

Column base plates under 3D loading

Laura da Silva Seco

▶ To cite this version:

Laura da Silva Seco. Column base plates under 3D loading. Structures. INSA de Rennes, 2019. English. NNT: 2019ISAR0009 . tel-03227367

HAL Id: tel-03227367 https://theses.hal.science/tel-03227367

Submitted on 17 May 2021

HAL is a multi-disciplinary open access archive for the deposit and dissemination of scientific research documents, whether they are published or not. The documents may come from teaching and research institutions in France or abroad, or from public or private research centers. L'archive ouverte pluridisciplinaire **HAL**, est destinée au dépôt et à la diffusion de documents scientifiques de niveau recherche, publiés ou non, émanant des établissements d'enseignement et de recherche français ou étrangers, des laboratoires publics ou privés.





THESE DE DOCTORAT DE

L'INSA RENNES Comue Universite Bretagne Loire

ECOLE DOCTORALE N° 602 Sciences pour l'Ingénieur Spécialité : Génie Civil

Par Laura DA SILVA SECO

Column base plates under 3D loading

Thèse présentée et soutenue à l'INSA de Rennes, le 29/11/2019 Unité de recherche : Laboratoire Génie Civil et Génie Mécanique Thèse Nº : 19ISAR 26 / D19 - 26

Rapporteurs avant soutenance : Composition du Jury :

Abdelhamid BOUCHA	R Professeur - Université Clermont Auvergne	Frantisek WALD Professeur – Czech Technical University, Prague I président
Frantisek WALD	Professeur - Czech Technical University Prague	Abdelhamid BOUCHAIR Professeur – Université Clermont Auvergne rapporteur Ana GIRAO COELHO Chargé de Recherche – Steel Construction Institute, Ascot UK examinateur Luís COSTA NEVES Professeur Associé – University of Coimbra, Portugal examinateur Mohammed HJIAJ Professeur – INSA Rennes Directeur de thèse Maël COUCHAUX Maître de Conférences – INSA Rennes Co-encadrant de thèse





Intitulé de la thèse :

Pieds de poteaux par platine d'assise sous sollicitations tridimensionnelles

Laura DA SILVA SECO

En partenariat avec :

Document protégé par les droits d'auteur

A pessimist sees the difficulty in every opportunity. An optimist sees the opportunity in every difficulty. Winston Churchill

ACKNOWLEDGEMENTS

This thesis has been the result of three years of hard work filled of obstacles but above all, achievements. This work was not possible without the support, encouragement, friendship and guidance of several people to whom I would like to express my gratitude.

First, I would like to thank my main supervisor, Professor Mohammed Hjiaj, for his guidance, support, patience and wise advices. I owe him the opportunity and conditions to embrace this project.

I would also like to address my most sincere gratitude to my co-supervisor, Professor Maël Couchaux, for his dedication, knowledge, constant support given that made possible the accomplishment of this work. It has been a privilege to work with both.

My acknowledgements to the LGCGM team: Anas Alhasawi, Christian Garand, David Cvetkovic, François-Xavier Bourdoulous, Frederic Marie, Jean-Luc Métayer, Quang Huy Nguyen, Quentin Lavazay and Raphaël Léon. Special thanks to Professor Christophe Lanos, Professor Habib Mesbah and Guy Bianéis from IUT civil engineering department for having kindly received me in the laboratory to perform the concrete compressive tests.

A word of gratitude to all my colleagues Noussaiba Graine, Hamza Bennani, Phuong Nguyen Viet and Tuan-Anh Nguyen that provided me an enriching experience in so many levels.

A special word to my mentor and friend Professor Luís Costa Neves for his support, concern and friendship over the years.

Thanks to my Portuguese friends for their constant encouragement, endless support and for understanding my absence.

Finally, to my parents Maria Isabel and Carlos, and my sister Véronique, there is not enough words to say thank you. Thank you for your love, encouragement, motivation and unconditional support. This thesis is dedicated to you.

ABSTRACT

Column bases have a major influence on the stability and stiffness of steel structures. This thesis focuses on the estimation of the resistance of column base plates subjected to a combination of axial force and biaxial bending moment. The investigated connections consist of a steel column welded to a base plate and fixed to the concrete block by means of four anchor bolts.

In Chapter II, an extensive literature review is presented, summarizing the existing models for the estimation of the resistance of column base plates as well as the results of previous experimental test campaigns.

Chapter III is dedicated to the analysis of the results of a test campaign carried out on 9 column base plates, subjected to in-plane, out-of-plane and biaxial bending moments. Particular attention is given to the influence of the base plate thickness and orientation of the applied bending moment. Results are presented and discussed, to better understand the elastic and postlimit behaviors. These experimental results are completed with refined numerical simulations presented in Chapter IV. A parametric study is conducted to broaden this investigation, by adding a normal force and other geometrical configurations (column steel profile HEA, IPE, diameter of the anchor bolts).

Next, an analytical model is proposed, based on the Component Method in Eurocode 3 Part 1-8, to calculate the resistance of column base plates under uniaxial and biaxial bending moments. The conservative nature of the model is demonstrated by comparing the model predictions against experimental and numerical results.

Keywords: Column base plate, biaxial bending, anchoring, M-N interaction

RÉSUMÉ

Les pieds de poteaux ont une influence prépondérante sur la stabilité et la rigidité des charpentes métalliques. Cette thèse porte sur la résistance des pieds de poteaux par platine soumis à un effort normal et un moment biaxial. Les pieds de poteaux étudiés sont composés d'une platine soudée à l'extrémité du poteau et reliée au bloc béton par 4 tiges.

Dans le chapitre II, une étude bibliographique dresse un bilan des modèles permettant de calculer la résistance de ce type d'assemblage ainsi que des résultats d'essais.

Le chapitre III est ensuite dédié à la description et l'analyse des résultats d'une campagne d'essais menée sur 9 assemblages soumis à un moment dans le plan, hors plan ou de la flexion biaxiale. Une attention particulière est accordée à l'influence de l'épaisseur de la platine et de l'orientation du moment. Les résultats sont présentés et discutés afin de mieux comprendre le comportement élastique et plastique. Cette étude est complétée par des analyses par éléments finis dans le chapitre IV validées par confrontation aux résultats d'essais. Une étude paramétrique a ensuite permis d'étendre le domaine d'investigation en ajoutant un effort normal et d'autres configurations géométriques (sections en HEA, IPE, diamètres des tiges d'ancrage).

Un modèle analytique, basé sur la méthode des composants de l'Eurocode 3 partie 1-8, est ensuite proposé afin de calculer la résistance des pieds de poteau en flexion uniaxiale (dans le plan et hors plan) et biaxiale. Le caractère conservatif de la méthode est démontré par comparaison aux résultats des essais et des études numériques.

Mots-clés: Pied de poteau, platine, flexion biaxiale, ancrage, interaction M-N

RÉSUMÉ ÉTENDU EN FRANÇAIS

Chapitre 1 : Introduction

Le chapitre 1 présente le cadre d'études où s'insère la thèse ainsi que les objectifs principaux:

- proposer un modèle de dimensionnement simple, basé sur la méthode des composants de l'Eurocode 3 partie 1-8, afin d'évaluer la résistance des pieds de poteaux soumis à la combinaison d'un effort normal de traction/compression et d'un moment fléchissant uniaxial (dans le plan, hors plan) ou biaxial. Les deux derniers cas ne sont pas couverts par les normes de dimensionnement actuelles,
- mener une étude expérimentale pour caractériser le comportement élastique et elastoplastique de ce type d'assemblage jusqu'à la ruine afin de valider le modèle analytique,
- développer un modèle de calcul par éléments finis sur le logiciel ABAQUS permettant de modéliser de façon fiable le comportement mécanique des assemblages testés,
- réaliser une étude paramétrique approfondie à l'aide du modèle de calcul par éléments finis afin d'étudier l'influence des paramètres tels que : le niveau de chargement axial, l'orientation du moment fléchissant appliqué, l'épaisseur de la platine, le type de profilé du poteau, le diamètre et l'emplacement des tiges d'ancrage.

Chapitre 2 : État de l'art

Le chapitre 2 comporte une étude bibliographique approfondie sur les pieds de poteaux. Une description détaillée des principaux développements et investigations menés est présentée. Cette section est organisée chronologiquement, en fonction de l'orientation du moment de fléchissant appliqué, en présentant les campagnes d'essais, les études numériques et les modèles analytiques proposés dans la littérature. Ce chapitre présente également une synthèse des principales méthodes de dimensionnement (élastiques et plastiques) applicables aux pieds de poteaux.



Figure 1: Type d'assemblage étudiée

Chapitre 3 : Étude expérimentale

Ce chapitre est dédié à la présentation de la campagne d'essais expérimentaux menée sur les pieds de poteaux au sein du Laboratoire de Génie Civil et Génie Mécanique de l'INSA Rennes. L'objectif de cette campagne de six essais est d'étudier le comportement mécanique des pieds de poteaux sous chargement monotone et notamment l'influence de l'épaisseur de la platine d'assise ainsi que l'orientation du moment fléchissant appliqué (dans le plan, hors plan et biaxial à 45°). Six spécimens à échelle 1 ont été dimensionnés, fabriqués et testés jusqu'à la ruine. L'épaisseur de la platine varie entre 10 et 20 mm.



Figure 2: Spécimens d'essais et orientations considérées pour le moment fléchissant appliqué

Le poteau était constitué d'un HEA 200 en acier S275. Les platines de dimensions 330×300×10 (ou 20 mm) étaient composées d'un acier S355. Quatre tiges d'ancrage M16 de classe 5.6 étaient ancrées à 300 mm de profondeur dans un bloc béton de dimensions 1450×900×610 mm composé d'un C25/30.

Tableau 1: Définition des spécimens d'essais				
Test	Moment	Épaisseur de la		
Test	Moment	platine (mm)		
SPE1-M0	Dans le plan	10		
SPE2-M0	Dans le plan	20		
SPE1-M90	Hors plan	10		
SPE2-M90	Hors plan	20		
SPE1-M45	Biaxial (45°)	10		
SPE2-M45	Biaxial (45°)	20		



Figure 3: Dimensions des éléments constituants les spécimens testés

Lors des essais, deux types de mesures ont été mises en œuvre:

- 7 LVDT placés le long du poteau et du bloc de béton, mesurant les déplacements verticaux; 4 LVDT placés en haut et en bas de la platine d'assise mesurant les déplacements horizontaux (voir Figure 4),
- 4 jauges mesurant la déformation du profilé HEA 200 au voisinage des soudures en présence d'un moment hors plan (voir Figure 5).



Figure 4: Configuration des essais (cas de SPE1/2-M0)

Figure 5: Positionnement des jauges de déformations

Le montage expérimental est présenté à la Figure 4. Les spécimens ont été chargés par un vérin hydraulique positionné à 1085 mm du bloc de béton. Les principales conclusions de ces essais sont:

- la ruine est obtenue à par rupture des boulons en traction avec une plastification de la platine pour les spécimens de 10 mm d'épaisseur,
- la résistance et la rigidité initiale en rotation augmentent avec l'épaisseur de la platine pour les trois orientations du moment étudiées, mais aussi lorsque le bras de levier augmente soit en modifiant l'orientation du moment,

- la capacité de rotation de l'assemblage augmente en diminuant l'épaisseur de la platine,
- la redistribution des efforts internes entre les tiges d'ancrage dépend de l'orientation du moment,
- l'orientation du moment fléchissant a un effet sur le développement des forces de levier,
- l'étendue de la zone comprimée dépend fortement de l'épaisseur de la platine ainsi que de l'orientation du moment fléchissant appliqué,
- les concentrations de contraintes sont importantes dans les semelles du poteau au voisinage des soudures.







Figure 7: Comparaison des courbes moment-rotation pour évaluer l'influence de l'orientation du moment appliqué



Figure 8: Déformée de la platine à la fin des essais SPE1-M0, SPE1-M90 et SPE1-M45



Figure 9: Déformée de la platine à la fin des essais SPE2-M0, SPE2-M90 et SPE2-M45



Figure 10: Distribution des déformations sur la semelle du poteau

Chapitre 4 : Étude numérique et paramétrique

Dans ce chapitre, une étude numérique du comportement des assemblages testés sous chargement statique est présentée. Le logiciel ABAQUS a été utilisé pour créer un modèle de calcul par éléments finis 3D. Les non linéarités matérielles et géométriques ainsi que le contact ont été pris en compte. Le béton a été modélisé à l'aide du modèle «concrete damage plasticity». La figure 11 montre le type d'élément utilisé ainsi que le maillage adopté. Des éléments volumiques ont été utilisés pour le béton, le poteau, la platine et les tiges, des éléments filaires ont permis de modéliser les armatures dans le béton.





Figure 12: Interactions considérées dans la modélisation

Le modèle de calcul par éléments finis est d'abord validé par comparaison aux résultats d'essais présentés dans le chapitre 3. Les principaux résultats de cette comparaison sont présentés cidessous:

- les résultats montrent que le modèle EF prédit assez bien la résistance maximale ainsi que la rigidité initiale (voir Figure 13). La déviation maximale est de 20%,
- les éléments plastifiés et les modes de ruine obtenus sont similaires. De plus, les modèles numériques permettent de vérifier la plastification du poteau et des soudures au niveau des assemblages SPE1-M0, SPE1-M90 et SPE1-M45,
- les modèles numériques permettent d'évaluer la contribution des tiges d'ancrage à la résistance aux charges appliquées. Pour les séries M90 et M45, on observe une redistribution des efforts entre les tiges d'ancrage,
- la distribution des contraintes à la base des semelles du poteau pour les échantillons SPE1-M90 et SPE2-M90 est correctement estimée par le modèle numérique,
- la position du centre de compression varie en fonction du moment fléchissant appliqué,
- pour toutes les séries de tests (M0, M90 et M45), la distance du centre compression au centre du poteau est plus élevée pour les platines épaisses, ce qui permet de conclure que z_c dépend fortement de la flexibilité de la platine.



Le modèle de calcul par éléments finis a été utilisé dans le cadre d'une étude paramétrique afin d'étudier l'influence de l'effort axial, de l'épaisseur de la platine, du type de poteau (HEA 200

ou IPE 200) et du diamètre des tiges.

Tableau 2: Caractéristiques des assemblages de l'étude paramétrique

-			
ID	Poteau	Épaisseur de la platine <i>t</i> _{bp} (mm)	Diamètre des tiges (mm)
P1	HEA 200	10	16
P2	HEA 200	20	16
P3	IPE 200	10	16
P4	IPE 200	20	16
P5	HEA 200	10	20
P6	HEA 200	20	20



Figure 14: Dimensions des assemblages étudiées

Les principales conclusions de cette étude sont :

- les modes de ruine dépendent fortement du niveau de l'effort axial appliqué et de la direction du moment fléchissant,
- l'augmentation de l'effort de traction diminue le moment résistant ainsi que la rigidité initiale,
- l'augmentation de l'effort de compression appliqué augmente la résistance et la rigidité initiale dans certains cas (M0 et M45) pour des valeurs allant jusqu'à la moitié de la résistance en compression pure de l'assemblage,
- en présence d'efforts de compression importants la ruine est obtenue par instabilité locale des semelles du poteau,
- l'augmentation de l'épaisseur de la platine tend à améliorer la résistance et la rigidité initiale des assemblages, sauf en présence d'un effort de compression conséquent,
- l'utilisation d'un profil en acier HEA au lieu d'un IPE augmente le bras de levier en présence d'un moment fléchissant hors plan dominant,
- l'augmentation du diamètre des tiges d'ancrage augmente le moment résistant ainsi que la rigidité initiale, dans les cas où la ruine correspond à la ruine des tiges d'ancrage.

Chapitre 5 : Modèle analytique

Ce chapitre est consacré au développement d'une méthode de calcul de la résistance des pieds de poteaux soumis à la combinaison d'un effort normal et d'un moment fléchissant (dans le plan, hors plan et flexion biaxiale). Cette méthode est basée principalement sur la méthode des composantes de l'Eurocode 3 Partie 1-8 ainsi que l'Eurocode 2. Une des principales inconnues du problème est le mécanisme de transfert des efforts du poteau aux éléments de l'assemblage.

Pour le cas du moment dans le plan, deux modèles ont été développés – nommés simplifié et complet – afin de considérer un cas sans la contribution de l'âme du poteau en compression et l'autre avec cette contribution. Pour un moment fléchissant hors plan et dans les deux directions, un seul modèle est proposé. Les modèles considèrent que la ruine ce produit dans l'assemblage (Modes de tronçons en T selon l'Eurocode 3 Partie 1-8) ou par la plastification du poteau. La procédure de calcul est liée au type d'effort dominant. Les modes de fonctionnement considérés sont les suivants:

- effort de traction dominant,
- moment fléchissant dominant avec traction critique,
- moment fléchissant dominant avec compression critique,
- effort de compression dominant.

Après vérification du type de fonctionnement, l'objectif du modèle analytique proposé est de déterminer la résistance de l'assemblage à partir de l'équilibre des efforts développés dans les côtés tendu et comprimé (résistances des T-stubs en traction et compression $F_{t,u}$ et $F_{c,u}$). Les résultats obtenus par les modèles de calcul sont comparés aux résultats expérimentaux et aux simulations numériques. Il est montré que le modèle de dimensionnement de l'assemblage prédit avec une bonne précision la résistance en flexion des pieds de poteaux (voir Tableau 3).

Tableau 3: Comparaison des résultats obtenus par les modèles de dimensionnement et les essais expérimentaux					
	Expérimentale		Analytique		Ana/Exp
ID	Résistance ultime	Mode de	Résistance ultime	Mode de	M
	$M_{j,u,exp}$ (kNm)	rupture	M _{j,u,ana} (kNm)	rupture	<i>W</i> I j,u
SPE1-M0	43,2	Mode 2	43,4	Mode 2	1,00
SPE2-M0	48,5	Mode 3	50,7	Mode 3	1,04
SPE1-M90	36,2	Mode 2	33,0	Mode 2	0,92
SPE2-M90	43,7	Mode 3	40,9	Mode 3	0,94
SPE1-M45	39,4	Mode 2	41,5	Mode 2	1,05
SPE2-M45	47,3	Mode 3	46,3	Mode 3	1.03





TABLE OF CONTENTS

ACKNOWLEDGEMENTS

ABSTRAC	CT
RÉSUMÉ	
RÉSUMÉ	ÉTENDU EN FRANÇAIS
TABLE O	F CONTENTS
MAIN NO	TATIONS
1. INTR	ODUCTION
1.1. S	cope of the thesis
1.2. O	bjectives2
1.3. C	hapters organization
2. STAT	TE OF ART
2.1. Ir	ntroduction
2.2. H	listorical background
2.3. P	revious analytical, numerical and experimental studies
2.3.1	In-plane bending studies
2.3.2	Out-of-plane bending studies
2.3.3	Biaxial bending studies
2.3.4	Rotational stiffness and rotation capacity
2.4. D	esign methods
2.4.1	Elastic design methods
2.4.2	Design method EN 1993-1-8: Component method
2.5. C	oncluding remarks
3. EXPE	ERIMENTAL TESTS
3.1. Ir	ntroduction
3.2. T	ests set-up51
3.3. G	eometry of specimens tested
3.3.1	Column base plates
3.3.2	Beam-to-concrete slab connections
3.4. Ir	nstrumentation

3.4	.1	Column base plates experimental program	58
3.4	.2	Beam-to-concrete slab connections experimental program	59
3.5	Mat	erial characteristics	60
3.5	5.1	Column base plate	60
3.5	5.2	Beam-to-concrete slab connections	65
3.6	Tes	ting procedure	67
3.7	Exp	erimental results	68
3.7	'.1	Column base plates experimental program	68
3.7	7.2	Beam-to-concrete slab connections experimental program	97
APPEN	DIX	A	107
A.1 C	Concr	ete block reinforcement of the column base plates	107
A.2 C	Concr	ete block reinforcement of the beam-to-concrete slab connections	113
4. NU	JMEI	RICAL STUDY	119
4.1	Intr	oduction	119
4.2	Mo	del definition	119
4.2	2.1	Geometry	119
4.2	2.2	Materials	121
4.2	2.3	Steps	127
4.2	2.4	Interactions	127
4.2	2.5	Loading and boundary conditions	130
4.2	2.6	Mesh	130
4.3	Con	nparisons against test results	133
4.3	8.1	Column base plate tests	133
4.3	8.2	Beam-to-concrete slab connection tests	147
4.4	Para	ametric study	152
4.4	.1	Introduction	152
4.4	.2	Geometry and loading conditions	153
4.4	.3	Failure modes/influence of the loading conditions	155
4.4	.4	Influence of the geometric parameters	170
4.4	.5	Concluding remarks	183
5. AN	JALY	TICAL MODELS FOR RESISTANCE	185
5.1	Intr	oduction	185

5.2	Mai	in assumptions	
5.2	.1	Components in tension	186
5.2	.2	Components in compression	186
5.3	In-p	plane bending moment	
5.3	.1	Simplified analytical model	
5.3	.2	Full analytical model	191
5.3	.3	Comparison against experimental and numerical results	196
5.4	Out	-of-plane bending moment	
5.4	.1	Analytical model/general hypothesis	
5.4	.2	Comparison against experimental and numerical results	
5.5.	Bia	xial bending moment	
5.5	.1	Analytical model/general hypotheses	
5.5	.2	Comparison against numerical and experimental results	
5.6	Con	cluding remarks	210
APPEN	DIX	В	211
B.1 J	oint c	configuration	211
B.2 Iı	n-plai	ne bending moment resistance: simplified model	212
B.3 Iı	n-plai	ne bending moment resistance: full model	214
B.4 C	out-of	f-plane bending moment resistance	215
B.5 B	iaxia	l bending moment resistance	216
6. CC	NCL	USIONS AND RECOMMENDATIONS	219
6.1	Obj	ectives	219
6.2	Con	nclusions	
6.2	.1	Experimental program	
6.2	.2	Numerical study	
6.2	.3	Analytical study	221
6.3	Fort	thcoming works	222
BIBLIO	GRA	APHY	225

MAIN NOTATIONS

Latin lower cases

ac	First term of the second degree equation for biaxial bending moment resistance
	in dominant compression
a_{M}	First term of the second degree equation for biaxial bending moment resistance
	in dominant bending moment
a_{T}	First term of the second degree equation for biaxial bending moment resistance
	in dominant tension
a	Weld throat thickness
$b_{ m bp}$	Width of the base plate
$b_{ m fc}$	Width of the column flange
$b_{\rm C}$	Second term of the second degree equation for biaxial bending moment
	resistance in dominant compression
b_{M}	Second term of the second degree equation for biaxial bending moment
	resistance in dominant bending moment
b_{T}	Second term of the second degree equation for biaxial bending moment
	resistance in dominant tension
$b_{ m eff,c}$	Column flange effective width
$b_{ m eff,w}$	Column web effective width
$b_{ m f}$	Width of the concrete block
С	Additional effective width
c^*	Modified additional effective width
CC	Third term of the second degree equation for biaxial bending moment resistance
	in dominant compression
CM	Third term of the second degree equation for biaxial bending moment resistance
	in dominant bending moment
CT	Third term of the second degree equation for biaxial bending moment resistance
	in dominant tension
d_0	Diameter of the anchor bolts holes
$d_{ m f}$	Depth of the concrete block
$d_{ m w}$	Diameter of the anchor bolts nut
е	Distance from the centre line of the anchor bolt to the edge of the base plate, in
	the direction parallel to the width $b_{\rm bp}$ of the base plate
eb	Distance between the edge of the base plate and the edge of the concrete block
	in the direction parallel to $b_{\rm bp}$
e_{h}	Distance between the edge of the base plate and the edge of the concrete block
	in the direction perpendicular to $b_{\rm bp}$
$e_{\rm N}$	Eccentricity

$e_{\rm x}$	Distance from the centre line of the anchor bolt to the edge of the base plate, in
	the direction perpendicular to the width $b_{\rm bp}$ of the base plate
$f_{ m ck}$	Compressive strength of concrete
fctm	Tensile strength of concrete
fc	Specified compressive strength of grout
fcm	Mean compressive strength at 28 days
fc,u	Uniformly distributed resistance along the column effective width in
c *	compression
<i>f</i> jd	Modified design bearing strength
<i>f</i> jd,max	Maximum design bearing strength of concrete
<i>f</i> jd,f	Design bearing strength of the column flange
fjd,w	Design bearing strength of the column web
fm	Ultimate strength corresponding to beginning of the strain hardening
f _{t,u}	Uniformly distributed resistance along the column effective width in tension
fub	Ultimate strength of anchor bolts
$f_{ m ubp}$	Ultimate strength of the base plate
$f_{ m uc}$	Ultimate strength of the column
$f_{ m yc}$	Yield strength of the column
$f_{ m ub}$	Yield strength of the anchor bolts
$f_{ m ybp}$	Yield strength of the base plate
$h_{ m bp}$	Height of the base plate
$h_{ m c}$	Height of the column cross-section
$h_{ m f}$	Height of the concrete block
k_2	Safety coefficient for the resistance of the anchor bolt in tension
kc	Stiffness coefficient in compression
k_{T}	Stiffness coefficient in tension
l	Total length of the coupon test at failure
l_0	Initial total length of the coupon test
l _{cp,i}	Effective length for circular pattern <i>i</i>
<i>l</i> _{eff,c}	Column flange T-stub effective length
l _{eff,i}	Effective length for failure mode <i>i</i>
$l_{\rm eff,w}$	Column web T-stub effective length
l _{nc,i}	Effective length for non-circular pattern <i>i</i>
m	Distance from the centre line of the anchor bolt to the edge of the weld, in the
	direction perpendicular to the width b_{bp} of the base plate
$m_{ m pl}$	Plastic moment per unit length of yield line
n	Number of activated anchor bolts
n _b	Number of activated anchor bolts rows
р	Distance between the anchor bolts in the direction of the height of the base plate
	$h_{ m bp}$
<i>S</i> ₁	Relative displacements limit for bond stress-slip law

<i>s</i> ₂	Relative displacements limit for bond stress-slip law
<i>S</i> ₃	Relative displacements limit for bond stress-slip law
<i>t</i> _{bp}	Thickness of the base plate
t _{fc}	Column flange thickness
t _{wc}	Column web thickness
W	Distance between the anchor bolts in the direction of the width of the base plate
Xc	Neutral axis position
$X_{ m W}$	Portion of the effective web in compression
Zc	Compressive lever arm
Z.	Total lever arm
Zcol	Width of the column for out-of-plane bending
ZC0	Compressive lever arm from EC3-1-8
Zт	Tensile lever arm
Zw	Half of the total effective length of the web
Z	Average of the total lever arm

Latin upper cases

A_1	Area of the base plate concentrically bearing on the grout (or concrete)
A_2	Maximum area of the portion of the supporting surface that is geometrically
	similar to and concentric with the loaded area
$A_{ m eff}$	Effective bearing area
$A_{\rm s}$	Total cross sectional area of the bolts (threaded part)
$A_{ m v}$	Shear area of the column cross-section
$E_{\rm c}$	Concrete Young modulus
$E_{\rm s}$	Steel Young modulus
F	Applied force
$F_{\rm C}$	Reduced resistance of the T-stub in compression
$F_{\rm c,fc,u}$	Resistance of the column flange in compression
$F_{\rm c,Rd}$	Design resistance of the T-stub in compression
$F_{\mathrm{C,u}}$	Resistance of the column T-stub in compression depending on the considered
	behaviour type
$F_{\mathrm{C,u,f}}$	Resistance of the column flange T-stub in compression
$F_{\mathrm{C,u,w}}$	Resistance of the column web T-stub in compression
F_{\max}	Maximum reached force on the experimental tests
$F_{ m Rdu}$	Concentrated design resistance force
F_{T}	Reduced resistance of the T-stub in tension
F _{t,1}	Applied tensile force in the left side
F _{t,r}	Applied tensile force in the right side
$F_{\rm t,Rd}$	Anchor bolt design resistance
$F_{\mathrm{T,u}}$	Resistance of the column T-stub in tension
G	Shear modulus of the column

Ι	Moment of inertia	
Κ	Ratio between the distances amongst the hydrostatic axis and the compression	
	and tension meridian	
L	Column length	
$L_{ m b}$	Active anchor bolt length	
${L_{\mathrm{b}}}^{*}$	Prying limit	
Lbe	Effective embedment length of the anchor bolt	
$L_{ m bf}$	Effective exposed length of the anchor bolt	
$L_{ m jack}$	Distance between the load application point and the column base plate	
M_0	Bending moment corresponding to the yield point	
M_1	Bending moment at the top of the column	
$M_{ m bp}$	Bending moment per unit length acting on the base plate	
$M_{\mathrm{C,u}}$	Bending moment in the compressive side	
$M_{ m ip}$	In-plane bending moment	
$M_{ m j,pl}$	Plastic bending moment resistance	
$M_{ m j,Rd}$	Bending moment design resistance	
$M_{ m j,u}$	Ultimate bending moment resistance	
$M_{ m op,l}$	Out-of-plane bending moment in the left side	
$M_{ m op,r}$	Out-of-plane bending moment in the right side	
$M_{ m op,u,max}$	Maximum out-of-plane bending moment	
$M_{\mathrm{T,u}}$	Bending moment in the tensile side	
Ν	Applied axial force	
$N_{\mathrm{C,u}}$	Resistance of the connection in pure compression	
$N_{ m Mmax}$	Axial load value corresponding to the maximum bending moment	
$N_{ m moy}$	Average between the resistances in pure compression and pure tension	
$N_{\mathrm{T,u}}$	Resistance of the connection in pure tension	
Q	Prying force	
S_0	Initial cross-sectional area of the coupon test	
$S_{ m j,ini}$	Initial stiffness	
$S_{ m u}$	Cross-sectional area of the coupon test at failure	
$U_{ m b}$	Horizontal displacement at the bottom of the base plate	
$U_{ m t}$	Horizontal displacement at the top of the base plate	
$W_{\rm y,pl}$	Plastic modulus of the cross-section in the strong-axis direction	
W _{z,pl}	Plastic modulus of the cross-section in the weak-axis direction	
One els la		

Greek lower cases

α	Curve fitting coefficient; coefficient that takes into account the concrete		
	confinement		
αc	Ratio between the lever arm z_T and the average lever arm \underline{z}		
α _T	Ratio between the lever arm $z_{\rm C}$ and the average lever arm \underline{z}		

$eta_{ m j}$	Foundation joint material coefficient
γь	Relative concrete block rotation
γc	Partial safety factor for the concrete
γмо	Partial safety factor for resistance of cross-sections whatever the class is
γм1	Partial safety factor for resistance of members to instability assessed by member
	checks
γм2	Partial safety factor for resistance of cross-sections in tension to fracture
$\delta_{ m t1}$	Displacement corresponding to the tensile strength of concrete
З	Strain
Ec1	Strain corresponding to the maximum strength of the concrete
\mathcal{E}_{h}	Strain corresponding to the beginning of the yield plateau
$arepsilon_{ m m}$	Strain corresponding to the beginning of the strain hardening
<i>E</i> nom	Nominal strain
Eu	Ultimate strain
€у	Yield strain
θ_0	Rotation corresponding to the yield point
$ heta_{ m c}$	Rotation of the concrete block
$ heta_{ m j}$	Rotation of the connection
$ heta_{\mathrm{j,u}}$	Ultimate rotation of the connection
$ heta_{ ext{j,max,exp}}$	Maximum rotation of the connection from the experimental tests
$ heta_{ ext{j,max,num}}$	Maximum rotation of the connection from the numerical analysis
θ_{x}	Rotation at a distance <i>x</i>
σ	Normal stress
$\sigma_{ m c}$	Concrete compressive stress
$\sigma_{ m e}$	Concrete compressive stress that corresponds to yielding of a cantilever with
	length $e_x + m$
$\sigma_{\rm nom}$	Nominal stress
τ	Shear stress
$ au_{ m f}$	Ultimate shear stress
$ au_{ m max}$	Maximum shear stress
ξs	Elongation of the anchor bolts

Greek upper cases

	Δ_1	Vertical displacement read by the displacement transducer V_1
--	------------	---

- Δ_4 Vertical displacement read by the displacement transducer V₄
- $\Delta_{\rm C}$ Discriminant of the quadratic formula for biaxial bending moment resistance in dominant compression
- Δ_M Discriminant of the quadratic formula for biaxial bending moment resistance in dominant bending moment

- $\Delta_{\rm T}$ Discriminant of the quadratic formula for biaxial bending moment resistance in dominant tension
- Φ_c Resistance factor for bearing

Abbreviations

HR	High resistance
LVDT	Linear variable differential transformer
M0	In-plane bending moment
M90	Out-of-plane bending moment
M45	Bending moment at 45°
SPE	Specimen

Subscripts

45	Biaxial (45°)
ana	Analytical
b	Bolts
bp	Base plate
c	Column, concrete, compression
exp	Experimental
f	flange
ip	In-plane
1	Left side
max	Maximum
num	Numerical
ор	Out-of-plane
r	Right side
S	Steel
t	Tension
u	Ultimate
W	Web

1. INTRODUCTION

1.1. Scope of the thesis

During the last decades, steel structures have been widely adopted for its simplicity, quick erection and less environmental impact. Commonly used for the construction of industrial, commercial and habitational buildings, bridges and recently, wind towers, steel structures provide an excellent alternative to current reinforced concrete structures (Simões, 2014). With multiple benefits such as the high-quality production, the reduction on the self-weight, no need formwork and the use of a fully recyclable material enables to obtain cheaper solutions, which led steel structures gain a worldwide leading place. But the civil engineering world is in permanent evolution with the development of new materials and revolutionary construction techniques, and because of its importance, steel structures continue to be the focus amongst researchers.

Steel joints are structural elements which play an important role in the connection between different elements of the steel structure, such as beams, columns, foundations. For this reason, their behavior have a strong influence on the global behavior of the structure. Furthermore, design engineers of steel structures from today need to pay special attention to find adaptable and economical solutions, with easy fabrication, high resistance and deformation capacity (Kuhlmann *et al.* 2014), which increases the difficulty on the design of such elements as joints.

Column base plates are one of the most critical and influential elements on steel structures, since their efficiency and performance strongly affects the whole behavior of the structure (Stamatopoulos and Ermopolous, 2011). The main function of these type of connection is to transfer to the foundation the self-weight and the loads applied to the structure, representing a great influence on the stability and durability of the overall structure. However, comparing with other types of connections, such as beam-to-column or beam-to-beam, there is a lack of information on regulatory documents about column base plates. Within the existing studies carried out by other researchers, it is very common to find numerical or experimental programs mainly focused on strong axis bending, unlike the cases with weak axis bending or biaxial bending.

Thus, it is crucial to have a correct estimation of the mechanical properties of this type of connections, in the matter of strength, stiffness and rotation capacity, for cases with biaxial bending. For this reason, the main objective of this work is to develop an accurate model, allowing to predict the resistance of column bases under complex loading conditions.



Figure 1.1: Transmission of forces applied in a structure to the foundation

1.2. Objectives

Aiming to obtain an economical structural design, the main objective of this thesis is the development of an analytical model that allows to obtain the resistance of steel column base plates anchored to a concrete foundation under different loading conditions: axial force, uniaxial and biaxial bending moment. As this work has the final purpose to be consistent with Eurocode 3, the analytical model will be based on the existing component method presented in EN 1993-1-8.

To develop a simple and suitable approach, this work consists on an extended literature review about column base plates, preceded by the experimental, numerical and analytical studies. The numerical simulations are based on two experimental programs carried out at *Laboratoire de Génie Civil et Génie Mécanique de l'INSA de Rennes*, in order to compare and validate the obtained results with experimental data. The development of an analytical model to predict the behavior of the column base plates subjected to uni-axial and biaxial bending moment comes right after, with its validation against the existing numerical and experimental results. This research work allows to evaluate how accurate are the predictions made until now based on the component method.

1.3. Chapters organization

The different chapters are described below:

• Chapter 1 – Introduction: brief presentation of the scope of the thesis, as well as the main objectives, including the description of each chapter,

- Chapter 2 State of art: dedicated to the description of the developments and investigations previously carried out on column base plates, this chapter includes a regulatory review about the existing methodologies provided to evaluate the resistance of column base plates,
- Chapter 3 Experimental tests: characterization of the experimental testing program. Abbreviated description of the tests set-up and the chosen variable parameters. Analysis and discussion of the obtained results,
- Chapter 4 Numerical study: description of the numerical models created in ABAQUS, this chapter presents the numerical study and the obtained results by the numerical simulations based on the presented experimental tests,
- Chapter 5 Analytical models for resistance: development of an analytical model, based on the component method, to predict and evaluate the resistance of column base plates under axial force and uni-axial/biaxial bending moment,
- Chapter 6 Conclusions and recommendations: final considerations are made. Additionally, a few suggestions of future works are also given.

2. STATE OF ART

2.1. Introduction

Column base plates are one of the most important connections in steel structures, since they link the superstructure to the foundation. However, the gap of knowledge is evident on the existing literature. Comparing to other types of connections, the number of available numerical and experimental data is quite limited and mostly focused on strong axis bending moment. As steel structures are very sensitive on the redistribution of internal forces caused by the rotational stiffness of column bases (Wald, 2000), researchers felt an immediate need to find suitable design methods for this type of connections.

The present chapter contains a description of the main numerical and experimental works carried out, as well as a review and comparison of the existing methodologies available on different national design codes.

A particular attention is given to the component method presented in Eurocode 3. Although it only takes into account the in-plane bending moment, this method is nowadays widely used to predict the behavior of column base plates in terms of resistance and stiffness.

2.2. Historical background

In Europe, column base plates were the subject of first studies in 1971 carried by Delesques in France. At the time, the calculation was made based on elastic methods used for reinforced concrete structures. Seventeen years later, Lescouarc'h adopted the same model for the development of the methodology presented in Lescouarc'h (1988) for column base plates subjected to biaxial bending moment. Also, in 1987 Colson developed two and three-dimensional models to investigate the nonlinear bending flexibility of column base plates.

In 1986, David Thambiratnam analysed by means of an experimental program, the behavior of column base plates subjected to eccentric axial loads and bending moment. Later, Krishnamurthy and Thambiratnam (1990) made great advances studying the column base plates behavior. The same way in 1992, Astaneh and Nakashima studied the parameters which play a major influence on the behavior of several column base plates configurations.

In the United States, before the Northridge earthquake, the design of column base plates under bending moments was based on published works by Gaylord and Gaylord (1957 and 1972), Salmon *et al.* (1957), Blodgett (1966), Soifer (1966), McGuire (1968), Maitra (1978), DeWolf and Sarisley (1980), DeWolf (1982), Ballio and Mazzolani (1983), Thambiratnam and Paramasivam (1986) and AISC Design Guide No. 1 "Column Base Plates" (DeWolf and Ricker, 1990; Lee *et al.*, 2008a). However, the earthquake that occurred in 1994 allowed

researchers to conclude that the performance of column base plates did not fulfil the requirements, exhibiting considerable and irreversible damages. Thus, an urgent need to develop new methods arise to design column base plates, in order to achieve a satisfactory response in case of large lateral displacements.

2.3. Previous analytical, numerical and experimental studies

In general, the design methods are built to design the connections to resist to in-plane bending moment, neglecting the effect of out-of-plane bending moment. However, this non-consideration of the effects of bending along the weak axis can lead to serious risks on the stability of the structure and to a premature failure of the column base plates.

Along the past years, the uncertainty related to the hypothesis taken on the design of this type of connection led some researchers to study column base plates with more detail. The analytical and numerical works described below represent the best-known studies on column base plates under 3D loading that were developed all over the world. Also, test campaigns were carried out on column base plates in order to have a better understanding about the behavior of such connections subjected to different loading conditions. In some cases, experimental tests represented the starting point to the development and validation of new calculation procedures. Comparisons between the existing component method and experimental laboratory tests on column base plates are also done and presented in this section.

2.3.1 In-plane bending studies

Back in 1985, at University of Laval in Canada, Picard and Beaulieu (1985) performed an experimental investigation on 15 column base plates in order to evaluate their behavior when subjected to axial forces combined to in-plane bending moment and estimate the rigidity in function of the combination of loading.

Table 2.1 summarizes the geometry of each tested specimen. For the series of tests F, the parameter *L* corresponds to the distance between the location of the attachment of the load jack and the top of the base plate. For the series CF, e_N represents the eccentricity of the load applied to the steel column. The test set-up and the base plate configurations can be seen in Figure 2.1 and Figure 2.2.
	Tuble 2.1. Characteristics of the		
Specimen	Column cross sections and base plate dimensions	<i>L</i> (mm)	$e_{\rm N}$ (mm)
1F*		730	-
2F*		733	-
3F*	M 100 × 19	737	-
4CF†	$130 \times 140 \times 11 \text{ mm}$	-	149
5CF†	(2 anchor bolts)	-	162
6CF†		-	305
7CF†		-	305
8F*	W 150 × 27	692	-
9F*	$\frac{100}{200} \times \frac{20}{20}$ mm	689	-
10CF†	$190 \times 500 \times 29$ IIIII	-	305
11CF†	(4 anchor bolts)	-	305
12F*		692	-
13F*	$HSS 152,4 \times 152,4 \times 12,7$ 100 × 200 × 20 mm	699	-
14CF†	$190 \times 300 \times 29$ IIIII (A anahar halta)	-	438
15CF†	(4 anchor bons)	-	440

Table 2.1: Characteristics of the tests

(F) – flexion: specimen subjected to shear force and bending moment

† (CF) - compression and flexion: specimen subjected to axial force, shear and bending moment



Figure 2.1: Tests set-up (Picard and Beaulieu, 1985)

Figure 2.2: Base plate configurations

In presence of large eccentricities, Picard and Beaulieu (1985) proposed the following equation for the ultimate bending moment resistance:

$$M_{j,u} = 0.85 \alpha f_{ck} d_{eff} b_{bp} [0.5(h_{bp} - x_c)] + n A_s f_{ub} (0.5h_{bp} - e)$$
(2.1)

with

 α : coefficient that takes into account the concrete confinement,

 f_{ck} : compressive strength of concrete,

 $d_{\rm eff}$: depth of the rectangular stress block,

 $h_{\rm bp}$: base plate depth,

 b_{bp} : base plate width,

- *n* : number of anchor bolts in the tensile zone,
- $A_{\rm s}$: tensile stress area of an anchor bolt,
- $f_{\rm ub}$: ultimate tensile stress of anchor bolt,
- *e* : distance from the base plate edge to the axis of the anchor bolt.

For the tests in pure bending (series F), the results showed that for all cases, the measured ultimate bending moment was smaller than the nominal plastic moment capacity of the column cross section about the strong axis. For the three first cases -1F, 2F and 3F - tests stopped when the piston stroke reached its maximum capacity, and consequently, the anchor bolts did not reach failure. After the test, the observed rotation and deformation of the connection were very large, showing a considerable gap between the grout and the base plate. However, the deformation was not sufficient to break the anchor bolts. For the other tests, the rupture of the anchor bolts was reached with great damage on the grout layer.

Results of tests subjected to a combination of bending and axial force (series CF) revealed that the applied moment at the top of the column was significantly higher than comparing to the nominal plastic moment capacity of the column cross section when the tests stopped. Local buckling occurred in the column flange at the end of the analysis 4CF to 7CF, 10CF and 11CF. For specimens 14CF and 15CF with tubular columns, tests stopped when the applied moment was about 70% larger than the plastic bending moment capacity. After the analysis, the uplift in the tensile zone of the steel base plate was evident.

Being one of the first experimental and analytical studies in the field of column base plates in bending, and therefore, considered satisfactory for the time, given the results, it was concluded that when subjected to simple bending moment, the predicted ultimate moment resistance of the connection from the presented theoretical relationships was conservative. Also, cases with four anchor bolts showed higher values of resistance and stiffness comparing to the ones with two anchor bolts. A significant increase in the flexural stiffness was noticed in presence of a compressive force.

Ermopolous and Stamatopoulos (1996) developed an analytical procedure for the classification of column base connections in proportion with the level of concrete stresses under the base plate. This work aimed to develop a relation between the bending moment and the rotation for different types of connections (size and thickness of the base plate; size, length and location of the anchor bolts; material properties of the different elements; geometry of the concrete block; level of the applied axial load).

The design procedure for the calculation of the rotation, for a given combination M+N (bending moment and axial force), is characterized by an iterative calculation to obtain the neutral axis position and then, the stress distribution. Then, the rotation can be calculated, allowing to obtain any point from the moment-rotation curve. This calculation procedure is divided into three

models, according to the level of the concrete stresses that are developed under the base plate (see Figure 2.3).



Figure 2.3: Distribution of the stresses for Type models I, II and III

Curves obtained from the formulas proposed in this work were compared experimentally against the results from Astaneh *et al.* (1992), allowing to conclude the satisfactory accuracy of the analytical results (for further information, see Ermopolous and Stamatopoulos, 1996). However, the application of this method is difficult in practice.

Jaspart and Vandegans (1998) made great advances in the study of the behavior of column base plates subjected to bending moment in the strong axis direction. The objective was to develop a mechanical model to accurately calculate the moment-rotation relationship.

To validate the mechanical model, twelve experimental tests were performed at University of Liège. As represented in Figure 2.4, the tests set-up consisted in a steel column welded to a steel base plate, with two different values of thickness, by two or four anchor bolts. The geometry of specimens is summarized in Figure 2.5, Figure 2.6 and Table 2.2. A thin layer of grout was placed between the base plate and the concrete to guarantee a good contact. Specimens were loaded by a combination of a compressive axial force and bending moment in the strong axis.





Figure 2.5: Base plate configurations

Figure 2.6: M20 anchor bolt dimensions

Table 2.2 summarizes the tests and the	e respective parameters t	that were used.
--	---------------------------	-----------------

<u></u>	Table 2.2: Test designation	ns (Jaspart and Vandegans, 1998)	Normal famor (I-NI)
Specimen	Number of anchor bolts	Base plate thickness (mm)	Normal force (kin)
PC2.15.100	2	15	100
PC2.15.600	2	15	600
PC2.15.1000	2	15	1000
PC2.30.100	2	30	100
PC2.30.600	2	30	600
PC2.30.1000	2	30	1000
PC4.15.100	4	15	100
PC4.15.400	4	15	400
PC4.15.1000	4	15	1000
PC4.30.100	4	30	100
PC4.30.400	4	30	400
PC4.30.1000	4	30	1000

Observing Figure 2.7 with moment-rotation curves from test series PC2.15, it was concluded that the bending moment resistance increases as the compressive force increases. Higher load levels lead to less base plate deformations and consequently, delaying the failure of the anchor bolts. Concerning the initial stiffness, it changed abruptly when in the tensile zone, the base plate began to separate from the concrete block. Once again, the lower the applied compressive load, the more quickly this phenomena occurs. Figure 2.9 from tests PC2.30, showed the influence of a thicker base plate in the values of the ultimate resistance, which were clearly higher. The difference on the initial stiffness values for test PC2.30.100 was due to the poor conditions of the concrete when vibrated, having as a consequence a lower resistance. Figure 2.8 represent the moment-rotation curves obtained from tests PC4.15. The difference on the strength between test PC4.15.1000 and the others is noticeable. In this case, the axial compressive force was much higher than the others having as consequence a positive effect on the resistance and stability. Tests PC4.30 with the thickest base plate, as seen in Figure 2.10, showed the highest values for resistance and stiffness. However, in this case, the conclusion that the higher value of compressive force corresponds to higher values of resistance was not valid. In fact, for PC4.30.1000, yielding of the end section of the steel column and local column 80 80 M (kNm) M (kNm) 60 60 40 40 PC2.15.100 PC2.30.100 20 20 PC2.15.600 PC2.30.600 PC2.15.1000 θ_i (mrad) PC2.30.1000 θ_i (mrad) 0 0 20 40 60 80 20 40 60 80 100 0 0 Figure 2.7: Moment-rotation curves for PC2.15 tests Figure 2.9: Moment-rotation curves for PC2.30 tests 100 120 M (kNm) M (kNm) 100 80 80 60 60 PC4.30.100 40 PC4.30.400 40 PC4.30.1000 PC4.15.100 20 PC4.15.400 20 PC4.15.1000 θ_i (mrad) θ_i (mrad) 0 0 20 40 60 80 20 40 60 80 100 0 Figure 2.8: Moment-rotation curves for PC4.15 tests Figure 2.10: Moment-rotation curves for PC4.30 tests

flange buckling occurred due to the large value of the applied load, resulting in lower values of resistance comparing to PC4.30.400.

The ultimate resistances and the registered collapse modes were summarized in Table 2.3.

Table 2.3: Moment resistances and failure modes registered on the experimental tests (Jaspart and Vandegans, 1998)

$M_{\rm Ru,test}$ (kNm)	Failure mode
40	Failure of the anchor bolts
56	Failure of the anchor bolts
63	Crushing of the concrete
35	Failure of the anchor bolts
57	Failure of the anchor bolts
75	Failure of the anchor bolts
62	Yielding of the plate
68	Collapse of the plate and of the anchor bolts
97	Yielding of the plate
86	Tearing of the anchor bolts
117	Tearing of the anchor bolts
110	Yielding and local buckling of HEB 160
	M _{Ru,test} (kNm) 40 56 63 35 57 75 62 68 97 86 117 110

From the table above, it is concluded that the most rigid and resistant column base plates are the ones in which the applied compressive force is higher when failure occurs on the connection. The resistance can decrease with increasing value of the compression force when yielding/buckling develops in the column itself. Results also indicated that column base plates have a high semi-rigid behavior, even when dealing with classical pinned connections, which is positive for the design of building frames.

Within the scope of this experimental program, as aforementioned, a specific model for column base plates was developed by Jaspart and Vandegans (1998) in order to have a better understanding of the behavior of such connections in terms of individual components, interactions between elements and possible failure modes. Based on the component method, this method allowed to integrate numerous aspects of the behavior of column base plates. With support from the observations of the experimental tests, some assumptions listed below were taken into account on the analytical model:

- the complexity of the contact between the base plate and the concrete block, by extensional springs for the concrete under the plate,
- the bond-slip relationship between the anchor bolts and the concrete. As the connection starts to deform, the bond between these two elements is activated and quickly disappears, so it was considered that the anchor bolts in tension were free to elongate from the beginning of the loading,
- the consideration of an equivalent rigid plate is preserved, in order to transmit properly the compressive forces under the base plate,
- as it was observed on the experimental tests, yield lines can be seen on the extension of the end-plate on the compressive side,
- the yielding of the steel profile due to the combination of an axial force and a bending moment,
- the global deformation of the connection, which led to variations on contact zones and lever arms.



Figure 2.11 represents the model considering the observations made from the experimental results. In the model, springs replace the main components, simulating the deformation of the column at the bottom, subjected to tension or compression (1), the deformation of the base plate and the anchor bolt in tension (2), the compression of the concrete block located under the base plate (3) and, the yielding of the base plate in the compressive side of the connection.

To represent the behavior of the anchor bolts and the base plate in tension (2), strongly related to the thickness of the base plate and the location of the bolts, Annex J from Eurocode 3 was used. For the component base plate in compression (4), as the base plate deforms, a plastic hinge is locally formed. A spring in bending can model this phenomenon, considering an elastic-plastic law in the compressive part of the connection. Springs (1) with an elastic-plastic behavior allowed to perform the behavior of the steel column subjected to axial forces and bending moment. However, this model does not take into account the buckling of the flanges and web of the column that might develop.

The moment rotation curves obtained by the refined model and experimental tests are presented in Figure 2.12 and Figure 2.13.



The ultimate bending resistance is accurately estimated by the model. The moment-rotation curves are also satisfactory, particularly for PC4.30.400. The rotational stiffness of the connection PC4.15.400 is different from the test. This specimen is more sensitive to the base plate deformation in bending in the tensile and compressive parts, since this component is modelled in a very simplified way considering Annex J of ENV 1993, that neglect shear deformation of the base plate and use simplified formulations. Moreover, the modelling of the concrete in compression can probably be improved. The rotation capacity of the connections was not considered as well.

Stamatopoulos and Ermopoulos (2011) performed tests on eight specimens and developed finite element models to evaluate the moment-rotation curves of column base plates. The tested connections were subjected to strong axis bending moment with different levels of compressive force. The test set-up is presented in Figure 2.14 and the geometry of specimens in Table 2.4.



Figure 2.14: Tests set-up (Stamatopoulos and Ermopolous, 2011)

	Column	Base plate		Anchor ro	Concrete		
Specimen		$h_{\rm p} \times b_{\rm p} \times t_{\rm bp}$ (mm)	$f_{\rm yp}$ (N/mm ²)	Туре	$f_{\rm ub}~({ m N/mm^2})$	$A_{\rm s}~({\rm mm^2})$	$h_{\rm f} \times b_{\rm f} \times d_{\rm f}$ (mm)
SP1		240×140×16	416	M12	536,5	84,3	
SP2		240×140×12	320	M16	846,5	157	
SP3		240×140×16	277	M12	536,5	84,3	
SP4		240×140×12	429	M16	846,5	157	
SP5	HEB 120	240×140×16	277	M16	846,5	157	500×500×400
SP6		240×140×16	416	M16	846,5	157	
SP7		240×140×12	320	M12	536,5	84,3	
SP8		240×140×12	429	M12	536,5	84,3	

 Table 2.4: Geometry and material properties of the tests (Stamatopoulos and Ermopolous, 2011)

Experimental moment-rotation curves were compared with the numerical and analytical curves in order to validate the analytical formula proposed by the authors that relates the moment M with the rotation of the connection θ_j :

$$M = \alpha \, M_0 \frac{\theta_{\rm j}}{\theta_0 + \theta_{\rm j}} \tag{2.2}$$

with

 α : curve fitting coefficient,

 M_0, θ_0 : moment and rotation corresponding to the yield point.

Figure 2.15 of the moment-rotation curves for tests SP1, shows that the formula proposed by the authors is in good agreement with the ones obtained from the experiments and the FEM models for the different levels of axial force.



Figure 2.15: Comparison of experimental, numerical and analytical moment-rotation curves for tests SP1

Although the moment-rotation curves obtained by Equation (2.2) are quite close to the numerical and experimental curves, the model requires to determine *a priori* the value of the rotation θ_j . This parameter results from an iterative process presented in Ermopolous and Stamatopoulos (1996) in which the position of the neutral axis must be first calculated. Next, the parameters required to obtain the rotation for a given bending moment *M* and axial force *N*. The relation expressed by Equation (2.2) also depends on a coefficient α , obtained from an extrapolation procedure, which takes into account the connection configuration. Thus, despite the fact that the moment-rotation curve obtained through the equation is quite simple and direct, considering the non-linearity between these two parameters, the calculation process behind this relationship requires some analysis results to determine specific points and parameters of the curve, which is impractical and time-consuming.

A similar nonlinear model was proposed by Abdollahzadeh and Ghobadi (2013). The model consists in the prediction of moment-rotation curves M- θ of column base plates subjected to monotonic loading. This work was based on the study proposed by Stamatopoulos and Ermopolous (2011) described above. The existing data was used to compare and validate the model. The proposed equation for the prediction of the M- θ curve of column base plates under monotonic loading is:

$$M = M_{n} \left(1 + \left(\frac{M_{y}}{M_{n}} - 1 \right) \frac{\theta}{\theta_{y}} \right) \left(1 - e^{-\left(1 + 0.25 \frac{\theta}{\theta_{y}} \right) \frac{\theta}{\theta_{y}}} \right)$$
(2.3)

with

 $M_{\rm n}$: obtained by Equation (2.4),

 M_y , θ_y : moment and rotation corresponding to the yield point.

To evaluate the accuracy of the proposed formula, the results were directly compared to experimental and FEM results from Stamatopoulos and Ermopolous (2011).

Table 2.5: Calculated parameters for the model (Abdollahzadeh and Ghobadi, 2013)									
Specimen	M_{in} (kNm)	$M_{\rm p}$ (kNm)	$M_{\rm y}$ (kNm)	$\theta_{\rm v}(3\rm rad)$	$N(\mathbf{kN})$	$K_{ m i}$	$K_{ m p}$		
speemen	171j,u (111 (111)		ing (ki (iii)	oy (Sidd)	11 (M 1)	(kNm/rad)	(kNm/rad)		
SP1	17,5	14,0	14,9	3,3	0	5052,2	252,6		
SP2	22,8	18,3	19,4	3,6	99,3	6038,3	301,9		
SP3	21,6	17,4	18,4	3,8	198,5	5461,5	273,1		
SP4	29,3	23,5	24,9	4,4	297,8	6270,4	313,5		
SP5	23,2	18,5	19,6	3,2	0	6767,2	338,4		
SP6	31,7	25,5	26,9	4,2	99,3	7185,3	359,3		
SP7	18,7	15,0	15,9	3,5	198,5	5023,9	251,2		
SP8	21,2	16,9	17,9	3,9	297,8	5121,9	256,1		



Figure 2.16: Comparison of the moment-rotation curves for test SP1

From Figure 2.16, the comparison of the curves between the existing experimental, analytical and numerical data and the proposed model for test SP1 reveal a good agreement. However, despite the good correlation between the results, the application of this model, in a similar way to the previous one presented in Stamatopoulos and Ermopolous (2011), needs a previous calculation of parameters such as: the initial stiffness K_i , the intercept constant moment M_n and the strain hardening stiffness K_p , represented in Figure 2.17. The first parameter can be obtained from the general analytical model presented in EN 1993-1-8 for the estimation of stiffness properties. From Figure 2.17, M_n depends on the ultimate bending moment of the connection $M_{j,u}$, calculated by EN 1993-1-8. Then the authors present the value of M_n as being a portion of $M_{i,u}$ calculated using the methodology of Eurocode:

$$M_{\rm n} = 0.8025 M_{\rm j,u} \tag{2.4}$$

This calculation approach for the moment M_n is surrounded by some uncertainties, since its origin is not explained, representing a clear disadvantage for the model. Similarly, the estimation of the parameter K_p , since there is no exact applicable analytical method for its calculation, Abdollahzadeh and Ghobadi (2013) used values available in the literature predicted by previous researchers, considering the ratio K_p/K_i equal to 5%. Once again, the determination of this value requires a clearer and more objective verification.



Figure 2.17: Moment-rotation curve parameters

Márai *et al.* (2014) proposed to integrate the effect of the column web in compression in the evaluation of the resistance of column base plates subjected to in-plane bending moment and axial force. The geometry of the compressive effective area was assumed to be equal to the cross section increased by a strip of effective width c calculated according to EN 1993-1-8. Under this area, the stress distribution was taken as perfectly plastic. For the tensile and compressive sides, the resistances were assumed as:

$$F_{\text{tens,Rd}} = F_{\text{t,Rd,i}} \tag{2.5}$$

$$F_{\text{comp,Rd}} = \min(F_{c,Rd}; F_{c,fc,Rd})$$
(2.6)

with

 $F_{t,Rd,i}$: anchor bolts resistance, $F_{c,Rd}$: concrete resistance, $F_{c,fc,Rd}$: column flange and web in compression resistance.

The calculation procedure consists in a sequence in which the equilibrium of the vertical forces is guaranteed. In each step of this sequence, the position of the neutral axis and consequently the compressive area changes, stopping when the activated part of the effective concrete area and the activated bolt-rows positions intersect each other or when the concrete resistance value is higher than the resistance of the flange and web in compression component (Márai *et al.*, 2014). Thus, the proposed formula for the connection resistance is:

$$M_{j,Rd} = F_{comp,Rd} \left(y_{s,conc} - y_{G,full} \right) + \sum F_{t,Rd,i} \left(h_i - y_{G,full} \right)$$
(2.7)

The parameters $y_{s,conc}$, $y_{G,full}$ and h_i can be obtained from Figure 2.18.



Figure 2.18: Assumed effective areas and equilibrium of forces for the design resistance calculation (Márai et al., 2014)

The M-N interaction curves obtained were compared against the Component Method, for a stiffened I-profile connection with two inner rows and one outer row of anchor bolts (see Figure 2.18). The resistance of the connection is thus significantly increased in presence of compressive force. However this procedure is iterative and the results were not compared against those from experimental tests.



Figure 2.19: M-N interaction diagram resulting from the model of Márai et al. (2014)

2.3.2 Out-of-plane bending studies

Concerning the study of column bases subjected to out-of-plane bending moment, Lee *et al.* (2008a) conducted first numerical and experimental studies.

The study began with an extensive numerical evaluation. For that, FE models were created varying the base plate thickness, anchor bolt size and grout compressive strength. The obtained numerical results were compared with the Drake and Elkin method, exposing its main weaknesses. For the parametric study, two anchoring configurations for column base plates were studied to compare the results with the predictions obtained from Drake and Elkin design method. The effects of shear force and bending moment were taken into account for the study of the connections. The main geometric characteristics can be found in Figure 2.20 and Table 2.6.



Figure 2.20: Base plate configurations assumed for the parametric study (units are in inches)

Table 2.6: Geometry of the connection elements							
Column	Base plate Anchor bolts						Grout
<i>L</i> (mm)	$h_{ m p} imes b_{ m p} imes t_{ m bp}$ (mm)	f_{yp} (N/mm ²)	φ (mm)	L _b (mm)	f_{yb} (N/mm ²)	f _{ub} (N/mm ²)	\dot{f}_{c} (N/mm ²)
2032	508×508×57	400	28	813	896	1034	41

The Drake and Elkin method was used to estimate the base plate thickness and the anchor bolts diameter. For that, the resultant bearing force R_u and the total tensile strength of the anchor bolts T_u must be calculated first. As it can be seen in Figure 2.21, a uniform distribution of the bearing stress block q_i located under the base plate is assumed. Then, using the equilibrium of forces, R_u and T_u can be effortlessly calculated.



Figure 2.21: Description of loads and parameters used in the Drake and Elkin method

An extensive parametric study was carried out for both four and six-bolt configurations. The main goal was to evaluate the impact of the base plate thickness, anchor bolts stiffness and grout compressive strength on the overall behavior of the connections. It was concluded that lower values for the strength and higher rotation capacity were observed for the most flexible base plate due to the early yielding of the base plate. The influence of anchor bolts stiffness's was also studied and results concluded that the initial rotational stiffness is strongly related to this parameter. For less stiff anchor bolts, larger base plate rotations were registered, with significant curvature on the compressive side. Also from the numerical results it was proved that although the Drake and Elkin method is highly sensitive to the grout compressive strength f_c , this parameter had no big influence on the variation of the resultant bearing force, the latter being more dependent on parameters such as the base plate thickness and anchor bolt stiffness. This study allowed to conclude that the assumption of bearing stress distribution as in Figure 2.21 is not fully adequate. Also, the proposed method do not take into account the number/position of the anchor bolts.

In the scope of this work, with the aim of enriching the comparison and evaluation of the results obtained from the Drake and Elkin method and the validity of the numerical parametric study, a test campaign with four exposed-type column base plates was carried out at University of Michigan. The influence of parameters such as the number of anchor bolts, relative strength between the base plate and the anchor bolts, and different filler metal and welding detail was also investigated. The mechanical and geometrical characteristics of specimens tested and calculated numerically are similar (see Figure 2.20 and Table 2.6). Table 2.7 summarizes the additional characteristics of each tested specimen.

Table 2.7: Tests characteristics (Lee, Goel and Stojadinovic, 2008b)								
	6-bolt specin	nen	4-bolt specin	nen				
	SP 6-1	SP 6-2	SP 4-1	SP 4-2				
Number of anchor bolt	6	6	4	4				
Base plate thickness (mm)	57	57	57	57				
Weld yield strength (N/mm ²)	440	475	440	475				
Weld tensile strength (N/mm ²)	525	593	525	593				



A comparison of the numerical and experimental lateral force-drift curves is depicted in Figure

2.23 and Figure 2.24. Results show better approximation for the six-bolt configuration. From Figure 2.25 and Figure 2.26 it is possible to verify that the higher stiffness of the anchor bolts from four-bolt tests led to higher values of tensile bolt forces.



By analyzing the obtained results, it was concluded that the Drake and Elkin method is not suitable to predict the behavior and performance of column base plates when subjected to weak axis bending moment. Besides this, from the observation of the deformed shape of base plates in tension, Lee *et al.* (2008b) could prove that the location and number of anchor bolts have influence on the yielding pattern of the base plate in tension and the bearing stress distribution in the compressive side. The investigation of the influence of filler metals and welding details allowed to verify that more ductile welding detail should be adopted to resist to high stress concentration and improve the connection performance under seismic conditions.

2.3.3 Biaxial bending studies

In this sub-chapter, the main works whose focus was the study of the behavior of connections subjected to biaxial bending and axial forces are presented.

In 2014, at Brno University of Technology, an experimental program on column base plates subjected to compressive force and uniaxial/biaxial bending was performed by Bajer *et al.* (2014). The tested column base plates were fixed to the grout layer and reinforced concrete foundation by means of four anchor bolts. Additionally, a shear lug was welded to the bottom of the base plate. An axial load of 400 kN was firstly applied and then, an horizontal load was increased until failure, creating uniaxial (Joints 1 and 2) or biaxial (Joints 3 and 4) bending moments (see Figure 2.27). The four specimens have the same geometrical characteristics (see Table 2.8).





Table 2.8: Adopted geometry and materials for tests						
Calumn	HEB 240					
Column	L = 2020 mm					
Base plate dimensions (mm)	440×330×20					
Steel grade (column, base plate)	S235					
Concrete dimensions (mm)	1500×1000×400					
Concrete grade	C16/20					
Anchor bolts	M20 8.8					
Channellan a	IPE 100					
Shear lug	L = 100 mm					
w*	160 mm					
p^*	340 mm					

*w, p – distance between the centre line of the anchor bolts in the yy' and zz' axis direction, respectively.

Failure was due to anchor bolts rupture in tension and yielding of the base plate in bending. Cracks were also observed on the concrete foundation but were not critical for the whole structure. The moment-rotation curves are plotted in Figure 2.28 and Figure 2.29. In addition the curve calculated with Eurocode 3 for strong axis bending moment is added.



The Eurocode approach overestimates the stiffness of column bases and underestimate the resistance in presence of a strong axis bending moment. For strong axis bending moment, the results obtained with the two specimens with the same geometry are quite close. In presence of biaxial bending moment, the rotation capacity of the two specimens is quite different mainly due to the material variability. The mechanical properties (rotational stiffness and resistance) obtained in presence of biaxial bending moment are lower than that in presence of strong axis bending moment mainly due to the modifications of the lever arm of the connection. However the type of failure mode was not affected by this orientation. By the end, the mechanical properties of the base plates are not provided, and this constitute a difficulty for a complete calibration of numerical models with these experimental tests.

At University of Porto, a numerical study was carried out by Amaral (2014), based on the experimental results presented by Bajer *et al.* (2014). The main goal of this work was to evaluate the structural response of column base plates subjected to a combination of axial compressive force and in-plane and biaxial bending and develop an analytical model based on the existing

methodology presented in EN 1993-1-8 capable to predict the moment design resistance and initial stiffness.

From the comparison of the results obtained numerically and experimentally, the approximation was quite satisfactory for both in-plane and biaxial bending, with ratios between 0,92 and 0,93 for the moment capacity and 1,42 to 1,27 for the initial stiffness. Small discrepancies were explained by the chosen material properties and the type of interactions assumed in the numerical model between the elements. As in the experiments, the failure of the models was governed by the yielding of the base plate and the rupture of anchor bolts in tension.

A model based on the component method was proposed to evaluate the bending resistance in presence of axial force and biaxial bending moment. The components used for the evaluation of the in-plane bending resistance are considered, only the effective lengths are modified to be cinematically consistent. Amaral (2014) proposed the following effective lengths in Table 2.9, based in the ones from the case of in-plane bending moment.





	Yield pattern	Circular pattern - $l_{cp,i}$	Non circular pattern - $l_{np,i}$
	*	$l = 2\pi m$	$l = 2\pi m$
Circular patterns		$r_{\rm cp,1} = 2\pi m_{\rm X}$	$r_{\rm np,1} = 2\pi m_{\rm X}$
		$l_{\rm cp,3} = \pi m_x + 2e$	$l_{\rm np,3} = \pi m_x + 4e$

 Table 2.9: Effective length calculation of a T-stub for the case of a weak axis bending moment (Amaral, 2014)



with

 $l_{cp,i}$: effective length for circular pattern *i*,

 $l_{np,i}$: effective length for non-circular pattern *i*,

 m_x : distance from the centre line of the anchor bolt to the edge of the weld, in the direction perpendicular to the width *b* of the base plate,

 e_x : distance from the centre line of the anchor bolt to the edge of the base plate, in the direction perpendicular to the width *b* of the base plate,

e: distance from the centre line of the anchor bolt to the edge of the base plate, in the direction parallel to the width b of the base plate.

The effective length is then chosen as the smallest value of the effective lengths of each possible mechanism for the different available failure modes:

- for Mode 1: $l_{eff,1} = \min(l_{cp,1}; l_{cp,3}; l_{cp,4}; l_{cp,6})$
- for Mode 2: $l_{eff,2} = \min(l_{cp,4}; l_{cp,6})$
- for Mode 1-2: $l_{\text{eff},1-2} = \min(l_{\text{np},1}; l_{\text{np},3}; l_{\text{np},4}; l_{\text{np},6})$

It is noted that from the table above, some effective lengths are assumed to correspond to circular and non-circular patterns whereas it is not the case. For example, the first mechanism is only associate to circular pattern.

For each pattern (circular and non-circular) the effective length can be obtained by a single expression. Then, the design resistance of the T-stub and the stiffness are calculated according to EN 1993-1-8.

For the base plate in bending and concrete in compression components, the method assumes that the stresses are transmitted to the foundation by the footprint of the column. However, the bearing area considered for in-plane bending must be adapted for the case of out-of-plane bending. For the resistance of the component, it can be easily defined by the effective compressive area A_{eff} and the concrete strength f_{jd} .

$$F_{\rm c,pl,Rd} = f_{\rm jd} \cdot A_{\rm eff} \tag{2.8}$$

with

 f_{jd} : bearing strength,

 $A_{\rm eff}$: effective bearing area.

To define the effective area, the location of the neutral axis for the case where the tensile part develops its full resistance needs to be calculated first. Thus, the validity of this procedure is limited to the cases where the failure is governed by the components in tension. The assumptions made for this model are listed below:

- 1. anchor bolts row develop its full resistance,
- 2. the effective area around the web is not taken into account for the compressive resistance,
- 3. the area under compression cannot cross the zz axis of the connection.

The effective area A_{eff} is then obtained by the equilibrium of the axial forces represented in Figure 2.30.

$$A_{\rm eff} = \frac{F_{\rm T,Rd} - N_{\rm Ed}}{f_{\rm jd}}$$
(2.9)

To determine the effective area, the dimensions b_{eff} and l_{eff} can be obtained from the following equations:

$$b_{\rm eff} = b_{\rm eff,l} + b_{\rm eff,r} = 2\left(t_{\rm f} + c + \min\left\{c; \frac{h_{\rm bp} - h_{\rm c}}{2}\right\}\right)$$
 (2.10)

$$l_{\rm eff} = \min \begin{cases} A_{\rm eff}/b_{\rm eff} \\ b_{\rm c}/_2 + c \\ b_{\rm bp}/2 \end{cases}$$
(2.11)

The lever arm z_c is given by:

$$z_{\rm c} = \min\left(\frac{b_{\rm c}}{2} + c; \frac{b_{\rm bp}}{2}\right) - \frac{l_{\rm eff}}{2}$$
 (2.12)

with

 $b_{\rm c}$, $h_{\rm c}$ and $t_{\rm f}$: width, length and thickness of the column flange, $b_{\rm bp}$: width of the base plate.

The stiffness coefficient is obtained as follows:

$$k_{\rm c} = k_{13} = \frac{E_{\rm c}\sqrt{b_{\rm eff,s}l_{\rm eff,s}}}{1,275E}$$
(2.13)

with

$$b_{\rm eff,s} = t_{\rm fc} + 2,5t$$
$$l_{\rm eff,s} = b_{\rm fc} + 2,5t$$

For the column flange in compression, the resistance is calculated considering that the resistance in pure bending develops (see Figure 2.31).



Figure 2.31: Equilibrium of forces for the column flange in compression component (Amaral, 2014)

The design resistance is then:

$$F_{\rm c,fc,Rd} = \frac{M_{\rm c,Rd}}{b_{\rm c}/2}$$
(2.14)

with

 $M_{c,Rd}$: design moment capacity of the column cross section, $W_{z,Pl}$: plastic modulus of the cross-section in the weak-axis.

After calculating all the necessary design resistances, the assembly of the components is done in order to obtain the design moment resistance. For that, the applied forces, the eccentricity, the resistance of both compressive and tensile parts and the lever arms z, z_c and z_t are required (see Figure 4.9 in Amaral, 2014).

An elliptical interaction curve is proposed for the resistance in presence of biaxial bending moment:



Figure 2.32: Determination of the design moment resistance $M_{j,u}$

The comparisons between the proposed analytical model and numerical simulations show that the proposed approach is conservative. The author explained that this fact could come from the conservative parameters provided by the Component Method. Although the discrepancies of the results, the project represented a major step on the study of the behavior of column base plates, being one of the first research works on the development of a new model for the calculation of the resistance and initial stiffness when subjected to biaxial bending moment.

One of the great advantages of this model, which was compared to a numerical and experimental database, is that it is based on the methodology of the Eurocode, which makes its comprehension and implementation easier for engineers used to European standard documentation. However, after a detailed evaluation of the model, it was concluded that it presents some limitations, which seems to be pertinent to improve in models developed and presented from now on. Regarding the connection resistance in compression, this model neglects the additional resistance provided by area of the concrete surrounding the column web.

Fasaee *et al.* (2018) proposed an analytical model to characterize the resistance of base plates subjected to weak axis and biaxial bending moments. The authors developed a numerical model validated by comparisons to tests of Gomez *et al.* (2010) performed on column bases plates subjected to in-plane bending moment and compressive force. The studied configuration are described in Figure 2.33 and Table 2.10.



Figure 2.33: Base plate configuration

|--|

Test Colu size ^t	Column	Plate size	Base plate material		Rod material ^a		Axial load M_{test}	$M_{\rm FEM}$		
	size ^b	$b_{ m bp}, h_{ m bp}, t_{ m bp}$ (mm)	f _{y,bp} (MPa)	f _{u,bp} (MPa)	f _{y,b} (MPa)	f _{u,b} (MPa)	<i>N</i> (kN)	(kNm)	(kNm)	IM FEM/IM test
1	W200×71	356×356×25	278	473	786	1010	0	123	125	1,02
2	W200×71	356×356×25	278	473	786	1010	0	123	125	1,02
4	W200×71	356×356×38	255	468	335	492	411	127	121	0,95
5	W200×71	356×356×25	278	473	786	1010	411	167	169	1,01
6	W200×71	356×356×51	265	497	786	1010	411	187	200	1,07
7	W200×71	356×356×25	278	473	786	1010	690	196	197	1,01
Mean										1,01
COV	COV (coefficient of variation = standard deviation/mean)									0,04

 $b_{\rm bp}, h_{\rm bp}, t_{\rm bp}$ = width, height and thickness of the base plate;

 f_{y} , f_{u} = yield strength and ultimate strength;

 $M_{\text{test}}, M_{\text{FEM}}$ = measured and calculated moment capacities, respectively.

^a All anchor rods were 19 mm in diameter and placed at 38 mm from the edge of the plate.

^b The column was fabricated from steel A992.

Concrete and grout compressive strengths were, respectively, 26,6 MPa and 58,3 MPa. After validating the numerical models, the study of the behavior under biaxial bending for models corresponding to Tests No. 1 (including Tests No.5 and 7), 4 and 6 was performed. An horizontal displacement was applied at the top of the column with different inclinations: 0° for strong axis bending, 30°, 45° and 60° for biaxial bending, and 90° for weak axis bending, as represented in Figure 2.34.



Figure 2.34: Assumed inclination angles (Fasaee et al., 2018)

The numerical results were normalized with respect to the uniaxial moment resistances in interaction curves (see Figure 2.35) and highlight an elliptical shape that was proposed in the interaction relationship of Equation (2.16).

$$\left(\frac{M_{\rm x}}{M_{\rm ux}}\right)^2 + \left(\frac{M_{\rm y}}{M_{\rm uy}}\right)^2 = 1 \tag{2.16}$$

with

 M_x and M_y : applied in-plane and out-of-plane bending moments, M_{ux} and M_{uy} : in-plane and out-of-plane bending moment resistances.



Figure 2.35: Proposed elliptical curve (Fasaee et al., 2018)

For the in-plane bending resistance Fasaee *et al.* (2018) used the approaches of AISC steel design guide one that assume failure on base plate in bending or anchor bolts in tension (see Figure 2.36).



Figure 2.36: Failure mechanisms for the design bending resistance under major axis bending moment (Fasaee et al., 2018)

To calculate the moment capacity of I-shaped column base plates subjected to minor (weak) axis bending moment, the proposed analytical model assumed the calculation procedure of mechanism 1 in which base plates behave as flexible. For this, three possible yield line patterns (Figure 2.37) for the tensile side were assumed.



Figure 2.37: Yield patterns (Fasaee *et al.*, 2018)

In the compression area, a yield line parallel to the column web was assumed. Fasaee *et al.* (2018) found out that its position could not be fixed as in the Drake and Elkin method, which assumes the yield line at 40% of flange width from the centre line. In reality, they concluded that it was strongly depended on the value of the applied axial load. As example, for Test No. 1, the yield line was practically under the edge of the flange (no axial load). Increasing the applied axial load, the yield line moved towards the column web. This fact was explained by the significant change on the stiffness of both tensile and compressive sides as the load level increased. The accuracy of the model proposed by Fasaee *et al.* (2018) was quite satisfactory when compared to experimental and numerical analysis.

2.3.4 Rotational stiffness and rotation capacity

Wald *et al.* (2000) proposed a design model to predict the stiffness of column base plates that was compared to tests from COBADAT database and tests made in Czech Technical University. The model consisted in the calculation of each individual component stiffness k_{15} and k_{16} (base plate in bending under tension; anchor bolts in tension) with the formulas provided in EN 1993-1-8. An additional stiffness coefficient for concrete in compression was proposed as being equal to:

$$k_{10} = 0.5 \ \frac{l_{\rm eff} b_{\rm eff} E_{\rm c}}{h_{\rm conc} E_{\rm s}} \tag{2.17}$$

with

 $b_{\rm eff}$, $l_{\rm eff}$: base plate effective dimensions (simplified rectangular area for the considered effective rigid plate),

 $E_{\rm s}, E_{\rm c}$: steel and concrete elasticity modulus,

 h_{conc} : depth of the concrete block.

The rotational stiffness of the connection was proposed as:

$$S_{\rm j} = \frac{Ez^2}{\mu \sum_{\rm i} \frac{1}{k_{\rm i}}} \tag{2.18}$$

with

 k_j : stiffness coefficient for component *i*,

z: lever arm as in Figure 2.53,

$$\mu = \frac{S_{j,\text{ini}}}{S_j} = \left(\kappa \cdot \frac{M_{\text{Ed}}}{M_{\text{Rd}}}\right)^{\zeta} \ge 1 \text{ (for } \kappa \text{ and } \zeta \text{ see Table 2.11).}$$

To calculate the values of the parameters needed for the stiffness calculation, three different failures modes (patterns) were considered as follows:

- pattern 1 when the column base plate is subjected to a low axial force and the concrete bearing stress f_j is never reached. The rupture can come from the yielding of the anchor bolts or plastic mechanisms in the base plate,
- pattern 2 when the column base plate is subjected to medium axial force, the concrete bearing stress f_i and the effective stress on the tension part f_t are reached,
- pattern 3 when the column base plate is subjected to high axial force and the concrete rupture occurs.

Pattern	Stiffness coefficients k_i to be considered	κ	ξ	Z
Pattern 1 Low axial force	k_{15}, k_{16}, k_{10}	1,1	6	$\frac{M_{\mathrm{Ed}}+0.5pN}{N+2A_{\mathrm{s}}f_{\mathrm{y}}}$; $M_{\mathrm{Ed}} \ge \frac{M_{\mathrm{Rd}}}{\kappa}$
Pattern 2 Medium axial force	k_{15}, k_{16}, k_{10}	Linear	transition	between patterns 1 and 3
Pattern 3 High axial force	k_{10}	1,5	8	$\frac{h_{\rm bp}}{\sqrt{3}}$

Table 2.11: Parameters for the calculation of the rotational stiffness (Wald, Sokol and Steenhuis, 2000)

Kavoura and Genvturk (2018) proposed a methodology for the evaluation of the rotational stiffness of column base plate in which the anchor bolts are placed inside column flanges. The type of column bases are generally classified as "pinned" in practice but can provide a non-negligible stiffness that is suitable for the global stability of the structure. Contrary to EN 1993-1-8, the center of compression is not positioned behind the column flange (see Figure 2.38-a) but is based on a uniformly distributed concrete bearing pressure that reduce the lever arm and is more realistic than Eurocode 3 (see Figure 2.38-b). These modifications lead to a significant reduction on the rotational stiffness (see Table 2.12) but the accuracy of the model is limited.



Figure 2.38: Lever arm according to the EN 1993-1-8 (a) and to the modified model (b) (Kavoura and Genvturk, 2018)

Specimen	Experime rotational (kN-m/rae	ental stiffness d)	ll Eurocode 3 ffness rotational stiffness, Si ini (kN-m/rad)		Modified Eurocode 3 rotational stiffness, S _{j,ini_mod}		Difference of Eurocode 3 S _{j,ini} from experiment		Difference of modified Eurocode 3 S _{j,ini_mod} from	
	, D	, 		, D II	(kN-m/r	ad)	(%)	D 11	experim	ent (%)
	Push	Pull	Push	Pull	Push	Pull	Push	Pull	Push	Pull
S01	4700	13200	39100	31200	5500	7500	732	136	17	-43
S02	8300	10900	20000	36800	7200	9100	133	238	-13	-17
S03	9300	16200	51700	26800	6800	7400	456	65	-27	-54
S04	5800	9800	58700	26900	8800	7800	912	174	52	-20
S05	14200	10500	58700	15700	5000	3900	92	-50	-65	-63
S06	13400	20800	16300	14400	4400	4400	22	-31	-67	-79
S07	38200	78800	336800	170600	30000	52200	782	116	-21	-34
S08	113700	115700	156800	160200	47900	38800	34	17	-58	-66
S09	60900	182500	156800	160200	47900	38800	151	-26	-21	-79
S10	163600	154300	170800	176700	54100	44200	4	15	-67	-71
S11	2500	5500	50700	15900	6300	4300	1928	136	152	-22
AVERAGE	Ξ						477	91	51	50

 Table 2.12: Comparison of the stiffness results between the Eurocode 3 method and the proposed modified model (Kavoura and Genvturk, 2018)

Latour and Rizzano (2013) proposed an analytical model to estimate the rotation capacity of column base plates. This model was confronted to three tests. The tests set-up consisted in a vertical column fixed to a concrete foundation by means of a steel base plate. Tests were subjected to monotonic loading. More detailed information about the specimens is shown in Table 2.13. The analytical model consisted in the definition of four possible collapse mechanisms presented in Figure 2.39 that corresponds to the failure of the two main components:

- base plate in bending with prying effect (mechanism 1) or without prying effect (mechanism 1^{*}),
- anchor bolt failure in tension with prying effect (mechanism 2) or without prying effect (mechanism 3).

The rotation that develop locally at the base plate were calculated according to the previous study of Piluso *et al.* (2001).

Table 2.13: Tests geometries and materials						
Test	Concrete	Base plate	Steel grade	Concrete	Dimensions and grade	
1050	dimensions (mm)	thickness (mm)	(column, base plate)	grade	of the anchor bolts	
HE240B-15		15			MO O 9 9	
HE160A-15	$1400\times 600\times 600$	15	S275	C20/25	M20.8.8 I = 400 mm	
HE240B-25		25			L = +00 mm	

Table 2.14 summarizes the comparison of the experimental results against analytical values for the rotation capacity and the results are in quite good agreement.

	Table 2.14: Comparison between experimental and analytical results (Latour and Rizzano, 2013)						
Test	θ_{p1} (rad)	θ_{p2} (rad)	$\delta_{ m pb}$ (mm)	$\phi_{p,mod}$ (mrad)	$\phi_{p,exp}$ (mrad)	$\phi_{p,mod}/\phi_{p,exp}$	
HE240B-15	0,5519	0,5519	0	115	131,6	0,874	
HE160A-15	0,466	0	23,3	173,5	183,7	0,944	
HE240B-25	0,466	0	23,3	168,9	178,5	0,946	
Average						0,92	
Standard Deviation 0,04							





Figure 2.39: Assumed kinematic collapse mechanisms for base plate connections (Latour and Rizzano, 2013)

2.4. Design methods

The calculation procedure to predict the resistance and stiffness of column base plates nowadays available for engineers in EN 1993-1-8 resulted from the evolution and improvement of several researchers that in the past, throughout their work, provided numerical and experimental data, proposing also analytical models based on the most diverse assumptions. This sub-chapter present the previous elastic design methods used in Europe and US and the current design procedure of EN 1993-1-8.

2.4.1 Elastic design methods

Lescouarc'h (1988) proposed a calculation procedure for the evaluation of the resistance of column base plates subjected to the combination of axial force and biaxial bending. The model was developed for rigid column base plates configuration with four anchor bolts, subjected to a combination of axial force N, bending moment in both directions M_y and M_z and shear forces V_y and V_z as in Figure 2.40.



Figure 2.40: Column base plate configuration (Lescouarc'h, 1988)

The model consider the assumptions that the base plate is rigid and thus the distribution of contact pressure with concrete is linear and can spread potentially over the complete area of the base plate. The system can be analyzed as a concrete cross section area, considering the compatibility deformations of the tensile and compressive area. The stress distribution and the position of the neutral axis can be calculated with the equilibrium equation. This model is not realistic even in presence of thick flange as the contact pressure generally develop around the flange and web of the column base plate. In addition, the model of Lescouarc'h completely neglect the existence of prying effect and the impact of the base plate rigidity on the redistribution of forces on the connection. A similar method is used in AISC design guide.



Figure 2.41: Forces distribution for $N+M_y+M_z$ (Lescouarc'h, 1988)

2.4.2 Design method EN 1993-1-8: Component method

2.4.2.1.Background review

Eurocode introduce the concept of plasticity in the evaluation of the resistance of elements and this concept can be applied to column base plate in presence of an axial force and in-plane bending moment.

Some disadvantages that have been pointed out are the focus of the work here developed:

- no consideration of the contribution of the effective area under the column web for the global resistance of the connection,
- different calculation procedures for different loading situations: pure compression and combination of axial force and bending moment, leading to a discontinuity on the M+N interaction diagram,
- impossibility to calculate the bending resistance when the axial compressive force is larger than the compression resistance provided by the effective areas under the column flange,
- absence of guidelines for the calculation of column base plates subjected to out-ofplane/biaxial bending moments.

Table 2.15 summarizes the different available design models that were developed based on the component method for the determination of the column base resistances.

Model type	Mechanical	Analytical				
Authors	(Guisse <i>et al.</i> , 1996)	(Wald, 1995)	(Wald <i>et al</i> ., 1996)	(Wald <i>et al</i> ., 1996)	(Guisse <i>et al.</i> , 1996)	(Steenhuis, 1998)
Model accuracy	Sophisticated	Complex	Simplified stiffness	Simplified strength	Simplified strength	Simplified strength and stiffness
Effective	Cross section shape	Rectangular sl	nape	Cross section shape	Equivalent rectangular shape	Web neglected
area					c	

 Table 2.15: Comparison between the different existing models for the calculation of the design resistance and stiffness (Márai et al., 2014)

From the summarized information exposed in Table 2.15, it can be concluded that a full range of calculation procedures is available on the literature, from very sophisticated methods that take into account the full non-linear behavior to simple resistance and stiffness calculation models. The simplified model developed by Wald et al. (1996) for the resistance was based on a geometry that follows the shape of the cross section, including this way the additional resistance provided by the area under the column web. The simplified model developed by Guisse *et al.* (1996) assumed an equivalent rectangular shape with a perfectly plastic stress distribution in the compression side. The definition of the equivalent shape depended on the applied bending moment, axial force and the tensile resistance of the anchor bolts. The last model do not take into account the effective area under the column web for the calculation of the resistance and stiffness of column base plates subjected to a combination of axial force and bending moment. In this model, springs are used to model the behavior of both tensile and compressive parts: two springs on the compressive side under the column flanges and two other springs on the tensile side in the location of the anchor bolts. The fact that it does not consider the effective area around the column web leads to conservative solutions for the case of large compressive loads acting with bending moment, since for a certain level it improves the resistance and the stability of the connections. Wald et al. (2000) clearly highlights the effect of the column web component (see Figure 2.42) for different thicknesses of the base plate.





2.4.2.2. Component method for column base plates under strong axis bending

The design procedure to calculate the resistance and stiffness of column base plates by the Component method is presented next for a combination of axial force and in-plane bending moment. This model is valid for exposed column base plates, as represented in Figure 2.43, composed by the following main elements:

- stiffened/unstiffened steel column (HEA, HEB, IPE steel profiles),
- steel base plate welded to the column,
- grout layer,
- anchor bolts,
- concrete foundation.



Figure 2.43: Main elements composing an exposed column base plate (Amaral, 2014)

The calculation procedure consist in the evaluation of the complex non-linear joint response through the subdivision into basic joint components, determining its individual resistance and stiffness in order to obtain the overall structural behavior. The procedure steps are listed below:

- 1. identification of the basic components,
- 2. characterization of the mechanical properties of each individual component (resistance and stiffness),
- 3. assembly of the individual component properties to obtain the overall properties of the connection,
- 4. classification of the connection in terms of resistance and stiffness.

Figure 2.44 summarizes the main individual components for this type of connections, according to the following list:

- base plate in bending,
- anchor bolts under tension,
- concrete in compression including grout and base plate under bending,
- column flange and web in compression.



Figure 2.44: Typical individual components of column base plates

The next step is to obtain the main mechanical properties of each individual joint, such as: the resistance F_{Rd} , the stiffness k_i and the deformation capacity. Combining the mechanical properties of each individual component, the moment-rotation curve can be obtained, defining the global behavior of the connection. Finally, depending on the value of $S_{j,\text{ini}}$, the connections can be classified in terms of its stiffness by the limits established in the EN 1993-1-8, defining the connection as rigid, semi-rigid or pinned.

In the following sections, the calculation procedure for the resistance and stiffness of each aforementioned components is described.

1. Base plate in bending under tension

When subjected to a bending moment, part of the connection is under tension. In this side, the anchor bolts, which are the only elements capable to work in tension, are activated and consequently elongated, and the steel base plate deforms in bending. Thus, the failure modes that govern the tensile part are the yielding of the base plate, the rupture of the anchor bolts or both.

To study the behavior of this component, the tensile part is replaced by a T-stub with width equal to the effective length l_{eff} in order to create a T-section with the same resistance as the component (see Figure 2.45).



Figure 2.45: T-stub nomenclature (EN1993-1-8)

To calculate the equivalent resistance of the T-stub, two different cases are considered depending on the presence of prying forces. These forces develop when the base plate is thin and consequently, more flexible, forcing the contact between the edge of the base plate and the concrete, resulting in an additional force Q resisted by the anchor bolts, as in Figure 2.46. For thicker base plates with rigid behavior, the steel base plate separates from the concrete block.



Figure 2.46: Forces applied on the T-stub in tension

The presence of prying effect can be determined by the established limit in EN 1993-1-8. If the active length of the anchor bolt, L_b , is smaller than this limit, prying effect must be taken into

$$L_{\rm b}^* = \frac{8.8m^3 A_{\rm s} n_{\rm b}}{\sum l_{\rm eff,1} t_{\rm bp}^3}$$
(2.19)

with

m: distance represented in Figure 2.46, $A_{\rm s}$: threaded area of the anchor bolt, $n_{\rm b}$: number of activated anchor bolts rows, $\Sigma l_{\rm eff,1}$: effective length of the T-stub, $t_{\rm bp}$: base plate thickness.

The active length of the anchor bolt L_b can be obtained from the following formula:

$$L_{\rm b} = L_{\rm be} + L_{\rm bf} = 8d + L_{\rm bf} \tag{2.20}$$

with d and L_{bf} defined according to the figure below:



Figure 2.47: Anchor bolt equivalent length

I.1. Design resistance - $F_{T,Rd}$

According to EN 1993-1-8, the value of the resistance depends on the failure mode of the T-stub. Three collapse mechanisms are considered, as shown in Figure 2.48.



Figure 2.48: Available failure modes for a T-stub in tension (EN1993-1-8)

Mode 1 occurs when the base plate is flexible and the anchor bolts show high resistance. Then, the failure mode is characterized by the complete yielding of the base plate. Mode 2 results from a combination of anchor bolts failure and base plate yielding in presence of prying forces. Mode 3 develops for configurations with thick base plates and low anchor bolt resistances. Thus, the failure mode is governed by the collapse of the anchor bolts without prying effects. EN 1993-1-8 add an additional failure mode: Mode 1-2, that corresponds to base plate yielding without prying effect.

The following table is provided by EN 1993-1-8 for the calculation of the resistances according to the aforementioned failure modes:

Table 2.16: Design resistance of a T-stub						
	Prying forces $(L_b \leq L_b^*)$		No prying forces			
Mode 1	Method 1	Method 2 (alternative)				
	$F_{\mathrm{T},1,\mathrm{Rd}} = \frac{4M_{\mathrm{pl},1,\mathrm{Rd}}}{m}$	$F_{\rm T,1,Rd} = \frac{(8n-2e_{\rm w})M_{\rm pl,1,Rd}}{2mn-e_{\rm w}(m+n)}$	$F_{\mathrm{T},1-2,\mathrm{Rd}} = \frac{2M_{\mathrm{pl},1,\mathrm{Rd}}}{m}$			
Mode 2 $F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\sum F_{t,Rd}}{m+n}$						
Mode 3 $F_{T,3,Rd} = \sum F_{t,Rd}$						
<i>m</i> and <i>n</i> : distances taken from Figure 2.46						
$e_{\rm w}$: $d_{\rm w}/4$ with $d_{\rm w}$ equal to the diameter of the nut						

with

the plastic moment capacities of the base plate are given by:

$$M_{\rm pl,1,Rd} = 0.25 \sum l_{\rm eff,1} t_{\rm bp}^2 f_{\rm y} / \gamma_{\rm M0}$$
(2.21)

$$M_{\rm pl,2,Rd} = 0.25 \sum l_{\rm eff,2} t_{\rm bp}^2 f_{\rm y,bp} / \gamma_{\rm M0}$$
(2.22)

the individual tensile design resistance of the anchor bolts is given by:

$$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}}$$
(2.23)

with $k_2 = 0,9$.

 $t_{\rm bp}$ and $f_{\rm y,bp}$ are respectively the thickness and yield strength of the base plate.

Then, in presence of prying effect, the design resistance of the T-stub is taken as:

$$F_{\rm T,Rd} = \min(F_{\rm T,1,Rd}; F_{\rm T,2,Rd}; F_{\rm T,3,Rd})$$
(2.24)

For cases without prying effects, the resistance of the T-stub is:

$$F_{\rm T,Rd} = \min(F_{\rm T,1-2,Rd};F_{\rm T,3,Rd})$$
(2.25)

The effective lengths used in failure modes 1, 2 and 1-2 depends on the potential plastic mechanism that can develop on the base plate:

	Yield pattern	Effective length
Circular patterns		$l_{\rm cp,1} = 2\pi m_{\rm x}$
		$l_{\rm cp,2} = \pi m_{\rm x} + w$
		$l_{\rm cp,3} = \pi m_{\rm x} + 2e$
		$l_{\rm np,1} = 4m_{\rm x} + 1,25e_{\rm x}$
Non-circular		$l_{\rm np,2} = 0.5w + 2m_{\rm x} + 0.625e_{\rm x}$
patterns		$l_{\rm np,3} = e + 2m_{\rm x} + 0,625e_{\rm x}$
		$l_{\rm np,4} = 0.5 b_{\rm bp}$

Table 2.17: Effective lengths of a T-stub subjected to strong axis bending

with

 $m_{\rm x}$: distance from the center line of the anchor bolt to the flange, see Figure 2.45,

w: distance between the center line of both anchor bolts in the same row, see Figure 2.45,

e: distance from the center line of the anchor bolts to the edge of the base plate, see Figure 2.45,

 e_x : distance from the center line of the anchor bolts to the edge of the base plate, see Figure 2.45,

 $b_{\rm bp}$: base plate width.
From Table 2.17, the effective lengths are classified according to the yield line patterns that are divided in two groups (see Figure 2.49):

- circular patterns *l*_{eff,cp},
- non-circular patterns $l_{\rm eff,nc}$.

The effective length of circular patterns corresponds to the minimum effective length considering the three mechanisms of Table 2.17:

$$l_{\rm eff,cp} = \min(l_{\rm cp,1}; l_{\rm cp,2}; l_{\rm cp,3})$$
(2.26)

The effective length of non-circular patterns corresponds to the minimum effective length considering the four mechanisms of Table 2.17:

$$l_{\rm eff,np} = \min(l_{\rm np,1}; l_{\rm np,2}; l_{\rm np,3}; l_{\rm np,4})$$
(2.27)



Figure 2.49: Possible yield patterns – circular and non-circular (Amaral, 2014)

For the cases when failure is given by Mode 1, both circular and non-circular patterns can occur. Then:

$$l_{\rm eff,1} = \min(l_{\rm eff,cp}; l_{\rm eff,nc})$$
(2.28)

For T-stub failure mechanisms governed by Mode 2, only non-circular patterns are taken into account, consequently:

$$l_{\rm eff,2} = l_{\rm eff,nc} \tag{2.29}$$

I.2. Component stiffness $-k_{\rm T}$

Similarly to the design resistance, the stiffness calculation depends on the presence of prying effect. From EN 1993-1-8, the stiffness coefficient of an anchor bolt row is:

$$k_{\rm T} = \frac{1}{\frac{1}{k_{15}} + \frac{1}{k_{16}}} \tag{2.30}$$

The stiffness coefficients k_{15} and k_{16} can be calculated from:

• Presence of prying effect:

$$k_{15} = \frac{0.85 \ l_{\rm eff} \ t_{\rm bp}^3}{m^3} \tag{2.31}$$

$$k_{16} = 1.6 A_{\rm s} / L_{\rm b} \tag{2.32}$$

• Absence of prying effect:

$$k_{15} = \frac{0.425 l_{\rm eff} t_{\rm bp}^3}{m^3} \tag{2.33}$$

$$k_{16} = 2.0 A_{\rm s} / L_{\rm b} \tag{2.34}$$

with

 $l_{\rm eff}$: T-stub effective length,

*t*_{bp} : base plate thickness,

m: distance taken from Figure 2.46,

 $L_{\rm b}$: anchor bolts active length, as in Figure 2.47.

2. Concrete in compression including grout

The resistance of the compressive area depends mainly on the bearing strength of concrete. For this case, an equivalent T-stub model in compression is used to obtain the resistance values for the combination of the following components:

- steel base plate in bending under the bearing pressure of the foundation,
- concrete and/or grout in bearing.

II.1. Design resistance $- F_{c,Rd}$

The design resistance of the T-stub flange in compression can be obtained as follows:

$$F_{\rm c,Rd} = f_{\rm jd} \, b_{\rm eff} \, l_{\rm eff} \tag{2.35}$$

with

 f_{jd} : design bearing strength of concrete,

*b*_{eff} : effective width of the T-stub:

$$b_{\rm eff} = \begin{cases} t_{\rm f} + 2c & \text{for large projection} \\ \frac{h_{\rm bp} - h_{\rm c}}{2} + t_{\rm f} + c & \text{for short projection} \end{cases}$$

 l_{eff} : effective length of the T-stub:

$$l_{\rm eff} = \begin{cases} b_{\rm c} + 2c & \text{for large projection} \\ b_{\rm bp} & \text{for short projection} \end{cases}$$

c: additional bearing width:

$$c = t_{bp} \sqrt{\frac{f_{yd}}{3f_{jd} \gamma_{M0}}}$$
(2.36)

 $t_{\rm bp}$: thickness of the base plate, $f_{\rm y,bp}$: yield strength of the base plate, $\gamma_{\rm M0}$: safety coefficient, taken equal to 1.

From Equation (2.35) it can be understood that the behavior of the component is modelled by replacing the flexible base plate by an equivalent rigid plate under the flange in compression with dimensions b_{eff} and l_{eff} . The pressures transferred to the T-stub are assumed to be uniformly distributed on the equivalent area of the T-stub in compression as shown in Figure 2.50:



Figure 2.50: Equivalent area of a T-stub in compression (EN1993-1-8)

The pressure on the effective bearing area should not exceed the design bearing strength f_{jd} obtained as follows:

$$f_{\rm jd} = \beta_{\rm j} \alpha_{\rm bf} f_{\rm cd} \tag{2.37}$$

with

 β_j : foundation joint material coefficient, equal to 2/3 when the characteristic strength of the grout is not less than 0,2 times the characteristic strength of the concrete foundation and the grout thickness is not greater than 0,2 times the smallest width of the steel base plate. For cases where the thickness of the grout is more than 50 mm, the characteristic strength of the grout should be at least the same as that of the concrete block,

and:

$$\alpha_{\rm bf} = \min\left[1 + \frac{d_{\rm f}}{\max(h_{\rm bp}, b_{\rm bp})}; 1 + 2\frac{e_{\rm h}}{h_{\rm bp}}; 1 + 2\frac{e_{\rm b}}{b_{\rm bp}}; 3\right]$$
(2.38)

$$f_{\rm cd} = \frac{\alpha_{\rm cc} f_{\rm ck}}{\gamma_{\rm c}}$$
(2.39)

Coefficient α_{cc} is considered equal to 1, γ_c equal to 1,5 and d_f is the depth of the concrete foundation. Dimensions e_h and e_b are represented below:



Figure 2.51: Nomenclature of the dimensions of the concrete foundation according to EN 1992-1-1

II.2. Component stiffness $-k_{\rm C}$

The compression stiffness coefficient should be taken as equal to the stiffness coefficient k_{13} given by EN 1993-1-8, then:

$$k_{\rm C} = k_{13} = \frac{E_{\rm c} \sqrt{b_{\rm eff} \, l_{\rm eff}}}{1,275E} \tag{2.40}$$

with

 $E_{\rm c}$: concrete elastic modulus, E: steel elastic modulus, $b_{\rm eff}$: effective width, $l_{\rm eff}$: effective length.

3. Column flange and web in compression

The design resistance of the column flange and web in compression can be calculated from:

$$F_{\rm c,fc,Rd} = \frac{M_{\rm c,Rd}}{(h_{\rm c} - t_{\rm fc})}$$
(2.41)

with

 $M_{c,Rd}$: design moment resistance of the column cross-section that in presence of class 1 can be taken as:

$$M_{\rm C,Rd} = M_{\rm pl,Rd} = \frac{W_{\rm pl,y} f_{\rm y}}{\gamma_{\rm M0}}$$
 (2.42)

 $h_{\rm c}$: column cross section height,

*t*_{fc} : column flange thickness,

 $W_{pl,y}$: column plastic section modulus (strong-axis).

EN 1993-1-8 also provides guidelines for the calculation of the resistance of column base plates subjected to pure compression. The design resistance $N_{j,Rd}$ is assumed equal to the sum of the design resistance in compression $F_{C,Rd}$ for the three T-stubs considered in Figure 2.52.



Figure 2.52: Considered T-stubs for the design resistance in pure compression (EN1993-1-8)

$$N_{j,Rd} = F_{C,Rd,1} + F_{C,Rd,2} + F_{C,Rd,3}$$
(2.43)

After calculating the design resistances and stiffness of each of the abovementioned individual components, the calculation proceeds with the assembly of the mechanical properties to obtain the overall properties of the connection.

For the design moment resistance of the column base, $M_{j,Rd}$, the loading case must be identified within the following considered cases:

- a) connection under dominant axial compressive force. Two T-stubs in compression are activated and failure occurs in the most requested ones,
- b) connection under dominant axial tensile force. Two T-stubs in tension are activated and failure occurs in the most requested ones,
- c) and d) connection under dominant bending moment. T-stubs are loaded in tension and compression and failure develops in one of them.



Figure 2.53: Determination of the lever arm *z* for column base plates (EN1993-1-8)

Then, considering for the calculation of $M_{j,Rd}$:

• In presence of prying effect:

$$F_{\rm T,Rd} = \min(F_{\rm T,1,Rd}; F_{\rm T,2,Rd}; F_{\rm T,3,Rd})$$
(2.44)

• In absence of prying effect:

$$F_{\rm T,Rd} = \min(F_{\rm T,1-2,Rd}; F_{\rm T,3,Rd})$$
(2.45)

and:

$$F_{\rm C,Rd} = \min(F_{\rm c,Rd}; F_{\rm c,fc,Rd})$$
(2.46)

From EN 1993-1-8, the following table summarizes the calculation of the design moment resistances for the available cases:

Table 2.18: Design moment resistance of column base plates				
Loading	Lever arm z	Design moment resistance $M_{j,Rd}$		
Left side in tension		$N_{\rm Ed} > 0$ and $e > z_{\rm T,l}$	$N_{\rm Ed} \leq 0$ and $e \leq -z_{\rm C,r}$	
Right side in compression	$z = z_{\mathrm{T},\mathrm{l}} + z_{\mathrm{C},\mathrm{r}}$	The smaller of $\frac{P_{T,l,Rd^Z}}{\frac{z_{C,r}}{e}+1}$ and $\frac{-P_{C,r,Rd^Z}}{\frac{z_{T,l}}{e}-1}$		
		$N_{\rm Ed} > 0$ and $0 < e < z_{\rm T,l}$	$N_{\rm Ed} > 0$ and $-z_{\rm T,r} < e \le 0$	
Left and right side in	7 = 7 + 7	The smaller of	The smaller of	
tension	2 - 2T, 1 + 2T, r	$\frac{F_{T,l,Rdz}}{ZT}$ and $\frac{F_{T,r,Rdz}}{ZT}$	$\frac{F_{T,l,Rdz}}{T_{T}}$ and $\frac{F_{T,l,Rdz}}{T_{T}}$	
		$\frac{-1,1}{e} + 1$ $\frac{-1,1}{e} - 1$	$\frac{\frac{-1}{2}}{e} + 1$ $\frac{\frac{2}{2}}{e} - 1$	
Left side in		$N_{\rm Ed} > 0$ and $e \leq -z_{\rm T,r}$	$N_{\rm Ed} \leq 0$ and $e > z_{\rm C,l}$	
compression Right side in tension	$z = z_{\mathrm{C},\mathrm{l}} + z_{\mathrm{T},\mathrm{r}}$	The smaller of $\frac{-F_{C,l,Rdz}}{\frac{z_{T,r}}{r}+1}$ and $\frac{F_{T,r,Rdz}}{\frac{z_{C,l}}{r}-1}$		
		$N_{\rm Ed} \leq 0$ and $0 < e < z_{\rm C,l}$	$N_{\rm Ed} \leq 0$ and $-z_{\rm C,r} < e \leq 0$	
Left and right side in	$7 = 7_{0} + 7_{0}$	The smaller of	The smaller of	
compression	2 = 2C, l + 2C, r	$\frac{-F_{C,l,RdZ}}{\frac{Z_{C,r}}{Z_{r+1}}}$ and $\frac{-F_{C,r,RdZ}}{\frac{Z_{C,l}}{Z_{r+1}}}$	$\frac{F_{C,l,RdZ}}{\frac{z_{C,r}}{z_{r+1}}+1}$ and $\frac{F_{C,r,RdZ}}{\frac{z_{C,l}}{z_{r+1}}-1}$	
$M_{\rm Ed} > 0$ is clockwise an	nd $N_{\rm Ed} > 0$ is tension	e e	e e	
$e = \frac{M_{\rm Ed}}{M_{\rm Ed}} = \frac{M_{\rm Rd}}{M_{\rm Rd}}$	Lu			
$N_{\rm Ed} N_{\rm Rd}$				

Lever arms can be calculated according to Figure 2.53. Additionally, EN 1993-1-8 provides formulas to calculate the stiffness of the connections and limits to the classification in terms of its strength and stiffness.

2.5. Concluding remarks

The information presented above allowed to identify not only the aspect to improve on the subsequent chapters but also the existing gaps on the literature. The main points that are highlighted from this extensive literature review are:

- the information found about the experimental studies on out-of-plane/biaxial bending subjected to monotonic loading do not provide full data on the considered geometry and material properties. This fact limits the use of the published experimental data for validation/calibration of analytical models and FE simulations,
- the experimental studies on out-of-plane bending moment under cyclic loading, although providing complete information, as they are subjected to a different type of

loading than the one intended in this work, correspond to very distinct behaviours and failure modes from those observed and analyzed in chapters 3, 4 and 5,

- the experimental and numerical studies exposed allowed to identify which parameters have the greatest influence on the overall behavior of column base plates. This information is extremely important for establishing the experimental test program exposed in Chapter 3, which analyzes the influence of the bending moment orientation and the base plate thickness,
- although it is clear that most of the theoretical studies are based on an interaction with in-plane bending moment, the need to develop a full analytical model for the resistance that considers the contribution of the column web is necessary. Additionally, previous studies have proved that for cases of *M*-*N* interactions, for high levels of compressive axial forces, failure is due to buckling/yielding of the column steel profile rather than from the connection. However, the study of these failure modes is done separately. For this reason, it is considered crucial to develop a single analytical model that takes into account the different failure modes that may arise from the different axial force/bending moment combinations,
- out-of-plane analytical models presented are based on the principles of the component method. However, its low accuracy in predicting the resistance of these connections under these conditions has been proven, providing very conservative results. Thus, the objective for the chapter related to the development of the models for resistance is to develop a simple model, taking into account the different internal force systems and corresponding failure mechanisms,
- studies carried out for biaxial bending are essentially numerical and experimental. The developed theoretical models are based in a very simplified approach, using an elliptical interaction curve. A new calculation procedure for the resistance of column base plates subjected to biaxial bending moment will also be in focus in the following presented works,
- from the previous research works, a solid and comprehensive parametric study to evaluate the influence of several geometrical parameters for different levels of tensile/compressive axial force and bending moment combination is required to compare and validate the obtained results.

3. EXPERIMENTAL TESTS

3.1. Introduction

A campaign of six experimental tests on steel column base plates connected to a concrete block was carried out at *Laboratoire de Génie Civil et Génie Mécanique* – INSA de Rennes with the purpose of studying the behavior of this type of connections subjected to monotonic loading and, validate the numerical and analytical models. The main variables are the base plate thickness and the loading conditions: major/minor axis bending and biaxial bending moment. The effect of the axial force is investigated in the parametric study. The base plates are in direct contact with the concrete block and connected to it using washer plates without any addition of grout or embedded plates. In the project TREPOS, other type of connection between the base plate and the concrete block have been investigated and correspond to post-installed anchor bolts or embedded plates. The results of these three experimental tests are also presented and will be used to validate the numerical model.

3.2. Tests set-up

Both experimental campaigns use the same test set-up. The column is positioned horizontally, working as a cantilever beam, fixed to a concrete block by means of four fasteners and loaded vertically by a load-jack at a distance of 1250 mm from the centre of portal frame A (see Figure 3.1). The concrete foundation is supported in the front and back by means of two portal frames. At the level of the portal A, the concrete block is directly placed on the top of two neoprene supports which allowed its rotation. At portal frame B, rollers are welded to a distribution plate positioned on the concrete block (see Figure 3.1). It is noted that each concrete block was reused twice as the foundation was designed to avoid any major cracking due to concrete cone failure, concrete blow-out or bending of the concrete block. Anchor bolts were placed on both sides. Lateral supports located at mid-length of the column were used to prevent lateral torsional buckling of the steel profile. Welds were painted in white colour in order to observe cracks initiation.





Figure 3.1: Test set-up for in-plane bending moment

As aforementioned, specimens were subjected to three different loading cases: pure bending moment about the strong axis, pure bending moment about the weak axis and biaxial bending moment. To ease the load application for the two last loading cases, a specific test set-up was devised in which the column is split in two parts. The part of the column attached to the load jack was left as for the case of pure bending moment about the strong axis (see Figure 3.3-II). To produce pure bending moment about the weak axis or biaxial bending moment, the second part which is in contact with the concrete block was rotated by 90° and 45° degrees (see Figure 3.2 and see Figure 3.3-I). These two parts were connected by two bolted circular plates, with eight preloaded bolts HR M20 class 10.9. Lateral plates were placed at the free part of the column – see Figure 3.3 (II) – and directly welded to the circular plate. This not only prevented the lateral torsional buckling but also transformed the column open cross-section into a closed cross-section, simplifying the transfer of the forces from this part to the inner column part.



Figure 3.2: Test set-up for out-of-plane (a) and biaxial (b) bending moment



Figure 3.3: Assembly for out-of-plane and biaxial bending moment tests

Specimens were loaded by a vertical force applied at the top of the column and increased until failure, generating a bending moment at the level of the connection. Specimens were expected to fail by anchor bolts rupture in tension (Mode 3 according to EN 1993-1-8) or by a combination of the anchor bolts failure with prying effect and yielding of the base plate (Mode 2 according to EN 1993-1-8).

Specimens were monitored under displacement control and monotonic loading conditions, varying only the orientation of the applied force as follows:

- in-plane load applied at the top of the column in order to create a strong-axis bending moment at the column base plate (Figure 3.4-a)),
- out-of-plane load applied at the top of the column in order to create a weak-axis bending moment at the column base plate (Figure 3.4-b)),
- load applied with a 45° inclination at the top of the column in order to create a biaxial bending moment at the column base plate (Figure 3.4-c)).



Figure 3.4: Loading cases: in-plane bending moment a); out-of-plane bending moment b); biaxial 45° bending moment c).

For these different loading combinations, the objective was to evaluate the rotational stiffness, the bending moment resistances, the failure modes and the rotation capacity of the column base plates.

3.3. Geometry of specimens tested

As mentioned, the main purpose of these tests is to characterize the non-linear behavior of column base plates subjected to a bending moment applied in different directions and to evaluate the influence of various parameters on the overall performance of the connections, such as: column and base plate steel grade, thickness of the base plate; anchor bolts, diameter, steel grade and embedment length of the anchor bolts/bolts/dowels; column steel profile cross-section geometry (HE or IPE). Sections 3.3.1 and 3.3.2 provide a detailed description of the geometry and the materials parameters for each tested specimen.

3.3.1 Column base plates

The tested specimens consist in a steel column HEA 200 welded to a rectangular base plate 330×300 mm by means of fillet welds of 7 mm throat thickness. The base plate was fixed to a concrete block $1450 \times 900 \times 610$ of class C25/30 by four anchor bolts M16 class 5.6 (see Figure 3.5). The embedded anchor bolts were connected to the concrete block by $50 \times 50 \times 10$ mm washers.



Figure 3.5: Tested column base plate configuration (dimensions in mm)

The geometrical and mechanical characteristics of the specimens are given in Table 3.1. The designations given to the tests consist of the capital letters *SPE* followed by a number indicating the base plate thickness - 1 for 10 mm and 2 for 20 mm - and then, the indication of the loading conditions – M0 for in-plane bending moment, M90 for an out-of-plane bending moment and M45 for biaxial bending moment.

	Table 3.1: Identification of the column base plates test specimens					
Test ID	T	Column profile	Column steel	Base plate thickness	Base plate steel	
Test ID	Loading conditions		grade	$t_{\rm bp}({ m mm})$	grade	
SPE1-M0	In-plane bending			10		
SPE2-M0	In-plane bending			20		
SPE1-M90	Out-of-plane bending		8255	10	8075	
SPE2-M90	Out-of-plane bending	ПЕА 200	3333	20	3275	
SPE1-M45	Biaxial bending			10		
SPE2-M45	Biaxial bending			20		

Base plates with a thickness of 10 and 20 mm were selected to evaluate the impact of the flexibility of the base plate on the failure mode as well as the overall response of the connection.

In order to be able to resist to the applied loadings, the concrete block was slightly reinforced. The steel reinforcement (see Figure 3.6) consists in a steel reinforcing mesh ST 25C supplemented on the top surface with 6 HA 10 of steel grade S500 with spacing 150 mm along the longitudinal direction (in green, see Figure 3.6). The cover to reinforcement was 25 mm. Additional C-shape plain round bars were placed along the supports zones to prevent concrete cone failure (in blue, see Figure 3.6). Shear reinforcements were added near the supports. More details about reinforcement can be found in the appendix at the end of this chapter.



Figure 3.6: Concrete block reinforcement (top view)

3.3.2 Beam-to-concrete slab connections

The main objective of these tests, which are part of the project TREPOS, is to study the behavior of steel beam-to-concrete slab with steel end plate connections subjected to in-plane bending moment. The behaviour of these connections loaded under in-plane bending moment and the transfer of forces to the anchoring system is quite similar to the column base plates. Hence, the test device used was the same for both test campaigns. Due to the geometric and material

differences in terms of steel column profile, base plate, anchor bolts and shear lug from the tests in 3.3.1, the inclusion of these tests allowed to understand more about the influence of these parameters on the behavior and failure modes of these connections and to validate the analytical model developed in Chapter 5 in a more comprehensive way. Table 3.2 summarize the main features concerning the specimens presented.

	Table 3.2. Deam-to-concrete state connections					
Test ID	Loading	Beam	Beam steel	Stiffeners	Base plate	Base plate steel
Test ID	conditions	profile	grade	Suitenets	thickness t _{bp} (mm)	grade
PB-1				No	20	
PB-2	In-plane bending	IPE 200	S275	Yes	15	S235
PB-4				Yes	15	

The geometry of the three specimens is presented in Figure 3.7, Figure 3.8 and Figure 3.9, respectively for specimens PB-1, PB-2 and PB-4. An IPE 200 is welded to a rectangular base plate of dimensions 340×160 mm. For the two latter specimens, stiffeners have been added as it affect the stiffness and resistance of these type of connections. The base plate is connected to the concrete by means of an embedded plate and bolts fully threaded SB M16 class 8.8 for the case of specimens PB-1 and PB-2. PB-4 has been connected by chemical post-installed anchor bolts HIT-Z M20. These tests provide an opportunity to estimate the impact of the fastening on the global behaviour of the connection. The specimens are fixed to a concrete slab of 200 mm thickness with a lintel of 450 mm height and 200 mm thickness. The concrete slab was reinforced in order to avoid premature failure of the slab in bending. Detailed information about the reinforcement can be found in the appendix at the end of this chapter.



Figure 3.7: Specimen PB-1



Figure 3.8: Specimen PB-2



Figure 3.9: Specimen PB-4

1450



Figure 3.10: Concrete slab dimensions (in mm)

3.4. Instrumentation

3.4.1 Column base plates experimental program

Three different measured values were recorded during the experimental tests:

- vertical displacements of the column and the concrete block as well as horizontal displacements of the base plate,
- vertical force applied at the top of the column,
- axial strains at the base of the column steel profile.

3.4.1.1 Displacement sensors placed along the column and the base plate

To monitor the displacements, seven LVDT's (linear variable differential transformer) were placed along the specimens. V_1 was placed below the load application point located at 1085 mm from the connection. To measure the vertical displacements along the column, V_2 , V_3 and V_4 were placed at 685 mm, 285 mm and immediately below the base plate. V_5 , V_6 and V_7 located below the concrete block at 100 mm, 325 mm and 1125 mm allowed the calculation of the block rotation θ_c . Due to the location of V_3 , the latter was not considered for specimens SPE1-M45 and SPE2-M45 as the column steel profile was rotated by 45° and can compromise the accuracy of the measurement. Horizontal transducers U_{t1} , U_{t2} , U_{b1} and U_{b2} were placed at the level of the column flanges and used to evaluate the uplift of the base plate in the tensile zone and the crushing at the compressive zone. The location of these devices is depicted in Figure 3.11 and Figure 3.12.



Portal frame B Portal frame A Figure 3.11: Displacement sensors arrangement for column base plate tests



Figure 3.12: Location of the horizontal displacement sensors for M0, M90 and M45 test series.

3.4.1.2 Hydraulic jack

Vertical forces were applied to the structure by a SCHENCK load-jack with capacity \pm 1500 kN. To obtain a greater level of accuracy on the measured results, an additional tensile/compression load cell DELTECH LC 403 TC of \pm 200 kN capacity was attached to the load-jack.

3.4.1.3 Strain gauges

For specimens SPE1-M90 and SPE2-M90, a total of eight strain gauges, four on each external face of the HEA 200 steel profile flanges at a distance of 20 mm from the welds. In this area, stress concentration due to redistribution of the forces between the compressive and tensile parts is expected. The longitudinal distance between the strain gauges is 50 mm (see Figure 3.13). Figure 3.14 shows the identification of the strain gauges for tests SPE1-M90 and SPE2-M90.





 Figure 3.13: Location of strain gauges for SPE1-M90 and SPE2-M90
 Figure 3.14: Strain gauge designations for SPE1-M90 and SPE2-M90

3.4.2 Beam-to-concrete slab connections experimental program

Eight displacement sensors, V_1 to V_8 , were placed along the test specimen in order to evaluate the vertical displacements during the loading and providing a complementary estimation of the slab rotation. Similarly to the previous tests in sub-chapter 3.4.1, four horizontal displacement transducers attached to the column steel profile were placed to evaluate the deformed shape of the base plate during the tests until failure. The position of LVDT's can be seen in Figure 3.15.



Portal frame B

Portal frame A

Figure 3.15: Displacement sensors arrangement for beam-to-concrete slab tests

3.5 Material characteristics

Raw materials used for specimen fabrication have been tested in order to define its actual properties. The following material characteristics have been measured:

- modulus of elasticity of steel *E*,
- yield strength of steel elements f_y ,
- ultimate strength of steel elements $f_{\rm u}$,
- necking of steel elements,
- elongation of steel elements,
- compressive cylinder strength of concrete f_{cm} .

3.5.1 Column base plate

Following the guidelines provided by EN 10002-1: 2001, tensile tests were carried out for the subsequent elements:

- column steel profile,
- 10 mm and 20 mm base plates,
- anchor bolts.

Concrete cylinder tests were also performed to determine the compressive strength $f_{\rm cm}$ according to EN 12390-3.

3.5.1.1 Mechanical properties of the column steel profile and base plates

Tensile tests were carried out on rectangular steel coupons (see Figure 3.16) from the column and base plates, using a testing machine of type INSTRON 3369 with 50 kN capacity. Tests were performed under displacement control. The axial deformation was given by an extensioneter placed in the longitudinal direction. Twelve samples were tested, as follows:

- three samples from the HEA 200 column flange,
- three samples from the HEA 200 column web,
- three samples from the 10 mm base plate,
- three samples from the 20 mm base plate.



Figure 3.16: Dimensions of rectangular steel samples



Figure 3.17: Rectangular steel samples after tensile tests

Mechanical characteristics of column and base plates are presented in Table 3.3, based on EN 10002-1: 2001:

Table 3.3: Average mechanical characteristics of column and base plates						
Element sample	Elasticity	Yield strength	Ultimate strength	Necking	Elongation	
	modulus (MPa)	(MPa)	(MPa)	(%)	(%)	
Base plate 10 mm	213122	423,6	512,7	70	37	
Base plate 20 mm	209288	401,3	503,1	71	30	
Column web	210000 ⁽¹⁾	378,4	457,3	60	25	
Column flange	211691	350,1	433,3	69	34	

Note 1 : This value corresponds to the nominal one as the measured value was not coherent



The yield and ultimate strengths are higher than the nominal properties provided by EN 1993-1-1. The plotted stress-strain curves in Figure 3.18, Figure 3.19, Figure 3.20 and Figure 3.21 are representative of hot rolled mild steel, showing a yield plateau followed by a hardening phase until the maximum strength. The tested samples show adequate ductility, respecting the minimum requirements established by EN 1993-1-1.

3.5.1.2 Mechanical properties of anchor bolts

Tensile tests on anchor bolts M16 class 5.6 round samples (see Figure 3.22) were performed. The testing machine used for tests was an INSTRON 3369 with a 50 kN capacity. Three samples were tested in total. Average mechanical characteristics of anchor bolts are given in Table 3.4.



Figure 3.22: Dimensions of the round steel samples



Figure 3.23: Round steel samples after tensile tests

Diameter	Elasticity	Yield strength	Ultimate strength	Necking	Elongation
	modulus (MPa)	(MPa)	(MPa)	(%)	(%)
M16	208295	519,1*	626,8	72	9

Table 3.4: Average mechanical characteristics of anchor bolts

*Defined as the value corresponding to $\varepsilon = 0.2\%$.



Figure 3.24: Stress-strain curve for anchor bolts

The maximum yield strength is higher than the predicted nominal value estimated at 300 MPa. The necking phase was quite substantial and of the same order of magnitude as for column and the base plates, demonstrating the great ductility of the material.

3.5.1.3 Concrete properties

A concrete class C25/30 was used for the concrete blocks. Nine cylinders of dimensions 11×22 cm were casted at the same time as the concrete blocks and subsequently tested. Standard cylinder tests were carried out at 31, 73 and 80 days, to coincide as closely as possible with the dates of test. The compression tests were performed with a hydraulic compressive testing machine, the capacity of which is 3000 kN.

Table 3.5: Compressive strength of concrete (column base plate tests)					
Time (days)	Maximum compressive	Average compressive			
Time (days)	strength (MPa)	strength (MPa)			
	25,2				
31	25,9	25,0			
	23,9				
	32,4				
73	32,5	32,6			
	33,1				
	33,9				
80	33,3	33,4			
	32,8				



Figure 3.25: Concrete compressive test – a); tested cylinder at 31 days – b); tested cylinder at 80 days – c)

Table 3.6 provide an estimation of the concrete strength at the day of test for each specimen. These values were obtained by interpolation and will be used in the numerical analysis (Chapter 4).

Table 3.0: Coller	ete compressive suengu	is for tests SFET and SFE2 evaluated by intear interpolation
Test ID	Time (days)	Maximum compressive strength f_{cm} (MPa)
SPE1-M0	43	28,6
SPE2-M0	41	28,2
SPE1-M90	51	30,4
SPE2-M90	48	29,5
SPE1-M45	79	33,3
SPE2-M45	70	32,3

Table 3.6: Concrete compressive strengths for tests SPE1 and SPE2 evaluated by linear interpolation

3.5.2 Beam-to-concrete slab connections

Coupon tests were carried out, following recommendations of EN 10002-1: 2001 and EN 12390-3, on steel samples and concrete cylinders for:

- beam steel profile IPE 200,
- 15 mm and 20 mm base plates,
- bolts and post-installed anchor bolts,
- concrete.

3.5.2.1 Mechanical properties of the beam steel profile and base plates

Tensile tests were performed on rectangular steel samples from the column flanges and web, and for both 15 mm and 20 mm base plates, with an INSTRON 3369 testing machine of 50 kN capacity. The axial deformation was given by an extensometer device placed in the longitudinal direction. In total, twelve samples were tested, as follows:

- three samples from the IPE 200 beam flange,
- three samples from the IPE 200 beam web,
- three samples from the 15 mm base plate,
- three samples from the 20 mm base plate.

Mechanical characteristics of the beam and base plates are summarized in Table 3.7.

Table 3.7: Mechanical properties of beam and base plates				
Element sample	Elasticity	Yield strength	Ultimate strength	Necking
	modulus (MPa)	(MPa)	(MPa)	(%)
Base plate 15 mm	206355	277,5	446,8	70
Base plate 20 mm	206140	229,1	396,7	63
Beam web	220255	340,1	483,5	64
Beam flange	218827	333,0	489,9	65



The results presented in Table 3.7 suggest that the properties of the materials are in overall agreement with those expected from the material grades, being higher than the nominal values provided in EN1993-1-1 except for the 20 mm base plate. Curves plotted from Figure 3.26 to Figure 3.29 represent the stress-strain curves obtained from the tensile coupon tests.



3.5.2.2 Mechanical properties of fasteners

Two bolts M16 class 8.8 samples and two post-installed anchor bolt M20 samples were tested (see Figure 3.22). The mechanical characteristics are exposed in Table 3.8.

Table 3.8: Mechanical characteristics of fasteners				
Elasticity		Yield strength	Ultimate strength	Necking
Diameter	modulus (MPa)	(MPa)	(MPa)	(%)
M16	198004	1063,5*	1082,2	69
M20	188765	724,4*	764,5	78

1400 1000 Stress Stress (MPa) (MPa) 1200 800 1000 600 800 AB-1 B-1 600 400 AB-2 **B-2** 400 200 200 Strain (mm/mm) Strain (mm/mm) 0 0 0 0.02 0.04 0.06 0.08 0.1 0 0.1 0.2 0.3 Figure 3.30: Stress-strain curve for the bolts M16 Figure 3.31: Stress-strain curve for the post-installed anchor bolts M20

*Defined as the value corresponding to $\varepsilon = 0.2\%$.

3.5.2.3 Concrete properties

Concrete cylinders of dimensions 11×22 cm were tested using the same testing device as in subsection 3.5.1.3. The concrete strength is provided in Table 3.9 for PB-1 and in Table 3.10 for PB-2 and PB-4. The properties of each test evaluated by interpolation are given in Table 3.11.

	Table 3.9: Compressive strength for PB-1					
Time (days)	Compressive strength	Average compressive				
Time (days)	(MPa)	strength (MPa)				
57	31,8	22.0				
57	32,2	32,0				
Tab	le 3.10: Compressive streng	th for PB-2 and PB-4				
Time (days)	Compressive strength	Average compressive				
Time (days)	(MPa)	strength (MPa)				
	29,5					
28	28,2	29,2				
	29,8					
	38,4					

Table 3.11: Concrete compressive strengths for tests PB-1, PB-2 and PB-4 evaluated by linear interpolation

38.2

Test ID	Time (days)	Compressive strength $f_{\rm cm}$ (MPa)	
PB-1	57	32,0	
PB-2	83	34,1	
PB-4	132	36,2	

3.6 Testing procedure

205

37,7

38,7

Specimens were subjected to a monotonic vertical force producing either unidirectional bending moment (in-plane or out-of-plane) or biaxial bending moment. All specimens were tested under displacement control. For a clearer evaluation of the behavior of the individual elements and the distribution of forces during the tests, the loading was divided in two main phases: the elastic phase and the plastic phase in which the connections were loaded until failure.

• Elastic phase:

The loading applied during this stage was estimated in order to maintain the connection in the elastic regime. This phase consisted in four loading/unloading cycles, the first three belonging to the elastic phase 1 and the fourth, to the elastic phase 2. In the elastic phase 1, the objective was to apply a load equal to 50% of the design resistance F_{Rd} calculated from EN1993-1-8. For elastic phase 2, specimens were subjected to the full design resistance F_{Rd} . For more detailed information on the loading, the reader will find in Table 3.12 a summary of the maximal forces applied by the load-jack during each phase. The design resistance F_{Rd} was considered as the force corresponding to a bending moment equal to $2/3M_{j,Rd}$, which corresponds to the theoretical elastic limit in Eurocode 3 Part 1-8.

• Plastic phase:

During this phase, the connections worked in the plastic domain and some elements started to yield. The applied vertical force increased gradually until one or more elements failed.

Test ID	Applied vertical force (kN)			
	Elastic phase 1	Elastic phase 2	Plastic phase	
SPE1-M0	7,5	15,0	37,6	
SPE2-M0	10,0	25,0	42,1	
SPE1-M90	6,0	12,0	28,9	
SPE2-M90	8,5	20,0	35,0	
SPE1-M45	7,5	15,0	34,2	
SPE2-M45	10,0	20,0	41,1	
PB-1	10,0	20,0	61,7	
PB-2	9,5	20,0	74,8	
PB-4	10,0	20,0	40,7	

3.7 Experimental results

3.7.1 Column base plates experimental program

3.7.1.1 Description of the tests

• Test SPE1-M0

The test started with three loading/unloading cycles up to 7,5 kN during phase 1. When the maximal force was applied, no deformations were visible in the base plate except the initial imperfections (about 1 mm at the top and bottom free edge) due to heat effect produced during welding of the base plate to the column (see Figure 3.91).

During the fourth cycle within the second phase, the applied load reached 15 kN at a rate of 1 mm/min. At this load level, the deformation of the base plate is clearly observable (see Figure 3.33). An uplift between the concrete and the base plate of about 1,5 mm was measured nearby the upper column flange in tension. At the end of the unloading stage, the base plate returned to its initial position. During these elastic stages, the contact did not develop at the upper free edge of the base plate and prying effect did not appear which contrast with the prediction of EN 1993-1-8. The initial imperfection and the rotation of the connection clearly favor this absence of prying effect. In addition, the uplift is maintained at the bottom free edge of the base plate (see Figure 3.33).



Figure 3.32: Initial base plate position



Figure 3.33: Base plate at 15kN



Figure 3.34: Main phases of the SPE1-M0 test (plastic domain)

During the plastic phase, the bending deformation of the base plate appears quite soon in conjunction with anchor bolts elongation in the tensile area. The bottom part of the extended end-plate remain in contact with concrete and uplift starts probably behind the bottom column flange (see Figure 3.35). At the same time, the deformation of the base plate increases significantly with the anchor bolts being still able to resist increasing applied loads. As a result of this deformation, the free edge in the tensile zone of the base plate is getting closer to the concrete surface. However, as shown in Figure 3.34, during the whole test contact was not observed at the outer edge of the base plate in the tensile area. Consequently, prying effect did not develop although the base plate was relatively thin. This load transfer mechanism between

the base plate and the anchor bolts was maintained until the system reached the maximum force (37,6 kN). At this load level, the deformation of the base plate and more generally the connection was already quite substantial with horizontal displacements at the top of the base plate reaching 4,5 mm and -0,6 mm at the bottom (compression).



Figure 3.35: Base plate deformation at 37,6 kN (side)



Figure 3.36: Base plate deformation at 37,6 kN (top)

The force applied to the connection decreased due to necking of the anchor bolts in tension. Due to this elongation, the uplift of the upper outer edge of the base plate can be clearly observed. The failure appears for a force of 31,9 kN caused by the rupture of one anchor bolt in tension. Cracks were observed on the concrete block surface around the tensile anchors as a consequence of their elongation (see Figure 3.39 and Figure 3.40).



Figure 3.37: Base plate deformation after failure (side)



Figure 3.38: Base plate deformation after failure (top)



Figure 3.39: Cracks at the concrete surface after failure (left activated anchor bolt)



Figure 3.40: Cracks at the concrete surface after failure (right activated anchor bolt)

• Test SPE2-M0

Initially, the base plate was in full contact with the concrete foundation and no initial gap can be observed (see Figure 3.41). The loading started with the initial three loading/unloading cycles up to 10 kN at a displacement rate of 0,5 mm/min. After each cycle, no residual deformations were visible in the overall assembly.



Figure 3.41: Initial base plate position

Figure 3.42: Base plate deformation at 25kN

During the fourth cycle, the connection was loaded up to 25 kN at a rate of 1 mm/min. At this load level, a horizontal displacement of 2 mm at the free edge of the base plate was measured (see Figure 3.42). At the end of the unloading phase, similarly to the previous test, it was observed that the connection return to its initial position. In contrast with specimen SPE1-M0, the base plate remain straight in the tensile area.

During the plastic stage, the base plate remain rigid in the tensile part and the deformation in this area is mainly due to bolt elongation in tension. This behavior is quite different to that of SPE1-M0. The compression area is clearly located at the bottom of the extended base plate, outside the column.



Figure 3.43: Main phases of the SPE2-M0 test (plastic domain)

The maximal force applied to the connection is equal to 42,1 kN, and the subsequent drop of the applied force is due to necking of the anchor bolts in tension. The horizontal displacement transducers indicated a horizontal displacement of 8,4 mm and 0,9 mm at the level of the upper and lower column flange, respectively. This result confirm that at these points the base plate was not in contact with the concrete block. This behavior suggests that unlike specimen SPE1-M0 and as a result of the plate stiffness (20 mm thickness), the location of the center of compression moved in the direction of the bottom outer edge of the base plate. At 36,2 kN, rupture of the bolts in tension took place (see Figure 3.45). Similarly to test SPE1-M0, no particular damages were recorded in the welds and in the concrete block. Furthermore, local cracks develops on concrete surface close to the anchors bolts loaded in tension due to excessive elongation (see Figure 3.47).



Figure 3.44: Base plate deformation at 42,1 kN



Figure 3.46: Base plate deformation after failure (top)



Figure 3.45: Base plate deformation after failure (side)



Figure 3.47: Cracks at the concrete surface after failure (right anchor bolt)

• Test SPE1-M90

Prior loading, the bottom extended base plate was in full contact with the concrete foundation, whereas gaps of about 0,4 mm were measured for the rest of the connection (see Figure 3.48). For SPE1-M90, the elastic phase 1 was performed at a displacement rate of 0,5 mm/min and stopped at 6 kN. During the fourth loading/unloading cycle, the displacement rate was adjusted to 1 mm/min and the maximum applied vertical force reached 12 kN. Throughout both elastic phases and due to the limited thickness of the base plate, the deformation of the latter was visible. Its initial position was recovered after each unloading phase. Laterally, an uplift between the base plate and the concrete surface was recorded, halfway between the upper and lower face of the column flanges (see Figure 3.49). From the top view, the base plate formed a concave surface with respect to the concrete surface.



Figure 3.49: Base plate deformation at 12 kN (side)

Figure 3.50: Base plate at the end of the fourth cycle

Even if an initial gap exists at the location of the upper anchor bolts, this gap closed during loading and prying effect probably starts to develop which is in contrast with what happened during the test of specimen SPE1-M0.

The deformation of the base plate is concave (see Figure 3.52). The maximum resistant capacity of the connection was reached at 28,9 kN. From this load level, and similarly to test SPE1-M0, the anchors bolts began to stretch excessively, causing significant deformation in the global assembly (see Figure 3.53 and Figure 3.54). The contact between the baseplate and concrete surface was lost and thus prying effect. The rupture of one of the anchor bolts occurred at a force equal to 25,1 kN. At the end of the test, local cracks initiation in the welds and on the concrete surface were observed (see Figure 3.57 and Figure 3.58).



Figure 3.51: Base plate deformation before F_{max} (side)



Figure 3.53: Base plate deformation after F_{max} (side)



Figure 3.52: Base plate deformation before F_{max} (top)



Figure 3.54: Base plate deformation after F_{max} (top)



Figure 3.55: Base plate deformation after failure (side)



Figure 3.57: Local cracks in the weld

• Test SPE2-M90



Figure 3.56: Base plate deformation after failure (top)



Figure 3.58: Cracks at the concrete surface after failure

Initially, gaps ranging between 0,15 and 0,55 mm were measured over the full depth of the base plate. The test started with three loading/unloading cycles with a maximum load of 8,5 kN considering a displacement rate of 0,5 mm/min. This rate was adjusted to 1 mm/min through the next loading/unloading cycles carried out until the maximum force value of 20 kN. Meanwhile, at 10 kN, uplift between the top end of the base plate and the concrete surface was about 1,5 mm, increasing to 2 mm for 16 kN. At 20 kN, uplift was about 2,5 mm. At the end of the unloading phase, no residual displacements could be observed. Similarly to what happened during test SPE2-M0, the base plate behaved like a rigid plate in the tensile area. Throughout the loading, the deformed configuration of the base plate indicates that the contact area had moved towards the lower free edge of the base plate, between the lower anchor bolts row and the free edge of the base plate. The connection reached its maximum resistant capacity for a load close to 35 kN. Then, the upper anchor bolts row started to elongate excessively as the applied load began to drop. The failure of the connection is obtained at 32,9 kN by the rupture

of one of the upper anchor bolts. Local cracks in the concrete were observed due to the activated upper and lower anchors bolts rows, as both bolt rows were subjected to tensile forces, contrary to the M0 test serie.



Figure 3.59: Base plate deformation at 33 kN (top)



Figure 3.61: Base plate deformation after failure (top)



Figure 3.60: Base plate deformation at 33 kN (side)



Figure 3.62: Base plate deformation after failure (side)



Figure 3.63: Local cracks at the surface of the concrete (upper anchor bolts row)



Figure 3.64: Local cracks at the surface of the concrete (lower anchor bolts row)

• Test SPE1-M45

At the beginning of the test, the three corners C_1 , C_3 and C_4 were in contact with the concrete surface and a gap of 1 mm was measured at the upper corner C_2 (see Figure 3.65). The elastic phase consisted of three loading/unloading cycles up to 7,5 kN at a rate of 0,5 mm/min and then a fourth cycle up to 15 kN in which the displacement rate was adjusted to 1 mm/min and thereafter increased to the end of the test. During the first three cycles, a 1,5 mm displacement began to be visible at C_2 . During the fourth cycle, at 12 kN, there was a 2 mm uplift between the base plate and the concrete surface, halfway between C_1 and C_2 and C_3 . At the end of this cycle, the base plate returned to its initial position.



Figure 3.65: Identification of the base plate corners and anchor bolts

During the plastic phase, anchor bolts AB_1 , AB_2 and AB_3 were activated. Anchor bolt AB_4 remained inactive throughout the loading as it was positioned in the compressive area. Initially, the base plate began to deform. The corner C_2 starts to move away from the concrete surface while the remaining corners C_1 , C_3 and C_4 stayed in contact. Prying effect also develops around anchor bolts AB_1 and AB_3 but not AB_2 . This difference is due to the global rotation of the connection that limit the development of prying effect as it induce an initial rotation of the base plate at the junction with the column.
At 19 kN, the point located halfway between C_1 and C_2 moved away from the concrete surface by 3 mm whereas corners C_1 , C_3 and C_4 remained in contact. A substantial bending of the base plate was observed between P_4 and C_4 . At 28 kN, first cracks initiate in the welds (see Figure 3.71). Increasing deformation of the base plate led to the activation of anchor bolts AB_1 and AB_3 , C_3 was no longer in contact with the concrete surface. At 33 kN, the uplift at C_2 was 5 mm and about 8 mm between C_1 and C_2 and, C_2 and C_3 . Due to the reduced thickness of the base plate and the location of the center of compression, close to P_4 , corner C_4 has begun to lose contact with the concrete surface. The most tensioned anchor bolt AB_2 reached its maximum capacity for a load close to 34,2 kN. At this load level, the relative displacements for points P_1 , P_2 , P_3 and P_4 between the plate and the concrete surface were respectively: 3,3 mm (uplift), 6,4 mm (uplift), -0,5 mm (crushing) and -2,90 mm (crushing). The rupture of AB_2 occurred for a value of the load equal to 32,9 kN. The residual force resisted by the remaining anchor bolts was about 17,5 kN and thus corresponds to half of the maximum force reached during the test.



Figure 3.66: Base plate deformation at 30 kN (side)



Figure 3.67: Base plate deformation at 30 kN (top)



Figure 3.68: Base plate deformation at 33 kN between the extremities of the base plate C1 and C2



Figure 3.69: Base plate deformation after failure (right side)



Figure 3.70: Base plate deformation after failure (top)



Figure 3.71: Cracks initiation on the weld at 28 kN

• Test SPE2-M45

At the beginning of the test, a gap of about 1 mm was measured at corners C_1 and C_2 . C_3 and C_4 were in contact with the concrete block. During the first three cycles, the maximum load was 10 kN and no deformations of the base plate were visible in the connection. For the last elastic cycle, the maximum load reached was about 20 kN and caused an uplift at point C_2 of 1 mm. Between C_1 and C_2 , the uplift was equal to 2,5 mm. After unloading, the base plate recovered its initial configuration (see Figure 3.72).

As observed for SPE2-M0 and SPE2-M90 tests, deformations of the base plate were less significant. As the applied load increased, bending of the base plate between points P_4 and C_4 was clearly evident. At the beginning of the fifth loading cycle, the gap read at C_1 and C_2 was approximately 1 mm. At 30 kN, these displacements increased to reach 2,5 mm and 3 mm, respectively. At 34,4 kN, the displacement at C_2 was 4,5 mm and 3 mm at C_1 . At 37,7 kN, the deformation of the connection was already quite advanced, with a displacement of 6 mm at C_2 and 5 mm at C_1 . At this moment, anchor bolts AB₁, AB₂ and AB₃ were all activated. The most tensioned anchor bolt AB₂ reached its maximum strength at 41,1 kN. Next, the load began to decrease and at 39,9 kN AB₂ failed. The maximum displacement swere 5,7 mm (uplift), 5,9 mm (uplift), 2,2 mm (crushing) and 2,8 mm (crushing). At the end of the test, the load was approximately equal to 23 kN, confirming that anchor bolts AB₁ and AB₃ contributed to resist half of the maximum force. Due to the high level of tensile forces resisted by these elements, damage on the concrete surface was visible (see Figure 3.77).



Figure 3.72: Base plate initial position



Figure 3.73: Base plate deformation at 40 kN (side)



Figure 3.75: Base plate deformation after failure (side)



Figure 3.74: Base plate deformation at 40 kN (top)



Figure 3.76: Base plate deformation after failure (uplift between C1 and C2)



Figure 3.77: Cracks at the surface of the concrete foundation

3.7.1.2 Force-vertical displacement curves and failure modes

For the six experimental tests, the failure was due to the rupture of one or more activated anchor bolts in tension, in the threaded area (see Table 3.13). Yielding of the base plate was clearly observed only for the base plate of 10 mm thickness (serie SPE1).

Table 3.13: Failure modes for SPE tests				
Test ID	Yielded elements	Failure mode		
SPE1-M0	Anchor bolts/base plate	Anchor bolt (one)		
SPE2-M0	Anchor bolts	Anchor bolts (two)		
SPE1-M90	Anchor bolts/base plate	Anchor bolt (one)		
SPE2-M90	Anchor bolts/column	Anchor bolt (one)		
SPE1-M45	Anchor bolts/base plate	Anchor bolt (one)		
SPE2-M45	Anchor bolts	Anchor bolt (one)		

The force applied by the load-jack as a function of the displacement sensor V_1 is presented below for all tests. Curves corresponding to tests SPE1/2-M0 and SPE1-M90 follow a similar shape, decreasing significantly after reaching the maximum force as consequence of anchor bolt necking prior failure. For tests SPE2-M90 and SPE1/2-M45, this drop is less significant as it was compensated by a slight redistribution of the tensile forces between the anchor bolt rows.





These curves highlight the influence of the base plate thickness and the orientation of the applied bending moment on the deformability of the connection. For all cases, although less pronounced for the M45 test series, the curves corresponding to specimens with a 10 mm base plate exhibited a higher deformability than that of the 20 mm base plate tests. Although noticeably more resistant, with higher values of vertical applied load, the maximum displacements reached for the tests with a base plate of 20 mm were smaller, since the only source of deformation of the connections was the elongation of the anchor bolts in tension. In contrast, the maximum displacements with a thinner base plate increased due to the development of plastic yield line in the tensile area of the base plate.

The direction of the applied load also influenced the overall displacement of the connection. A trend on the maximum displacement for each series of tests can be clearly observed. Overall, maximum displacements observed in M90 test serie are larger than that measured in M0 test serie, exhibiting a less deformation capacity. Regarding M45 test serie, the maximum displacements are comprised between those of M0 and M90 test series. The difference on the magnitude of the maximum displacement is mainly due to the lever arm that is lower for specimens M90 and greater for specimens M0. The possibility to redistribute the forces between the different anchor bolts also contribute to increase the rotation capacity of the connection.

3.7.1.3 Moment-rotation curves

The moment-rotation curves for all specimens are illustrated in Figure 3.82 and Figure 3.83. The bending moment, M, was calculated by the following expression:

$$M = F \times L \tag{3.1}$$

with

F : vertical force,

L : distance between the load application point and the column base plate.

The rotation θ_j of the connection was calculated based on the displacements provided by LVDT's U_{t1}, U_{t2}, U_{b1} and U_{b2} as follows:

$$\theta_{\rm j} = \frac{U_{\rm t} - U_{\rm b}}{z_{\rm col}} \tag{3.2}$$

with

 U_t : average horizontal displacement of U_{t1} and U_{t2} (see Figure 3.11),

 U_{b} : average horizontal displacement of U_{b1} and U_{b2} (see Figure 3.11),

 z_{col} : height of the column cross-section for in-plane bending and width of the column for out-of-plane bending.

Regarding the calculation of θ_j for cases M45, Equation (3.2) is no longer valid. The values given by U_t and U_b should be replaced with those of U_{t2} and U_{b1} , respectively (see Figure 3.12). The parameter z_{col} is considered as the distance between the upper and lower column flanges.

For SPE2, the curves of cases M45 and M90 are quite different from that of M0. A clear hardening phase can be observed as a result of a redistribution of the forces in the inner anchor bolts. Due to the absence of prying effect and base plate yielding, the curve of SPE2-M0 is directly proportional to the force-displacement curve of the anchor bolts and thus characterised by a rapid loss of stiffness due to necking.



Figure 3.82: Influence of the base plate thickness on the moment-rotation curves



The plastic bending moment $M_{j,pl}$, the ultimate bending moment $M_{j,u}$, the initial stiffness $S_{j,ini}$ and the rotation capacity $\theta_{j,u}$ are summarized in Table 3.14. To define the yield point and consequently $M_{j,pl}$ and $S_{j,ini}$, the method proposed by CECM (1986) described in Figure 3.84 was used. For that, the tangent to the moment-rotation curve at the origin is drawn, the slope of which is $S_{j,ini}$. Next, a straight line with a slope equal to $S_{j,ini}/10$ and tangent to the momentrotation curve is drawn. The yield point corresponds to the intersection point between these two straight lines. Values of $\theta_{j,u}$ are obtained by the intersection of the moment-rotation curve in Figure 3.84 (after $M_{j,u}$) with a straight horizontal line of value equal to $0,8M_{j,u}$ at the origin. For some cases, this intersection point does not exist and $\theta_{j,u}$ is equal to the maximum value reached during the test.

Table 3.14: Resistances, initial stiffness and rotation capacities of SPE test series					
Test	Plastic bending	Ultimate bending	Initial stiffness	Rotation capacity	
	moment $M_{j,pl}$ (kNm)	moment $M_{j,u}$ (kNm)	$S_{\rm j,ini}$ (kNm/rad)	$\theta_{j,u}$ (mrad)	
SPE1-M0	37,7	43,2	4117,4	44,7	
SPE2-M0	42,9	48,5	7189,9	33,2	
SPE1-M90	28,2	33,3	2028,7	69,5	
SPE2-M90	35,9	40,2	3120,5	48,1	
SPE1-M45	32,0	39,4	2440,5	62,7	
SPE2-M45	36,6	47,3	3205,0	54,8	



Figure 3.84: Definition of the experimental yield point

The moment-rotation curves were also obtained from the vertical LVDT's readings for tests SPE1-M0 and SPE2-M0. The displacement at the column end comprises the flexural and the shear displacements of the column in addition to the displacement produced by the connection and the concrete block rotation. The rotation of the connection obtained from the vertical displacement measured by LVDT's reads is given by:

$$\theta_{\rm j} = \frac{(\Delta_{\rm l} - \frac{FL^3}{3EI} - \frac{FL}{GA_{\rm v}} - \Delta_{\rm 4})}{L} - \gamma_{\rm b}$$
(3.3)

with

- θ_j : rotation of the connection,
- Δ_1 : vertical displacement read by the displacement transducer V₁,
- *F* : applied force,
- L : distance between the load-jack and the connection,
- E: elasticity modulus of the column,
- *I* : inertia of the column,

G: shear modulus of the column,

- $A_{\rm v}$: shear area of the column cross-section,
- Δ_4 : vertical displacement read by the displacement transducer V₄,
- $\gamma_{\rm b}$: concrete block rotation obtained from $\frac{|V_6 V_7|}{L_6 T}$ (see Figure 3.11).

Curves were plotted and compared to those depicted in Figure 3.82 and Figure 3.83. It can be seen that up to $M_{j,u}$, the shape of the curves is similar, proving that the horizontal and vertical displacements measurements are consistent (see Figure 3.85).



Figure 3.85: Comparison of the moment rotations curves obtained from Equation 3.2, Equation 3.3 for M0 test series

From the curves depicted in Figure 3.82 and Figure 3.83, it can be concluded that the ultimate bending moment, the initial stiffness and the rotation capacity strongly depend on the applied loading case and the base plate thickness. For each base plate thickness, a trend can be observed.

For cases with thicker base plates, the plastic/ultimate bending moments and initial rotational stiffness are visibly higher. For minor-axis bending, which was the most severe loading case in terms of ultimate bending moment and initial stiffness for both base-plate thicknesses, the difference between the maximum bending moments was approximately 20%. As far as the initial rotational stiffness is concerned, this variation is even more pronounced. Indeed, the initial stiffness for the 20 mm base plate configuration is approximately three times that of 10 mm base plate thickness. This is due to the fact that increasing the base plate thickness results in a more rigid base plate behavior. The nature of the base plate behavior during the tests exercised a strong influence on the position of the center of compression. It can be seen in Figure 3.86 that if the thickness is higher and hence more rigid, the center of compression tends to move away from the footprint of the column flange towards the edge of the base plate. On the other hand, for thinner and more deformable base plates, the yield line in the compression side tends to develop near the column flange leading to a shorter distance between $F_{c,Rd}$ and $F_{t,Rd}$. The development of yield line in the compressive area has not been demonstrated experimentally but observed in the numerical simulations. As mentioned before, the contribution of the base plate thickness to the rotation capacity is substantial. For tests with a thicker base plate, yielding of the anchor bolts may occurred before yielding of the base plate which developed smoothly with the formation of a yield line along the base plate in the compressive side.



Figure 3.86: Variation of the position of $F_{t,Rd}$ and $F_{c,Rd}$ according to the base plate thickness

As the moment resistance was calculated from Equation (3.4), logically, the greater is the distance *z* between $F_{t,Rd}$ and $F_{c,Rd}$, the greater is the moment resistance:

$$M_{j,Rd} = 2 \times F_{t,Rd} \times z \tag{3.4}$$

with

 $M_{i,Rd}$: moment resistance,

 $F_{t,Rd}$: anchor bolt design resistance,

z: lever arm between $F_{c,Rd}$ and $F_{t,Rd}$.

The loading case also influences the main characteristics of the moment-rotation curves (see Figure 3.83). Test results show that, on one hand, specimens subjected to major-axis bending

(M0 series) revealed greater bending moment resistances and initial rotational stiffness but lower rotation capacity (see Table 3.14). On the other hand, specimens under minor-axis bending (M90 series) were the weakest cases from the bending moment capacity and initial stiffness point of view but with higher rotation capacity. For all, connection failure was due to anchor bolt failure in tension with or without base plate yielding in bending. The main difference concerns the lever arm between the center of tension and the center of compression. This lever arm is shorter for series M90 than series M0 and thus the bending moment resistance decreases for a constant tensile resistance of the anchor bolts.



Figure 3.87: Variation of the position of $F_{t,Rd}$ and $F_{c,Rd}$ according to bending moment orientation

3.7.1.4 Lateral displacements of the base-plate

Image correlation have been used to evaluate the relative displacement between the concrete block and the base plate and also to estimate the deformed shape of the base plate. A photograph was taken every 20 seconds. These photographs were then processed using a specific program for digital image correlation GOM Correlate. The outputs are an estimation of the horizontal displacements of points (markers) along the base plate. These results were used as a complementary information to evaluate the area of the compressive and tensile zones and better understand the development of prying effect.

The initial deformations as well as the lateral displacements (see Figure 3.88 and Figure 3.89) were recorded for specimens SPE1-M0/90, SPE2-M0/90. In addition, the displacement of the top of the base plate of specimens SPE1-M90 and SPE2-M90 was measured. It is noted that the given results correspond only to the displacement on the lateral parts of the base plate.





(M90) and at the concrete surface



Figure 3.90: Location of the photographic devices for digital image correlation (front view)

For SPE1-M0, the displacement of the base plate is quite substantial in the tensile area (Figure 3.92) and a yield line develops close to the tensile beam flange. After failure, bending residual deformations of the base plate can be clearly seen, confirming its yielding during the test. Once F_{max} is reached, the anchor bolts began to elongate excessively until failure. This phenomenon can be captured from the evolution of the base plate deformation. In the lower part, the initial gap due to initial imperfections was closing during loading until contact with the concrete foundation was achieved. At the beginning of the loading, the uplift point is positioned somewhere between 90 mm and 120 mm from the bottom of the end-plate, the bottom beam flange being at 70 mm from this point. The contact area developed also over the entire area of the bottom extend end-plate but also over about 25% of the column depth. This uplift point move down when yielding starts to develop in both the base-plate and anchor bolt as a result of the modification of the ratio between the stiffness of compressive components and the stiffness of the tensile components. Once the maximum force is reached, the contact area extend towards the end plate edges. It can also be observed that in the tensile area the point of inflexion is not located at the weld toe as supposed in EN 1993-1-8 but behind the upper beam flange.



Figure 3.92: Evolution of the lateral base plate displacements with initial imperfections for SPE1-M0

In the tensile area, the base plate of specimen SPE2-M0 remains rigid during the whole test (see Figure 3.93). One may conclude that the base plate remained elastic throughout the test. At the end of loading, the extend of the contact area is comprised between 40 and 50 mm. Accordingly, the assumption of EN 1993-1-8 on the position of the center of compression is not valid.



Figure 3.93: Evolution of the lateral base plate displacements with initial imperfections for SPE2-M0



Figure 3.94: Evolution of the lateral base plate displacements with initial imperfections for SPE1-M90

For specimen SPE1-M90, the deformation of the base plate was also very significant (see Figure 3.94). The drawn deformed shapes indicate the existence of a yield line around the activated anchor bolts row and at the level of the column flange tips in compression. Since the bottom anchor bolt row was above the contact area, the contribution of these anchor bolts to the load carrying capacity is confirmed. The base plate remain in contact with the concrete surface behind the upper anchor bolts. This confirm the development of prying effect. The contact is

released when the maximal force is reached due to the excessive anchor bolt elongation. The negative displacement obtained in this tensile part is due to closing of the initial imperfection caused by welding process.

From the plotted deformed shapes, it can be concluded that, at the end of the test, contact between the base plate and the concrete block in the compressive part was maintained over 60 mm from the bottom of the base plate. However at early stages of loading, this contact length was 90 mm comprising a portion of the column cross-section depth. The deformed shape of the top of the base plate was concave (see Figure 3.96). After failure, the base plate kept the same deformed shape confirming yielding of the element.

Similarly to SPE2-M0 specimen, the behavior of the base plate of specimen SPE2-M90 was rigid and anchor bolts were the only source of deformation. A maximum uplift of 12 mm was recorded at the upper free edge of the base plate. The displacements of the base plate in the lower part were essentially due to the initial gap closing at the end of the test resulting in the contact of the bottom edge of the base plate with the concrete surface.



Figure 3.95: Evolution of the lateral base plate displacements with initial imperfections for SPE2-M90



3.7.1.5 Strain distribution

The longitudinal strains measured on the column flanges of specimens SPE1-M90 and SPE2-M90 are presented in Figure 3.98. The positions of the strain gauges along the column flanges with their corresponding labels and the location of the plastic neutral axis are given in Figure 3.97 (the exact positions are given in Figure 3.13). The distribution of stresses and strains along the column flange width can be seen in Figure 3.99, Figure 3.100, Figure 3.101 and Figure 3.102.



Figure 3.97: Strain gauge designation



Strains obtained from strain gauges SG_14 , SG_r4 of specimen SPE1-M90 and SG_r4 of specimen SPE2-M90 are negative as they are located in the compressive part of the column flanges. Concerning SG_r3 of specimen SPE2-M90, negative values of the strain were measured for forces below 30 kN whereas positive values were recorded for larger values of the applied force corresponding to anchor bolt yielding and necking. This change of the stress sign is the result of the evolution of the relative stiffness between the tensile and compressive parts. With the yielding of anchor bolts, the tensile parts become more flexible and thus the size of the compressive area decrease. It can also be noticed that this decrease of the column width. The hypothesis of linear plane strain of the beam theory is more or less respected at the beginning of the loading.

A similar phenomena is observed with SG₁3 of specimen SPE1-M90 but the strain became positive for a force level of 10 kN. The initial preloading of anchor bolts is probably at the origin of the initial distribution of stresses. Once the effect of preloading is mitigated, ³/₄ of the column width works under tension. This phenomena appears sooner than in specimen SPE2-M90 as the tensile part of specimen SPE1-M90 is more flexible. Even if the tensile area increases during the test, the yielding of the edge of this part is observed. This effect is a consequence of the low flexibility of the base plate. The tensile stresses are thus located around the upper anchor bolts.



Figure 3.101: Stress distribution along the column flange height for SPE2-M90



3.7.1.6 Concluding remarks

The tests performed on six column base plates with different base plate thicknesses and bending moment orientations allowed to gain further insight into the behavior of this type of connections. The main conclusions are:

- the bending moment resistance and the initial stiffness of the connection increases with increasing value of the base plate thickness whatever the orientation of the bending moment,
- the bending moment resistance and the initial stiffness of the connection increases as the distance between the centers of compression and tension *z* increases (higher for major-axis bending and shorter for minor-axis bending),
- the rotation capacity of the connection increases with decreasing values of the base plate thickness,

- the base plate thickness and the orientation of the bending moment have limited influence on the behavior of the connection after reaching the maximum force as the necking of the anchor bolts develops in these cases,
- the redistribution of the internal forces to the anchoring system is strongly dependent on the base plate thickness,
- the orientation of the bending moment have an effect on the development of prying forces,
- the spread of the compressive area is strongly dependent on the base plate thickness and on the orientation of the applied bending moment.

3.7.2 Beam-to-concrete slab connections experimental program

3.7.2.1 Description of the tests

• Test PB-1

During elastic phases 1 and 2, the specimen remained in the elastic domain. Next, loading was increased until failure at a rate of 1 mm/min. This loading rate was continuously increased until the end of the test. At a load level of 27 kN, an uplift of 2 mm between the base plate and the concrete surface began to be visible. When the applied force reached 35,6 kN, the first cracks appeared on the surface of the concrete block and the uplift of the base plate with respect to the concrete block was about 3mm. At 40 kN, the embedded plate started to emerge from the concrete block and the cracks on the concrete surface began to spread. At 58,2 kN, the deformation of the base plate was substantial and the gap between the base plate and the concrete in the vicinity of the footprint of the tensioned column flange was about 5 mm (see Figure 3.103 a)). The maximum capacity of the connection was reached for a vertical force equal to 61,7 kN. At this load level, the base plate deformations were considerable (see Figure 3.103 b). The embedded plate moved out from the concrete block (see Figure 3.103 c)). After that, the load began to decrease due to necking while the deformation of the tensioned bolts increased. The test stopped with the failure of both upper anchor bolts. After the test, significant yielding of the base plate and cracks in the welds and on the concrete surface were visible (see Figure 3.103 d) and e)). The contact between the top of the end plate and the embedded plate was maintained during the whole test meaning that prying effect was developing. It can be noted that specimen PB-1 is equivalent to specimen SPE2-M0 as these specimens have M16 fasteners, 20 mm base plate thickness and a profile (I or H) of 190/200 mm height. The two main differences are the base plate width and the fastening to concrete support. In contrast to specimen PB-1, prying effect did not develop with specimen SPE2-M0 as the base plate is more rigid (due to greater width) and the fasteners more flexible.



Base plate deformation at 58,2 kN (side) a)



Base plate deformation at 61,7 kN (side) b)



Embedded plate detachment at 61,7 kN c)



d) Cracks on the welds



Cracks on the concrete at the end of the test e) Figure 3.103: Specimen PB-1 deformation during and after the test

Test PB-2 •

During the first loading/unloading cycles within elastic phases 1 and 2, no yielding of the steel elements or cracks on the concrete surface were observed. This was expected as the assembly was still working in the elastic domain. Thereafter, the load was increased until failure at a rate of 1 mm/min. This rate was appropriately and continuously adjusted until the end of the test. When the applied force reached 43 kN, the first cracks appeared on the surface of the concrete block. The deformation of the base plate began to be visible at 56 kN and at the same time, the embedded plate started to move out from the concrete block. The maximum force resisted by the connection was 74,6 kN. At this load level, the connection deformations were largely visible. The embedded plate was out of the concrete block. The crushing of the base plate against the concrete measured by U_{b1} and U_{b2} was approximately 0,7 mm. After that, the load began to decrease with increasing value of the displacements due to bolts necking. The test stopped with the failure of both activated anchor bolts. After the test, large yielding of the base plate and cracks in the welds and on the concrete surface were visible. Furthermore, the corner of the upper outer edge of the end-plate was in contact with the concrete foundation which means that prying effect developed.



a) Base plate deformation at 56 kN (top)



b) Base plate deformation after failure (top)





c) Base plate deformation after failure (side)
 d) Cracks on the welds
 Figure 3.104: Specimen PB-2 deformation during and after the test

• Test PB-4

During the first four cycles, the specimen was loaded up to 10 kN (phase 1) and 20 kN (phase 2). Some residual deformations were observed at the end of phase 2. Similarly to the previous tests, the test proceeded with a cycle in which the vertical load was increased until the failure of the connection. The main difference in the configuration between specimen PB-4 and PB-2 was the type of anchoring system. This test allowed to evaluate the influence of the anchoring system on the performance of the connection. It was found that the deformation of the base plate and the initiation of cracks in the concrete occurred for a load level well under the previous

test. At 32,3 kN, cracking was already at an advanced stage. The base plate started to slightly deform. As a consequence of excessive cracking and concrete spalling, the load decreased abruptly. Contact did not develop at the outer edge of the upper end-plate and prying effect was not present in contrast to what was observed in specimen PB-2. This can be explained by the fact that the tensile force was transferred by more flexible fasteners.



a) Cracks in the concrete surface at 35,6 kN



c) Cracks in the concrete at the end of the test



b) Base plate deformation at 35,6 kN (top)



d) Base plate deformation at the end of the test (top)



e) Base plate deformation at the end of the test (side) Figure 3.105: Specimen PB-4 deformation during and after the test

3.7.2.2 Force-vertical displacement curves and failure modes

The failure modes observed during the tests can be divided into two types:

• rupture of the activated bolts with yielding of the base plate in bending (PB-1, PB-2),

• concrete cone failure (PB-4).

As mentioned in the previous sub-chapters, this aspect is directly related to the type of anchoring system. The redistribution of the internal forces between the anchoring system and the concrete reinforcement was quite different from test to test. In specimens PB-1 and PB-2, the embedded length and the presence of bolt washers allowed the steel elements to govern the overall behavior of the connection until failure, with benefits in terms of bending resistances, initial rotational stiffness and rotation capacity. The resistance and deformation capacity of specimen PB-4 are clearly smaller, due to the reduced embedded length and also to difficulties in transferring correctly the tensile force to the foundation. Table 3.15 summarizes the yielded elements at the end of the tests and the observed failure modes.

Table 3.15: Failure modes for PB tests						
Test ID	Maximal vertical force F_{max} (kN)	Yielded elements	Failure mode			
PB-1	61,7	Anchor bolts/base plate/welds	Anchor bolts (both)			
PB-2	74,8	Anchor bolts/base plate/welds	Anchor bolts (both)			
PB-4	40,7	-	Concrete cone failure			

The curves relating the applied load and the corresponding displacement measured by displacement sensor V_1 for tests PB-1, PB-2 and PB-4 are plotted in Figure 3.106.



Figure 3.106: Force displacement curves for specimens PB-1, PB-2 and PB-4

The curve from PB-2 test reached values of applied vertical force significantly higher than tests PB-1 and PB-4. This was essentially due to the presence of stiffeners and the configuration of the anchoring system. Although the thickness of the base plate in specimen PB-1 (20 mm) was higher than that in specimen PB-2 (15 mm), the existence of stiffeners as shown in Figure 3.8 and Figure 3.104 proved to be favorable to the resistance and initial stiffness of the connection. In fact, the absence of stiffeners favor prying effect in test PB-1. The development of these additional forces Q resisted by the activated anchor bolts led to failure of these elements for reduced values of the applied force. However, the absence of stiffeners increases the rotation

capacity of the connection thanks to the full development of a yield line parallel to the beam flange on the end-plate.

As mentioned above, the configuration of the anchoring system was a crucial aspect for the performance of the connections. Specimen PB-2 revealed to be considerably more resistant than PB-4 due to its anchoring system. For specimen PB-2, the bolts were the element governing failure of the specimen, producing a substantially more ductile behavior and resisting to higher levels of the applied force. For test PB-2, the loss of resistance after reaching F_{max} was mostly due to necking of the anchor bolts. Throughout the test and as a result of the plate thickness, a yield line in the tensile side of the connection developed. The existence of stiffening plates also influenced the shape of the deformed plate (see Figure 3.104). For specimen PB-4, the redistribution of internal forces in the anchoring system during the test was quite different. It was reflected on the behavior of the sudden load drop after reaching the maximum force is quite different from that of the previous test, as explained below. In this test, there was a significant loss of concrete strength, which was in a very advanced cracking state when the decision of stopping the test was taken. This fact contributed to the brittle behavior of the connection after reaching F_{max} , causing the sharp decrease in the curve (see Figure 3.106).

3.7.2.3 Moment-rotation curves

The moment rotation curves of specimens PB-1, PB-2 and PB-4 are depicted in Figure 3.107. The initial rotational stiffness, the plastic/ultimate bending moments and the rotation capacity are also given in Table 3.16.

	Table 3.16: Resistances, in	nitial rotational stiffness's	and rotation capacities of	PB test series
Test	Plastic bending	Ultimate bending	Initial stiffness	Rotation capacity
	moment $M_{j,pl}$ (kNm)*	moment $M_{j,u}$ (kNm)	S _{j,ini} (kNm/rad)*	$\theta_{\rm j,max}({ m mrad})^*$
PB-1	56,0	71,0	5433,8	85,4
PB-2	66,5	82,3	7984,0	51,2
PB-4	41,6	44,7	4398,3	14,1

*Values obtained from Figure 3.84.



Laura da Silva Seco

The plastic and ultimate bending moments for test PB-2 are about 20% higher than that of PB-1 and 80% higher than that of PB-4. The resistance of specimen PB-2 is increased comparatively to specimen PB-1 as prying effects decreases due to the presence of stiffeners. However, the rotation capacity of specimen PB-2 is less than that of specimen PB-1. The ductility of specimen PB-1 is greater than that of specimen PB-2. The failure mode of PB-1 is a pure Mode 2 whereas specimen PB-2 fails according to a combination of Mode 2 and Mode 3. The rotation capacity of specimens PB-2 and PB-1 is above 50 mrad and these connections can be considered as ductile. This is not the case for specimen PB-4, as its rotation capacity is equal to 14,1 mrad and involve a non-ductile failure mode.

The use of an embedded plate, washer, and stiffeners clearly improve the mechanical characteristics of the connection. For specimen PB-4, the non-linear behavior result mainly from cracking of the concrete block. The steel components remain probably elastic during the whole test. For PB-1 and PB-2, the non-linearity results from yielding of both the end-plate and the bolts in tension.

For specimen PB-4, the moment increases linearly to a value close to 30 kNm. At this load level, the first cracks began to appear on the surface of the concrete slab. The inefficiency of the anchoring system to resist to the internal forces and transfer to the reinforcement results in crack initiation which spread rapidly. This phenomenon had implications not only on the resistance and rotation capacity of the connection but also on the shape of the moment-rotation curve. The latter is characterized by a sudden increase of resistance until reaching $M_{j,u}$ followed by a sharp decrease, typical of a brittle failure due to concrete cracking/spalling.

Concerning the initial rotational stiffness, as said earlier, the use of stiffeners visibly influence and improve the initial stiffness of the specimen PB-2 over specimen PB-1. The use of an embedded plate and an end washer anchoring system rather than simple post-installed anchor bolts as in PB-4 lead to a significantly more rigid behavior of the connection.

3.7.2.4 Lateral displacements of the end-plate

Digital image correlation was also used to evaluate the lateral displacements of the base plates. The evolution of the horizontal displacements during tests are presented in Figure 3.108, Figure 3.109 and Figure 3.110.

According to the figures below, it can seen that the deformation of the base plate was significantly higher for specimens PB-1 and PB-2 with a maximum uplift value of 11 mm at the level of the upper bolt row. The horizontal displacements noticeably increased after reaching the maximum force F_{max} thanks to bolts necking. As it has been said earlier, prying developed in test PB-1. From Figure 3.108, an uplift can be observed at the top of the base plate where the marker was located. However, this position does not coincide exactly with the extremity of the base plate explaining why an uplift is observed on the graph. Additionally, for high levels of load, the embedded plate detached and moved away from the concrete surface.

Regarding specimen PB-2, it is found that throughout the test, the upper end of the base plate gradually moved away from the concrete surface as well as the embedded plate (see Figure 3.109). Prying may have developed in the corners. Despite the occurrence of a yield line at the level of the tensioned bolts, the base plate did not return to contact with the concrete slab. Comparing the compressive zones pertaining to specimens PB-1 and PB-2 in Figure 3.108 and Figure 3.109, it is realized that the extend of the contact zone is greater for the case without stiffeners. Although specimen PB-2 has a thinner base plate, the stiffeners limited the deformation in bending of the base plate during the test, pushing the center of compression towards the lower free edge. On the other hand, the base plate of specimen PB-1 was thicker but without stiffeners, its deformability was higher, allowing the center of compression to develop closer to the column flange.



Figure 3.109: Evolution of the lateral plate horizontal displacements for PB-2

The deformations of the plate of specimen PB-4 were less significant compared to that of specimen PB-2. The evolution of the distribution of the horizontal displacements suggest that the yielding of the plate was limited as a result of the brittle failure mode that occurred before steel elements started to yield.



Figure 3.110: Evolution of the lateral plate horizontal displacements for PB-4

3.7.2.5 Concluding remarks

The results of the tests on beam-to-concrete slab connections considering different base plate thickness and anchoring system allowed to have a better understanding of the impact of such parameters on the overall behavior of the connections. The following conclusions could be drawn from the testing programme:

- the bending moment and initial stiffness increases with the presence of stiffeners;
- the bending moment, initial stiffness and rotation capacity increases with increasing value of the embedded length of the bolts, with the presence of an embedded plate sufficiently reinforced and, with the presence of bolt washers embedded in the concrete slab,
- prying effects are strongly influenced by the presence of stiffeners,
- the redistribution of the internal forces between the anchoring system and the concrete slab reinforcement is strongly dependent on the embedded length of the anchor bolts,
- the anchoring system (length of the embedded anchor bolts, bolt washers, embedded base plate) has great influence on the behavior of the connection during the elastic and plastic phases until failure.

APPENDIX A

A.1 Concrete block reinforcement of the column base plates

• Specimens tested under in-plane bending moment (SPE1-M0; SPE2-M0)







• Specimens tested under out-of-plane bending moment (SPE1-M90; SPE2-M90)

APPENDIX A





• Specimens tested under biaxial bending moment (SPE1-M45; SPE2-M45)



A.2 Concrete block reinforcement of the beam-to-concrete slab connections

• Specimens PB-1




• Specimen PB-2 and PB-4





• Anchoring system details of specimens PB-1 and PB-2



4. NUMERICAL STUDY

4.1 Introduction

The behaviour of column base plates is known for being highly complex involving multiple phenomena such as geometrical, material and contact non-linearity's resulting from the interaction of the different components that constitute the joint. For a better understanding of these structural elements under various loading conditions and considering different typologies and materials, it is crucial to develop an accurate numerical model capable to capture the response of the connection and to provide further insight into the behaviour of its components by analysing the different stress states to which the joint is subjected when statically loaded. Transfer of internal forces, interactions between interfaces in contact, performance of the anchoring system, damage of the concrete foundation and yielding/failure of steel elements are examples of situations that can be easily and quite accurately predicted by numerical model, avoiding time-consuming and expensive experimental studies. Moreover some phenomena such as contact pressure, cannot be measured during experimental tests but the numerical model provide these information's which are crucial for the development of an accurate and precise analytical model.

This chapter presents a detailed description of the numerical models created with the Finite Element software ABAQUS in order to investigate the behaviour of column base plates and beam-to-concrete slab connections under different loading conditions. The numerical results are compared against the experimental test data presented in Chapter 3 and used to calibrate the analytical model developed in Chapter 5. Additionally, an extensive parametric study is presented and the influence of parameters such as the base plate thickness, the column steel profile and the arrangement of the anchoring systems on the resistances, initial rotational stiffness and rotation capacity is analysed.

4.2 Model definition

4.2.1 Geometry

All connection components (base/end-plate, column/beam, weld, concrete, anchors ...) except the reinforcing bars, are modelled as three-dimensional deformable solids using eight-node linear brick elements. Two-node linear truss elements are used for the concrete reinforcement. The actual (measured) dimensions of the tested specimens have been considered in the finite element model.

4.2.1.1 Column base plate model

The mean measured dimensions considered in the model are given in Table 4.1 and Table 4.2. The anchoring system consist in four M16 anchor bolts which are modelled with nuts and

washers, and assuming a constant cross section throughout their length equal to the threaded area A_s .

Table 4.1: Measured geometry of the column (HEA 200)							
Test ID	$h_{\rm c}$ (mm)	$b_{\rm c} ({\rm mm})$	$t_{\rm fc}({\rm mm})$	$t_{\rm wc}({\rm mm})$			
SPE1-M0	194,5	200,5	6,5	10,8			
SPE2-M0	191,0	201,0	6,5	11,0			
SPE1-M90	192,0	200,5	6,5	10,3			
SPE2-M90	193,0	202,0	6,5	10,0			
SPE1-M45	194,5	200,0	6,5	10,0			
SPE2-M45	193,0	202,0	6,5	10,0			

Table 4.2: Measured geometry of the steel base plates								
Test ID	$h_{\rm p}~({\rm mm})$	$b_{\rm p}$ (mm)	t _{bp} (mm)	$d_0 (mm)$	<i>e</i> (mm)	$e_{\rm x}$ (mm)	<i>p</i> (mm)	<i>w</i> (mm)
SPE1-M0	330,0	300,0	10,1	18,0	70,0	34,0	262,0	160,0
SPE2-M0	329,0	299,0	20,0	18,0	70,5	35,5	258,0	158,0
SPE1-M90	330,0	299,0	10,0	18,0	70,0	35,0	260,0	159,0
SPE2-M90	330,0	299,0	20,0	18,0	70,0	36,0	258,0	159,0
SPE1-M45	330,0	300,0	10,0	18,0	70,0	35,0	260,	160,0
SPE2-M45	330,0	300,0	20,0	18,0	70,0	36,0	258,0	160,0



Figure 4.1: Column steel profile and base plate parameters

4.2.1.2 Beam-to-concrete slab connection model

For models of specimen PB-1 and PB-2, the embedded plate system with the shear lug is modelled as in the tested specimens (see Figure 4.2).



Figure 4.2: Anchoring system from PB-1 and PB-2 numerical models

4.2.2 Materials

In order to obtain numerical results with higher accuracy, the material non-linearity is considered and the experimental stress-strain relationship obtained from steel coupon tests is adopted. Regarding concrete, the evolution of concrete resistance over the time provided by the compressive tests is also considered.

4.2.2.1 Steel

For steel elements, such as the HEA and IPE columns, base/end-plates, embedded plate, anchor bolts and post-installed bolts, the material characteristics obtained from the coupon tests presented in Chapter 3 and given in Table 4.3 are used. Elasto-plastic behavior following the Von-Mises yield criterion with strain hardening is considered to model steel elements. The stress-strain curve introduced in the numerical models is multi-linear (see Figure 4.3), based on the proposition of Couchaux *et al.* (2017) that is built with the following stress-strain pairs: $(f_y;\varepsilon_y)$, $(f_y;\varepsilon_h)$, $(f_m;\varepsilon_m)$ and $(f_u;\varepsilon_u)$ taken from the tensile coupon tests. The behavior is assumed to be perfectly elastic up to yield strength f_y . Next, the behavior is elasto-plastic until ε_u . Material failure is simply characterized by an abrupt decrease of the stress to 10 N/mm².



True stress and true strain can be determined based on the nominal stress and strain as follows:

$$\sigma = \sigma_{\text{nom}}(1 + \varepsilon_{\text{nom}}) \tag{4.1}$$

$$\varepsilon = \ln(1 + \varepsilon_{\text{nom}}) \tag{4.2}$$

with

$$\sigma_{\rm nom} = \frac{F}{S_0} \tag{4.3}$$

$$\varepsilon_{\rm nom} = \frac{l - l_0}{l_0} \tag{4.4}$$

After reaching the ultimate strength f_m , Equations (4.3) and (4.4) are no longer valid due to necking and the ultimate stress and strain are obtained from Equations (4.5) and (4.6).

$$\sigma_{\rm u} = \frac{F_{\rm u}}{S_{\rm u}} \tag{4.5}$$

$$\varepsilon_{\rm u} = \ln\left(\frac{S_0}{S_{\rm u}}\right) \tag{4.6}$$

with

 $F_{\rm u}$, $S_{\rm u}$: ultimate tensile force and cross-sectional area of the coupon test at failure (taken from the information given by tests).



The relationship between the true stress and the true strain taken from Couchaux *et al.* (2017) (see Figure 4.3) is described by the following set of equations:

• For $0 \le \varepsilon \le \varepsilon_y$:

$$\sigma = E\varepsilon \tag{4.7}$$

• For $\varepsilon_{y} \leq \varepsilon \leq \varepsilon_{h}$:

 $\sigma = f_{\rm v} \tag{4.8}$

• For $\varepsilon_h \leq \varepsilon \leq \varepsilon_m$:

$$\sigma = f_{\rm y} - c_{\rm m} + \left(\frac{b_{\rm m}}{a_{\rm m}}\right) \sqrt{a_{\rm m}^2 - (\varepsilon_{\rm m} - \varepsilon)^2} + E_{\rm u}(\varepsilon_{\rm m} - \varepsilon_{\rm h})$$
(4.9)

• For $\varepsilon_m \leq \varepsilon \leq \varepsilon_u$:

$$\sigma = f_{\rm m} + E_{\rm u}(\varepsilon - \varepsilon_{\rm m}) \tag{4.10}$$

with

$$a_{\rm m}^2 = (\varepsilon_{\rm m} - \varepsilon_{\rm h})(\varepsilon_{\rm m} - \varepsilon_{\rm h} + c_{\rm m}/E)$$

$$b_{\rm m} = \sqrt{c_{\rm m}(\varepsilon_{\rm m} - \varepsilon_{\rm h})E + c_{\rm m}^2}$$
$$c_{\rm m} = \frac{\left(f_{\rm m} - f_{\rm y}\right)^2}{(\varepsilon_{\rm m} - \varepsilon)E - 2\left(f_{\rm m} - f_{\rm y}\right)}$$
$$E_{\rm u} = \frac{f_{\rm u} - f_{\rm m}}{(\varepsilon_{\rm u} - \varepsilon_{\rm m})}$$

Table 4.3: Material properties introduced in the FE models							
Connection	Elements	f_y (MPa)	$\boldsymbol{\varepsilon}_{\mathrm{h}}\left(\% ight)$	$f_{\rm m}$ (MPa)	$\varepsilon_{\mathrm{m}}(\%)$	f_u (MPa)	$\boldsymbol{\varepsilon}_{\mathrm{u}}\left(\% ight)$
	Base plate 10 mm	423,6	2,6	599,8	15,7	1146,1	121,9
Column hass plate	Base plate 20 mm	401,3	1,7	581,6	14,5	1196,9	125,4
(Spol M0)	Column web	378,4	2,7	525,3	13,9	1048,0	92,6
(Spe1-1010,)	Column flange	350,1	2,6	504,4	15,2	1240,0	116,3
	M16	520,4	0,1	626,8	1,4	1367,9	127,5
	End plate 15 mm	277,5	0,8	534,7	17,9	1032,2	120,3
Deem to concrete	End plate 20 mm	229,1	0,7	485,0	20,1	773,9	98,9
slab (PB-1/2)	Beam web	340,1	2,1	575,6	17,4	976,4	102,7
	Beam flange	333,0	1,8	582,6	17,3	989,5	106,0
	M16	1063,5	0,5	1121,0	3,5	1913,0	115,8

In order to validate the material parameters, standard tensile tests (see Figure 3.16 and Figure 3.22) are simulated with the FE program and the numerical results are compared against the corresponding experimental data. Cross-sections of the samples are modelled with a total of 20 elements for the round coupons and 24 elements for the rectangular samples. Good agreement between the numerical and experimental results for tensile tests can be observed in Figure 4.6 and Figure 4.7.



Figure 4.6: Modelling of coupon tests of anchor bolts and bolts



4.2.2.2 Concrete

Standard cylinder tests presented in Chapter 3 enabled an estimation of the evolution of the maximum compressive strength f_{ck} . The concrete behavior law herein considered (see Figure 4.8) taken from EN 1992-1-1, is expressed by:

$$\frac{\sigma_{\rm c}}{f_{\rm cm}} = \frac{k\eta - \eta^2}{1 + (k - 2)\eta}$$
(4.11)

with

$$f_{cm} = f_{ck} + 8,$$

$$\eta = \varepsilon_c / \varepsilon_{c1},$$

$$k = 1,05 E_{cm} \times |\varepsilon_{c1}| / f_{cm}.$$

Concrete Damaged Plasticity model available on ABAQUS library is used to model the concrete material behavior. CDP model is based on the Drucker-Prager strength criterion and allows to define the plasticity by damage parameters (see Table 4.5) as well as the behaviour in



tension and compression of the concrete (see Figure 4.8). Nominal values suggested in EN 1992-1-1 are used to define the stress-displacement law in tension.

rigure 4.0. Concrete suess-strain law i	of compression and	a suess-uispiacement iaw	for tension

Table 4.5: Concrete damaged plasticity parameters							
Dilation angle (°)	Eccentricity	$f_{ m b0}/f_{ m c0}$	K	Viscosity parameter			
36	0,1	1,16	0,667	0			

The parameters used in the CDP model for all models are the ones recommended in ABAQUS user's guide. The dilation angle is described as the concrete internal friction angle and usually assumes values between 30° and 40°. The eccentricity is obtained by the ratio between the tensile strength and the compressive strength (recommended value equal to 0,1). The strength ratio between the biaxial state and the uniaxial state f_{b0}/f_{c0} , represents the point where concrete failure occurs under biaxial compression. Kupler experimental tests in 1969 indicated that the uniform biaxial compression strength f_{b0} can be considered equal to 1,16 f_{c0} . ABAQUS user's guide suggests to assume f_{b0}/f_{c0} according to Kupler's results. From the deviatoric cross section of the failure surface of the CDP model (see Figure 4.9), K is defined as the ratio between the distances amongst the hydrostatic axis and the compression and tension meridian (C.M. and T.M.). It can be observed that this ratio assumes values between 0.5 and 1. The maximum value corresponds to the case when the deviatoric cross section of the failure surface is circumscribed into the Drucker-Prager surface (circle). Through experimental tests, Majewski assumed this value equal to 0,6 for a zero mean normal stress, increasing as the mean stress decreases Kmiecik and Kaminski (2011). Michal and Andrzej (2015) demonstrated that the viscosity parameter should not be assumed other than zero, preventing inaccurate results. Adopting higher values leads to a wider damage area, originating a dispersed cracking pattern, which do not represent the reality.



Figure 4.9: Deviatoric cross section of failure surface in the CDP model

4.2.2.3 Bond stress-slip law

A special attention was given to the contact between the anchor bolts surface and the concrete block, since the way these elements interact with each other strongly affects the overall stiffness of the specimens. For that, it is necessary to adopt a bond stress-slip law able to predict realistically the behavior at the interface between these two elements. Bond properties of the embedded anchor bolts are defined by a local bond-slip relationship. This bond-slip law, for monotonic loading conditions, adopted by FIP Model Code 2010 (International Federation for Structural Concrete (fib), 2010) is expressed by the following equations. Shear stress τ , developed at the interface between the anchor bolt and the concrete, depends on the relative displacement (slip) *s* between these two elements.

• For $0 \le s \le s_1$:

$$\tau = \tau_{\max} (s/s_1)^{\alpha} \tag{4.12}$$

• For $s_1 \leq s \leq s_2$:

$$\tau = \tau_{\max} \tag{4.13}$$

• For $s_2 \leq s \leq s_3$:

$$\tau = \tau_{\max} - (\tau_{\max} - \tau_f)(s - s_2)/(s_3 - s_2)$$
(4.14)

• For $s_3 < s$:

with

 τ , τ_{max} , τ_{f} : shear stress limits obtained from FIP Model Code 2010 (see Figure 4.10), s_1 , s_2 , s_3 : relative displacements limits taken from FIP Model Code 2010 (see Figure 4.10).

 τ_{f}

(4.15)



Figure 4.10: Bond stress-slip law from FIP Model Code 2010 (International Federation for Structural Concrete (fib), 2010)

 Table 4.6: Shear stress and slip limits for smooth bars (hot rolled bars) according to FIP Model Code 2010

 $\tau_{max} (N/mm^2)$ $\tau_f (N/mm^2)$ $s_1 = s_2 = s_3 (mm)$

 0,36(f_{ck})^{0.5}
 0
 0,1

4.2.3 Steps

The loading is applied in two subsequent steps. The initial step is defined to set the contact interactions between the elements and the boundary conditions. The second step is used to reproduce the loading conditions from the experimental tests. For that, a displacement perpendicular to the centre line of the column steel profile is applied at the location of the loadjack. As a consequence, an increasing bending moment at the column base plate with direction of the applied displacement is created up to failure. As it will be seen later, for the parametric study in which the models are subjected to a combination of axial force and bending moment, three steps are required: the initial step, Step-1 for the application of the axial force and Step-2 for the application of the aforementioned displacement (bending moment).

Steps are created as *Static General*. This type of analysis has been found to be the most suitable for the considered cases since it can be linear or nonlinear and is assumed when inertia and time-dependent material effects (as creep, swelling or viscoelasticity) can be ignored. Option *Nlgeom* is activated to take into account nonlinearities inherent to large-displacements.

4.2.4 Interactions

Contact interactions between elements strongly affect the computing process (time, convergence, accuracy...). In order to obtain numerical results with great accuracy, special attention must be given to contact due to the highly nonlinear and non-smooth nature of the phenomenon. For the same model and depending on the applied loading, the contact conditions between elements can vary widely and consequently affect the internal forces distribution. Interactions in ABAQUS are characterized as step dependent and thus, must be defined in the

correct analysis steps. To create it, a contact pair between two rigid or deformable threedimensional surfaces has to be defined. Although it is not necessary to guarantee matching meshes on the connected surfaces, the establishment of a "master" and a "slave" surface is required. As master surfaces, analytical rigid surfaces and rigid-element-based surfaces, smaller surfaces in case of contact with a larger surface, stiffer body surface, and coarser mesh surface should be considered. Regarding the sliding formulation, the main difference between smallsliding and finite-sliding contact is the way the contact area and the contact pressures are calculated. For the first case, the analysis is based on the undeformed configuration of the model. On the opposite side, finite-sliding contact considers the deformed configuration of the model.

Several contact interactions between elements are created in the models. The types of contact interactions are listed below:

- "rigid body" constraint: this type of constraint is used to create a rigid cross-section at the top of the column where the load is applied. The reference point is located at the geometric centre of the column cross-section for all models. This constraint allows to guarantee the uniformity of imposed displacement throughout the section. According to Figure 4.11 and Figure 4.12, this constraint corresponds to constraint 1,
- "tie" constraint: this type of constraint is used to tie two surfaces in contact during the simulation. Constraints involving two surfaces connected by a weld as 2, 3 and 6 in Figure 4.11 and 2 and 3 in Figure 4.12 are created using "tie" constraint. Also, as explained in Chapter 3, it is considered that a considerable part of the resistance of the anchoring system results from to the embedment of the washer plates in the concrete. This type of constraint is adopted (constraint 10 and 7 in Figure 4.11 and Figure 4.12, respectively),
- steel-steel interaction: to model the contact between two steel surfaces (4, 5 and 7 in Figure 4.11 and 4 in Figure 4.12), a surface-to-surface discretization method with finite sliding formulation is considered with a friction coefficient equal to 0,3. For normal behavior, the hard contact is selected allowing separation and preventing penetration of surfaces in contact,
- steel-concrete interaction (8 in Figure 4.11 and 5 in Figure 4.12): same as above with a friction coefficient equal to 0,5,
- steel-concrete with bonding interaction: to model the contact between the lateral surface of the anchor bolts and the concrete block (9 in Figure 4.11; 6 and 8 in Figure 4.12), a surface-to-surface contact type is selected with finite sliding formulation. For further details about the shear stress-relative displacement relationship, see subsection 4.2.2.3. The shear stress limit τ_{max} is obtained from Table 4.6. The normal behavior is defined as a normal steel-concrete interaction,
- "embedded region" constraint: in order to create the contact interaction between the concrete and the steel reinforcement, an "Embedded region" type constraint is adopted,

with the steel reinforcement as embedded elements and the concrete foundation as the host element (constraint 11 and 9 in Figure 4.11 and Figure 4.12 respectively).



Figure 4.11: Interactions considered for the column base plate models

with

- 1. top of the column loading plate,
- 2. column side plates welds,
- 3. column circular plate welds,
- 4. circular plate bolt nut,
- 5. circular plate (upper part) circular plate (lower part),
- 6. column base plate welds,
- 7. anchor bolt washers base plate,
- 8. base plate concrete foundation,
- 9. anchor bolt concrete foundation,
- 10. anchor bolt washers concrete foundation,
- 11. reinforcement rebars concrete foundation.



Figure 4.12: Interactions considered for the beam-to-concrete slab connection models

with

- 1. top of the beam loading plate,
- 2. beam stiffeners welds (for PB-2),
- 3. beam end plate welds,
- 4. bolt nuts end plate,
- 5. end plate/embedded plate concrete slab,
- 6. bolts/post-installed bolts concrete slab,
- 7. bolt washers concrete foundation,
- 8. shear lug concrete slab (for PB-1 and PB-2),
- 9. reinforcement rebars concrete slab.

4.2.5 Loading and boundary conditions

As described in Chapter 3, specimens are subjected to in-plane bending M0, out-of-plane bending M90 and bending at 45°. In a displacement-controlled simulation, the externally applied load is simulated by imposing a vertical displacement, which is increased until failure of one or more elements. This displacement is applied at the reference point created at the geometric centre of the cross-section, to which all nodes from the surface are rigidly connected. For the parametrical study, presented in sub-chapter 4.4, the influence of an axial force combined with bending is also analysed. For these cases, an additional concentrated load is applied, as explained in sub-section 4.2.3.

Boundary conditions are set for the concrete foundation/slab, the column base plate and the beam-to-concrete slab connection models in order to create the same support conditions as in the experimental tests described in Chapter 3. Vertical displacements are blocked at the neoprene supports of portal frame A and at the rollers attached to portal frame B (see Figure 3.11). To simulate the support conditions provided by the lateral steel plates placed in both portal frames, displacements are blocked in the horizontal direction, perpendicular to the portal frames.

4.2.6 Mesh

In order to reduce the computing time needed for the simulation, different mesh sizes are adopted according to the importance of the elements. From the experimental tests it was concluded that anchoring systems (anchor bolts, washers, embedded plates), and base/end plates are the elements that influence the most the behavior of the connections during the whole tests, until failure. As a consequence, a particular attention is given to the discretization of these elements. For the anchor bolts, in order to simulate the complex stress distribution during the computation until failure, a refined mesh is adopted considering the same number of elements for the cross-section as that used in the tensile tests simulations (subsection 4.2.2.1). To produce reasonable and physical sound stress distributions at the interface between the anchor bolts and the concrete, a smaller mesh size is selected for the layer of concrete material in contact with the anchor bolts. For the remaining concrete material, a coarser mesh is considered as this zone is of little interest.

In regions with high stress concentrations, such as welds, mesh size needs to be reduced. For this reason, four elements are used through the thickness. Similarly, in regions were buckling and/or bending are expected such as column flanges and base/end plates, three elements are assigned to the thickness of these elements (four for the 20 mm thickness base plates). Typically, this limits inaccuracies in the results and convergence problems due to the severe changes of stresses and strain distributions during the simulation. As the beam-to-concrete slab connection models are subjected to pure in-plane bending moment, only half of the connection is modelled (see Figure 4.14-b), resulting in reduced time analysis.



Figure 4.14. Examples of mesh discretization for the different elements of the beam-to-concrete stab connection models

As mentioned, besides anchor bolts, both base plates (10 mm and 20 mm) and the concrete block are elements that govern the behavior of the connections. For this reason, a mesh

sensibility analysis is carried out to evaluate the impact of both the number of elements across the base plate thickness direction and a refined mesh of the concrete block on the moment-rotation curve. The number of elements along the thickness direction adopted for the thinner base plate in specimen SPE1-M0 is increased from 3 to 5 and, in specimen SPE2-M0, from 4 to 6 elements (see Figure 4.15). The response of specimen SPE2-M0 is also analyzed for the concrete mesh depicted in Figure 4.16.



a) Base plate with 5 elements through thickness (10 mm)
 b) Base plate with 6 elements through thickness (20 mm)
 Figure 4.15: Refined mesh of the base plates for models SPE1-M0 and SPE2-M0



Figure 4.16: Refined mesh of the concrete block for model SPE2-M0

To evaluate the influence of the mesh density on the connection response, moment-rotations curves are drawn for specimens SPE1-M0 and SPE2-M0 considering both sets of mesh parameters (see Figure 4.17 and Figure 4.18). It can be seen that the number of elements through the base plate thickness does not modify the maximum bending moment. Nevertheless, a larger number of elements across the thickness direction of the base plate may results in a slightly larger joint rotation although curves clearly suggest that this difference is insignificant. For the analysis presented in subchapters 4.3 and 4.4, base plates are discretized using 3 and 4 elements for 10 mm and 20 mm thicknesses, respectively. From Figure 4.18 it can be seen, that curves in red and blue overlap. This means that increasing the total number of elements of the concrete

block does not alter the connection response but requires greater computational time. As a result, future analysis consider the mesh depicted in Figure 4.13 a).



4.3 Comparisons against test results

In this sub-chapter, numerical results are compared against experimental data presented in Chapter 3 to assess the finite element model accuracy. The comparison is carried out considering moment-rotation curves, failure modes, anchor bolts tensile forces, strain distribution over the column flanges and the variation of the contact zones for different load levels.

4.3.1 Column base plate tests

4.3.1.1 Moment-rotation curves and failure modes

A comparison of the plastic bending moment as well as the initial rotational stiffness obtained from the numerical model and the experimental tests is given in Table 4.7. The numerical plastic bending moment is obtained using the method described in Figure 3.84.

Table 4.7: Comparison of the plastic resistances and initial stiffness from the numerical models and experimental tests							
	Experimental		Numerical		Exp/Num		
Test	Initial stiffness S _{j,ini,exp} (kNm/rad)	Plastic bending moment <i>M</i> _{j,pl,exp} (kNm)	Initial stiffness S _{j,ini,num} (kNm/rad)	Plastic bending moment <i>M</i> _{j,pl,num} (kNm)	$S_{ m j,ini}$	$M_{ m j,pl}$	
SPE1-M0	4117,4	37,7	4163	35,1	0,99	1,07	
SPE2-M0	7189,9	42,9	7536	46,2	0,95	0,93	
SPE1-M90	2028,7	28,2	2061	26,4	0,98	1,07	
SPE2-M90	3120,5	35,9	2712	37,9	1,15	0,95	
SPE1-M45	2440,5	32,0	2853	32,5	0,86	0,98	
SPE2-M45	3205,0	36,6	4571	38,2	0,70	0,96	

In general, the plastic bending moment and the initial rotational stiffness obtained from the numerical simulations for all specimens are in good agreement with the experimental data. The values of the plastic bending moment predicted by the numerical models are very satisfactory,

with an error less than 7%. The value of the computed initial rotational stiffness for specimen SPE2-M45 is less satisfactory with an error not exceeding 30%, which is still acceptable. This discrepancy between the results obtained experimentally and numerically results from the simplified and rather conservative approach used to model the adherence (τ_{max}) between the concrete and the anchor bolts (see subsection 4.2.2.3). This has a strong effect on the initial stiffness $S_{j,ini}$. Also, the fact that the initial geometrical imperfections of the base plates are not taken into account in the FE models may have an impact on the way the connection deform in the elastic regime and consequently, on the stiffness and plastic bending moment values.

A comparison of the yielded elements and the failure modes obtained numerically and experimentally is given in Table 4.8. For all specimens, the yielded elements predicted by the FE model are quite the same as those observed experimentally. For specimens SPE1-M0, SPE1-M90 and SPE1-M45, numerical simulations allow to conclude that the column steel profile slightly yielded. In contrast to specimen SPE1-M90, this phenomenon could not be observed in specimens SPE1-M0 and SPE1-M45 during experimental tests, as a result of the absence of strain gauges. Local yielding of the welds is also observed as a result of the high deformability of the thinner base plate and consequently, high deformation of the assembly under bending. In all cases, anchor bolts yield as a consequence of tension and bending. For the six models, yield lines in the base plate and at the bottom of the column flanges can be observed. Von-Mises Plastic Strain distributions at the base plates are depicted in Figure 4.19.

labl	e 4.8: Comparis	son of the yielded eleme	ents and failure mode	es from the numerical mo	dels and experimental tests
		Experimental		Numerical	
Test		Yielded elements*	Failure mode	Yielded elements*	Failure mode
	SPE1-M0	AB/BP/W	AB	AB/BP/C/W	AB
	SPE2-M0	AB	AB	AB/BP	AB
	SPE1-M90	AB/BP/C/W	AB	AB/BP/C/W	AB
	SPE2-M90	AB/C	AB	AB/BP	AB
	SPE1-M45	AB/BP/W	AB	AB/BP/C/W	AB/W
	SPE2-M45	AB	AB	AB/W	AB

. . ..

*AB: anchor bolts yielding due to tension and bending; BP: base plate yielding due to bending; C: column yielding due to bending; W: weld yielding due to bending.





Figure 4.19: Yield line patterns obtained from the numerical simulations for column base plate models

As mentioned in Chapter 3, yield lines parallel to the column flanges develop at the base plate of specimen SPE1-M0 in both compressive and tensile sides. In the tensile side, the yielding develop behind welding. The latter is also significantly deformed. In contrast to specimen SPE1-M0, the base plate in specimen SPE2-M0 behave as rigid body in the tensile side. In the compressive side, a yield line develop between the anchor bolt row and the column flange. Due to the higher deformability of the base plate in specimen SPE1-M90, the development of yield lines occur in both tensile and compressive sides. The same is not observed for specimen SPE2-M90 in which the base plate remained elastic in the tensile side. For specimen SPE1-M45, a yield line parallel to the column flange develop similarly to that observed in specimen SPE1-M0. Yield line pattern of specimen SPE1-M45 suggests a local yielding of the base plate. Similarly to specimen SPE2-M0 and as a result of the base plate thickness in specimen SPE2-M45, the yield line occur only in the compressive side.

Depending on the direction of the applied bending moment, it is concluded that for all specimens, the failure obtained numerically and experimentally is due to the rupture of one or two activated anchor bolts (see Table 4.8).

	Experimental		Numerical		Exp/Nur	n
Test	Maximum Ultimate rotation		Maximum Ultimate rotation			
	bending moment	capacity $\theta_{j,max,exp}$	bending moment	capacity $\theta_{j,max,num}$	$M_{ m j,u}$	$ heta_{ ext{j,max}}$
	$M_{\rm j,u,exp}$ (kNm)	(mrad)	M _{j,u,num} (kNm)	(mrad)		
SPE1-M0	43,2	44,7	41,5	45,1	1,04	0,99
SPE2-M0	48,5	33,2	50,8	35,3	0,95	0,94
SPE1-M90	33,3	69,5	32,4	63,4	1,03	1,10
SPE2-M90	40,2	48,1	41,1	45,7	0,98	1,05
SPE1-M45	39,4	62,7	39,0	60,1	1,01	1,04
SPE2-M45	47,3	54,8	45,7	52,2	1,04	1,05

Table 4.9: Comparison of the numerical and experimental maximum bending resistances and rotation capacities

The agreement between the numerical and experimental results regarding the maximum bending resistance and the ultimate rotation capacity is also satisfactory. The ratio experimental/numerical indicates higher deviation for the ultimate rotation capacity, around 10%. In general, the numerical models underestimate the ultimate rotation capacity with an acceptable error. This is a consequence of several factors that compromise the accuracy of the estimation of this parameter, such as: the variability of the material properties; the difficulty in defining the exact failure point in simple models such as those presented and, the precision of the computation of the connection rotation.

The moment-rotation curves obtained numerically and experimentally are depicted in Figure 4.20. It can be seen that the shape of both the experimental tests and FE models are quite the same from the beginning of the loading (elastic domain) until the end of the tests (failure). Numerical moment-rotation curves are derived from Equations 3.1 and 3.2. The resultant force F and the relative horizontal displacements of the base plate at level of the upper and lower column flanges U_t and U_b are directly extracted from ABAQUS. From the results below, the numerical resistances, initial rotational stiffness, rotation capacity and the overall shape of the curves are in good agreement with the experimental results. The failure was considered when the Von Mises strain reached the ultimate strain ε_u in one component.





The FE models are able to capture the global behavior of the tested column base plates. As can be observed from Figure 4.21 to Figure 4.26, the deformed shapes produced by the FE models are in agreement with the observations made during the experimental tests.



Figure 4.21: Failure of the anchor bolts and yielding of the base plate for SPE1-M0



Figure 4.22: Failure of the anchor bolts and yielding of the base plate for SPE1-M90



Figure 4.23: Failure of the anchor bolts for SPE2-M0





Figure 4.24: Failure of the anchor bolts for SPE2-M90



Figure 4.25: Failure of the anchor bolts and yielding of the base plate for SPE1-M45

Figure 4.26: Failure of the anchor bolts for SPE2-M45

To widen the scope of the present study, the response of specimens SPE1 and SPE2 under different bending moment orientations is considered. Two cases are considered: an applied bending moment with an inclination of 30° with respect to the strong axis direction and an inclination of 60° with respect to same axis. Although these results cannot be compared against experimental data, the aptitude of the numerical models to simulate the actual response of the

connections has been proven. Interaction curves M_0 - M_{90} obtained from experimental data and numerical results are drawn and compared in Figure 4.27 and Figure 4.28. Results suggest that for these specimens, the evolution of the bending resistance can be accurately predicted by an elliptical curve.



4.3.1.2 Anchor bolt forces

The evolution of the tensile force in the anchor bolts with increasing bending moment predicted by the FE model is depicted in Figure 4.29 for all specimens. These curves allow to evaluate the redistribution of internal forces in the anchor bolts up to the maximal bending moment $M_{j,u}$. Due to the symmetry of the loading for specimens SPE1-M0, SPE2-M0, SPE1-M90 and SPE2-M90, the evolution of the tensile forces is represented only for the most activated anchor bolts. As explained in Chapter 3, for specimen SPE1-M90 and specimen SPE2-M90, a redistribution of internal forces from the upper to the lower anchor bolt rows is observed. Thus, for these cases the evolution of the tensile force in the anchor bolts comprises the lower anchor bolt row. For specimens SPE1-M45 and SPE2-M45, anchor bolts AB1, AB2 and AB3 (see Figure 3.65) contribute to resist to the applied loads. The evolution of the tensile forces is compared in these three elements.





Figure 4.29: Normal force on the anchor bolts-applied moment curves for SPE models

Regarding specimens SPE1-M0 and SPE2-M0, it is confirmed that the evolution of the tensile force in the most tensioned anchor bolt (upper row) is almost linear until the bending moment reaches 30-40 kNm. This results from the absence of prying effect during these simulations, leading to a linear relationship between the bending moment and the bolt tensile forces. At the end of the loading of specimen SPE1-M0, a slight decrease of the slope is observed. This is a consequence of the modification of the lever arm (see subsection 4.3.1.4). As mentioned earlier, the configuration of the anchoring system limit the redistribution of internal forces from the upper to the lower anchor bolts rows, the latter remaining inactive (compressive side). It can also be concluded that the great flexibility of the base plate in specimen SPE1-M0 leads to a smaller lever arm z. Consequently, for a given bending moment, the anchor bolts are more loaded than in specimen SPE2-M0.

For specimen SPE2-M90, the tensile force in the upper anchor bolt increase linearly until a bending moment of 30-35 kNm. This results from the absence of prying effect leading to a linear relationship between the bending moment and the bolt tensile force. For specimen SPE1-M90 a slight decrease of the slope of the curve is visible at about 25 kNm. At the same time, the slope of the lower anchor bolts increase for the same loading level. This indicates the beginning of the redistribution of tensile forces between both anchor bolt rows. For specimen SPE2-M90, yielding of the upper row in tension occurs for a bending moment greater than 30 kNm and initiates redistribution. At the same time, the lever arm z is modified. This phenomena is intensified when the upper anchor bolts enter in the necking phase. At the maximum bending moment, the force resisted by the lower anchor bolts corresponds to a quarter of the force applied to the upper anchor bolts for connection SPE1-M90. For SPE2-M90, the lower anchor bolts is less loaded due to the reduced ductility of SPE2-M90 comparatively to SPE1-M90.

The evolution of the tensile force in anchor bolts AB1, AB2 and AB3 for specimens SPE1-M45 and SPE2-M45 is depicted in Figure 4.29 c) and d). As a result of AB4 being located in the compressive zone, the latter remain inactive during the simulation for both cases. It can be

observed that AB2 is the most tensioned anchor bolt. For specimen SPE1-M45, the slope of the curve of AB2 starts to decrease for a bending moment equal to 30 kNm. At the same time, the force in anchor bolts AB1 and AB3 starts to increase. Anchor bolt AB3 is located immediately under AB2 and contributed to the load carrying mechanism nearly from the beginning of the loading. The magnitude of the tensile forces resisted by anchor bolts AB2 and AB3 is similar at the end of the test/simulation. AB1 being located near the compressive zone, its contribution to the load carrying mechanism is the least, withstanding approximately 3/4 of the maximum tensile force of AB2 at the end of the loading.

4.3.1.3 Strain distribution

The axial strain obtained from the numerical simulations are compared against the experimental measurements (see chapter 3) for specimens SPE1-M90 and SPE2-M90 (see Figure 4.30). In general, the agreement between the curves is satisfactory. Due to the flexibility of the base plate in specimen SPE1-M90, a local yielding occur at the top of the column flanges in tension. For specimen SPE2-M90, as expected from the experimental results, the column yields in the compressive side, as a result of the decreasing compressive area during the loading. The distribution of stresses and strains along the column flange width can be seen in Figure 4.31, Figure 4.32, Figure 4.33 and Figure 4.34. As during experimental tests, the length of the compressive area provided by the FE model is equal to 50 mm in SPE1-M0 and the tensile area yields at the end of the loading. For SPE2-M90, the length of the compressive area is greater as the tensile area is stiffer. The strain distribution is closer to that given by Bernoulli beam theory.



Figure 4.30: Comparison of the strain distribution obtained from the experimental tests and numerical simulations



Figure 4.33: Numerical stress distribution along the column Fig flange width for SPE2-M90

Figure 4.34: Numerical strain distribution along the column flange width for SPE2-M90

4.3.1.4 Contact zones

In contrast to the assumption made in the Component method in EN 1993-1-8, the position of the centre of compression is not fixed at the column flange in compression (for in-plane bending). In reality, as mentioned above, contact zones vary with the loading due to the continuously changes of the relative flexibility of the different components. Hereupon, the assumption that the centre of compression remains at the same position during the elastic and plastic phases is safe but not realistic. The evolution of the centre of compression location z_c is depicted below.



Figure 4.35: Position of the centre of compression zc for M0, M90 and M45 loading cases



Figure 4.36: Evolution of the position of the centre of compression obtained from the SPE numerical models

The comparison between the positions of the center of compression obtained numerically and from the Component method in EN 1993-1-8 for specimens SPE1-M0 and SPE2-M0 suggests that the approach adopted in the Eurocode is quite conservative for thick flanges. From the numerical simulations, it can be seen that z_c is not constant. In fact, z_c remains nearly constant during the first stage of loading characterized by a value of the bending moment below 5 kNm. However, z_c increases with increasing values of the applied bending moment. This evolution is a consequence of the modification of the relative stiffness between the tensile and the compressive areas caused by the yielding of several components.

Regardless the loading case (M0, M90 or M45), the centre of compression for thicker base plates (20 mm) tends to move away from the footprint of the column flange towards the base plate free edge. This trend is apparently less pronounced for the cases with a thinner base plate (10 mm), for which z_c does not vary substantially and remain nearly constant, near the column flange.

The evolution of specimen SPE1-M0 confirms that the centre of compression at the end of the analysis is located above the lower anchor bolt row. For specimen SPE2-M0, *z*_c moved towards

the base plate edge, and near collapse, its location is between the lower anchor bolt row and the base plate free edge. For specimen SPE1-M90, the evolution of z_c is insignificant. Consequently, the centre of compression remain at the level of the column flange tips, with all the anchor bolts located in the tensile side. For specimen SPE2-M90 the evolution is more pronounced than in the previous case and, at the end of the loading z_c is located close to the base plate end.

Contact pressures on the base plate bottom surface (in contact with the concrete) in the elastic regime and at collapse are depicted in Figure 4.37. According to what has been said above, contact pressures tend to move towards the lower edge for thicker base plate, with a more or less uniform distribution over the base plate width. Conversely, for more deformable base plates (10 mm), contact pressures are located closer to the column flanges as a consequence of the less uniform stress distribution. For M0, M90 and M45, the development of prying effect at an initial stage of the loading can be observed for SPE1 test serie. For specimen SPE1-M0, contact pressure develops between the lower column flange and the lower anchor bolt row during the elastic regime, with development of prying effect in the tensile area. Near collapse, contact pressures are located around the lower anchor bolts in the compressive area. Prying effect disappears as a result of the excessive anchor bolt row and the free edge of the base plate during the elastic regime. Near collapse, pressures are located at the lower free edge of the base plate during the elastic regime. Near collapse, pressures are located at the lower free edge of the base plate.

For specimen SPE1-M90, contact pressure develops locally around the column flanges in the compressive area during the elastic regime and near failure. At the beginning of the loading, prying forces develop in the tensile area and disappear near collapse. For specimen SPE2-M90, contact pressures are located around the column flange and the free edge of the base plate at the beginning of the loading, moving closer to the free edge at collapse. For specimens SPE1-M45 and SPE2-M45, contact pressure distribution does not significantly change between the elastic and plastic regimes.









Contact pressures on the base plates along lines 1, 2 and 3 (see Figure 4.38) are depicted in Figure 4.39. As can be seen in Figure 4.37, contact pressure distribution changes more or less from the elastic regime to collapse. Three types of contact pressure distribution can be observed:

- for low values of the length of the contact area, contact pressure distribution is linear,
- for large values of the contact area length (about $4-5 \times t_{bp}$), contact pressure distribution is parabolic with two uplift points,
- for intermediate length of the contact area, the distribution shape is more complex.



Figure 4.38: Location of the contact pressure measurement lines





4.3.1.5 Concluding remarks

The results of the experimental tests on column base plates presented in Chapter 3 are enriched with results of detailed finite elements models. From the comparison and validation of the numerical results against experimental data, it was concluded that:

- the resistance, initial stiffness and rotational capacity predicted by the FE models are very close to those given by the experimental tests for the six specimens. The maximum deviation is 30%,
- the yielded elements and obtained failure modes are similar. Additionally, the numerical models allow to confirm the yielding of the column and the welds in specimens SPE1-M0, SPE1-M90 and SPE1-M45,
- the numerical models allow the evaluation of the contribution of the anchor bolts to the load carrying mechanism for all specimens. For M90 and M45 series, a redistribution of the internal forces is observed between the most tensioned anchor bolts and the remain activated anchor bolts,
- the strain distribution at the base of the column flanges for specimens SPE1-M90 and SPE2-M90 are correctly predicted by the numerical models,
- the position of the centre of compression *z*_c varies with the applied bending moment which contrast with the assumption made in the Component Method,
- for all test series (M0, M90 and M45), the depth of the centre of compression is higher for thicker base plates. Consequently, *z*_c strongly depends on the flexibility of the base plate.

4.3.2 Beam-to-concrete slab connection tests

4.3.2.1 Moment-rotation curves and failure modes

The main objective of this work is to study the behaviour of connections with ductile failure modes. Thus, plastic bending moment and initial rotational stiffness obtained from the FE

	Experimental		Numerical	Numerical		Exp/Num	
Test	$M_{\rm j,pl,exp}({ m kNm})$	S _{j,ini,exp} (kNm/rad)	$M_{\rm j,pl,num}({ m kNm})$	S _{j,ini,num} (kNm/rad)	$M_{ m j,pl}$	$S_{ m j,ini}$	
PB-1	56,0	5433,8	58,5	7572,6	0,96	0,72	
PB-2	66,5	7984,0	68,4	10750,0	0,97	0,74	

Table 4 10: Comparison of the plastic resistances and initial stiffness from the numerical models and experimental tests

calculations are compared against the experimental data only for specimens PB-1 and PB-2. As can be seen from Table 4.10, good agreement are obtained.

Table 4.11 compares the yielded elements and the failure modes observed in experimental tests and numerical simulations. As observed in the experimental tests, the yielding of the bolts and the base plate govern the behavior and deformation of the FE models. Furthermore, finite element analysis allow to verify that the large thickness of the base plate in specimen PB-1 or the existence of stiffeners in specimen PB-2, do not prevent local yielding of the IPE steel profiles and welds. For specimens PB-1 and PB-2, bolts govern the failure of the specimen.

Table 4.11: Comparison of the yielded elements and failure modes from the numerical models and experimental tests

	Experimental		Numerical		
Test	Yielded elements*	Failure mode	Yielded elements*	Failure mode	
PB-1	B/BP/W	В	B/BP/C/W	В	
PB-2	B/BP/W	В	B/BP/C/W	В	

*B: bolts yielding due to tension and bending; BP: base plate yielding due to bending; C: local column yielding due to bending; W: weld yielding due to bending.

Due to bending, the development of yield lines occurs in the base plates. As it can be observed, for specimens PB-1 and PB-2, the location of the yield lines is quite different (see Figure 4.40). Specimen PB-1 consists in a simple plate subjected to in-plane bending in which plastic yield lines develop at the level of the column flanges in compression and tension. In contrast to this, the presence of stiffeners in specimen PB-2 modify substantially the yield line pattern in the tensile side where the yield line tends to develop vertically between the column flange and the free edge of the plate. Consequently, the maximum bending resistance increases (see Table 4.12).



Figure 4.40: Yield line patterns obtained from the numerical simulations for beam-to-concrete slab connection models (half)



Table 4.12: Comparison of the maximum bending resistances and rotation capacities

Numerical moment-rotation curves are compared against the ones from the experimental tests. Both moments and rotations are calculated according to the procedure described in subsection 4.3.2.1. The agreement between the curves is satisfactory and we can conclude that the FE models are able to predict the global behavior of the connection with good accuracy. According to Table 4.12, a maximum deviation of 15% is observed for the ultimate rotation capacity for specimen PB-1. Similarly to the column base plate specimens, bolts have an important role on the deformability of the beam-to-concrete slab connections. The observed discrepancy may be a consequence of the simplified/approximate modelling of these elements. The deformed shape of the base plate in the numerical simulations are compared to the observations of the specimen after failure (see Figure 4.42 and Figure 4.43).



Figure 4.42: Failure of the anchor bolts and yielding of the base plate for PB-1



Figure 4.43: Failure of the anchor bolts and yielding of the base plate for PB-2

4.3.2.2 Anchor bolt forces

The evolution of the tensile forces in the bolts for specimens PB-1 and PB-2 (upper bolt row) up to the maximum bending moment $M_{j,u}$ is presented in Figure 4.44. The evolution for both cases is quite linear until a bending moment of about 50 kNm. It can be observed that the presence of stiffeners in PB-2 allow the bolts to resist higher values of the applied bending moment. It is expected that a greater thickness of the base plate of specimen PB-1 would result in a redistribution of the internal forces between this element and the bolts which is more favorable to the bending resistance. Figure 4.44 indicates the contrary. This behavior is a result of the high deformability of the base plate in bending and consequently, to the development of prying effect, that in specimen PB-2 is limited due to the presence of stiffeners. Thus, the magnitude of the additional force Q applied to the bolts due to the presence of prying is higher for specimen PB-1, leading these elements to resist a greater level of applied tensile force. Consequently, bolts from specimen PB-1 reach their maximum tensile resistance for a value of the bending moment 20% lower.



Figure 4.44: Tensile force on the anchor bolts-applied moment curves for PB models
4.3.2.3 Contact zones

A comparison of the position of the centre of compression z_c (see Figure 4.35) for specimens PB-1 and PB-2 obtained from the numerical simulations and the Component Method is depicted in Figure 4.45. As it can be seen, due to the higher deformability of the base plate in specimen PB-1, the evolution of z_c is clearly visible. At the maximum bending moment level $M_{j,u}$, the centre of compression is located near the lower free edge of the base plate and far from the column flange footprint. For this case, the assumption made in the Component Method (EN 1993-1-8) is quite conservative. However, due to the presence of stiffeners, as explained before, the deformations of the base plate are limited and consequently, the position of z_c , close to the lower anchor bolt row, remained practically constant throughout the loading.



Figure 4.45: Evolution of the position of the centre of compression obtained from the PB numerical models

Contact pressure distributions on the base plate bottom surface (in contact with the concrete) in the elastic regime and collapse are depicted in Figure 4.46. It is observed that due to the high deformability of the base plate in PB-1, the highest concentration of the contact pressures lies in the intermediate zone between the column flange in compression and the lower anchor bolts row. On the other hand, due to the presence of stiffeners in the column in PB-2, the contact pressures, as noted, extend along the footprint of these reinforcements, from the column flange to the free end of the base plate.







4.3.2.4 Concluding remarks

From the comparison of the numerical and experimental results for the beam-to-concrete slab connections it can be concluded that:

- the plastic bending moment, initial stiffness, ultimate bending moment and the rotation capacity are accurately predicted by the FE models,
- the yielded elements and the failure modes from the numerical simulations are in agreement with the experimental tests,
- the presence of stiffeners strongly influences the maximum bending resistance and the yielding pattern,
- the presence of stiffeners restricts the development of prying effect, limiting additional forces *Q* applied to the bolts and improving the overall resistance of the connection,
- the position of the centre of compression *z*_c varies according to the existence or not of stiffeners welded to the beam steel profile, since this parameter is highly dependent on the deformability of the base plate.

4.4 Parametric study

4.4.1 Introduction

A parametric study on column base plates have been performed in order to enlarge the set of results covering cases that have not been considered yet and to investigate the effects of the base plate thickness t_{bp} , the column steel profile type (HEA or IPE), the diameter of the anchor bolts and the arrangement of the anchoring system. The response of the joint is evaluated for a combination of tensile/compressive axial force and bending moment (in-plane, out-of-plane or biaxial). In particular, the influence of the loading conditions on the failure mode is investigated. The specimen geometries considered for the parametric study are inspired from specimens SPE1, SPE2 and PB-1.

4.4.2 Geometry and loading conditions

Six column base plate configurations are analysed in this parametric study. In order to reduce the computing time, the dimensions of the concrete block are reduced to $460 \times 360 \times 725$ mm and the height of the column to 500 mm (see Figure 4.47). Geometry and material properties are given in Table 4.13 and Table 4.14. The throat thickness of the weld is equal to 7 mm. The nominal characteristics of the materials are selected. A multi-linear steel stress-strain curve is considered (see Figure 4.49). At the bottom and lateral surfaces of the concrete block, displacements are restrained in all directions. The anchor bolts are modelled with an area equivalent to $A_s = \pi d^2/4$.



		Table 4.15: Geo	metries of the paral	neuric study			
Configuration	Column steel profile	Base plate thickness t _{bp} (mm)	Anchor bolt diameter (mm)	<i>e</i> (mm)	$e_{\rm x}$ (mm)	<i>p</i> (mm)	w (mm)
P1	HEA 200	10	16	70	35	260	160
P2	HEA 200	20	16	70	35	260	160
P3	IPE 200	10	16	40	35	270	80
P4	IPE 200	20	16	40	35	270	80
P5	HEA 200	10	20	70	35	260	160
P6	HEA 200	20	20	70	35	260	160

Fable 4.13: Geometries of the parametric study



Figure 4.48: Geometry of the specimens

	Table 4.14: Material characteristics of the parametric study											
Column	1	Base Plate Anchor bolts				Concrete						
$f_{ m y,c}$	$f_{ m u,c}$	$\boldsymbol{\varepsilon}_{\mathrm{u,c}}$	$f_{ m y,bp}$	$f_{ m u,bp}$	E u,bp	$f_{ m y,b}$	$f_{ m u,b}$	€ u,b	$f_{ m ck}$	$\boldsymbol{\varepsilon}_{\mathrm{cu},1}$		
(MPa)	(MPa)	(%)	(MPa)	(MPa)	(%)	(MPa)	(MPa)	(%)	(MPa)	(‰)		
355	568	15	275	499	15	300	575	10	25	3,5		



To evaluate the influence of a compressive/tensile axial force on the bending moment resistance and the failure modes of the connections, specimens are subjected to a combination of an axial force and mono-axial (in-plane/out-of-plane) or biaxial bending moment. To obtain the *M-N* ultimate interaction curves, specimens described in Table 4.13 are firstly loaded by an axial force equal to a percentage of $N_{j,c,u}$ or $N_{j,t,u}$ which corresponds to the resistances obtained numerically in pure compression and tension, respectively. Next, they are loaded by a bending moment in one or two directions. Table 4.15 summarizes the studied loading cases for specimens P1, P2, P3, P4, P5 and P6.

Loading case	Axial force	Bending moment*	Loading case	Axial force	Bending moment*
1	N _{j,c,u}	-	32	Nj,t,u	-
2	-	M0			
3	$0,1 \times N_{j,c,u}$		33	$0,1 \times N_{j,t,u}$	
4	$0,2 \times N_{j,c,u}$		34	$0,2 \times N_{j,t,u}$	
5	$0,3 \times N_{j,c,u}$		35	$0,3 \times N_{j,t,u}$	
6	$0,4 \times N_{\rm j,c,u}$		36	$0,4 \times N_{j,t,u}$	
7	$0,5 \times N_{j,c,u}$	M0	37	$0,5 \times N_{j,t,u}$	M0
8	$0,6 \times N_{j,c,u}$		38	$0,6 \times N_{j,t,u}$	
9	$0,7 \times N_{\rm j,c,u}$		39	$0,7 \times N_{j,t,u}$	
10	$0,8 \times N_{\rm j,c,u}$		40	$0,8 \times N_{j,t,u}$	
11	$0,9 \times N_{\rm j,c,u}$		41	$0,9 \times N_{j,t,u}$	
12	-	M90			
13	$0,1 \times N_{j,c,u}$		42	$0,1 \times N_{j,t,u}$	
14	$0,2 \times N_{j,c,u}$		43	$0,2 \times N_{j,t,u}$	
15	$0,3 \times N_{j,c,u}$		44	$0,3 \times N_{j,t,u}$	
16	$0,4 \times N_{j,c,u}$		45	$0,4 \times N_{j,t,u}$	
17	$0,5 \times N_{j,c,u}$	M90	46	$0,5 \times N_{j,t,u}$	M90
18	$0,6 \times N_{j,c,u}$		47	$0,6 \times N_{j,t,u}$	
19	$0,7 \times N_{\rm j,c,u}$		48	$0,7 \times N_{j,t,u}$	
20	$0,8 \times N_{\rm j,c,u}$		49	$0,8 \times N_{j,t,u}$	
21	$0,9 \times N_{\rm j,c,u}$		50	$0,9 \times N_{j,t,u}$	
22	-	M45			
23	$0,1 \times N_{j,c,u}$		51	$0,1 \times N_{\mathrm{j,t,u}}$	
24	$0,2 \times N_{j,c,u}$		52	$0,2 \times N_{j,t,u}$	
25	$0,3 \times N_{j,c,u}$		53	$0,3 \times N_{j,t,u}$	
26	$0,4 \times N_{j,c,u}$		54	$0,4 \times N_{j,t,u}$	
27	$0,5 \times N_{j,c,u}$	M45	55	$0,5 \times N_{j,t,u}$	M45
28	$0,6 \times N_{j,c,u}$		56	$0,6 \times N_{\rm j,t,u}$	
29	$0,7 \times N_{\rm j,c,u}$		57	$0,7 \times N_{\rm j,t,u}$	
30	$0,8 \times N_{\rm j,c,u}$		58	$0,8 \times N_{\rm j,t,u}$	
31	$0,9 \times N_{\rm j,c,u}$		59	$0,9 \times N_{\rm j,t,u}$	

Table 4.15: Loading cases of the parametric study

*M0: in-plane bending moment; M90: out-of-plane bending moment; M45: biaxial bending moment.

4.4.3 Failure modes/influence of the loading conditions

Plastic and ultimate bending moments, initial rotational stiffness, yielded elements and failure modes for specimens P1, P2, P3, P4, P5 and P6 are summarized in the tables below. $M_{j,pl}$ is evaluated based on the method depicted in Figure 3.84. Compressive forces are positive (+) and tensile forces negative (-). The caption code for the yielded elements is as follows:

- AB anchor bolts yielding due to tension and bending (one or more),
- BP base plate yielding due to bending,
- C column yielding/local buckling due to bending,
- W weld yielding due to bending.

Specimen	Loading case	<i>N</i> (kN)	<i>M</i> _{j,pl} (kNm)	M _{j,u} (kNm)	$M_{ m j,u}/M_{ m j,pl}$	S _{j,ini} (kNm/rad)	Yielded elements	Failure mode
	1	1971	-	-	-	-	С	С
	11	1774	17	23	1,35	14257	C/W	С
	10	1577	37	54	1,45	12534	C/BP/W	С
	9	1380	50	73	1,46	14527	C/BP/W	С
	8	1183	73	85	1,16	14428	C/BP/W	С
	7	986	81	97	1,19	13788	C/BP/W/AB	С
	6	788	81	106	1,31	16316	C/BP/W/AB	AB
	5	591	63	92	1,46	16541	C/BP/W/AB	AB
	4	394	47	76	1,62	15120	C/BP/W/AB	AB
	3	197	36	58	1,61	7212	C/BP/W/AB	AB
P1	2	0	25	38	1,12	5305	C/BP/W/AB	AB
	33	-29	23	35	1,56	3507	C/BP/W/AB	AB
	34	-59	20	32	1,64	3191	C/BP/W/AB	AB
	35	-88	18	29	1,61	2524	C/BP/W/AB	AB
	36	-117	14	26	1,85	2103	C/BP/W/AB	AB
	37	-147	11	23	2,16	2289	C/BP/W/AB	AB
	38	-176	7	20	2,75	1832	C/BP/W/AB	AB
	39	-205	4	14	3,76	979	C/BP/W/AB	AB
	40	-235	~0	9	-	215	C/BP/W/AB	AB
	41	-264	~0	4	-	149	C/BP/W/AB	AB
	32	-293	-	-	-	-	C/BP/W/AB	AB

Table 4.16: Resistances and failure modes of P1 for M0

Table 4.17: Resistances and failure modes of P2 for M0

Specimen	Loading case	<i>N</i> (kN)	M _{j,pl} (kNm)	M _{j,u} (kNm)	$M_{ m j,u}/M_{ m j,pl}$	S _{j,ini} (kNm/rad)	Yielded elements	Failure mode
	1	1966	-	-	-	-	С	С
	11	1769	20	44	2,20	14558	C/W	С
	10	1573	39	62	1,58	19387	C/W	С
	9	1376	59	74	1,25	15263	C/BP/W	С
	8	1180	74	89	1,20	17657	C/BP/W	С
	7	983	87	109	1,25	17171	C/BP/W/AB	С
	6	786	84	113	1,34	13205	C/BP/W/AB	AB
	5	590	72	107	1,48	15927	C/BP/W/AB	AB
	4	393	61	92	1,50	10415	C/BP/W/AB	AB
	3	197	42	73	1,73	9554	C/BP/W/AB	AB
P2	2	0	27	53	1,96	11234	C/BP/W/AB	AB
	33	-37	17	48	2,84	5234	BP/W/AB	AB
	34	-73	18	43	2,40	4144	BP/W/AB	AB
	35	-109	13	37	2,86	3981	AB/W	AB
	36	-147	9	32	3,61	3921	AB	AB
	37	-183	4	26	6,43	2930	AB	AB
	38	-220	~0	21	-	551	AB	AB
	39	-257	~0	14	-	411	AB	AB
	40	-293	~0	9	-	-	AB	AB
	41	-330	~0	5	-	-	AB	AB
	32	-367	-	-	-	-	AB	AB

Specimen	Loading case	<i>N</i> (kN)	<i>M</i> _{j,pl} (kNm)	M _{j,u} (kNm)	$M_{ m j,u}/M_{ m j,pl}$	S _{j,ini} (kNm/rad)	Yielded elements	Failure mode
	1	1052	-	-	-	-	С	С
	11	947	9	22	2,44	7643	С	С
	10	842	20	32	1,60	9000	С	С
	9	737	30	41	1,36	11415	С	С
	8	631	40	55	1,37	10291	С	С
	7	526	45	64	1,42	10979	C/BP/W/AB	С
	6	421	47	70	1,48	12200	C/BP/W/AB	AB
	5	316	45	65	1,44	11737	C/BP/W/AB	AB
	4	210	36	56	1,55	10334	C/BP/W/AB	AB
	3	105	31	48	1,54	7293	C/BP/W/AB	AB
P3	2	0	24	41	1,70	4826	C/BP/W/AB	AB
	33	-37	19	37	2,01	4679	C/BP/W/AB	AB
	34	-74	19	34	1,76	3191	C/BP/W/AB	AB
	35	-110	14	29	2,07	3174	C/BP/W/AB	AB
	36	-147	8	25	3,37	3603	C/BP/W/AB	AB
	37	-184	8	21	2,36	3160	C/BP/W/AB	AB
	38	-221	6	18	2,85	2638	C/BP/W/AB	AB
	39	-257	5	13	1,90	1680	C/BP/W/AB	AB
	40	-294	5	8	1,55	1370	C/BP/W/AB	AB
	41	-331	2	3	1,73	980	C/BP/W/AB	AB
	32	-368	-	-	-	-	C/BP/W/AB	AB

Table 4.18: Resistances and failure modes of P3 for M0

Table 4.19: Resistances and failure modes of P4 for M0

Specimen	Loading case	<i>N</i> (kN)	M _{j,pl} (kNm)	M _{j,u} (kNm)	$M_{ m j,u}/M_{ m j,pl}$	S _{j,ini} (kNm/rad)	Yielded elements	Failure mode
	1	1062	-	-	-	-	С	С
	11	956	11	20	1,81	7285	С	С
	10	849	20	31	1,55	11012	С	С
	9	743	28	41	1,46	11242	С	С
	8	637	38	51	1,34	12006	С	С
	7	531	48	65	1,35	12501	C/W/AB	С
	6	425	49	72	1,46	12233	C/BP/W/AB	С
	5	319	50	76	1,52	10903	C/BP/W/AB	AB
	4	212	46	70	1,52	10691	C/BP/W/AB	AB
	3	106	39	62	1,58	10151	C/BP/W/AB	AB
P4	2	0	30	51	1,72	7522	BP/AB	AB
	33	-39	28	48	1,70	6558	BP/AB	AB
	34	-78	29	44	1,49	5823	BP/AB	AB
	35	-116	18	39	2,13	6028	BP/AB	AB
	36	-155	23	34	1,49	3908	BP/AB	AB
	37	-194	21	29	1,34	2374	AB	AB
	38	-233	18	22	1,24	1931	AB	AB
	39	-272	12	15	1,24	1627	AB	AB
	40	-311	6	10	1,53	1515	AB	AB
	41	-349	3	5	1,57	1314	AB	AB
	32	-388	-	-	-	-	AB	AB

Specimen	Loading case	<i>N</i> (kN)	<i>M</i> _{j,pl} (kNm)	M _{j,u} (kNm)	$M_{ m j,u}/M_{ m j,pl}$	S _{j,ini} (kNm/rad)	Yielded elements	Failure mode
	1	2091	-	-	-	-	С	С
	11	1883	11	26	2,36	8215	C/BP/W	С
	10	1674	28	48	1,71	11943	C/BP/W	С
	9	1464	53	74	1,39	13440	C/BP/W	С
	8	1255	65	87	1,33	15199	C/BP/W	С
	7	1046	79	104	1,32	16213	C/BP/W	С
	6	837	83	111	1,33	17436	C/BP/W/AB	AB
	5	628	74	105	1,41	16126	C/BP/W/AB	AB
	4	418	59	96	1,62	16728	C/BP/W/AB	AB
	3	209	51	80	1,58	10621	C/BP/W/AB	AB
P5	2	0	36	61	1,71	6442	C/BP/W/AB	AB
	33	-55	32	54	1,69	6001	C/BP/W/AB	AB
	34	-109	28	49	1,78	4971	C/BP/W/AB	AB
	35	-164	21	43	2,02	4158	C/BP/W/AB	AB
	36	-218	17	37	2,15	4183	C/BP/W/AB	AB
	37	-273	16	29	1,86	3271	C/BP/W/AB	AB
	38	-327	11	26	2,26	3095	C/BP/W/AB	AB
	39	-382	3	19	6,47	2512	C/BP/W/AB	AB
	40	-436	2	10	7,04	2694	C/BP/W/AB	AB
	41	-491	~0	4	-	-	C/BP/W/AB	AB
	32	-545	-	-	-	-	C/BP/W/AB	AB

Table 4.20: Resistances and failure modes of P5 for M0

Table 4.21: Resistances and failure modes of P6 for M0

Specimen	Loading case	<i>N</i> (kN)	M _{j,pl} (kNm)	M _{j,u} (kNm)	$M_{ m j,u}/M_{ m j,pl}$	S _{j,ini} (kNm/rad)	Yielded elements	Failure mode
	1	2096	-	-	-	-	С	С
	11	1887	11	18	1,63	8202	C/W	С
	10	1678	30	49	1,63	11899	C//W	С
	9	1467	52	73	1,40	13483	C/BP/W	С
	8	1258	64	83	1,29	15440	C/BP/W	С
	7	1048	80	104	1,30	17125	C/BP/W	С
	6	838	93	117	1,26	18911	C/BP/W/AB	С
	5	629	89	122	1,37	17258	C/BP/W/AB	AB
	4	419	79	117	1,48	15651	C/BP/W/AB	AB
	3	209	74	101	1,36	13507	C/BP/W/AB	AB
P6	2	0	58	81	1,39	11322	C/BP/W/AB	AB
	33	-63	49	75	1,54	9741	BP/W/AB	AB
	34	-127	33	68	2,06	8603	BP/W/AB	AB
	35	-191	32	62	1,94	8310	AB/W	AB
	36	-254	31	53	1,72	6779	AB	AB
	37	-318	25	45	1,77	5734	AB	AB
	38	-382	~0	34	-	2781	AB	AB
	39	-445	~0	23	-	-	AB	AB
	40	-509	~0	15	-	-	AB	AB
	41	-572	~0	6	-	-	AB	AB
	32	-635	-	-	-	-	AB	AB

Specimen	Loading case	<i>N</i> (kN)	<i>M</i> _{j,pl} (kNm)	M _{j,u} (kNm)	$M_{ m j,u}/M_{ m j,pl}$	S _{j,ini} (kNm/rad)	Yielded elements	Failure mode
	1	1971	-	-	-	-	С	С
	21	1774	15	25	1,67	6068	С	С
	20	1577	29	43	1,48	7419	С	С
	19	1380	40	54	1,35	9061	C/BP/W	С
	18	1183	45	65	1,44	9383	C/BP/W/AB	С
	17	986	44	68	1,54	10054	C/BP/W/AB	С
	16	788	44	65	1,47	10407	C/BP/W/AB	С
	15	591	39	63	1,61	8276	C/BP/W/AB	С
	14	394	38	57	1,50	7360	C/BP/W/AB	С
	13	197	32	49	1,53	5497	C/BP/W/AB	С
P1	12	0	20	34	1,70	3855	C/BP/W/AB	AB
	33	-29	18	31	1,72	2139	C/BP/W/AB	AB
	34	-59	15	28	1,84	1578	C/BP/W/AB	AB
	35	-88	13	26	1,93	1325	C/BP/W/AB	AB
	36	-117	11	23	2,07	1065	C/BP/W/AB	AB
	37	-147	8	20	2,44	916	C/BP/W/AB	AB
	38	-176	5	17	3,28	783	C/BP/W/AB	AB
	39	-205	4	14	4,82	539	C/BP/W/AB	AB
	40	-235	~0	8	-	73	C/BP/W/AB	AB
	41	-264	~0	4	-	64	C/BP/W/AB	AB
	32	-293	-	-	-	-	C/BP/W/AB	AB

Table 4.23: Resistances and failure modes of P2 for M90

Specimen	Loading case	<i>N</i> (kN)	M _{j,pl} (kNm)	M _{j,u} (kNm)	$M_{ m j,u}/M_{ m j,pl}$	S _{j,ini} (kNm/rad)	Yielded elements	Failure mode
	1	1966	-	-	-	-	С	С
	21	1769	19	34	1,78	21427	C/W	С
	20	1573	28	46	1,64	16373	C/W	С
	19	1376	28	57	2,03	15969	C/W	С
	18	1180	48	66	1,37	13817	C/BP/W	С
	17	983	51	78	1,52	12179	C/BP/W/AB	С
	16	786	54	81	1,50	10815	C/BP/W/AB	С
	15	590	42	76	1,80	8346	C/BP/W/AB	С
	14	393	45	68	1,51	8268	C/BP/W/AB	С
	13	197	38	60	1,57	6603	C/BP/W/AB	С
P2	12	0	22	46	2,09	5791	C/BP/W/AB	AB/W
	33	-37	19	42	2,21	3303	C/BP/W/AB	AB
	34	-73	16	36	2,25	3023	C/BP/W/AB	AB
	35	-109	13	31	2,38	2290	C/BP/W/AB	AB
	36	-147	8	27	3,37	1760	C/BP/W/AB	AB
	37	-183	3	22	7,33	1560	C/BP/W/AB	AB
	38	-220	~0	18	-	-	AB	AB
	39	-257	~0	12	-	-	AB	AB
	40	-293	~0	7	-	-	AB	AB
	41	-330	~0	3	-	-	AB	AB
	32	-367	-	-	-	-	AB	AB

Specimen	Loading case	<i>N</i> (kN)	<i>M</i> _{j,pl} (kNm)	M _{j,u} (kNm)	$M_{ m j,u}/M_{ m j,pl}$	S _{j,ini} (kNm/rad)	Yielded elements	Failure mode
	1	1052	-	-	-	-	С	С
	21	947	4	5	1,25	1007	С	С
	20	842	9	10	1,11	1057	С	С
	19	737	13	13	1,00	1109	С	С
	18	631	14	17	1,21	1192	С	С
	17	526	13	18	1,38	1285	С	С
	16	421	13	19	1,46	1115	С	С
	15	316	14	20	1,42	1141	С	С
	14	210	14	19	1,35	1380	C/BP/W/AB	С
	13	105	14	18	1,28	1122	C/BP/W/AB	AB
P3	12	0	15	18	1,20	1141	C/BP/W/AB	AB
	33	-37	14	17	1,21	1049	C/BP/W/AB	AB
	34	-74	13	16	1,23	944	C/BP/W/AB	AB
	35	-110	13	13	1,00	654	C/BP/W/AB	AB
	36	-147	12	12	1,00	450	C/BP/W/AB	AB
	37	-184	9	9	1,00	330	C/BP/W/AB	AB
	38	-221	~0	7	-	228	C/BP/W/AB	AB
	39	-257	~0	5	-	219	C/BP/W/AB	AB
	40	-294	~0	2	-	259	C/BP/W/AB	AB
	41	-331	~0	1	-	203	C/BP/W/AB	AB
	32	-368	-	-	-	-	AB	AB

Table 4.24: Resistances and failure modes of P3 for M90

Table 4.25: Resistances and failure modes of P4 for M90

Specimen	Loading case	<i>N</i> (kN)	<i>M</i> _{j,pl} (kNm)	M _{j,u} (kNm)	$M_{ m j,u}/M_{ m j,pl}$	S _{j,ini} (kNm/rad)	Yielded elements	Failure mode
	1	1062	-	-	-	-	С	С
	21	956	5	9	1,80	1182	С	С
	20	849	9	11	1,22	1270	С	С
	19	743	10	15	1,50	1414	С	С
	18	637	14	18	1,28	1153	С	С
	17	531	11	19	1,72	1224	С	С
	16	425	12	19	1,58	1248	С	С
	15	319	13	20	1,53	1251	С	С
	14	212	15	20	1,33	1013	С	С
	13	106	14	19	1,35	932	C/BP/W/AB	С
P4	12	0	11	18	1,63	818	C/W/AB	С
	33	-39	11	16	1,45	667	C/W/AB	AB
	34	-78	11	15	1,36	565	C/W/AB	AB
	35	-116	9	15	1,66	393	C/W/AB	AB
	36	-155	9	14	1,55	304	C/W/AB	AB
	37	-194	7	12	1,71	273	C/W/AB	AB
	38	-233	4	10	2,50	212	AB	AB
	39	-272	2	7	3,50	286	AB	AB
	40	-311	2	4	2,00	372	AB	AB
	41	-349	1	2	2,00	383	AB	AB
	32	-388	-	-	-	-	AB	AB

Specimen	Loading case	<i>N</i> (kN)	<i>M</i> _{j,pl} (kNm)	M _{j,u} (kNm)	$M_{ m j,u}/M_{ m j,pl}$	S _{j,ini} (kNm/rad)	Yielded elements	Failure mode
	1	2091	-	-	-	-	С	С
	21	1883	15	20	1,33	6431	С	С
	20	1674	28	32	1,14	7533	C/BP/W	С
	19	1464	39	57	1,46	8139	C/BP/W	С
	18	1255	40	67	1,67	8852	C/BP/W/AB	С
	17	1046	43	70	1,62	9745	C/BP/W/AB	С
	16	837	43	68	1,58	9981	C/BP/W/AB	С
	15	628	37	67	1,81	11424	C/BP/W/AB	С
	14	418	41	66	1,60	8955	C/BP/W/AB	С
	13	209	32	60	1,87	8793	C/BP/W/AB	AB
P5	12	0	29	49	1,68	4006	C/BP/W/AB	AB
	33	-55	27	46	1,70	3694	C/BP/W/AB	AB
	34	-109	24	38	1,58	2787	C/BP/W/AB	AB
	35	-164	22	36	1,63	2025	C/BP/W/AB	AB
	36	-218	18	31	1,72	1788	C/BP/W/AB	AB
	37	-273	13	25	1,92	1377	C/BP/W/AB	AB
	38	-327	5	22	4,42	1191	C/BP/W/AB	AB
	39	-382	4	12	2,93	-	C/BP/W/AB	AB
	40	-436	1	9	5,82	-	C/BP/W/AB	AB
	41	-491	~0	2	-	-	C/BP/W/AB	AB
	32	-545	-	-	-	-	AB	AB

 Table 4.26: Resistances and failure modes of P5 for M90

Table 4.27: Resistances and failure modes of P6 for M90

Specimen	Loading case	<i>N</i> (kN)	<i>M</i> _{j,pl} (kNm)	M _{j,u} (kNm)	$M_{ m j,u}/M_{ m j,pl}$	S _{j,ini} (kNm/rad)	Yielded elements	Failure mode
	1	2096	-	-	-	-	С	С
	21	1887	9	17	1,88	9941	C/W	С
	20	1678	28	40	1,42	8848	C/W	С
	19	1467	45	59	1,30	9194	C/W	С
	18	1258	44	68	1,54	9075	C/W	С
	17	1048	58	80	1,37	9886	C/BP/W/AB	С
	16	838	56	82	1,46	10411	C/BP/W/AB	С
	15	629	52	81	1,55	10402	C/BP/W/AB	С
	14	419	52	78	1,50	9908	C/BP/W/AB	С
	13	209	43	74	1,72	8692	C/BP/W/AB	AB
P6	12	0	44	64	1,45	6599	C/BP/W/AB	AB/W
	33	-63	40	59	1,47	5046	C/BP/W/AB	AB
	34	-127	38	54	1,41	4737	C/BP/W/AB	AB
	35	-191	37	48	1,29	3784	C/BP/W/AB	AB
	36	-254	28	43	1,53	3464	C/BP/W/AB	AB
	37	-318	26	36	1,38	2424	C/BP/W/AB	AB
	38	-382	~0	28	-	-	AB	AB
	39	-445	~0	16	-	-	AB	AB
	40	-509	~0	9	-	-	AB	AB
	41	-572	~0	4	-	-	AB	AB
	32	-635	-	-	-	-	AB	AB

Specimen	Loading case	<i>N</i> (kN)	M _{j,pl} (kNm)	M _{j,u} (kNm)	$M_{ m j,u}/M_{ m j,pl}$	S _{j,ini} (kNm/rad)	Yielded elements	Failure mode
	1	1971	_	_	_	_	C	C
	31	1774	25	27	1.08	23307	C/BP/W	C
	30	1577	36	43	1.19	17489	C/BP/W	C
	29	1380	39	65	1.66	15915	C/BP/W/AB	C
	28	1183	56	79	1,41	13315	C/BP/W/AB	C
	27	986	60	83	1,38	15414	C/BP/W/AB	С
	26	788	56	83	1,48	15250	C/BP/W/AB	С
	25	591	51	81	1,58	15644	C/BP/W/AB	С
	24	394	45	71	1,57	13653	C/BP/W/AB	С
	23	197	38	57	1,50	10303	C/BP/W/AB	C/W
P1	22	0	24	38	1,58	4945	C/BP/W/AB	AB
	51	-29	22	35	1,59	3068	C/BP/W/AB	AB
	52	-59	19	32	1,64	2638	C/BP/W/AB	AB
	53	-88	17	28	1,66	2101	C/BP/W/AB	AB
	54	-117	14	25	1,81	1938	C/BP/W/AB	AB
	55	-147	10	22	2,20	1878	C/BP/W/AB	AB
	56	-176	6	17	3,07	1287	C/BP/W/AB	AB
	57	-205	2	13	6,43	939	C/BP/W/AB	AB
	58	-235	~0	7	-	-	C/BP/W/AB	AB
	59	-264	~0	3	-	-	C/BP/W/AB	AB
	32	-293	-	-	-	-	AB	AB

 Table 4.28: Resistances and failure modes of P1 for M45

 Table 4.29: Resistances and failure modes of P2 for M45

Specimen	Loading case	<i>N</i> (kN)	<i>M</i> _{j,pl} (kNm)	M _{j,u} (kNm)	$M_{ m j,u}/M_{ m j,pl}$	S _{j,ini} (kNm/rad)	Yielded elements	Failure mode
	1	1966	-	-	-	-	С	С
	31	1769	17	20	1,17	29844	C/W	С
	30	1573	22	39	1,77	29847	C/W	С
	29	1376	42	63	1,50	28512	C/BP/W	С
	28	1180	49	85	1,73	25110	C/BP/W/AB	С
	27	983	59	91	1,54	20670	C/BP/W/AB	С
	26	786	60	93	1,55	18253	C/BP/W/AB	С
	25	590	57	89	1,56	13809	C/BP/W/AB	С
	24	393	48	79	1,66	12431	C/BP/W/AB	С
	23	197	35	65	1,91	11516	C/BP/W/AB	С
P2	22	0	26	49	1,81	8985	C/BP/W/AB	AB
	51	-37	22	45	2,02	4765	C/BP/W/AB	AB
	52	-73	20	40	1,98	3733	C/BP/W/AB	AB
	53	-109	15	36	2,33	2908	C/BP/W/AB	AB
	54	-147	10	31	2,98	2798	C/BP/W/AB	AB
	55	-183	5	26	5,30	1927	C/BP/W/AB	AB
	56	-220	~0	21	-	-	C/BP/W/AB	AB
	57	-257	~0	13	-	-	AB	AB
	58	-293	~0	8	-	-	AB	AB
	59	-330	~0	4	-	-	AB	AB
	32	-367	-	-	-	-	AB	AB

Specimen	Loading case	<i>N</i> (kN)	<i>M</i> _{j,pl} (kNm)	M _{j,u} (kNm)	$M_{ m j,u/} \ M_{ m j,pl}$	S _{j,ini} (kNm/rad)	Yielded elements	Failure mode
	1	1052	-	-	-	-	С	С
	31	947	7	10	1,42	4666	С	С
	30	842	13	21	1,61	5578	С	С
	29	737	22	29	1,31	6184	С	С
	28	631	26	32	1,23	6256	C/BP/W	С
	27	526	29	35	1,20	6092	C/BP/W/AB	С
	26	421	31	36	1,16	5263	C/BP/W/AB	AB
	25	316	34	37	1,08	5990	C/BP/W/AB	AB
	24	210	28	36	1,28	5453	C/BP/W/AB	AB
	23	105	24	32	1,33	4571	C/BP/W/AB	AB
P3	22	0	22	28	1,27	8985	C/BP/W/AB	AB
	51	-37	19	27	1,42	2908	C/BP/W/AB	AB
	52	-74	16	24	1,50	2068	C/BP/W/AB	AB
	53	-110	17	21	1,23	2209	C/BP/W/AB	AB
	54	-147	10	18	1,80	1450	C/BP/W/AB	AB
	55	-184	8	12	1,50	978	C/BP/W/AB	AB
	56	-221	6	8	1,33	865	C/BP/W/AB	AB
	57	-257	3	4	1,33	572	C/BP/W/AB	AB
	58	-294	2	2	1,00	532	C/BP/W/AB	AB
	59	-331	1	1	1,00	274	C/BP/W/AB	AB
	32	-368	-	-	-	-	AB	AB

 Table 4.30: Resistances and failure modes of P3 for M45

Table 4.31: Resistances and failure modes of P4 for M45

Specimen	Loading case	<i>N</i> (kN)	<i>M</i> _{j,pl} (kNm)	M _{j,u} (kNm)	$M_{ m j,u}/M_{ m j,pl}$	S _{j,ini} (kNm/rad)	Yielded elements	Failure mode
	1	1062	-	-	-	-	С	С
	31	956	7	10	1,42	4422	С	С
	30	849	10	16	1,60	5605	С	С
	29	743	12	20	1,66	6099	С	С
	28	637	15	29	1,93	6293	С	С
	27	531	24	38	1,58	6632	C/W/AB	С
	26	425	30	44	1,46	6721	C/BP/W/AB	С
	25	319	34	51	1,50	6605	C/BP/W/AB	AB
	24	212	34	51	1,50	6226	C/BP/W/AB	AB
	23	106	33	44	1,33	5566	C/BP/W/AB	AB
P4	22	0	30	39	1,30	4894	C/BP/W/AB	AB
	51	-39	29	35	1,21	4087	C/BP/W/AB	AB
	52	-78	26	32	1,23	3276	C/BP/W/AB	AB
	53	-116	24	27	1,12	3411	C/BP/W/AB	AB
	54	-155	19	23	1,21	3231	C/BP/W/AB	AB
	55	-194	14	19	1,35	2358	C/BP/W/AB	AB
	56	-233	10	14	1,40	946	C/BP/W/AB	AB
	57	-272	9	9	1,00	786	C/BP/W/AB	AB
	58	-311	2	4	2,00	587	C/BP/W/AB	AB
	59	-349	1	1	1,00	430	C/BP/W/AB	AB
	32	-388	-	-	-	-	AB	AB

Specimen	Loading case	<i>N</i> (kN)	<i>M</i> _{j,pl} (kNm)	M _{j,u} (kNm)	$M_{ m j,u'} \ M_{ m j,pl}$	S _{j,ini} (kNm/rad)	Yielded elements	Failure mode
	1	2091	-	-	-	-	С	С
	31	1883	14	22	1,57	11473	C/BP/W	С
	30	1674	22	37	1,68	12571	C/BP/W	С
	29	1464	44	62	1,40	14506	C/BP/W/AB	С
	28	1255	58	75	1,29	16325	C/BP/W/AB	С
	27	1046	56	80	1,42	15226	C/BP/W/AB	С
	26	837	56	80	1,42	15211	C/BP/W/AB	AB
	25	628	49	75	1,53	12485	C/BP/W/AB	AB
	24	418	48	68	1,41	11446	C/BP/W/AB	AB
	23	209	37	62	1,67	9460	C/BP/W/AB	AB
P5	22	0	37	52	1,41	6430	C/BP/W/AB	AB
	51	-55	33	48	1,45	5212	C/BP/W/AB	AB
	52	-109	29	40	1,37	4119	C/BP/W/AB	AB
	53	-164	25	35	1,40	3208	C/BP/W/AB	AB
	54	-218	20	27	1,35	2762	C/BP/W/AB	AB
	55	-273	19	23	1,21	1917	C/BP/W/AB	AB
	56	-327	6	16	2,66	-	C/BP/W/AB	AB
	57	-382	~0	13	-	-	C/BP/W/AB	AB
	58	-436	~0	5	-	-	C/BP/W/AB	AB
	59	-491	~0	2	-	-	C/BP/W/AB	AB
	32	-545	-	-	-	-	AB	AB

 Table 4.32: Resistances and failure modes of P5 for M45

 Table 4.33: Resistances and failure modes of P6 for M45

Specimen	Loading case	<i>N</i> (kN)	M _{j,pl} (kNm)	M _{j,u} (kNm)	$M_{ m j,u}/M_{ m j,pl}$	S _{j,ini} (kNm/rad)	Yielded elements	Failure mode
	1	2096	-	-	-	-	С	С
	31	1887	9	19	2,11	11863	C/W	С
	30	1678	36	49	1,36	10563	C/W	С
	29	1467	51	68	1,33	12620	C/BP/W	С
	28	1258	59	84	1,42	14803	C/BP/W/AB	С
	27	1048	63	96	1,52	13011	C/BP/W/AB	С
	26	838	66	102	1,54	15854	C/BP/W/AB	AB
	25	629	60	101	1,68	15947	C/BP/W/AB	AB
	24	419	54	94	1,74	14846	C/BP/W/AB	AB
	23	209	56	83	1,48	11389	C/BP/W/AB	AB
P6	22	0	47	72	1,53	9269	C/BP/W/AB	AB
	51	-63	48	66	1,37	7448	C/BP/W/AB	AB
	52	-127	45	61	1,35	6149	C/BP/W/AB	AB
	53	-191	38	54	1,42	5895	C/BP/W/AB	AB
	54	-254	27	46	1,69	4993	C/BP/W/AB	AB
	55	-318	25	42	1,68	2960	C/BP/W/AB	AB
	56	-382	11	31	2,78	1920	C/BP/W/AB	AB
	57	-445	~0	22	-	-	C/BP/W/AB	AB
	58	-509	~0	15	-	-	C/BP/W/AB	AB
	59	-572	~0	6	-	-	AB	AB
	32	-635	-	-	-	-	AB	AB

Two main failure modes are observed:

- failure of the column steel profile characterized by the local buckling of the flanges due to the high compressive axial force applied to the specimen,
- failure of the anchor bolts in tension with or without base plate yielding.



Figure 4.50: Failure modes: local buckling of the column; anchor bolts failure with or without base plate yielding (for inplane bending moment)



Figure 4.51: Failure modes: local buckling of the column; anchor bolts failure with or without base plate yielding (for out-ofplane bending moment)

From the results presented above, it can be seen that failure modes are strongly influenced by the loading. Under tensile force, all specimens considered in the present study will fail by anchor bolt rupture (tension failure), regardless of the direction of the applied bending moment. For values up to $0.5N_{j,t,u}$, the anchor bolts on the tensile side fail as a result of the combined action of bending moment and tensile force. Above this value, the tensile force is dominant and nearly equally distributed among the anchor bolts. For this range of values of the applied tensile force, the elastic stage is very short and the post-limit strength is quickly reached. This can be verified from the comparison of the moment-rotation curves drawn for each specimens for different levels of tensile/compressive axial load (see Figure 4.52). Under compressive force, the connection behavior is stiffer than that of connection subjected to a tensile force.



compressive axial load; - tensile axial load)

In contrast to what has been explained, when subjected to a compressive axial force, a distinction of the behaviour at the end of the loading is made. It can be understood from the above results that the behavior of the connections in compression cannot be so easily characterized and generalized. In fact, for these cases, the behavior is much more complex due

to its dependence on the orientation of the applied bending moment. For in-plane (M0) and biaxial bending (M45) and for values of the applied force up to $0.5N_{j,c,u}$, failure results from anchor bolts rupture. For values of the applied force above this limit combined with a dominant bending moment, the development of local buckling of the flanges at the bottom of the column is responsible for the failure of the connections. However, for out-of-plane bending (M90) and for relatively low values of applied compressive force, failure is due to local buckling of the flange at the bottom of the column steel profile. It was demonstrated from experimental tests that high stress concentration occurs at the bottom of the column steel profiles (SPE1-M90 and SPE2-M90) producing a local yielding in the compression side. Thus, even with relatively low axial force magnitude, its combination with a bending moment causes a high concentration of stresses.

The applied axial force also influence the plastic bending moment $M_{j,pl}$, the ultimate bending moment $M_{j,u}$ and the initial rotational stiffness $S_{j,ini}$. From Table 4.16 to Table 4.33 it can be observed that regardless of the orientation of the applied bending moment, the ultimate bending moment and the initial stiffness tend to decrease with increasing value of the applied tensile force. This is a consequence of the fact that an increase of the tensile force produces a gradual decrease of the compressive area up to the point where the entire effective area of the base plate is working under tension (pure tension). The compressive area being more rigid than the tensile area, the decrease of its area results in a decrease of the stiffness.

When a compressive axial load is applied, combined with bending moment, the effect of the former can be beneficial for the moment resistance and the initial stiffness for values of axial force up to $0.5N_{j,c,u}$. Until this limit, $M_{j,u}$ and $S_{j,ini}$ increase with increasing value of the axial force. For M0 and M45 series, in which the anchoring system is the element governing failure, this increase of resistance and initial stiffness is a consequence of the late activation of the anchor bolts in the tensile side occurring for larger values of the applied compressive force. For M90 serie, even if the failure is due to the local buckling of the column, the anchor bolts are subjected to high levels of stress. Then, the comments made above are also valid for this case. However, for values of the axial compressive force above $0.5N_{j,c,u,}$ the resistance decreases as the failure is exclusively due to local buckling of the column.

Interactions curves M_{ip} - M_{op} are depicted in Figure 4.53 and Figure 4.54 for different compressive/tensile axial load levels. In presence of a tensile force, the points of the numerical analysis fit quite well with an elliptical curve. However, for specimens P1 and P5 and value of the axial force close to the tensile resistance, this elliptical curve is unsafe. It seems that a linear approximation is more suitable. Even in presence of a tensile force, the elliptical curve is far from what is obtained numerically for specimens P3 and P4. In presence of a compressive force, the elliptical curve is generally conservative except for specimen P2 under high compressive force.



Figure 4.53: Interaction curves M_{ip} - M_{op} for specimens subjected to a compressive axial load





4.4.4 Influence of the geometric parameters

4.4.4.1 Base plate thickness

As observed and discussed in Chapter 3, the overall behavior of the column base plates strongly depend on the thickness of the base plates. The influence of the base plate thickness is analyzed below in terms of failure mode, resistance and initial stiffness. From Table 4.16 to Table 4.33, it can be verified that this parameter has no particular influence on the failure mode. As abovementioned, under combined bending moment and tensile axial force, anchor bolts govern failure of the connection regardless of the base plate thickness. On the other hand, for connections subjected to combined compressive axial force and bending moment, two different failure modes can be observed: rupture of the anchor bolts and local buckling of the column steel profile. However, the FE computations of the plastic bending moment $M_{i,pl}$ and the maximum bending moment $M_{i,u}$ show an increase of the connection resistance with increasing values of the base plate thickness. As can be seen in Figure 3.86, increasing the base plate thickness leads to an increase of the lever arm z. In contrast to this, as it was observed during the experimental tests and numerical simulations as well, highly deformable thin base plates (configurations P1, P3 and P5) produce a rather non-uniform stress distribution. Consequently, a high concentration of stresses is located near the column flange and the lever arm is reduced (see Figure 4.62 and Figure 4.63). The maximum gain in resistance for M0, M90 and M45 are 60%, 48% and 63%, respectively. The increasing of the base plate thickness also affects the initial stiffness of the connections. Although less pronounced in some cases than in others, values of S_{i,ini} are greater for thicker base plates due to a greater lever arm and lower deformation of the base plate. To get a better insight of the gain of resistance for the 20 mm base plate configuration, the *M*-*N* interaction curves are plotted below.



Figure 4.55: M-N interaction for in-plane bending moment for P1 and P2



Figure 4.56: M-N interaction for in-plane bending moment for P3 and P4



Figure 4.57: M-N interaction for in-plane bending moment for P5 and P6



Figure 4.59: M-N interaction for out-of-plane bending moment for P3 and P4



Figure 4.61: M-N interaction for biaxial bending moment for P1 and P2



Figure 4.58: M-N interaction for out-of-plane bending moment for P1 and P2



Figure 4.60: M-N interaction for out-of-plane bending moment for P5 and P6



Figure 4.62: M-N interaction for biaxial bending moment for P3 and P4



for P5 and P6

As observed, the gain in resistance is more or less evident, depending on the configuration of the connection and the applied loading conditions. Interaction curves *M*-*N* demonstrate that varying the base plate thickness has no particular influence on the failure mode. For all cases, although less pronounced for out-of-plane bending, the gain in resistance is substantially higher when the failure mode is governed by the anchor bolts rupture in tension. However, for values of the applied compressive force above $0,5N_{j,c,u}$, the curves overlap each other as a consequence of the local buckling in the column flanges. In these cases, the gain in resistance is insignificant. Under out-of-plane bending moment, the shape of the M-N curve is parabolic. For in-plane bending moment, it seems locally parabolic but in general, most of the segment of the curve is linear. The curve for biaxial bending moment are in between these two cases.

The contact pressure distribution obtained for connections P1 and P2 are presented in Figure 4.64 and Figure 4.65, for in-plane and out-of-plane. If the axial force is equal to zero, the compression area is located close to the lower anchor bolt row for specimen P1 and at the outer edge of the base plate for connection P2. Under the maximal bending moment and the corresponding axial force, failure can occur either by anchor bolts failure or local buckling of the column. For this loading case, the contact spread over the column flange. As it could be expected the size of the compressive area is greater for thicker base plates.







4.4.4.2 Column steel profile

Although the HE steel profiles are considered as the most suitable cross-sections for columns, IPE steel profiles are still widely used for this type of elements. As discussed before, when subjected to high levels of axial compressive force, the column base plate's failure mode is often governed by local instability (buckling) of the column steel profile. Moreover, under out-of-plane bending moment, the lever arm depends strongly on the width of the column flange. Thus, the influence of the steel column cross-section geometry (HEA 200 or IPE 200) column steel profile on the connection response is discussed below. Interaction curves *M-N* depicted in Figure 4.66 to Figure 4.71, compare specimen P1 to specimen P3 and specimen P2 to specimen P4 subjected to strong-axis, weak-axis and biaxial bending moments.

As expected, it can be seen that when the element governing failure is the column, the column steel section has a pronounced influence on the resistance. Under both in-plane and biaxial bending moment combined with a axial compressive force whose magnitude is larger than $0,5N_{j,c,u}$, the gain in resistance when using an HEA 200 steel profile instead of an IPE 200 is very significant.

Under in-plane bending moment combined with a tensile force or low intensity value of the compressive force, the gain in connection resistance is limited. The lever arms being quite close, only the width of the base plate is modified and influence its local bending resistance in the tensile and the compressive areas (see Figure 4.72).

For out-of-plane bending resistance, the gain of resistance also exist in presence of tensile force or low value of the compressive force due to:

- the increase of the anchor bolt pitch that is beneficial in presence of dominant tensile force,
- the increase of the width of the column flange that permit to increase the lever arm in presence of dominant bending moment (the distance between the resultant forces in tension $F_{t,Rd}$ at the level of the activated anchor bolts, and compression $F_{c,Rd}$ around the column flange tips increases for the HEA 200 steel profile see Figure 4.73).

Similar results are obtained with biaxial bending moment.



Figure 4.66: M-N interaction force for in-plane bending moment for P1 and P3



Figure 4.68: M-N interaction force for out-of-plane bending Figure 4.69: M-N interaction force for out-of-plane bending moment for P1 and P3



Figure 4.70: M-N interaction force for biaxial bending moment for P1 and P3



Figure 4.67: M-N interaction force for in-plane bending moment for P2 and P4



moment for P2 and P4



Figure 4.71: M-N interaction force for biaxial bending moment for P2 and P4

The contact pressure distribution obtained for connections P1 and P3 (Figure 4.72 and Figure 4.73), are quite similar whatever the loading conditions. It can be observed that contact pressure due to prying effect is more important in connections P3. For this connection, prying is more important due to a lower width of the base plate.









Figure 4.73: Contact pressures of specimens P1 and P3 at failure for out-of-plane bending moment (in *MPa*)

4.4.4.3 Anchor bolts position

As demonstrated in Chapter 3, the resistance of the connections strongly depends on the distance *z* between the resultant of tensile forces $F_{t,Rd}$, and compressive forces $F_{c,Rd}$. The first being considered permanently located at the centre line of the activated anchor bolts, the value of the distance *z* depends essentially on the position of the resultant of the compressive forces. From Figure 3.86 and Figure 4.36, it can be seen that, as expected the location and the extend of the contact area in compressive side depend on the base plate thickness. As a result of the substantial deformability of thinner base plates, this parameter remain closer to the column flange footprint. Also, from Figure 3.87, it can be seen that the greater the distance between the tensile and compressive resultant forces, the greater the lever arm *z* is and consequently, the greater is the resistance. This fact is not only related to the direction of application (orientation) of the moment but also to the configuration of the base plate and the anchoring system. For that, through the parametric study, it is possible to evaluate the impact of different base plate geometries and the position of the anchor bolts on the yield lines pattern and on the mechanical properties of the connections.

The influence of the different geometries presented in Figure 4.48 can be assessed. This is valid for the cases in which the failure is governed by rupture of the activated anchor bolts. In Figure 4.66, Figure 4.67, Figure 4.70 and Figure 4.71, the part of the curves corresponding to failure of the anchor bolts, occurring for either $0,1N_{j,t,u}$ or $0,9N_{j,t,u}$ in tension or $0,1N_{j,c,u}$ or $0,5N_{j,c,u}$ in compression, is very close. This is easily explained by the fact that when subjected to an inplane or 45° bending moment, both configurations (P1/2 and P3/4) have approximately the same geometry and consequently, no significant changes in terms of resistance and initial stiffness are noticeable. However, for M90 cases, the same is not true. Under an out-of-plane bending moment, the geometry of the base plate and the anchor bolts arrangement are quite different in specimens P1/2 and specimens P3/4. The distance *p* between the centre of the anchor holes changes from 160 mm to 80 mm. The same for the distance *e* between the edge and the centre line of the holes that changes from 70 mm to 40 mm. This variation necessarily reduces the lever arm *z* between the pair of forces $F_{c,Rd}$ and $F_{t,Rd}$ for specimens P3 and P4 which leads to a loss of resistance and a significantly lower initial stiffness.

4.4.4.4 Anchor bolts diameter

As previously concluded, the anchor bolts largely influence the behavior of the connections. For this reason, the parametric study is extended to evaluate the gains in terms of resistance and initial stiffness corresponding to the variation of the anchor bolts diameter. It was the objective of the analysis of connections P5 and P6 that use anchor bolts M20 instead of M16 for connections P1 and P2. From the curves depicted in Figure 4.74 to Figure 4.79, the gain of resistance of anchor bolts M20 over anchor bolts M16 is visible when constituting the elements governing failure. For the other cases where failure is driven by local buckling of the column (axial compressive force above $0,5N_{j,c,u}$), curves overlap.



Figure 4.76: M-N interaction force for out-of-plane bending Figure 4.77: M-N interaction force for out-of-plane bending moment for P1 and P5

moment for P2 and P6



As said, when failure is a result of the rupture of the anchor bolts in tension, a significant improvement of the resistance is observed between the curves corresponding to M16 (red) and M20 anchor bolts (black). The gain in resistance seems logical since increasing the bolt diameter result in a 50% increase of the maximum force to be resisted by these elements. This increase reaches 60% when comparing the response of specimens P1 and P5 for in-plane bending moment. For the same loading case, an increase of about 70% is observed when comparing specimen P2 and specimen P6. For out-of-plane and biaxial bending moment, the gains are also significant, up to 50% between specimens P1 and P5 and 60% between specimens P2 and P6. From the values presented in Table 4.16 to Table 4.33, a visible impact on the initial stiffness $S_{j,ini}$ is also observed. The anchor bolts being more rigid, the flexibility of the connection increases as well as prying effect, which becomes more evident for specimen P5.

The contact pressure distribution obtained for connections P1 and P5 are presented in Figure 4.80 and Figure 4.81.







Laura da Silva Seco





4.4.5 Concluding remarks

The numerical investigations presented at the beginning of this chapter provide detailed and accurate information about the behaviour of the connections subjected to different loading conditions and allow identifying the most influential parameters on this type of joints. The parametric study help to understand the role of each parameter and its influence on the global behavior of the structure in terms of resistance, initial stiffness and rotation capacity. The main conclusions drawn from this study are enumerated below:

- the failure modes strongly depend on the level of the applied axial force and the direction of the bending moment,
- increasing the applied tensile force decreases the bending moment resistance and the initial rotational stiffness whatever the orientation of the bending moment,
- increasing the applied compressive force up to half of the compressive resistance of the connection is advantageous in terms of resistance and initial stiffness for some cases (M0 and M45),
- high levels of applied axial compressive force develop local instabilities/yielding in the column flanges, decreasing the resistance and initial stiffness of the connections,
- increasing the base plate thickness tends to improve the resistance and the initial stiffness of the connections, except for the cases when the applied axial compressive force is too large,
- the use of an HEA steel profile instead of an IPE increases the lever arm in presence of dominant out-of-plane bending moment and consequently, increase the resistance,
- the arrangement of the anchor bolts and the dimensions of the base plate in the direction of the applied loads strongly affect the distance z between the binary of forces $F_{c,Rd}$ and $F_{t,Rd}$, and consequently the resistance of the connections,
- increasing the anchor bolts diameter increases the bending resistance and the initial stiffness of the connections, for the cases in which the failure mode is governed by the anchor bolts in tension.

5. ANALYTICAL MODELS FOR RESISTANCE

5.1 Introduction

The present chapter focuses on the development of simple yet effective mechanical models to estimate the resistance of column base plates subjected to biaxial bending moment and axial force. The models are based on the so-called Component Method presented in EC3-1-8 and their predictions are compared against experimental and numerical results from Chapters 3 and 4. The proposed analytical models applies to column base plates connected to the base with four anchor bolts. The main features of the connection behavior identified during the experimental campaign as well as the numerical study are accounted for:

- the resistance in compression not only considers the resistance provided by the flange but also the additional resistance of the effective area around the web (full analytical model),
- the contact areas and lever arms of the compressive area are not predefined,
- for high levels of applied axial compressive force, yielding can occur in the column. The resistance of the compressive area cannot exceed the resistance of the concrete component but also of the column in compression, limiting the design bearing strength f_{jd} .

5.2 Main assumptions

The connections considered by the proposed model are rigid/semi-rigid exposed column base plates, capable to transfer axial forces and bending moments. These connections consist in a column steel profile welded to a steel base plate, fixed to the concrete foundation by means of four outer anchor bolts. Welds and concrete foundation must be designed according to EC2 Part 1-1 and EC3 Part 1-8 to not fail before the base plate/anchor bolts. Additionally, good contact conditions between the concrete and the embedment length of the anchor bolts is assumed. The model considers a plastic distribution of the internal forces at failure.

Similarly to the Component Method, the analytical models for in-plane and out-of-plane bending considers the connections as a set of basic components. Therefore, the calculation procedure consists in the following steps:

- 1. identification of the individual components in tension and compression,
- 2. characterization of the individual resistance for each of the components,
- 3. assembly of the components resistances to calculate the overall resistance.

The sources of resistance considered in the analytical model are divided in two groups:

- the tensile components consisting on the base plate in bending and the anchor bolts in tension,
- the components in compression including the concrete in compression, the base plate in bending under the bearing pressure on the foundation and the column in compression.

Depending on the loading, the connections can work in two different modes:

- the bending moment is dominant: a tensile and a compressive area are identified and their behavior is modelled by T-stubs in tension/compression. For the full analytical model, the resistance of the web T-stub in compression is added,
- the normal tensile/compressive force is dominant: the connection is fully under compression or tension. The resistance of the connection is reached when either the T-stub in compression fails (for the full analytical model, a third T-stub is considered corresponding to the web of the column), or the resistance of the T-stub in tension is attained.

5.2.1 Components in tension

The components in tension of the analytical model are the base plate in bending and the anchor bolts in tension (see Figure 2.44). As in the Component Method presented in section 2.4.2, the behavior of this component is based on the classic T-stub model, that depends not only on the resistance of the abovementioned elements but also on the stiffness of the anchor bolts which govern the development of prying effect. The resistance of the connection in tension $F_{T,u}$ is the lowest resistance associated to the potential failure modes given by Equations (2.44) and (2.45). It is noted that for the resistance in tension of the anchor bolts, a value of 1 is taken into account for coefficient k_2 , instead of the suggested value 0,9. Partial factors γ_{M0} , γ_{M2} and γ_c are considered equal to 1. For Mode 1, the ultimate tensile strength, f_u , is used instead of the yield strength, f_y , corresponding to the failure resistance. It is also considered that for the resistance corresponding to Mode 2, the base plate yield strength f_y is replaced by $(2f_y+f_u)/3$ (see Packer *et al.*, 1989) due to the fact that f_u cannot be reached on both components (base plate and anchor bolts).

5.2.2 Components in compression

According to the method suggested in Eurocode, the T-stub in compression is modelled considering the contribution of the base plate, concrete foundation and in some cases, the grout layer (see Figure 2.44). As presented in section 2.4.2, the proposed model considers the design resistance $F_{C,u}$ calculated by the procedure given in EN 1992. However, some modifications are made in order to obtain less conservative results, since this aspect should be improved in the Component Method.

From the parametric study, it can be observed that for high levels of applied compressive axial force, the collapse of the connections tends to occur by yielding/buckling of the column. Thus, for the calculation of the resistance of the T-stub in compression, the maximum resistances
from both the concrete foundation and the column are taken into account. The proposed method to evaluate $F_{C,u,f}$ for column flange T-stub is given below

$$F_{\mathrm{C},\mathrm{u},\mathrm{f}} = \min\left(b_{\mathrm{eff},\mathrm{c}}l_{\mathrm{eff},\mathrm{c}}f_{\mathrm{jd},\mathrm{max}};F_{\mathrm{c},\mathrm{u}}\right) = b_{\mathrm{eff},\mathrm{c}}l_{\mathrm{eff},\mathrm{c}}f_{\mathrm{jd}}^{*}$$
(5.1)

$$f_{jd}^{*} = f_{jd,max} \le \frac{F_{c,u}}{b_{eff,c} l_{eff,c}}$$
(5.2)

with

 $b_{\rm eff,c}$, $l_{\rm eff,c}$: column flange T-stub effective length and width in compression,

 $f_{jd,max}$: design bearing strength of concrete calculated according to Eurocode 2 and 3, $F_{c,u}$: maximum resistance of the column in compression that depends on the model considered "simplified analytical model" or "full analytical model".

As seen in Chapter 2, the design resistance of the flange T-stub in compression depends not only on the design bearing strength but also, on the effective width and length of the T-stub, $b_{\rm eff,c}$ and $l_{\rm eff,c}$ calculated as follows (see Figure 2.50):

$$b_{\rm eff,c} = \begin{cases} t_{\rm fc} + 2c & \text{for large projection} \\ \frac{h_{\rm bp} - h_{\rm c}}{2} + t_{\rm fc} + c & \text{for short projection} \end{cases}$$
(5.3)
$$(b_{\rm c} + 2c & \text{for large projection} \end{cases}$$

$$l_{\rm eff,c} = \begin{cases} b_{\rm c} + 2c & \text{for large projection} \\ b_{\rm bp} & \text{for short projection} \end{cases}$$
(5.4)

with

 $t_{\rm fc}$: column flange thickness, c: additional effective width, $h_{\rm bp}$: height of the base plate, $h_{\rm c}$: height of the column cross-section, $b_{\rm bp}$: width of the base plate, $b_{\rm c}$: width of the column cross-section.

In the Component Method, the deformations of the base plate are limited to the elastic ones, assuming a uniform stress under the base plate and ensuring that the yield strength of the base plate is not exceeded. The effective bearing area is based on the effective width c (Equation (2.36)), that consider the elastic bending moment resistance per unit length of the base plate. In the numerical analysis, yielding of the base plate in the compressive area was clearly observed and the plastic bending moment of the base plate was reached. In the proposed analytical model presented in sections 5.3 and 5.4, the effective width c is obtained by equalizing the plastic bending moment resistance per unit length of the base plate (Equation (5.5)) to the bending moment per unit length acting on the base plate, represented by a cantilever of span c (Equation (5.6)):

$$M_{\rm pl} = \frac{1}{4} t_{\rm bp}^2 f_{\rm y, bp}$$
(5.5)

$$M_{\rm bp} = \frac{1}{2} c^2 f_{\rm jd}$$
 (5.6)

Resulting in:

$$c = t_{bp} \sqrt{\frac{f_{y,bp}}{2f_{jd}}}$$
(5.7)

with

 $M_{\rm pl}$: plastic bending moment resistance per unit length of the base plate,

 $M_{\rm bp}$: bending moment per unit length acting on the base plate,

 $t_{\rm bp}$: thickness of the base plate,

 $f_{y,bp}$: yield strength of the base plate.

This way, the value of the effective width c is increased and thus the effective bearing area under the base plate, allowing to obtain values of the effective width closer to those obtained by numerical simulations and therefore less conservative. Numerical analysis show that fillet welds permit to transfer compressive forces to the foundation. The width of the fillet welds is also added and the plastic bending moment is supposed to develop at the weld toe. Then, the additional effective width c is replaced by a modified effective width c^* :

$$c^* = t_{\rm bp} \sqrt{\frac{f_{\rm y,bp}}{2f_{\rm jd,max}}} + \sqrt{2}a$$
 (5.8)

with

a : weld throat thickness.

Thus, the new effective length and width of the T-stubs in compression are taken as follows: For T-stub 1 and 3 (column flanges – see Figure 2.52):

$$b_{\rm eff,c} = t_{\rm fc} + 2c^* \tag{5.9}$$

$$b_{\rm eff,c} = b_{\rm c} + 2c^* \le b_{\rm bp}$$
 (5.10)

For T-stub 2 (column web – see Figure 2.52):

$$b_{\rm eff,w} = t_{\rm wc} + 2c^* \tag{5.11}$$

$$l_{\rm eff,w} = h_{\rm c} - 2c^* - 2t_{\rm fc} \ge 0 \tag{5.12}$$

5.3 In-plane bending moment

5.3.1 Simplified analytical model

As abovementioned, the model herein assumes different internal forces systems, according to the level of the applied loading, excluding the consideration of the additional resistance provided by the column web in compression, as shown in Figure 5.1. The analytical model considers that the connections can work according to four distinct behaviour types (see Figure

5.1). This assumption allows to simplify the application of the calculation procedure and to consider that the position of the resultant force in compression F_c is not located at the footprint of the column flange. Then, depending on the level of the applied axial force N, the model is divided into:

- dominant tensile force,
- dominant bending moment with critical tensile force,
- dominant bending moment with critical compressive force,
- dominant compressive force.



compressive force – behaviour type 3

	Table 5.1: Validity range of the mechanisms of the simplified analytical model						
Behaviour type	Loading	Range of validity					
1	Dominant tensile force	$-N_{\mathrm{T},\mathrm{u}} \leq N \leq N_1$					
		$N_1 = -F_{T,u}; N_{T,u} = 2F_{T,u}$					
2	Dominant bending moment with critical tensile force	$N_1 < N \le N_2$					
		$N_2 = F_{\mathrm{C,u}} - F_{\mathrm{T,u}}$					
3	Dominant bending moment with critical compressive force	$N_2 < N \le N_3$					
		$N_3 = F_{C,u}$					
4	Dominant compressive force	$N_3 < N \leq N_{\mathrm{C,u}}$					
		$N_{\rm C,u} = 2F_{\rm C,u}$					

The validity domains of the aforementioned mechanisms are summarized below:

In presence of dominant tensile force (see Figure 5.1 a)), Eurocode 3 Part 1-8 is applied for the determination of the ultimate bending moment resistance $M_{j,ip,u}$. The internal forces system contain two resultant forces $F_{T,u}$ and F_T acting on both sides at the anchor bolts axes. Then, the in-plane bending resistance is given by:

$$M_{j,ip,u} = 2z_{\rm T} F_{\rm T,u} + z_{\rm T} N \tag{5.13}$$

with

 $F_{T,u}$: resistance of the T-stub in tension according to 5.2.1,

N: axial force.

 $z_{\rm T}$: lever arm (distance between the line of action of the anchor bolts and the center of the column).

In presence of dominant bending moment and critical tensile force (see Figure 5.1 b)), the position of the neutral axis depends on the axial force and the resistance of the tensile area. The equilibrium of axial forces gives:

$$N = f_{id}^{*} l_{eff,c} x_{c} - F_{T,u}$$
(5.14)

with

$$f_{jd}^* = \frac{N_{C,u}}{2b_{\text{eff,c}}l_{\text{eff,c}}} \le f_{jd,\max}$$
(5.15)

In the present case, the compressive resistance of the column, $F_{c,u}$, is taken equal to $N_{c,u}/2$ that corresponds to half of the total resistance of the column in compression. It is the only possibility to match with the pure compressive resistance of the column and it suppose that strain hardening can develop locally on the column flange. It assumes compact flange and can not be considered with flanges of class 4 that are not commonly used in practice.

One obtain the position of the neutral axis x_c :

$$x_{\rm c} = \frac{N + F_{\rm T,u}}{f_{\rm jd}^* l_{\rm eff,c}} \le t_{\rm fc} + 2c^*$$
(5.16)

The in-plane bending resistance is thus:

$$M_{j,ip,u} = (z_T + z_C)F_{T,u} + z_C N$$
(5.17)

with

 $z_{\rm C}$: compressive lever arm (distance between the line of action of the resultant compressive force $F_{\rm C}$ and the centre of the column):

$$z_{\rm C} = \frac{h_{\rm c}}{2} + c^* - \frac{x_{\rm c}}{2} \le z_{\rm C0}$$
(5.18)

 z_{C0} : compressive lever arm given by EC3-1-8 (distance between the action line of the resultant compressive force F_{C} located at the column flange and the centre of the column).

In presence of dominant bending moment and critical compressive force (see Figure 5.1 c)), the force applied in the tensile side is lower than $F_{T,u}$ and the compressive force, located behind the column flange, is equal to $F_{C,u}$. The in-plane bending resistance is determined according to Eurocode 3 part 1-8:

$$M_{j,ip,u} = (z_T + z_{C0})F_{C,u} - z_T N$$
(5.19)

with

$$F_{\rm C,u} = b_{\rm eff,c} l_{\rm eff,c} f_{\rm id}^{*}$$
(5.20)

In presence of dominant compressive force (see Figure 5.1 d)) two compressive forces $F_{\rm C}$ and $F_{\rm C,u}$ are applied behind the column flange on the left and on the right sides, respectively. The ultimate bending moment resistance is calculated according to:

$$M_{j,ip,u} = 2F_{C,u}z_{C0} - z_{C0}N$$
(5.21)

An example of the application of this calculation procedure for the calculation of the design resistance is given in the appendix at the end of this chapter (section B.2).

5.3.2 Full analytical model

As mentioned in Chapter 2, studies carried out by Wald (2000) and Márai *et al.* (2014) highlighted the influence of the additional resistance provided by the concrete area located around the column web for base plates subjected to a combination of compressive force and inplane bending moment. This additional resistance clearly increases the resistance of the connection in presence of dominant compressive force or bending moment comparatively to values given in Eurocode 3 Part 1-8. The full analytical model proposed herein consists in an improvement of the method presented in 5.3.1, considering the additional resistance provided by the concrete around the column web in compression. In addition, the force transferred by the anchor bolts in tension decreases when more than half of the web is under compression. The calculation procedure allows to solve the problem of the discontinuity of the *M-N* interaction curve produced by the Component Method, when the connection changes from being subjected to a combination of axial force and bending moment (and therefore does not consider the area

of the column web in compression) to pure compression where the maximum resistance $N_{C,u}$ is defined by the sum of the resistances of the T-stubs 1, 2 and 3 depicted in Figure 2.52. Similarly to the simplified analytical model, the calculation procedure consists in identifying four types of internal forces systems, depending on the value of the axial force *N*. This allows to simplify the determination of the web contribution to the ultimate bending moment resistance. In addition, the model considers the failure of the column.

The four working behaviour types of the connection are:

- dominant tensile force,
- dominant bending moment with critical tensile force,
- dominant bending moment with critical compressive force,
- dominant compressive force.



a) Dominant tensile force – behaviour type 1





d) Dominant bending moment with critical compressive force – mechanism behaviour type 4



e) Dominant compressive force – behaviour type 5 **Figure 5.2:** Mechanisms considered in the full analytical model

The validity domains of the aforementioned mechanisms are summarized below:

Behaviour type	Loading	Range of validity
1	Dominant tensile force	$-N_{\mathrm{T,u}} \le N \le N_1$
		$N_1 = -F_{T,u}; N_{T,u} = 2F_{T,u}$
2	Dominant bending moment with critical tensile force	$N_1 < N \le N_2$
		$N_2 = F_{\mathrm{C,u,f}} - F_{\mathrm{T,u}}$
3	Dominant bending moment with critical tensile force	$N_2 < N \le N_3$
		$N_3 = F_{\rm C,u,f} + F_{\rm C,u,w}/2 - F_{\rm T,u}$
4	Dominant bending moment with critical compressive force	$N_2 < N \le N_3^*$
		$N_3^* = F_{C,u,f} + F_{C,u,w}$
5	Dominant compressive force	$N_3^* < N \le N_{\mathrm{C,u}}$
		$N_{\rm C,u} = 2F_{\rm C,u,f} + F_{\rm C,u,w}$

Table 5.2: Validity range of the mechanisms of the full analytical mode

The resistance obtained under dominant tensile force (see Figure 5.2 a)) is similar to that of the simplified analytical model. The ultimate in-plane bending moment resistance is given by:

$$M_{j,u,ip} = 2z_{\rm T} F_{\rm T,u} + z_{\rm T} N \tag{5.22}$$

Again behaviour type 2 (see Figure 5.2 b)) is quite similar to that of the simplified analytical model. However, the bearing resistance is not limited to half of the resistance of the column in compression but to the resistance in compression of the flange and column web evaluated according to Eurocode 3 part 1-8, $F_{c,fc,u}$.

$$f_{\rm jd,f}^{*} = \frac{F_{\rm c,fc,u}}{b_{\rm eff,c} l_{\rm eff,c}} \le f_{\rm jd,max}$$
(5.23)

$$F_{c,fc,u} = \frac{W_{pl,c}f_{y,c}}{h_c - t_{fc}}$$
(5.24)

with

 $W_{pl,c}$: column cross-section plastic modulus, $f_{y,c}$: column yield strength.

The in-plane bending resistance is thus:

$$M_{j,jp,u} = (z_{T} + z_{C})F_{T,u} + z_{C}N$$
(5.25)

with

$$x_{\rm c} = \frac{N + F_{\rm T,u}}{f_{\rm jd,f}^{*} l_{\rm eff,c}} \le t_{\rm fc} + 2c^{*}$$
(5.26)

$$z_{\rm C} = \frac{h_{\rm c}}{2} + c^* - \frac{x_{\rm c}}{2} \tag{5.27}$$

Behaviour type 3 (see Figure 5.2 c)) corresponds to the case in which the effective width of the column flange $b_{\text{eff,c}}$ is fully under compression ($x_c \ge t_{\text{fc}} + 2c^*$). As a result, $F_{\text{C,u,f}}$ and $F_{\text{T,u}}$ can develop. Also, part of the column web is contributing to the bending moment resistance, given by $F_{\text{C,w}}$. This system of internal forces is valid provided that the following criteria are respected:

$$x_{\rm c} = b_{\rm eff,c} + x_{\rm w} \tag{5.28}$$

$$x_{\rm w} \le z_{\rm w} \tag{5.29}$$

with

 $z_{\rm w}$: half of the total effective length of the web $l_{\rm eff,w}$ in compression computed according to Equation (5.12),

 $x_{\rm w}$: portion of the effective web in compression.

The equilibrium of axial forces gives:

$$N = F_{\rm C,u,f} + f_{\rm jd,w} b_{\rm eff,w} x_{\rm w} - F_{\rm T,u}$$
(5.30)

with

 $F_{C,u,f}$: resistance of the column flange T-stub in compression according to 5.2.2 considering $f_{jd} = f_{jd,f}^*$,

 $b_{\rm eff,w}$: column web effective width according to Equation (5.11),

 $f_{jd,w}$: design bearing strength of the web:

$$f_{jd,w} = \frac{N_{C,u} - 2F_{C,u,f}}{b_{eff,w}z_w} \le f_{jd,max}$$
(5.31)

Thus the length of the web in compression is:

$$x_{\rm w} = \frac{N + F_{\rm T,u} - F_{\rm C,u,f}}{b_{\rm eff,w} f_{\rm jd,w}}$$
(5.32)

The in-plane bending moment resistance is given by:

$$M_{j,ip,u} = F_{T,u}z_{T} + z_{C0}F_{C,u,f} + x_{w}b_{eff,w}f_{jd,w}\left(z_{w} - \frac{x_{w}}{2}\right)$$
(5.33)

Behaviour type 4 (see Figure 5.2 d)) corresponds to the case in which the effective width of the column flange and more than half of the web effective length are fully under compression. This system of internal forces is valid provided that the following criteria are respected:

$$b_{\rm eff,c} + z_{\rm w} \le x_{\rm c} \le b_{\rm eff,c} + 2z_{\rm w} \tag{5.34}$$

$$z_{\rm w} \le x_{\rm w} \le 2z_{\rm w} \tag{5.35}$$

With more than half of the effective area of the base plate under compression, it has been observed numerically that failure does not occur in the tensile area and, as a result, it is proposed to reduce the tensile force $F_{\rm T}$ considering the following expression:

$$F_{\rm T} = F_{\rm T,u} \frac{2z_{\rm w} - x_{\rm w}}{z_{\rm w}} \le F_{\rm T,u}$$
(5.36)

The tensile force transmitted by the anchor bolts is thus equal to zero when the left T-stub in compression start to be activated.

The equilibrium of the axial forces gives:

$$N = F_{\mathrm{C,u,f}} + f_{\mathrm{id,w}} b_{\mathrm{eff,w}} x_{\mathrm{w}} - F_{\mathrm{T}}$$

$$(5.37)$$

Inserting (5.36) in (5.37), one obtain the length of the web in compression:

$$x_{\rm w} = \frac{N + 2F_{\rm T,u} - F_{\rm C,u,f}}{b_{\rm eff,w} f_{\rm jd,w} + \frac{F_{\rm T,u}}{z_{\rm w}}}$$
(5.38)

The in-plane bending moment resistance is given by:

$$M_{j,ip,u} = F_{T}z_{T} + z_{C0}F_{C,u,f} + x_{w}b_{eff,w}f_{jd,w}\left(z_{w} - \frac{x_{w}}{2}\right)$$
(5.39)

Behaviour type 5 (see Figure 5.2 e)) corresponds to the dominant compressive force. The ultimate in-plane bending moment resistance is calculated according to:

$$M_{j,ip,u} = \left[b_{eff,c} - (x_c - x_w - 2z_w)\right] l_{eff,c} f_{jd,f} \left[\frac{h_c}{2} + c^* - \frac{\left(b_{eff,c} - (x_c - x_w - 2z_{wc})\right)}{2}\right]$$
(5.40)

with

$$x_{\rm w} = \frac{N - F_{\rm C,u,f} - F_{\rm C,u,w} + 2z_{\rm w} l_{\rm eff,c} f_{\rm jd,f}}{l_{\rm eff,c} f_{\rm jd,f}}$$
(5.41)

An example of the application of this calculation procedure is given in the appendix at the end of this chapter (section B.2).

5.3.3 Comparison against experimental and numerical results

• Comparison against the experimental tests of INSA Rennes

The bending moment resistances obtained from the analytical approach are compared against the experimental tests carried out at INSA Rennes and presented in detail in sub-chapter 3.7. The ultimate bending moment resistances as well as the ratio between the experimental and analytical results and the failure modes are presented in Table 5.3. The ratio between results is very satisfactory.

Table 5.3: Comparison of the analytical results against the experimental tests for in-plane bending moment							
	Experimental		Analytical		Ana/Exp		
Specimen	Maximum bending	Failura mode	Maximum bending	Failure mode	M:		
	moment $M_{j,u,exp}$ (kNm)	Failure mode	moment M _{j,u,ana} (kNm)	Fanure mode	<i>141</i> j,u		
SPE1-M0	43,2	Mode 2 [*]	43,4	Mode 2	1,00		
SPE2-M0	48,5	Mode 3	50,7	Mode 3	1,04		
PB-1	71,0	Mode 2	67,0	Mode 2	0,94		
PB-2	82,3	Mode 2	81,2	Mode 2	0,99		

*During tests, prying effect did not develop due to initial imperfection. However, Mode 2 is still considered for this case, since the base plate yielded.

• Comparison against the numerical results of the parametric study

The *M-N* interaction curves corresponding to the bending moment resistances obtained from both models presented in 5.3.1 and 5.3.2 are compared against the curves given by the parametric study. In most cases, the resistance obtained by the analytical models is lower than the numerical resistances and so the models can be used with confidence as their predictions are mostly on the safe side. The differences between the numerical and analytical results, for both proposed models, exhibit better agreement when the failure occurs by anchor bolt rupture in tension than by column yielding. This fact has to do with the complexity in characterizing the behavior of the assembly when subjected to high axial compressive loads and also to the development of strain hardening. For the cases in which the connections were subjected to a tensile force, the results obtained by the models are identical (blue and green curves). The maximum bending resistance obtained with the simplified analytical model is slightly unsafe for specimens P5 and P6 mainly due to the fact that half the resistance of the column in compression is considered in the T-stub in compression. For safer results it could be preferable to replace it by the resistance of the column flange and web in compression as for the full analytical model.





5.4 Out-of-plane bending moment

5.4.1 Analytical model/general hypothesis

A simplified model is here proposed for the calculation of the resistance of column base plates subjected to axial force and out-of-plane bending moment. The final proposal is based on a parabolic interaction that was suggested by the parametric study. Before developing these equations, it is shown that with the force distribution presented in Figure 5.9, the parabolic interaction can be adequate. It is assumed that the out-of-plate bending moment is balanced by an uniformly distributed force in the tensile and compressive area along the effective width of the column flange $b_{\text{eff,c}}$ (see Figure 5.9). Concerning the compressive area, this hypothesis seems rational. For the tensile area, the lever arm can be underestimated or overestimated (depending on the position of the anchor bolts) and so the maximum bending moment. To evaluate the contribution of both compressive and tensile sides $f_{c,u}$ and $f_{t,u}$, the position of the neutral axis x_c must be determined.



Figure 5.9: Mechanism of internal forces considered in the analytical model for out-of-plane bending moment

The uniformly distributed resistances along the column flange effective width $f_{t,u}$ and $f_{c,u}$ are calculated as follows:

$$f_{t,u} = \frac{N_{T,u}}{b_{eff,c}}$$
(5.42)

$$f_{\rm c,u} = \frac{N_{\rm C,u}}{b_{\rm eff,c}} \tag{5.43}$$

with

 $N_{T,u}$: tensile resistance of the connection,

 $N_{C,u,}$: compressive resistance of the connection,

 $b_{\text{eff,c}}$: column flange effective width according to Equation (5.9).

The equilibrium of axial forces gives:

$$N = f_{c,u} x_c - f_{t,u} (b_{eff,c} - x_c)$$
(5.44)

Inserting (5.42) and (5.43) in (5.44), one obtain the position of the neutral axis x_c :

$$x_{\rm c} = b_{\rm eff,c} \frac{N + N_{\rm T,u}}{N_{\rm C,u} + N_{\rm T,u}}$$
(5.45)

The ultimate bending moment resistance $M_{j,op,u}$ is obtained from:

$$M_{j,op,u} = M_{C,u} + M_{T,u} + N\left(\frac{b_{eff,c}}{2} - x_c\right)$$
(5.46)

with

 $M_{C,u}$: bending moment about neutral axis produced by the force acting in the compressive area:

$$M_{\rm C,u} = f_{\rm c,u} \cdot \frac{x_{\rm c}^2}{2}$$
(5.47)

 $M_{T,u}$: bending moment about neutral axis produced by the force acting in tensile area:

$$M_{\rm T,u} = f_{\rm t,u} \cdot \frac{\left(b_{\rm eff,c} - x_{\rm c}\right)^2}{2}$$
(5.48)

The previous equation is parabolic (see Figure 5.10) and can be conveniently rewritten as:

$$M_{\rm j,op,u} = M_{\rm op,u,max} \left[1 - \left(\frac{N - N_{\rm Mmax}}{N_{\rm moy}} \right)^2 \right]$$
(5.49)

with

$$N_{\rm Mmax} = \frac{N_{\rm C,u} - N_{\rm T,u}}{2}$$
(5.50)

$$N_{\rm moy} = \frac{N_{\rm C,u} + N_{\rm T,u}}{2}$$
(5.51)

$$M_{\rm op,u,max} = \frac{b_{\rm eff,c}}{4} \frac{N_{\rm C,u} + N_{\rm T,u}}{2}$$
(5.52)



Figure 5.10: Parabolic interaction curve M-N

The choice of $b_{\text{eff,c}}$ is acceptable for the compressive area, but the proposal regarding the tensile area is rather questionable. The lever arm can be underestimated or overestimated (depending on the position of the anchor bolts) and so the maximum bending moment. To avoid these inconsistencies, the contribution of the tensile area to the maximum bending moment is evaluated considering that the tensile resistance develops at the anchor bolts location. The maximum bending moment becomes:

$$M_{\rm op,u,max} = z_{\rm T,op} F_{\rm T,u} + z_{\rm C,op} F_{\rm c,f,u}$$
 (5.53)

with

$$F_{\rm c,f,u} = b_{\rm eff,c} l_{\rm eff,c} f_{\rm jd,max} \le b_{\rm c} t_{\rm fc} f_{\rm y,c}$$
(5.54)

Parameters $z_{T,op}$, $z_{C,op}$ are given by:

$$z_{\mathrm{T,op}} = \frac{w}{2} \tag{5.55}$$

$$z_{\rm C,op} = \frac{b_{\rm c}}{4} + \frac{c^*}{2} \le \frac{b_{\rm bp}}{4}$$
(5.56)

An example of the application of this calculation procedure is given in the appendix at the end of this chapter (section B.4).



Figure 5.11: Mechanism of internal forces considered for the calculation of $M_{op,max}$

5.4.2 Comparison against experimental and numerical results

• Comparison against experimental test results

The bending moment resistances obtained with the model described in 5.4.1 are compared against the experimental tests carried out at INSA Rennes and presented in detail in sub-chapter 3.7. The ultimate moment bending resistances as well as the ratio between the experimental and analytical results and the failure modes are given in Table 5.4.

	Experimental	6	Analytical		Ana/Exp
Specimen	Maximum bending	Failura moda	Maximum bending	Esilura moda	M
	moment $M_{j,u,exp}$ (kNm)	Failure mode	moment M _{j,u,ana} (kNm)	Failure mode	1 /1 j,u
SPE1-M90	36,2	Mode 2	33,3	Mode 2	0,92
SPE2-M90	43,7	Mode 3	40,9	Mode 3	0,94

Table 5.4. Comparison of the analytical results against the experimental tests for out-or-plane bending moment

The results presented above show a fairly good agreement between the experimental tests and the predictions of the proposed analytical models for the estimation of the out-of-plane bending resistance of column base plates. For SPE1-M90 and SPE2-M90, the proposed model predict the resistance on the safe side with an error of 8% and 6%, respectively. Moreover, the failure modes predicted are identical to those observed experimentally.

• Comparison against the numerical results of the parametric study

The bending moment resistances obtained from the model described in 5.4.1 are compared against the results of the parametric study (see sub-chapter 4.4). The interactions curves M-N produced by the parametric study and those predicted by the analytical models are depicted below. In general, the values predicted by the analytical models are in good agreement with the numerical models. Except for specimens P1 and P4, in which the resistance is overestimated in certain cases, the analytical curves are circumscribed by the numerical interaction curves.





5.5. Biaxial bending moment

5.5.1 Analytical model/general hypotheses

Recent studies on the behavior of column base plates subjected to bending moment about both strong and weak axis, discussed in section 2.3.3 of this thesis, proposed elliptical curves for the interaction between in-plane and out-of-plane bending moments. The parametric study of the present report confirmed that this proposal is adequate in presence of H profile and dominant bending moment or compressive force. For I profile, this elliptical interaction is quite conservative, whereas it becomes unsafe in presence of a dominant tensile force. For these reason, a more rigorous model, based on mechanical principles, is necessary for the evaluation of the interaction between in-plane and out-of-plane bending moments.



Figure 5.18: Plastic interaction curves *M*₀-*M*₉₀-*N*

The analytical model presented here for the resistance of column base plates subjected to an axial force and bending moment in both directions $M_{j,\alpha,u}$ consists in dividing and analyzing the applied forces on both left and right sides of the column base plate (see Figure 5.19). Both sides of the connections reach their plastic bending resistance and thus the connection should be sufficiently ductile. Following the procedures described in section in sections 5.3 and 5.4,

the analytical model is divided into three cases, according to the presence of tensile or compressive forces applied to the sides of the connection. Then, depending on the level of the axial force N, the model is divided into:

• dominant tensile force (see Figure 5.19-a),

* * * * *

- dominant bending moment (see Figure 5.19-b),
- dominant compressive force (see Figure 5.19-c).



The validity domains of the aforementioned loading type are summarized below:

	Table 5.5: Validity rang	ge of the mechanisms of the analytical model
Behaviour type	Loading	Range of validity
1	Dominant tensile force	$-N_{\mathrm{T,u}} \leq N \leq N_1$ and $M_{\mathrm{ip}} \leq 2z_{\mathrm{T}}F_{\mathrm{T,u}} + z_{\mathrm{T}}N$
		$N_1 = -F_{T,u}; N_{T,u} = 2F_{T,u}$
2	Dominant bending moment	$N_1 < N \le N_2$ and $M_{ip} \le \min\left((z_C + z_T)F_{T,u} + z_C N; (z_C + z_T)F_{C,u} - z_T N\right)$
		$N_2 = F_{C,u}$
3	Dominant compressive force	$N_2 < N \le N_{\rm C,u}$ and $M_{\rm ip} \le 2 z_{\rm C} F_{\rm C,u} - z_{\rm C} N$
		$N_{\rm C,u} = 2F_{\rm C,u}$

Dominant tensile force (see Figure 5.19-a) consists in a system of internal forces acting on both sides left and right. $F_{t,l}$ and $F_{t,r}$ that result from the axial force N and the in-plane bending moment M_{ip} are obtained as follows:

$$F_{\rm t,l} = \frac{N}{2} - \frac{M_{\rm ip}}{2z_{\rm T}} < 0 \tag{5.57}$$

$$F_{\rm t,r} = \frac{N}{2} + \frac{M_{\rm ip}}{2z_{\rm T}} < 0 \tag{5.58}$$

with

 z_{T} : lever arm (distance between the anchor bolts axis and the center of the connection).

These forces are applied on each side of the connection concomitantly with the out-of-plane bending moments $M_{op,l}$ and $M_{op,r}$. These two bending moments equilibrate the total out-of-plane bending moment applied to the connection:

$$M_{\rm op,u} = M_{\rm op,l} + M_{\rm op,r} \tag{5.59}$$

At failure, the resistance is reached at each side of the connection. Considering the parabolic interaction curve proposed in the previous paragraph, the out-of-plane bending moment of each side can be calculated.

On the left side, the limit bending moment is:

$$M_{\rm op,l} = \frac{M_{\rm op,u,max}}{2} \left[1 - \left(\frac{F_{\rm t,l} - \frac{N_{\rm Mmax}}{2}}{\frac{N_{\rm moy}}{2}} \right)^2 \right]$$
(5.60)

with N_{Mmax} , N_{moy} and $M_{\text{op},u,\text{max}}$ calculated from Equations (5.50), (5.51) and (5.53).

Inserting (5.57) in (5.60), Equation (5.60) becomes:

$$M_{\rm op,l} = \frac{M_{\rm op,u,max}}{2} \left[1 - \left(\frac{N - N_{\rm Mmax} - \frac{M_{\rm ip}}{z_{\rm T}}}{N_{\rm moy}} \right)^2 \right]$$
(5.61)

On the right side, the limit bending moment is:

$$M_{\rm op,r} = \frac{M_{\rm op,u,max}}{2} \left[1 - \left(\frac{F_{\rm t,r} - \frac{N_{\rm Mmax}}{2}}{\frac{N_{\rm moy}}{2}} \right)^2 \right]$$
(5.62)

Inserting (5.58) in (5.62), Equation (5.62) becomes:

$$M_{\rm op,r} = \frac{M_{\rm op,u,max}}{2} \left[1 - \left(\frac{N - N_{\rm Mmax} + \frac{M_{\rm ip}}{z_{\rm T}}}{N_{\rm moy}} \right)^2 \right]$$
(5.63)

Finally, inserting expressions (5.61) and (5.63) in (5.59), an expression for the out-of-plane bending resistance is obtained:

$$M_{\rm op,u} = M_{\rm op,u,max} \left[1 - \left(\frac{N - N_{\rm Mmax}}{N_{\rm moy}} \right)^2 - \left(\frac{M_{\rm ip}}{z_{\rm T} N_{\rm moy}} \right)^2 \right]$$
(5.64)

For the specific case of a 45° biaxial bending moment where:

$$M_{\rm op} = M_{\rm ip} = M_{\rm j,45,u} = \frac{M_{\rm 45}}{\sqrt{2}}$$
(5.65)

Resistance $M_{i,45,u}$ is obtained by solving a quadratic equation as follows:

$$M_{j,45,u}^{2}a_{\rm T} + M_{j,45,u}b_{\rm T} + c_{\rm T} = 0$$
(5.66)

with

$$\Delta_{\rm T} = b_{\rm T}^2 - 4a_{\rm T}c_{\rm T} \tag{5.67}$$

$$a_{\rm T} = \frac{1}{\left(z_{\rm T} N_{\rm moy}\right)^2} \tag{5.68}$$

$$b_{\rm T} = \frac{1}{M_{\rm op,u,max}} \tag{5.69}$$

$$c_{\rm T} = \left(\frac{N - N_{\rm Mmax}}{N_{\rm moy}}\right)^2 - 1 \tag{5.70}$$

The positive solution of this equation is:

$$M_{j,45,u} = \frac{-b_{\rm T} + \sqrt{\Delta_{\rm T}}}{2a_{\rm T}}$$
(5.71)

In presence of dominant bending moment (see Figure 5.19-b), a tensile force is applied on the left side $F_{t,l}$ and a compressive force on the right side $F_{c,r}$, that can be obtained by:

$$F_{t,l} = -\frac{M_{ip}}{2\bar{z}} + \frac{\alpha_{\rm T}N}{2} < 0$$
(5.72)

$$F_{\rm c,r} = \frac{M_{\rm ip}}{2\bar{z}} + \frac{\alpha_{\rm C}N}{2}$$
(5.73)

with

$$\overline{z} = \frac{z_{\rm C} + z_{\rm T}}{2} \tag{5.74}$$

$$\alpha_{\rm T} = \frac{z_{\rm C}}{\bar{z}} \tag{5.75}$$

$$\alpha_{\rm C} = \frac{z_{\rm T}}{\bar{z}} \tag{5.76}$$

These forces applied on each side of the connection concomitantly with the out-of-plane bending moments $M_{op,l}$ and $M_{op,r}$, and ultimate limit states is reached on each side. Again, considering the parabolic interaction curve, the limit for the out-of-plane bending moments are: On the left side:

$$M_{\rm op,l} = \frac{M_{\rm op,u,max}}{2} \left[1 - \left(\frac{F_{\rm t,l} - \frac{N_{\rm Mmax}}{2}}{\frac{N_{\rm moy}}{2}} \right)^2 \right]$$
(5.77)

On the right side:

$$M_{\rm op,r} = \frac{M_{\rm op,u,max}}{2} \left[1 - \left(\frac{F_{\rm c,r} - \frac{N_{\rm Mmax}}{2}}{\frac{N_{\rm moy}}{2}} \right)^2 \right]$$
(5.78)

Inserting equation (5.72) and (5.73) in (5.77) and (5.78) respectively it is obtained: On the left side:

$$M_{\rm op,l} = \frac{M_{\rm op,u,max}}{2} \left[1 - \left(\frac{\alpha_{\rm T} N - N_{\rm Mmax} - \frac{M_{\rm ip}}{\bar{z}}}{N_{\rm moy}} \right)^2 \right]$$
(5.79)

On the right side:

$$M_{\rm op,r} = \frac{M_{\rm op,u,max}}{2} \left[1 - \left(\frac{\alpha_C N - N_{\rm Mmax} + \frac{M_{\rm ip}}{\bar{z}}}{N_{\rm moy}} \right)^2 \right]$$
(5.80)

The total out-of-plane bending moment resistance is thus:

$$M_{j,op,u} = M_{op,u,max} \left[1 - \frac{1}{2} \left(\frac{\alpha_{\rm T} N - N_{\rm Mmax}}{N_{\rm moy}} \right)^2 - \frac{1}{2} \left(\frac{\alpha_{\rm C} N - N_{\rm Mmax}}{N_{\rm moy}} \right)^2 - \left(\frac{M_{\rm ip}}{\bar{z} N_{\rm moy}} \right)^2 - \frac{M_{\rm ip}}{2\bar{z} N_{\rm moy}^2} N(\alpha_{\rm C} - \alpha_{\rm T}) \right] (5.81)$$

For the specific case of a 45° biaxial bending moment where:

$$M_{\rm op} = M_{\rm ip} = M_{\rm j,45,u} = \frac{M_{45}}{\sqrt{2}}$$
 (5.82)

Resistance $M_{j,45,u}$ is obtained by solving a quadratic equation as follows:

$$M_{j,45,u}^2 a_M + M_{j,45,u} b_M + c_M = 0$$
(5.83)

with

$$\Delta_{\rm M} = b_{\rm M}^2 - 4a_{\rm M}c_{\rm M} \tag{5.84}$$

$$a_{\rm M} = \frac{1}{\left(\bar{z}N_{\rm moy}\right)^2} \tag{5.85}$$

$$b_{\rm M} = \frac{1}{M_{\rm op,u,max}} + \frac{N(\alpha_{\rm C} - \alpha_{\rm T})}{\bar{z}N_{\rm moy}^2}$$
(5.86)

$$c_{\rm M} = \frac{1}{2} \left[\frac{\left(\alpha_{\rm C} N - N_{\rm Mmax} \right)^2 + \left(\alpha_{\rm T} N - N_{\rm Mmax} \right)^2}{N_{\rm moy}^2} \right] - 1$$
(5.87)

The positive solution of this equation is:

$$M_{\rm j,45,u} = \frac{-b_{\rm M} + \sqrt{\Delta_{\rm M}}}{2a_{\rm M}}$$
(5.88)

In presence of dominant compressive force (see Figure 5.19-c)), the system of internal forces $F_{c,l}$ and $F_{c,r}$, resulting from the applied axial load N and the in-plane bending moment M_{ip} is calculated from:

$$F_{\rm c,l} = \frac{N}{2} - \frac{M_{\rm ip}}{2z_{\rm C0}} > 0 \tag{5.89}$$

$$F_{\rm c,r} = \frac{N}{2} + \frac{M_{\rm ip}}{2z_{\rm C0}} > 0 \tag{5.90}$$

These forces are applied on each side of the connection concomitantly with the out-of-plane bending moments $M_{op,l}$ and $M_{op,r}$ and failure is reached on each side. Again, considering the parabolic interaction curve, the limit out-of-plane bending moments are: On the left side:

$$M_{\rm op,1} = \frac{M_{\rm op,u,max}}{2} \left[1 - \left(\frac{F_{\rm c,1} - \frac{N_{\rm Mmax}}{2}}{\frac{N_{\rm moy}}{2}} \right)^2 \right]$$
(5.91)

On the right side:

$$M_{\rm op,r} = \frac{M_{\rm op,u,max}}{2} \left[1 - \left(\frac{F_{\rm c,r} - \frac{N_{\rm Mmax}}{2}}{\frac{N_{\rm moy}}{2}} \right)^2 \right]$$
(5.92)

Inserting equation (5.89) and (5.90) in (5.91) and (5.92) respectively it is obtained: On the left side:

$$M_{\rm op,l} = \frac{M_{\rm op,u,max}}{2} \left[1 - \left(\frac{N - N_{\rm Mmax} - \frac{M_{\rm ip}}{z_{\rm C}}}{N_{\rm moy}} \right)^2 \right]$$
(5.93)

On the right side:

$$M_{\rm op,r} = \frac{M_{\rm op,u,max}}{2} \left[1 - \left(\frac{N - N_{\rm Mmax} + \frac{M_{\rm ip}}{z_{\rm C}}}{N_{\rm moy}} \right)^2 \right]$$
(5.94)

The total out-of-plane bending moment resistance is thus:

$$M_{\rm j,op,u} = M_{\rm op,u,max} \left[1 - \left(\frac{N - N_{\rm Mmax}}{N_{\rm moy}} \right)^2 - \left(\frac{M_{\rm ip}}{z_{\rm C} N_{\rm moy}} \right)^2 \right]$$
(5.95)

For the specific case of a 45° biaxial moment where:

$$M_{\rm op} = M_{\rm ip} = M_{\rm j,45,u} = \frac{M_{\rm 45}}{\sqrt{2}}$$
(5.96)

Resistance $M_{j,45,u}$ is obtained by solving a quadratic equation as follows:

$$M_{j,45,u}^{2}a_{\rm C} + M_{j,45,u}b_{\rm C} + c_{\rm C} = 0$$
(5.97)

with

$$\Delta_{\rm C} = b_{\rm C}^2 - 4a_{\rm C}c_{\rm C} \tag{5.98}$$

$$a_{\rm C} = \frac{1}{\left(z_{\rm C} N_{\rm moy}\right)^2} \tag{5.99}$$

$$b_{\rm C} = \frac{1}{M_{\rm op,u,max}} \tag{5.100}$$

$$c_{\rm C} = \left(\frac{N - N_{\rm Mmax}}{N_{\rm moy}}\right)^2 - 1 \tag{5.101}$$

The positive solution of this equation is:

$$M_{\rm j,45,u} = \frac{-b_{\rm C} + \sqrt{\Delta_{\rm C}}}{2a_{\rm C}} \tag{5.102}$$

An example of the application of this calculation procedure is given in the appendix at the end of this chapter (section B.5).

5.5.2 Comparison against numerical and experimental results

Comparison against the experimental tests of INSA Rennes

The analytical ultimate bending moment resistances are compared against the experimental test results carried out at INSA Rennes and presented in detail in sub-chapter 3.7. The ultimate bending moment resistances deduced from the experimental test results and those predicted by the analytical approach are summarized in Table 5.6.

Table 5.6: Comparison of the analytical results against the experimental tests for biaxial bending moment

	Experimental	Analytical	Ana/Exp
Specimen	$M_{j,u,exp}$ (kNm)	$M_{\rm j,u,ana}$ (kNm)	-
SPE1-M45	39,4	41,5	1,05
SPE2-M45	47,3	46,3	0,98

The results presented above show a fairly good agreement between the experimental test results and the proposed analytical models for the estimation of the resistance of column base plates. For specimens SPE1-M45, the proposed model overestimate the resistance by 5%.

• Comparison against the numerical results of the parametric study

The bending moment resistances estimated by the calculation procedure given in 5.5.1 are compared against the results obtained in the parametric study (see sub-chapter 4.4). In general, the values obtained by the analytical models underestimate the resistances obtained by the FE analysis. Looking carefully at the interaction curves at collapse depicted in Figure 5.20 to Figure

5.25, it can be seen that the analytical curves are generally circumscribed to the numerical curves, revealing a good accuracy of the obtained results with an acceptable safety margin, except for specimens P3 and P4. Similarly to the cases studied for in-plane and out-of-plane bending moment, for biaxial bending moment the model gives accurate results when subjected to a tensile force.





Figure 5.23: M-N interaction curves for P4



5.6 Concluding remarks

Two analytical models (simplified and full analytical models) were developed and presented for the estimation of the ultimate resistance of four bolts column base connections subjected to the combination of an axial tensile/compressive force and in-plane bending moment. The calculation procedures are treated separately for in-plane and out-of-plane bending moment. For out-of-plane bending moment a parabolic interaction curve is proposed. In presence of biaxial bending moment and axial force, a simplified approach was presented for the calculation of the ultimate bending moment resistance, based on a plastic redistribution of the applied forces between both left and right sides of the column base plate (see section 5.5.1).

The results obtained by these analytical models were compared against the experimental and numerical results, in order to demonstrate its applicability within the range of the studied base plate configurations and applied loading cases. The agreement between the ultimate resistances is overall satisfactory. The analytical models underestimate the resistances and therefore can be used with confidence (safe side). For some cases, an overestimation of the resistance was observed, although with an acceptable error. Due to its simplification, the analytical results for high levels of applied axial compressive force show less good agreement. The predictions for the resistance showed higher accuracy for the cases in which the collapse was governed by anchor bolts rupture in tension.

Regarding out-of-plane and biaxial bending moments, the proposed analytical model assume a plastic redistribution of the forces in the connection. This assumption excludes failures developed in the concrete components activated by the transfer of tensile forces and columns of class 4.

APPENDIX B

B.1 Joint configuration

The proposed analytical model is applied to a column base plate in order to present the main steps of the calculation. The studied column base plate is built considering the configuration and nominal material properties of SPE2-M0. This information is summarized in Figure B.1 and Table B.1. The design bending resistances of the connection are calculated for a compressive force of 800 kN and contain also:

- the in-plane bending resistance calculated according to the simplified and full analytical model in section B.2 and B.3 respectively,
- the out-of-plane bending resistance in section B.4,
- the biaxial bending resistance for 45° in section B.5.

Partial safety factors and coefficients are considered as: $k_2 = 0.9$; $\gamma_c = 1.5$; $\gamma_{M0} = 1$; $\gamma_{M2} = 1.25$.



Figure B.1: Joint configuration of the worked example

Luoie Dill Ocomen ; of the soluted estample
--

	Column steel	profile	Base plate		Anchor bolt	S	Concrete	
	$h_{\rm c} ({\rm mm})$	190	$h_{\mathrm{bp}} (\mathrm{mm})$	330	<i>d</i> (mm)	16	$h_{\rm f}$ (mm)	610
	$b_{\rm c}$ (mm)	200	$b_{ m bp}$ (mm)	300	$d_0 (\mathrm{mm})$	18	$b_{\rm f}$ (mm)	900
	$t_{\rm wc}$ (mm)	6,5	$t_{\rm bp}~({\rm mm})$	20	$A_{\rm s}({\rm mm^2})$	157	$d_{\rm f}$ (mm)	1450
	$t_{\rm fc} ({\rm mm})$	10	$e_{\rm x}({\rm mm})$	35	$L_{\rm b}({\rm mm})$	350		
Geometry	$A \text{ (mm}^2)$	5380	e_{x1} (mm)	35	$L_{\rm emb}({\rm mm})$	300		
	$I_{\rm y}~({\rm cm}^4)$	3692	<i>p</i> (mm)	260	$h_{\mathrm{bh}}(\mathrm{mm})$	10		
	$W_{\rm pl,y}({\rm cm}^3)$	429,5	<i>e</i> (mm)	70	$d_{\rm w}$ (mm)	50		
	$I_z (cm^4)$	1336	<i>w</i> (mm)	160				
	$W_{\rm pl,z}({\rm cm}^3)$	203,8						

-	Column steel	l profile	Base plate	Anchor bolts	Concrete
Motorial	E (GPa)	210	210	210	<i>E</i> _{cm} (GPa) 31
properties	$f_{\rm y}$ (MPa)	355	275	300	<i>f</i> _{cm} (MPa) 33
properties	<i>f</i> _u (MPa)	490	430	500	<i>f</i> _{ck} (MPa) 25

 Table B.2: Material properties of the worked example

B.2 In-plane bending moment resistance: simplified model

• Tensile resistances

The length m_x is:

$$m_{\rm x} = e_{\rm x1} - 0,8a\sqrt{2} = 27,1 \text{ mm}$$

The circular and non-circular effective lengths are:

$$l_{\rm eff,cp} = \min(2\pi m_{\rm x}; \pi m_{\rm x} + 2e; \pi m_{\rm x} + w) = 170 \text{ mm}$$

 $l_{\text{eff,nc}} = \min(4m_x + 1,25e_x; 2m_x + 0,625e_x + e; 2m_x + 0,625e_x + w/2; b_p/2) = 146 \text{ mm}$

The effective length of failure mode 1 is:

 $l_{\text{eff},1} = \min(l_{\text{eff},\text{cp}}; l_{\text{eff},\text{nc}}) = 146 \text{ mm}$

The anchor bolt elongation length and its limit are:

$$L_{\rm b} = t_{\rm bp} + t_{\rm wa} + 0.4d + 8d = 164.4 \text{ mm}$$
$$L_{\rm b}^* = \frac{8.8m_{\rm x}^3 A_{\rm s} n_{\rm b}}{\sum l_{\rm eff,1} t_{\rm bp}^3} = 23.5 \text{ mm}$$

And thus prying effects does not develop as:

 $L_{\rm b}^* < L_{\rm b}$

Failure modes 1-2 and 3 should be considered for the evaluation of the resistance of the T-stub in tension.

The plastic bending moment resistance of the base plate and the resistance of anchor bolts in tension are:

$$M_{\rm pl,1,Rd} = 0.25 l_{\rm eff,1} t_{\rm bp}^2 f_{\rm y,bp} / \gamma_{\rm M0} = 4015 \text{ kNmm}$$

 $F_{\rm t,Rd} = \frac{k_2 f_{\rm ub} A_{\rm s}}{\gamma_{\rm M2}} = 56.5 \text{ kN}$

The resistances of failure mode 1-2 and 3 are:

$$F_{\text{T},1-2,\text{Rd}} = \frac{2M_{\text{pl},1,\text{Rd}}}{m_{\text{x}}} = 297 \text{ kN}$$

 $F_{\text{T},3,\text{Rd}} = 2F_{\text{t},\text{Rd}} = 113 \text{ kN}$

Then, the design resistance of the T-stub in tension is given by:

$$F_{\text{T,Rd}} = \min(F_{\text{T,1-2,Rd}}; F_{\text{T,3,Rd}}) = 113,0 \text{ kN}$$

And the tensile resistance of the connection:

$$N_{\rm T,Rd} = 226,1 \, \rm kN$$

• Compressive resistances

From the section 2.4.2, the bearing strength of the concrete $f_{jd,max}$ is taken equal to:

$$f_{\rm jd,max} = \beta_{\rm j} \alpha_{\rm bf} f_{\rm cd} = \alpha_{\rm bf} \frac{f_{\rm ck}}{\gamma_{\rm c}} = 20,5 \text{ MPa}$$

with

$$\alpha_{\rm bf} = \min\left[1 + \frac{d_{\rm f}}{\max(h_{\rm bp}; b_{\rm bp})}; 1 + 2\frac{e_{\rm h}}{h_{\rm bp}}; 1 + 2\frac{e_{\rm b}}{b_{\rm bp}}; 3\right] = 1,85$$
$$\beta_{\rm j} = 2/3$$

The additional effective width c^* is:

$$c^* = t_{\rm bp} \sqrt{\frac{f_{\rm y,bp}}{2f_{\rm jd,max}}} + \sqrt{2}a_{\rm w} = 61,6 \text{ mm}$$

For column flange T-stubs, the effective width and length are:

$$b_{\rm eff,c} = t_{\rm fc} + 2c^* = 133,3 \,\mathrm{mm}$$

$$l_{\rm eff,c} = \min(b_{\rm c} + 2c^*; b_{\rm bp}) = 300 \text{ mm}$$

For the column web T-stub, the effective width and length are:

$$b_{\text{eff,w}} = t_{\text{wc}} + 2c^* = 129,8 \text{ mm}$$

 $l_{\text{eff,w}} = h_c - 2c^* - 2t_{\text{fc}} = 46,7 \text{ mm}$

The resistance in compression of the column is:

$$N_{\rm c,Rd} = \frac{Af_{\rm y,c}}{\gamma_{\rm M0}} = 1909,9 \text{ kN}$$

The effective bearing strength of concrete is:

$$f_{jd}^* = \min\left(\frac{N_{c,Rd}}{2b_{eff,c}l_{eff,c}}; f_{jd,max}\right) = 20,5 \text{ MPa}$$

The resistance of a column flange T-stub in compression is:

$$F_{\rm C,Rd} = f_{\rm jd}^* b_{\rm eff,c} l_{\rm eff,c} = 821 \text{ kN}$$

Laura da Silva Seco

The resistance in compression of the connection is thus:

$$N_{\rm C,Rd} = 2F_{\rm C,Rd} = 1643 \text{ kN}$$

• In-plane bending resistance

The axial force applied to the connection is equal to 800 kN and it is necessary to classify the behavior of the connection based on the three limits N_1 , N_2 and N_3 :

$$N_1 = -F_{T,Rd} = -113 \text{ kN}$$

 $N_2 = F_{C,Rd} - F_{T,Rd} = 708 \text{ kN}$
 $N_3 = F_{C,Rd} = 821 \text{ kN}$

Behaviour type 3 should be considered (dominant bending moment with critical compressive force):

$$N_2 < N = 800 \text{ kN} \le N_3$$

The lever arms are:

$$z_{\rm T} = h_{\rm c}/2 + e_{1,\rm x} = 130 \text{ mm}$$

$$z_{\rm C0} = h_{\rm c}/2 - t_{\rm fc}/2 = 90 \text{ mm}$$

The in-plane bending moment resistance $M_{j,ip,Rd}$ is obtained as follows:

$$M_{j,ip,Rd} = (z_T + z_{C0})F_{C,Rd} - z_T N = 76,7 \text{ kNm}$$

B.3 In-plane bending moment resistance: full model

• Resistances in compression

For the application of the full analytical model, it is necessary to calculate new values of the bearing strength of concrete.

The resistance of the column flange and web in compression is:

$$F_{\rm c,fc,Rd} = \frac{W_{\rm pl,y} f_{\rm y,c}}{h_{\rm c} - t_{\rm fc}} = 847.1 \text{ kN}$$

The effective bearing strength of concrete on the flange T-stub is:

$$f_{jd,f}^* = \min\left(\frac{F_{c,fc,Rd}}{b_{eff,c}l_{eff,c}}; f_{jd,max}\right) = 20,5 \text{ MPa}$$

The resistance of a column flange T-stub in compression is:

$$F_{\mathrm{C,f,Rd}} = f_{\mathrm{jd,f}}^{*} b_{\mathrm{eff,c}} \ l_{\mathrm{eff,c}} = 821 \text{ kN}$$

The effective bearing strength of concrete on the web T-stub is:

$$f_{jd,w} = \min\left(\frac{N_{C,Rd} - 2F_{c,fc,Rd}}{b_{eff,w}l_{eff,w}}; f_{jd,max}\right) = 20,5 \text{ MPa}$$

The resistance of a column web T-stub in compression is:

$$F_{C,w,Rd} = 125 \text{ kN}$$

• In-plane bending resistance

The axial force applied to the connection is equal to 800 kN and it is necessary to classify the behavior of the connection based on the four limits N_1 , N_2 , N_3 and N_3^* :

• •

$$N_{1} = F_{T,Rd} = -113 \text{ kN}$$

$$N_{2} = F_{C,f,Rd} - F_{T,Rd} = 708 \text{ kN}$$

$$N_{3} = F_{C,f,Rd} + F_{C,w,Rd}/2 - F_{T,Rd} = 771 \text{ kN}$$

$$N_{3}^{*} = F_{C,f,Rd} + F_{C,w,Rd} = 946 \text{ kN}$$

Behaviour type 4 should be considered (dominant bending moment with critical compressive force):

$$N_3 < N = 800 \text{ kN} \le N_3^*$$

The length of the web under compression is:

$$x_{\rm w} = \frac{N + 2F_{\rm T,Rd} - F_{\rm C,f,Rd}}{b_{\rm eff,w} f_{\rm jd,w} + \frac{F_{\rm T,Rd}}{z_{\rm w}}} = 27,3 \text{ mm}$$

with

$$z_{\rm w} = z_{\rm C0} - c^* - \frac{t_{\rm fc}}{2} = 23,3 \text{ mm}$$

Then, the in-plane bending resistance is:

$$M_{j,ip,Rd} = F_{T,Rd} \frac{2z_w - x_w}{z_w} z_T + z_{C0} F_{C,f,Rd} + x_w b_{eff,w} f_{jd,w} \left(z_w - \frac{x_w}{2} \right) = 86,8 \text{ kNm}$$

B.4 Out-of-plane bending moment resistance

When the maximum out-of-plane bending moment develops, the lever arms are:

$$z_{\mathrm{T,op}} = \frac{w}{2} = 80 \mathrm{mm}$$

$$z_{\rm C,op} = \min\left(\frac{b_{\rm c}}{4} + \frac{c^*}{2}; \frac{b_{\rm bp}}{4}\right) = 75 \text{ mm}$$

The compressive force that develop is equal to:

$$F_{c,f,Rd} = \min (b_{eff,c} l_{eff,c} f_{jd,max}; b_c t_{fc} f_{y,c}) = 710 \text{ kN}$$

The maximum out-of-plane bending resistance is:

$$M_{\rm op,Rd,max} = z_{\rm T,op} F_{\rm T,Rd} + z_{\rm C,op} F_{\rm c,f,Rd} = 62,3 \text{ kNm}$$

The necessary forces for the evaluation of the final bending resistance are taken as:

$$N_{\rm Rd,Mmax} = \frac{N_{\rm C,Rd} - N_{\rm T,Rd}}{2} = 708 \text{ kN}$$

 $N_{\rm Rd,moy} = \frac{N_{\rm C,Rd} + N_{\rm T,Rd}}{2} = 934 \text{ kN}$

The out-of-plane bending resistance calculated for an applied axial force of 800 kN comes:

$$M_{\rm j,op,Rd} = M_{\rm op,Rd,max} \left[1 - \left(\frac{N - N_{\rm Rd,Mmax}}{N_{\rm Rd,moy}} \right)^2 \right] = 61,7 \text{ kNm}$$

B.5 Biaxial bending moment resistance

Assuming an elliptical interaction, the bending resistance for an inclination of 45° is:

$$M_{j,45,Rd} = \frac{\sqrt{2}}{\sqrt{\frac{1}{(M_{j,ip,Rd})^2} + \frac{1}{(M_{j,op,Rd})^2}}} = 68 \text{ kNm}$$

The limits N_1 and N_2 for the application of the analytical model are:

$$N_1 = F_{T,Rd} = -113 \text{ kN}$$

 $N_2 = F_{C,Rd} = 821 \text{ kN}$

According to the values of the limits N_1 and N_2 , behaviour type 2 must be considered (dominant bending moment). For the calculation of the ultimate bending moment $M_{j,45,u}$, parameters a_M , b_M , c_M and Δ_M are:

$$a_{\rm M} = \frac{1}{\left(\bar{z}N_{\rm Rd,moy}\right)^2} = 9 \times 10^{-5} \ (\rm kNm)^{-2}$$
$$b_{\rm M} = \frac{1}{M_{\rm op,Rd,max}} + \frac{N(\alpha_{\rm C} - \alpha_{\rm T})}{\bar{z}N_{\rm Rd,moy}^2} = 0,019 \ (\rm kNm)^{-1}$$
$$c_{\rm M} = \frac{1}{2} \left[\frac{\left(\alpha_{\rm C}N - N_{\rm Rd,Mmax}\right)^2 + \left(\alpha_{\rm T}N - N_{\rm Rd,Mmax}\right)^2}{N_{\rm Rd,moy}^2} \right] - 1 = -0,97$$
$$\Delta_{\rm M} = b_{\rm M}^2 - 4a_{\rm M}c_{\rm M} = 7,3 \times 10^{-4} \ (\rm kNm)^{-2}$$

with

$$\bar{z} = \frac{z_{\rm C} + z_{\rm T}}{2} = 110 \text{ mm}$$
$$\alpha_{\rm C} = \frac{z_{\rm T}}{\bar{z}} = 1,18$$

$$\alpha_{\rm T} = \frac{z_{\rm C}}{\bar{z}} = 0,82$$

Then, the bending resistance with an inclination of 45° and a compressive force of 800 kN is:

$$M_{\rm j,45,u} = \sqrt{2} \ \frac{-b_{\rm M} + \sqrt{\Delta_{\rm M}}}{2a_{\rm M}} = 59.3 \ \rm kNm$$

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Objectives

The main goal of this thesis was to develop a model based on the Component Method for the evaluation of the resistance of column base plates and beam-to-concrete slab connections. The scope of this research work was restricted to rigid or semi-rigid column base plates with four anchor bolts subjected to a combination of axial tensile/compressive load and bending moment in one or both axis directions. This study aimed to analyse several configurations and its behaviour under the abovementioned loading conditions.

This objective was achieved by conducting four main research lines:

- 1. Literature review allowed to analyze and document the existing methodologies and identify the aspects to be improved in order to develop a simple but innovative model;
- 2. Experimental program a total of six experimental tests on column base plates and three on beam-to-concrete slab connections subjected to pure bending (in-plane, out-of-plane and biaxial bending moment) were carried out and constitute a reliable database for the validation of the proposed analytical models since there are few records of tests performed in bending other than in-plane. These tests provided an insight view into the actual behavior of these types of connection in the elastic and plastic domain. The main parameters that influenced directly the joints response (base plate thickness, diameter and steel grade of the anchor bolts) were identified due to the literature review and analysed. The impact of the arrangement of the anchoring system as well as the base plate configuration, the column cross-section and the presence of stiffeners welded to the column flanges were also addressed;
- 3. Numerical study the experimental program was supplemented by several FE analyses conducted on different configurations subjected to different loading conditions (M+N). These models combined geometrical, material and contact nonlinearities allowing to understand quantitatively and qualitatively the global behavior of the connections. An extensive parametric study was carried out to evaluate the influence of the geometry and material properties on the ultimate response of column base plates;
- 4. Analytical study the analytical approach was divided into in-plane and out-of-plane bending moment. The model for in-plane bending moment resistance was divided into two models simplified and full to take into account or not the resistance of the column web in compression. The calculation procedure to estimate the ultimate bending moment resistance was compared against the experimental tests and validated for the range of configurations studied on the parametric study.

6.2 Conclusions

6.2.1 Experimental program

The experimental tests carried out at *Laboratoire de Génie Civil et Génie Mécanique de l'INSA de Rennes* and included in Chapter 3 of this thesis allowed to have a better understanding of the behavior of column base plates with different configurations subjected to bending moment in different directions. From the observed results, some patterns could be drawn in terms of resistance, initial stiffness, rotation capacity and failure modes.

One main goal of this experimental program was to study the influence of the base plate thickness. For that, two different thicknesses were purposely tested, 10 and 20 mm, in order to evaluate the differences on the overall behavior with a "flexible" and "rigid" base plate. It was concluded that the bending moment resistance and the initial stiffness of the connections increased for the thicker base plate. On the other hand, the increase of the base plate thickness decrease the rotation capacity. Also, the distribution of the internal forces on the base plates were strongly affected by this parameter. For thinner base plates, a great concentration was located at the footprint of the column flange in compression and at the level of the activated anchor bolt row in tension. The same could not be observed for the cases with thicker base plates, in which the stress distribution was more uniform on the compressive area. Finally, it was also found that the base plate thickness was responsible for the different redistributions of the internal forces.

The other objective was to evaluate the overall behavior of the column base plates subjected to different bending moment orientations and characterize in terms of resistance, failure modes and yielded elements. This variation of the direction of application of the bending moment meant a decrease of the resistance for out-of-plane bending moment cases due to decrease of the distance between the centers of compression and tension. It was also a preponderant factor for the development of prying and the location of the contact areas. However, its contribution had no impact on the obtained failure modes.

6.2.2 Numerical study

Numerical models were created with the Finite Element software ABAQUS for further investigation of the behavior of the tested column base plates and beam-to-concrete slab connections subjected to different loading conditions. The components of the connection (base/end-plate, column/beam, weld, concrete) were generated considering three-dimensional deformable solids with eight-node linear brick elements, except for the concrete reinforcement modelled as two-node linear beam element. The measured dimensions of the specimens tested have been considered in the finite element models. The main goal of these models was to develop an accurate numerical simulation able to capture the response of the different parts of the connection, as well as the different stress states, taking into account the complex behavior characterized by multiple phenomena such as geometrical, material and contact non-linearities.

Due to the observations made during the experimental tests, a special attention was given to the interaction between the concrete and the anchor bolts since the way the anchor bolts interact with the foundation strongly influenced the overall stiffness. Also, in order to obtain numerical results with higher accuracy, the material non-linearity was carefully taken into account. This way, the post-limit behavior of the elements could be accurately modelled. This was particularly relevant during the necking of the anchor bolts.

Next, the numerical results were compared and calibrated against the experimental tests presented in Chapter 3. From this confrontation, it was concluded that the agreement between the results obtained numerically and from the experimental tests in terms of resistance, initial stiffness, rotation capacity and strain distribution was quite satisfactory. Besides the fact that the yielded elements and the obtained failure modes were the same, the numerical analyses allowed to confirm the assumptions of yielding of the column and the welds in specimens SPE1-M0, SPE1-M90 and SPE1-M45 from the experimental tests. As seen from Chapter 3, although the anchor bolts played an important role on the post-limit response of the connections, the evolution of the stress distribution on these elements could not be evaluated. The numerical models completed the experimental tests and permitted to assess the contribution of the anchor bolts to resist to the applied loads for all specimens. It was observed that for the cases of out-of-plane and biaxial bending moments, a redistribution of the internal forces took place between the anchor bolts. It was also interesting to analyse the influence of the base plate thickness and the bending moment orientation on the variation of the location of the centre of compression.

As the numerical analyses based on the experimental tests provided accurate information about the behaviour of the connections subjected to different loading conditions and allowed to identify the most influential parameters on this type of connections, an extensive parametric study was carried out and helped to understand the roll of each parameter and its influence on the global behavior of the structure in terms of resistance and failure modes. It was concluded that the loading conditions (level of the applied tensile/compressive axial load; orientation of the applied bending moment) strongly affect the obtained failure modes as well as the bending moment resistance and the initial stiffness. As the level of tensile force increased, the resistance and the initial stiffness of the studied configurations tended to decrease. However, for compressive loads, this was only valid for higher levels of applied force. It was also possible to observe the development of a local buckling at the level of the column flanges for significant axial compressive load levels, appearing sooner or later depending on the chosen column steel profile and the orientation of the applied bending moment. Additionally, the increase of the anchor bolts diameter resulted in an increase of the bending resistance and the initial stiffness of the connections, for the cases the failure mode was due to the rupture of anchor bolts in tension.

6.2.3 Analytical study

From the literature review and the observations of the experimental tests and FE analyses, three main aspects were identified as issues to be improved on the existing calculation procedures for

the resistance of column base plates. The first has to do with the fact that the location of the centre of compressions varies along the loading in the elastic and plastic regions. The second aspect is the consideration of the contribution of the resistance of the concrete and the column in compression to the overall resistance. Furthermore, the main objective was to develop a calculation procedure not only for in-plane bending moment but also for cases subjected to out-of-plane bending moment with an axial force, easy to apply and based on the Component Method.

Considering loading cases in which the axial tensile/compressive force acts simultaneously with bending moment, three distinct design models were developed for in-plane, out-of-plane and biaxial bending moment. For the first case, two analytical models were created in order to calculate the ultimate resistance of the connection, considering or not the contribution of the column web in compression (simplified and full models). For the out-of-plane bending moment case, the proposed model was developed using a parabolic interaction curve. In presence of biaxial bending moment and axial force, a simplified approach was presented for the calculation of the ultimate bending moment resistance. It consisted on the division of the applied system of internal forces on both left and right sides of the connection, assuming a plastic redistribution of the out-of-plane bending moment.

Finally, the results obtained by these analytical models were compared against the experimental tests and the results from the parametric study, in order to demonstrate its applicability within the range of the studied base plate configurations and applied loading cases. The agreement between the ultimate resistances was quite satisfactory, placing generally the analytical results on the safe side.

6.3 Forthcoming works

During the accomplishment of this work, some relevant aspects were identified and deserve further consideration. The issues exposed below are suggested as future works in the scope of this thesis in terms of experimental program, numerical study and analytical models:

- from the experimental tests, it was proved that the anchor bolts were the governing element in failure and strongly influenced the overall behavior of the connections in terms of resistance, initial rotational stiffness, rotation capacity and phenomena like the development of prying effect, also due to the thickness of the base plate. For this reason, it is thought to be interesting to carry out tests in the same conditions but with different diameters of the anchor bolts in order to be able to evaluate the differences according to the aspects abovementioned,
- extend the experimental study to include an important compressive force combined with bending moment and thus observe local buckling of the column,
- extend the parametric study for the cases of biaxial bending moment to inclination angles of 30° and 60° with respect to the column major axis (and not only at 45° as
presented), allowing to evaluate the impact in terms of resistance, failure modes, yielded elements, yielding patterns and compare the results against the proposed procedure for these cases,

- the analytical models proposed in Chapter 5 were developed for rigid column base plates with four outer anchor bolts. However, due to the diversity of configurations available for these types of connections, it would also be important to proceed to some modifications, namely at the level of the effective areas, to include the contribution of additional inner anchor bolts rows,
- further research to propose analytical models that permit to calculate the initial rotational stiffness of column base plates in presence of out-of-plane and biaxial bending moment,
- the tests were performed on ductile column base plates, however in presence of concrete cone failure or local elastic buckling of the column flange, the plastic redistribution probably does not occur. The analytical model developed for plastic redistribution should be modified.

BIBLIOGRAPHY

Abdollahzadeh, G. R. and Ghobadi, F. (2013) 'Mathematical modeling of column-base connections under monotonic loading', *Civil Engineering Infrastructures journal*.

Alfarah, B., López-Almansa, F. and Oller, S. (no date) New methodology for calculating damage variables evolution in plastic damage model for RC structures.

Amaral, P. M. (2014) Steel column bases under biaxial loading conditions. University of Porto.

American Institute of Steel Construction (1990) Steel design guide series: Column base plates.

Bajer, M. *et al.* (2014) 'Influence of selected parameters on design optimisation of anchor joints', in *12th International Conference on Steel, Space and Composite Structures*. Prague.

Baniotopolous, C. and Sokol, Z. (no date) Column base connections.

^cCECM: recommended testing procedures for assessing the behavior of structural elements under cyclic loads' (1986) *European Convention for Constructional Steelwork, Technical Commitee 1, TWG 13 - Seismic Design*, No 45.

Comite Euro-international du Béton (1995) Design of fastenings in concrete - design guide.

Couchaux, M. (2010) Comportement des assemblages par brides circulaires boulonnées. INSA de Rennes.

Couchaux, M. (2015) Projet J3D Pieds de poteaux encastrés, Report No. DRV/14-RCM-105/003-A.

Couchaux, M. et al. (2017) 'Tensile resistances of L-stubs', Journal of Constructional Steel Research, 138, pp. 131–149.

Cowie, K., Hyland, C. and Mago, N. (no date) Column base plate design - lapping strip method.

CTICM (2015a) Revue construction métallique n°3/2015 Assemblages de contreventements par goussets boulonnés centrés sous chargements monotones et cycliques.

CTICM (2015b) Revue construction métallique n°4/2015 Rupteur thermique par platine d'about pour les structures métalliques rapportées.

CTICM (no date) Guide Eurocode: Assemblages des pieds de poteaux en acier.

Delhomme, F. *et al.* (2015) 'Static and cyclic pullout behavior of cast-in-place headed and bonded anchors with large embedment depths in cracked concrete', *Nuclear Engineering and Design*, 287, pp. 139–150.

Delhomme, F. and Debicki, G. (2010) 'Numerical modelling of anchor bolts under pullout and

relaxation tests', Construction and Building Materials, 24, pp. 1232–1238.

Ermopolous, J. C. and Stamatopoulos, G. N. (1996) 'Mathematical modelling of column base plate connections', *Journal of Constructional Steel Research*, 2, pp. 79–100.

European Committee for Standardisation (2001) CEN, EN 10002-1:2001, Metallic materials -Tensile testing - Part 1: Method of test at ambient temperature. Brussels.

European Committee for Standardisation (2002) CEN, Eurocode, ENV 1990:2002, Eurocode: Basis of structural design.

European Committee for Standardisation (2004) CEN, Eurocode 2, ENV 1992-1-1:2004, Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings. Brussels.

European Committee for Standardisation (2005) CEN, Eurocode 3, ENV 1993-1-8:2005, Eurocode 3: Design of steel structures - Part 1-8: Design of joints. Brussels.

Fasaee, M. A. K., Banan, M. R. and Ghazizadeh, S. (2018) 'Capacity of exposed column base connections subjected to uniaxial and biaxial bending moments', *Journal of Constructional Steel Research*, 148, pp. 361–370.

Gomez, I. R., Kanvinde, A. and Deierlein, G. G. (2010) *Exposed column base connections* subjected to axial compression and flexure, Final Report AISC. Chicago.

Guisse, S., Vandegans, D. and Jaspart, J. P. (1996) *Application of the component method to column bases - experimentation and development of a mechanical model for characterization, Report No. MT195.*

Heisinuo, M., Laine, V. and Lehtimaki, E. (2009) 'Enlargement of the component method into 3D', in *Nordic Steel Construction Conference 2009*, pp. 430–437.

Heisinuo, M., Romni, H. and Perttola, H. (2011) 'Experimental study of end plate joints in biaxial bending', in *EUROSTEEL*, pp. 327–332.

Hong, S. and Park, S. (2012) 'Uniaxial bond stress-slip relationship of reinforcing bars in concrete', *Advances in Materials Science and Engineering*.

International Federation for Structural Concrete (fib) (2010) FIP Model Code 2010. Lausanne.

Jalalifar, H., Aziz, N. and Hadi, M. (2004) 'Modelling of sheared behaviour bolts across joints', in *Coal Operator's Conference, University of Wollongong & the Australasian Institute of Mining and Metallurgy*, pp. 225–232.

Jaspart, J. P. (1990) Étude de la semi-rigidité des noeuds poutre-colonne et son influence sur la résistance et la stabilité des ossatures en acier. University of Liège.

Jaspart, J. P. (1997) Recent advances in the field of steel joints column bases and further configurations for beam-to-column joints and beam splices. University of Liège.

Laura da Silva Seco

Jaspart, J. P. and Vandegans, D. (1998) 'Application of the component mehtod to column bases', *Journal of Constructional Steel Research*, 48, pp. 89–106.

Kavoura, F. and Genvturk, B. (2018) 'Evaluation of existing provisions for design of pinned column base-plate connections', *Journal of Constructional Steel Research*, 148, pp. 233–250.

Khodaie, S., Mohamadi-shooreh, M. R. and Mofid, M. (2012) 'Parametric analyses on the initial stiffness of the SHS column base plate connections using FEM', *Engineering Structures*, 34, pp. 363–370.

Kmiecik, P. and Kaminski, M. (2011) 'Modelling of reinforced concrete structures and composite structures with concrete strength degradation taken into consideration', *Archives of Civil and Mechanical Engineering*, XI, pp. 623–636.

Krishnamurthy, N. and Thambiratnam, D. P. (1990) 'Finite element analysis of column base plates', *Computers & Structures*, 34, pp. 215–223.

Kuhlmann, U. et al. (2014) Design of steel-to-concrete joints design manual II. Prague, Stuttgart, Coimbra, Brussels: Research fund for Coal & Steel.

Latour, M. and Rizzano, G. (2013) 'A theoretical model for predicting the rotational capacity of steel', *Journal of Constructional Steel Research*, 91, pp. 89–99.

Lee, D. Y., Goel, S. and Stojadinovic, B. (2008a) 'Exposed column-base plate connections bending about weak axis: I. Numerical parametric study', *Steel Structures*, 8, pp. 11–27.

Lee, D. Y., Goel, S. and Stojadinovic, B. (2008b) 'Exposed column-base plate connections bending about weak axis: II. Experimental study', *Steel Structures*, 8, pp. 29–41.

Lemonis, M. and Gantes, C. (2008) 'Evaluation of the complete moment-rotation curve', in *EUROSTEEL 2008, Graz, Austria*, pp. 579–584.

Lescouarc'h, Y. (1988) Les pieds de poteaux encastrés en acier. France: CTICM.

Lim, W., Lee, D. Y. and You, Y. (2017a) 'Cyclic loading tests on exposed column-base plate weak-axis connections of small-size steel structures', *Engineering Structures*, 153, pp. 653–664.

Lim, W., Lee, D. Y. and You, Y. (2017b) 'Exposed column-base plate strong-axis connections for small-size steel construction', *Journal of Constructional Steel Research*, 137, pp. 286–296.

Márai, P. et al. (2014) 'Resistance model for fixed column bases', in EUROSTEEL. Naples.

Michal, S. and Andrzej, W. (2015) 'Calibration of the CDP model parameters in Abaqus', *Advances in Structural Engineering and Mechanics*.

Neumann, N. and Nuhic, F. (2011) 'Design of structural joints connecting H or I sections', in *EUROSTEEL 2011, August 31 - September 2, Budapest, Hungary*, pp. 303–308.

Nguyen, H. T. and Kim, S. E. (2009) 'Finite element modeling of push-out tests for large stud

shear connectors', Journal of Constructional Steel Research, 65, pp. 1909–1920.

Packer, J., Bruno, L. and Birkemoe, P. (1989) 'Limit analysis of bolted RHS flange plate joints', *Journal of Structural Engineering*, 115, pp. 2226–2242.

Pavlovic, M. et al. (2013) 'Bolted shear connectors vs. headed studs behaviour in push-out tests', Journal of Constructional Steel Research, 88, pp. 134–149.

Pereira, H., Cunha, V. and Sena-Cruz, J. (2015) 'Numerical simulation of galvanized rebars pullout', *Frattura ed Integrità Strutturale*, 31, pp. 54–66.

Pertold, J., Xiao, R. Y. and Wald, F. (2000) 'Embedded steel column bases I. Experiments and numerical simulation', *Journal of Constructional Steel Research*, 56, pp. 253–270.

Perttola, H. and Heisinuo, M. (no date) *Experimental study on flanged joints of tubular members under biaxial bending*.

Picard, A. and Beaulieu, D. (1985) 'Behavior of a simple column base connection', *Canadian Journal of Civil Engineering*, 12, pp. 126–136.

Ruopp, J. and Kuhlmann, U. (2017) 'Steel-to-concrete joints with large anchor plates under shear loading', *Steel Construction*, 10, pp. 115–124.

Shaheeen, M. A., Tsavdaridis, K. D. and Salem, E. (2017) 'Effect of grout properties on shear strength of column base connections: FEA and analytical approach', *Engineering Structures*.

Simões da Silva, L. (2008) 'Towards a consistent design approach for steel joints under generalized loading', *Journal of Constructional Steel Research*, 64, pp. 1059–1075.

Simões da Silva, L. and Santiago, A. (2003) Manual de ligações metálicas.

Simões, R. (2014) Manual de dimensionamento de estruturas metálicas Eurocódigo 3: projecto de estruturas de aço Parte 1-1: regras gerais e regras para edifícios. CMM Editions.

Stamatopoulos, G. N. and Ermopolous, J. C. (1997) 'Interaction curves for column base plate connections', *Journal of Constructional Steel Research*, 44, pp. 69–89.

Stamatopoulos, G. N. and Ermopolous, J. C. (2011) 'Experimental and analytical investigation of steel column bases', *Journal of Constructional Steel Research*, 67, pp. 1341–1357.

Steenhuis, C. M. (1998) Assembly procedure for base plates. Delft.

Sumer, Y. and Aktas, M. (2015) 'Defining parameters for concrete damage plasticity model', *Challenge Journal of Structural Mechanics 1*, 3, pp. 149–155.

Tsavdaridis, K. D. *et al.* (2016) 'Analytical approach of anchor rod stiffness and steel base plate calculation under tension', *Structures*, 5, pp. 207–2018.

Wald, F. (1995) Patky Sloupù - Column bases. CVUT Praha.

Wald, F. (2000) Resistance and stiffness model of column bases by component method. Prague.

Wald, F., Sokol, Z. and Steenhuis, M. (2000) Proposal of the stiffness design of column bases, COST Project C1.

Yayla, P., Kaluc, E. and Ural, K. (2007) 'Effects of welding processes on the mechanical properties of HY 80 steel weldments', *Materials and Design*, 28, pp. 1898–1906.

INSA de RENNES Service des Formations

AVIS DU JURY SUR LA REPRODUCTION DE LA THESE SOUTENUE

Titre de la thèse:

Pieds de poteaux par platine d'embase sous sollicitations tridimensionelles

Nom Prénom de l'auteur : DA SILVA SECO LAURA

Membres du jury :

- Monsieur BOUCHAIR Abdelhamid

- Madame COELHO Ana

- Monsieur WALD Frantisek

- Monsieur COSTA NEVES Luis - Monsieur HJIAJ Mohammed
- Monsieur COUCHAUX Maël

Président du jury : FRANTISEL WALD

Date de la soutenance : 29 Novembre 2019

Reproduction de la these soutenue

X

Thèse pouvant être reproduite en l'état Thèse pouvant être reproduite après corrections suggérées

Fait à Rennes, le 29 Novembre 2019

Signature du président de jury

frahisch hall

Le Directeur,

M'hamed DRISS

UNIVERSITE BRETAGNE SCIENCES LOIRE POUR L'INGENIEUR



Titre : Pieds de poteaux par platine d'assise sous sollicitations tridimensionnelles

Mots clés : Pied de poteau, platine, flexion biaxiale, ancrage, interaction M-N

Résumé : Les pieds de poteaux ont une influence prépondérante sur la stabilité et la rigidité des charpentes métalliques. Cette thèse porte sur la résistance des pieds de poteaux par platine soumis à un effort normal et un moment bi-axial. Les pieds de poteaux étudiés sont composés d'une platine soudée à l'extrémité du poteau et reliée au bloc béton par 4 tiges. Dans le chapitre II, une étude bibliographique dresse un bilan des modèles permettant de calculer la résistance de ce type d'assemblage ainsi que des résultats d'essais. Le chapitre III est ensuite dédié à la description et l'analyse des résultats d'une campagne d'essais menée sur 9 assemblages soumis à un moment dans le plan, hors plan ou de la flexion bi-axiale. Une attention particulière est accordée à l'influence de l'épaisseur de la platine et de l'orientation du moment.

Les résultats sont présentés et discutés afin de mieux comprendre le comportement élastique et plastique. Cette étude est complétée par des analyses par éléments finis dans le chapitre IV validées par confrontation aux résultats d'essais. Une étude paramétrique a ensuite permis d'étendre le domaine d'investigation en ajoutant un effort normal et d'autres configurations géométriques (sections en HEA, IPE, diamètres des tiges d'ancrage). Un modèle analytique, basé sur la méthode des composants de l'Eurocode 3 partie 1-8, est ensuite proposé afin de calculer la résistance des pieds de poteau en flexion uniaxiale (dans le plan et hors plan) et biaxiale. Le caractère conservatif de la méthode est démontré par comparaison aux résultats des essais et des études numériques.

Title : Column base plates under 3D loading

Keywords : Column base plate, biaxial bending, anchoring, M-N interaction

Abstract : Column bases have a major influence on the stability and stiffness of steel structures. This thesis focuses on the estimation of the resistance of column base plates subjected to a combination of axial force and biaxial bending moment. The investigated connections consist of a steel column welded to a base plate and fixed to the concrete block by means of four anchor bolts. In Chapter II, an extensive literature review is presented, summarizing the existing models for the estimation of the resistance of column base plates as well as the results of previous experimental test campaigns. Chapter III is dedicated to the analysis of the results of a test campaign carried out on 9 column base plates, subjected to in-plane, out-of-plane and bi-axial bending moments. Particular attention is given to the influence of the base plate thickness and orientation of the applied bending moment.

Results are presented and discussed, to better understand the elastic and post-limit behaviors. These experimental results are completed with refined numerical simulations presented in Chapter IV. A parametric study is conducted to broaden this investigation, by adding a normal force and other geometrical configurations (column steel profile HEA, IPE, diameter of the anchor bolts). Next, an analytical model is proposed, based on the Component Method in Eurocode 3 Part 1-8, to calculate the resistance of column base plates under uniaxial and biaxial bending moments. The conservative nature of the model is demonstrated by predictions model against comparing the experimental and numerical results.