YRKESHÖGSKOLAN NOVIA

Comparative Study of AISC Specifications & Idea StatiCa

Development of AISC Based Excel Tools

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EXAMENSARBETE

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Titel: Jämförande studie av AISC-specifikationer & Idea StatiCa
Utveckling av AISC-baserade Excel-beräkningsbottnar

Datum: 18.4.2018 Sidantal: 50 Bilagor: 3

Abstrakt

Dimensionering av olika stålförband enligt normer kan vara tidskrävande och kräver att designern gör en del antaganden eftersom normerna är framtagna för olika standardfall, vilket också gäller en del dimensioneringsprogram. Syftet med detta examensarbete var att undersöka ett dimensioneringsprogram, Idea StatiCa, som har ett nytt tillvägagångssätt som kombinerar Finita elementmetoden och komponentmetoden baserat på olika normer.

Arbetet består av tre huvuddelar: utveckling av AISC-baserade Excel-verktyg, teoretisk bakgrund om Idea StatiCa samt jämförelser med framtagna Excel-verktyg. Först beskrivs och förklaras AISC-specifikationer och behövda AISC-designguider, för att sedan fortsätta med beskrivningen av Excel-verktygens skapande. I andra delen förklaras Idea StatiCa, hur det fungerar och löser olika situationer. I sista delen diskuteras jämförelserna mellan Excel-verktygen och Idea StatiCa, vilket som ger strängare resultat i olika situationer och andra upptäckter i undersökningen. Resultatet är tre Excel-verktyg för olika förband, K-förband, T- & Y-förband och pelarfot, samt resultat och iakttagelser från jämförelserna, presenterade i punktform.

Språk: engelska Nyckelord: stålförband, Idea StatiCa, AISC, finita elementmetoden

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Nimike: AISC-spesifikaatioiden ja Idea StatiCa:n välinen vertaileva tutkimus
AISC:iin perustettujen Excel-laskentapohjien kehittäminen

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Tiivistelmä

Eri teräsrakenteiden liitosten suunnittelu koodien mukaan on usein aikaa vievää ja vaatii suunnittelijan tekevän useita oletuksia koska koodit ovat tuotettuja muutamalle vakiotapaukselle. Tämä koskee myös joitakin suunnitteluohjelmia. Tämän työn tarkoitus oli tutkia suunnitteluohjelmaa, Idea StatiCa, joka käyttää uutta menettelytapaa, joka yhdistää elementtimenetelmän ja erilaisiin koodeihin perustuvan komponentti-menetelmän.

Opinnäytetyö koostuu kolmesta pääosasta: AISC:iin perustettujen Excel-laskentapohjien kehittäminen, Idea StatiCa:n teoreettinen tausta ja vertailut Excel-laskentapohjiin. Ensimmäiseksi selitetään AISC-spesifikaatiot ja tarvittavat AISC-suunnitteluoppaat, jatkaakseen Excel-laskentapohjien kehittämisellä. Toisessa osassa Idea StatiCa selitetään, miten se toimii ja miten suorittaa erilaisia tapauksia. Viimeiseksi keskustellaan Excel-laskentapohjien ja Idea StatiCa:n vertailusta, kumpi on konservatiivinen erilaisissa tapauksissa ja millaisia erilaisia löydöksiä on tehty. Tulos on kolme Excel-laskentapohjaa erilaisille teräsliitoksille: K-liitos, T- & Y-liitos ja pilarin liitos betonirakenteihin. Ensisijainen tulos työstä on lyhytsanaisesti pistemuodossa esiteltyjä tuloksia ja löydöksiä suoritetuista vertailuista sekä Excel-laskentapohjissa että Idea StatiCa:ssa.

Kieli: englanti Avainsanat: teräsrakenneliitos, Idea StatiCa, AISC, elementtimenetelmä

BACHELOR'S THESIS

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Abstract

Design of steel connections according to codes can be time consuming and require various assumptions by the designer since the codes are produced for standard cases, which is also the case with some design software's. The purpose of this thesis work is to examine a software, Idea StatiCa, with a new approach of combining Finite Element Method and component model based on different codes. Comparisons are performed with developed Excel tools based on AISC specifications.

The thesis consists of three main parts, development of AISC based Excel tools, Idea StatiCa theoretical background and comparisons with the developed Excel tools. Firstly, AISC specifications and the needed AISC design guides are explained, to then continue with the development of the corresponding Excel tools. For the second part Idea StatiCa is explained, how it works and performs certain tasks. In the last part the comparisons between the Excel tools and Idea StatiCa is discussed, which one is conservative in different situations and other findings. The result is three Excel tools for different connections, K-connections, T- & Y-connections and column base plates. And succinctly presented in bullet points lists the obtained results and detections from the performed comparisons, in both Idea StatiCa and the Excel tools.

Language: English Key words: Steel connections, Idea StatiCa, AISC, Finite Element Method

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1 INTRODUCTION

Design of steel connections can be very time consuming, especially when the connections get more complex. The common 3D design software's are often limited to few basic connections therefore Excel tools are oftentimes developed for various connections. Which is not ideal since it will only work for a specific connection, and thus tools must be developed for every connection type.

This thesis work is divided into three main parts. Straighten out the provisions of AISC, developing AISC based Excel tools for three types of connections, and investigation of a design software, Idea StatiCa, specifically for connections. Idea StatiCa uses a new approach of analysing steel connections, a combination of Finite Element Method and component method used in codes. The purpose of a design software is to minimize workload, time saving and more efficient connections, Idea StatiCa is a software that can analyse connections with an unlimited topology.

The first part of the thesis work, development of Excel tools, is performed first and foremost for results that can be compared to the results obtained from software analysis. In this way the customer will get a better understanding of how the software behaves in different situations with different data or settings for the software. Today the designing of steel connections at Citec is performed in different software's and own developed Excel tools since the software's often are not capable of analysing certain connections. Which ideally should be replaced by a main design procedure or design software capable of analysing a larger range of connections. The truss connections were chosen by the steel team on Citec because the company has no such tools based on AISC, and one of their customers is also expanding in US. The connections were also considered as good objects for a comparative study.

For the comparison's American specifications will be used, "Specification for Structural Steel Buildings", developed by American Institute of Steel Construction(AISC), which will hereafter be referred to as AISC 360. Design guides used for this thesis work are, "Design Guide 24 Hollow Structural Section Connections" and "Design Guide 1 Base Plate and Anchor Rod Design". Both developed by AISC and based on AISC 360, Design Guide 1 is also partly based on ACI (American Concrete Institute) codes, as base plate connections include concrete. And for the welds AISC has developed "Design Guide 24, Welded

Connections”. The design guides will hereafter be referred to as *design guide 24*, *design guide 1* and *design guide 21*, to make referring in the text shorter and neater.

1.1 Customer

The customer of this thesis is Citec Oy Ab, a company that provides engineering services to the energy and power industry and other technology dependent industries[1]. Citec was founded in 1984 with the headquarters located in Vaasa and internationally operative all over the world, and Citec offices can be found in 11 different countries. In 2011 Citec Engineering Oy Ab and Citec Information Oy Ab were consolidated into Citec Oy Ab. The company consists of 6 different sectors, energy, oil & gas, process, manufacturing, civil and vehicles. Citec civil offer a broad range of different services, geotechnical and infrastructure design, foundation design, architectural and building design and construction management, to name a few. Their services also include dynamic analyses and FEM calculations, bill of quantity and mass calculations and other such calculations.[2]

1.2 Purpose and goals

The purpose of this thesis is through examinations and comparisons to find out how the program works, if and how it would benefit the work of Citec’s steel team. Idea StatiCa claims their program can analyse any type of connections with unlimited topology, save time and modelling of more efficient connections. And this being performed without the number of assumptions that frequently must be done by the designer.[3]

But the aim is also to get usable tools for the future. As the customer wants a benchmark, and at times manual calculations are also performed besides software analysis. If quick results are needed, they can often be obtained easier and faster from a simple and working Excel tool than through a software analysis, as that requires modelling of the connection. And one never knows when a customer requires a check manually of connections. For the development of the tools there are various tools on the market, Excel was chosen because of the versatility, suitability for this case, and how widely used it is.

[1] Citec, 10.1.2018 http://www.citec.fi/en-US/Company/Business_idea
[2] Citec, 10.1.2018 http://www.citec.fi/en-US/Sectors/Civil
[3] https://www.ideastatica.com/steel/

How does Idea StatiCa differ from design procedures of AISC. Which situations and settings can be critical in design of a connection in Idea Statica. How reliable is the design software by Idea StatiCa. These are underlying research questions for this thesis work.

2 EXCEL TOOLS & AISC SPECIFICATIONS

This chapter, covering the development of the Excel tools based on AISC Specifications, will firstly handle the Specifications in general. Further the Excel tools will be explained, the buildup, how they work and how certain parts were reasoned and interpreted. Welds are attended both under its own heading and separately for the connections, as they might have certain provisions just for a specific connection.

The terms building code and specifications are sometimes used as synonyms. Which is not very accurate, more correctly building code is a comprehensive document, covering a wide range of topics, generally also all the different aspects correlated to safety. Meanwhile specifications are additional requirements beyond codes and standards, they might overlap one another, most often specifications refer to rules determined by architects or engineers.[4]

2.1 AISC Specifications

The calculations are performed according to the Specifications by American Institute of Steel Construction, which in this case are *AISC 360, design guide 1* and *design guide 24*. *AISC 360* is the result of combining conclusions drawn from researches and the success of engineers practicing. The obtained results from researching have been synthesized into practical design methods, the target is both safe and economical structures.[5] The intention with the Specification is not to provide criteria's and design rules for infrequently occurring problems, but for routine use in design procedures.[6]

AISC 360 provides two different design methods, Allowable Strength Design(ASD) and Load and Resistance Factor Design(LRFD). Structural design has for the past two decades or so been moving towards a more rational and probability-based design procedure, known as "limit state design".[7] Load and Resistance Factor Design will be the approach for this

[4] Salmon, Johnson and Malhas, (2009) p. 38
[5] Salmon, Johnson and Malhas, (2009) p. 38
[6] AISC 360, (2010) p. iii
[7] Salmon, Johnson and Malhas, (2009) p. 38-39

thesis work, and they will hereafter be referred to as ASD and LRFD. The approach of LRFD is built up on probability, calibration with ASD, and judging earlier experience and studies.

$$\boldsymbol{\phi R_n \geq [R_u = \sum \gamma_i Q_i]}$$

R_n	nominal resistance, which is multiplied by resistance factor ϕ for LRFD
R_u	required strength
γ_i	overload factors
Q_i	various load effects

Formula 1 *Design strength formula for LRFD.*[8]

The design strength formula of LRFD can be written as presented in *formula 1*. Where the nominal resistance is multiplied with factor (ϕ), and is required to be at least equal the sum of the factored loads. Where Q is the load type and γ is the factor, which change for different loads.[9]

Advantages of LRFD design are several. LRFD has a more rational approach whereas ASD has a more approximate method. It is the rationality that makes LRFD such attractive, and becomes favorable since it results in more economical use of material for some load combinations. Achieved through use of multiple load factor combinations. LRFD should have a better awareness of structural behavior, which results in safer structures.[10]

2.2 Fillet welds according to AISC

For the examined connections only fillet welds are used, therefore only fillet welds will be covered. For the design of the welds everything can be found in *AISC 360* combined with the *design guide 1* or *design guide 24*, depending on connection type being designed. Hence the use of *design guide 21* is minimal in this thesis work.

Fillet welds (see *Figure 2*) have a triangular cross-section can be applied either to the surface or the edge of the material being joined. They are extensively used in fabrications of steel structures and may be used to add strength to PJP groove welds (see *Figure 1*). The size of a fillet weld is specified through leg size, while determination of weld strength is performed using throat size.[11]

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[8] Salmon, Johnson and Malhas, (2009) p. 40
[9] Salmon, Johnson and Malhas, (2009) p. 40-41
[10] Salmon, Johnson and Malhas, (2009) p. 47
[11] Miller, (2006) p. 37

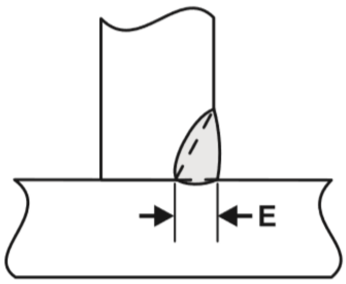


Figure 1 *PJP Groove weld .*[12]

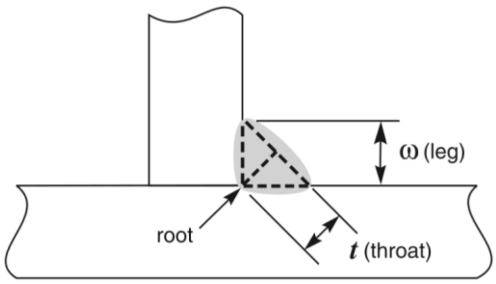


Figure 2 *Fillet weld.*[13]

2.2.1 Size and length limitations for fillet welds

The provisions of maximum fillet weld size are frequently misunderstood, they are only applied in cases of welds along material edges. If material thickness less than 6mm, the leg size can't be greater than the material thickness, while for material thicknesses greater than 6mm the maximum leg size is material thickness minus 2mm. As these provisions are only for welds along material edges it doesn't apply on T-joints. If required capacity is not reached with maximum leg size, it may be possible to use an unequal legged fillet, as long as one of the connected parts is either a surface or thicker than the other joined material.[14]

To insure fusion of the two materials and minimize distortion *AISC 360* provides a minimum weld size, which is based on the thicker of the joined materials. This is shown in a table of *AISC 360-10*, table J2.4 (see *table 1*).[15]

Table 1 *Minimum size of fillet welds depending on material thickness.*[16]

TABLE J2.4
Minimum Size of Fillet Welds

Material Thickness of Thinner Part Joined, in. (mm)	Minimum Size of Fillet Weld, [a] in. (mm)
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19)	5/16 (8)

[a] Leg dimension of fillet welds. Single pass welds must be used.
Note: See Section J2.2b for maximum size of fillet welds.

[12] Miller, (2006) p. 35
[13] Miller, (2006) p. 39
[14] Miller, (2006) p. 38
[15] Salmon, Johnson and Malhas, (2009) p. 201
[16] AISC 360, (2010) p. 111

For end-loaded fillet welds the length of a weld less than 100 times the weld size, the actual length can be set as the effective length. When the length of the weld exceeds the limit, length is determined by multiplying with reduction factor β, which is determined as shown in *Formula 2*. If weld length also exceeds 300 times leg size, effective weld length should be taken as 180 times leg size.[17] Even the stricter limitation is rarely exceeded, and a weld longer than 300 times leg size is extremely rare.[18]

$$\beta = 1.2 - 0.002 * \left(\frac{l}{w}\right) \leq 1.0$$

l is actual weld length

w leg size of weld

Formula 2 *Reduction factor for effective weld length.*[19]

There is always a slight tapering off in both the beginning of the weld and the end of the weld when a fillet weld is applied. Therefore, a minimum fillet weld length is used, which is four times the nominal leg size, if this criterion is not met the size of the weld shall be considered to one fourth of the length of the weld.[20]

2.2.2 Effective areas of fillet welds

The strength of a fillet welds in based on effective area, which is determined by multiplying the effective throat size the with effective length of the weld. Effective throat dimension is the shortest distance from the root to the face of the weld. Thus, the throat size for a normal T-joint with a symmetrical fillet weld will be 0.707 times leg size.[21] An increase of effective throat size is permitted when the welding process produces a penetration beyond the root (see *Figure 3*). Which means leg size can be reduced while maintaining the strength of the weld but before increasing the effective throat a validated testing is required.[22] Effective weld length is determined according to the limitations in previous subparagraph, but for certain connections different provisions and formulas are used, and will be discussed further on in the chapter of the specific connection.

[17] AISC 360, (2010) p.111-112
[18] Miller, (2006) p. 38
[19] AISC 360, (2010) p. 111
[20] Salmon, Johnson and Malhas, (2009) p. 202
[21] Salmon, Johnson and Malhas, (2009) p.204
[22] Miller, (2006) p. 39

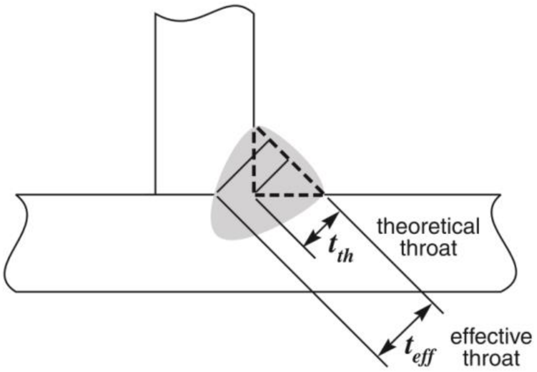


Figure 3 *Effective throat, fillet weld with penetration.*[23]

2.2.3 Nominal strength of welds

The traditional approach for strength of a fillet weld assumed to be through shear stress on effective area, whether the shear transfer is parallel or perpendicular to the weld. Experience and experimentation, have shown that a perpendicularly loaded fillet welds have approximately 50% greater ultimate strength than longitudinally loaded welds.[24] Although the strength is greater for perpendicular shear, they might be treated the same for simplicity.[25] The available strength for a linear weld group with a uniform leg size, loaded through center of gravity is determined as follows:

$$R_n = F_{nw} * A_{we}$$

$F_{nw} = 0.60 * F_{EXX} * (1.0 + 0.5 * \sin^{1.5}\theta)$	nominal stress of weld metal
A_{we}	effective area of weld
F_{EXX}	filler metal classification strength
θ	angle of loading. from weld longitudinal axis (0°=parallel loading)

Formula 3 *Available strength for linear weld group.*[26]

When longitudinal and transversal welds are combined in a weld group, full strength of both welds simultaneously is not permitted. This due to the difference in deformation capacity. And since the load/deformation curves are nonlinear, it is difficult to determine the capacity

[23] Miller, (2006) p. 39
[24] Miller, (2006) p. 39
[25] Salmon, Johnson and Malhas, (2009) p. 204-205
[26] AISC 360. (2010) p. 115

provided by each element to the combination. For this AISC permits the use of the greater of the following:[27]

$R_n = R_{nwl} + R_{nwt}$ or $R_n = 0.85 * R_{nwl} + 1.5 * R_{nwt}$

R_{nwl} nominal strength of longitudinally loaded welds

R_{nwt} nominal strength of transversely loaded welds

Formula 4 *Longitudinal and transversal welds combined.[28]*

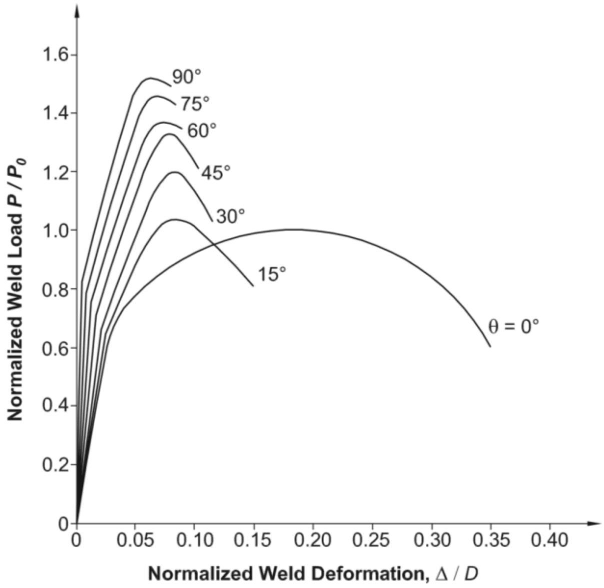


Figure 4 *Deformation capacity for welds with different orientation.[29]*

The design strength of welded joints shall be the lower of base material strength and weld metal strength. Design strength for LRFD received by multiplying the nominal strength by ϕ, which is 0.75 for welded joints. Base metal strength is determined based on tensile rupture strength, as follows:[30]

$R_n = F_{nBM} * A_{BM}$

F_{nBM} nominal stress of base metal

A_{BM} cross-sectional area of base metal

Formula 5 *Nominal strength of base metal in welded joints.*

[27] Miller, (2006) p. 40
[28] Miller, (2006) p. 40
[29] Miller, (2006) p. 40
[30] AISC 360, (2010) p. 113-114

For T-joints there is always a risk for gaps, the two members should be brought as closely as possible to contact but achieving full contact is not always possible. A gap between the members will occur, and as the gap increases the actual throat size decreases. The requirements for this is the following, if gap is larger than 1/16 inch the leg size of the fillet weld should be increased with the same amount as the gap. Designer must also check if gap size meets the requirements for certain material thicknesses.[31]

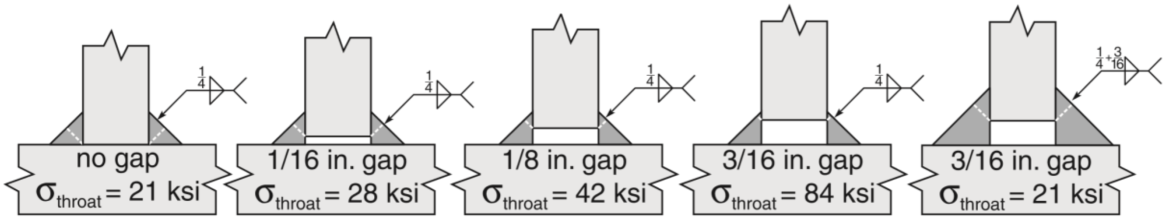


Figure 5 *T-joints, effect of gap on throat.*[32]

2.3 AISC regulations for HSS connections

Design guide 24, Chapter 8 HSS-to-HSS Truss Connections, covers planar truss-type connections of HSS which are connected by welds, and it is based on Chapter K2 of *AISC 360-10*.[33] It contains designing of both round- and square members, since Excel tools are built for box members, round members won't be discussed.

Design guide 24, and Chapter K2 in *AISC 360* (which the design guide is based upon) both presume that the branches are only loaded by axial forces. Designing of truss connections are based on failure modes, or limit states, obtained from international research on HSS. T-, Y-, cross-, K-, N-, gapped or overlapped, are the different forms handled in Chapter 8 of *design guide 24*. The classification of connection is determined by the method of force transfer in the connection, not the physical appearance of the connection.

- T-connection, when the punching load in a branch equals the shear in the chord member, which is when the branch is perpendicular to the chord. Otherwise the connection is classified as a Y-connection.

- K-connection, when punching load in a branch can be equilibrated within 20% by loads of the other branch. If gap size is large the value of noding eccentricity (see

[31] Miller, (2006) p. 42
[32] Miller, (2006) p. 42
[33] Packer, Sherman and Lecce, (2010) p. 91

Figure 7) might be exceeded the limit, in such cases it should be treated as two separate Y-connections. Also, if one of the branches has very little loading the connection can be treated as a Y-connection.

- Cross-connection, when the punching load is transmitted through the chord. Can be 2 chords with compression on top of chord or branches with equal forces on both sides of the chord (with one exception of 2 branches on top of chord, one with compression and one with tension, and reversed on opposite side. Such connection is considered as 2 K-connections on opposite sides.) .[34]

The welded connections in a truss will be semi-rigid, and the stiffness of branches will be significantly less than the stiffness of the chord. Which results in very low bending moments in branches, less than would be reflected by a rigid frame analysis. Hence, it is recommended to use a pin-jointed analysis, or an analysis using web members pin-connected to the chord. The extremely stiff member should have properties greater than the chord member, and a length equal to the noding eccentricity, e. Both these methods will give zero bending moments on the branch members. To produce efficient connections relatively hefty chord members should be chosen, typical square chord members are in the range of 15≤B/t≤25. While branch members should have a high B/t ratio, but within the limits permitted.[35]

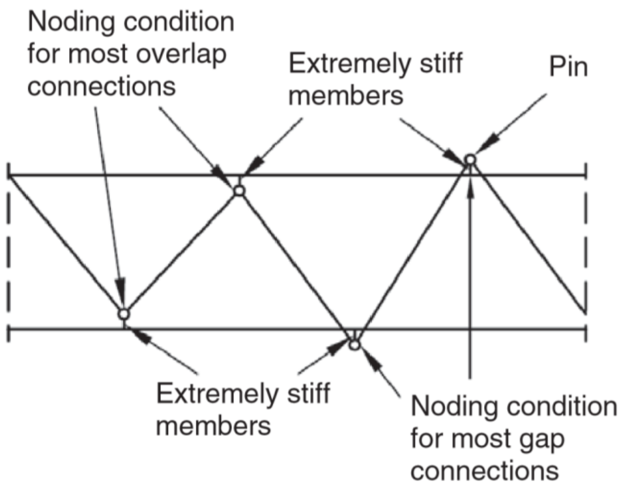


Figure 6 *Modeling assumption, web member pin-connected to continuous chord member.*[36]

[34] Packer, Sherman and Lecce, (2010) p. 91-93
[35] Packer, Sherman and Lecce, (2010) p. 94-95
[36] Packer, Sherman and Lecce, (2010) p. 102

2.3.1 Gapped K-connection & T-connection

Design guide 24 (and *AISC 360-10*) has tabulated succinctly the different failure modes and the design formula for each mode. As well as limits for geometry and materials. Gapped K-connections (and N-connections) require checks for chord shear and effective width of web member, while overlapped connections only need checks for effective width of web.[37] This is obviously considered in the tables of design guide 24. Gapped K-connections are preferred over overlapped because the fabrication is easier, consequently an overlapped connection is more expensive, but likely to provide greater strength and stiffness.[38]

Table 8-2 (continued). Nominal Strengths of Rectangular HSS-to-HSS Truss Connections

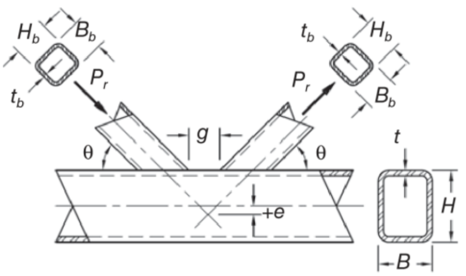
Connection Type	Connection Nominal Axial Strength*
Gapped K-Connections 	Limit State: Chord Wall Plastification, for all β $P_n \sin\theta = F_y t^2 \left(9.8\beta_{eff}\gamma^{0.5}\right)Q_f$ (K2-20) $\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
	Limit State: Shear Yielding (Punching), when $B_b < B - 2t$ Do not check for square branches. $P_n \sin\theta = 0.6 F_y t B \left(2\eta + \beta + \beta_{eop}\right)$ (K2-21) $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
	Limit State: Shear of Chord Sidewalls in the Gap Region. Determine $P_n \sin\theta$ in accordance with *Spec.* Sect. G5. Do not check for square chords.
	Limit State: Local Yielding of Branch/Branches Due to Uneven Load Distribution. Do not check for square branches or if $B/t \geq 15$ $P_n = F_{yb} t_b \left(2H_b + B_b + b_{eoi} - 4t_b\right)$ (K2-22) $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD) $b_{eoi} = \frac{10}{B/t}\left(\frac{F_y t}{F_{yb} t_b}\right) B_b \leq B_b$ (K2-23)

Figure 7 *Gapped K-connection, geometry and design formulas.*[39]

Similar to N-connections being considered as a particular K-connection, T-connection is a particular type of Y-connection. The difference between these two types, one or two branches, is that a single branched connection is resisted by shear and bending in the chord member. Whereas the K- and N-connections are primarily balanced between the branches.[40] Similar to the K-connection, tabulated design formulas provided by AISC can be found in design guide 24 for T- and Y-connections with similar failure modes.

[37] Packer and Henderson, (1997) p. 79
[38] AISC, (1997) Chapter 8 p. 4
[39] Packer, Sherman and Lecce, (2010) p. 98
[40] Packer and Henderson, (1996) p. 76

It should be observed that not all checks are necessary for all geometries when designing according to *Figure 7*, often depending on the branch to chord width ratio. This applies for T- and Y-connections also, since they're based on same failure modes. One should also note that K-connections are restricted to one compression and one tension branch.

2.3.2 HSS-to-HSS moment connections

Achieving full rigidity in unstiffened truss connections is difficult, only connections with $\beta \approx 1{,}0$[41] and a low chord width to chord wall thickness ratio can approach it. All connections apart from such can be considered semi-rigid.[42]

Available testing for moment connections is much less extensive than for axially loaded truss connections. Hence, the limit states from axially loaded connections is used as a basis for the possible limit states for moment connections. *Eurocode 3* and *CIDECT design guide NO 9* are both used as basis for equations of section K3 in *AISC 360-10*, which also *design guide 24* is based on.[43] The connections may be subjected to in plane- and/or out of plane bending moments. For rectangular HSS only T-connections and cross-connections of 90 degrees are covered in *design guide 24,* and the branch centerline is required to be in line with chord center line. Failure modes are succinctly tabulated with design formulas and limits for geometry and materials.[44] While *AISC 360* provides in section K4 tabulated formulas for weld design of moment connections.

[41] Width ratio between chord and branch.
[42] Packer and Henderson, (1996) p. 185
[43] AISC 360-10, (2010) p. 436
[44] Packer, Sherman and Lecce, (2010) p. 123

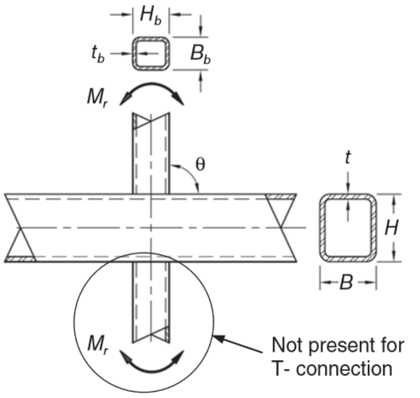
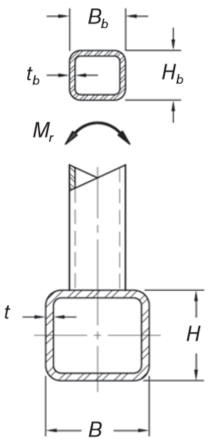
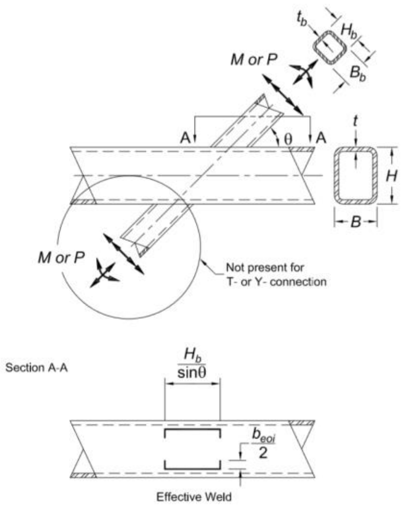
Table 9-2. Nominal Capacities of Rectangular HSS-to-HSS Moment Connections		
Connection Type	**Connection Nominal Moment Capacity***	
Branch(es) under In-Plane Bending T- and Cross-Connections 	Limit State: Chord Wall Plastification, when $\beta \le 0.85$ $M_n = F_y t^2 H_b\left(\frac{1}{2\eta}+\frac{2}{\sqrt{1-\beta}}+\frac{\eta}{(1-\beta)}\right)Q_f$ $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)	(K3-11)
	Limit State: Sidewall Local Yielding, when $\beta \ge 0.85$ $M_n = 0.5F_y^* t\left(H_b + 5t\right)^2$ $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)	(K3-12)
	Limit State: Local Yielding of Branch/Branches Due to Uneven Load Distribution, when $\beta \ge 0.85$ $M_n = F_{yb}\left[Z_b - \left(1-\frac{b_{eoi}}{B_b}\right)B_b H_b t_b\right]$ $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)	(K3-13)
Branch(es) under Out-of-Plane Bending T- and Cross-Connections 	Limit State: Chord Wall Plastification, when $\beta \le 0.85$ $M_n = F_y t^2\left[\frac{0.5H_b(1+\beta)}{(1-\beta)}+\sqrt{\frac{2BB_b(1+\beta)}{(1-\beta)}}\right]Q_f$ $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)	(K3-15)
	Limit State: Sidewall Local Yielding, when $\beta \ge 0.85$ $M_n = F_y^* t(B-t)(H_b+5t)$ $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)	(K3-16)
	Limit State: Local Yielding of Branch/Branches Due to Uneven Load Distribution, when $\beta \ge 0.85$ $M_n = F_{yb}\left[Z_b - 0.5\left(1-\frac{b_{eoi}}{B_b}\right)^2 B_b^2 t_b\right]$ $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)	(K3-17)
	Limit State: Chord Distortional Failure, for T-Connections and Unbalanced Cross-Connections $M_n = 2F_y t\left[H_b t + \sqrt{BHt(B+H)}\right]$ $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)	(K3-19)

Figure 8 *Tabulated design formulas for HSS-to-HSS moment connections.*[45]

Formulas for in plane bending moments and out of plane bending moments are provided as shown in *Figure 8*. For biaxial bending the sum of the utilization ratios should be less than 1 or 100%, if branch is subjected to axial loads its utilization ratio should also be included in the summation. It should be noted that these formulas are limited to T-connections, in other words with an angle of approximately 90 degrees.

[45] Packer, Sherman and Lecce, (2010) p. 126

T-, Y- and Cross-Connections Under Branch Axial Load or Bending



Effective Weld Properties

$$l_e = \frac{2H_b}{\sin\theta} + 2b_{eoi} \quad \text{(K4-5)}$$

$$S_{ip} = \frac{t_w}{3}\left(\frac{H_b}{\sin\theta}\right)^2 + t_w b_{eoi}\left(\frac{H_b}{\sin\theta}\right) \quad \text{(K4-6)}$$

$$S_{op} = t_w\left(\frac{H_b}{\sin\theta}\right)B_b + \frac{t_w}{3}\left(B_b^2\right) - \frac{(t_w/3)(B_b - b_{eoi})^3}{B_b} \quad \text{(K4-7)}$$

$$b_{eoi} = \frac{10}{B/t}\left(\frac{F_y t}{F_{yb} t_b}\right)B_b \le B_b \quad \text{(K2-13)}$$

When $\beta > 0.85$ or $\theta > 50°$, $b_{eoi}/2$ shall not exceed $2t$.

Figure 9 *Tabulated formulas for weld design of HSS-to-HSS moment connections.*[46]

In *Figure 9* the formulas for effective length and the effective elastic section modulus of the welds for bending in each direction. Since connection with bending moments are limited to 90 degrees and sin(90) equals 1, the angle is unnecessary in the formula for connections with moments.

2.4 Excel tools for HSS connections

This chapter describes the build-up of the Excel tools for HSS connections, decisions and assumptions for the development, and how it is structured with different functions.

2.4.1 Excel tools for K-connections

Decisions and assumptions for the K-connection design tool:

- The design tool for K-connections takes load input for axial forces only, since the design guide by AISC does not provide design guides for K-connections with branches subjected to bending moment. For the chord member there is separate inputs for axial force on both sides of the connection, it can be tensioned or compressed. While axial force in branches is restricted to one tension member and one compression member, since the connection would be considered a X-connection with compression in both branches.

[46] AISC 360, (2010) p. 160

- If the difference between branch forces become too large, the connection is calculated partly as a K-connection and partly as a Y-connection (see previous chapter for regulations).

- If noding eccentricity limit for branches is exceeded because of a large gap, the user is directed to the T- & Y- design tool (to calculate them as two separate, in such cases the shear in chord member must be checked).

- Welds are calculated as a weld group.

- All calculations are performed even if every failure mode check is not always required, but the calculations for the ones not required are hidden.

2.4.2 Excel tool for T- & Y-connections

Decisions and assumptions for the T- & Y-connection design tool:

- Chord axial forces are similar as for K-connection, but if branch angle is 90 degrees and connection considered a T-connection, both in plane- and out of plane bending input is possible besides the axial force input.

- For weld strength formulas tabulated in AISC 360 is used, but for base metal strength the force per mm of weld caused by bending moment, separately in each direction is summarized. Axial force in branch is also translated to N/mm of weld, the sum of these is used to get the utilization ratio.

- Other checks and calculations have a similar build up as the tool for K-connections.

2.4.3 Structure of tools for K-, T- and Y-connections

Since the purpose of the development of the tools was to get results based on AISC for comparison, the strive was to keep the tools as simple as possible without using any coding. The tools are built up mostly by using logical functions, as “IF”, “AND” and “OR”. In purpose to make the checking clearer and hiding unnecessary calculations to prevent misunderstanding “Conditional formatting” was used. Hiding of different calculations was performed with formulas in “Conditional formatting” and turning the text white, which is not the most elegant way but the only way without coding and saves a lot of time.

In *Appendix 2* it can be noted that there are inputs for bracings, this however is not included in the thesis and therefore only the part for T- and Y- connections is presented.

2.5 AISC regulations for column base plates

Column base plate connections are crucial, as they're the parts connecting the steel structure with the foundation, which in this study will be concrete. AISC have developed a design guide solely for column base plates, covering fabrication, installation, repairs and design of base plate connections. For this thesis work the design of base plates is the only subject studied. The design guide covers typical base plate connections, discussing five different design load cases, which are following:[47]

- Concentric compressive axial loads
- Tensile axial loads
- Base plates with small moments
- Base plates with large moments
- Design for shear

Design for shear force and design for bending moments are executed separately. This leads to an assumption of no significant interaction of the two. Column base plate connections have an elastic behavior until one of the following failures occur:

- a plastic hinge occurs in the column
- plastic mechanism occurs in base plate
- tension in anchor rod causes yielding
- supporting concrete is crushed
- pullout strength in concrete is reached for anchor rod group

if any of first four failure modes have a lower strength than the pullout strength of the concrete, the connection will generally be ductile.[48]

[47] Fischer and Kloiber, (2006) p.6-8
[48] Fischer and Kloiber, (2006) p.13

2.5.1 OSHA requirements

Occupational Safety and Health Administration has regulations concerning column base plates, in “Safety Standards for Steel Erection”. The regulations limit base plates to a minimum of 4 anchor bolts, unless the column doesn’t weigh under 135kg (300 pounds). And a minimum moment strength to resist an eccentric gravity load of 135 kg (300 pounds), placed 450 mm (18 inches) from the outer face of the column, to all directions.[49]

2.5.2 Concentric compressive axial loads

For axial compression loads, the base plate must be large enough for the bearing strength of the concrete. Base plate thickness will be controlled by the bending strength of the plate, by the cantilever method for base plates with significantly larger base plates than columns. And for base plates not extending much beyond the edge of the column, a different approach is required. In this case it can be treated as uniformly loaded over a H-shaped area.[50]

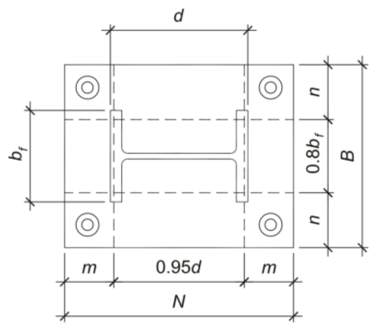


Figure 10 *Column base plate with significantly larger plate than column, dashed lines shows the line where bending is assumed to occur.*[51]

Design guide 1 offers two alternatives to determine concrete compressive strength, one from *AISC 360* and one from *ACI 318-02* (American Concrete Institute). The formulas are similar for nominal strength, but resistance factor ϕ for LRFD differs, as *AISC 360* stipulates it to 0,6 *and ACI-318* on the other hand stipulates a factor of 0,65. The authors of *design guide 1* recommend use of the factor specified by ACI. The concrete bearing strength can be determined as follows:[52]

[49] Fischer and Kloiber, (2006) p. 14
[50] Salmon, Johnson and Malhas, (2009) p. 747
[51] Fischer and Kloiber, (2006) p. 15
[52] Fischer and Kloiber, (2006) p. 14

$$f_{pu.max} = \phi * 0{,}85 * f_c' * \sqrt{\frac{A_2}{A_1}} \quad \text{and} \quad \sqrt{\frac{A_2}{A_1}} \leq 2 \quad (f_{pu.max} = \phi P_p)$$

$f_{pu.max}$	max bearing strength of concrete (LRFD)
$f_c{}'$	specified compressive strength of concrete support
A_1	Area of base plate.
A_2	Maximum of area of supporting surface, geometrically similar and concentric with the load.

Formula 6 *Concrete bearing strength for compressive axial loads, for base plates.*[53]

Column base plates can be supported by a layer of grout as an alternative to concrete. Since the compressive strength for grout is greater than compressive strength of concrete, it is recommended to use grout strength as two times concrete strength as f_c'.

For axially loaded columns the compressive stress will cause a bending moment on the plate for the part extending beyond the column outer face (see *Figure 11*). More precisely at the yield lines, shown in *Figure 10* for W shape columns. The required strength of the base plate can be determined as follows:[54]

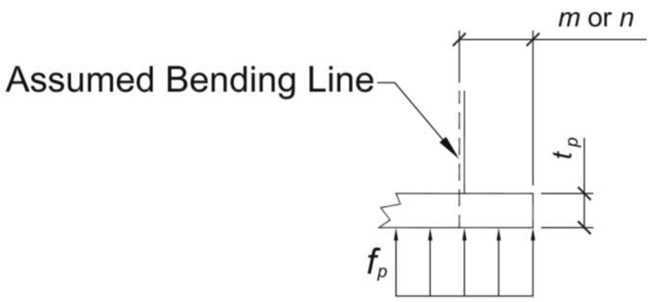


Figure 11 *Plate thickness determined by bending moment.*[55]

[53] Fischer and kloiber, (2006) p. 14
[54] Fischer and Kloiber, (2006) p. 15-16
[55] Fischer and Kloiber, (2006) p. 15

$M_{pl} = f_{pu} * (\frac{l^2}{2})$ for LRFD

$l = \max(m, n, \lambda n')$ cantilever length (see *Figure 8 or 9*)

n' cantilever distance from column web or flange

$\lambda = \frac{2*\sqrt{X}}{1+\sqrt{1-X}}$

$X = \frac{4db_f}{(d+b_f)^2} * \frac{P_u}{\phi P_p}$ for LRFD

P_u required axial compressive load, LRFD

Formula 7 *Required bending strength of base plate.*[56]

Base plates subjected only to axial compression have three general cases in the design procedure (where A_1 and A_2 is same as in *Formula 6*):

1. Case I: $A_2 = A_1$
2. Case II: $A_2 \geq 4 * A_1$
3. Case III: $A_1 < A_2 < 4 * A_1$

A direct and conservative approach is to use Case I, without taking the size and shape of the supporting surface into account. And the most economical plates are usually received when *m* and *n* (show in *Figure 10*) are equal.

What differs in use of HSS columns is: 0,95 times both profile depth and width, and for pipes 0,8 times the diameter of the pipe is used. Irrespective of which shape of HSS is used, the term λ is not used in design.[57]

2.5.3 Concentric tensile forces

Tensile forces on base plate connection occur when the uplift caused by wind loads, exceed the dead load of the roof and other possible loads on the roof. Design of base plate and anchors for tensile forces require checks of tensile strength of anchor, base plate yielding strength and concrete strength (except if additional reinforcement is used to resist the tensile forces).[58]

[56] Fischer and Kloiber, (2006) p. 16
[57] Fischer and Kloiber, (2006) p. 15-17
[58] Fischer and Kloiber, (2006) p. 18

Base plate design against uplift should be performed in the following order: anchor rods are checked by dividing the uplift to force per rod, then compared to the strength per anchor rod. For anchor rod design prying forces are usually neglected, since it's usually justified in base plate thickness design by assuming bending yield lines at column flanges or web. Base plate thickness for larger plates is determined by bending moment about flanges, generated by rod loads. If the connection is pinned and rods are places between column flanges, one way bending about the web can be used for simplicity. If the web is designed to take the anchor loads, the web and its attachment to the base plate should be checked.[59]

The design tensile strength for anchor rods in tension is taken as the smaller of summarized steel tensile strength and concrete tensile strength of the anchor group. To determine the tensile strength of the steel, there is two methods. One defined by ANSI/ASME[60] and one by AISC specifications.

$\left(D - \frac{0{,}7854}{n}\right)^2$		tensile stress area, by ANSI/ASME
	D	major diameter
	n	number of threads per in.
$0{,}7 * A_b$		tensile area, by AISC specifications
	A_b	bolt area without decrease for threads

Formula 8 *2 methods for tensile strength of bolts.*[61]

AISC specifications uses a reduction factor of 0,7, thus it relates to the unthreaded part. The direct tensile stress area is also stipulated by ACI 318 in Appendix D. The designer should pay attention to the difference and be consistent through the design process.[62] Concrete pullout strength is based on ACI Appendix D, and is determined as follows:

$\phi N_p = \phi \psi_4 A_{brg} 8 f_c'$		design strength, LRFD ($\phi = 0{,}75$)
	ψ_4	1,4 for uncracked support, otherwise 1,0
	A_{brg}	bearing area of rod head
	f_c'	concrete compressive strength

[59] Fischer and Kloiber, (2006) p. 18
[60] American National Standards Institute and American Society of Mechanical Engineers
[61] Fischer and Kloiber, (2006) p. 19
[62] Fischer and Kloiber, (2006) p. 19

Formula 9 *Concrete pullout design strength for anchor.*

For anchors of higher strength, washer might be necessary to obtain full strength of anchor. But the washer size should be kept as small as possible, since unnecessarily large ones could reduce the concrete resistance to pullout.[63]

Concrete Capacity Design (CCD) method considers the breakout to be cone shaped, with an angle of approximately 34 degrees, 1:1,5 slope for simplicity in calculations. Consequently, the strength increase in the CCD method is proportional to the embedment depth to the power of 1,5, or to the power of 5/3 if the embedment depth exceeds the limit. The method is valid for anchor dimensions not exceeding 50mm in diameter or an embedment length of 635mm. The concrete breakout strength for an anchor group is determined in *ACI 318-02, Appendix D* as follows:[64]

$$\phi N_{cbg} = \phi \psi_3 24\sqrt{f_c{}'} h_{ef}^{1,5} * (A_N/A_{No}) \qquad \text{when } h_{ef} < 280\ mm$$

$$\phi N_{cbg} = \phi \psi_3 16\sqrt{f_c{}'} h_{ef}^{5/3} * (A_N/A_{No}) \qquad \text{when } h_{ef} \geq 280\ mm$$

ϕ	for LRFD design strength (=0,7)
ψ_3	1,25 for uncracked concrete at service loads, otherwise 1,0 (and 80% of concrete capacity values should be used)
A_N	breakout cone area for anchor group
A_{No}	breakout cone area for single anchor

Formula 10 *Design strength for concrete breakout for a group of anchors.*[65]

In the same appendix by ACI, criteria listed for anchor rods to prevent lateral bursting forces at the anchor head are also found, caused by the tension force. This failure is also assumed to be cone shaped, from the anchor head to the concrete surface. To avoid this a minimum concrete cover (c_1) of 6 times the rod diameter is recommended. Use of washers increases the bearing area, and thus increases the side face blowout strength. In some cases, the concrete area is too small to develop a tensile strength enough, one example is piers. Such cases require additional steel reinforcement to be able to handle the anchor tension forces. If steel reinforcement with an overlap on the anchor is used, the anchor strength can be taken as:[66]

[63] Fischer and Kloiber, (2006) p. 20-21
[64] Fischer and Kloiber, (2006) p. 20-21
[65] Fischer and Kloiber, (2006) p. 21
[66] Fischer and Kloiber, (2006) p. 22

$\phi A_{se} F_y$ $\phi = 0{,}9$ for LRFD

A_{se} effective cross-sectional area

F_y anchor rod material yield strength

Formula 11 *Anchor rod design tensile strength when anchor is designed to lap with reinforcement.*[67]

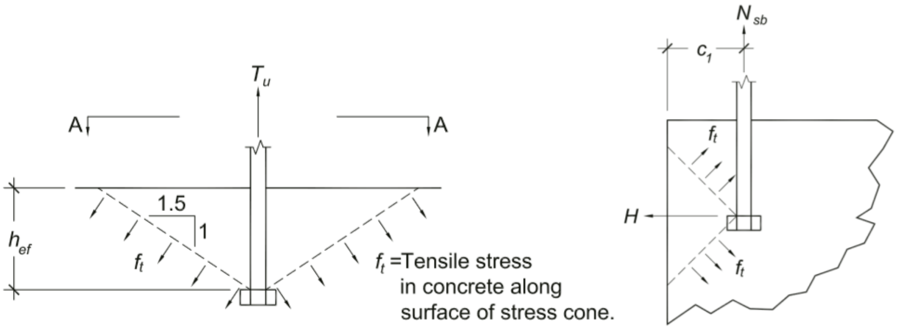


Figure 12 *Breakout cones caused by tension and lateral bursting force when anchor is near edge.*[68]

2.5.4 Bending moments

In addition to axial forces, base plates are often required to resist bending moments. If the axial force is compression, the base plate is precompressed and when moment is applied the compression is reduced in the section of tension for bending.[69] AISC provides different design procedures for small and large bending moments, which is based on the moment to axial force ratio. The definition of small and large eccentricities is based on the assumption of uniform bearing stress. If the eccentricity of column compressive divided by the bending moment exceeds the eccentricity of the resultant force of maximum bearing pressure, the loads can't be resisted by bearing alone and anchors will have to handle the remaining forces as tension. This leads to following inequity:[70]

$e \leq e_{crit}$ satisfied for small, else large bending moment

$e = \frac{M_u}{P_u}$ eccentricity of column force

$e_{crit} = \epsilon_{max} = \frac{N}{2} - \frac{P_u}{2q_{max}}$ critical eccentricity, max eccentricity for bearing resultant

q_{max} max bearing stress of concrete

Formula 12 *Bending moment eccentricity, provisions to determine small and large bending moments (see Figure 12).*

[67] Fischer and Kloiber, (2006) p. 22
[68] Fischer and Kloiber, (2006) p. 21&23
[69] Salmon, Johnson and Malhas, (2009) p. 751
[70] Fischer and Kloiber, (2006) p. 23-24

2.5.5 Small bending moments

If the eccentricity limit is satisfied, the bending moment is considered small and anchors won't be subjected to tension. Bearing stress between concrete and base plate will cause bending moment on the cantilever of the plate. Formula for required plate thickness depends on if bearing length (Y) is larger or smaller than cantilever length:[71]

$t_{preq} = 1{,}5m\sqrt{\frac{f_p}{F_y}}$ required plate thickness for $Y \leq m\ or\ n$

$t_{preq} = 2{,}11\sqrt{\frac{f_p Y\left(m-\frac{Y}{2}\right)}{F_y}}$ required plate thickness for $Y > m\ or\ n$

m	cantilever length (n for bending in the other axis)
F_y	yield strength of base plate material
$Y = N - 2 * e$	bearing length for compression
e	eccentricity of column force
$f_p = \frac{q}{B}$	bearing stress between plate and concrete
B	plate width (N in other direction)

Formula 13 *Required plate thickness with small bending moments (see Figure 13 for clarifying).*[72]

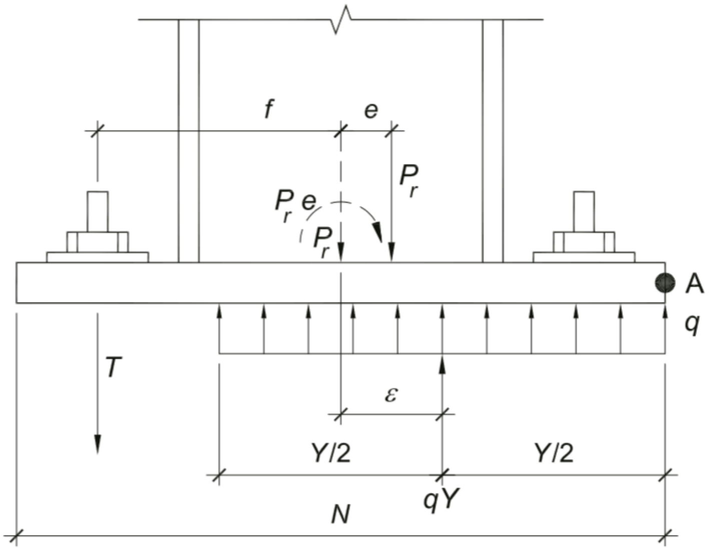


Figure 13 *Base plate subjected to bending moment.*[73]

71 Fischer and Kloiber, (2006) p. 24-25
72 Fischer and Kloiber, (2006) p. 25
73 Fischer and Kloiber, (2006) p. 23

2.5.6 Large bending moments

When bending moment is large relative to axial force, the anchors are required to take some tension to prevent the column from not tipping over. The idea is to use the concrete compressive strength to its maximum for the bearing part and remaining forces transmitted as tensile forces to opposing anchors. To get the bearing length for large bending moments, tension anchor is set as rotation point for moment calculation and the summation of moments caused by concrete stress and base plate compression must equal to zero. Hence, the formula for bearing length (Y) can be derived from the moment equilibrium, and bearing length can be taken as (see *Figure 14* for clarification):[74]

$$Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2P_u(e+f)}{q_{max}}}$$

q_{max}	maximum concrete bearing pressure
P_u	axial force in column

Formula 14 *Compression bearing length for large bending moments.*[75]

For certain force, moment and geometry combinations a valid solution is not possible for the equation for bearing length. In such cases a increased plate size is required, which is determined by the following inequity (see *Figure 14* for clarification):

$$\left(f + \frac{N}{2}\right)^2 \geq \frac{2P_u(e+f)}{q_{max}}$$

Formula 15 *Inequity for checking of validity of connection, if not satisfied an increase of plate size is required.*[76]

Required base plate thickness for the bearing interface is determined with the same formulas as used for small bending moments, except that bearing stress is changed to maximum bearing stress (f_p to $f_{p(\max)}$). The final base plate thickness is taken as the smaller of required thickness at compression interface and tension interface. At the tension interface the anchors must be checked for tensile forces and the base plate for bending caused by tension force in anchors. Check can be performed either by determining required bending strength or by determining the required plate thickness. Required plate thickness can be determined as follows:[77]

[74] Fischer and Kloiber, (2006) p. 25-26
[75] Fischer and Kloiber, (2006) p. 26
[76] Fischer and Kloiber, (2006) p. 27
[77] Fischer and Kloiber, (2006) p. 26-27

$t_{preq} = 2{,}11\sqrt{\frac{T_u x}{BF_y}}$ required plate thickness, LRFD

$T_u = q_{max}Y - P_u$ tensile force in anchors

q_{max} maximum bearing pressure

Formula 16 *Required plate thickness due to tension force in anchors, for large bending moments.*[78]

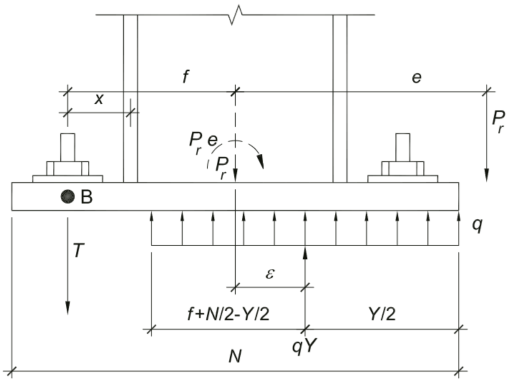


Figure 14 *Base plate with large bending moment which means eccentricity of axial load is larger than the eccentricity of resultant force of bearing pressure. Consequently, bearing alone is not enough and anchors are subjected to tension.*[79]

2.5.7 Shear forces

Shear forces can be transferred in 3 different ways to the concrete support:

- Friction between base plate and concrete/grout.
- Shear lug or base plate embedded in concrete.
- Shear in anchor rods.

Compression force in column might produce enough friction between base plate and concrete. But if friction is used to handle the shear, the most unfavorable load combination must be used for compression. Shear strength can be calculated as stipulated in ACI:[80]

$\phi V_n = \phi \mu P_u \leq 0{,}2 f_c' A_c$ shear strength by friction

μ friction coefficient, 0,55 for concrete and 0,7 for grout

A_c area of supporting surface

Formula 17 *Friction shear strength developed by compression.*[81]

[78] Fischer and Kloiber, (2006) p. 27
[79] Fischer and Kloiber, (2006) p. 26
[80] Fischer and Kloiber, (2006) p. 27
[81] Fischer and Kloiber, (2006) p. 27

The second way of handling the shear forces is by welding a shear lug to the plate, or by embedment of the column in concrete. ACI permits use of increased strength by initially transferring shear through anchors to concrete by tension developed in anchors and the shear is progressed into a shear friction mode. The shear strength for shear lugs or bearing on column is expressed in the following formula, where second half of formula is confinement strength by anchors:[82]

$\phi P_n = 0{,}8 f_c' A_l + 1{,}2(N_y - P_a)$	for shear lugs
$\phi P_n = 0{,}55 f_c' A_{brg} + 1{,}2(N_y - P_a)$	for bearing on column or side of base plate
$A_l \; and \; A_{brg}$	bearing area of shear lug or column
$N_y = nA_{se}F_y$	yield strength of tension anchors
P_a	external axial load (positive for tension)

Formula 18 *Shear strength of shear lug or embedded column, with additional strength by anchors.*[83]

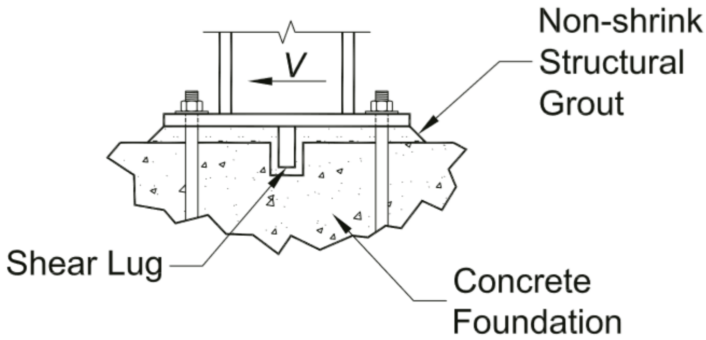


Figure 15 *Detail of shear lug.*[84]

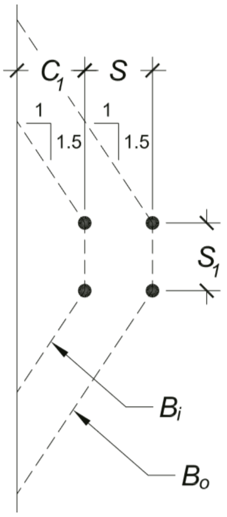
The last way of designing the shear strength is by using the anchors. Which should be carefully performed, since the design requires several assumptions. A cautious approach of using 2 anchors to resist the shear is recommended by the authors. This because of a small slip of the base plate may occur before bearing against the rods, and the placement tolerances will lead to an uneven distribution of forces between the anchors. Even force distribution can be achieved by welding washers to the base plate. This on the other hand will cause bending moment on the anchor within the thickness of the base plate, which must be taken into consideration. For a typical cast-in-place anchor group, the shear strength for concrete breakout is determined as follows:[85]

[82] Fischer and Kloiber, (2006) p. 28
[83] Fischer and kloiber, (2006) p. 28
[84] Fischer and Kloiber, (2006) p. 28
[85] Fischer and Kloiber, (2006) p. 29

$\phi V_{cbg} = \phi \frac{A_v}{A_{vo}} \psi_5 \psi_6 \psi_7 V_b$	shear concrete breakout strength
A_v	total breakout area for a group of anchors
$A_{vo} = 4{,}5 c_1^2$	area of full shear cone for a single anchor
ϕ	0,7 for LRFD
ψ_5	1 for all anchors at same load
ψ_6	modifier to adjust the capacity if when side cover limits the breakout cone size
ψ_7	1,0 for uncracked concrete, otherwise 1,4
$V_b = 7 \left(\frac{l}{d_o}\right)^{0{,}2} \sqrt{d_o} \sqrt{f_c{}'} c_1^{1{,}5}$	concrete strength for single anchor
c_1	anchor to concrete edge distance
l	embedment depth
d_o	rod diameter

Formula 19 *Shear strength of anchors for concrete breakout.*[86]

There are two potential breakout surfaces for anchors, the breakout starting from the anchors closer to edge or anchors further away. If the anchors closer the edge determine the strength the total shear force should carry all the shear. And if the anchor further away from edge control the strength the shear is even on anchors. The breakout surface is determined in the following table.[87]



S_1/S	C_1/S For B_o To Control
0.5	> 2.33
2/3	> 2.31
1.0	> 2.26
1.5	> 2.19
2.0	> 2.12

Figure 16 *Slope of breakout cone and directives for controlling breakout surface for shear on anchors.*[88]

[86] Fischer and Kloiber, (2006) p. 29
[87] Fischer and Kloiber, (2006) p. 30
[88] Fischer and Kloiber, (2006) p. 30

In addition to the concrete breakout strength ACI also provides checks for pryout strength. This rarely determine the strength and is expressed as:

$V_{cp} = k_{cp}N_{cp}$		pryout strength
	k_{cp}	1,0 for $h_{ef} \leq 2{,}5\ in.$ otherwise 2,0
	h_{ef}	effective embedment length of anchor
	N_{cp}	nominal concrete breakout strength for tension

Formula 20 *Pryout strength for anchors in shear.*[89]

For interaction between tension and shear design guide 1 refers the reader to *ACI 318 Appendix D*. If utilization rate of shear is equal to or less than 20% the full strength of tension is permitted, the same goes other way around. If both shear and tension exceeds 20% the summation of the utilization ratios shouldn't exceed 120%.[90]

2.6 Excel tool for column base plates & anchors

Decisions and assumptions for development of the excel tool:

- The tool is made for rectangular HSS and W shape columns, for W shape column options of HEA and HEB. Both fixed and pinned are possible for W shape.

- Possible load inputs are: moment in both weak and strong axis, axial force which can be both compression or tension, and shear force in both directions.

- Inputs for geometry and size of supporting concrete, for concrete checks.

- For anchor selection products of 2 fabricators is found as options. The strengths found in fabricators documents aren't used, but the dimensions for strength calculations according to *design guide 1*.

- *Design guide 1* does not cover biaxial bending, but a decision was made during the thesis to try a simplified method by adding the required plate thicknesses obtained from separate calculations for both directions with SRSS[91]. For tension forces in anchors the critical anchor is checked in the corner which is subjected to tension by both bending moments.

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[89] Fischer and Kloiber, (2006) p. 30
[90] ACI 318, (2005) p. 403
[91] Square root of the sum of the squares

- Weld checks are performed by splitting up bending moments to compression and tension components and calculating the resultant force of shear- and tension forces per mm of weld. The critical part with the highest stress/force is then checked against weld strength per mm.

For the buildup of the excel tool the system of using logical functions and “Conditional formatting” is similar to the previous excels. For selection of different profiles and materials “vlookup” is used, different sheets are created for columns, anchor rods and weld materials. Since this excel includes more options and some of the might be dependent on another one, logical functions are used in “data validation” to make a dropdown list dependent on earlier selections. For some selections, like column, a combination of logical functions and “vlookup” is used to get the tool to search in the corresponding table. The many options might confuse the user with the different options of dimensions and knowing what is what, to prevent this formulas for different pictures was made. This by creating a sheet with pictures and using “index” and “match” as shown in *Figure 17*.

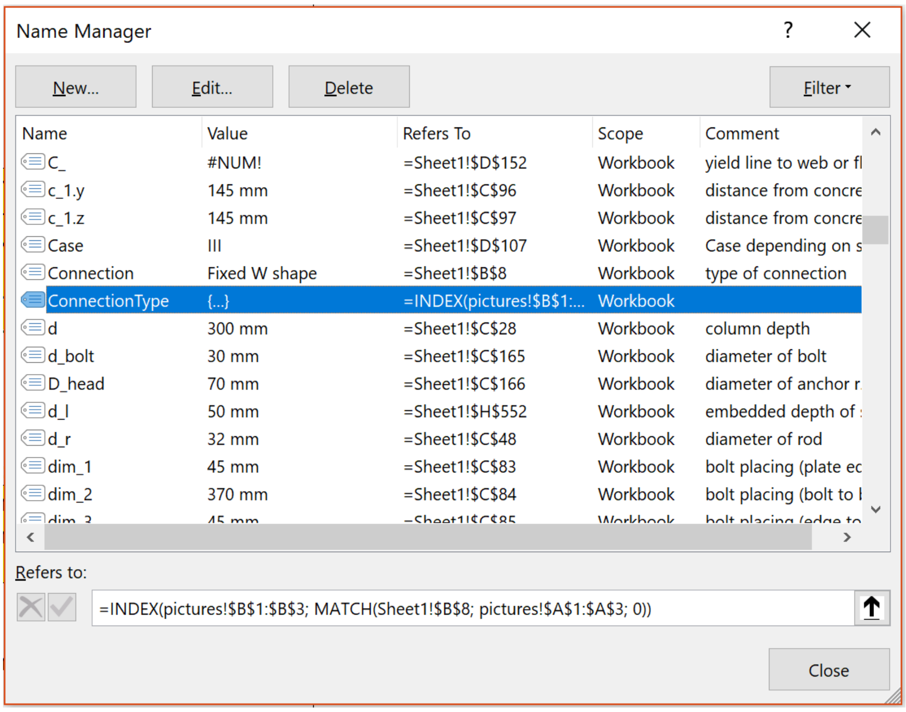

Figure 17 *The corresponding connection type and the dimensions for input is displayed with the help of this formula.*

3 BACKGROUND OF IDEA STATICA

Idea StatiCa is a relatively new company, founded in 2009. In March 2014 they released the first version of the application, this after 5 years of research and theoretical preparations. Documentation of Theoretical background, user guides, verification and articles and such can be found on their recourse center at their website. In this chapter the theoretical background of Idea StatiCa steel connections will be discussed. To keep the thesis work from not being too extensive it will be limited to general discussion about finite elements and which theories are applied in Idea StatiCa, without going deeper into how the calculations are performed.

The theory of bar members instead of a real model is not valid in many cases, like welded joints, bolted connections, footing and holes in walls for a few examples. Nonlinearities in such connections must be respected, therefore analysis of the connections requires special attention.[92]

3.1 CBFEM

The component method solves connections as system of interconnected items, and determining forces and stresses in each component. Each component is then checked, each by the corresponding formula. This means it is required to create a model for each joint type, and the method is limited to general shapes and loads. Therefore, Idea StatiCa together with project team of Department of Steel and Timber Structures of Faculty of Civil engineering in Prague and Institute of metal and timber structures of Faculty of Civil engineering of Brno University of Technology developed a new method for advanced design and analysis of steel connections. The result is the CBFEM method (Component Based Finite Element Model), and the idea is to keep the most verified and useful parts of the component method. And finite elements method replacing the component methods weakness, which is its generality in the approach of analyzing stresses for individual components. [93] The advantage of component model is the experimental and analytical knowledge of the behavior of connections components such as bolts, welds and plates. This results in an accurate prediction of elastic and ultimate loading.[94]

[92] Idea StatiCa, (2017) p. 5
[93] Idea StatiCa, (2017) p. 4
[94] Sabatka et al. (2014) p. 1

3.2 Components of CBFEM

FEM (Finite Element Method) is a computer based procedure for analyze of structures and continua. It is used in almost every scope of engineering analysis, for its versatile numerical method it uses for problem solving. Evolvement in computer hardware have made the use of finite element software for problem solving easy and efficient, even from personal computers. The main difference between classical methods and finite elements is how they approach the structure and the solution procedure. Classical methods consider the structure as a continuum, while the finite element method considers the structure to be built up of small finite-sized particles, finite elements. Classical method determines the behavior of the structure by partial or ordinary differential equations. Finite elements use a system of algebraic equations to determine the behavior of the elements and overall structure, commonly solved by a computer.[95]

Finite element method is a commonly used method for structural analysis, and FEM for connections has been used since the 70's. Thanks to the ability to express the real behavior of connections makes it an alternative to testing.[96] Usually the steel plasticizes in structures, and the elastic-plastic analysis is required. Hence, results from a linear analysis for joint design are useless.[97]

3.2.1 Material

The three most common material diagrams used in FEM of structural steel are ideal plastic, elastic model with strain hardening and true stress-strain (see *Figure 10*). True stress-strain is calculated from the material properties of mild steels obtained from tensile tests, the true stress and strain can be obtained as shown in *Formula 21*. Material behavior is based on von Mises yield criterion, and assumed to be elastic until reaching the yield strength.[98]

[95] Spyrakos, (1994) p. 1-2
[96] Sabatka et al. (2014) p. 2
[97] Idea StatiCa, (2017) p. 5
[98] Idea StatiCa, (2017) p. 5

$$\sigma_{true} = \sigma(1+\epsilon) \qquad \text{and} \qquad \epsilon_{true} = \ln(1+\epsilon)$$

σ	nominal stress
ϵ	nominal strain
σ_{true}	true stress
ϵ_{true}	true strain

Formula 21 *True stress and strain.*[99]

The stress and strain curves are determined by values for stress obtained by dividing the load by the cross-section area, and strain values by dividing the elongation by the original length, this is known as an engineering stress-strain. The already mentioned true stress-strain uses the actual cross-section area even after the material is tensioned and thus a locally reduced cross-section area.[100]

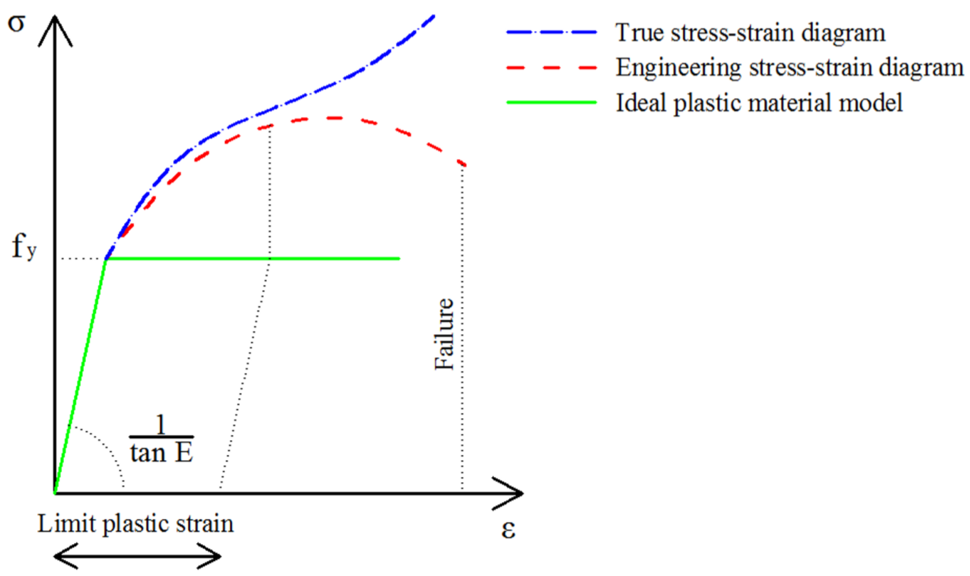

Figure 18 *Material diagrams in numerical models.*[101]

For ideal plastic model, the limit value has a low sensitivity on the ultimate load. This is shown in the document of theoretical background by Idea StatiCa. The influence of the limit on ultimate load is shown with a IPE 180 beam connected to a HEB 300, loaded by bending moment. The following figure shows the influence for limits from 2% to 8%. Eurocodes recommend a limit of 5%.[102]

[99] Idea StatiCa, (2017) p. 7
[100] Salmon, Johnson and Malhas, (2009) p. 59-60
[101] Idea StatiCa, (2017) p. 7
[102] Idea StatiCa, (2017) p. 6-7

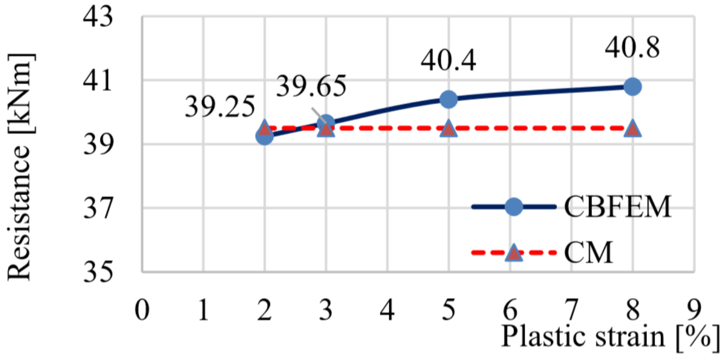

Figure 19 *influence of limit value on ultimate load, for ideal plastic model.*[103]

According to *AISC 360-10* commentary the strength of a connection can be determined by an ultimate limit-state model of the connection, or physical test. When the moment rotation response doesn't show any peaks, the strength can be taken as the moment at a rotation of 0,02 rad.[104] From examples performed by Idea StatiCa, with a plastic strain limit of 5% and the limit of 0,02 rad rotation by the moment governed by AISC, strengths with small differences are obtained. The example is a W shape beam welded to a W shape column, with a bending moment of 45,5kip-in (kilo pound-inch, 45,5kip-in≈5,15kNm). The bending resistance obtained with the 5% strain limit is 408,5 kip-in, and at rotation of 0,02 rad is 402,5 kip-in. Therefore 5% is also recommended as the limit for AISC.[105]

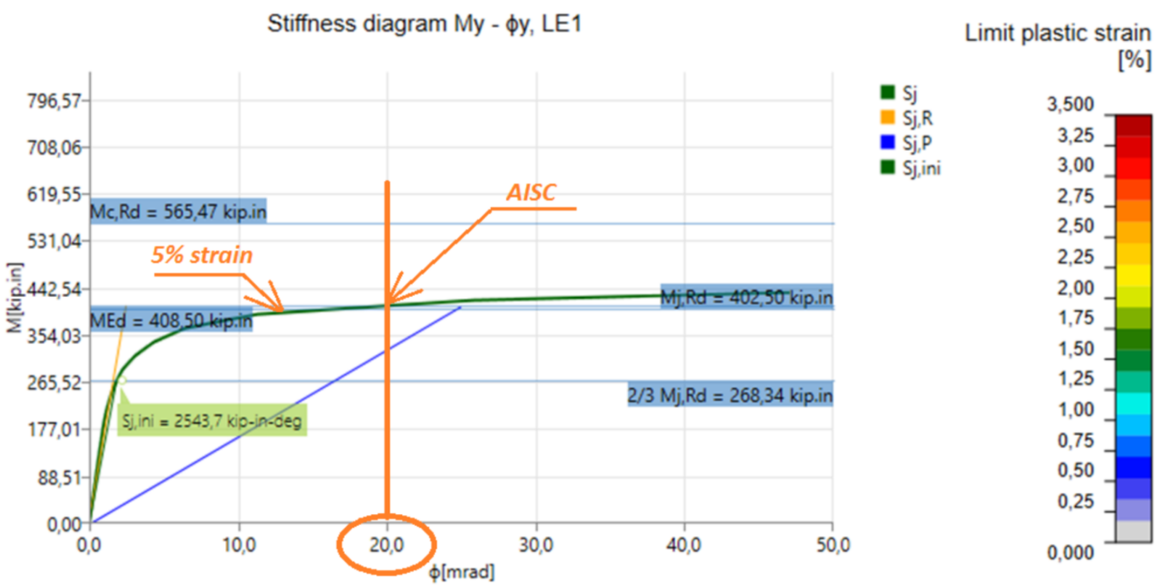

Figure 20 *Strain limit and rotation limit comparison.*[106]

[103] Idea StatiCa, (2017) p. 7
[104] AISC 360, (2010) p. 264
[105] www.ideastatica.com/resource
[106] www.ideastatica.com/resource

3.2.2 Plates and mesh convergence

Plates appear as quadrangle shell elements, with a node in each corner. Each of the nodes is considered to have six degrees of freedom, linear (u_x, u_y, u_z) and rotations $(\varphi_x, \varphi_y, \varphi_z)$. And it applies the following: membrane behavior, based on the work of Ibrahimbegovic (1990), Mindlin hypothesis, for the out of plane shear deformations, MITC4 (Mixed Interpolation of Components) by Dvorkin (1984), Gauss-Lobatto integration, divides the shell into five integration points along the height of the shell and plastic behavior is analyzed at each point.[107]

With too loose convergence tolerances the obtained results will be inaccurate, and with tolerance too strict computational workload is large for needless accuracy.[108] For generation of mesh in connection models there are a few criteria's. Mesh size for separate plates is insignificant, but for complex connections such as stiffened panels, T-stubs and base plates attention should be paid. Mesh elements should be of equal size between plates of a beam cross-section, with a minimal size of 10mm and maximal size of 50mm. Mesh of end plates are independent on other parts of a connection. Following is an example of a IPE 220 beam connected to a HEA 200 column, and the beam is subjected to bending moment.[109]

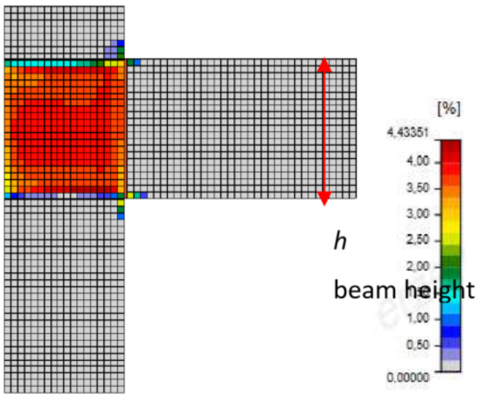

Figure 21 *Connection example for mesh size sensitivity.*[110]

A comparison of results with 4 finite elements along the cross-section height and results with 40 finite elements, the recommended number of elements for beam height is 8. The critical part of the connection is the slender compression plate in the column. The results are presented in *Figure 22 and 23*. For stiffener the critical load for buckling is also shown.

[107] Idea StatiCa, (2017) p. 8
[108] Bathe, (1982) p. 494
[109] Idea StatiCa, (2017) p.8-9
[110] Idea StatiCa, (2017) p. 10

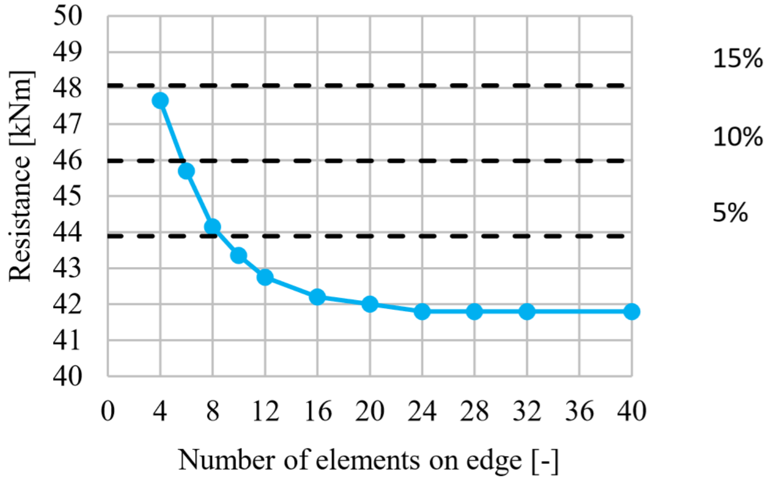

Figure 22 *graph of influence of number of elements on bending resistance.*[111]

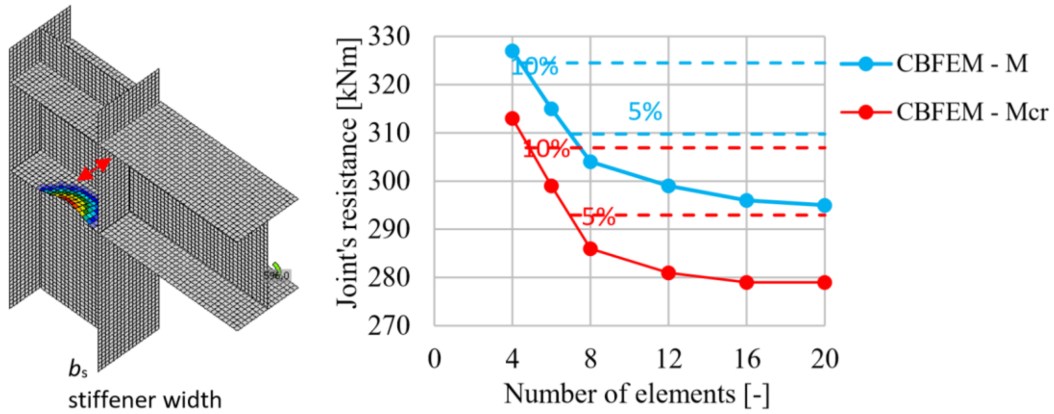

Figure 23 *Buckling and influence on moment resistance by number of elements along the stiffener.*[112]

3.2.3 Contacts between plates

One member can't be allowed to penetrate into the opposite member. The arising friction means that a relation between the normal force and shear force occurs for slipping.[113] Contacts between plates have a significant impact on the redistribution of the forces in the connection, and the standards penalty method is recommended. When a node of one plate penetrates into the opposite plate, penalty stiffness is added between the nodes (penetration distance). Points of penetration is automatically detected by the solver and solves the distribution of contact force between the node to the nodes of opposite plate. Contacts can be added between 2 surfaces, two edges and edge to surface.[114]

[111] Idea StatiCa, (2017) p. 10
[112] Idea StatiCa, (2017) p. 11
[113] Fredriksson, (1978) p. 307
[114] Idea StatiCa, (2017) p. 11-12

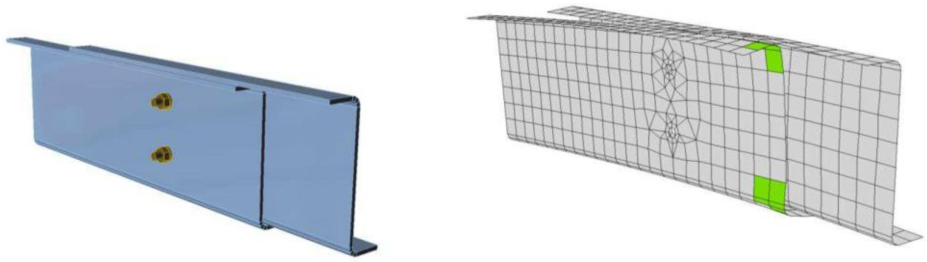

Figure 24 *Plate contact with different meshes.*[115]

3.2.4 Welds

For weld design the most often used material model is the rate-independent plasticity model, based on von Mises yield criterion.[116] Von Mises states that plastic deformation initiates when the principal stresses at a certain point satisfy the relationship, shown in *Formula 22*. The quantity of the right-hand side of the formula is the von Mises stress. Therefore, yielded areas can be identified by the comparison of the von Mises stress to σ_y .[117]

$$\sigma_y = \sqrt{0{,}5 * [(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\ \sigma_3 - \sigma_1)^2}$$

Formula 22 *von Mises yield criterion.*[118]

Two used approaches will be presented, first direct connection of plates. This option is merging the meshes of the connected parts, and the forces are transmitted to the opposing plate through force and deformation constraints based on lagrangian method. The connection is a multipoint constraint (MPC), which means the nodes are placed to relate to the nodes of the opposite plate, but without being connected directly. It respects the real weld configurations and calculates the stresses in the throat section. The program calculates the exact values in the welds, and the user can choose how to evaluate the values. Since stress peaks occur in plate edges, maximal stress gives a conservative stress distribution.[119]

[115] Idea StatiCa, (2017) p. 12
[116] Idea StatiCa, (2017) p. 11
[117] Spyrakos, (1994) p. 32
[118] Spyrakos, (1994) p. 32
[119] Idea StatiCa, (2017) p. 13-14

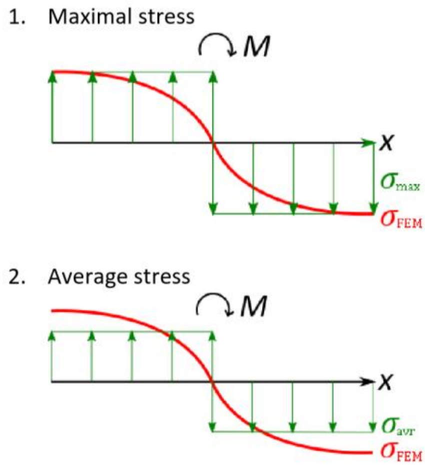

Figure 25 *Maximal stress and average stress for welds.*[120]

The second method for weld design is the plastic method. For this a special elastoplastic element which respects the throat size, orientation and position is added between the two joined parts. Welds are analyzed through nonlinear material analysis and elastoplastic behavior determined for the weld solid. As in the previous method the stress in the weld throat section controls, and the stress peaks are redistributed along the weld length. Ideal plastic model is used and the plastic strain is limited to 5% as for the plates.[121]

3.2.5 Bolts

In CBFEM the behavior of the bolt is determined by nonlinear spring models. Bolt in tension determined by axial initial stiffness, design resistance, initialization of yielding and deformation capacity. The axial initial stiffness is produced analytically from guideline 2230 by VDI[122]. The model also corresponds to the experimental data presented by Gödrich in "Advanced modelling of end plates". For deformation capacity it is assumed that plastic yielding occurs only in the threaded part of the bolt, and elastic deformation of bolt shank. The limit value for plastic strain is 5%, and the total deformation capacity is taken as the sum of the two. Bolt holes can be set as standard or slotted, standard bolts take shear in all directions while slotted exclude one direction in which the bolt can move freely.[123]

Bolt preloading has large influence on the rotational stiffness of bolted connections. The stiffening effect of the connection when preloaded bolts are used is related to two phenomena. Firstly, preloading increases the axial stiffness of the system made of bolt and

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[120] Idea StatiCa, (2017) p. 14
[121] Idea StatiCa, (2017) p. 14-15
[122] VDI, a standard ogranization in Germany, producing standards based on latest technical developments.
[123] Idea StatiCa, (2017) p. 15-16

the plates considered as a whole. And for connections like T-stubs the preloading modifies the connection behavior which depends on the flexural stiffness between the plates and the bolt axial stiffness.[124] Preloaded bolts can be used when a decreased deformation is strived. The tension model is similar to standard bolts, but shear is transferred solely via friction. The program checks for pre-slipping, if a slipping effect occurs the bolts doesn't satisfy the check and a standard bearing check is performed for shear on bolts and bolt holes. Options for design are resistance to major slip or post-slipping state, it is assumed that bolts have normal bolt behavior after major slip. The influence of bending moment on shear capacity is solved by checking each bolt separately.[125]

3.2.6 Anchor rods and concrete blocks

Anchor bolts are modeled with similar procedures as structural bolts. The bolts are fixed to the concrete block with a length for stiffness calculations determined in accordance to the component method (codes). Checks for stiffness of anchor in shear is taken as the stiffness of a structural bolt in shear.[126]

The connection between concrete block and base plate resists only compressive loads. Compression is transferred with Winkler-Pasternak subsoil model, which is commonly used for simplified calculations of foundations for deformations of concrete blocks.[127] Use of Winkler-Pasternak model means interaction of the concrete block and an elastic material supporting the block.[128] Shear force can be transferred through friction, shear lug and by bending of anchors and friction.[129]

3.3 Analysis

Fast analysis of complex geometries is enabled by the newly developed Component Based Finite Element Method (CBFEM). The designer doesn't create the FEM model, but the program automatically generates an analyzed FEM model. The model consists of members and manufacturing operations, cuts, welds, plates and stiffeners for example. These operations are chosen by the designer to construct the joint.[130]

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[124] Faella, Piluso and Rizzano, (2000) p. 164
[125] Idea StatiCa, (2017) p. 17
[126] Idea StatiCa, (2017) p. 18
[127] Idea StatiCa, (2017) p. 18-19
[128] Bittnar and Sejnoha, (1996) p. 86
[129] Idea StatiCa, (2017) p. 19
[130] Idea StatiCa, (2017) p. 20

One member is always bearing, chosen by the designer, and the other connected. The bearing member can be continuous or ended, ended is supported in one end while continuous is supported in both. Different connection might be able to transfer different number of internal force components. All joints are in the state of equilibrium during the analysis of frame structure. The default length of members is set to two times the height, the height of the member should be at least equal to the height after manufacturing operations. Each node of a 3D model is required to be in equilibrium unless it's the case of a simple connection. Therefore, there are two modes as options for the designer:[131]

- Simplified - for this mode the bearing member is continuous, supported at both ends. Which means no forces are defined on the member only for the connected members.
- Advanced - the bearing members is ended and supported on just one end. Hence, loads can be applied to all members and equilibrium must be found in the connection.

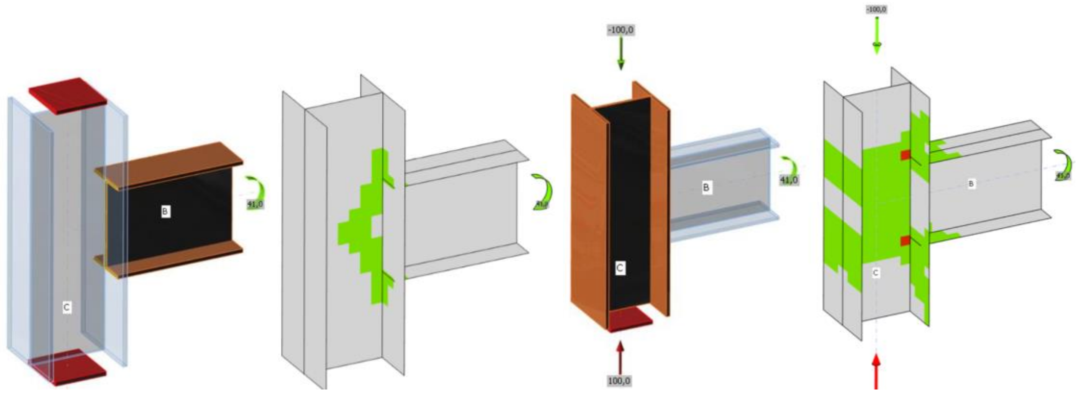

Figure 26 *Example of simplified and advanced connection design in Idea StatiCa, advanced with higher utilization to the right and simplified to the left.*[132]

In the example is a beam to column connection the beam subjected to a bending moment of 41kNm and the column is compressed by 100kN. The simplified method does not take the axial force into account since the column is support at both ends. Therefore, the simplified method shouldn't be used for heavily loaded connections or if behavior of whole joint is wanted.[133]

[131] Idea StatiCa, (2017) p. 20-23
[132] Idea StatiCa, (2017) p. 24
[133] Idea StatiCa, (2017) p. 23-24

3.3.1 Loads

The analysis model created by CBFEM method corresponds very accurately to the real model. Whereas analysis of internal forces is performed on a 3D FEM bar model with member centerlines, and joints using immaterial nodes. Consequently, internal forces carried out from a bar model won't correspond with the CBFEM model since the bar model does not respect the eccentricities. Therefore, the user is provided three options for point of force, node, bolt or position chosen manually.[134]

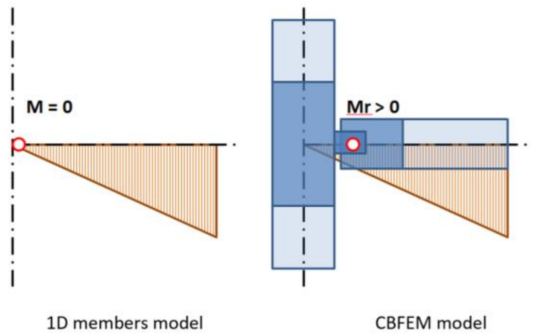

Figure 27 *Issues with nodes and forces using bar models and CBFEM models.*[135]

3.3.2 Strength analysis

For strength analysis two approaches are offered, first one is by response of structure to overall load. All loads defined are applied and the stresses and deformation is calculated. The second option is to set the program to stop when the limit of plastic strain is reached, and applied % of loads is displayed. Second option is recommended for practical design and first option is preferable for more detailed analysis of complex connections.[136]

3.3.3 Stiffness analysis

Stiffness analysis for individual members of a joint is possible in CBFEM. For a proper analysis the analysis must be performed separately for each member, which is possible by setting all members expect the one to be analyzed as bearing members. In this way, only the loads of the analyzed member are used of all the applied loads and other members are set as supports. The program performs axial and rotational stiffness checks (depending on applied loads) and automatically generates complete diagrams.[137]

[134] Idea StatiCa, (2017) p. 24-27
[135] Idea StatiCa, (2017) p. 26
[136] Idea StatiCa, (2017) p. 28-29
[137] Idea StatiCa, (2017) p. 29-30

4 COMPARISONS AND RESULTS

In this chapter obtained results from various test performed in both Idea StatiCa and the excel tools are presented, compared and analyzed. The connections were examined with a range different geometries, settings and loadings, in aim of finding differences and possibly preferable settings for certain connections. Following, a few chosen examples of three connection types with findings that may be useable. The used limit value for plastic strain is 5%. For each tested situation it is presented which one is conservative, that is the one requiring bigger profiles or welds. The credibility of the Excel tools is strengthened by comparisons to available example in *AISC Specifications* and the design guides and by checks by inspectors from Citec, these comparisons are not presented in the thesis work.

4.1 Tests for K-connections

4.1.1 Example K-connection

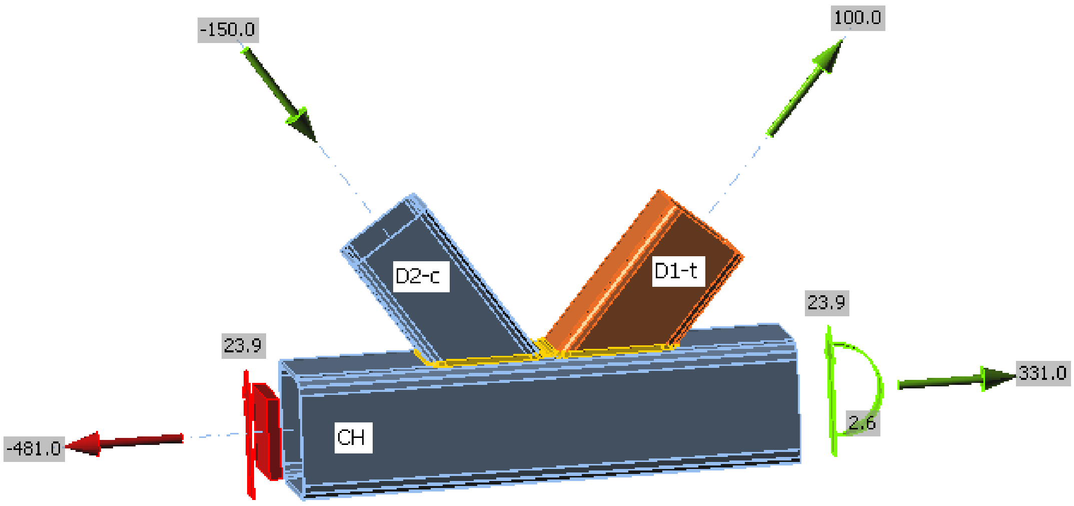

Figure 28 *Example, one of the studied K-connections in the examination.*

A chord of 100x100x5 and branches of 70x70x3 are the dimensions of the members in the example, the forces are displayed in the figure. The following results were obtained:

- In Idea StatiCa both chord and compression member fails the plastic strain limit of 5%. While excel results were following, the chord member passes the check and branches doesn’t exceed the geometry limit for required branch check.

- Welds for compression branch passes on both, but welds of tension branch fail the check in Idea StatiCa while excel gives a very low utilization ratio.

4.1.2 Other conclusions and findings for K-connections

- Member checks for the examined connection is extensively more conservative in the CBFEM model than the excel results.

- An explanation for the lower utilization of welds in excel could possibly be the use of formulas for weld strength of a weld group, a combination of longitudinal and transverse welds (described in Chapter 2.2.3, *Formula 4*). This could give a higher strength than the method of CBFEM, comparing strength per millimeter of weld to the critical weld stress.

- With a branch width equal to the chord width, plastic redistribution method for welds gives significantly smaller utilization rates than average value. Chosen method for cutting does also have a major impact on the check, “surface” should be chosen as the cutting method, since branches will be connected at the rounded corners of the chord member. In these cases, excel seems more conservative for welds.

- If possible a decrease in weld size could decrease the utilization rate of the members.

4.2 Tests for T- & Y-connections

4.2.1 Example T-connection

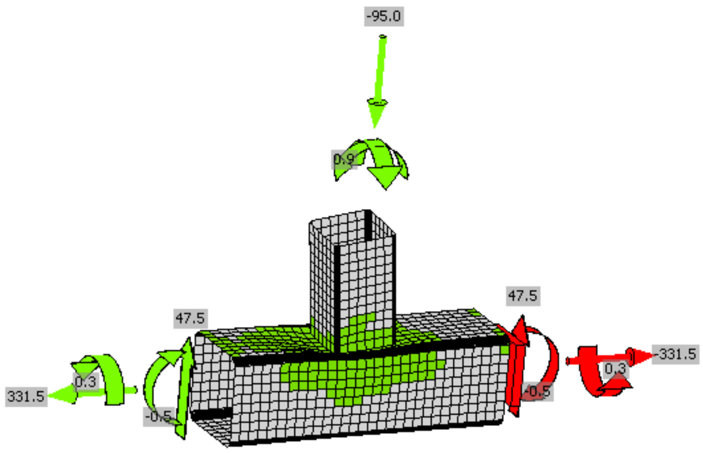

Figure 29 *Example, T-connection subjected to biaxial bending moment and axial force.*

The example connection is a 100x100x5 chord welded to a 70x70x3 branch, the loads are displayed in the figure, a snip from Idea StatiCa. The following results were obtained in the comparison:

- Member check – the connection barely passed in Idea StatiCa, whereas the connection failed in the excel tool with a utilization ratio of 124%.

- Weld check – in the excel tool the welds had a utilization ratio of 90%, in Idea StatiCa the welds both failed and passed, depending on which settings and evaluation method was chosen. The different methods displayed the following, average stress=61%, maximal stress=92%, and plastic redistribution=110%.

4.2.2 Conclusions and findings for T- & Y-connections

- Excel tool being conservative for the member check is mostly because of the bending moments. Tests performed with the bending moments only showed a significant difference between the excel and Idea StatiCa, excel being extensively more conservative. The conservativeness decreased as the plate thickness of the chord member was increased. The reason for the conservativeness in cases of bending moments could be the slim availability of tests performed on T-connections of HSS with such loadings as mentioned in the Chapter of HSS-to-HSS moment connections.

- For small branches relative to the chord the excel is very conservative compared to Idea StatiCa. Weld checks of Idea StatiCa gives very different results, average stress method usually the only one not failing.

- Number of elements in corners of HSS plays a big role for member checks, as the failure mode of the chord is commonly at the face where branch is connected. And for branches the highest stress is found in the corners. Hence, too few number of elements won't provide an accurate result and utilization rate increases as the number of elements (until a sufficient number is found).

- As for member checks the welds are also affected by the number of elements but the other way around, the stress might decrease when number of elements is increased. 7-8 was found optimal in the performed checks, a larger number of elements doesn't make a significant difference in results but increases the workload. Especially for maximal stress method the divergences in weld checks were large in some cases.

- Average stress method was the most consistent in the variety of tests performed. It was also least sensitive to the number of elements in HSS corners.

- Idea StatiCa was found conservative compared to the excel for large members in T-connections with large axial forces. But as the connection was translated to a Y-connection with a decreased angle on the branch the results came closer together. At 40 degrees results obtained in tests were very close to equal.

4.3 Tests on column base plates

4.3.1 Example case for column base plates

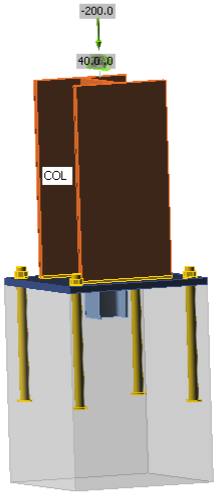

Figure 30 *Base plate connection, used as an example which includes all force types. 200kN of compression, 78kNm bending moment in strong axis, 40kNm bending moment in weak axis and 50kN of shear force in strong axis of column.*

The obtained results from the example connection:

- Anchors – excel generates less tension, but as only the larger of the bending moments is considered large this is the only bending moment generating tension in anchors. Critical anchor in Idea SatiCa was subjected to 139kN of tension, whereas excel produces only 44kN.

- Plate thickness – Idea StatiCa passes the check with a 13mm thick plate, at a plastic strain of 2,1%. Excel required 32mm (24mm by larger moment and 21mm by smaller moment), which is a significant difference.

- Welds – plastic redistribution was used for the check and provided a utilization ratio of 96%, which is equal to the 96% that excel provided. With a change to average value the utilization ratio in Idea StatiCa is only 41%.

- Shear lug – the HEA profile used as shear lug had a utilization ratio of 45% in excel and only 14% in Idea StatiCa, as friction is also taken into account.

4.3.2 Conclusions and findings for base plate connections

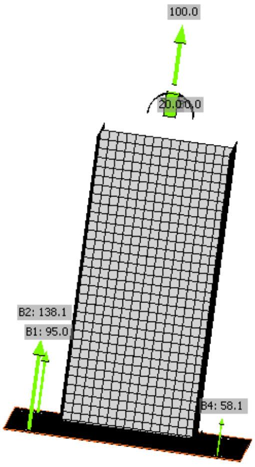

Figure 31 *Tension forces in each anchor with 100kN uplift, 30kNm bending in strong axis and 20kNm in weak.*

Tension and tension combined with bending:

- Concrete breakout strengths are considerably larger in the excel tool. Possible casual factors are, effective embedment length was found smaller in Idea StatiCa and effective area of the breakout surface. In Idea StatiCa breakout for the anchor group had a smaller area than for a single anchor. Also with a trial of choosing a very small pier knowing that the edge distances will limit the area thus the breakout area should equal the pier area, excel and Idea StatiCa didn't provide same breakout area.

- Plate thicknesses in pinned connections with axial loads in excel are conservative, often excessively so and for both tension and compression.

- Bolts forces with bending – in most cases bolt forces were close, with excel producing slightly lower values and the difference might increase in cases with large bending moments relative to the tension force. This was found for both uniaxial and biaxial bending.

- Plate thickness with biaxial bending – the plate thickness obtained for the critical cantilever was found close to Idea StatiCa, thus the thickness obtained by combining the thicknesses becomes very conservative. The reason why choosing the higher was valid in the tests is that anchor tension forces are calculated for each anchor and the sum of the two critical is used as the force causing the bending in corresponding direction. In *Figure 30* it would be 138kN+95kN.

- Welds – with both tension and bending Idea StatiCa requires large welds with plastic redistribution (recommended by Idea StatiCa) whereas use of average value is closer to excel which requires a lot smaller.

Compression and bending:

- Anchor tension - in most cases excel developed a slightly smaller tension force in anchors. A reason could be the approach of max compressive stress for the bearing length in *design guide 1*, which leads to less tension to create equilibrium. The fact that the concrete utilization was relatively low in Idea StatiCa gives same indications.

- Anchor tension – evaluation method for welds can also affect the tension force in welds, in one of the examined connection the following tension force were obtained: 345kN with plastic redistribution, and 273kN with average value.

- Member check – in some cases the evaluation method of welds also affects the members. Flange was found failing with plastic redistribution whereas average value for welds passed the member check.

- Base plate - excel results for base plate thickness are conservative compared to Idea Statica. In tested connections the larger of the thickness results obtained in excel was enough, and still slightly conservative.

- Welds – excel excessively conservative in many cases in comparison with Idea StatiCa.

Compression and biaxial bending with HSS column:

- Anchors – obtained anchor tension forces were comparable to each other. But as moments get larger relative to compression the tension is higher in excel.

- Plate thickness – excel requires a little thicker but that has been the case with studied cases of uniaxial bending also and the thickness for biaxial seems to be in relation to uniaxial bending. A case with a 180x180 profile Idea StatiCa required a 13mm base plate, whereas excel required 20mm.

- Welds – use of plastic redistribution gave conservative results compared to excel, whereas results from average value usually are lower than excel results.

Shear:

- Anchors – shear breakout strength of anchors have same problems as anchors in tension.

- Plate – with a plate as shear lug the strengths obtained were comparable. But in some cases, the plate could fail due to bending in excel, which is not checked in Idea StatiCa. Neither does Idea StatiCa check for concrete breakout, but neither was concrete breakout critical in excel for any of studied cases.

4.4 General conclusions

A lot of the settings play a big role for the strength of the connections but it also has a considerable impact on the computational workload. This was experienced during the examination when running a lot of different geometries and load cases. In some cases, for all the different connections the evaluation method of welds was found to have a significant impact on the plates and members. Situations where plastic redistribution could be failing whereas the two other methods gave low utilization ratios, this could be depending on the multipoint constraint method used in the direct connection of plates (used in average value and maximal value) which provides a better evaluation of connections with members of different mesh densities.

Modelling of the connections is also possible in Idea StatiCa but some for some connections the modelling can be quite clumsy, even for simple connections. A perfect example is K-connections, where branches doesn’t have an input for the gap size. The options for input are in the Y-, X- and Z-axis of the branch being moved, and for K-connections where the branches doesn’t have a 90 degrees angle this means there is not an input for moving the branch in the direction of the chord.

5 CONCLUSION

The purpose of this thesis was to find out how a design software, Idea StatiCa, works and the theoretical background of it. In addition, development of excel tools based on AISC specifications for results comparable to results obtained from Idea StatiCa.

For the development of the excel tools I was forced to familiarize with AISC specifications, since the only codes I've used in the past are Eurocodes. After some use I didn't find it complicated, but the units could be challenging during times. Metric units is most of the times provided but not always, and since some formulas are developed specifically for imperial units the translation to metric units was at times clumsy.

The other challenge with the development was the lack of experience in excel use. Since the tools were primarily made for comparison they were kept as simple as possible, so I managed with the help of Microsoft's own forums. Yet the biggest challenge was making the excels to manage as many different situations as possible, for as many comparative cases as possible. This means combining several cases into one calculation and still getting the excel to perform a correct calculation for all the different situations.

After all the thesis work has been challenging but very instructive since none of the used programs or standards were familiar. My Excel skills got improved a lot, I got familiar with new standards and I learned to use Idea StatiCa by trial and error. In addition to that I also improved my knowledge of steel connections.

6 BIBLIOGRAPHY

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EXAMPLE CONNECTION				Page 1
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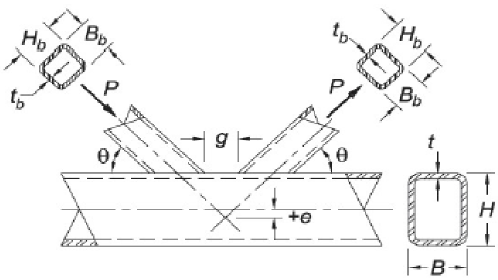
Title HOLLOW SECTION K-CONNECTION				

According to AISC 360-10
and AISC Design Guide 24



Controlling load combination: **LC 24**
for: **K-truss connection**
Node in calculation model: **24**
(force direction in accordance to picture)
LOAD INPUT:

Chord:	(negative = tension)	
$F_{x,l}$=	-331,0 kN	axial force, left
$F_{x,r}$=	-331,0 kN	axial force, right
Axial forces in branch:		
positive input only, since force direction is in picture		
$P=F_{xc}$=	150,0 kN	compression branch
$P=F_{xt}$=	100,0 kN	tension branch

Pr*sinθ / Pr*sinθ (max 20%): 43,1 % ⟹ calculated as K- and cross-connection

Compr. branch: 180x180x7,1		Tension branch: 180x180x7,1		Chord: 220x220x10	
H_{bc}=	180,0 mm	H_{bt}=	180,0 mm	H=	220,0 mm
B_{bc}=	180,0 mm	B_{bt}=	180,0 mm	B=	220,0 mm
t_{bc}=	7,1 mm	t_{bt}=	7,1 mm	t=	10,0 mm
$A_{branch.c}$=	4910 mm²	$A_{branch.t}$	4910 mm²	A_{chord}=	8400 mm²

Strengths: **S355**

yield	F_y=	355 MPa	throat size	a=	6,0 mm	minimum
tensile	F_u=	470 MPa	leg length	w=	8,5 mm	5
	E=	200000 MPa	thinner material	t_{min}=	7,1 mm	

Welds:

Parameters:

g=	21,0 mm	gap between diagonals	21>=17	Verified
e=	43,4 mm	joint eccentricity (limit: e/H)	-0,55>=0,2<=0,25	Verified
θ_c=	56,0 deg	angle compression branch	56>=30	Verified
θ_t=	45,0 deg	angle tension branch	45>=30	Verified

Limits (AISC design guide 24, table 8-2A):

chord wall slenderness:	B/t and H/t ≤ 35	22 and 22>=35	Verified
material strength:	$F_y \le 360$ MPa	355>=360	Verified
branch size(if both are square):	smaller B_b > 0.63*(larger B_b)	180>113,4	Verified
branch wall slenderness			
compression branch:	B_b/t_b and $H_b/t_b \le 1.25(E/F_y)$^0.5 and 35	25,4 and 25,4<=29,7 and 35	Verified
tension branch:	B_b/t_b and $H_b/t_b \le 35$	25,4 and 25,4<=35	Verified
aspect ratio branch, compr.:	$0.5 \le H_b/B_b \le 2.0$	0,5<=1<=2	Verified
aspect ratio branch, tension:	$0.5 \le H_b/B_b \le 2.0$	0,5<=1<=2	Verified
aspect ratio chord:	$0.5 \le H/B \le 2.0$	0,5<=1<=2	Verified
ductility:	$F_y/F_u \le 0.8$	0,76<=0,8	Verified

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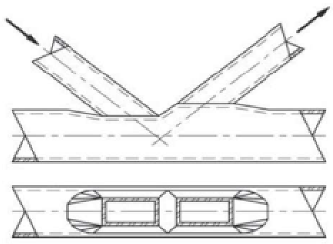
1. Chord wall plastification (AISC design guide 24, table 8-2)



$P_r = F_x =$ Tension in chord, which means Qf is 1

$F_c = F_y =$ 355 MPa yield strength

$M_r = M_y =$ Neglected moment in chord

S= 562545 mm³ elastic section modulus

A= 8400 mm² area of cross section

$F_{xc} =$ 100 Balanced forces, since difference exceeds limit

$F_{xt} =$ 100

$$P_n \sin\theta = F_y t^2 \left(9.8\beta_{eff}\gamma^{0.5}\right)Q_f$$

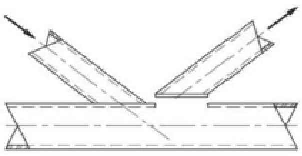
$P_n =$	for compression branch	1138,7 kN	
$\phi P_n =$		**1024,9 kN**	
$P_n =$	for tension branch	1335,1 kN	
$\phi P_n =$		1201,6 kN	
$\beta_{eff} =$	[(Bb+Hb)compression branch + (Bb+Hb)tension branch]/4*B	0,8	effective width ratio
$Q_f =$	1,3-0,4*(U/β_{eff}) ≤ 1,0	1,0	
U=	(P_r / A*F_c)+(M_r / S*F_c)		
γ=	B / 2t=		chord slenderness ratio
ϕ=	factor for LRFD	0,9	

Tension in chord, means Qf is limited to 1, so value for QF not needed

Utilization: 9,8 % compression branch; 8,3 % tension branch

2. Shear yielding (punching of chord) (design guide 24, table 8-2)

when Bb<B-2t and if branches aren't square: Required



$$P_n \sin\theta = 0.6F_y tB\left(2\eta + \beta + \beta_{eop}\right) \quad \text{(K2-21)}$$

P_n*sinθ=	shear yielding strength	2867,8 kN	
$\phi P_n =$		**2724,4 kN**	
β=	B_b/B	0,82	width ratio, tension branch
$\beta_{eop} =$	5β/γ ≤ β	0,37	eff. outside punch parameter
η=	l_b/B	1,6	load length parameter
$l_b =$	H_b/sinθ	345,1	length of bearing
t=	steel thickness	10,0 mm	
B=	chord width	220	
ϕ=	factor for LRFD	0,95	

Utilization: 3,7 %

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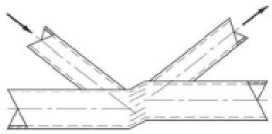
3. **Shear in chord sidewalls in gap region (Spec. sect. G5)**

NO check for square chords: NOT CALCULATED



$$V_n = 0.6F_y A_w C_v$$

A_w= 2*H*t

t= 0.93*nominal thickness design wall thickness

h= height resisting shear force

C_v=

k_v=

E=

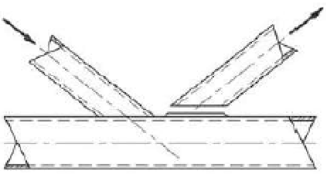
ϕ= factor for LRFD

V_n=

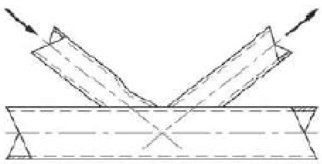
ϕV_n=

V= shear force

Utilization:



(c) Local yielding of tension branch, due to uneven load distribution.



(d) Local yielding of compression branch, due to uneven load distribution.

4. **Local shear yielding of branches, due to uneven load distribution (design guide 24, table 8-2)**

No check for square branches or if B/t≥15: NOT CALCULATED

$$P_n = F_{yb} t_b \left(2H_b + B_b + b_{eoi} - 4t_b\right)$$ **(K2-22)**

t_b= steel thickness

H_b= branch height

B_b= branch width

B_{eoi}= $(10/ B/t)*(F_y*t / F_{yb}*t_b)*B_b \le B_b$

ϕ= factor for LRFD

P_n=

ϕP_n=

Utilization:

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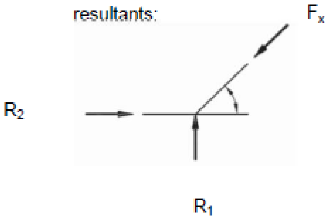
b **Cross-connection (design guide 24, table 8-2)**

1.b **Chord wall plastification (K2-13)**

resultants:



calculated if branch forces differ more than 20%
Required as difference between forces= 43,1 %

$$P_n \sin\theta = F_y t^2 \left(\frac{2\eta}{(1-\beta)} + \frac{4}{\sqrt{1-\beta}} \right) Q_f$$

Calculated part: **compression branch** (Fx and R in accordance to picture)

dimensions for compression branch:

H_b=	180,0 mm
B_b=	180,0 mm
t_b=	7,1 mm

anlge: 56,0 deg

F_x=	remaining force after balancing	50,0 kN	
θ=	angle of branch	56,0 deg	
R_1=	F_x*sin(θ)	41,5 kN	vertical resultant
R_2=	F_x*cos(θ)	28,0 kN	horizontal resultant
β=	B_b / B	0,818181818	
η=	H_b / B*sinθ	0,99	
Q_f=	1,3-0,4*(U/β) ≤ 1,0	1,0	
U=	(P_r / A*F_c)+(M_r / S*F_c)		
P_r= P_u=	R_2	28,0 kN	

Tension in chord, means Qf is limited to 1, so values for QF not needed

P_n=	for remaining force, as Y-connection	866,6 kN	
ϕP_n=	ϕ=1.0 for LRFD	**866,6 kN**	
Utilization:	cross-connection	0,058	
	K-connection	0,098	
	TOTAL	15,5 %	

2.b **shear yielding (K2-14)**

calculated if 0,85<Beta<=1-1/γ or B/t<10
Required as

Beta= 0,8	0,85
	0,91
B/t= 22	10

γ=	B/2*t	11	
β=	B_b/B	0,818181818	
B_{eop}=	5*β/γ <= β	0,37	
P_n=	0,6*Fy*t*B/(2*η+2*Beop)	1533,9 kN	
ϕP_n=	**φ=0,95 for LRFD**	**1457,2 kN**	
Utilization:	cross-connection	0,034	
	K-connection	0,098	
	TOTAL	13,2 %	

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4.b local shear yielding of branches (K2-18)

calculated if Beta>=0,85 NOT CALCULATED as β= 0,82

P_n= F_y*t_b*(2*H_b+2*B_{eoi}-4*t_b)

ϕP_n= φ=0,95 for LRFD

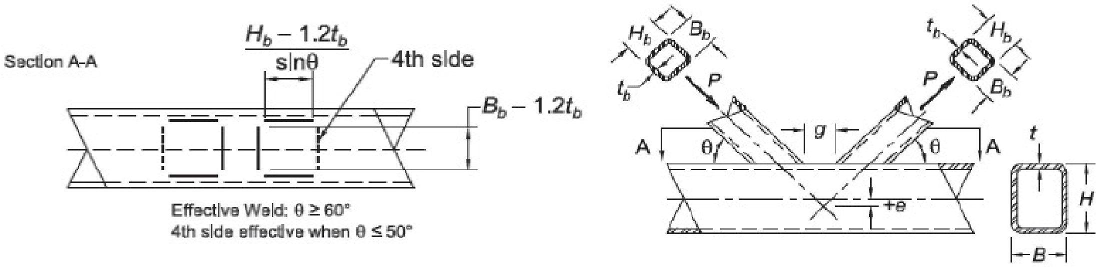
Utilization: cross-connection

K-connection

TOTAL

5. Weld check

(force directions are according to picture, similar as first picture in sheet)



5.1a effective lengths, COMPRESSION branch (all effective weld lengths from, AISC spec. table K4.1)

$l_{e.c}$= 668,6 mm

θ= 56,0 deg when θ ≤ 50 deg NOT calculated

$l_{e.c}$= $l_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + 2(B_b - 1.2t_b)$ (K4-8)

790,7 mm (50deg for interpolation)

when θ ≥ 60 deg NOT calculated

$l_{e.c}$= $l_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + (B_b - 1.2t_b)$ (K4-9)

567,5 mm (60deg for interpolation)

when 50 < θ < 60 deg Used

$l_{e.c}$= interpolated value 668,59 mm

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5.1b effective lengths, TENSION branch 45,0 deg

$l_{e.t}$= **828,0 mm**

when θ ≤ 50 deg — Used

$l_{e.t}$= $l_e = \dfrac{2(H_b - 1.2t_b)}{\sin\theta} + 2(B_b - 1.2t_b)$ (K4-8) 828,0 mm

when θ ≥ 60 deg — NOT calculated

$l_{e.t}$= $l_e = \dfrac{2(H_b - 1.2t_b)}{\sin\theta} + (B_b - 1.2t_b)$ (K4-9)

when 50 < θ < 60 deg — NOT calculated

$l_{e.t}$= interpolated value

5.2 weld strengths

5.2a welds for compression branch

obtuse side angle: 124,0 deg acute side angle: 56,0 deg

$w_{eq.a}$=	weld size for acute side, =leg size*factor	factor: 1,41	12,0 mm
$w_{eq.o}$=	for obtuse side, =leg size*factor	factor: 0,816	6,9 mm
w_{eq}=	for 90 deg side, =leg size*factor	factor: 1	8,5 mm
t_w=	throat for longitudinal weld		6,0 mm

Rn is determined by the greater of formula (i) and (ii):
AISC spec. J2.4

(i) $R_n = R_{nwl} + R_{nwt}$ (J2-10a) Filler metal: E70XX

or

(ii) $R_n = 0.85\ R_{nwl} + 1.5\ R_{nwt}$ (J2-10b)

F_{exx}=	filler metal classification strength
$l_{e.l}$=	effective length longitudinal weld
$l_{e.t}$=	effective length transversal weld
$A_{we.l}$=	effective AREA longitudinal weld
$A_{we.t}$=	effective AREA transversal weld
R_{nwl}=	longitudinal, 0,6*F_{exx}*$A_{we.l}$
R_{nwt}=	transversal, 0,6*F_{exx}*$A_{we.t}$

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$R_n(i)$=	alternative 1	
$R_n(ii)$=	alternative 2	
R_n=	**greater of (i) and (ii)**	
ϕR_n=	**ϕ = 0,75**	
	Utilization:	15,7 %

5.2b welds for tension branch

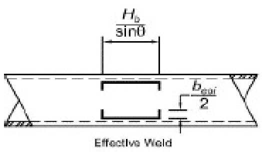
obtuse side angle: 135,0 deg acute side angle: 45,0 deg

$w_{eq.a}$=	weld size for acute side, =leg size*factor	factor: 1,41	12,0 mm
$w_{eq.o}$=	for obtuse side, =leg size*factor	factor: 0,78	6,6 mm
w_{eq}=	for 90 deg side, =leg size*factor	factor: 1	8,5 mm
t_w=	throat size, w*cos(θ/2)		6,0 mm

$l_{e.l}$=	effective length longitudinal weld	485,0 mm
$l_{e.t}$=	effective length transversal weld	343,0 mm
$A_{we.l}$=	effective AREA longitudinal weld	2910,0 mm²
$A_{we.t}$=	effective AREA transversal weld	2058,0 mm²
R_{nwl}=	longitudinal, $0{,}6{*}F_{exx}{*}A_{we.l}$	1113,7 kN
R_{nwt}=	transversal, $0{,}6{*}F_{exx}{*}A_{we.t}$	787,6 kN

$R_n(i)$=	alternative 1	1901,3 kN
$R_n(ii)$=	alternative 2	2128,0 kN
R_n=	**greater of (i) and (ii)**	**2128,0 kN**
ϕR_n=	**ϕ = 0,75**	**1596,0 kN**
	Utilization:	6,3 %

5.3 adjusted effective weld lengths (when calculated as Y-connection)
(calculated if branch forces differ >20%) Required



$$l_e = \frac{2H_b}{\sin\theta} + 2b_{eoi}$$ (K4-5) calculated branch: **compression**

$H_{b.y}$=	branch height for Y-connection		180 mm
$B_{b.y}$=	branch width for Y-connection		180 mm
$t_{b.y}$=	branch thickness for Y-connection		7 mm
θ=	angle of calculated branch	θ>50	56,0 deg
β=	Bb_y/B		0,81818182
$b_{eoi}/2$=	when β>0,85 or θ>50 deg, b_eoi/2 shall not exceed 2*t		20 mm
$l_{e.y}$=	effective weld for partly Y-connection		519 mm
l_e=	compression branch, smaller of le_c and le_y		519 mm

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$w_{eq.a}$=	weld size for acute side, for branch considered Y	12,0 mm
$w_{eq.o}$=	for obtuse side	6,9 mm
w_{eq}=	for 90 deg side	8,5 mm
t_w=	throat for longitudinal weld	6,0 mm
$l_{e.l}$=	effective length longitudinal weld	413,7 mm
$l_{e.t}$=	effective length transversal weld	105,8 mm
$A_{we.l}$=	effective AREA longitudinal weld	2482,2 mm²
$A_{we.t}$=	effective AREA transversal weld	706,4 mm²
R_{nwl}=	longitudinal, $0,6*F_{exx}*A_{we.l}$	1015,3 kN
R_{nwt}=	transversal, $0,6*F_{exx}*A_{we.t}$	288,9 kN
R_n(i)=	alternative 1	1304,2 kN
R_n(ii)=	alternative 2	1296,4 kN
R_n=	**greater of (i) and (ii)**	**1304,2 kN**
ϕR_n=	**ϕ = 0,75**	**978,1 kN**
	Utilization:	15,3 %

This gives a conservative result, since the effective weld length is shorter for Y-connections.

5.4 **shear yielding and rupture of base metal**

nominal shear rupture strength for base metal — tension branch

ϕ=		0,75
Rn=	$F_{bm}*A_{bm}$	2335,0 kN
ϕRn=		**1751,2 kN**
Controlling:	0,76 > 0,75 therefore shear rupture controls	
F_{bm}=	Fu*0.6	282 MPa
A_{bm}=	effective weld length*base metal thickness	8280,0 mm²
l_e=	effective length for K-connection used	828,0 mm
	Utilization:	8,6 %

nominal shear rupture strength for base metal — compression branch

ϕ=		0,75
R_n=	$F_{bm}*A_{bm}$	1464,9 kN
ϕR_n=		**1098,7 kN**
Controlling:	0,76 > 0,75 therefore shear rupture controls	
F_{bm}=	Fu*0.6	282 MPa
A_{bm}=	effective weld length*base metal thickness	5194,8 mm²
l_e=	effective length for Y-connection used	519,5 mm
	Utilization:	13,7 %

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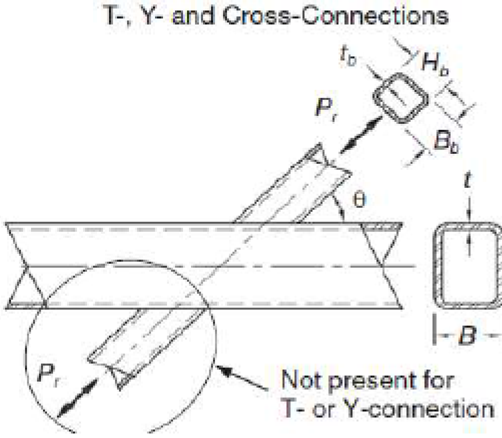
Title HOLLOW SECTION T-CONNECTION & BRACING				



T-connection
According to AISC 360-10
and AISC Design Guide 24

Controlling load combination **LC 24**
for base plate
Node 9 in calculation model
(negative force = tension)
LOAD INPUT:

Chord:			
	F_x=	-100,0 kN	axial force left of post
	F_x=	-100,0 kN	axial force right of post
Post:			
	F_x=	50,0 kN	axial force, compression
	M_{ip}=	6,5 kNm	in plane, bending
	M_{op}=	4,5 kNm	out of plane, bending
Bracing:			
	F_x=	0,0 kN	axial force, bracing 1
	F_x=	0,0 kN	axial force, bracing 2

Bracing: 0x0x0

H_b=	0,0 mm
B_b=	0,0 mm
t_b=	0,0 mm
$A_{bracing}$=	0 mm²

Post: 80x80x6

H_p=	80,0 mm
B_p=	80,0 mm
t_p=	6,0 mm
θ=	90,0 deg
A_{post}=	1776 mm²

Chord: 100x100x12

H=	100,0 mm
B=	100,0 mm
t=	12,0 mm
A_{chord}=	4224 mm²

Strengths: **S355**

yield	F_y=	355 MPa
tensile	F_u=	470 MPa
	E=	200000 MPa

Welds:

			minimum
leg length	w=	9,9 mm	5
throat	a=	7,0 mm	
thinner member	t_{min}=	6,0 mm	
filler metal:		E70XX	
	F_{exx}=	483 MPa	

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Limits:

chord wall slenderness:	B/t and H/t ≤ 35	8,33 and 8,33>=35	Verified
material strength:	F_y ≤ 360 MPa	355>=360	Verified
compression branch:	B_p/t_p and H_p/t_p ≤1.25(E/F_y)^0.5 and 35	13,3 and 13,3<=29,7 and 35	Verified
width ratio:	B_p/B and H_p/B >= 0.25	0,8 and 0,8>=0,25	Verified
aspect ratio chord:	0.5<= H/B <=2.0	0,5<=1<=2	Verified
aspect ratio post:	0.5<= H_p/B_p <=2.0	0,8 and 0,8>=0,25	Verified
ductility:	B_p/B and H_p/B >= 0.25	0,8 and 0,8>=0,25	Verified

Formulas for axial forces from AISC Design guide 24 table 8-2, moments from table 9-2

1.0 Chord wall plastification

calculated if β<=0.85 β=B_p/B= 0,8 required

$$P_n \sin\theta = F_y t^2 \left(\frac{2\eta}{(1-\beta)} + \frac{4}{\sqrt{1-\beta}}\right) Q_f \qquad \text{(K2-13)}$$

ϕ=	LRFD factor	1,0	
S=	elastic section modulus, chord	93504 mm³	
M_r=	moment in chord	0	Nmm
F_c= F_y=	yield strength	355 MPa	
P_r=	stress in chord, side of branch with higher value	-100000 N	(tension)
η=	load length parameter =l_b/B	0,8	
l_b=	H_p/sinθ	80,0 mm	
Q_f=	1.3-0.4*(U/β)<=1.0 and 1 if chord in tension	1,00	
U=	(P_r/A*F_c)+(M_r/S*F_c)	0,07	

			with angle	Utilization
P_n=		866 kN	866,2 kN	
ϕP_n=	limit, LRFD	**866 kN**	**866,2 kN**	
F_x=	axial force in post	50,0 kN		5,8 %

$$M_n = F_y t^2 H_b \left(\frac{1}{2\eta} + \frac{2}{\sqrt{1-\beta}} + \frac{\eta}{(1-\beta)}\right) Q_f \qquad \text{(K3-11)}$$

post bending in plane

$M_{n.ip}$=	moment capacity in plane	36,3 kNm	
ϕ$M_{n.ip}$=	ϕ=1.0	**36,3 kNm**	17,9 %

$$M_n = F_y t^2 \left[\frac{0.5H_b(1+\beta)}{(1-\beta)} + \sqrt{\frac{2BB_b(1+\beta)}{(1-\beta)}}\right] Q_f \qquad \text{(K3-15)}$$

post bending out of plane

$M_{n.op}$=	moment capacity out of plane	37,8 kNm	
ϕ$M_{n.op}$=	ϕ=1.0	**37,8 kNm**	11,9 %

Utilization for combined loads: (Pu/ϕPn)+(Mu.ip/ϕMn.ip)+(Mu.op/ϕMn.op) 35,6 %

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2.0 Shear yielding (punching)

when 0,85< β <=1-1/γ or B/t<10 0,85<0,8<=0,76 or 8,33<=10 required

$$P_n \sin\theta = 0.6F_y tB\left(2\eta + 2\beta_{eop}\right) \quad \text{(K2-14)}$$

ϕ=	LRFD factor	0,95
γ=	chord slenderness ratio =B / 2*t	4,2
η=	load length parameter =l_b/B	0,8
l_b=	Hp/sinθ	80,0 mm
B_{eop}=	(5*β)/γ <= β	0,8

			with angle
P_n=		818 kN	817,9 kN
ϕP_n=	limit, LRFD	**777 kN**	**777,0 kN**
F_x=	axial force in post	50,0 kN	

Utilization: 6,4 %

3.0 Local yielding of chord sidewalls

when β>=0,85 (limit for M_n) β= 0,8 NOT Calculated

$$P_n \sin\theta = 2F_y t\left(5k + l_b\right) \quad \text{(K2-9)}$$

ϕ=	LRFD factor	1,0
k=	outside corner radies =1,5*t	18,0

		with angle
P_n=		
ϕP_n=	limit, LRFD	
F_x=	axial force in post	

$$M_n = 0.5F_y^* t\left(H_b + 5t\right)^2 \quad \text{(K3-12)}$$

$M_{n.ip}$=	moment capacity in plane
$\phi M_{n.ip}$=	ϕ=1.0

$$M_n = F_y^* t\left(B - t\right)\left(H_b + 5t\right) \quad \text{(K3-16)}$$

$M_{n.op}$=	moment capacity out of plane
$\phi M_{n.op}$=	ϕ=1.0

Utilization for combined loads: (Pu/ϕPn)+(Mu.ip/ϕMn.ip)+(Mu.op/ϕMn.op)

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4.0 Local crippling of chord sidewalls

when β=1.0 and post is in compression β= 0,8 **NOT Calculated**

$$P_n \sin\theta = 1.6t^2\left(1+\frac{3l_b}{H-3t}\right)\sqrt{EF_y}Q_f$$ (K2-10) (AISC spec.)

ϕ= LRFD factor 0,75

with angle

P_n=

ϕP_n= limit, LRFD

F_x= axial force in post

Utilization:

5.0 Local yielding of branch due to uneven load distribution

when β>=0,85 β= 0,8 **NOT Calculated**

$$P_n = F_{yb}t_b\left(2H_b + 2b_{eoi} - 4t_b\right)$$ (K2-18)

ϕ= LRFD factor 0,95

B_{eoi}= (10 / B/t)*(F_y*t / F_y*t_p)*B_p <= B_p 80,0 mm

with angle

P_n=

ϕP_n= limit, LRFD

F_x= axial force in post

$$M_n = F_{yb}\left[Z_b - \left(1-\frac{b_{eoi}}{B_b}\right)B_bH_bt_b\right]$$ (K3-13) in plane bending

$M_{n.ip}$= moment capacity in plane

$\phi M_{n.ip}$= ϕ=0.95

Z_b= plastic section modulus for branch in axis of bending

$$M_n = F_{yb}\left[Z_b - 0.5\left(1-\frac{b_{eoi}}{B_b}\right)^2 B_b^2 t_b\right]$$ (K3-17) out of plane bending

$M_{n.op}$= moment capacity out of plane

$\phi M_{n.op}$= ϕ=0.95

$Z_{b.op}$= plastic section modulus for branch in axis of bending

Utilization for combined loads: (Pu/ϕPn)+(Mu.ip/ϕMn.ip)+(Mu.op/ϕMn.op)

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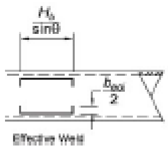
6.0 Chord distorional failure

only if there is bending out of plane

$$M_n = 2F_y t\left[H_b t + \sqrt{BHt(B+H)}\right]$$ (K3-19)

$M_{n.op}$=	moment capacity out of plane	49,9 kNm	
$\phi M_{n.op}$=	ϕ=1.0	**49,9 kNm**	9,0 %

7.0 Welds for post connecting to chord (AISC spec. Table K4.1)



$l_{e.p}$=	Effective weld length =(2*H_b/sinθ)+2*B_{eoi}	256 mm
S_{ip}=	effective elastic section modulus of welds, in plane bending	59733 mm³
S_{op}=	for out of plane bending	44613 mm³
F_{nw}=	nominal weld metal stress	423 MPa

when β>0.85 or θ>50 deg, B_{eoi}/2 shall not exceed 2t

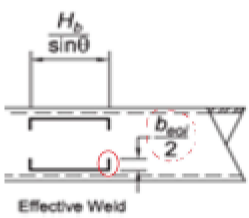
B_{eoi}/2= 24 mm

ϕ=	LRFD factor for fillet welds	0,75		
R_n=	φ*F_{nw}*t_w*l_e	**569,0 kN**	(K4-1)	8,8 %
$M_{n.ip}$=	φ*F_{nw}*S_{ip}	**19,0 kNm**	(K4-2)	34,3 %
$M_{n.op}$=	φ*F_{nw}*S_{op}	**14,2 kNm**	(K4-3)	31,8 %

Utilization for combined loads: (Ru/ϕRn)+(Mu.ip/ϕMn.ip)+(Mu.op/ϕMn.op) 74,8 %

7.1 Base metal strength Bending moments occuring: both directions

ϕ=		0,75	
R_n=	F_{bm}*A_{bm}	3384 N	/mm of weld
ϕR_n=		**2538 N**	/mm of weld
Controlling:	Fy/Fu<0.75 -> 0,76 > 0,75 therefore the controlling is:	rupture	
F_{bm}=	Fu*0.6	282 MPa	
t=	base metal thickness	12,0 mm	
l_e=	effective weld length	256,0 mm	
F_{mo}=	force caused by moment out of plane	879 N	/mm of weld
F_{mi}=	by moment of in plane bending	1270 N	/mm of weld
F_{mt}=	force by moment on critical part (corner if biaxial bending)	2148 N	/mm of weld
F=	axial force per mm of weld	195 N	
F_{crit}=	force per mm of weld in critical spot	2344 N	92,3 %



Corner is critical if both bending out of plane and in plane occurs, as shown in picture.

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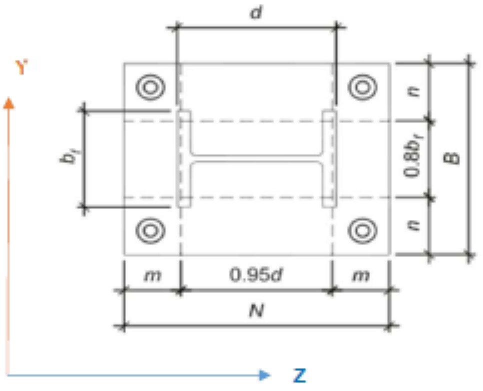
Fixed W shape

Column base plate connection
According to AISC 360-10 and
AISC design guide 1 column base plates



load combination:	**1**
Node in model:	**1**

Load input:

M_z=	78,0 kNm	moment in Z direction
M_y=	40,0 kNm	moment in Y direction
P_u=	200,0 kN	axial force (minus=uplift)
V_z=	50,0 kN	shear Z direction
V_y=	0,0 kN	shear Y direction

Type:	**HEB**	
Column:	**HEB300**	
b_f=	300 mm	flange width
d=	300 mm	height
t_w=	11,0 mm	web thickness
t_f=	19,0 mm	flange thickness
A=	14910 mm²	area of cross section
steel:	**S355**	
Fy=	355 MPa	yield strength
Fu=	470 MPa	tensile strength
Es=	200000 MPa	modulus of elasticity

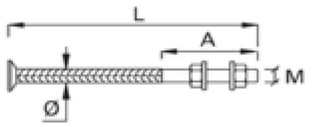
Base plate:	**460x460x30**	
B=	460 mm	
N=	460 mm	
t_p=	30 mm	plate thickness
A_1=	211600 mm²	area of base plate
m=	88 mm	edge - bending line X-dir.
n=	110 mm	in Y-direction

Custom: when custom is used, set type to "custom" and column to "X"

b=	0 mm	column width
d=	0 mm	depth
t_f=	0,0 mm	flange thickness
t_w=	0,0 mm	web thickness
A=	0 mm²	area for RHS

Rods:	**Peikko HPM Rebar**	
	HPM L 30	
dimensions:		
l=	500 mm	total length
Ø=	32 mm	bar diameter
M=	30 mm	bolt size
A=	190 mm	minimum A distance
Washer=	65-8	diamter and thickness

HPM L



Stengths:

nuts:	8	property class
F_{yn}=	640 MPa	yield strength
F_{un}=	800 MPa	tensile strength

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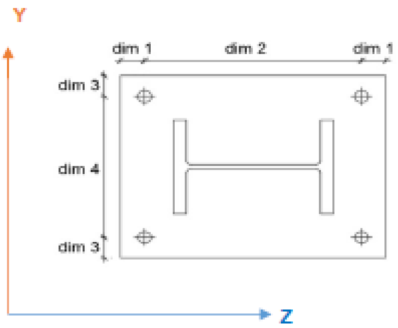
Concrete support:

f_c'=	28 MPa	compressive strength MPa for calculations
f_c'=	4 ksi	compressive strength input in ksi
A_2=	435600 mm²	maximum area of supporting surface

Bolts:

Fixed W shape

4 number of bolts



connection type and bolt placing

Connection: Fixed W shape 4 bolts

$n_{bolts,y}$=	2	bolts along sides in Y-axis
$n_{bolts,z}$=	2	bolts along sides in Z-axis

dimensions: **distance to column (bolt center)**

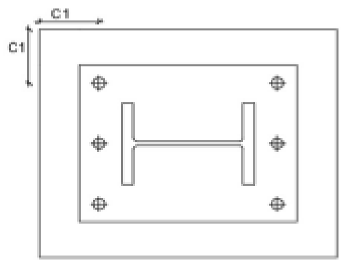
dim 1=	45 mm	35,0 mm
dim 2=	370 mm	
dim 3=	45 mm	35,0 mm
dim 4=	370 mm	

support type: **pier**

(base plate is concentric on pier, means that c_1 are equal on both sides of plate)

distance from base plate edge to concrete edge:

Y-axis=	100 mm
Z-axis=	100 mm



$c_{1,y}$=	145 mm	in Y-axis
$c_{1,z}$=	145 mm	in Z-axis

(recommended distance from anchor to concrete edge is >=6*anchor diameter) = 180 mm

anchor to concrete edge distance

1.0 Compressive axial loads (design guide 1, 3.1.)

Applied Case: III meaning: **A1<A2<4*A1**

where: A1= area of base plate

A2= maximum area of the part of supporting surface, geomatrically similar and concentric to loaded area

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1.1 Concrete bearing (design guide 1 3.1)

nominal bearing strength determined as: fp(max)=0.85*fc'*sqrt(A2/A1)

fp(max)=	nominal strength	33,6MPa	
sqrt(A1/A2)=	limited to <=2	1,434782609	
ϕfp(max)=	ϕ=0.65	21,9MPa	4,3 %

1.2 Base plate yielding limit, compression (design guide 1, 3.1.2)

ϕP_p=	ϕ*0,85*fc'*A1*sqrt(A2/A1)	4626 kN	OK, >Pu=200 kN
f_{pu}=	Pu/B*N	0,95 MPa	
M_u=	fpu*(l^2/2)	2630435 Nmm	
l=	larger of m, n and λn'	110 mm	
λn'=	λ*(sqrt(d*b_f)/4)	15,8 mm	
λ=	2*sqrt(X)/1+sqrt(1-X)<=1	0,21	
X=	(4*d*b_f/(d+b_f)^2)*Pu/ϕc*Pp	0,04	

$t_{min} = l\sqrt{\frac{2P_u}{\phi F_y BN}}$ (LRFD) equation solved for Pu Pu=B*N*t^2*phi*Fy/2*l^2

Pmax= max axial compression in column **2514,3 kN** 8,0 %

1.3 Yield line approac for small plates, compression (AISC design guide 1, 3.1.2)

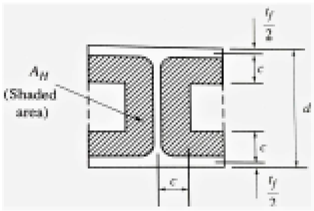
c= λn'= 15,8 mm

A_H= 16727 mm²

Pmax=

tp.req=

concrete bearing utilization:



yield lines for small plates

2.0 Tensile axial loads (design guide 1, 3.1.2)

d_{bolt}=	30 mm	bolt diameter
D_{head}=	70 mm	diameter of rod head
n_{bars}=	1	number of bars
A_{brg}=	3848 mm²	of anchor rod head(s)
A_{bolt}=	707 mm²	bolt area

F_{yb}=	640 MPa	bolt yield strength
F_{ub}=	800 MPa	bolt tensile strength

Reinforcement bar is always B500B, which gives:

F_y=	430 MPa
F_u=	550 MPa

tensile strength of bolt:

ϕRn= 0,75*0,75*F_u*A_b **318,1 kN** 13,7 %

tensile strenth of anchor:

ϕRn= 0,75*F_u*A_{ts} **331,8 kN** 13,1 %

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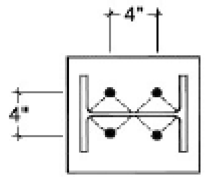
2.1 Plate thickness determined by bending (AISC design guide 1, 3.2)

2.1.1 Small plates (pinned W shape)

width of base plate resisting bending b_{eff}=
plastic section modulus for plate Z=
moment in plate caused by tension M_u=
required plate thickness $t_{req'd}$=



load distribution from rods to plate

2.1.2 Fixed plates with cantilevers

required plate thickness $t_{req'd}$= 6,9 mm 23,0 %

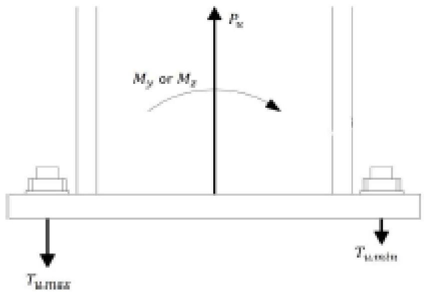
2.2 Uplift with moment (design guide 1, 3.1)

Positive = tension, Negative = compression

per bolt

$T_{u.max.y}$=
$T_{u.min.y}$=

$T_{u.max.z}$=
$T_{u.min.z}$=



bolt forces with combined uplift and moment

Combined:

$T_{u.max}$= critical bolt for tension (if bending is biaxial)
$T_{u.min}$= bolt with biggest compression

5.1 base plate yielding (design guide 1, 3.1)

required plate thickness, bending in Y-axis $t_{p.req.y}$=
required plate thickness, bending in Z-axis $t_{p.req.z}$=

required plate thickness with biaxial bending $t_{p.req}$=

combining bolt forces for pullout force $P_{u.M}$=
used for concrete breakout utilization

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2.3 concrete pullout strength

ϕNp= $\phi*\Psi_4*A_{brg}*8*f_c'$ **832,1 kN** 7,1 %

Ψ_4= 1,4

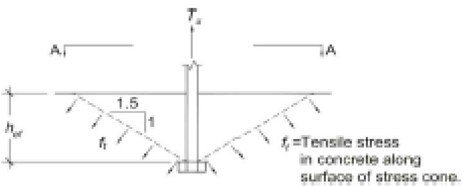
ψ_4 = 1.4 if the anchor is located in a region of a concrete member where analysis indicates no cracking ($f_t - f_r$) at service levels, otherwise ψ_4 = 1.0

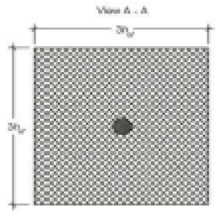
2.4 concrete capacity

this method is valid for anchor diameters of 50mm or less and tensile embedment length not exceeding 625mm

d= 30 mm

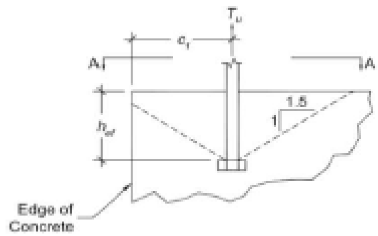
h_{ef}= 335 mm





when h_ef>=275mm strength can be determined by: ϕ*Ψ3*16*(√fc')*hef^(5/3)*AN/Ano

if base plate is near edge of concrete and closest anchor is closer than 1,5*h_{ef}, c_1 is used to decrease the cone area



close to concrete edge, reduction of area

A_N= 435600 mm reduced breakout cone area

ϕN_{cbg}= 512,4 kN strength after reduction

utilization= 33,1 %

where

Ψ_3= 1,25=concrete uncracked at service loads other 1,0 1,25

A_N= concrete breakout cone area for group 1890625 mm²

A_{no}= concrete breakout cone area for one anchor 1010025 mm²

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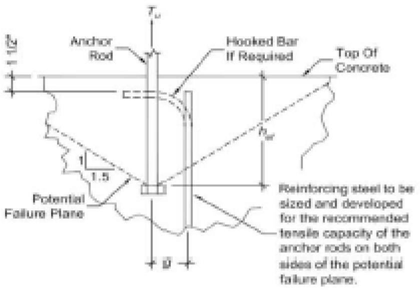
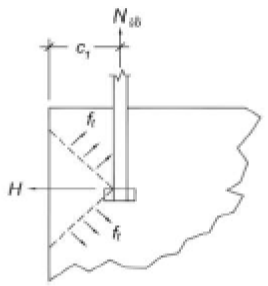
reinforcement overlap anchors

lap splice ensures ductile behavior

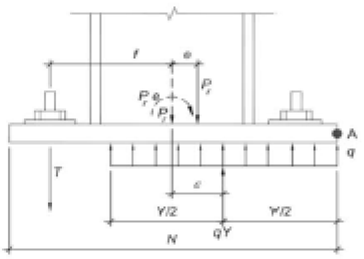
$\phi Rn=$	$\phi * A_{se} * F_y$	**407,2 kN**
$\phi=$	from ACI 318-02 chapter 9	0,9
	utilization=	**12,3 %**



3.0 Base plate with SMALL moments (design guide 1, 3.3)



picture of lateral bursting force for anchor in tension.



picture of column base plate with small bending moment

Y-axis = weak axis and Z-axis = strong axis for column

eccentricities

	critical		existing	makes moment considered:
$e_{crit,y}=$	220,1 mm	$e_y=$	200,0 mm	**Small**
$e_{crit,z}=$	220,1 mm	$e_z=$	390,0 mm	**Large**

3.1 Concrete bearing stress

in Y-axis:

			q_{max}
q=	bearing stress =Pr/Y	3333 N/mm	10057 N/mm
where:			
Y=	bearing length	60,0 mm	

in Z-axis:

			q_{max}
q=	bearing stress =Pr/Y		
where:			
Y=	bearing length		

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required strength of base plate is determined as (plate bending per unit width):

in Y-axis:

M_{pl}=	fp.max*Y*(n-Y/2)	**34783 Nmm**	48,4 %
where:			
n=	cantilever length	110,0 mm	
f_p=	Pr/B*Y	7,2 N/mm²	

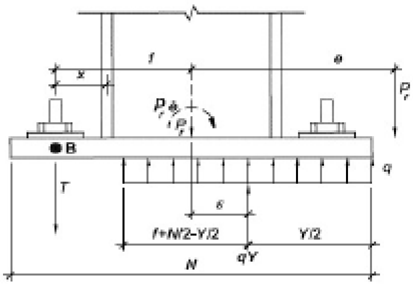
in Z-axis:

M_{pl}=	fp.max*Y*(m-Y/2)		
where:			
m=	cantilever length		
f_p=	Pr/B*Y		
ϕR_n=	beding resistance per unit width of the plate	**71888 Nmm**	

required plate thickness:

$t_{p.req.y}$=	Y-axis, with cantilever n	20,9 mm	69,6 %
$t_{p.req.z}$=	Z-axis, with cantilever m		
$t_{p.req}$=	combined with SRSS		

4.0 Base plate with LARGE moments (design guide 1, 3.3)



Y= weak direction (B = width)
Z= strong direction (N = width)

considered large if $e>e_{crit}$ (determined earlier):

M_y= Small
M_z= Large

picture of connection subjected to large bending moment

inequity satisfied for Y-axis 172225 >= 15313
inequity satisfied for Z-axis 172225 >= 22870

$$\left(f+\frac{N}{2}\right)^2 \geq \frac{2P_r(e+f)}{q_{max}} \quad 3.4.4)$$

if previous ineguity isn't satisfied, choose a larger base plate

$q_{max.y}$= 10057 N/mm $q_{max.z}$= 10057 N/mm

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Compression section (base plate yielding):

in Y-axis

f=	column center to center of bolt in tension		
Y=	bearing length		
	first part of formula		
	second part of formula		
tp.req=	required plate thickness on bearing interface		

$$Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2P_r(e+f)}{q_{max}}} \qquad (3.4.3)$$

used for both Y and Z

in Z-axis

f=	column center to center of bolt in tension	185 mm	
Y=	bearing length	28,5 mm	
	first part of formula	415,00	
	second part of formula	386,47	
tp.req=	required plate thickness on bearing interface	**23,9 mm**	79,8 %

Tension section:

in Y-axis

x=	bolt center to flange center		
T_u=	anchor rod tension		
$t_{p.req}$=	minimum plate thickness		

in Z-axis

x=	bolt center to flange center	44,5 mm	
T_u=	anchor rod tension	87,0 kN	
$t_{p.req}$=	minimum plate thickness	**10,3 mm**	34,2 %

Combined

$T_{u.crit}$=	tension in critical bolt	43,5 kN	

final plate thickness, larger value of $t_{p.req}$ chosen between tension and compression

$t_{p.req}$=	combined as sqrt(t1^2+t2^2)	**31,8 mm**	105,9 %

(does also combine plate thicknesses between small and large moments if needed)

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4.1 Anchor rod strength (design guide 1, 3.3)

ϕN_p=	$\phi * \Psi_4 * A_{brg} * 8 * f_c'$	**832,1 kN**	9,2 %
Ψ_4=		1,4	

$\psi_4 = 1.4$ if the anchor is located in a region of a concrete member where analysis indicates no cracking $(f_t - f_r)$ at service levels, otherwise $\psi_4 = 1.0$

4.2 Breakout cone for anchors in tension (design guide 1, 3.3)

in Y-axis

when h_ef>=275mm strength can be determined by:

ϕN_{cbg}=	$\phi * \Psi 3 * 16 * (\sqrt{fc'}) * hef^{(5/3)} * AN/Ano$	**775,0 kN**	0,7 %
where			
Ψ_3=	1,25=concrete uncracked at service loads, other 1,0	1,25	
A_N=	concrete breakout cone area for group	658831 mm²	
A_{no}=	concrete breakout cone area for one anchor	1010025 mm²	

in Z-axis

ϕN_{cbg}=	$\phi * \Psi 3 * 16 * (\sqrt{fc'}) * hef^{(5/3)} * AN/Ano$	**775,0 kN**	5,6 %
where			
Ψ_3=	1,25=concrete uncracked at service loads, other 1,0	1,25	
A_N=	concrete breakout cone area for group	658831 mm²	
A_{no}=	concrete breakout cone area for one anchor	1010025 mm²	

Combined

ϕN_{cbg}=	$\phi * \Psi 3 * 16 * (\sqrt{fc'}) * hef^{(5/3)} * AN/Ano$	**1056,8 kN**	7,8 %
where			
A_N=	combined group breakout cone area	898406 mm²	

5.0 Design for shear forces (design guide 1, 3.5)

5.1 Friction between plate and supporting surface (design guide 1, 3.5.1)

supporting surface for base plate: **concrete**

$$\phi V_n = \phi \mu P_u \leq 0.2 f_c' A_c$$

determines the shear strength due to friction

ϕV_n=		**105,0 kN**	0,0 %	Y-axis
where			47,6 %	Z-axis
ϕ=	LRFD factor	0,75		
μ=	friction coefficient	0,7		
A_c=	bearing area	211600 mm²		

No actions needed if shear strength due to friction is greater than the acting shear force

User must also verify that compression is present while shear force is occuring

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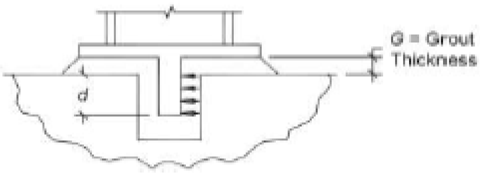
5.2 Concrete bearing, shear lug (design guide 1, 3.5.2)

G= 50 mm grout thickness



Shear lug detail

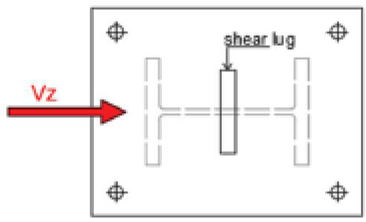
Dimensions of shear lug:

h=	96 mm	height of profile
w=	100 mm	widht
d=	100 mm	embedded depth
t_f=	8 mm	flange thickness
t_w=	5 mm	web thickness
R=	12 mm	radius, flange to web

shear force acts in: Z-axis
dominant shear force: Z-axis
plate in Y-axis recommended

Type and size of shear lug:

type	HEA	
size	HEA100	
L=	150 mm	profile length
w=	0 mm	plate widht



if plate is chosen it is placed to resist the dominant shear

shear forces and the suggested shear lug

$$\phi P_n = 0.80 f_c' A_\ell + 1.2(N_y - P_a) \text{ for shear lugs}$$

shear in Y-axis

ϕP_n=	just first part of formula for concrete crusching	**211,8 kN**	0,0 %
where			
A_l=	bearing area of shear lug	9600 mm²	

shear in Z-axis

ϕP_n=	just first part of formula for concrete crusching	**220,6 kN**	22,7 %
where			
A_l=	bearing area of shear lug	10000 mm²	

$$M_l = V(G + d/2)$$

Moment caused by the cantilever

$M_{l,y}$=	caused moment by cantilever in Y-axis	0,0 kNm	0,0 %
$M_{l,z}$=	in Z-axis	5,0 kNm	18,9 %

$$M_l = \phi F_y Z$$

Moment capacity

$M_{l,y}$=	in Y-axis	13,1 kNm
$M_{l,z}$=	in Z-axis	26,5 kNm

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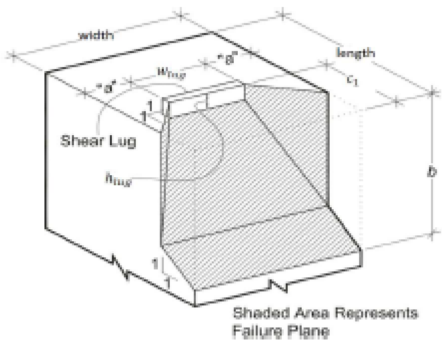
Z_y= plastic section modulus for Y-axis 41100 mm³
Z_z= for Z-axis 83000 mm³

5.2.1 Concrete break out at edges (design guide 1, 3.5.2)

$$V_u = 4\phi\sqrt{f_c'}A_v$$ shear capacity of concrete in front of lug

shear in Y-axis

V_u=		**3996,2 kN**
where			
ϕ=	LRFD factor	0,75
A_v=	projected area of shear plane	253648 mm²
c1=		282 mm
b=	(from picture)	382 mm
a_1=	(from picture)	282 mm
a_2=		282 mm



picture of projected shear area

shear in Z-axis

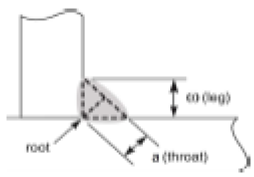
V_u=		**4471,3 kN**
where		
A_v=	projected area of shear plane	283800 mm²
c1=		282 mm
b=	(from picture)	430 mm
a_1=	(from picture)	280 mm
a_2=		280 mm

Y-axis: **0,0 %**
Z-axis: **1,1 %**

5.2.2 Shear lug welds (AISC 360-10, J2-5)

flange weld properties:

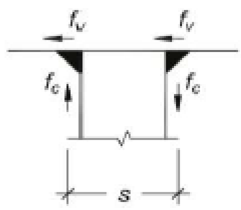
a=	4,0 mm	weld throat size		
w=	5,7 mm	leg size	**min leg size=**	**5 mm**
filler metal=	E70XX			
F_{EXX}=	483 MPa	filler metal strength		
t_i=	8 mm	thinner of the joined parts		



weld dimensions

web weld properties:

a=	4,0 mm	weld throat size		
w=	5,7 mm	leg size	**min leg size=**	**5 mm**
filler metal=	E70XX			
F_{EXX}=	483 MPa	filler metal strength		
t_i=	5 mm	thinner of the joined parts		



forces on welds

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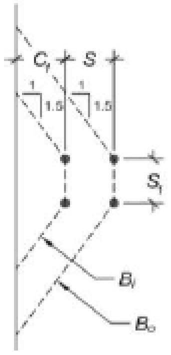
Flange welds:

effective length of flange welds	l_{eff}=	195 mm	
effective area of flange welds	A_{eff}=	2356 mm²	
radius of inner corner (W shapes)	R=	12 mm	
strength of fillet welds at one flange (kN/mm of weld)	**ϕR_n=**	**869 N/mm**	33,5 %
shear force in Y-axis	V_y=	0 N/mm	
force on weld caused by bending moment in Y-axis	T_{My}=	0 N/mm	
force on weld caused by bending moment Z-axis	T_{Mz}=	291 N/mm	
resultant force of shear and bending	f_r=	**291 N/mm**	

Web welds:

effective length of web welds	l_{eff}=	304 mm	
effective area of web welds	A_{eff}=	1216 mm²	
strength of fillet welds for	**ϕR_n=**	**869 N/mm**	18,9 %
shear force in Y-axis	V_y=	0 N/mm	
shear force in Z-axis	V_z=	164 N/mm	
tension force on weld by bending in Y-axis	T_{My}=	0,0 N/mm	
tension force on weld by bending in Z-axis	T_{Mz}=	0,0 N/mm	
resultant force of tension and bending	f_r=	**164 N/mm**	

5.3 Anchor rods shear strength (AISC design guide 1, 3.5.3)



	Y-axis		Z-axis
C_1=	145 mm	C_1=	145 mm
S=	370 mm	S=	370 mm
S_1=	370 mm	S_1=	370 mm

ϕV_a= single anchor shear strength **85,5 kN** 14,6 %

	Y-axis		Z-axis
S_1/S=	1	S_1/S=	1
C_1/S=	0,39	C_1/S=	0,39
controlling line:	Bi	controlling line:	Bi

S_1/S	C_1/S For B_o To Control
0.5	> 2.33
2/3	> 2.31
1.0	> 2.26
1,5	> 2,19
2,0	> 2,12

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Y-axis

$$\phi V_{cbg} = \phi \frac{A_v}{A_{vo}} \Psi_5 \Psi_6 \Psi_7 V_b$$ concrete break strength out for anchor group

where

A_v=	total breakout shear area for anchor our group of anchors	175088 mm²	
A_{vo}=	$4{,}5{*}C_1^{\ 2}$, area of shear cone for a single anchor	94613 mm²	
Ψ_5=	when all anchors have same load =1	1	
Ψ_6=	capacity reduction when side cover limits size of cone	1	
Ψ_7=	1,4 for uncracked or supplementary reinforcement	1,4	
V_b=	$7{*}(l/d_0)^2{*}\mathrm{sqrt}(d_0){*}\mathrm{sqrt}(f_c'){*}c_1^{\ 1,5}$	550352	
d_0=		32 mm	
l=	embedment depth	335 mm	
l/d_0=	limited to 8, maximum load bearing length $=8d_0$	8	
ϕV_{cbg}=		**926,8 kN**	0,0 %

Z-axis

where

A_v=	total breakout shear area for anchor our group of anchors	175088 mm²	
A_{vo}=	$4{,}5{*}C_1^{\ 2}$, area of shear cone for a single anchor	94613 mm²	
Ψ_6=	capacity reduction when side cover limits size of cone	1	
V_b=	$7{*}(l/d_0)^2{*}\mathrm{sqrt}(d_0){*}\mathrm{sqrt}(f_c'){*}c_1^{\ 1,5}$	550352	
ϕV_{cbg}=		**926,8 kN**	5,4 %

V_{cp}=	pryout strength	**1664,3 kN**	3,0 %
where			
K_{cp}=		2	
N_{cp}=		832,1 kN	

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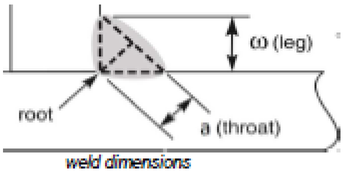
6.0 Column to base plate welds (AISC 360-10, J2-5)

flange weld properties:

a=	5 mm	weld throat size		
w=	7,1 mm	leg size	min leg size=	6 mm
filler metal=	E70XX			
F_{EXX}=	483 MPa	filler metal strength		
t_i=	19 mm	thinner of the joined parts		



web weld properties:

a=	4 mm	weld throat size		
w=	5,7 mm	leg size	min leg size=	5 mm
filler metal=	E70XX			
F_{EXX}=	483 MPa	filler metal strength		
t_i=	11 mm	thinner of the joined parts		

Flange welds:

effective length of flange welds	l_{eff}=	589 mm	
effective area of flange welds	A_{eff}=	2945 mm²	
radius of inner corner (W shapes)	R=	27 mm	
strength of fillet welds at one flange (kN/mm of weld)	**ϕR_n=**	**1087 N/mm**	96,6 %
shear force in Y-axis	V_y=	0 N/mm	
force on weld caused by bending moment in Y-axis	T_{My}=	453 N/mm	
force on weld caused by bending moment Z-axis	T_{Mz}=	471 N/mm	
tension force on welds by tension in column	P_u=	125 N/mm	
resultant force of shear and bending	f_r=	**1049 N/mm**	

Web welds:

effective length of web welds	l_{eff}=	416 mm	
effective area of web welds	A_{eff}=	1664 mm²	
strength of fillet welds for	**ϕR_n=**	**869 N/mm**	20,0 %
shear force in Z-axis	V_z=	120 N/mm	
tension force on welds by tension in column	P_u=	125 N/mm	
force on weld caused by bending in Y-axis(only for HSS)	T_{My}=	0 N/mm	
resultant force of tension and bending	f_r=	**174 N/mm**	