Anchor bolt design manual

1. Design of Steel-to-Concrete Joints Design Manual II Ulrike Kuhlmann František Wald Jan Hofmann et al UlrikeKuhlmann,JanHofmann,FrantišekWald,,etal Design of Steel-to-Concrete Joints This Design manual I summarises the reached knowledge in the RFCS Project RFSR- CT-2007-00051 New Market Chances for Steel Structures by Innovative Fastening Solutions between Steel and Concrete, INFASO. The material was prepared in cooperation researchers from Institute of Structural Design and Institute of Construction Materials, Universität Stuttgart, Department of Steel and Timber Structures, Czech Technical University in Prague, and practitioners from Gabinete de Informática e Projecto Assistido Computador Lda., Coimbra, Goldbeck West GmbH, Bielefeld, stahl+verbundbau GmbH, Dreieich and European Convention for Constructional Steelwork, Bruxelles, one targeting on fastening technique modelling and others focusing to steel joint design. The models in the text are based on component method and enable the design of steel to concrete joints in vertical position, e.g. beam to column or to wall connections, and horizontal ones, base plates. The behaviour of components in terms of resistance, stiffness, and deformation capacity is summed up for components in concrete and steel parts: header studs, stirrups, concrete in compression, concrete panel in shear, steel reinforcement, steel plate in bending, threaded studs, embedded plate in tension, beam web and flange in compression and steel contact plate. In the Chapters 5 and 6 are described the possibility of assembly of components behaviour into the whole joint behaviour for resistance and stiffness separately. The presented assembly enables the interaction of normal forces, bending moments and shear forces acting in the joint. The global analyses in Chapter 7 is taken into account the joint behaviour. The connection design is sensitive to tolerances, which are recapitulated for beam to column connections and base plates in Chapter 8. The worked examples in Chapter 9 demonstrates the application of theory to design of pinned and moment resistant base plates, pinned and moment resistance beam to column connections and the use of predicted values into the global analyses. Deliverable of a project carried out with a financial grant from the Research Fund for Coal and Steel of the European Community DesignofSteel-to-ConcreteJoints ISBN 978-80-01-05439-0 February 2014 František Wald, Jan Hofmann, Ulrike Kuhlmann et al Printing in Publishing house of CTU in Prague - production 2. Design of Steel-to-Concrete Joints Design Manual I Prague, Stuttgart, Coimbra, and Brussels, February 2014 Deliverable of a project carried out with a financial grant from the Research Fund for Coal and Steel of the European Community 3. II Design of steel-to-concrete joints, Design manual I Although all care has been taken to ensure the integrity and quality of this publication and the information herein, no liability is assumed by the project partners and the publisher for any damage to property or persons as a result of the use of this publication and the information contained herein. Reproduction for non-commercial purpose is authorised provided the source is acknowledged and notice is given to the project coordinator. Publicly available distribution of this publication through other sources than the web sites given below requires the prior permission of the project partners. Requests should be addressed to the project coordinator: Universität Stuttgart, Institut für Konstruktion und Entwurf / Institute for Structural Design, Pfaffenwaldring 7, 70569 Stuttgart, Germany. The present document and others related to the research project INFASO RFSR-CT-2007-00051 New Market Chances for Steel Structures by Innovative Fastening Solutions between Steel and Concrete and the successive dissemination project RFS2-CT- 2012-00022 Valorisation of Knowledge for Innovative Fastening Solution between Steel and Concrete, which have been co-funded by the Research Fund for Coal and Steel (RFCS) of the European Community. ISBN 978-92-9147-119-5 František Wald, Jan Hofmann, Ulrike Kuhlmann, Šárka Bečková, Filippo Gentili, Helena Gervásio, José Henriques, Markus Krimpmann, Ana Ožbolt, Jakob Ruopp, Ivo Schwarz, Akanshu Sharma, Luis Simoes da Silva, and Jörg van Kann. Printing by European Convention for Constructional Steelwork February 2014 178 pages, 138 figures, 32 tables 4. III Contents

7. VI Symbols Lower case a factor considering the shoulder width, length b length c minimum edge distance, effective width, length ccr,N critical edge distance, ccr,N 1.5 hef cw drag coefficient d diameter db diameter of the bolt dh diameter of the head of headed stud ds diameter of the shaft of headed stud ds,re diameter of the stirrup ds,nom nominal diameter of the anchor shaft dw diameter of the washer ex,y length between the bolt axis and the edge of the plate e eccentricity fbd design bond strength according to EN1992-1-1:2004 fcd design strength of concrete fck characteristic strength of concrete fck,cube characteristic square strength of concrete fu strength of structural steel fub strength of the bolt fuk characteristic strength of steel fy nominal yield strength of steel fya average yield strength fyb nominal value of yield strength of the bolt fyd design yield strength of steel fyd,re design yield strength of the stirrups fyk characteristic yield strength of steel h height hef effective embedment depth according to product specifications k coefficient depending on the type of forming k1 factor for concrete cone strength in case of headed studs k2 factor for the headed studs for component P kA factor considering the cross-section ka form factor at porous edge sections kb stiffness of the bolt kb,re bond stiffness due to supplementary reinforcement, stirrups kC1 stiffness due to the displacement of the anchorage in case of concrete cone failure with supplementary reinforcement, combination C1 kC2 stiffness due to the displacement of the head, due to the pressure under the head on the concrete, and steel elongation, combination C2 kc,de stiffness of the descending branch for component CC kc,soft stiffness of the concrete cone in the softening branch kj concentration factor kp stiffness coefficient of the plate kp,de stiffness of the descending branch for component P ks stiffness of the anchor shaft for component S ks,re steel stiffness due to supplementary reinforcement, stirrups kv empirical value depending on the type of anchor l1 anchorage length lep elongated length leff effective length of T-stub, defined in accordance with EN1993 1-8:2006 lv,eff effective length of shear area m distance between threaded and headed studs mpl plastic moment resistance per unit, defined as m . · · n location of the prying force, number nre total number of legs of stirrups p internal pressure r radius of the fillet of a rolled profile s actual spacing of anchors 8. VII scr,N critical spacing for anchors t thickness tf thickness of the T-stub, flange tw thickness of T-stub, column tp1 thickness of the anchor plate tp2 thickness of the base plate wfic fictive effective width x distance between the anchor and the crack on the concrete surface assuming a crack propagation from the stirrup of the supplementary reinforcement to the concrete surface with an angle of 35° z distance of tension/compressed part Upper case A cross section area Ac0 loaded area Ac1 maximum spread area Ac,N actual projected area of concrete cone of the anchorage at the concrete surface, limited by overlapping concrete cones of adjacent anchors (s scr,N), as well as by edges of the concrete member (c ccr,N) A , reference area of the concrete cone of an individual anchor with large spacing and edge distance projected on the concrete surface Aeff effective area Ah area on the head of the headed stud Anet net cross section area As tensile stress area in the bolt As,nom nominal cross section area of all shafts As,re nominal cross section area of all legs of the stirrups Bt.Rd design tension resistance of a single bolt-plate assembly B , 0.9 · f · A /γ D diameter of column E modulus of elasticity the steel E 210 000 MPa F force or load Fc.Rd resistance of compressed part Fd design load Fk characteristic load Fmemb axial force Ft.Ed external tensile force Ft.Rd external ultimate resistance FT.Rd resistance of tension part I moment of inertia It torsion constant K general stiffness L length Lb length of anchor bolt Lcr buckling length LD the elongation length of the bolt, which may be taken as the total grip length (thickness of material plus washers) plus half the sum of the height of the bolt head and the height of the nut Lh length of the anchor shaft Ip,bp equivalent moment of inertia Mc,Rd bending moment capacity Mj,Rd design moment resistance of a joint MN,Rd interaction resistance bending with compression Mpl.Rd plastic moment resistance defined as M , l · m Mt,Rd torsion capacity Nact actual load on the anchor Nb,Rd design buckling resistance Ncr critical buckling load NEd tension/compression load NETA tension load for which the displacements are derived in the product specifications Npl,Rd design capacity in tension/compression NRd design capacity NRd,b,re design tension resistance for bond failure of stirrups NRd.C3 design failure load or the combined model NRd,c design tension resistance for concrete cone failure of headed stud 9. VIII NRd,cs design failure load for the concrete strut NRd,p design tension resistance for pull out failure of headed stud NRd,re design failure load for the supplementary reinforcement NRd,s design tension resistance for steel failure of headed stud NRd,s,re design tension resistance for steel failure of stirrups N , characteristic resistance of a single anchor without edge and spacing effects Nu ultimate resistance Ny yielding resistance Q prying force Rd design capacity Rk characteristic resistance Si elastic stiffness Sj,ini initial stiffness VETA shear load for which the displacements are derived in the product specifications Vpl,Rd shear capacity VRd design failure load for the anchor under shear VRd,c design shear resistance for concrete cone failure VRd,cp design shear resistance for concrete pryout VRd,p design shear resistance for pullout VRd,s design shear resistance for steel failure We external work Weff section modulus of effective area Wel elastic section modulus Wi internal work Wpl plastic section modulus Greek symbols α factor according to EN1992:2006 for hook effect and large concrete cover αc factor of component concrete break out in tension αp factor for the component head pressing αs factor of component stirrups βj material coefficient γF partial safety factor for actions γM material safety factor γMb partial safety factor for bolts γMb = 1.25 γMc partial safety factor for concrete γMc = 1.5 γMs partial safety factor for steel γMs = 1.15 γMV partial safety factor for shear resistance of studs γMV = 1.25 γMw partial safety factor for welds γMw = 1.25 γM0 partial safety factor for resistance of Class 1, 2 or 3 cross-sections γM0 = 1.0 γM1 partial safety factor for resistance of a member to buckling γM1 = 1.0 γM2 partial safety factor resistance of net section at bolt holes γM2 = 1.25 δ deformation, displacement δact displacement corresponding to Nact δc displacement corresponding to Nact for concrete cone δf corresponding displacement at failure load NRd,sre or NRd,bre δN,ETA displacement given in the product specifications for a corresponding tension load δRd,b,re deformation corresponding to design resistance for bond failure of stirrups δRd,c deformation corresponding to design resistance for concrete cone failure δRd,p deformation corresponding to design resistance for pull out failure δRd,s deformation corresponding to NRd δRd,s,re deformation corresponding to design resistance for steel failure of stirrups 10. IX δRd,sy deformation corresponding to design yield resistance for steel failure δu elongation δV,ETA displacement given in the product specifications for a corresponding shear load εbu,re strain limit for the stirrups due to bond εsu ultimate design strain limit for steel εsu,re strain limit for the stirrups under tension εsu,re strain limit for the stirrups under tension εu ultimate strain θ angle λ slenderness of member μ coefficient of friction ν Poisson`s ratio, ν 0.30 σ stress c reduction factor ψA,N factor accounting for geometric effects in anchor group, ψA,N = A , /A , ψre,N factor accounting for negative effect of closely spaced reinforcement in the concrete member on strength of anchors with hef 100 mm ψs,N factor accounting for the influence of edges of the concrete member on the distribution of stresses in the concrete ψs,N = 0.7 0. ,3 · c/c , 1.0 ψsupp support factor considering the confinement of the stirrups ψsupp = 2.5 x/h 1.0 Φ rotation Subscripts A area act actual b bolt, bond bd design bond c column, concrete cb concrete block ck characteristic concrete cp concrete pry out cs concrete strut cr critical d design e external eff effective ETA European technical approval g grout h head i internal k characteristic lim limit Mc material concrete Ms material steel N tension nom nominal po pullout p plate pl plastic Rd resistance design Rk characteristic resistance re failure rec reinforcement Sd internal design s shaft of anchor, stud soft softening supp support T tension part t tension tot total p plate p1 anchor plate p2 base plate u ultimate uk characteristic ultimate V shear w column web x, y directions y yield yd design yield yk characteristic yield 11. 10 1 INTRODUCTION The mixed building technology allows to utilise the best performance of all structural materials available such as steel, concrete, timber and glass. Therefore the building are nowadays seldom designed from only one structural material. Engineers of steel structures in practice are often faced with the question of economical design of steel to concrete joints, because some structural elements, such as foundations, stair cases and fire protection walls, are optimal of concrete. A gap in knowledge between the design of fastenings in concrete and steel design was abridged by standardized joint solutions developed in the INFASO project, which profit from the advantage of steel as a very flexible and applicable material and allow an intelligent connection between steel and concrete building elements. The requirements for such joint solutions are easy fabrication, quick erection, applicability in existing structures, high loading capacity and sufficient deformation capacity. One joint solution is the use of anchor plates with welded headed studs or other fasteners such as post-installed anchors. Thereby a steel beam can be connected by butt straps, cams or a beam end plate connected by threaded bolts on the steel plate encased in concrete. Examples of typical joint solutions for simple steel-to-concrete joints, column bases and composite joints are shown in Fig. 1.1. a) b) c) Fig. 1.1 Examples for steel-to-concrete joints, a) simple joint, b) composite joint, c) column bases The Design Manual I gives an overview of the existing design rules and introduces components developed. To present the use of the developed design rules, worked examples are given within the Design Manual I. More detailed information about the background documents, the experiments and development of the new design rules might be found in the final report, (Kuhlman et al 2013) and in Design manual II. This manual is focused to more complex worked examples, the application of a software tool for design, sensitivity study of proposed analytical models and its boundary conditions as well as design tables of optimal solutions. Chapter 2 gives a general overview about the component method and presents the existing models for steel-to-concrete joints. It also includes a short summary of the joint models and components developed in the project. In Chapter 3 and Chapter 4 the concrete and the steel components for the modelling of steel-to-concrete joints are described in more detail. The components already described in the codes as well as components of the newly derived models are introduced. Values for stiffness and resistance are presented. In Chapter 5 the single components are assembled to evaluate the overall joint resistance. Chapter 6 shows how the joint stiffness can be derived due to the stiffness’s of the single components. For the global analysis of a structure the joint behaviour/stiffness may have an influence. The effects of the joint modelling on the global analysis are explained in Chapter 7. The tolerances for steel-to-concrete joints and their effect on the construction are discussed in Chapter 8. In the Chapter 9 worked examples for the whole range of the steel-to-concrete joints are prepared. This examples are demonstrating the possibilities of the new design rules and allow an easy 12. 11 access for the engineers in practice. The references are given to this Design manual I (DM I) and to Eurocodes (EN199x-1-x). Chapter 10 summarises the offered opportunity for innovations. Chapters 1 and 2 were prepared by U. Kuhlman and J. Ruopp, Chapter 3 by J. Hofmann and A. Sharma, Chapters 4, 5 and 6 by F. Wald, Bečková Š. and Schwarz I., Chapter 7 by da Silva L. Simoes, H. Gervásio and J. Henriques and F. Gentili and Chapter 8 by M. Krimpmann. The worked examples 9.1 to 9.3 were set by Š. Bečková and I. Schwarz, 9.4 by Š. Bečková, I. Schwarz and M. Krimpmann, 9.5 by J. Ruopp, 9.6 and 9.7 by J. Henriques and F. Gentili, with help of the headed studs design models by A. Sharma. 13. 12 2 COMPONENT METHOD FOR STEEL TO CONCRETE JOINTS 2.1 Design method In the past decades, the component method has been set as a unified approach for the efficient analysis of steel and composite joints, see (Da Silva 2008). The basic principle of the component method consists of determining the complex non-linear joint response through the subdivision into basic joint components. The joint can be regarded as a set of individual basic components that contribute to its structural behaviour by means of resistance, stiffness and deformation capacity. The component method allows designers to take options more efficiently because the contribution of each component to the joint behaviour may be optimized according to the limiting components. Thus, one of the main advantages of the component method is that the analysis of an individual component can be done independently of the type of joint. In a second calculation step the single components are assembled by the designers according to the joint configuration. Joint components may be divided by the type of loading. Accordingly, three groups of components are usually identified: components for tension, compression and shear. Additionally, a second division may be done according to their location: panel zone or connecting zone. In Fig. 2.1 these two definitions are illustrated based on a double sided composite joint. Fig. 2.1 Division of joint into groups and zones In practice these components are modelled by translational springs with non-linear force- deformation response that are exposed to internal forces. The joint may then be represented by a spring model as illustrated in Fig. 2.2. 14. 13 Fig. 2.2 Component model for composite joint with separated the panel zone in shear The component method is given by EN1993-1-8:2006 and EN1994-1-1:2010 for the analysis of steel and composite joints. The application of the method requires following steps: 1. Identification of the basic joint components 2. Characterization of the structural properties of the basic joint components 3. Assembly of the component properties In the referred codes, a list of basic joint components is provided for the most common joint configurations. Basic joint components are then characterized in terms of strength, stiffness and deformation capacity allowing to obtain the F-δ curve, see Fig. 2.3, reproducing its behaviour. Finally, through the assembly procedure the joint properties are determined. The joint behaviour may be later reproduced by an M-Φ curve, see Fig. 2.4, in the structural analysis. Fig. 2.3 Component force deformation, F-δ, curve, experiment in black and model in grey line Fig. 2.4 Joint moment rotation, M-Φ, curve experiment in black and model in grey line 2.2 Classification of joints 2.2.1 Global analyses The classification of the joints is prepared to examine the extent to which the stiffness or strength have to be considered in the calculation according the design accuracy. In total there are three different calculation methods which require different joint properties. These calculation methods and the joint properties are compared within Tab. 2.1Tab. 2.1 Relation between method of global analysis and considered joint behaviour 15. 14 Method of global analysis Considered joint behaviour Elastic Rigid plastic Elastic plastic Elastic method If the elastic calculation method is applied, only the joint stiffness Sj is considered. Sj is implemented in the structural calculation as spring element or one-dimensional beam element in order to determine the internal forces. If the bending moment does not exceed 2/3 of the moment resistance of the joint the initial stiffness Sj,ini can be used to describe the elastic behaviour. For calculations, where the plastic moment capacity is reached, the joint stiffness can be calculated with the secant stiffness Sj,ini/. The joints are classified for this method by taking into consideration the rotational stiffness. Rigid plastic method In the second calculation method the elastic behaviour of the joint is neglected. Internal forces of the structural calculation are calculated from 1st order plastic hinge theory only satisfying equilibrium conditions. Within this method only the plastic moment capacity is considered, but the joints must have sufficient deformation capacity to allow full plastic redistribution. In this case the joints are classified by the resistance. Elastic plastic method If the third method is applied the overall moment-rotation-relationship of the joint has to be considered. This relationship is used within the joint modelling of the structural calculation. For simplification a bilinear approach of the moment rotation curve may be used. Typically the reduced secant stiffness is applied. If the elastic plastic method is used, the joint has to be classified by stiffness and strength. The advantages of this method are shown in the following example. In Fig. 2.5 a steel frame with horizontal and vertical loading is shown. Instead of modelling the column bases as a pinned joint as it is common in practice, the column bases may be classified as semi-rigid and modelled with a rotational spring. Thereby the column bases may stabilise the structure and reduce the bending moment in the steel-to-steel beam to column joints. So a classification of the column bases as semi-rigid instead of pinned makes the steel structure more safe and economical. 16. 15 Fig. 2.5 Considering the rotation stiffness of joints with springs It is also important not to underestimate the stiffness of the column bases, because big rotational stiffness might cause unexpected high bending moments in the joints which may lead to failure. The classification of the joints, may be found in cl 5 of EN1993-1-8:2006 and is explained in the following section. 2.2.2 Stiffness The first part of this chapter deals with the classification of beam to column/wall and beam to beam joints, the second part with the classification of column bases. Depending on its initial rotational stiffness S , a joint may be classified as pinned, rigid or semi-rigid. Normally pinned joints can transfer axial and shear force. Rotation of the joint does not cause significant bending moments. If a joint cannot be classified as normally pinned or rigid it is classified as semi-rigid. Rigid joints have a rotational stiffness which legitimise to treat the joint as rigid in the global analysis. Fig. 2.6 Classification due to stiffness Joints classified according to the connecting beams Rigid joints, in Fig. 2.6 zone 1, is classified as rigid if S , K E I /L (2.1) If a bracing system reduces the horizontal displacement more than 80 %, then Kb 8. For other frames provided that in every storey the following equation (2.2) is valid, then Kb 25. K K 0.1 (2.2) 17. 16 Semi-rigid joints, in Fig. 2.6 zone 2, are all joints which are not classified as pinned or rigid. For frames where Eq. 2.3 applies the joints should be classified as semi rigid and not as rigid. K K 0.1 (2.3) Nominally pinned joints, in Fig. 2.6 zone 3, are expecting to have a limited bending stiffness compared to the bending stiffness of the connected beam. S , 0.5 E I /L (2.4) where K is mean value of I /L for all the beams at the top of that storey K is mean value of I /L for all columns of that storey I is the second moment of area of beam I is the second moment of area of column L is the span of beam L is the storey height of a column Column bases classified according to the connecting column Column bases are classified as rigid if the following conditions are satisfied. There are two possible cases which have to be considered. If there is an additional bracing in a frame and the additional bracing reduces the horizontal movement at least by 80 %, then the column base affects the accuracy of the column design, which depends on the column relative slenderness. This column base might be assumed as rigid according to EN1993-1-8:2006 cl. 5.2a, if λ 0.5 (2.5) for 0.5 λ 3.93 is S , 7 2 λ 1 E I /L (2.6) and for λ 3.93 and S , 48 E I /L (2.7) where λ is the relative slenderness of a column in which both ends are assumed as pinned. For all other constructions, the cases where the storey´s sway is not prevented, the column base might be classified according to cl. 5.2d in EN1993-1-8:2006 as rigid if S , 30 EI /L (2.8) 2.2.3 Strength A joint is classified for strength as pinned, full-strength or partial-strength, see Tab. 2.1 and Fig. 2.7. The classification by strength may be found in EN1993-1-8:2006 cl 5.2.3. Nominally pinned joint should have a design moment resistance less than 25 % of the design moment resistance, which would be required for a full-strength joint. They must have sufficient rotational capacity. A Partial-strength joint is a joint, which cannot be classified as pinned or full-strength. 18. 17 The design moment resistance of a full-strength joint is bigger than the design moment resistance of the beam or column connected to it. Fig. 2.7 Classification due to resistance If the design resistance of the beam M , , is smaller than the design resistance of the column M , , , M , , is replaced for connections at the top of a column by M , , see Fig. 2.7. If the design resistance of the beam M , , is smaller than the double design resistance of the column M , , than in the figure above M , , is replaced for connections within the column height by 2 M , , . 2.2.4 Deformation capacity In EN1993-1-8:2006 an explicit classification for deformation or rotational capacity of the joint is not implemented. The complexity on classification according to deformation capacity is in the lack of knowledge of the upper values of material properties by designers, which do not allow a safe prediction of the failing component. In EN 1993-1-8 cl 6.4 design rules for the rotation capacity are given based on best engineering practice. If the system is calculated with a plastic global analysis a sufficient rotation capacity is needed. No investigation of the rotation capacity of the joint is necessary, if the moment resistance of the joint M , is at least 20 % bigger than the plastic moment resistance M , of the connected beam, see (2.9). Then the plastic hinge appears in the beam and the rotational capacity has to be satisfied by the beam section. M , 1.2 M , (2.9) If the moment resistance of the joint is not 1.2 times the plastic moment resistance of the connected beam and a plastic hinge is assumed in the joint, minimum rotational capacities for bolted and welded joints have to be checked. Bolted joints The rules for bolted joints may be found in EN1993-1-8:2006 cl 6.4.2. A bolted joint is assumed to have a sufficient rotation capacity if following conditions can be applied: If the failure load M , is determined by the resistance of the column web panel and for this panel d/t 69 ε where d is the nominal bolt diameter and tw is the thickness of the web 19. 18 If the thickness of the flange of the column or the beam end plate is sufficiently thin to satisfy the following formula. t 0.36 d f /f (2.10) where f is ultimate strength of the bolts f is yield strength of the flange or the end plate Welded joints The rules for welded joints may also be found in EN1993-1-8:2006 cl 6.4. For a welded beam to column connection the rotation capacity ∅ may be calculated with the following equation. In this case the web has to be stiffened in the compression area but not in the tension are and the moment resistance is not determined by the resistance of the column web panel. ∅ 0.025 h /h (2.11) where h is the depth of the column h is the depth of the beam For a welded beam to column connection where the compression and the tension area in the column are not stiffened, the rotation capacity may be assumed to be at least 0.015 rad. 2.3 Steel-to-concrete joints 2.3.1 Available models Design models for steel-to-concrete joints are currently available in the three standard documents: EN1993-1-8:2006 includes values for stiffness and resistance for all steel components and values for stiffness and resistance for concrete components in compression. There are no rules for concrete components in tension or shear. EN1994-1-1:2010 enhancement of the rules from EN 1993-1-8 on composite joints such as the connection of composite girder to steel columns. CEN/TS 1992-4-1:2009 summarises values for the design resistance of fasteners in concrete. But no values for stiffness and ductility are available. 2.3.2 Steel and composite structures Design rules in the Eurocode are given for different joint configurations. The model for the column bases is described in the EN1993-1-8:2006 and the model for the composite joint in EN1994-1-1:2010. Column bases with base plates The analytical prediction model for column base with base plate is described in the EN1993- 1-8:2006. With these design rules column bases loaded by axial force and bending moments are calculated. The model is only including concrete components for the compression forces. 20. 19 For the tension force only steel components are considered. The design resistance of column bases with steel base plates is described in EN1993-1-8:2006, cl 6.2.8. First according to the eccentricity of the axial force and the geometry of the column base one of the four loading types is chosen, and the lever arm is calculated. For this see Tab. 2.2. Then the loading of the column and the compression components are calculated. The failure load is determined by the weakest activated component. These components are for: Tension Base plate in bending under tension cl 6.2.6.11 in EN1993-1-8 Anchor bolt in tension cl 6.2.6.12 in EN1993-1-8 Column web in tension cl 6.2.6.8 in EN1993-1-8 Compression Base plate in bending under compression cl 6.2.6.10 in EN1993-1-8 Concrete in compression cl 6.2.6.9 in EN1993-1-8 Column web and flange in compression cl 6.2.6.7 in EN1993-1-8 Shear Anchor bolts in shear cl 6.2.2.6 to 6.2.2.9 in EN1993-1-8 According to procedure in EN1993-1-8:2006 cl 6.3.4 one of the four cases of the loading and geometry is chosen, see Tab.2.2. Then the rotational stiffness is calculated. One complexity creates change of the loading type depending on the loading cases. From this different rotational stiffness values for different combinations of bending moment and axial forces are resulting. The design of the embedded column base according to Eurocodes was developed by (Pertold et al, 2000) based on set of tests and finite element modelling. This model is prepared to approve resistance to combine base plate with embedding. Composite joints The composite joint is described in the Section 8 in EN1994-1-1:2010. The composite joint may be used for the connection of composite beams to steel columns. The design rules are an enhancement of the rules according to EN1993-1-8:2006 and new components are added. These additional components are: - Longitudinal steel reinforcement in tension cl. 8.4.2.1 EN1994-1-1:2010 - Steel contact plate in compression cl 8.4.2.2 EN1994-1-1:2010 - Column web in transverse compression cl 8.4.3 EN1994-1-1:2010 - Reinforced components cl 8.4.4 EN1994-1-1:2010 - Column web panel in shear cl 8.4.4.1 EN1994-1-1:2010 - Column web in compression cl 8.4.4.2 EN1994-1-1:2010 For all other components EN1993-1-8:2006 is applied. 21. 20 Tab. 2.2 The loading situations for the definition of the lever arm Number Description of loading Sketch Explanation 1 Left side in tension Right side in compression , , Bending moment is dominating 2 Left side in tension Right side in tension , , Tensile force is dominating 3 Left side in compression Right side in tension , , Bending moment is dominating 4 Left side in compression Right side in compression , , Compression force is dominating Fig. 2.8 Composite joint 22. 21 Tab. 2.3 Failure modes observed for anchors in concrete Loading Failure modes Tension Steel failure Concrete cone failure Pull-out / Pull-through Splitting failure Local Blow-out failure Steel failure Shear Steel failure Concrete edge failure Pry-out failure Pull-out failure 2.3.3 Concrete structures In CEN/TS1992-4-1:2009 the design of fastenings in concrete is given. In these rules the failure modes of the fasteners and the concrete are described in a detailed way. For tension and shear loading various failure modes exist. Failure modes are given according to CEN/TS 1992- 4-1:2009, see Tab. 2.3.. All possible failure modes are determined. The smallest resistance defines the design resistance of the joint. The design rules for the resistance include different types of geometries. Also edge effects, concrete with and without cracks and different kinds of fasteners are considered. However for stiffness no design rules are given and the use of additional stirrups is covered in a very conservative way. 2.3.4 Components for joints with anchor plate Headed studs in tension / Headed studs with stirrups in tension Load-displacement-curves of test specimens have shown, that in cases were additional reinforcement is used, also other components besides the reinforcement have a contribution on the overall load bearing capacity of the fixture. If, for instance, the reinforcement starts to yield, compression struts may develop and a small concrete cone failure can be the decisive component. With the design model the interaction of the concrete cone and the stirrups is considered. This allows the increase of the design resistance and the determination of the stiffness of the two combined components concrete cone and stirrups in tension in cases, where both of them are interacting. In Fig. 2.9 a headed stud with additional reinforcement and the assembly of single components is shown. 23. 22 Headed stud failure in tension Pull-out failure Concrete cone failure with stirrups in tension Fig. 2.9 Component headed studs with stirrups in tension Embedded plate in tension Ductile behaviour and a larger rotation capacity of column bases can be initiated with a thin anchor plate in combination with a base plate welded to the end of the column. In Fig. 2.10 three different kinds of geometries of embedded plates are shown, see Kuhlman et al, 2013. a) b) c) Fig. 2.10 Example of different positions of headed and treaded studs, a) above, b) in distance in one major direction, c) in distance in general The headed studs are welded on the bottom side of the base plate to connect the thin plate to the concrete. The column base plate is connected to the anchor plate by the threaded bolts. If the threaded bolts and the headed studs are in one line like, see Fig. 2.10, the anchor plate has no influence on the behaviour of the joint. If the threaded bolts and the headed studs are not in one line the anchor plate is activated. The model of the embedded plate represents an additional failure mode for the T-stub in tension. If the T-stub reaches its limit state, the thin base plate may still increase its capacity due to the membrane effect. The component embedded plate in tension shows a ductile behaviour as large deformations occur before failure. A detailed explanation of this component is given in Chapter 7. The Tab. 2.4 summarises the components, which are used to model the simple and rigid steel beam to concrete column/wall joints and column bases using anchor plates. Tab. 2.4 Components for joints with anchor plates Component Headed stud in tension Concrete breakout in tension Stirrups in tension Pull-out failure of the headed stud Headed stud in shear Figure Chapter 3.1.1 3.1.2 3.1.4 3.1.5 3.1.6 Component Friction Concrete in compression Concrete panel in shear Longitudinal steel Slip of the composite beam 24. 23 reinforcement in tension Figure Chapter 3.3.7 3.4 3.5 3.6 3.7 Component Threaded studs in tension/ shear Punching of the anchor plate Anchor plate in bending and tension Colum/beam flange and web in compression Steel contact plate Figure Chapter 4.7 4.3 4.4 4.5 4.6 25. 24 3 COMPONENTS IN CONCRETE 3.1 Component model for headed studs For components embedded in concrete the displacement behaviour and therefore the F-curve is influenced by the concrete properties itself and the interaction between the anchorage and the concrete. The influence of concrete on the behaviour of anchorages in tension have to be considered. The scatter in concrete is much larger than that observed for the material steel, see (Pallarés and Hajjar, 2009). For design, a material safety factor for concrete according to EN1992-1-1:2004 of Mc 1.5 is used. The characteristic values for the resistances are derived by assuming a normal distribution and a probability of 90 % for the 5 % fractal that corresponds to the characteristic value. The given displacements and stiffness’s are mean values and can scatter with coefficient of variation up to 50 %. The complete F-curve for the design of a headed stud in tension is described by a rheological model using and combining different components for the headed stud. The individual components for anchorages with supplementary reinforcement are: Component S Steel failure of the headed stud (Rd,s / NRd,s) Component CC Concrete cone failure (Rd,c / NRd,c) Component RS Steel failure of the stirrups (Rd,s,re / NRd,s,re) Component RB Bond failure of the stirrups (Rd,b,re / NRd,b,re) Component P Pull out failure of the headed stud (Rd,p / NRd,p) The combination is given in Fig. 3.1. Components S P CC KS/RB Components S P CC a) with supplementary reinforcement b) without supplementary reinforcement Fig. 3.1 Spring models for the different components of anchorages embedded in concrete 3.1.1 Headed studs in tension, component S If a headed stud is loaded in tension, the load is first transferred from the loading point at the base plate to the bearing areas of the headed stud. Therefore the shaft will elongate up to the design yielding strength f f /γ . For design the behaviour is assumed as linear elastic up to the yielding load of the headed stud. The corresponding elongation due to the introduced stress is calculated with the equation using the Hooke´s law. The elongation corresponding to the yield load is given by 26. 25 δ , N , L A , E σ , L E [mm] (3.1) where Lh is length of the anchor shaft [mm] NRd,s is design tension resistance of the headed stud [N] Es is elastic modulus of the steel, Es 210 000 N/mm² [N/mm²] As,nom is nominal cross section area of all shafts A , d , 4 mm² (3.2) where ds,nom is nominal diameter of the shaft [mm] The design load at steel yielding failure is calculated as given below N , A , f γ n π d , 4 f γ N (3.3) where fuk is characteristic ultimate strength of the shaft material of the headed stud [N/mm²] n is number of headed studs in tension [-] Ms is partial safety factor for steel [-] Exceeding the design steel yielding strength fyd, the elongation will strongly increase without a significant increase in load up to a design strain limit su. For the design, this increase of strength is neglected on the safe side and the stiffness is assumed to be zero, ks 0 N/mm. Depending on the product the failure shall be assumed at the yielding point. In general, fasteners as headed studs are deemed to have an elongation capacity of at least su 0.8 %. This limit shall be used to determine the response of the fasteners unless it is proven by means of tests that they have a higher elongation capacity. Therefore the stiffness ks is described as given below depending on the displacement or load k A , E L for N N , N/mm (3.4) k 0 for δ δ , e and N N , N/mm (3.5) where δRd,sy is displacement at yielding of the shaft, see Eq. (3.1) [mm] εsu is maximum elongation capacity of the shaft, 0.8 % [-] 3.1.2 Headed studs in tension, component CC The component concrete breakout in tension is described using the design load NRd,c for concrete cone failure and the displacement in the softening branch after failure. Up to the design load the component can´t be assumed as absolutely rigid without any displacement. The displacement corresponding to design load is given by 27. 26 δ , N , k , [mm] (3.6) The design load at concrete cone failure is calculated as N , N , ψ , ψ , ψ , γ [N] (3.7) where N , is characteristic resistance of a single anchor without edge and spacing effects N , k h . f . [N] (3.8) where k1 is basic factor 8.9 for cracked concrete and 12.7 for non-cracked concrete [-] hef is embedment depth given according to the product specifications [mm] fck is characteristic concrete strength according to EN206-1:2000 [N/mm²] ψ , is factor accounting for the geometric effects of spacing and edge distance [-] ψ , , , [-] (3.9) where ψ , is factor accounting for the influence of edges of the concrete member on the distribution of stresses in the concrete ψ , 0.7 0.3 c c , 1 (3.10) where ψ , is factor accounting for the negative effect of closely spaced reinforcement in the concrete member on the strength of anchors with an embedment depth hef 100 mm 0.5 hef / 200 for s 150 mm (for any diameter) [-] or s 100 mm (for ds 10 mm) 1.0 for s 150 mm (for any diameter) [-] γMc is 1.5 for concrete [-] A , is reference area of the concrete cone of an individual anchor with large spacing and edge distance projected on the concrete surface [mm²]. The concrete cone is idealized as a pyramid with a height equal to hef and a base length equal to scr,N with s , 3.0 h [mm] (3.11) c , 0.5 s , 1.5 h [mm] (3.12) where Ac,N is actual projected area of concrete cone of the anchorage at the concrete surface, limited by overlapping concrete cones of adjacent anchors, s scr,N, as well as by edges of the concrete member, c ccr,N. It may be deduced from the idealized failure cones of single anchors [mm²] To avoid a local blow out failure the edge distance shall be larger than 0.5 hef. Due to sudden and brittle failure, the initial stiffness for concrete cone is considered as infinity, i.e. till the actual load, Nact is less than or equal to the design tension resistance for concrete cone, the 28. 27 displacement c is zero. Once the design load is exceeded, the displacement increases with decreasing load, descending branch. Thus, the load-displacement behaviour in case of concrete cone breakout is idealized as shown in Fig. 3.2. Fig. 3.2 Idealized load-displacement relationship for concrete cone breakout in tension The stiffness of the descending branch kc,de for the design is described with the following function k , α f h ψ , ψ , ψ , [N/mm] (3.13) where αc is factor of component concrete break out in tension, currently αc -537 hef is embedment depth of the anchorage [mm] fck is characteristic concrete compressive strength [N/mm²] Ac,N is projected surface of the concrete cone [mm2] A , projected surface of the concrete cone of a single anchorage [mm2] The displacement δc as a function of the acting load Nact is described using the design resistance and the stiffness of the descending branch. For ascending part N N , and δc 0 (3.14) For descending branch δ 0 mm and δ N N , k , (3.15) 3.1.3 Stirrups in tension, component RS The component stirrups in tension was developed based on empirical studies. Therefore the tests results were evaluated to determine the displacement of the stirrups depending on the load Nact acting on the stirrup. The displacement is determined like given in the following equation δ , , 2 N , , α f d , n [mm] (3.16) where Nact NRd,c kc,de δc 1 29. 28 αs is factor of the component stirrups, currently αs 12 100 [-] NRd,s,re is design tension resistance of the stirrups for tension failure [N] ds,re is nominal diameter of thereinforcement leg [mm] fck is characteristic concrete compressive strength [N/mm²] nre is total number of legs of stirrups [-] The design load for yielding of the stirrups is determined as given N , , A , f , n π d , 4 f , [N] (3.17) where As,re is nominal cross section area of all legs of the stirrups [mm²] ds,re is nominal diameter of the stirrups [mm] fyd is design yield strength of the shaft material of the headed stud [N/mm²] nre is total number of legs of stirrups [-] Exceeding the design steel yielding strength fyd,re the elongation will increase with no significant increase of the load up to a strain limit εsu,re of the stirrups. For the design this increase of strength is neglected on the safe side. In general reinforcement steel stirrups shall have an elongation capacity of at least εsu,re = 2,5 %. So the design strain limit εsu,re is assumed to be 2.5 %. The displacement as a function of the acting load is determined as k , n α f d , √2 δ for δ δ , , [N/mm] (3.18) k , 0 for δ δ , , ε , [N/mm] (3.19) 3.1.4 Stirrups in tension - bond failure, component RB The displacement of the concrete component stirrups in tension is determined under the assumption that bond failure of the stirrups will occur. This displacement is calculated with equation (3.19) as δ , , 2 N , , α f d , n [mm] (3.20) where αs is factor of the component stirrups, currently αs 12 100 [-] NRd,b,re is design tension resistance of the stirrups for bond failure [N] ds,re is nominal diameter of the stirrups [mm] fck is characteristic concrete compressive strength [N/mm²] The design anchorage capacity of the stirrups according CEN/TS-model [5] is determined the design tension resistance of the stirrups for bond failure N , , n , l π d , f α [N] (3.21) where ns,re is number of legs [-] 30. 29 l1 is anchorage length [mm] ds,re is nominal diameter of the stirrups [mm] fbd is design bond strength according to EN1992-1-1:2004 [N/mm²] α is factor according to EN1992-1-1:2004 for hook effect and large concrete cover, currently 0.7 · 0.7 = 0.49 [-] k , n α f d , √2 δ for δ δ , , [N/mm] (3.22) k , 0 for δ δ , , ε , [N/mm] (3.23) 3.1.5 Headed studs in tension, component P The pull out failure of the headed studs will take place if the local stresses at the head are larger than the local design resistance. Up to this level the displacement of the headed stud will increase due to the increasing pressure under the head. δ , , k · N , A · f · n [mm] (3.24) δ , , 2 k · min N , ; N , A · f · n δ , , [mm] (3.25) k α · k · k k (3.26) where Ah is area on the head of the headed stud [mm²] A π 4 · d d (3.27) where ka is form factor at porous edge sections [-] k 5/a 1 (3.28) where ap is factor considering the shoulder width [mm] a 0.5 · d d (3.29) where kA is factor considering the cross section depending on factor ka [-] k 0.5 · d m · d d 0.5 · d (3.30) where n is number of the headed studs [-] 31. 30 αp is factor of the component head pressing, currently is αp 0.25 [-] k2 is factor for the headed studs in non-cracked concrete, currently 600 [-] is factor for the headed studs in cracked concrete, currently 300 [-] m is pressing relation, m 9 for headed studs [-] dh is diameter of the head [mm] ds is diameter of the shaft [mm] NRd,p is design load at failure in cases of pull out N , n p A /γ (3.31) where puk is characteristic ultimate bearing pressure at the headed of stud [N/mm2] NRd,c is design load for concrete cone failure without supplementary reinforcement N , N , ψ , ψ , ψ , γ [N] (3.32) where NRd,re design load at failure of the supplementary reinforcement minimum value of N , , A , f , n π , f , and N , , ∑ · · , · , [N] (3.33) The stiffness as a function of the displacement is determined as k , A f n δ k [N/mm] (3.34) k , A f n δ δ , 2 δ k [N/mm] (3.35) k , min N , ; N , /δ k , 1 δ , , /δ [N/mm] (3.36) The stiffness kp,de depends on the failure modes. If the supplementary reinforcement fails by yielding (NRd,s,re NRd,b,re and NRd,s,re NRd,p) the design stiffness kp,de is assumed as 104 N/mm², negative due to descending branch. In all other cases (e.g. NRd,s,re NRd,b,re or NRd,s,re NRd,p) kp,de shall be assumed as infinite due to brittle failure. The stiffness in case of pull out failure is calculated using the minimum value of the stiffness’s calculated with equation (3.34) to (3.36). k , min k , ; k , ; k , [N/mm] (3.37) 3.1.6 Headed studs in shear, component V The load-displacement behaviour mainly depends on the pressure to the concrete near the surface of the concrete member. Due to concrete crushing at the surface of the concrete member, the displacement under shear loading varies very large with a coefficient of variation about 40 % to 50 %. However a semi-empirical calculation shows that the displacement at failure mainly depends on the acting loading, the diameter of the anchors and the embedment 32. 31 depth. Therefore the displacement under shear loading for a given load level is calculated, see (Hofmann 2005), using the following equation only as an estimation δ , k V d h . [mm] (3.38) where kv empirical value depending on the type of anchor [-], for headed studs kv 2 to 4 VRd design failure load as the minimum of the design failure loads calculated for the different failure modes (VRd,s, VRd,cp, VRd,c , VRd,p) given according to the technical product specification CEN/TS 1992-4-1 or (FIB Bulletin 58, 2011) The displacement at ultimate load up three times larger than the displacement at the design load level due to the assumption, that the concrete near the surface is not fully crushed at design load level. 3.2 Combination of components To come up with the total stiffness of the connection with headed studs anchored in concrete with or without supplementary reinforcement, the stiffness’s must be combined. The combination depends on whether the components are acting in parallel, equal displacements, or in serial, equal load. Three combinations are given, see (Hofmann, 2005): Combination C1 Concrete cone failure with or without supplementary reinforcement, ks,re = 0 and kb,re = 0 Combination C2 Displacement due to steel elongation and head pressure, pull out Combination C3 Total connection of headed studs anchored in concrete with supplementary reinforcement Fig. 3.3 Combinations of different single components for an anchorage with supplementary reinforcement 3.2.1 Combination of concrete cone and stirrups, C1 = CC + RS/RB If both components are summarized, the load is calculated using the sum of the loads at the same displacement due to the combination of the components using a parallel connection 33. 32 from the rheological view. Two ranges must be considered. The first range is up to the load level at concrete failure NRd,c the second up to a load level of failure of the stirrups NRd,s,re or NRd,b,re. k . k k , ∞ for N N , [N/mm] (3.39) This leads to the following equation k . n α f d , √2 δ for N N , [N/mm] (3.40) In the second range the load is transferred to the stirrups and the stiffness decreases. The stiffness is calculated if Nact is larger than NRd,c with the following equation k . k k , for N N , [N/mm] (3.41) This leads to a relative complex equation k . N , δ k , δ , δ n α f d , √2 δ (3.42) for N N , , N , , [N/mm] If the load exceeds the ultimate load given by NRd,s,re or NRd,b,re the stiffness of the stirrups are negligible. Therefore the following equation applies: k . k k , 0 for N N , , N , , [N/mm] (3.43) 3.2.2 Combination of steel and pullout, C2 = S + P If both components are summarized the load is calculated using the sum of the displacements at the same load Nact due to the combination of the components using a serial connection from the rheological view. This is done by summing up the stiffness’s as given below k 1 k 1 k [N/mm] (3.44) This loads to the following equation k L A , E 1 k L A , E 1 min k ; k ; k [N/mm] (3.45) where kp is the minimum stiffness in case of pullout failure as the minimum of kp1, kp2 and kp3 34. 33 3.2.3 Combination of all components, C3 = CC + RS/RB + P +S To model the whole load- displacement curve of a headed stud embedded in concrete with a supplementary reinforcement the following components are combined: concrete and stirrups in tension, components CC and RB/RS, as combination C1, shaft of headed stud in tension, component S, and pull-out failure of the headed stud component P as Combination 2. The combinations C1 and C2 is added by building the sum of displacements. This is due to the serial function of both components. That means that these components are loaded with the same load but the response concerning the displacement is different. The combination of the components using a serial connection leads to the following stiffness of the whole anchorage in tension: 1/k 1/k 1/k [N/mm] (3.46) where kC1 is the stiffness due to the displacement of the anchorage in case of concrete cone failure with supplementary reinforcement, see combination C1 [N/mm], if no supplementary reinforcement is provided kC1 is equal to kc kC2 is the stiffness due to the displacement of the head, due to the pressure under the head on the concrete, and steel elongation, see combination C2 [N/mm] 3.2.4 Design failure load In principle two failure modes are possible to determine the design failure load NRd,C3 for the combined model. These modes are failure of the concrete strut NRd,cs, the supplementary reinforcement NRd,re. The design failure load in cases of concrete strut failure is calculated using the design load in case of concrete cone failure and an increasing factor to consider the support of the supplementary reinforcement, angle of the concrete strut, N , ψ N , N (3.47) where NRd,c is design failure load in case of concrete cone failure, see Eq. 3.7 [N] Ψsupport is support factor considering the confinement of the stirrups 2.5 x h 1 (3.48) where x is distance between the anchor and the crack on the concrete surface assuming a crack propagation from the stirrup of the supplementary reinforcement to the concrete surface with an angle of 35° [mm] 35. 34 Fig. 3.4 Distance between the anchor and the crack on the concrete surface The load is transferred to the stirrups and the concrete cone failure load is reached. Depending on the amount of supplementary reinforcement the failure of the stirrups can decisive NRd,re NRd,cs. Two failure modes are possible: steel yielding of stirrups NRd,s,re, see equation (3.16), anchorage failure of stirrups NRd,b,re, see equation (3.20). The corresponding failure load is calculated according to equation (3.49) summarizing the loads of the corresponding components N , min N , , ; N , , N , δ · k , [N] (3.49) where NRd,c is design failure load in case of concrete cone failure, see equation (3.7), [N] NRd,s,re is design failure load in case of yielding of the stirrups of the supplementary reinforcement, see equation (3.16) [N] NRd,b,re is design failure load in case of bond failure of the stirrups of the supplementary reinforcement, see equation (3.20) [N] kc,de is stiffness of the concrete cone in the descending branch, see equation (3.13) [N/mm] δf is corresponding displacement at failure load NRd,s,re or NRd,b,re [mm] 3.2.5 Combination of tension and shear components The displacements in tension and shear is calculated by the sum of the displacement vectors. 3.3 Simplified stiffness’s based on technical specifications 3.3.1 Headed stud in tension without supplementary reinforcement For simplification the displacements and the stiffness of headed studs or anchorages is estimated using technical product specifications. The elongation δRd is estimated up to the design load NRd using the displacements given in the technical product specification. The displacement is estimated by the following equation δ , δ , N N (3.50) where δN,ETA is displacement given in the product specifications for a corresponding load NETA is tension load for which the displacements are derived in the product specifications 36. 35 NRd is design tension resistance The stiffness of the anchorage is calculated with the following equation k , δ , N (3.51) where δN,ETA is displacement given in the product specifications for a corresponding load NETA is tension load for which the displacements are derived in the product specifications 3.3.2 Headed stud in shear For the design the displacement δv is estimated up to the design load VRd using the displacements given in the technical product specification. The displacement is estimated using the displacements far from the edge δv,ETA for short term and long term loading. The displacement is estimated by the following equation δ , δ , V V (3.52) where δV,ETA is displacement given in the product specifications for a corresponding load VETA is shear load for which the displacements are derived in the product specifications VRd,c is design shear resistance The stiffness of the anchorage is calculated with the following equation k , δ , V (3.53) where δV,ETA is displacement given in the product specifications for a corresponding load VETA is shear load for which the displacements are derived in the product specifications 3.3.3 Concrete breakout in tension The characteristic load corresponding to the concrete cone breakout in tension for a single headed stud without edge influence is given by equation N , k h . f (3.54) where k1 is basic factor for concrete cone breakout, which is equal to 8.9 for cracked concrete and 12.7 for non-cracked concrete, for headed studs, [-] hef is effective embedment depth given according to the product specifications [mm] [-] fck is characteristic concrete strength according to EN206-1:2000 [N/mm²] The design load for concrete cone breakout for a single anchor, N , is obtained by applying partial safety factor of concrete γ to the characteristic load as 37. 36 N , N , γ (3.55) For concrete, the recommended value of is γ = 1.5. For a group of anchors, the design resistance corresponding to concrete cone breakout is given by equation (3.56), which is essentially same as equation (3.7) N , N , ψ , ψ , ψ , /γ (3.56) where N , is characteristic resistance of a single anchor without edge and spacing effects ψ , is factor accounting for the geometric effects of spacing and edge distance given as ψ , , , A , is reference area of the concrete cone for a single anchor with large spacing and edge distance projected on the concrete surface [mm²]. The concrete cone is idealized as a pyramid with a height equal to hef and a base length equal to scr,N with s , 3.0 h , thus A , 9 h . A , is reference area of the concrete cone of an individual anchor with large spacing and edge distance projected on the concrete surface [mm²]. The concrete cone is idealized as a pyramid with a height equal to hef and a base length equal to scr,N with s , 3,0 h mm Ac,N is actual projected area of concrete cone of the anchorage at the concrete surface, limited by overlapping concrete cones of adjacent anchors s scr,N, as well as by edges of the concrete member c ccr,N. It may be deduced from the idealized failure cones of single anchors [mm²] c is minimum edge distance c 1.5 hef [mm] ccr,N is critical edge distance ccr,N 1.5 hef [mm] ψre,N is factor accounting for the negative effect of closely spaced reinforcement in the concrete member on the strength of anchors with an embedment depth hef < 100 mm 0.5 + hef / 200 for s 150 mm, for any diameter [-] or s < 100 mm, for ds ≤ 10 mm 1.0 for s 150 mm (for any diameter) [-] γMc is 1.5 for concrete [-] 3.3.4 Pull out failure of the headed studs The design load corresponding to the pull out failure of the headed stud, NRd,p is given by N , p A /γ (3.57) where puk is characteristic ultimate bearing pressure at the head of stud [N/mm2] Ah is area on the head of the headed stud [mm²] A π 4 · d d (3.57b) dh is diameter of the head [mm] ds is diameter of the shaft [mm] γMc is 1.5 for concrete [-] 38. 37 3.3.5 Interaction of components for concrete and stirrups In case of headed stud anchored in concrete with supplementary reinforcement, stirrups, the stirrups do not carry any load till the concrete breakout initiates, i.e. till Nact is less than or equal to NRd,c. Once, the concrete breakout occurs, the load shared by concrete decreases with increasing displacement as depicted in Fig. 3.4. The load shared by concrete Nact,c corresponding to a given displacement δ is therefore given by equation N , N , k , δ (3.57) where kc,de is the slope of descending branch of Fig. 3.4, negative value, given by Eq. (3.7). Simultaneously, in case of concrete with supplementary reinforcement, the stirrups start to carry the load. The load carried by the stirrups corresponding to a given displacement δ is given by equation N , n d , α f δ 2 (3.58a) where s is factor of the component stirrups, currently is αs = 12 100 [-] ds,nom is nominal diameter of the stirrups [mm] fck is characteristic concrete compressive strength [N/mm²] nre is total number of legs of stirrups [-] The total load Nact carried by concrete cone and stirrups corresponding to any given displacement δ is therefore given as the sum of the two components: N N , N , N , k , δ min n d , α f δ 2 ; N , , ; N , , (3.59) The displacement corresponding to peak load of the system is obtained by differentiating the right hand side of Eq. (3.60) and equating it to zero. If the bond failure or steel failure of stirrups is not reached at an earlier displacement then the design peak load carried by the system Nu,c s is given by N , N , , , (3.60) where NRd,c is design load at concrete cone failure given by equation (3.7) s is factor of the component stirrups, currently is s = 12 100 [-] ds,re is Nominal diameter of the stirrups [mm] fck is characteristic concrete compressive strength [N/mm²] nre is total number of legs of stirrups [-] kc,de is stiffness of descending branch for concrete cone failure, given by eq. (3.13) In a relatively rare case of all studs loaded in tension, both the legs of the hanger reinforcement are not uniformly loaded and the distribution of forces is difficult to ascertain. Due to this reason and also to avoid the problems with serviceability requirements, it is recommended that in such a case, the contribution of hanger reinforcement is ignored. 39. 38 3.3.6 Determination of the failure load The failure load Nu is given by the minimum of the failure load corresponding to each considered failure mode 3.3.7 Friction For base plates the friction is defined in EN1993-1-8 cl 6.2.2. For the resistance the resistance values of friction and bolts may be added as long as the bolt holes are not oversized. For the friction between a base plate and the grout underneath the plate the following calculation may be used. F , C , N , (3.61) where C , is coefficient for friction, for sand-cement mortar C , 0.2 N , is axial compressive force of the column In this design manual the friction is not only applied to compression forces caused by axial forces but also for compression forces generated by bending moments. This principle is applied in EN1993-1-8:2006 for beam to the column end joints with end plates in cl 3.9.2(3). 3.4 Base plate in bending and concrete block in compression 3.4.1 Concrete 3D strength The components concrete in compression and base plate in bending represent the behaviour of the compressed part of a steel to concrete connection. The resistance of these components depends primarily on the bearing resistance of the concrete block under the flexible base plate, see (Melchers, 1992). The resistance of concrete is influenced by flexibility of base plate. In case of loading by an axial force, the stresses in concrete are not uniformly distributed, they are concentrated around the footprint of the column under the plate according to its thickness, see (Dewolf, Sarisley, 1980). For the design the flexible base plate is replaced by

reducing the effective fully rigid plate. The grout layer between the base plate and concrete block influences the resistance and stiffness of the component. That is why this layer is also included into this component, see (Penserini, Colson, 1989). Other important factors which influence the resistance are the concrete strength, the compression area, the location of the plate on the concrete foundation, the size of the concrete block and its reinforcement. The stiffness behaviour of column base connection subjected to bending moment is influenced mostly by elongation of anchor bolts. The Component concrete in compression is mostly stiffer in comparison to the component anchor bolts in tension. The deformation of concrete block and base plate in compression is important in case of dominant axial compressive force. The strength of the component FRd,u, expecting the constant distribution of the bearing stresses under the effective area, is given by F , A f (3.62) The design value of the bearing strength fjd in the joint loaded by concentrated compression, is determined as follows. The concrete resistance is calculated according to cl. 6.7(2) in EN1992-1-1:2004 see Fig. 3.6 is 40. 39 F , A f A A 3.0 A f (3.63) where A b d and A b d (3.64) where Ac0 is the loaded area and Ac1 the maximum spread area. The influence of height of the concrete block to its 3D behaviour is introduced by h b2 b1 and h d2 d1 3 b b and 3 d d (3.65) Load axes Fig. 3.5 Concrete compressive strength for calculation of 3D concentration From this geometrical limitation the following formulation is derived f β F , b l β A f A A A β f k 3 A f A 3.0 f (3.66) The factor j represents the fact that the resistance under the plate might be lower due to the quality of the grout layer after filling. The value 2/3 is used in the case of the characteristic resistance of the grout layer is at least 0.2 times the characteristic resistance of concrete and thickness of this layer is smaller than 0.2 times the smallest measurement of the base plate. In different cases, it is necessary to check the grout separately. The bearing distribution under 45° is expected in these cases, see (Steenhuis et al, 2008) and Fig. 3.5 Concrete compressive strength for calculation of 3D concentration Fig. 3. The design area Ac0 is conservatively considered as the full area of the plate Ap. 41. 40 Fig. 3.6 Modelling of grout 3.4.2 Base plate flexibility In case of the elastic deformation of the base plate is expected homogenous stress distribution in concrete block is expected under the flexible base plate based on the best engineering practice. The formula for the effective width c is derived from the equality of elastic bending moment resistance of the base plate and the bending moment acting on the base plate, see (Astaneh et al., 1992). Acting forces are shown in Fig. 3.7. Fig. 3.7 Base plate as a cantilever for check of its elastic deformation only Elastic bending moment of the base plate per unit length is M 1 6 t f γ (3.69) and the bending moment per unit length on the base plate of span c and loaded by distributed load is M´ 1 2 f c (3.70) where fj is concrete bearing strength and from Eq. (3.69) and (3.70) is c t f 3 · f · γ (3.71) The flexible base plate, of the area Ap, is replaced by an equivalent rigid plate with area Aeq, see Fig. 3.8. Then the resistance of the component, expecting the constant distribution of the bearing stresses under the effective area is given by c fj Column Base plate F t c tw Sd FRd L t 42. 41 F , A · f (3.72) The resistance FRd should be higher than the loading FEd F F , (3.73) Fig. 3.8 Effective area under the base plate 3.4.3 Component stiffness The proposed design model for stiffness of the components base plate in bending and concrete in compression is given also in (Steenhuis et al, 2008). The stiffness of the component is influenced by factors: the flexibility of the plate, the Young´s modulus of concrete, and the size of the concrete block. By loading with force, a flexible rectangular plate could be pressed down into concrete block. This flexible deformation is determined by theory of elastic semi-space δ F α a E A (3.74) where F is acting load α is shape factor of the plate ar is width of equivalent rigid plate Ec is elastic modulus of concrete Ap is area of the plate The factor depends on the material characteristics. The Tab. 3.1 gives values of this factor dependent on the Poison's ratio, for concrete is 0.15. The table shows also the approximate value of factor α, that is 0.58 · L/a . Tab. 3.1 Factor α and its approximation for concrete l / ar α Approximation as α 0.58 · L/a . 1 0.90 0.85 1.5 1.10 1.04 2 1.25 1.20 3 1.47 1.47 5 1.76 1.90 10 2.17 2.69 For steel plate laid on concrete block it is Aeq Ap A c cc c c c Aeq Ap A Aeq Ap A 43. 42 δ 0.85 F E l · a (3.75) where σr is deformation under the rigid plate l is length of the plate The model for the elastic stiffness behaviour of component is based on a similar interaction between concrete block and steel plate. The flexible plate is expressed as an equivalent rigid plate based on the same deformation, modelled in Fig. 3.9. Fig. 3.9 A flange of flexible T-stub Independent springs support the flange of a unit width. Then, the deformation of the plate is a sine function. δ δ sin ½ π x / c (3.76) The uniform stress on the plate is rewritten by the fourth differentiate and multiplied by E I´p δ E I´ ½ π/c δ sin ½ π x c E t 12 ½ π c δ sin ½ π x /c (3.77) where E is elastic modulus of steel I´p is moment of inertia per unit length of the steel plate (I´p t3 / 12) t is thickness of the plate δ σ h /E (3.78) where hef is equivalent concrete height of the portion under the steel plate Assume that h ξ c (3.79) Factor ξ expresses the rotation between hef and cfl. Hence δ σ ξ c /E (3.80) After substitution and using other expressing it is E Ip x cfl 44. 43 c t π/2 12 ξ E E (3.81) The flexible length cfl may be replaced by an equivalent rigid length c c 2 / π (3.82) The factor shows the ratio between heq and cfl. The value ar represents height heq. Factor α is approximated to 1.4 · a t 2c and t 0.5 c . Then it is written h 1.4 · 0.5 2 c 1.4 · 2.5 · c · 2 π 2.2 c (3.83) Hence 2.2. For practical joints is estimated by Ec 30 000 N / mm2 and E 210 000 N / mm2, what leads to c t π/2 12 ξ E E t π/2 12 2.2 210000 30000 1.98 t (3.84) or c c 2 π 1.98 · 2 π · t 1.25 t (3.85) The equivalent width ar is in elastic state replace with a , t 2.5 t 0.5 c t (3.86) or a , 0.5 · 1.25 t 2.5 t 3.125 t (3.87) From the deformation of the component and other necessary values which are described above, the formula to calculate the stiffness coefficient is derived k F δ E E a , L 1.5 · 0.85 E E a , L 1.275 E E · √t · L 0.72 · E (3.88) where aeq,el is equivalent width of the T-stub L is length of the T-stub 3.5 Concrete panel The resistance and deformation of the reinforced concrete wall in the zone adjacent to the joint is hereby represented by a joint link component, see (Huber and Cermeneg, 1998). Due to the nature of this joint, reinforced concrete, the developed model is based on the strut-and-tie method, commonly implemented in the analysis of reinforced concrete joints. The problem is 3D, increasing its complexity, as the tension load is introduced with a larger width than the 45. 44 compression, which may be assumed concentrated within an equivalent dimension of the anchor plate, equivalent rigid plate as considered in T-stub in compression. Thus, a numerical model considering only the reinforced concrete wall and an elastic response of the material has been tested to identify the flow of principal stresses. These show that compression stresses flow from the hook of the longitudinal reinforcement bar to the anchor plate. In this way the strut-and-tie model (STM) represented in Fig. 10a is idealized. Subsequently, in order to contemplate the evaluation of the deformation of the joint, a diagonal spring is idealized to model the diagonal compression concrete strut, as illustrated in Fig. 10. The ties correspond to the longitudinal steel reinforcement bars. The properties of this diagonal spring are determined for resistance and stiffness. The resistance is obtained based on the strut and nodes dimension and admissible stresses within these elements. The node at the anchor plate is within a tri-axial state. Therefore, high stresses are attained as confinement effect. In what concerns the strut, because of the 3D nature, stresses tend to spread between nodes. Giving the dimensions of the wall of infinite width, the strut dimensions should not be critical to the joint. Thus, the node at the hook of the bar is assumed to define the capacity of the diagonal spring. The resistance of the spring is then obtained according to the dimensions of this node and to the admissible stresses in the node and in the strut. For the latter, the numerical model indicates the presence of transverse tension stresses which have to be taken into consideration. The deformation of the diagonal spring is obtained by assuming a non-linear stress-strain relation for the concrete under compression, as defined in (Henriques, 2012). The maximum stress is given by the limiting admissible stress as referred above. Then, deformation is calculated in function of the length of the diagonal strut and the concrete strain. a) Strut-and-tie model b) Single diagonal spring Fig. 3.10 Joint link modelling Tab. 3.2 provides the stresses for nodes and struts according to EN1992-1-1:2004. Node 1 is characterized by the hook longitudinal reinforcement bar. The represented dimension is assumed as defined in CEB-FIP Model Code 1990. In what concerns the width of the node, based on the numerical observations, it is considered to be limited by the distance between the external longitudinal reinforcement bars within the effective width of the slab. The numerical model demonstrates that the longitudinal reinforcement bars are sufficiently close, as no relevant discontinuity in the stress field is observed. Though, this is an issue under further T T C C Strut Node 1 Node 2 11′ 46. 45 investigation and depending on the spacing of the reinforcing bars, this assumption may or may not be correct (Henriques, 2013). Tab. 3.2 Stresses in strut-and-tie elements according to EN1992-1-1:2004 Element Limiting stresses Node 1 0.75 ν fcd Node 2 3 ν fcd Strut 0.6 ν fcd with ν 1 - fck/250 Fig. 3.11 Definition of the dimension related to the hook of the longitudinal reinforcement bar in Node 1, according to the CEB Model Code Finally, to simplify the assembling of the joint model, the diagonal spring representing the joint link component is converted into a horizontal spring. The properties of the horizontal spring are directly obtained from the diagonal spring determined as a function of the angle of the diagonal spring. 3.6 Longitudinal steel reinforcement in tension In the composite joint configuration under consideration, the longitudinal reinforcement in tension is the only component able to transfer tension forces introduced by the bending moment to the supporting member e.g. a reinforced concrete wall. This component determines the behaviour of the joint. According to EN1994-1 the longitudinal steel reinforcement may be stressed to its design yield strength. It is assumed that all the reinforcement within the effective width of the concrete flange is used to transfer forces. The resistance capacity of the component may then be determined as in Eq. (3.89). Regarding the deformation of the component, the code provides stiffness coefficients for two composite joint configurations, single and double-sided joints. The stiffness coefficient for single-sided joints may be estimated as in Eq. (3.90). This stiffness coefficient depends essentially on the elongation length of the longitudinal reinforcement contributing to the deformation of the component. Analogous to the code provisions, the dimension h involved in Eq. (3.90) is assumed as shown in Fig. 3.12. F , A , f (3.89) k , A , 3.6 h (3.90) 41 ° 2 02 .35 26 Fc Ft1 Ft2 Ft1 Ft2 Fc θ r 47. 46 Fig. 3.12 Dimension h for elongation length The tension component of the joint is calculated according to F M , /h (3.91) 3.7 Slip of the composite beam The slip of composite beam does not directly influence the resistance of the joint. However, the level of interaction between concrete slab and steel beam defines the maximum load the longitudinal reinforcement can achieve. Therefore in such joint configuration, where reinforcement is the only tension component, the level of interaction affects the joint resistance. In the EN1994-1-1:2008, the influence of the slip of composite beam is taken into account. The stiffness coefficient of the longitudinal reinforcement, see Eq. (3.92) should be multiplied with the reduction factor kslip determined as follows: k 1 1 E k k (3.92) K N k ϑ ϑ 1 1 ξ h d (3.93) ϑ 1 ξ N k l d E I (3.94) ξ E I d E A (3.95) where hs is the distance between the longitudinal reinforcing bars and the centre of compression of the joint, that may be assumed as the midpoint of the compression flange of the steel beam ds is the distance between the longitudinal reinforcing bars and the centroid of the steel beam section, see Fig. 13 48. 47 Ia is the second moment area of the steel beam section l is the length of the beam in hogging bending adjacent to the joint, in the case of the tested specimens is equal to the beam's length N is the number of shear connectors distributed over the length l ksc is the stiffness of one shear connector Fig. 3.13 Dimensions hs and ds 4 STEEL COMPONENTS 4.1 T-stub in tension The base plate in bending and anchor bolts in tension is modelled by the help of T-stub model based on the beam to column end plate connection model. Though in its behaviour there are some differences. Thickness of the base plate is bigger to transfer compression into the concrete block. The anchor bolts are longer due to thick pad, thick base plate, significant layer of grout and flexible embedding into concrete block. The influence of a pad and a bolt head may be higher. eff Column flange Base plate F t e m Fig. 4.1 The T stub - anchor bolts in tension and base plate in bending Due to longer free lengths of bolts, bigger deformations could arise. The anchor bolts, compare to bolts, are expecting to behave ductile. When it is loaded by tension, the base plate is often separated from the concrete surface. This case is shown in (Wilkinson et al, 2009). By bending moment loading different behaviour should be expected. The areas of bolt head and pad change favourably distribution of forces on T-stub. This influence is not so distinctive during calculation of component stiffness. The all differences from end plate connections are involved 49. 48 in the component method, see EN1993-1-8:2006. The design model of this component for resistance as well for stiffness is given in (Wald et al, 2008). Lbf L d be Lb Fig. 4.2 Length of anchor bolt 4.1.1 Model When the column base is loaded by bending moment as it is shown in Fig. 4.3, anchor bolts transfer tensile forces. This case of loading leads to elongation of anchor bolts and bending of the base plate. Deformed bolts can cause failure as well as reaching of the yield strength of the base plate. Sometimes failure in this tensile zone is caused by both, see (Di Sarno et al, 2007). Fig. 4.3 Tensile zone and equivalent T-stub in case of loading by bending moment Column with connected base plate taken, as it is shown in Fig. 4.4, into model of T-stub. nm F Q = 0 Q = 0 Fig. 4.4 T-stub separated from the concrete block with no prying force There are two models of deformation of the T-stub of the base plate according to presence of prying. In the case the base plate separated from the concrete foundation, there is no prying force Q, see Fig. 4.4. In other case, the edge of the plate is in contact with concrete block, the bolts are loaded by additional prying force Q. This force is balanced just by the contact force at the edge of the T-stub, see Fig. 4.5. When there is contact between the base plate and the concrete block, beam theory is used to describe deformed shape of the T-stub. 50. 49 Fig. 4.5 Beam model of T-stub and prying force Q Deformed shape of the curve is described by differential equation E I δ" M (4.1) After writing the above equation for both parts of the beam model 1 and 2, application of suitable boundary conditions, the equations could be solved. The prying force Q is derived just from these solved equations as Q F 2 · 3 m n A 2 L I 2 n A 3 m n 3 L I (4.2) When the base plate is in contact with concrete surface, the prying of bolts appears and on the contrary no prying forces occur in the case of separated base plate from the concrete block due to the deformation of long bolts. This boundary, between prying and no prying has to be determined. Providing that n 1.25 m it may be expressed as L , 8.82 m A l t L (4.3) where As is the area of the bolt Lb is equivalent length of anchor bolt leff is equivalent length of T-stub determined by the help of Yield line method, presented in following part of work For embedded bolts length Lb is determined according to Fig. 4.2 as L L L (4.4) where Lbe is 8 d effective bolt length When the length of bolt L L , there is no prying. Previous formulae is expressed for boundary thickness tlim, see (Wald et al, 2008), of the base plate as t 2.066 m · A l L (4.5) F 2 F 2 + Q Q + x 2 1 51. 50 If the base plate are loaded by compression force and by bending moment and not by tensile force it is recommended to neglect these prying forces. In other cases it needs to be checked. 4.1.2 Resistance The design resistance of a T-stub of flange in tension of effective length *l*eff is determined as minimum resistance of three possible plastic collapse mechanisms. For each collapse mechanism there is a failure mode. Following collapse modes, shown in Fig. 4.6, is used for T-stub in contact with the concrete foundation, see in EN1993-1-8:2006. F B Rd.3 t.Rd Bt.Rd F B Rd.1 B Q Q e n m Q Q Bt.RdBt.Rd FRd.2 Mode 3 Mode 1 Mode 2a) b) c) Fig. 4.6 Failure modes of the T-stub in contact with the concrete foundation Mode 1 According to this kind of failure the T-stub with thin base plate and high strength anchor bolts is broken. In the base plate plastic hinge mechanism with four hinges is developed. F , 4 l m , m (4.6) Mode 2 This mode is a transition between failure Mode 1 and 3. At the same time two plastic hinges are developed in the base plate and the limit strength of the anchor bolts is achieved. F , 2 l m , Σ B , · n m n (4.7) Mode 3 Failure mode 3 occurs by the T-stub with thick base plate and weak anchor bolts. The collapse is caused by bolt fracture. F , Σ B , (4.8) The design strength FRd of the T-stub is derived as the smallest of these three possible modes: F min F , , F , , F , (4.9) 52. 51 Because of the long anchor bolts and thick base plate different failure mode arises compare to an end plate connection. When the T-stub is uplifted from the concrete foundation, there is no prying, new collapse mode is obtained, see Fig. 4.7. This particular failure mode is named Mode 1-2. F B B Rd,1-2 Fig. 4.7 T-stub without contact with the concrete foundation, Mode 1-2 Mode 1-2 The failure results either from bearing of the anchor bolts in tension or from the yielding of the plate in bending, where a two hinges mechanism develops in the T-stub flange. This failure does not appear in beam to column connection because of the small deformation of the bolts in tension, see (Wald et al, 2008). F , 2 l m , m (4.10) The relationship between Mode 1-2 and modes of T-stub in contact with concrete is shown in Fig. 4.8. F B/ T,Rd 0,0 0,2 0,4 0,6 0,8 1,0 0 0,5 1 1,5 2 Mode1 Mode 2 Mode 3 Mode 1-2 4 eff mpl,Rd / BT,Rd Fig. 4.8 Failure mode 1-2 The boundary between the mode 1-2 and others is given in the same way like the boundary of prying and no prying – according to the limiting bolt length Lb,min. During the Mode 1-2 large deformations of the base plate can develop. Finally these deformations could lead to contact between the concrete block and the edge of the T-stub (prying forces can arise even in this case). After loading Modes 1 or 2 should be obtained like the first. But for reaching this level of resistance, which is necessary to obtain these modes, very large deformations are required. And so high deformations are not acceptable for design. In conclusion, in cases where no prying forces develop, the design resistance of the T-stub is taken as 53. 52 F min F , , F , (4.11) where F , ΣB , (4.12) The equivalent length of T-stub leff, which is very important for the resistance determination, is calculated by the help of the yield line method, which is explained in the following part of the work. Yield line method Although numerical methods, based on extensive use of computers, are potentially capable of solving the most difficult plate problems, yield-line analysis is such an alternative computational technique (Thambiratnam, Paramasivam, 1986). It provides such an alternative design method for plates. This simple method, which uses concepts and techniques familiar to structural engineers, provides realistic upper bounds of collapse loads even for arbitrary shapes and loading conditions. The advantages of the yield-line method are: simplicity and economy, information is provided on the real load-carrying capacity of the slab, the basic principles used are familiar to structural engineers, the method also gives acceptable estimates for the ultimate load-carrying capacity of structural steel plates, and resulting designs are often more economical. On the other hand, the present limitations of the method are: the method fails in vibration analysis and cannot be used in the case of repeated static or dynamic loads (but is applied effectively for suddenly applied one-time loads), and theoretically, the law of superposition is not valid. The yield-line method offers, especially for the practicing engineer, certain advantages over the elastic stress analysis approaches. Assumptions The correct failure pattern is known, the critical load is obtained either from virtual work or from equilibrium considerations. Both approaches use the following basic assumptions: at impending collapse, yield lines are developed at the location of the maximum moments, the yield lines are straight lines, along the yield lines, constant ultimate moments mu are developed, the elastic deformations within the slab segments are negligible in comparison with the rigid body motions, created by the large deformations along the yield lines, from the many possible collapse mechanisms, only one, pertinent to the lowest failure load, is important. In this case the yield-line pattern is optimum, when yield lines are in the optimum position, only ultimate bending moments, but no twisting moments or transverse shear forces are present along the yield lines. The location and orientation of yield lines determine the collapse mechanism. The Fig. 4.9 introduces an example of yield line. Free edge Fig. 4.9 Possible yield line patterns

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