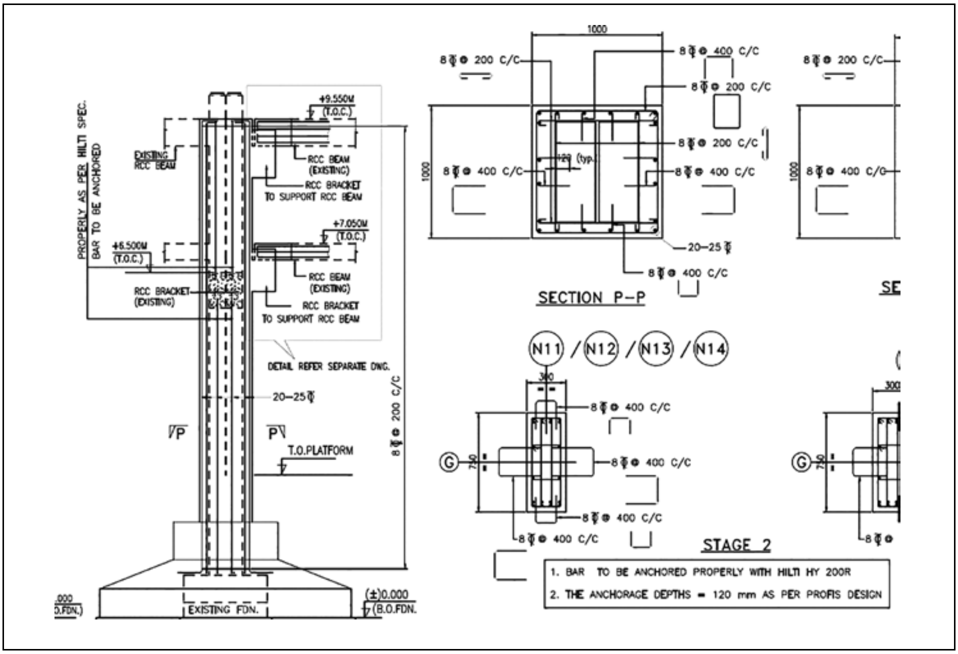
6.3.2 CHALLENGES

The primary constraint in this project was that no part of the existing structure could be dismantled (as there was a live railway line nearby and its service could not be interrupted). In addition, the same column beam frame system had to be utilized. This called for extensive strengthening of the entire existing RCC frame system below including its foundation system.

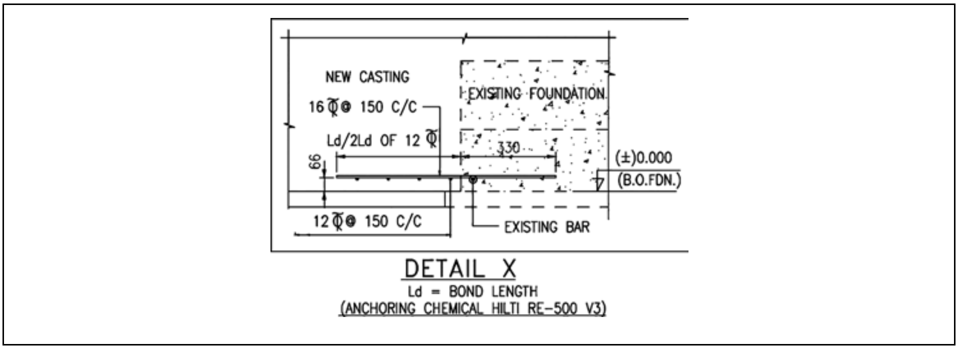
6.3.3 SUMMARY OF STRENGTHENING METHOD

At the very beginning of the project, a condition assessment of the existing structure was conducted to ascertain the grade of concrete of the members. Once this was done, structural analysis was carried out for the entire composite structure. Since no dismantling was allowed, the only way modification could be achieved was through the post-installed anchoring and rebaring technology. Keeping in mind the criticality of the work, for every connection, load parameters were shared with the fastening manufacturer's design team (in this case Hilti) based on which a design report was submitted. This was carried out for all steel-to-concrete connections as well as concrete-to-concrete connections. Typical connection schematic details are shown in Fig.6.4. To avoid over drilling or hitting existing reinforcement bars, scanning was recommended before drilling. Since the number of drill points was huge, to ensure proper workmanship, the

Safeset concept was introduced. Special installation notes were provided to ensure proper execution. The job was carried out by trained personnel.



(a) Post-installed anchoring application



(b) Post-installed rebar application

Fig. 6.4 Typical connection details

6.4 CASE STUDY 3

Title: Rehabilitation of RCC Structure Under Construction

Author: Er. Utpal Santra and Er. Ankit Agarwala, Manager Design, M. N. Consultants (Pvt.) Ltd.

6.4.1 INTRODUCTION

This case study is about rehabilitation of a commercial building that started showing signs of distress while partial occupation was there. This commercial building was located in North India. This project which houses office and retail spaces was meant to be 14 storeys. After completion of the structure, in the 10th year, the structure started showing signs of distress. At first, the signs of distress were observed in the columns in the fifth and sixth floors. There was extensive spalling and cracking in the columns (see Fig 6.5).

6.4.2 INVESTIGATION

To identify the cause of sudden distress investigation was carried out. Core tests were conducted at specific locations. In addition, certain non-destructive tests (NDTs) were also conducted. It was observed that the grade of concrete was well below the acceptable limit defined in design. It was not possible to demolish and rebuild. Hence, the only option was to bring the structural elements back to the desired strength.

6.4.3 SOLUTION

In order to rehabilitate the columns, it was decided to adopt a steel plate jacketing and strengthening of all the columns. The approach was either full plate jacketing or corner angle batten jacketing based on the capacity and load. Post installed anchoring was adopted in accordance with the international standards and guidelines. Typical connection details used to strengthen the column joint is shown in Fig. 6.6

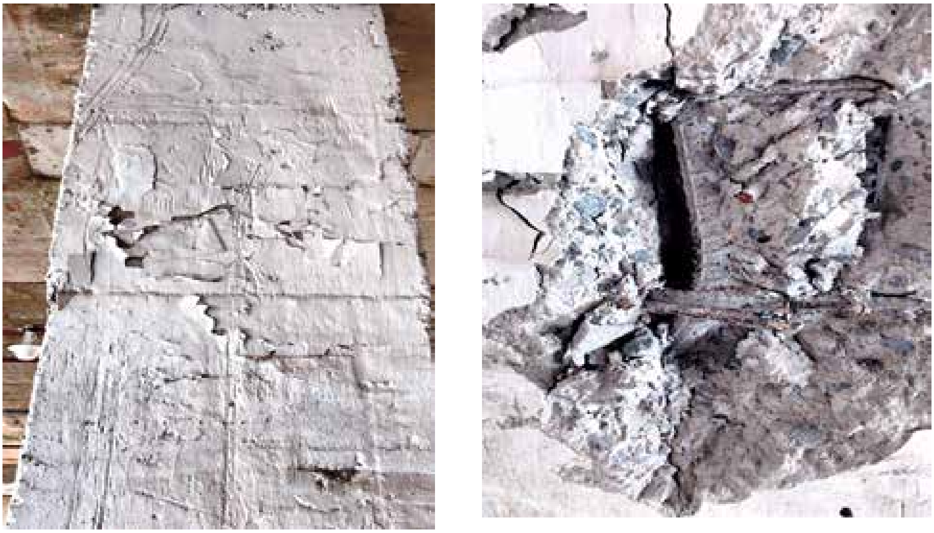
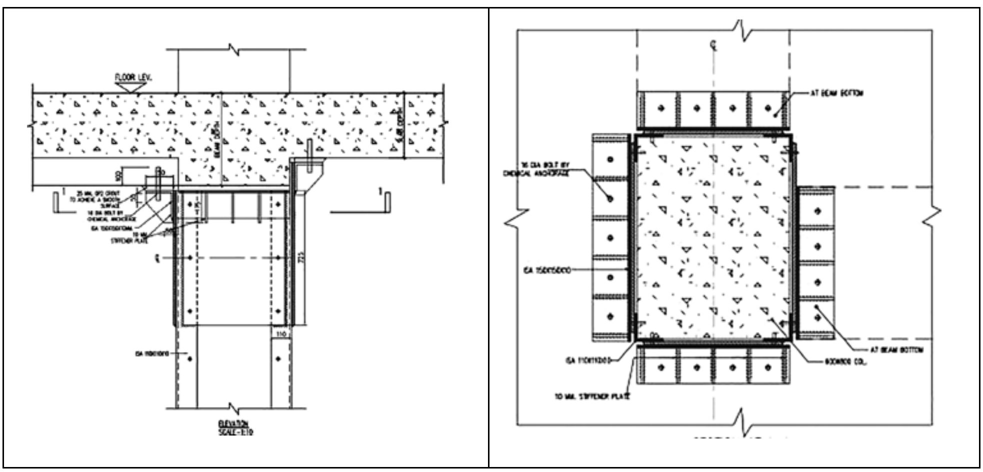


Fig. 6.5 Condition of distressed column



(a) Levation (b) Plan

Fig.6.6 Typical connection detail

6.5 CASE STUDY 4

Title: Retrofitting of Existing Occupied/in-use Commercial Building After Collapse of Cantilever Slab at Terrace Level

Author: Er. H. P. Yogesh, Director, Innotech Engineering Consultants Private Ltd

6.5.1 INTRODUCTION

This case study is about retrofitting of a portion of RCC building which collapsed. The Cantilever slab at terrace level suddenly collapsed. Innotech was tasked to investigate the cause for sudden collapse as well as come up with solution to retrofit the collapsed portion. The retrofitting project duration was about 1-2 Months.

6.5.2 INVESTIGATION

After the collapse, analysis was conducted by the team to find the reasons for collapse.The following were identified to be the main reasons for the collapse based on analysis post the incident:

- Rusting of bars of cantilever slab due to continuous water leakage from the chiller Plant
- In addition to this, the cantilever slab was subject to extra load, without prior approval from structural consultant.

6.5.3 CHALLENGES AND SOLUTION

Design of retrofitting work was taken up by Innotech Engineering Consultants Private Limited. The detailing of retrofitting work was done in a span of 15-20 days. The detailing work further had to go through 2-3 checks. Since all the other floors in the commercial building were operational, it was important to select a retrofitting process which would utilize the least amount of time. The proposed measure was to introduce deck slab at terrace level instead of RCC and structural steel brackets were provided at the floor below. Typical connection detail is illustrated in Fig.6.7. Connection detailing design was done as per TR029

[45][1].i.e. the design method of bonded anchors, as the use of other design methods would have required reconsideration of the necessary tests. The design of anchorages was in accordance with the general rules given in EN 1990, where the value of the design actions S_d does not exceed the value of the design resistance R_d (i.e. $S_d \leq R_d$). Actions to be used in design may be obtained from national regulations or in the absence of them from the relevant parts of EN 1991 i.e. Eurocode 1. The partial safety factors for actions may be taken from national regulations or in the absence of them according to EN 1990, which is Eurocode – Basis of Structural Design.

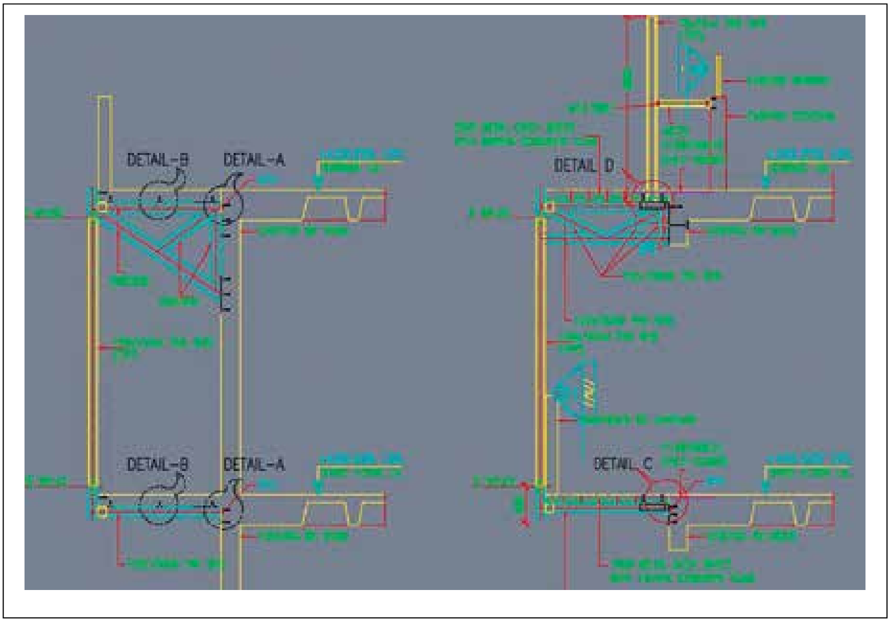


Fig.6.7 Typical connection detail

[1] TR029[18] was referred to for design of bonded anchors before it was superseded by EN 1992-4[6]

6.6 CASE STUDY 5

Title: Connection of ETFE Column Directly to Mother Slab

Author: Er. Srinivas Rao Bolimuntha, Technical Director- Structures, Design Tree

6.6.1 INTRODUCTION

This case study is of Connection (Anchoring) of ETFE column directly to the mother Slab. ETFE Columns were to be fixed and the entire thing was imported to India.

6.6.2 CHALLENGES

Major concerns were time, cost and also space constraint as office spaces were operational. Since time and cost were the major concerns in this project, construction of foundation and installation of J-bolts were not the preferred method, since it would have led to major delay and space availability for construction was also a big concern. In addition to this, the loads acting at the connection were tension and moment and very negligible amount of compression load.

6.6.3 CHALLENGES AND SOLUTION

The adopted method was post-installed anchoring, designs were done by Design Tree. The columns were supposed to be anchored directly to the mother slab. The connections were very critical and solution suggested was of bonded anchoring systems which had to withstand huge moment forces due to wind. Typical connection detail is illustrated in Fig.6.8. Entire Structure had a short deadline of completion so no foundation, directly anchoring on the mother slab resulted in saving huge cost and time for the ETFE structure. Baseplate was anchored to the slab using adhesive bolt designed as per TR029 [45], on which ETFE Column Baseplate was welded. Some site photographs are shown in Fig.6.9.

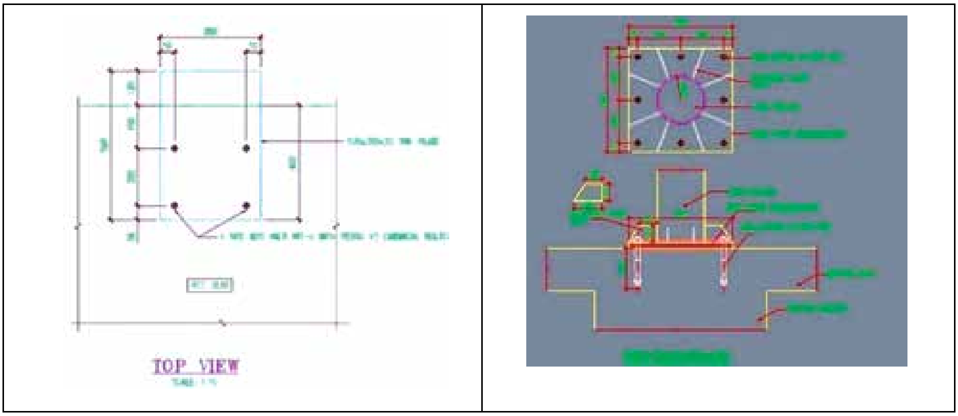


Fig. 6.8 Typical connection detail

Fig. 6.9 Site photographs

ANNEX I

Deutsches Institut für Bautechnik

DIBt

Member of EOTA

www.eota.eu

Approval body for construction products
and types of construction

Bautechnisches Prüfamt

An institution established by the Federal and Laender Governments

Designated according to Article 29 of Regulation (EU) No 305/2011 and member of EOTA (European Organisation for Technical Assessment)

European Technical Assessment

ETA-12/0084 of 28 July 2017

English translation prepared by DIBt - Original version in German language

General Part

Technical Assessment Body issuing the European Technical Assessment:	Deutsches Institut für Bautechnik
Trade name of the construction product	Injection system Hilti HIT-HY 200-R
Product family to which the construction product belongs	Bonded anchor for use in concrete
Manufacturer	Hilti Aktiengesellschaft 9494 SCHAAN FÜRSTENTUM LIECHTENSTEIN
Manufacturing plant	Hilti Werke
This European Technical Assessment contains	39 pages including 3 annexes
This European Technical Assessment is issued in accordance with Regulation (EU) No 305/2011, on the basis of	Guideline for European technical approval of "Metal anchors for use in concrete", ETAG 001 Part 5: "Bonded anchors", April 2013, used as European Assessment Document (EAD) according to Article 66 Paragraph 3 of Regulation (EU) No 305/2011.
This version replaces	ETA-12/0084 issued on 3 February 2017

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Specific Part

1 Technical description of the product

The Injection system Hilti HIT-HY 200-R is a bonded anchor consisting of a foil pack with injection mortar Hilti HIT-HY 200-R and a steel element according to Annex A.

The steel element is placed into a drilled hole filled with injection mortar and is anchored via the bond between metal part, injection mortar and concrete.

The product description is given in Annex A.

2 Specification of the intended use in accordance with the applicable European Assessment Document

The performances given in Section 3 are only valid if the anchor is used in compliance with the specifications and conditions given in Annex B.

The verifications and assessment methods on which this European Technical Assessment is based lead to the assumption of a working life of the anchor of at least 50 years. The indications given on the working life cannot be interpreted as a guarantee given by the producer, but are to be regarded only as a means for choosing the right products in relation to the expected economically reasonable working life of the works.

3 Performance of the product and references to the methods used for its assessment

3.1 Mechanical resistance and stability (BWR 1)

Essential characteristic	Performance
Characteristic resistance under static and quasi-static action, displacements	See Annex C1 to C12
Characteristic resistance for seismic performance category C1 and C2, displacements	See Annex C13 to C17

3.2 Safety in case of fire (BWR 2)

Essential characteristic	Performance
Reaction to fire	Anchorages satisfy requirements for Class A1
Resistance to fire	No performance determined (NPD)

3.3 Hygiene, health and the environment (BWR 3)

Regarding dangerous substances there may be requirements (e.g. transposed European legislation and national laws, regulations and administrative provisions) applicable to the products falling within the scope of this European Technical Assessment. In order to meet the provisions of Regulation (EU) No 305/2011, these requirements need also to be complied with, when and where they apply.

3.4 Safety in use (BWR 4)

The essential characteristics regarding Safety in use are included under the Basic Works Requirement Mechanical resistance and stability.

4 Assessment and verification of constancy of performance (AVCP) system applied, with reference to its legal base

In accordance with guideline for European technical approval ETAG 001, April 2013 used as European Assessment Document (EAD) according to Article 66 Paragraph 3 of Regulation (EU) No 305/2011 the applicable European legal act is: [96/582/EC].

The system to be applied is: 1

5 Technical details necessary for the implementation of the AVCP system, as provided for in the applicable European Assessment Document

Technical details necessary for the implementation of the AVCP system are laid down in the control plan deposited at Deutsches Institut für Bautechnik.

Issued in Berlin on 28 July 2017 by Deutsches Institut für Bautechnik

BD Dipl.-Ing. Andreas Kummerow
Head of Department

beglaubigt:
Lange

Installed condition

Figure A1:
Threaded rod and HIT-V-..., AM 8.8

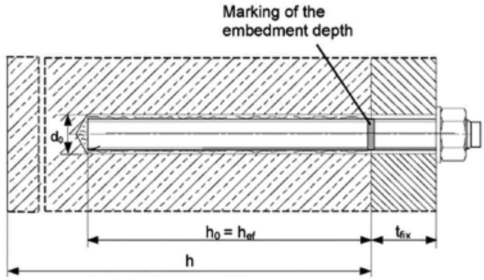


Figure A2:
Threaded rod and HIT-V-..., AM 8.8, with Hilti Filling Set

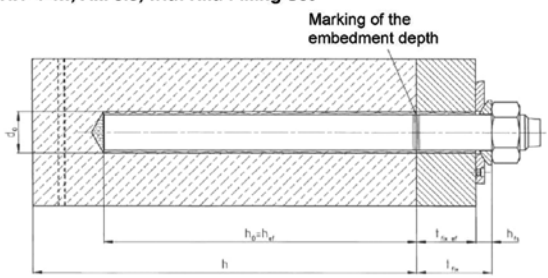
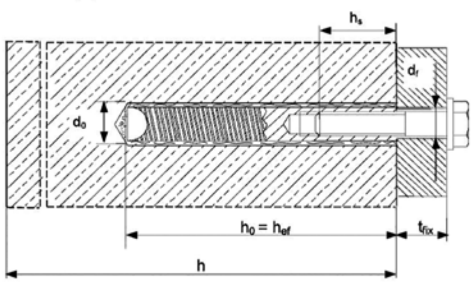


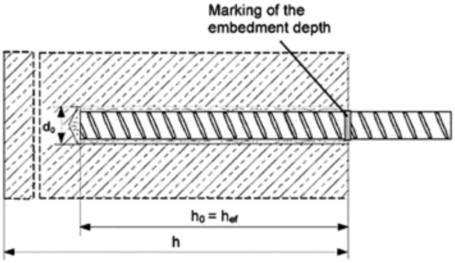
Figure A3:
Internally threaded sleeve HIS-(R)N



Injection System Hilti HIT-HY 200-R	Annex A1
Product description Installed condition	

Installed condition

Figure A4:
Reinforcing bar



Injection System Hilti HIT-HY 200-R	**Annex A2**
Product description Installed condition	

Product description: Injection mortar and steel elements

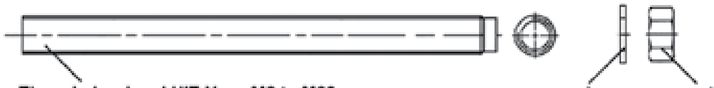
Injection mortar Hilti HIT-HY 200-R: hybrid system with aggregate
330 ml and 500 ml

Marking:
HILTI-HIT
Production number and
production line
Expiry date mm/yyyy

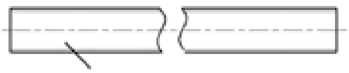
Product name: "Hilti HIT-HY 200-R"

Static mixer Hilti HIT-RE-M

Steel elements



Threaded rod and HIT-V-...: M8 to M30 washer nut



Hilti meter rod AM 8.8 electroplated zinc coated, AM HDG 8.8 hot dip galvanized
M8 to M30, 1m to 3m

Commercial standard threaded rod:
- Materials and mechanical properties according to Table A1.
- Inspection certificate 3.1 according to EN 10204:2004. The document shall be stored.
- Marking of embedment depth.

Internally threaded sleeve: HIS-(R)N M8 to M20

Hilti Tension Anchor: HZA M12 to M27 and HZA-R M12 to M24

Injection System Hilti HIT-HY 200-R	**Annex A3**
Product description Injection mortar / Static mixer / Steel elements	

Product description: Injection mortar and steel elements

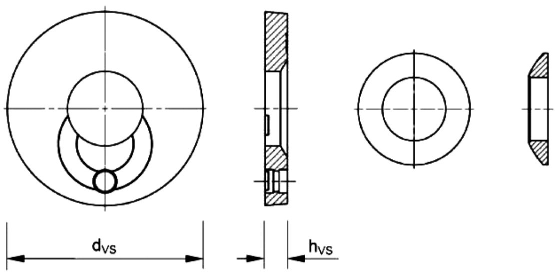
Reinforcing bar (rebar): ϕ 8 to ϕ 32

- Materials and mechanical properties according to Table A1
- Dimensions according to Annex B6

Hilti Filling Set to fill the annular gap between anchor and fixture

Sealing washer

Spherical washer



Filling Set			**M16**	**M20**	**M24**
Diameter of sealing washer	d_{VS}	[mm]	56	60	70
Thickness of sealing washer	h_{VS}	[mm]	6		

Injection System Hilti HIT-HY 200-R

Product description
Injection mortar / Static mixer / Steel elements

Annex A4

Table A1: Materials

Designation	Material
Reinforcing bars	
Rebar: EN 1992-1-1: 2004 and AC:2010, Annex C	Bars and de-coiled rods class B or C with f_{yk} and k according to NDP or NCL of EN 1992-1-1/NA:2013 $f_{uk} = f_{tk} = k \cdot f_{yk}$
Metal parts made of zinc coated steel	
Threaded rod, HIT-V-5.8(F)	Strength class 5.8, f_{uk} = 500 N/mm², f_{yk} = 400 N/mm², Elongation at fracture (l_0=5d) > 8% ductile Electroplated zinc coated ≥ 5 µm, (F) hot dip galvanized ≥ 45 µm
Threaded rod, HIT-V-8.8(F)	Strength class 8.8, f_{uk} = 800 N/mm², f_{yk} = 640 N/mm², Elongation at fracture (l_0=5d) > 12% ductile Electroplated zinc coated ≥ 5 µm, (F) hot dip galvanized ≥ 45 µm
Hilti Meter rod, AM 8.8 (HDG)	Strength class 8.8, f_{uk} = 800 N/mm², f_{yk} = 640 N/mm² Elongation at fracture (l_0 = 5d) > 12% ductile, Electroplated zinc coated ≥ 5 µm, (HDG) hot dip galvanized ≥ 45 µm
Hilti tension anchor HZA	Round steel with threaded part: electroplated zinc coated ≥ 5 µm Rebar: Bars class B according to NDP or NCL of EN 1992-1-1/NA:2013
Internally threaded sleeve HIS-N	Electroplated zinc coated ≥ 5 µm
Washer	Electroplated zinc coated ≥ 5 µm, hot dip galvanized ≥ 45 µm
Nut	Strength class of nut adapted to strength class of threaded rod Electroplated zinc coated ≥ 5 µm, hot dip galvanized ≥ 45 µm
Hilti Filling set (F)	Filling washer: Electroplated zinc coated ≥ 5 µm, (F) hot dip galvanized ≥ 45 µm Spherical washer: Electroplated zinc coated ≥ 5 µm, (F) hot dip galvanized ≥ 45 µm Lock nut: Electroplated zinc coated ≥ 5 µm, (F) hot dip galvanized ≥ 45 µm
Metal parts made of stainless steel	
Threaded rod, HIT-V-R	For ≤ M24: strength class 70, f_{uk} = 700 N/mm², f_{yk} = 450 N/mm²; For > M24: strength class 50, f_{uk} = 500 N/mm², f_{yk} = 210 N/mm²; Elongation at fracture (l_0=5d) > 8% ductile Stainless steel 1.4401, 1.4404, 1.4578, 1.4571, 1.4439, 1.4362 EN 10088-1:2014
Hilti tension anchor HZA-R	Round steel with threaded part: Stainless steel 1.4404, 1.4362, 1.4571 EN 10088-1:2014 Rebar: Bars class B according to NDP or NCL of EN 1992-1-1/NA:2013
Internally threaded sleeve HIS-RN	Stainless steel 1.4401, 1.4571 EN 10088-1:2014
Washer	Stainless steel 1.4401, 1.4404, 1.4578, 1.4571, 1.4439, 1.4362 EN 10088-1:2014
Nut	Strength class of nut adapted to strength class of threaded rod Stainless steel 1.4401, 1.4404, 1.4578, 1.4571, 1.4439, 1.4362 EN 10088-1:2014

Injection System Hilti HIT-HY 200-R	**Annex A5**
Product description Materials	

Metal parts made of high corrosion resistant steel	
Threaded rod HIT-V-HCR	For ≤ M20: f_{uk} = 800 N/mm², f_{yk} = 640 N/mm², For > M20: f_{uk} = 700 N/mm², f_{yk} = 400 N/mm², Elongation at fracture (l_0=5d) > 8% ductile High corrosion resistant steel 1.4529, 1.4565 EN 10088-1:2014
Washer	High corrosion resistant steel 1.4529, 1.4565 EN 10088-1:2014
Nut	Strength class of nut adapted to strength class of threaded rod High corrosion resistant steel 1.4529, 1.4565 EN 10088-1:2014

Injection System Hilti HIT-HY 200-R	**Annex A6**
Product description Materials	

Specifications of intended use

Anchorages subject to:
- Static and quasi static loading
- Seismic performance category C1 and C2 (see Table B1).

Base material:-
- Reinforced or unreinforced normal weight concrete according to EN 206-1:2000.
- Strength classes C20/25 to C50/60 according to EN 206-1:2000.
- Cracked and non-cracked concrete.

Temperature in the base material:
- **at installation**
  -10 °C to +40 °C
- **in-service**
  Temperature range I: -40 °C to +40 °C
  (max. long term temperature +24 °C and max. short time temperature +40 °C)
  Temperature range II: -40 °C to +80 °C
  (max. long term temperature +50 °C and max. short time temperature +80 °C)
  Temperature range III: -40 °C to +120 °C
  (max. long term temperature +72 °C and max. short time temperature +120 °C)

Table B1: Specifications of intended use

	HIT-HY 200-R with ...			
Elements	HIT-V ... AM 8.8	Rebar	HZA(-R)	HIS-(R)N
Hammer drilling with hollow drill bit TE-CD or TE-YD	✓	✓	✓	✓
Hammer drilling	✓	✓	✓	✓
Static and quasi static loading in cracked and non-cracked concrete	M8 to M30	ϕ 8 to ϕ 32	M12 to M27	M8 to M20
Seismic performance category C1	M10 to M30	ϕ 10 to ϕ 32	M12 to M27	-
Seismic performance category C2	M16 to M24, HIT-V 8.8, AM 8.8 HIT-V-F 8.8, AM HDG 8.8 Commercial standard rod (electroplated zinc coated only)	-	-	-

Injection System Hilti HIT-HY 200-R	Annex B1
Intended Use Specifications	

Use conditions (Environmental conditions):

- Structures subject to dry internal conditions (zinc coated steel, stainless steel or high corrosion resistant steel).
- Structures subject to external atmospheric exposure (including industrial and marine environment) and to permanently damp internal conditions, if no particular aggressive conditions exist (stainless steel or high corrosion resistant steel).
- Structures subject to external atmospheric exposure and to permanently damp internal conditions, if other particular aggressive conditions exist (high corrosion resistant steel).
  Note: Particular aggressive conditions are e.g. permanent, alternating immersion in seawater or the splash zone of seawater, chloride atmosphere of indoor swimming pools or atmosphere with extreme chemical pollution (e.g. in desulphurization plants or road tunnels where de-icing products are used).

Design:

- Anchorages are designed under the responsibility of an engineer experienced in anchorages and concrete work.
- Verifiable calculation notes and drawings are prepared taking account of the loads to be anchored. The position of the anchor is indicated on the design drawings (e. g. position of the anchor relative to reinforcement or to supports, etc.).
- Anchorages under static or quasi-static loading are designed in accordance with:
  "EOTA Technical Report TR 029, Edition September 2010"
- Anchorages under seismic actions (cracked concrete) are designed in accordance with:
  "EOTA Technical Report TR 045, Edition February 2013"
  Anchorages shall be positioned outside of critical regions (e.g. plastic hinges) of the concrete structure. Fastenings in stand-off installation or with a grout layer under seismic action are not covered in this European Technical Assessment (ETA).

Installation:

- Use category: dry or wet concrete (not in flooded holes)
- Overhead installation is admissible
- Anchor installation carried out by appropriately qualified personnel and under the supervision of the person responsible for technical matters of the site.

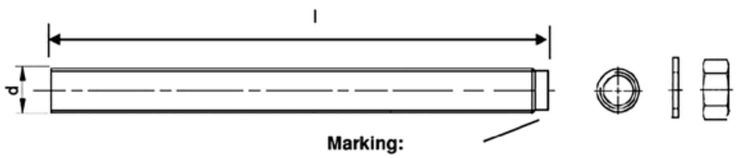
Injection System Hilti HIT-HY 200-R	**Annex B2**
Intended Use Specifications	

Table B2: Installation parameters of threaded rod and HIT-V-... and AM 8.8

Threaded rod and HIT-V-... AM 8.8			M8	M10	M12	M16	M20	M24	M27	M30
Diameter of element	d	[mm]	8	10	12	16	20	24	27	30
Nominal diameter of drill bit	d_0	[mm]	10	12	14	18	22	28	30	35
Effective embedment depth and drill hole depth	$h_{ef} = h_0$	[mm]	60 to 160	60 to 200	70 to 240	80 to 320	90 to 400	96 to 480	108 to 540	120 to 600
Maximum diameter of clearance hole in the fixture 1)	d_f	[mm]	9	12	14	18	22	26	30	33
Effective fixture thickness with seismic filling set $t_{fix,eff} = t_{fix} - h_{fs}$	h_{fs}	[mm]	-	-	-	11	13	15	-	-
Minimum thickness of concrete member	h_{min}	[mm]	$h_{ef} + 30$ ≥ 100 mm			$h_{ef} + 2 \cdot d_0$				
Maximum torque moment	T_{max}	[Nm]	10	20	40	80	150	200	270	300
Minimum spacing	s_{min}	[mm]	40	50	60	75	90	115	120	140
Minimum edge distance	c_{min}	[mm]	40	45	45	50	55	60	75	80

1) for larger clearance hole see "TR 029 section 1.1"

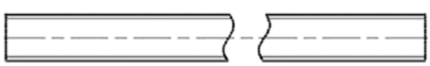
HIT-V-...



Marking:

5.8 - l = HIT-V-5.8 M...x l
5.8F - l = HIT-V-5.8F M...x l
8.8 - l = HIT-V-8.8 M...x l
8.8F - l = HIT-V-8.8F M...x l
R - l = HIT-V-R M ...x l
HCR - l = HIT-V-HCR M ...x l

Hilti meter rod AM (HDG) 8.8



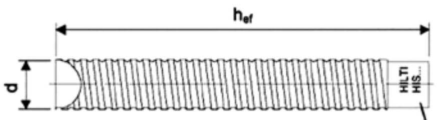
Injection System Hilti HIT-HY 200-R	**Annex B3**
Intended Use Installation parameters of threaded rod and HIT-V-... and AM 8.8	

Table B3: Installation parameters of internally threaded sleeve HIS-(R)N

Internally threaded sleeve HIS-(R)N...			M8	M10	M12	M16	M20
Outer diameter of sleeve	d	[mm]	12,5	16,5	20,5	25,4	27,6
Nominal diameter of drill bit	d_0	[mm]	14	18	22	28	32
Effective embedment depth and drill hole depth	$h_{ef} = h_0$	[mm]	90	110	125	170	205
Maximum diameter of clearance hole in the fixture [1)]	d_f	[mm]	9	12	14	18	22
Minimum thickness of concrete member	h_{min}	[mm]	120	150	170	230	270
Maximum torque moment	T_{max}	[Nm]	10	20	40	80	150
Thread engagement length min-max	h_s	[mm]	8-20	10-25	12-30	16-40	20-50
Minimum spacing	s_{min}	[mm]	60	75	90	115	130
Minimum edge distance	c_{min}	[mm]	40	45	55	65	90

[1)] for larger clearance hole see "TR 029 section 1.1"

Internally threaded sleeve HIS-(R)N...



Marking:
Identifying mark - HILTI and
embossing "HIS-N" (for C-steel)
embossing "HIS-RN" (for stainless steel)

Injection System Hilti HIT-HY 200-R	**Annex B4**
Intended Use Installation parameters of internally threaded sleeve HIS-(R)N	

Table B4: Installation parameters of Hilti tension anchor HZA-R

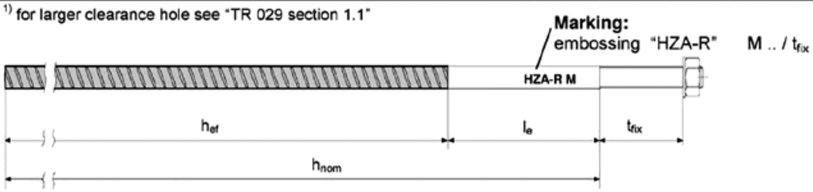
Hilti tension anchor HZA-R ...			M12	M16	M20	M24
Rebar diameter	ϕ	[mm]	12	16	20	25
Nominal embedment depth and drill hole depth	$h_{nom} = h_0$	[mm]	170 to 240	180 to 320	190 to 400	200 to 500
Effective embedment depth ($h_{ef} = h_{nom} - l_e$)	h_{ef}	[mm]	$h_{nom} - 100$			
Length of smooth shaft	l_e	[mm]	100			
Nominal diameter of drill bit	d_0	[mm]	16	20	25	32
Maximum diameter of clearance hole in the fixture [1)]	d_f	[mm]	14	18	22	26
Maximum torque moment	T_{max}	[Nm]	40	80	150	200
Minimum thickness of concrete member	h_{min}	[mm]	$h_{nom} + 2 \cdot d_0$			
Minimum spacing	s_{min}	[mm]	65	80	100	130
Minimum edge distance	c_{min}	[mm]	45	50	55	60

[1)] for larger clearance hole see "TR 029 section 1.1"

Table B5: Installation parameters of Hilti tension anchor HZA

Hilti tension anchor HZA...			M12	M16	M20	M24	M27
Rebar diameter	ϕ	[mm]	12	16	20	25	28
Nominal embedment depth and drill hole depth	$h_{nom} = h_0$	[mm]	90 to 240	100 to 320	110 to 400	120 to 500	140 to 560
Effective embedment depth ($h_{ef} = h_{nom} - l_e$)	h_{ef}	[mm]	$h_{nom} - 20$				
Length of smooth shaft	l_e	[mm]	20				
Nominal diameter of drill bit	d_0	[mm]	16	20	25	32	35
Maximum diameter of clearance hole in the fixture [1)]	d_f	[mm]	14	18	22	26	30
Maximum torque moment	T_{max}	[Nm]	40	80	150	200	270
Minimum thickness of concrete member	h_{min}	[mm]	$h_{nom} + 2 \cdot d_0$				
Minimum spacing	s_{min}	[mm]	65	80	100	130	140
Minimum edge distance	c_{min}	[mm]	45	50	55	60	75

[1)] for larger clearance hole see "TR 029 section 1.1"



Injection System Hilti HIT-HY 200-R	Annex B5
Intended Use Installation parameters of Hilti tension anchor HZA-(R)	

Table B6: Installation parameters of reinforcing bar

Reinforcing bar (rebar)			ϕ 8	ϕ 10	ϕ 12	ϕ 14	ϕ 16	ϕ 20	ϕ 25	ϕ 26	ϕ 28	ϕ 30	ϕ 32
Diameter	ϕ	[mm]	8	10	12	14	16	20	25	26	28	30	32
Effective embedment depth and drill hole depth	$h_{ef} = h_0$	[mm]	60 to 160	60 to 200	70 to 240	75 to 280	80 to 320	90 to 400	100 to 500	104 to 520	112 to 560	120 to 600	128 to 640
Nominal diameter of drill bit	d_0	[mm]	10 / 12[1]	12 / 14[1]	14[1] | 16[1]	18	20	25	32	32	35	37	40
Minimum thickness of concrete member	h_{min}	[mm]	h_{ef} + 30 ≥ 100 mm		$h_{ef} + 2 \cdot d_0$								
Minimum spacing	s_{min}	[mm]	40	50	60	70	80	100	125	130	140	150	160
Minimum edge distance	c_{min}	[mm]	40	45	45	50	50	65	70	75	75	80	80

[1] Each of the two given values can be used.

Reinforcing bar

For rebar bolt

- Minimum value of related rip area $f_{R,min}$ according to EN 1992-1-1:2004+AC:2010
- Rib height of the bar h_{rib} shall be in the range $0{,}05 \cdot \phi \leq h_{rib} \leq 0{,}07 \cdot \phi$
  (ϕ: Nominal diameter of the bar; h_{rib}: Rib height of the bar)

Injection System Hilti HIT-HY 200-R	**Annex B6**
Intended Use Installation parameters of reinforcing bar (rebar)	

Table B7: Maximum working time and minimum curing time

Temperature in the base material T	Maximum working time t_{work}	Minimum curing time t_{cure}
-10 °C to -5 °C	3 hours	20 hours
> -5 °C to 0 °C	2 hours	8 hours
> 0 °C to 5 °C	1 hour	4 hours
> 5 °C to 10 °C	40 min	2,5 hours
> 10 °C to 20 °C	15 min	1,5 hours
> 20 °C to 30 °C	9 min	1 hours
> 30 °C to 40 °C	6 min	1 hours

Injection System Hilti HIT-HY 200-R	Annex B7
Intended Use Maximum working time and minimum curing time	

Table B8: Parameters of cleaning and setting tools

Elements				Drill and clean			Installatic
Threaded rod, HIT-V-..., AM 8.8	HIS-(R)N	Rebar	HZA(-R)	Hammer drilling		Brush	Piston plu
					hollow drill bit		
							
size	size	size	size	d_0 [mm]	d_0 [mm]	HIT-RB	HIT-SZ
M8	-	ϕ8	-	10	-	10	-
M10	-	ϕ8 / ϕ10	-	12	12 [1)]	12	12
M12	M8	ϕ10 / ϕ12	-	14	14 [1)]	14	14
-	-	ϕ12	M12	16	16	16	16
M16	M10	ϕ14	-	18	18	18	18
-	-	ϕ16	M16	20	20	20	20
M20	M12	-	-	22	22	22	22
-	-	ϕ20	M20	25	25	25	25
M24	M16	-	-	28	28	28	28
M27	-	-	-	30	-	30	30
-	M20	ϕ25 / ϕ26	M24	32	32	32	32
M30	-	ϕ28	M27	35	35	35	35
-	-	ϕ30	-	37	-	37	37
-	-	ϕ32	-	40	-	40	40

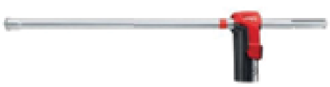
1) To be used in combination with Hilti vacuum cleaner with suction volume ≥ 61 l/s (VC 20/40 –Y in corded mode only

Cleaning alternatives

Manual Cleaning (MC):
Hilti hand pump for blowing out drill holes with diameters $d_0 \leq 20$ mm and drill hole depths $h_0 \leq 10 \cdot d$

Compressed air cleaning (CAC):
Air nozzle with an orifice opening of minimum 3,5 mm in diameter.

Automatic Cleaning (AC):
Cleaning is performed during drilling with Hilti TE-CD and TE-YD drilling system including vacuum cleaner.

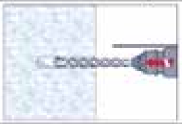


Injection System Hilti HIT-HY 200-R	Annex B8
Intended Use Cleaning and setting tools	

Installation instruction

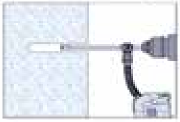
Hole drilling

a) Hammer drilling



Drill hole to the required embedment depth with a hammer drill set in rotation-hammer mode using an appropriately sized carbide drill bit.

b) Hammer drilling with Hilti hollow drill bit

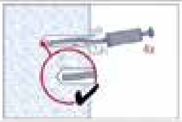


Drill hole to the required embedment depth with an appropriately sized Hilti TE-CD or TE-YD hollow drill bit attached to Hilti vacuum cleaner VC 20/40 (-Y) (suction volume ≥ 57 l/s) with automatic cleaning of the filter activated. This drilling system removes the dust and cleans the bore hole during drilling when used in accordance with the user's manual. When using TE-CD size 12 and 14 refer to Table B8.

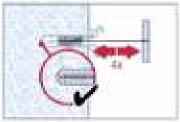
After drilling is completed, proceed to the "injection preparation" step in the installation instruction.

Drill hole cleaning — Just before setting an anchor, the drill hole must be free of dust and debris.

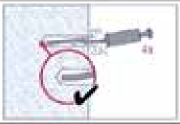
Manual Cleaning (MC) — **non-cracked concrete only**
for drill hole diameters $d_0 \leq 20$ mm and drill hole depths $h_0 \leq 10 \cdot d$



The Hilti hand pump may be used for blowing out drill holes up to diameters $d_0 \leq 20$ mm and embedment depths up to $h_{ef} \leq 10 \cdot d$.
Blow out at least 4 times from the back of the drill hole until return air stream is free of noticeable dust



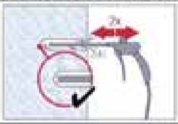
Brush 4 times with the specified brush (see Table B8) by inserting the steel brush Hilti HIT-RB to the back of the hole (if needed with extension) in a twisting motion and removing it.
The brush must produce natural resistance as it enters the drill hole (brush Ø ≥ drill hole Ø) - if not the brush is too small and must be replaced with the proper brush diameter.



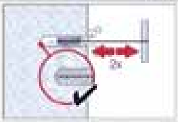
Blow out again with Hilti hand pump at least 4 times until return air stream is free of noticeable dust.

Injection System Hilti HIT-HY 200-R	**Annex B9**
Intended Use Installation instructions	

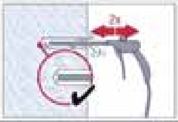
Compressed air cleaning (CAC) for all drill hole diameters d_0 and all drill hole depths h_0



Blow 2 times from the back of the hole (if needed with nozzle extension) over the hole length with oil-free compressed air (min. 6 bar at 6 m³/h) until return air stream is free of noticeable dust.
For drill hole diameters ≥ 32 mm the compressor has to supply a minimum air flow of 140 m³/h.



Brush 2 times with the specified brush (see Table B8) by inserting the steel brush Hilti HIT-RB to the back of the hole (if needed with extension) in a twisting motion and removing it.
The brush must produce natural resistance as it enters the drill hole (brush Ø ≥ drill hole Ø) - if not the brush is too small and must be replaced with the proper brush diameter.



Blow again with compressed air 2 times until return air stream is free of noticeable dust.

Injection preparation

Tightly attach new Hilti mixing nozzle HIT-RE-M to foil pack manifold (snug fit). Do not modify the mixing nozzle.
Observe the instruction for use of the dispenser.
Check foil pack holder for proper function. Do not use damaged foil packs / holders.
Insert foil pack into foil pack holder and put holder into the dispenser.

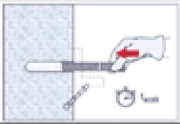
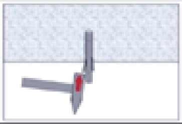
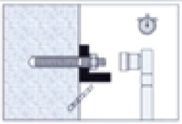
Discard initial adhesive. The foil pack opens automatically as dispensing is initiated. Depending on the size of the foil pack an initial amount of adhesive has to be discarded.
Discarded quantities are

2 strokes	for 330 ml foil pack,
3 strokes	for 500 ml foil pack,
4 strokes	for 500 ml foil pack ≤ 5 °C.

Injection System Hilti HIT-HY 200-R

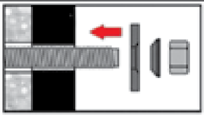
Intended Use
Installation instructions

Annex B10

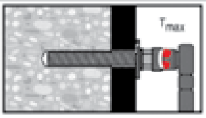
Inject adhesive from the back of the drill hole without forming air voids.	
	Inject the adhesive starting at the back of the hole, slowly withdrawing the mixer with each trigger pull. Fill holes approximately 2/3 full, to ensure that the annular gap between the anchor and the concrete is completely filled with adhesive along the embedment length.
	After injection is completed, depressurize the dispenser by pressing the release trigger. This will prevent further adhesive discharge from the mixer.
	Overhead installation and/or installation with embedment depth h_{ef} > 250mm. For overhead installation the injection is only possible with the aid of extensions and piston plugs. Assemble HIT-RE-M mixer, extension(s) and appropriately sized piston plug (see Table B8). Insert piston plug to back of the hole and inject adhesive. During injection the piston plug will be naturally extruded out of the drill hole by the adhesive pressure
Setting the element	
	Before use, verify that the element is dry and free of oil and other contaminants. Mark and set element to the required embedment depth until working time t_{work} has elapsed. The working time t_{work} is given in Table B7.
	For overhead installation use piston plugs and fix embedded parts with e.g. wedges.
	Loading the anchor: After required curing time t_{cure} (see Table B7) the anchor can be loaded. The applied installation torque shall not exceed the values T_{max} given in Table B2 to Table B5.

Injection System Hilti HIT-HY 200-R	**Annex B11**
Intended Use Installation instructions	

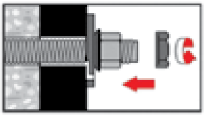
Installation of Filling Set



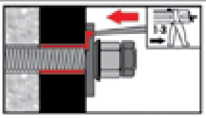
Use Hilti filling set with standard nut. Observe the correct orientation of filling washer and spherical washer.



The applied installation torque shall not exceed the values T_{max} given in Table B2 to Table B5.



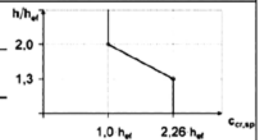
Optional:
Installation of lock nut. Tighten with a ¼ to ½ turn. (Not for size M24.)



Fill the annular gap between the anchor rod and fixture with 1-3 strokes of Hilti injection mortar HIT-HY 200 R.
Follow the installation instructions supplied with the HIT-HY 200 A foil pack.

Injection System Hilti HIT-HY 200-R	**Annex B12**
Intended Use Installation instructions	

Table C1: Characteristic values of resistance for threaded rod, HIT-V-... and AM 8.8 under tension loads in concrete

HIT-HY 200-R with threaded rod, HIT-V-..., AM 8.8			M8	M10	M12	M16	M20	M24	M27	M30
Installation safety factor	γ_2	[-]	1,0							
Steel failure										
Characteristic steel resistance	$N_{Rk,s}$	[kN]	$A_s \cdot f_{uk}$							
Partial safety factor grade 5.8	$\gamma_{Ms,N}$ 1)	[-]	1,5							
Partial safety factor grade 8.8	$\gamma_{Ms,N}$ 1)	[-]	1,5							
Partial safety factor HIT-V-R	$\gamma_{Ms,N}$ 1)	[-]	1,86						2,86	
Partial safety factor HIT-V-HCR	$\gamma_{Ms,N}$ 1)	[-]	1,5					2,1		
Combined pullout and concrete cone failure										
Characteristic bond resistance in non-cracked concrete C20/25										
Temperature range I: 40 °C/24 °C	$\tau_{Rk,ucr}$	[N/mm²]	18							
Temperature range II: 80 °C/50 °C	$\tau_{Rk,ucr}$	[N/mm²]	15							
Temperature range III: 120 °C/72 °C	$\tau_{Rk,ucr}$	[N/mm²]	13							
Characteristic bond resistance in cracked concrete C20/25										
Temperature range I: 40 °C/24 °C	$\tau_{Rk,cr}$	[N/mm²]	7,5		8,5			9,0		
Temperature range II: 80 °C/50 °C	$\tau_{Rk,cr}$	[N/mm²]	6,0		7,0			7,5		
Temperature range III: 120 °C/72 °C	$\tau_{Rk,cr}$	[N/mm²]	5,5		6,0			6,5		
Increasing factors for τ_{Rk} in concrete	ψ_c	C30/37	1,04							
		C40/45	1,07							
		C50/60	1,1							
Splitting failure										
Edge distance $c_{cr,sp}$ [mm] for		$h / h_{ef} \geq 2{,}0$	$1{,}0 \cdot h_{ef}$							
		$2{,}0 > h / h_{ef} > 1{,}3$	$4{,}6\ h_{ef} - 1{,}8\ h$							
		$h / h_{ef} \leq 1{,}3$	$2{,}26\ h_{ef}$							
Spacing	$s_{cr,sp}$	[mm]	$2 \cdot c_{cr,sp}$							

1) In absence of national regulations.

Injection System Hilti HIT-HY 200-R	**Annex C1**
Performances Characteristic values of resistance under tension loads in concrete Design according to „EOTA Technical Report TR 029, Edition September 2010“	

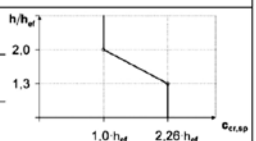
Table C2: Characteristic values of resistance for threaded rod, HIT-V-... and AM 8.8 under shear loads

HIT-HY 200-R with threaded rod, HIT-V-..., AM 8.8			M8	M10	M12	M16	M20	M24	M27	M30
Steel failure without lever arm										
Characteristic steel resistance	$V_{Rk,s}$	[kN]	$0{,}5 \cdot A_s \cdot f_{uk}$							
Partial safety factor grade 5.8	$\gamma_{Ms,V}$[1]	[-]	1,25							
Partial safety factor grade 8.8	$\gamma_{Ms,V}$[1]	[-]	1,25							
Partial safety factor HIT-V-R	$\gamma_{Ms,V}$[1]	[-]	1,56						2,38	
Partial safety factor HIT-V-HCR	$\gamma_{Ms,V}$[1]	[-]	1,25					1,75		
Steel failure with lever arm										
Characteristic bending moment	$M^0_{Rk,s}$	[Nm]	$1{,}2 \cdot W_{el} \cdot f_{uk}$							
Concrete pry-out failure										
Factor in equation (5.7) of Technical Report TR 029 for the design of bonded anchors	k	[-]	2,0							
Concrete edge failure										
The value of h_{ef} for calculation in equations (5.8a) and (5.8b) of Technical Report TR 029 is limited by:			min (h_{ef}; $12 \cdot d_{nom}$)							
Outside diameter of anchor	d_{nom}	[mm]	8	10	12	16	20	24	27	30

[1] In absence of national regulations.

Injection System Hilti HIT-HY 200-R	Annex C2
Performances Characteristic values of resistance under shear loads in concrete Design according to „EOTA Technical Report TR 029, Edition September 2010"	

Table C3: Characteristic values of resistance for internally threaded sleeve HIS-(R)N under tension loads in concrete

Hilti HIT-HY 200-R with HIS-(R)N			**M8**	**M10**	**M12**	**M16**	**M20**
Installation safety factor	γ_2	[-]	1,0				
Steel failure threaded rods							
Characteristic resistance HIS-N with screw grade 8.8	$N_{Rk,s}$	[kN]	25	46	67	125	116
Partial safety factor	$\gamma_{Ms,N}$ 1)	[-]	1,50				
Characteristic resistance HIS-RN with screw grade 70	$N_{Rk,s}$	[kN]	26	41	59	110	166
Partial safety factor	$\gamma_{Ms,N}$ 1)	[-]	1,87				2,4
Combined pull-out and Concrete cone failure							
Effective anchorage depth	h_{ef}	[mm]	90	110	125	170	205
Effective anchor diameter	d_1	[mm]	12,5	16,5	20,5	25,4	27,6
Characteristic bond resistance in non-cracked concrete C20/25							
Temperature range I: 40 °C/24 °C	$\tau_{Rk,ucr}$	[N/mm²]	13				
Temperature range II: 80 °C/50 °C	$\tau_{Rk,ucr}$	[N/mm²]	11				
Temperature range III: 120 °C/72 °C	$\tau_{Rk,ucr}$	[N/mm²]	9,5				
Characteristic bond resistance in cracked concrete C20/25							
Temperature range I: 40 °C/24 °C	$\tau_{Rk,cr}$	[N/mm²]	7				
Temperature range II: 80 °C/50 °C	$\tau_{Rk,cr}$	[N/mm²]	5,5				
Temperature range III: 120 °C/72 °C	$\tau_{Rk,cr}$	[N/mm²]	5				
Increasing factor for τ_{Rk} in concrete	ψ_c	C30/37	1,04				
		C40/45	1,07				
		C50/60	1,1				
Splitting failure relevant for non-cracked concrete							
Edge distance $c_{cr,sp}$ [mm] for	$h / h_{ef} \geq 2{,}0$		$1{,}0 \cdot h_{ef}$				
	$2{,}0 > h / h_{ef} > 1{,}3$		$4{,}6 \cdot h_{ef} - 1{,}8 \cdot h$				
	$h / h_{ef} \leq 1{,}3$		$2{,}26 \cdot h_{ef}$				
Spacing	$s_{cr,sp}$	[mm]	$2 \cdot c_{cr,sp}$				

1) In absence of national regulations.

Injection System Hilti HIT-HY 200-R	**Annex C3**
Performances Characteristic values of resistance under tension loads in concrete Design according to „EOTA Technical Report TR 029, Edition September 2010“	

Table C4: Characteristic values of resistance for internally threaded sleeve HIS-(R)N under shear loads in concrete

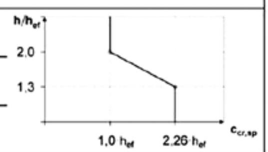
Hilti HIT-HY 200-R with HIS-(R)N			M8	M10	M12	M16	M20
Steel failure without lever arm							
Characteristic resistance HIS-N with screw grade 8.8	$V_{Rk,s}$	[kN]	13	23	34	63	58
Partial safety factor	$\gamma_{Ms,V}$ 1)	[-]	1,25				
Characteristic resistance HIS-RN with screw grade 70	$V_{Rk,s}$	[kN]	13	20	30	55	83
Partial safety factor	$\gamma_{Ms,V}$ 1)	[-]	1,56				2,0
Steel failure with lever arm							
Characteristic resistance HIS-N with screw grade 8.8	$M^0_{Rk,s}$	[Nm]	30	60	105	266	519
Partial safety factor	$\gamma_{Ms,V}$ 1)	[-]	1,25				
Characteristic resistance HIS-RN with screw grade 70	$M^0_{Rk,s}$	[Nm]	26	52	92	233	454
Partial safety factor	$\gamma_{Ms,V}$ 1)	[-]	1,56				
Concrete pry-out failure							
Factor in equation (5.7) of Technical Report TR 029 for the design of bonded anchors	k	[-]	2,0				
Concrete edge failure							
Outside diameter of anchor	d_{nom}	[mm]	12,5	16,5	20,5	25,4	27,6

1) In absence of national regulations.

Injection System Hilti HIT-HY 200-R	Annex C4
Performances Characteristic values of resistance under shear loads in concrete Design according to „EOTA Technical Report TR 029, Edition September 2010"	

Table C5: Characteristic values of resistance for Hilti tension anchor HZA / HZA-R under tension loads in concrete

Hilti HIT-HY 200-R with HZA, HZA-R			**M12**	**M16**	**M20**	**M24**	**M27**
Installation safety factor	γ_2	[-]	1,0				
Steel failure							
Characteristic resistance HZA	$N_{Rk,s}$	[kN]	46	86	135	194	253
Characteristic resistance HZA-R	$N_{Rk,s}$	[kN]	62	111	173	248	-
Partial safety factor	γ_{Ms} [1)]	[-]	1,4				
Combined pull-out and concrete cone failure							
Diameter of rebar	d	[mm]	12	16	20	25	28
Characteristic bond resistance in non-cracked concrete C20/25							
Temperature range I: 40 °C/24 °C	$\tau_{Rk,ucr}$	[N/mm²]	12				
Temperature range II: 80 °C/50 °C	$\tau_{Rk,ucr}$	[N/mm²]	10				
Temperature range III: 120 °C/72 °C	$\tau_{Rk,ucr}$	[N/mm²]	8,5				
Characteristic bond resistance in cracked concrete C20/25							
Temperature range I: 40 °C/24 °C	$\tau_{Rk,cr}$	[N/mm²]	7				
Temperature range II: 80 °C/50 °C	$\tau_{Rk,cr}$	[N/mm²]	5,5				
Temperature range III: 120 °C/72 °C	$\tau_{Rk,cr}$	[N/mm²]	5				
Increasing factor for τ_{Rk} in concrete	ψ_c	C30/37	1,04				
		C40/45	1,07				
		C50/60	1,1				
Effective anchorage depth for calculation of $N^0_{Rk,p}$ acc. Eq. 5.2a (TR 029, 5.2.2.3 Combined pull-out and concrete cone failure)	HZA h_{ef}	[mm]	$h_{nom} - 20$				
	HZA-R h_{ef}	[mm]	$h_{nom} - 100$				-
Concrete cone failure							
Effective anchorage depth for calculation of $N^0_{Rk,c}$ acc. Eq. 5.3a (TR 029, 5.2.2.4 Concrete cone failure)	HZA HZA-R h_{ef}	[mm]	h_{nom}				
Splitting failure relevant for non-cracked concrete							
Edge distance $c_{cr,sp}$ [mm] for	$h / h_{ef} \geq 2{,}0$		$1{,}0 \cdot h_{ef}$				
	$2{,}0 > h / h_{ef} > 1{,}3$		$4{,}6 \cdot h_{ef} - 1{,}8 \cdot h$				
	$h / h_{ef} \leq 1{,}3$		$2{,}26 \cdot h_{ef}$				
Spacing	$s_{cr,sp}$	[mm]	$2 \cdot c_{cr,sp}$				



[1)] In absence of national regulations.

Injection System Hilti HIT-HY 200-R

Performances
Characteristic values of resistance under tension loads in concrete
Design according to „EOTA Technical Report TR 029, Edition September 2010"

Annex C5

Table C6: Characteristic values of resistance for Hilti tension anchor HZA, HZA-R under shear loads in concrete

Hilti HIT-HY 200-R with HZA, HZA-R			**M12**	**M16**	**M20**	**M24**	**M27**
Steel failure without lever arm							
Characteristic resistance HZA	$V_{Rk,s}$	[kN]	23	43	67	97	126
Characteristic resistance HZA-R	$V_{Rk,s}$	[kN]	31	55	86	124	-
Partial safety factor	γ_{Ms} [1]	[-]	1,5				
Steel failure with lever arm							
Characteristic resistance HZA	$M^0_{Rk,s}$	[Nm]	72	183	357	617	915
Characteristic resistance HZA-R	$M^0_{Rk,s}$	[Nm]	97	234	457	790	-
Partial safety factor	γ_{Ms} [1]	[-]	1,5				
Concrete pry-out failure							
Factor in equation (5.7) of Technical Report TR 029 for the design of bonded anchors	k	[-]	2,0				
Concrete edge failure							
The value of h_{ef} for calculation in equations (5.8a) and (5.8b) of Technical Report TR 029 is limited by:			min (h_{nom}; 12 · d_{nom})				
Outside diameter of anchor	d_{nom}	[mm]	12	16	20	24	27

[1] In absence of national regulations.

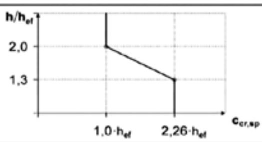
Injection System Hilti HIT-HY 200-R

Performances
Characteristic values of resistance under shear loads in concrete
Design according to „EOTA Technical Report TR 029, Edition September 2010"

Annex C6

Table C7: Characteristic values of resistance for rebar under tension loads in concrete

HIT-HY 200-R with rebar			ϕ 8	ϕ 10	ϕ 12	ϕ 14	ϕ 16	ϕ 20	ϕ 25	ϕ 26	ϕ 28	ϕ 30	ϕ 32
Installation safety factor	γ_2	[-]	1,0										
Steel failure													
Characteristic resistance for rebar B500B acc. to DIN 488:2009-08 [2]	$N_{Rk,s}$	[kN]	28	43	62	85	111	173	270	292	339	388	442
Partial safety factor [3]	$\gamma_{Ms,N}$ [1]	[-]	1,4										
Combined pull-out and Concrete cone failure													
Diameter of rebar	d	[mm]	8	10	12	14	16	20	25	26	28	30	32
Characteristic bond resistance in non-cracked concrete C20/25													
Temperature range I: 40°C/24°C	$\tau_{Rk,ucr}$	[N/mm²]	12										
Temperature range II: 80°C/50°C	$\tau_{Rk,ucr}$	[N/mm²]	10										
Temperature range III: 120°C/72°C	$\tau_{Rk,ucr}$	[N/mm²]	8,5										
Characteristic bond resistance in cracked concrete C20/25													
Temperature range I: 40°C/24°C	$\tau_{Rk,cr}$	[N/mm²]	-	5	7								
Temperature range II: 80°C/50°C	$\tau_{Rk,cr}$	[N/mm²]	-	4	5,5								
Temperature range III: 120°C/72°C	$\tau_{Rk,cr}$	[N/mm²]	-	3,5	5								
Increasing factor for τ_{Rk} in concrete	ψ_c	C30/37	1,04										
		C40/45	1,07										
		C50/60	1,1										
Splitting failure relevant for non-cracked concrete													
Edge distance $c_{cr,sp}$ [mm] for	$h / h_{ef} \geq 2,0$		$1,0 \cdot h_{ef}$										
	$2,0 > h / h_{ef} > 1,3$		$4,6 \cdot h_{ef} - 1,8 \cdot h$										
	$h / h_{ef} \leq 1,3$		$2,26 \cdot h_{ef}$										
Spacing	$s_{cr,sp}$	[mm]	$2\ c_{cr,sp}$										



[1] In absence of nationa regulations
[2] The characteristic tension resistance $N_{Rk,s}$ for rebars that do not fulfill the requirements acc. to DIN 488 shall be calculated acc. Technical Report TR 029, Equation (5.1).
[3] The partial safety factor $\gamma_{Ms,N}$ that do not fulfill the requirements acc. to DIN 488 shall be calculated acc. Technical Report TR 029, Equation (3.3a).

Injection System Hilti HIT-HY 200-R	**Annex C7**
Performances Characteristic values of resistance under tension loads in concrete Design according to „EOTA Technical Report TR 029, Edition September 2010“	

Table C8: Characteristic values of resistance for rebar under shear loads in concrete

HIT-HY 200-R with rebar			ϕ 8	ϕ 10	ϕ 12	ϕ 14	ϕ 16	ϕ 20	ϕ 25	ϕ 26	ϕ 28	ϕ 30	ϕ 32
Steel failure without lever arm													
Characteristic resistance for rebar B500B acc. to DIN 488:2009-08 [2]	$V_{Rk,s}$	[kN]	14	22	31	42	55	86	135	146	169	194	221
Partial safety factor [4]	$\gamma_{Ms,V}$ [1]	[-]	1,5										
Steel failure with lever arm													
Characteristic resistance for rebar B500B acc. to DIN 488:2009-08 [3]	$M^0_{Rk,s}$	[Nm]	33	65	112	178	265	518	1012	1139	1422	1749	2123
Concrete pry-out failure													
Factor in equation (5.7) of Technical Report TR 029 for the design of bonded anchors	k	[-]	2,0										
Concrete edge failure													
The value of h_{ef} for calculation in equations (5.8a) and (5.8b) of Technical Report TR 029 is limited by:			min (h_{ef}; 12 · d_{nom})										
Outside diameter of anchor	d_{nom}	[mm]	8	10	12	14	16	20	25	26	28	30	32

1) In absence of national regulations.
2) The characteristic shear resistance $V_{Rk,s}$ for rebars that do not fulfill the requirements acc. DIN 488 shall be calculated acc. Technical Report TR 029, Equation (5.5).
3) The characteristic bending resistance $M^0_{Rk,s}$ for rebars that do not fulfil the requirements acc. DIN 488 shall be calculated acc. Technical Report TR 029, Equation (5.6b).
4) The partial safety factor $\gamma_{Ms,V}$ for rebar that do not fulfill the requirements acc. to DIN 488 shall be calculated acc. Technical Report 029, Equation (3.3b) or (3.3c).

Injection System Hilti HIT-HY 200-R	**Annex C8**
Performances Characteristic values of resistance under shear loads in concrete Design according to „EOTA Technical Report TR 029, Edition September 2010“	

Table C9: Displacements under tension load

Hilti HIT-HY 200-R with threaded rod, HIT-V-…, AM 8.8-…			M8	M10	M12	M16	M20	M24	M27	M30
Non-cracked concrete temperature range I : 40°C / 24°C										
Displacement	δ_{N0}	[mm/(N/mm²)]	0,02	0,03	0,03	0,04	0,06	0,07	0,07	0,08
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,04	0,05	0,06	0,08	0,10	0,13	0,14	0,16
Non-cracked concrete temperature range II : 80°C / 50°C										
Displacement	δ_{N0}	[mm/(N/mm²)]	0,03	0,04	0,05	0,06	0,08	0,09	0,10	0,12
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,04	0,05	0,06	0,09	0,11	0,13	0,15	0,16
Non-cracked concrete temperature range III : 120°C / 72°C										
Displacement	δ_{N0}	[mm/(N/mm²)]	0,04	0,05	0,06	0,08	0,10	0,12	0,13	0,16
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,04	0,05	0,07	0,09	0,11	0,13	0,15	0,17
Cracked concrete temperature range I : 40°C / 24°C										
Displacement	δ_{N0}	[mm/(N/mm²)]	0,07							
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,16							
Cracked concrete temperature range II : 80°C / 50°C										
Displacement	δ_{N0}	[mm/(N/mm²)]	0,10							
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,22							
Cracked concrete temperature range III : 120°C / 72°C										
Displacement	δ_{N0}	[mm/(N/mm²)]	0,13							
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,29							

Table C10: Displacements under shear load

Hilti HIT-HY 200-R with threaded rod, HIT-V-…			M8	M10	M12	M16	M20	M24	M27	M30
Displacement	δ_{V0}	[mm/kN]	0,06	0,06	0,05	0,04	0,04	0,03	0,03	0,03
	$\delta_{V\infty}$	[mm/kN]	0,09	0,08	0,08	0,06	0,06	0,05	0,05	0,05

Injection System Hilti HIT-HY 200-R

Performances
Displacements with threaded rod, HIT-V-… and AM 8.8

Annex C9

Table C11: Displacements under tension load

Hilti HIT-HY 200-R with HIS-(R)N			M8	M10	M12	M16	M20
Non-cracked concrete temperature range I : 40°C / 24°C							
Displacement	δ_{N0}	[mm/(N/mm²)]	0,03	0,05	0,06	0,07	0,08
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,06	0,09	0,11	0,13	0,14
Non-cracked concrete temperature range II : 80°C / 50°C							
Displacement	δ_{N0}	[mm/(N/mm²)]	0,05	0,06	0,08	0,10	0,11
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,07	0,09	0,11	0,13	0,15
Non-cracked concrete temperature range III : 120°C / 72°C							
Displacement	δ_{N0}	[mm/(N/mm²)]	0,06	0,08	0,10	0,13	0,14
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,07	0,09	0,11	0,14	0,15
Cracked concrete temperature range I : 40°C / 24°C							
Displacement	δ_{N0}	[mm/(N/mm²)]	0,11				
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,16				
Cracked concrete temperature range II : 80°C / 50°C							
Displacement	δ_{N0}	[mm/(N/mm²)]	0,15				
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,22				
Cracked concrete temperature range III : 120°C / 72°C							
Displacement	δ_{N0}	[mm/(N/mm²)]	0,20				
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,29				

Table C12: Displacements under shear load

Hilti HIT-HY 200-R with HIS-(R)N			M8	M10	M12	M16	M20
Displacement	δ_{V0}	[mm/kN]	0,06	0,06	0,05	0,04	0,04
	$\delta_{V\infty}$	[mm/kN]	0,09	0,08	0,08	0,06	0,06

Injection System Hilti HIT-HY 200-R

Performances
Displacements with HIS-(R)N

Annex C10

Deutsches Institut für Bautechnik DIBt

Table C13: Displacements under tension load

Hilti HIT-HY 200-R with HZA, HZA-R			M12	M16	M20	M24	M27
Non-cracked concrete temperature range I : 40°C / 24°C							
Displacement	δ_{N0}	[mm/(N/mm²)]	0,03	0,04	0,06	0,07	0,08
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,06	0,08	0,13	0,13	0,15
Non-cracked concrete temperature range II : 80°C / 50°C							
Displacement	δ_{N0}	[mm/(N/mm²)]	0,05	0,06	0,08	0,10	0,11
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,06	0,09	0,14	0,14	0,15
Non-cracked concrete temperature range III : 120°C / 72°C							
Displacement	δ_{N0}	[mm/(N/mm²)]	0,06	0,08	0,10	0,12	0,14
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,07	0,09	0,14	0,14	0,16
Cracked concrete temperature range I : 40°C / 24°C							
Displacement	δ_{N0}	[mm/(N/mm²)]	0,11				
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,16				
Cracked concrete temperature range II : 80°C / 50°C							
Displacement	δ_{N0}	[mm/(N/mm²)]	0,15				
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,22				
Cracked concrete temperature range III : 120°C / 72°C							
Displacement	δ_{N0}	[mm/(N/mm²)]	0,20				
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,29				

Table C14: Displacements under shear load

Hilti HIT-HY 200-R with HZA, HZA-R			M12	M16	M20	M24	M27
Displacement	δ_{V0}	[mm/kN]	0,05	0,04	0,04	0,03	0,03
	$\delta_{V\infty}$	[mm/kN]	0,08	0,06	0,06	0,05	0,05

Injection System Hilti HIT-HY 200-R

Performances
Displacements with HZA and HZA-R

Annex C11

Table C15: Displacements under tension load

Hilti HIT-HY 200-R with rebar			ϕ 8	ϕ 10	ϕ 12	ϕ 14	ϕ 16	ϕ 20	ϕ 25	ϕ 26	ϕ 28	ϕ 30	ϕ 32
Non-cracked concrete temperature range I : 40°C / 24°C													
Displacement	δ_{N0}	[mm/(N/mm²)]	0,02	0,03	0,03	0,04	0,04	0,06	0,07	0,08	0,08	0,09	0,09
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,04	0,05	0,06	0,07	0,08	0,10	0,13	0,14	0,15	0,16	0,17
Non-cracked concrete temperature range II : 80°C / 50°C													
	δ_{N0}	[mm/(N/mm²)]	0,03	0,04	0,05	0,05	0,06	0,08	0,10	0,11	0,11	0,12	0,12
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,04	0,05	0,06	0,07	0,09	0,11	0,14	0,15	0,15	0,16	0,17
Non-cracked concrete temperature range III : 120°C / 72°C													
Displacement	δ_{N0}	[mm/(N/mm²)]	0,04	0,05	0,06	0,07	0,08	0,10	0,12	0,13	0,14	0,15	0,16
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,04	0,05	0,07	0,08	0,09	0,11	0,14	0,15	0,16	0,17	0,18
Cracked concrete temperature range I : 40°C / 24°C													
Displacement	δ_{N0}	[mm/(N/mm²)]	0,11										
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,16										
Cracked concrete temperature range II : 80°C / 50°C													
Displacement	δ_{N0}	[mm/(N/mm²)]	0,15										
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,22										
Cracked concrete temperature range III : 120°C / 72°C													
Displacement	δ_{N0}	[mm/(N/mm²)]	0,20										
	$\delta_{N\infty}$	[mm/(N/mm²)]	0,29										

Table C16: Displacements under shear load

Hilti HIT-HY 200-R with rebar			ϕ 8	ϕ 10	ϕ 12	ϕ 14	ϕ 16	ϕ 20	ϕ 25	ϕ 26	ϕ 28	ϕ 30	ϕ 32
Displacement	δ_{V0}	[mm/kN]	0,06	0,05	0,05	0,04	0,04	0,04	0,03	0,03	0,03	0,03	0,03
	$\delta_{V\infty}$	[mm/kN]	0,09	0,08	0,07	0,06	0,06	0,05	0,05	0,05	0,04	0,04	0,04

Injection System Hilti HIT-HY 200-R	Annex C12
Performances Displacements with rebar	

Seismic design shall be carried out according to the TR 045 „Design of Metal Anchors Under Seismic Action“

Table C17: Characteristic values of resistance for threaded rod, HIT-V-... and AM 8.8 under tension loads for seismic performance category C1

HIT-HY 200-R with threaded rod, HIT-V-..., AM 8.8			M8	M10	M12	M16	M20	M24	M27	M30
Steel failure										
HIT-V-5.8(F), threaded rod 5.8	$N_{Rk,s,seis}$	[kN]	-	29	42	79	123	177	230	281
HIT-V-8.8(F), threaded rod 8.8	$N_{Rk,s,seis}$	[kN]	-	46	67	126	196	282	367	449
HIT-V-R, threaded rod A4-70	$N_{Rk,s,seis}$	[kN]	-	41	59	110	172	247	230	281
HIT-V-HCR, threaded rod HCR-80	$N_{Rk,s,seis}$	[kN]	-	46	67	126	196	247	321	393
Combined pullout and concrete cone failure										
Characteristic bond resistance in cracked concrete C20/25										
Temperature range I: 40 °C/24 °C	$\tau_{Rk,seis}$	[N/mm²]	-	5,2	7,0					
Temperature range II: 80 °C/50 °C	$\tau_{Rk,seis}$	[N/mm²]	-	3,9	5,7					
Temperature range III: 120 °C/72 °C	$\tau_{Rk,seis}$	[N/mm²]	-	3,5	4,8					

Table C18: Characteristic values of resistance for threaded rod, HIT-V-... and AM 8.8 under shear loads for seismic performance category C1

HIT-HY 200-R with threaded rod, HIT-V-..., AM 8.8			M8	M10	M12	M16	M20	M24	M27	M30
Steel failure without lever arm										
HIT-V 5.8(F), threaded rod 5.8	$V_{Rk,s,seis}$	[kN]	-	11	15	27	43	62	81	98
HIT-V 8.8(F), threaded rod 8.8	$V_{Rk,s,seis}$	[kN]	-	16	24	44	69	99	129	157
HIT-V R, threaded rod A4-70	$V_{Rk,s,seis}$	[kN]	-	14	21	39	60	87	81	98
HIT-V HCR, threaded rod HCR-80	$V_{Rk,s,seis}$	[kN]	-	16	24	44	69	87	113	137

Table C19: Displacements under tension load for seismic performance category C1

HIT-HY 200-R with threaded rod, HIT-V-..., AM 8.8			M8	M10	M12	M16	M20	M24	M27	M30
Displacement [1]	$\delta_{N,seis}$	[mm]	-	0,8	0,8	0,8	0,8	0,8	0,8	0,8

[1] Maximum displacement during cycling (seismic event).

Table C20: Displacements under shear load for seismic performance category C1

HIT-HY 200-R with threaded rod, HIT-V-..., AM 8.8			M8	M10	M12	M16	M20	M24	M27	M30
Displacement [1]	$\delta_{V,seis}$	[mm]	-	3,5	3,8	4,4	5,0	5,6	6,1	6,5

[1] Maximum displacement during cycling (seismic event).

Injection System Hilti HIT-HY 200-R

Performances
Characteristic values for seismic performance category C1 and displacements
Design according to „EOTA Technical Report TR045, Edition February 2013 “

Annex C13

Table C21: Characteristic values of resistance for Hilti tension anchor HZA, HZA-R under tension loads for seismic performance category C1

HIT-HY 200-R with Hilti tension anchor HZA, HZA-R			M12	M16	M20	M24	M27
Steel failure							
Characteristic resistance HZA	$N_{Rk,s,seis}$	[kN]	46	86	135	194	253
Characteristic resistance HZA-R	$N_{Rk,s,seis}$	[kN]	62	111	173	248	-
Partial safety factor	$\gamma_{Ms,N,seis}$ [1]	[-]	1,4				
Combined pull-out and concrete cone failure[1]							
Diameter of rebar	d	[mm]	12	16	20	25	28
Characteristic bond resistance in cracked concrete C20/25							
Temperature range I: 40°C/24°C	$\tau_{Rk,cr}$	[N/mm²]	6,1				
Temperature range II: 80°C/50°C	$\tau_{Rk,cr}$	[N/mm²]	4,8				
Temperature range III: 120°C/72°C	$\tau_{Rk,cr}$	[N/mm²]	4,4				

[1] In absence of national regulations.

Table C22: Characteristic values of resistance for Hilti tension anchor HZA, HZA-R under shear loads for seismic performance category C1

HIT-HY 200-R with Hilti tension anchor HZA, HZA-R			M12	M16	M20	M24	M27
Steel failure without lever arm							
Characteristic resistance HZA	$V_{Rk,s,seis}$	[kN]	16	30	47	68	88
Characteristic resistance HZA-R	$V_{Rk,s,seis}$	[kN]	22	39	60	124	-
Partial safety factor	$\gamma_{Ms,V,seis}$ [1]	[-]	1,5				

[1] In absence of national regulations.

Table C23: Displacements under tension load for seismic performance category C1

HIT-HY 200-R with Hilti tension anchor HZA, HZA-R			M12	M16	M20	M24	M27
Displacement [1]	$\delta_{N,seis}$	[mm]	1,3	1,3	1,3	1,3	1,3

[1] Maximum displacement during cycling (seismic event).

Table C24: Displacements under shear load for seismic performance category C1

HIT-HY 200-R with Hilti tension anchor HZA, HZA-R			M12	M16	M20	M24	M27
Displacement [1]	$\delta_{V,seis}$	[mm]	3,8	4,4	5,0	5,6	6,1

[1] Maximum displacement during cycling (seismic event).

Injection System Hilti HIT-HY 200-R	Annex C14
Performances Characteristic values for seismic performance category C1 and displacements Design according to „EOTA Technical Report TR 045, Edition February 2013“	

Table C25: Characteristic values of resistance for rebar under tension loads for seismic performance category C1

HIT-HY 200-R with rebar			ϕ 8	ϕ 10	ϕ 12	ϕ 14	ϕ 16	ϕ 20	ϕ 25	ϕ 26	ϕ 28	ϕ 30	ϕ 32
Steel failure													
Characteristic resistance for rebar B500B acc. to DIN 488:2009-08 [1]	$N_{Rk,seis}$	[kN]	-	43	62	85	111	173	270	292	339	388	442
Combined pull-out and Concrete cone failure													
Diameter of rebar	d	[mm]	-	10	12	14	16	20	25	26	28	30	32
Characteristic bond resistance in cracked concrete C20/25													
Temperature range I: 40°C/24°C	$\tau_{Rk,cr}$	[N/mm²]	-	4,4	6,1								
Temperature range II: 80°C/50°C	$\tau_{Rk,cr}$	[N/mm²]	-	3,5	4,8								
Temperature range III: 120°C/72°C	$\tau_{Rk,cr}$	[N/mm²]	-	3	4,4								

[1] The characteristic tension resistance $N_{Rk,s,seis}$ for rebars that do not fulfil the requirements acc. DIN 488 shall be calculated acc. Technical Report TR 029, Equation (5.1), $N_{Rk,s,seis} = N_{Rk,s}$.

Table C26: Characteristic values of resistance for rebar under shear loads for seismic performance category C1

HIT-HY 200-R with rebar			ϕ 8	ϕ 10	ϕ 12	ϕ 14	ϕ 16	ϕ 20	ϕ 25	ϕ 26	ϕ 28	ϕ 30	ϕ 32
Steel failure without lever arm													
Characteristic resistance for rebar B500B acc. to DIN 488:2009-08	$V_{Rk,s,seis}$	[kN]	-	15	22	29	39	60	95	102	118	135	165

[1] The characteristic shear resistance $V_{Rk,s,seis}$ for rebars that do not fulfil the requirements acc. DIN 488 shall be calculated acc. Technical Report TR 029, Equation (5.5), $V_{Rk,s,seis} = 0,7 \times V_{Rk,s}$.

Table C27: Displacements under tension load for seismic performance category C1

Hilti HIT-HY 200-R with rebar			ϕ 8	ϕ 10	ϕ 12	ϕ 14	ϕ 16	ϕ 20	ϕ 25	ϕ 26	ϕ 28	ϕ 30	ϕ 32
Displacement [1]	$\delta_{N,seis}$	[mm]	-	1,3	1,3	1,3	1,3	1,3	1,3	1,3	1,3	1,3	1,3

[1] Maximum displacement during cycling (seismic event).

Table C28: Displacements under shear load for seismic performance category C1

Hilti HIT-HY 200-R with rebar			ϕ 8	ϕ 10	ϕ 12	ϕ 14	ϕ 16	ϕ 20	ϕ 25	ϕ 26	ϕ 28	ϕ 30	ϕ 32
Displacement [1]	$\delta_{V,seis}$	[mm]	-	3,5	3,8	4,1	4,4	5,0	5,8	6,2	6,2	6,8	6,8

[1] Maximum displacement during cycling (seismic event).

Injection System Hilti HIT-HY 200-R	Annex C15
Performances Characteristic values for seismic performance category C1 and displacements Design according to „EOTA Technical Report TR 045, Edition February 2013“	

Table C29: Characteristic values of resistance for threaded rod, HIT-V... and AM 8.8 under tension loads for seismic performance category C2

HIT-HY 200-R with threaded rod, HIT-V-..., AM 8.8			M8	M10	M12	M16	M20	M24	M27	M30
Steel failure										
HIT-V (-F) 8.8, AM (HDG) 8.8 Commercial standard threaded rod, electroplated zinc coated 8.8	$N_{Rk,s,seis}$	[kN]	-			126	196	282	-	
Combined pullout and concrete cone failure										
Characteristic bond resistance in cracked concrete C20/25 in hammer drilled holes and hammer drilled holes with Hilti hollow drill bit TE-CD or TE-YD										
Temperature range I: 40 °C/24 °C	$\tau_{Rk,seis}$	[N/mm²]	-			3,9	4,3	3,5	-	
Temperature range II: 80 °C/50 °C	$\tau_{Rk,seis}$	[N/mm²]	-			3,3	3,7	2,9	-	
Temperature range III: 120 °C/72 °C	$\tau_{Rk,seis}$	[N/mm²]	-			2,8	3,2	2,5	-	

Table C30: Characteristic values of resistance for threaded rod, HIT-V-...8.8 and AM 8.8 under shear loads for seismic performance category C2

HIT-HY 200-R with threaded rod, HIT-V-..., AM 8.8			M8	M10	M12	M16	M20	M24	M27	M30
Steel failure without lever arm with Hilti Filling set										
HIT-V 8.8, AM 8.8	$V_{Rk,s,seis}$	[kN]	-			46	77	103	-	
Steel failure without lever arm without Hilti Filling set										
HIT-V 8.8, AM 8.8	$V_{Rk,s,seis}$	[kN]	-			40	71	90	-	
HIT-V-F 8.8, AM HDG 8.8	$V_{Rk,s,seis}$	[kN]	-			30	46	66	-	
Commercial standard threaded rod, only electroplated zinc coated 8.8	$V_{Rk,s,seis}$	[kN]	-			28	50	63	-	

Injection System Hilti HIT-HY 200-R	**Annex C16**
Performances Characteristic values for seismic performance category C2 Design according to „EOTA Technical Report TR 045, Edition February 2013"	

Table C31: Displacements under tension load for seismic performance category C2

HIT-HY 200-R with threaded rod, HIT-V-..., AM 8.8			M8	M10	M12	M16	M20	M24	M27	M30
Displacement DLS, HIT-V (-F) 8.8, AM (HDG) 8.8	$\delta_{N,seis(DLS)}$	[mm]		-		0,2	0,5	0,4	-	
Displacement ULS, HIT-V (-F) 8.8, AM (HDG) 8.8	$\delta_{N,seis(ULS)}$	[mm]		-		0,6	0,8	1.0	-	

Table C32: Displacements under shear load for seismic performance category C2

HIT-HY 200-R with threaded rod, HIT-V-..., AM 8.8			M8	M10	M12	M16	M20	M24	M27	M30
Installation with Hilti Filling set										
Displacement DLS, HIT-V 8.8, AM 8.8	$\delta_{V,seis(DLS)}$	[mm]		-		1,2	1,42	1,1	-	
Displacement ULS, HIT-V 8.8, AM 8.8	$\delta_{V,seis(ULS)}$	[mm]		-		3,2	3,8	2,6	-	
Installation without Hilti Filling set										
Displacement DLS, HIT-V 8.8, AM 8.8	$\delta_{V,seis(DLS)}$	[mm]		-		3,2	2,5	3,5	-	
Displacement DLS, HIT-V-F 8.8, AM HDG 8.8	$\delta_{V,seis(DLS)}$	[mm]		-		2,3	3,8	3,7	-	
Displacement ULS, HIT-V, 8.8 AM 8.8	$\delta_{V,seis(ULS)}$	[mm]		-		9,2	7,1	10,2	-	
Displacement ULS, HIT-V-F 8.8, AM HDG 8.8	$\delta_{V,seis(ULS)}$	[mm]		-		4,3	9,1	8,4	-	

Injection System Hilti HIT-HY 200-R	**Annex C17**
Performances Displacements for seismic performance category C2 Design according to „EOTA Technical Report TR 045, Edition February 2013“	

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PROFIS ENGINEERING

The authors have made use of PROFIS Engineering software to develop the design examples presented in this book. PROFIS Engineering is a state-of-the-art software that handles calculations and analysis of the different elements of a steel-to-concrete connection including base material, steel plate, anchors, weld and stiffeners with easy iteration and thorough documentation - all from one cloud-based tool. It makes tackling even the most complex and challenging anchoring projects easier.

Advantages of PROFIS Engineering software are:

1. **Reduce design time -** combining anchor and baseplate calculations to reduce overall design time.

2. **Improve accuracy -** Ensure your specifications are right the first time. Automatic load transfers, simultaneous multiple load combination processing, and BIM / CAD model generation all help to reduce errors and rework.

3. **Increase efficiency -** Integrated data sharing in a single platform grants every stakeholder immediate access to the most up-to-date design files and reports. This helps to improve transparency, remove communication logjams and increase efficiency.

ASK HILTI

Questions related to fastening technology can be posted on Ask Hilti community. Ask Hilti is an online community platform for Engineering and Construction professionals. It offers opportunities for gaining knowledge on topics such as anchoring and passive fire protection systems through Q&A, Webinars and Articles with the end goal of improving safety across the construction industry.

The platform contains 3 sections for Ask Hilti members to use:

1. **Expert Advice** – ask questions for fast, expert advice from Hilti Engineers on some of the most challenging and innovative applications/concepts like seismic, retrofitting, concrete to concrete post-installed connection(s), fire safety etc.

2. **Education** – engage with Hilti on-demand and/or live webinars on innovative topics and be awarded with participation certificates.

3. **Articles** – regularly added to the platform, read up on recent projects, expand users' knowledge on anchor designs and be educated on the latest standards and ask questions on articles.

Finally, a handy search function allows users to easily delve into the archive of webinars, articles, and questions.

FASTENING SYSTEMS IN CONCRETE CONSTRUCTION

This book is a useful design guidebook for the practicing engineers who have to design different types of fasteners in their projects as well as for academician and students who are interested in this topic. In absence of national standards, it becomes difficult for the engineers to design the fastening systems which are also an integral aspect of the structure. Fortunately, there are several international standards as well as specialist literature on the subject. This book talks about innovative fastening technologies like cast-in anchor channels, post-installed anchors, post-installed rebar and direct fastening systems. This book introduces basic concepts related to fastening technology, explains the design method recommended in Eurocodes as well as gives an overview of assessment criteria as per various guidelines developed by "European Organisation for Technical Assessment (EOTA)". Several design examples are also covered in detail. Lastly, this book also discusses some case studies to showcase practical applications of these innovative technologies.

About the Authors

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