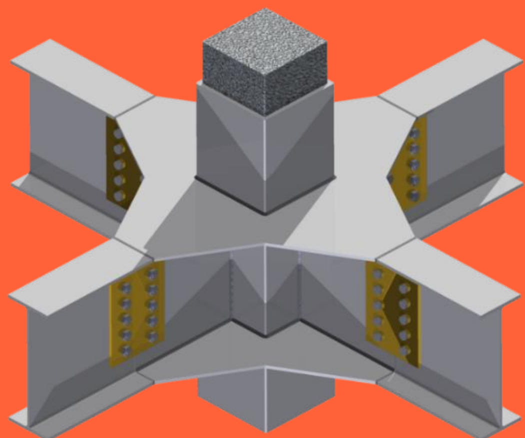
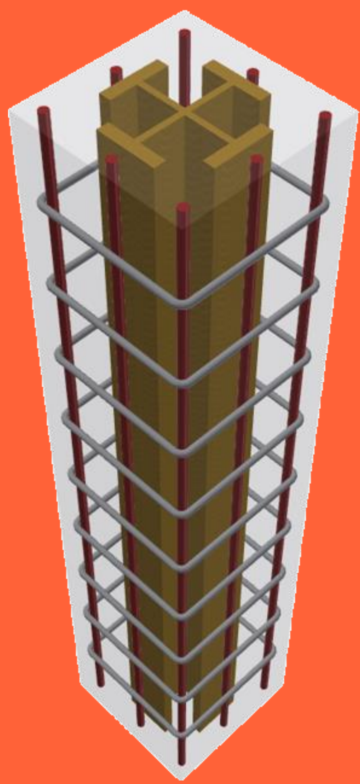
BC4:2021

Design Guide for Steel-Concrete Composite Columns with High Strength Materials –

- An Extension of Eurocode 4 Method to C90/105 Concrete and S550 Steel





Building and Construction Authority

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Design Guide for Steel-Concrete Composite Columns with High Strength Materials - An Extension of Eurocode 4 Method to C90/105 Concrete and S550 Steel

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Foreword

The previous version of BC4 (2015) only provides design guidance for concrete filled tubular members. This new design guide has been revised to provide additional guidance for concrete encased steel composite members. It provides a unified method for high strength steel-concrete composite members. The design guide is written with practicing structural engineers in mind. It emphasises professional applications, placing great emphasis on ready-to-use materials. It contains formulas and tables that give immediate solutions to common questions and problems arising from practical work related to composite columns.

Steel-concrete composite members, with the concrete either encasing the steel section or being confined by the steel tube, have been used in various applications, especially for columns in high rise buildings. The other applications include structural uses in civil infrastructural work, industrial construction, offshore oil and gas installations and protective structures. Modern design codes on steel-concrete composite members such as American, Chinese, European, and Japanese codes, do not provide guidance on the use of high strength construction materials, such as the high strength concrete and high tensile steel section. This design guide is an extension of EN1994-1-1 (Eurocode 4) for the design of steel-concrete composite members with special considerations for high strength concrete with cylinder compressive strength up to 90 N/mm^2 and high strength steel section with yield strength up to 550 N/mm^2. More than 2500 test data collected from the available literature on steel-concrete composite members with normal and high strength materials have been analysed to formulate the design methods proposed in this guide. Additional tests on high strength composite members were conducted in the Structural Laboratory at the National University of Singapore based on the research fund supported by the Ministry of National Development and the Building and Construction Authority, Singapore.

The design guide also provides good detailing practices for typical joints between steel-concrete composite columns and other structural components. Guideline is provided to select matching concrete and steel grades for the design of high strength composite columns. Special considerations for fire resistance design, fabrication of high tensile steel sections, and preparation of high strength concrete are emphasized. This design guide will endow structural engineers with the confidence to use high strength materials in a safe and economic manner to design and construct high rise buildings.

A comprehensive guidebook "Design of Steel-Concrete Composite Structures – An extension of EN1994-1-1 to High Strength Materials" will be published by Elsevier in 2021 (Liew et al., 2021). Readers may want to refer to this book for deeper understanding of the design concepts and approaches, including advanced calculation methods, test database, spreadsheets, and detailed work examples.

Acknowledgement

The Building and Construction Authority of Singapore (BCA) would like to thank the authors for developing this Guidebook as well as the local and international expert committee members for their valuable contributions.

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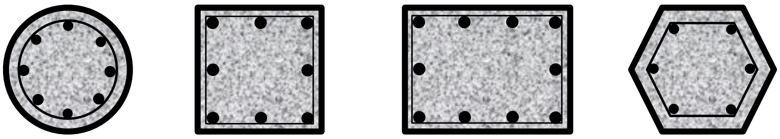
1 General

(1) This design guide is applicable for the design of steel-concrete composite members with concrete cylinder strength up to 90 N/mm^2 and steel of yield strength up to 550 N/mm^2.

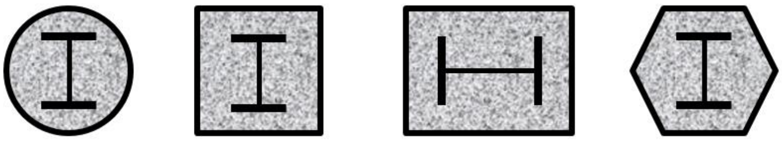
(2) This design guide extends the limitation of EN 1994-1-1 and EN 1994-1-2 for the design steel-concrete composite members made of high strength concrete and the high strength steel section.

(3) This design guide should be limited to the types of steel-concrete composite members of doubly symmetrical and uniform cross-section over the member length, as shown in Figure 1.1.

(a) Concrete filled steel tubes without reinforcement



(b) Concrete filled steel tubes with reinforcement



(c) Concrete filled steel tubes with an encased steel section

(d) Concrete encased steel sections

Figure 1.1 Types of double symmetric concrete filled steel tubular members and concrete encased steel members

(4) This design guide does not apply to laced or battened concrete filled steel tubular members which consist of two or more discontinuously connected sections. In which case the specialist advices should be consulted.

(5) The partial safety factors for concrete, steel, reinforcement, and shear connector as given in Table 1.1 should be conformed to.

Table 1.1 Partial safety factors of materials

Concrete	Steel	Reinforcement	Shear Connector
$\boldsymbol{\gamma_c = 1.5}$	$\gamma_a = 1.0$	$\gamma_s = 1.15$	$\gamma_v = 1.25$

(6) The ratio of the depth to the width of the composite cross-section should be within the limits of 0.2 and 5.0.

(7) The longitudinal reinforcements that may be used in calculation should not exceed 6% of the concrete area. The longitudinal reinforcements in concrete filled tube may not be necessary if they are not required for fire resistance.

(8) The steel contribution ratio is defined as

$$\delta = \frac{A_a f_{yd} + A_e f_{ed}}{N_{pl,Rd}} \tag{1.1}$$

where

A_a, A_e	are	the cross-sectional area of steel tube and encased steel section, respectively
f_{yd}, f_{ed}	are	the design strength of steel tube and encased steel section, respectively
$N_{pl,Rd}$	is	the axial resistance of the composite section

The range of the steel contribution should be within the following range:

$$0.2 \le \delta \le 0.9 \tag{1.2}$$

(9) The general method given in EN 1994-1-1 is applicable to steel-concrete composite members with high strength materials. The general method may be implemented by means of advanced finite element analysis.

2 Materials

2.1 Concrete

(1) The high strength concrete is defined as concrete with cylinder compressive strength greater than 50 N/mm^2.

(2) The concrete strength classes given in Table 2.1 and Table 2.2 should be conformed to for design of composite members.

Table 2.1 Strength classes of normal strength concrete

Strength class	C12/15	C16/20	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
Characteristic Cylinder strength (f_{ck}, N/mm^2)	12	16	20	25	30	35	40	45	50
Modulus of elasticity (E_{cm}, GPa)	27	29	30	31	33	34	35	36	37

Table 2.2 Strength classes of high strength concrete

Strength class	C55/67	C60/75	C70/85	C80/95	C90/105
Characteristic Cylinder strength (f_{ck}, N/mm^2)	55	60	70	80	90
Reduced Cylinder strength (ηf_{ck}, N/mm^2)	53.6	57	63	68	72
Modulus of elasticity (E_{cm}, GPa)	38.2	39.1	40.7	42.2	43.6
Reduced modulus of elasticity (E'_{cm}, GPa)	38.0	38.6	39.6	40.4	41.1

(3) For high strength concrete with f_{ck} > 50 N/mm^2 as shown in Table 2.2, the characteristic cylinder strength should be multiplied by a reduction factor defined as:

$$\eta = \begin{cases} 1.0 - (f_{ck} - 50)/200 & 50 \text{ N/mm}^2 < f_{ck} \leq 90 \text{ N/mm}^2 \\ 0.8 & f_{ck} > 90 \text{ N/mm}^2 \end{cases} \tag{2.1}$$

(4) For high strength concrete with f_{ck} > 50 N/mm^2, the reduced modulus of elasticity should be determined as follow:

$$E'_{cm} = 22[(\eta \cdot f_{ck} + 8)/10]^{0.3} \tag{2.2}$$

2.2 Steel Section

(1) Steel sections with yield strength higher than 460N/mm^2 is defined as high tensile steel.

(2) The technical delivery conditions for flat products of high strength quenched and tempered steels should conform to EN 10025-6.

(3) The technical delivery conditions of high tensile steel plates manufactured from thermo-mechanically controlled process should comply with EN 10149-2.

(4) The strength classes of steel given in Table 2.3 should be conformed to for the design of composite members.

Table 2.3 Strength classes of mild steel and high tensile steel

Grade	Nominal values of yield strength f_y (N/mm^2) with thickness (mm) less than or equal to					
	16	40	63	80	100	150
S235	235	225	215	215	215	195
S275	275	265	255	245	235	225
S355	355	345	335	325	315	295
S420	420	400	390	370	360	340
S460	460	440	430	410	400	380
S500	500	500	480	480	480	440
S550	550	550	530	530	530	490

(5) The limiting values of ratio f_u/f_y, elongation at failure, and the ultimate strain ε_u are recommended in Table 2.4.

Table 2.4 Limitations on ductility, elongation at failure and ultimate strain of steel

Steel	Ratio f_u/f_y	Elongation at failure	Ultimate strain ε_u
≤ 460 N/mm^2	≥ 1.10	15%	$\geq 15\varepsilon_y$
> 460 N/mm^2	≥ 1.05	10%	$\geq 15\varepsilon_y$

(6) The modulus of elasticity of structural steel should be taken as 210,000 N/mm^2.

(7) The maximum permissible plate thickness for mild steel and high tensile steel should be determined in accordance with EN 1993-1-10 and EN 1993-1-12, respectively.

(8) For the structural steel materials to be used in Singapore, they should be in compliance with BC1:2012.

2.3 Reinforcing Steel

(1) The yield strength of reinforcing steel should be limited to the range of 400 N/mm^2 to 600 N/mm^2 as conforming to EN 1992-1-1.

(2) The strength classes of reinforcing steel provided in Table 2.5 can be used in steel-concrete composite members.

Table 2.5 Strength classes of reinforcing steel

Class	Characteristic yield strength (f_{yk}, N/mm^2)	Ultimate/yield strength ratio	Ultimate elongation
B500A	500	1.05	2.5%
B500B	500	1.08	5.0%
B500C	500	$\geq 1.15, < 1.35$	7.5%

(3) The elastic modulus of reinforcing steel should be taken as 210,000 N/mm^2.

(4) The Grade 460 reinforcing steel is allowed in accordance with BS 4449.

2.4 Shear Connector

(1) The mechanical characteristics and nominal dimensions of shear studs should conform to BS EN ISO 13918 and BS EN ISO 898-1.

(2) Weldability and welding examination of shear studs should be checked in accordance with BS EN ISO 14555.

(3) High strength concrete provides higher confinement for shear studs locally. The design shear resistance of a headed stud, irrespective of concrete strength, may be calculated according to Eq.(2.3):

$$P_{Rd} = \min\left(\frac{0.8 f_{us} \pi d_s^2}{4\gamma_v}, \frac{0.29\alpha_s d_s^2 \sqrt{f_{ck} E_{cm}}}{\gamma_v}\right) \tag{2.3}$$

where

d_s	is	the diameter of the shank of the shear stud, $16\ \mathrm{mm} \leq d_s \leq 25\ \mathrm{mm}$
f_{us}	is	the ultimate strength of the shear stud, $\leq 500\ \mathrm{N/mm^2}$
h_{sc}	Is	the overall height of the shear stud
γ_v	is	the partial factor which is 1.25 for the ultimate limit state
α_s	=	$0.2\left(\frac{h_{sc}}{d_s}+1\right)$ for $3 \leq h_{sc}/d_s \leq 4$
		1.0 for $h_{sc}/d_s > 4$

(4) For applications in Singapore, shear studs not covered in this design guide shall be allowed provided they are in compliance with the provisions in BC1:2012.

(5) Shear connectors other than the stud type, such as weld beads, welded reinforcements, welded shear keys, etc., are allowed provided they can perform in accordance with the product manufacturer's recommendations or when specialist's advice is consulted.

2.5 Material Compatibility

(6) For structural steel and concrete used in the steel-concrete composite members, the material compatibility recommended in Table 2.6 should be conformed to.

(7) Alternatively, the maximum effective yield strength of the steel section should be determined in accordance with the concrete characteristic cylinder strength with class up to C90/105 using the following expression:

$$f_y \leq 0.7E_a\left(f_{ck}+8\right)^{0.31} \tag{2.4}$$

where

f_y	is	the characteristic yield strength of steel
E_a	is	the modulus of Elasticity of steel
f_{ck}	is	the characteristic cylinder strength of concrete

Table 2.6 Compatibility between steel and concrete

	S235	S275	S355	S420	S460	S500	S550
C12/15	√	√	√	×	×	×	×
C16/20	√	√	√	×	×	×	×
C20/25	√	√	√	×	×	×	×
C25/30	√	√	√	√	×	×	×
C30/37	√	√	√	√	×	×	×
C35/45	√	√	√	√	√	×	×
C40/50	√	√	√	√	√	×	×
C45/55	√	√	√	√	√	√	×
C50/60	√	√	√	√	√	√	×
C55/67	√	√	√	√	√	√	×
C60/75	√	√	√	√	√	√	×
C70/85	√	√	√	√	√	√	√
C80/95	√	√	√	√	√	√	√
C90/105	√	√	√	√	√	√	√

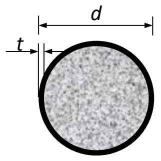
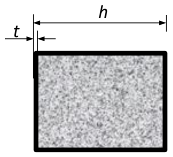
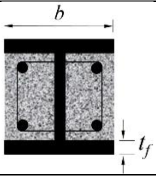
Notes: "√" indicates compatible materials and "×" is not recommended.

3 Design of Steel-Concrete Composite Members

3.1 Local Buckling

(1) The effects of local buckling may be neglected for the steel section fully encased by the concrete. For the outer steel tube and partially encased steel section, the maximum value of Table 3.1 should not be exceeded.

Table 3.1 Maximum values (*d*/*t*), (h/t) for local buckling

Concrete filled Cross-section	Maximum value in expressions	Maximum values according to steel grades				
		S275	S355	S460	S500	S550
	$\left(d/t\right)_{\max} = 90\dfrac{235}{f_y}$	77	60	46	42	38
	$\left(h/t\right)_{\max} = 52\sqrt{\dfrac{235}{f_y}}$	48	42	37	35	34
	$\left(b/t_f\right)_{\max} = 44\sqrt{\dfrac{235}{f_y}}$	41	36	31	30	29

(2) The local buckling of the outer steel tube and partially encased steel section may be prevented by increasing the plate thickness. Alternatively, stiffener plate may be welded on the steel plate along the longitudinal direction of the column to reduce the effective width of the steel plate.

3.2 Resistance of Cross Sections

3.2.1 Resistance to Compression

(1) The plastic design resistance to compression of a concrete filled tubular section with an encased steel section inside (refer to Figure 1.1(c)) or concrete encased steel section (refer to Figure 1.1(d)) should be calculated by adding the plastic resistances of its components:

For CFST section: $$N_{pl,Rd} = A_a f_{yd} + A_c f_{cd} + A_s f_{sd} + A_e f_{ed} \quad (3.1\text{ (a)})$$

For CES section: $$N_{pl,Rd} = 0.85 A_c f_{cd} + A_s f_{sd} + A_e f_{ed} \quad (3.1\text{ (b)})$$

where

A_a, A_c, A_s, A_e	are	the cross-sectional area of steel tube, concrete, reinforcements and encased steel section, respectively
$f_{yd}, f_{cd}, f_{sd}, f_{ed}$	are	the design strength of steel tube, concrete, reinforcements and encased steel section, respectively

(2) For circular concrete filled steel tubular cross-section, the increase of concrete strength due to confinement effect may be allowed provided that the relative slenderness $\bar{\lambda}$ of the member does not exceed 0.5 and $e/D < 0.1$, where e is the eccentricity of loading defined by M_{Ed}/N_{Ed} and D is the external dimension of the section. M_{Ed} and N_{Ed} are the design effects due to moment and axial compression, respectively. The plastic resistance to axial compression considering the confinement effect may be calculated as:

$$N_{pl,Rd} = \eta_a A_a f_{yd} + A_e f_{ed} + A_c f_{cd}\left(1 + \eta_c \frac{t}{d}\frac{f_y}{f_{ck}}\right) + A_s f_{sd} \quad (3.2)$$

where

t	is	the thickness of steel tube
η_a	=	$\eta_{a0} + (1-\eta_{a0})(10\,e/D)$ for $e/D \le 0.1$ 1.0 for $e/D > 0.1$
η_{a0}	=	$0.25(3+2\bar{\lambda})$ but ≤ 1.0
η_c	=	$\eta_{c0}(1-10\,e/d)$ for $e/D \le 0.1$ 1.0 for $e/D > 0.1$
η_{c0}	=	$4.9 - 18.5\bar{\lambda} + 17\bar{\lambda}^2$ but ≥ 0

(3) For high strength concrete (f_{ck} > 50 N/mm^2), the compressive strength f_{ck} and f_{cd} should be reduced by the factor of η given in Eq.(2.1).

3.2.2 Resistance to Shear Forces

(1) The shear forces $V_{a,Ed}, V_{e,Ed}, V_{c,Ed}$ acting on the outer steel tube, encased steel section and concrete section, respectively, should be calculated as:

$$V_{a,Ed} = V_{Ed}\frac{M_{pl,a,Rd}}{M_{pl,Rd}} \quad (3.3)$$

$$V_{e,Ed} = V_{Ed} \frac{M_{pl,e,Rd}}{M_{pl,Rd}} \tag{3.4}$$

$$V_{c,Ed} = V_{Ed} - V_{a,Ed} - V_{e,Ed} \tag{3.5}$$

where

$M_{pl,a,Rd}$	is	the plastic moment resistance of the steel tube
$M_{pl,e,Rd}$	is	the plastic moment resistance of the encased steel section inside the tube
$M_{pl,Rd}$	is	the plastic moment resistance of the composite section
V_{Ed}	is	the design shear force

(2) For simplification, the design shear force V_{Ed} may be assumed to be resisted by the steel tube and inner steel section only. Thus, the shear forces $V_{a,Ed}, V_{e,Ed}$ acting on the steel tube and the inner steel section, respectively, may be calculated as:

$$V_{a,Ed} = V_{Ed} \frac{M_{pl,a,Rd}}{M_{pl,a,Rd} + M_{pl,e,Rd}} \tag{3.6}$$

$$V_{e,Ed} = V_{Ed} \frac{M_{pl,e,Rd}}{M_{pl,a,Rd} + M_{pl,e,Rd}} \tag{3.7}$$

(3) In case where the shear force $V_{a,Ed}$ or $V_{e,Ed}$ exceeds 50% of the design shear resistance $V_{pl,a,Rd}$ or $V_{pl,e,Rd}$ of the steel sections, the influence of transverse shear forces on the moment resistance should be considered. The consideration may be taken into account by a reduced design steel strength in their shear areas, or a reduced web thickness of the shear area. The reduction factor should be determined according to Eq.(3.8) and Eq.(3.9).

For the outer steel tube

$$\phi = 1 - \left(\frac{2V_{a,Ed}}{V_{pl,a,Ed}} - 1 \right)^2 \tag{3.8}$$

For the encased steel section

$$\phi = 1 - \left(\frac{2V_{e,Ed}}{V_{pl,e,Ed}} - 1 \right)^2 \tag{3.9}$$

The design shear resistance $V_{pl,a,Rd}$ and $V_{pl,e,Rd}$ can be calculated in accordance with EN 1993-1-1 for the steel tube and encased steel section, respectively.

3.2.3 Resistance to Combined Compression and Bending

(1) The resistance of a cross-section to combined compression and moments may be calculated based on interaction curve assuming rectangular stress blocks as shown in Figure 3.1, taking account of the design shear force in accordance with Section 3.2.2. The tensile strength of the concrete may be neglected.

(2) As a simplification, the interaction curve may be assumed as a polygonal diagram as shown in Figure 3.2. The plastic stress distributions of a composite section for the points A, B, C and D are shown in Figure 3.2.

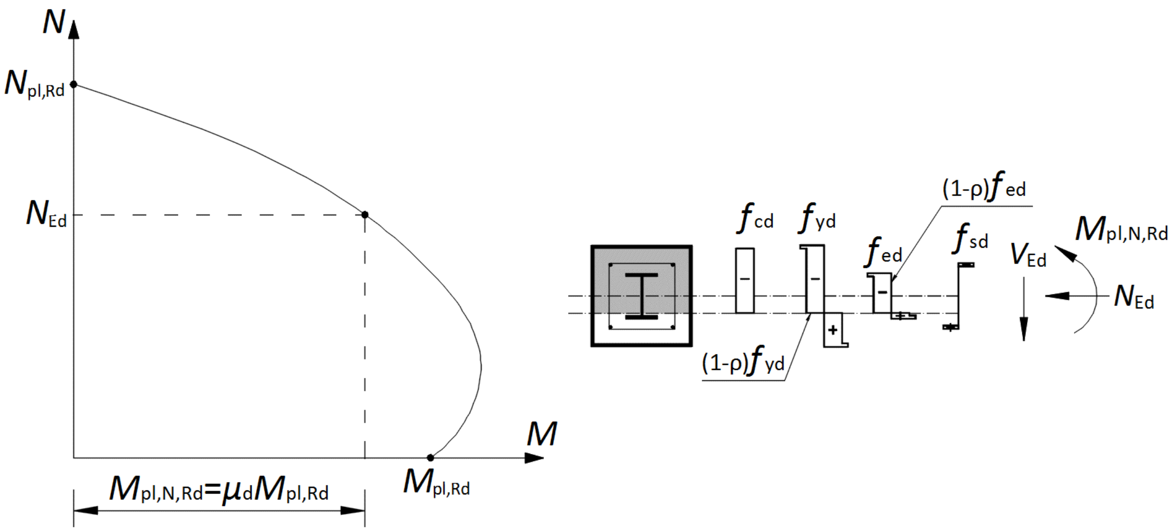


Figure 3.1 Interaction curve for combined compression and bending

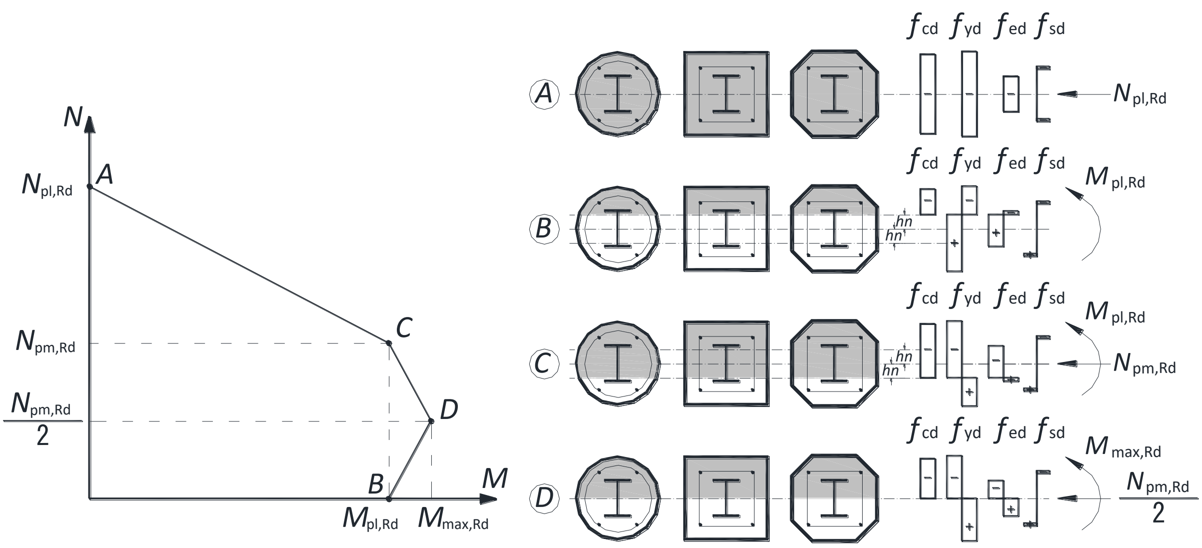


Figure 3.2 Simplified interaction curve and corresponding stress distributions

3.3 Resistance of Members

3.3.1 Resistance to Compression

(1) For simplification for members in axial compression, the design value of normal force N_{Ed} should satisfy:

$$\frac{N_{Ed}}{\chi N_{pl,Rd}} \leq 1 \tag{3.10}$$

(2) The value of χ for the appropriate non-dimensional slenderness $\bar{\lambda}$ should be determined from the relevant buckling curve according to:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad \text{but} \quad \chi \leq 1 \tag{3.11}$$

where

Φ	=	$0.5\left[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2\right]$
α	=	the imperfection factor as given in Table 3.2, the relevant buckling curves and member imperfections are given in Table 3.3
$\bar{\lambda}$	=	the relative slenderness for the plane of bending and equal to $\sqrt{\frac{N_{pl,Rk}}{N_{cr}}}$
$N_{pl,Rk}$	=	the characteristic value of the plastic resistance to compression $N_{pl,Rd}$ in which the material characteristic strengths rather than the design strength should be used.
N_{cr}	=	the elastic critical normal force for the relevant buckling mode determined by Eq. 3.12.

(3) The elastic critical normal force N_{cr} for the relevant buckling mode is determined by:

$$N_{cr} = \frac{\pi^2 (EI)_{eff}}{L_{eff}^2} \tag{3.12}$$

Where

L_{eff} is the buckling length of a composite column for the relevant buckling mode. In the absence of Eurocode guidance, method to calculate the effective buckling lengths given in BS 5950: Part 1 may be adopted.

(4) The effective flexural stiffness of a composite section $(EI)_{\text{eff}}$ should be determined from:

$$(EI)_{eff} = E_a I_a + E_s I_s + E_e I_e + 0.6 E_{cm} I_c \quad (3.13)$$

where

I_a, I_c, I_s, I_e	are	the second moments of area of the steel tube, the un-cracked concrete, the reinforcements and the encased steel section for the bending plane being considered
E_a, E_{cm}, E_s, E_e	are	the modulus of elasticity of the steel tube, the un-cracked concrete, the reinforcements and the encased steel section

(5) The influence of long-term effects on the effective flexural stiffness $(EI)_{\mathrm{eff}}$ should be accounted for. The modulus of elasticity of concrete E_{cm} should be reduced to the value $E_{\mathrm{c,eff}}$ in accordance with the following equation:

$$E_{c,eff} = E_{cm} \frac{1}{1 + \left(N_{G,Ed} / N_{Ed} \right) \varphi_t} \quad (3.14)$$

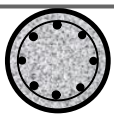
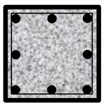
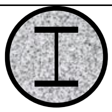
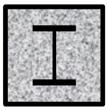
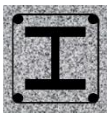
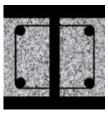
where

$N_{G,Ed}$	is	the part of the normal force that is permanent
φ_t	is	the creep coefficient

Table 3.2 Imperfection factors for buckling curves

Buckling curve	a	b	c
Imperfection factor	0.21	0.34	0.49

Table 3.3 Buckling curves and member imperfections for composite cross-sections

Cross section	Limits	Axis of buckling	Buckling curve	Member imperfection
	$\rho_s \leq 3\%$	any	a	$L/300$
	$\rho_s > 3\%$	any	b	$L/200$
	-	any	b	$L/200$
	-	any	b	$L/200$
	-	major	b	$L/200$
		minor	c	$L/150$
	-	major	b	$L/200$
		minor	c	$L/150$
Note: ρ_s is the area ratio of reinforcements relative to the concrete area.				

(6) For simplification, the creep coefficient φ_t may be conservatively taken as that for normal strength concrete when high strength concrete is used. The creep coefficient φ_t should be determined in accordance with EN 1992-1-1.

3.3.2 Resistance to Combined Compression and Uniaxial Bending

(1) The following expression based on the interaction curve determined according to Section 3.2.3 should be satisfied:

$$\frac{M_{Ed}}{M_{pl,N,Rd}} = \frac{M_{Ed}}{\mu_d M_{pl,Rd}} \leq \alpha_M \qquad (3.15)$$

where

M_{Ed}	is	the greatest of the end moments and the maximum bending moment within the column length, including imperfections and second order effects

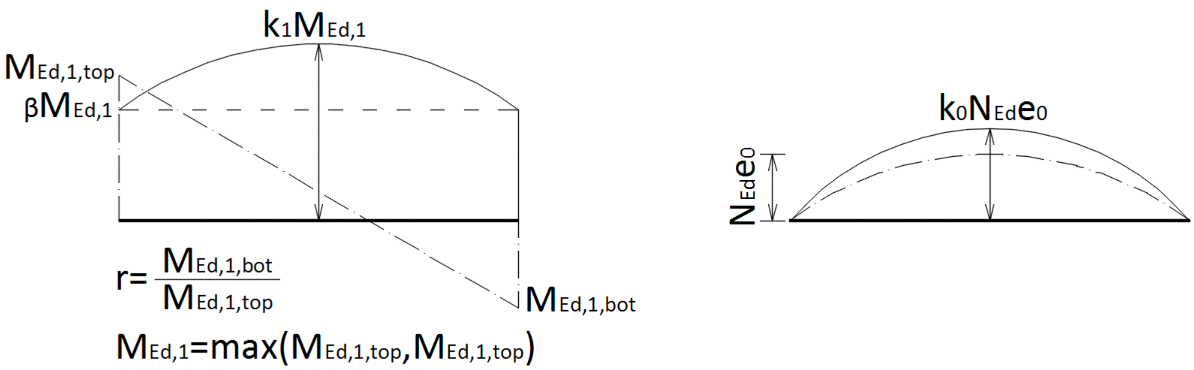
$M_{pl,N,Rd}$	is	the plastic bending resistance taking into account the normal force N_{Ed}, given by $\mu_d M_{pl,Rd}$
$M_{pl,Rd}$	is	the plastic bending resistance given by Point B in Section 3.2.3
α_M	=	0.9 for steel grades S235, S275, S355
		0.8 for other steel grades up to S550

(2) Within the column length, second-order effects may be allowed for by multiplying the greatest first-order design bending moment by a factor k given in Eq.(3.16). The second-order effect should be considered for both moments from initial member imperfection and first-order analysis as given in Figure 3.3 and calculated from Eq.(3.17).

$$k = \frac{\beta}{1 - N_{Ed} / N_{cr,eff}} \tag{3.16}$$

where

β	is	an equivalent moment factor given in Table 3.4
$N_{cr,eff}$	is	the critical normal force for the relevant axis and corresponding to the effective flexural stiffness given in Eq.(3.18), with the effective length taken as the column length



(a) Moment from first-order analysis (b) Moment from imperfection

Figure 3.3 Amplifications for moments from first-order analysis and member imperfection

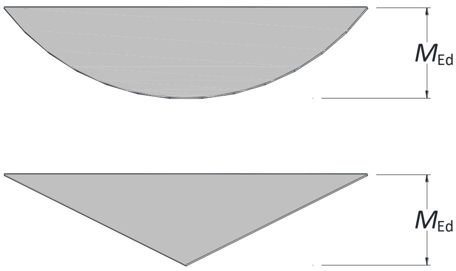
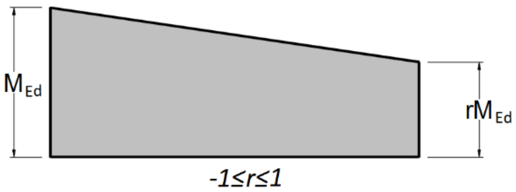
$$M_{Ed} = k_0 N_{Ed} e_0 + k_1 M_{Ed,1} \geq M_{Ed,1} \tag{3.17}$$

where

$M_{Ed,1}$	is	the maximum first-order design moment in column length
e_0	is	the member imperfection, given in Table 3.3

$$(EI)_{eff,II} = 0.9\left(E_a I_a + E_s I_s + E_e I_e + 0.5 E_{c,\mathrm{eff}} I_c\right) \tag{3.18}$$

Table 3.4 Equivalent moment factor $\boldsymbol{\beta}$

Moment distribution	Moment factors $\boldsymbol{\beta}$	Comment
	First-order bending moment from member imperfection or lateral load: $\beta = 1.0$	M_{Ed} is the maximum bending moment within the column length ignoring second-order effects
	End moments: $\beta = max(0.44, 0.66 + 0.44r)$	M_{Ed} and rM_{Ed} are the end moments from first-order or second-order global effects

3.3.3 Resistance to Combined Compression and Biaxial Bending

(1) For combined compression and biaxial bending, the following conditions should be satisfied for the stability check within the column length and for the check at the column ends:

$$\frac{M_{y,Ed}}{\mu_{dy} M_{pl,y,Rd}} \le \alpha_{M,y} \tag{3.19}$$

$$\frac{M_{z,Ed}}{\mu_{dz} M_{pl,z,Rd}} \le \alpha_{M,z} \tag{3.20}$$

$$\frac{M_{y,Ed}}{\mu_{dy} M_{pl,y,Rd}} + \frac{M_{z,Ed}}{\mu_{dz} M_{pl,z,Rd}} \le 1 \tag{3.21}$$

$M_{y,Ed}, M_{z,Ed}$	are	the design bending moments around $y-y$ or $z-z$ axis including second-order effects and imperfects
$M_{pl,y,Rd}, M_{pl,z,Rd}$	are	the plastic bending resistances around $y-y$ or $z-z$ axis
$\alpha_{M,y}, \alpha_{M,z}$	=	0.9 for steel grades S235, S275, S355 0.8 for other steel grades up to S550

(2) The value $\mu_d = \mu_{dy}\ or\ \mu_{dz}$ as shown in Figure 3.4 refers to the design plastic resistance moment $M_{pl,Rd}$ for the plane of bending being considered. Values μ_d greater than 1.0 should only be used where the bending moment M_{Ed} depends directly on the compression force N_{Ed}, for example where the moment M_{Ed} results from an eccentricity of the normal force N_{Ed}.

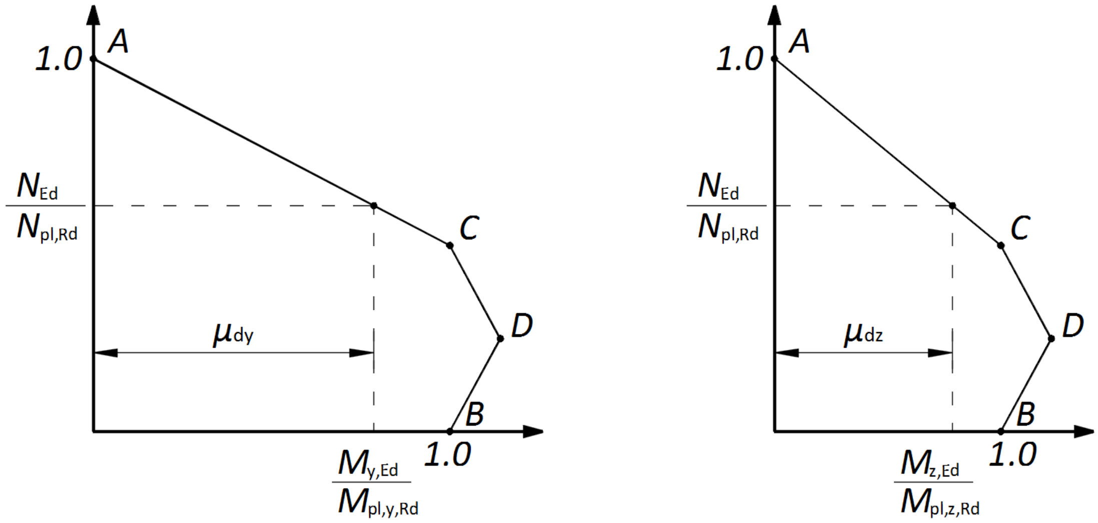


Figure 3.4 Interaction curves for design of combined compression and biaxial bending

(3) For composite columns and compression members with biaxial bending the values μ_{dy} and μ_{dz} as shown in Figure 3.4 may be calculated separately for each axis. Imperfections should be considered only in the plane in which failure is expected to occur. If it is not evident which plane is the more critical, checks should be made for both planes. Irrespective of axis, the value μ_d can be interpolated according to Figure 3.4 and Eqs. (3.22-3.24).

$$N_{Ed} \leq N_{pm,Rd}/2: \qquad \mu_d = 1 + \frac{2N_{Ed}}{N_{pm,Rd}}\left(\frac{M_{\max,Rd}}{M_{pl,Rd}} - 1\right) \tag{3.22}$$

$$N_{pm,Rd}/2 < N_{Ed} \leq N_{pm,Rd}: \qquad \mu_d = 1 + \frac{2\left(N_{pm,Rd} - N_{Ed}\right)}{N_{pm,Rd}}\left(\frac{M_{\max,Rd}}{M_{pl,Rd}} - 1\right) \tag{3.23}$$

$N_{Ed} > N_{pm,Rd}$: $$\mu_d = \frac{N_{pl,Rd} - N_{Ed}}{N_{pl,Rd} - N_{pm,Rd}} \quad (3.24)$$

3.4 Longitudinal Shear

(1) The longitudinal shear at the interface between concrete and steel should be checked if it is caused by transverse loads and/or end moments. Shear connectors should be provided where the design shear strength τ_{Rd} is exceeded.

(2) For simplicity, the design shear strength at the interface between high strength concrete and steel may be conservatively taken as between normal strength concrete and steel.

(3) Provided that the surface of the steel section in contact with the concrete is unpainted and free from oil, grease and loose scale or rust, the design shear strength at interface of steel and concrete can be taken as the values in Table 3.5.

Table 3.5 Design shear strength at the interface between concrete and steel

Type of cross section	τ_{Rd} (N/mm^2)
Fully concrete encased steel sections	0.30
Circular hollow sections	0.55
Rectangular hollow sections	0.40
Flanges of partially encased section	0.20
Webs of partially encased section	0.00

(4) The values of design shear strength given in Table 3.5 is for a minimum concrete cover of 40 mm. For greater concrete cover, higher values of τ_{Rd} may be used with an amplification factor β_c given by:

$$\beta_c = \min\left[1 + 0.02c_z\left(1 - \frac{40}{c_z}\right), 2.5\right] \quad (3.25)$$

where

c_z is the nominal value of concrete cover in mm

3.5 Load Introduction

(1) Shear studs should be provided in the load introduction area if the design shear strength is exceeded at the interface between concrete and steel section. In absence of a more

accurate method, the introduction length should not exceed $2d$ or $L/3$ as shown in Figure 3.5, where d is the minimum transverse dimension of the composite column and L is the system length of the column.

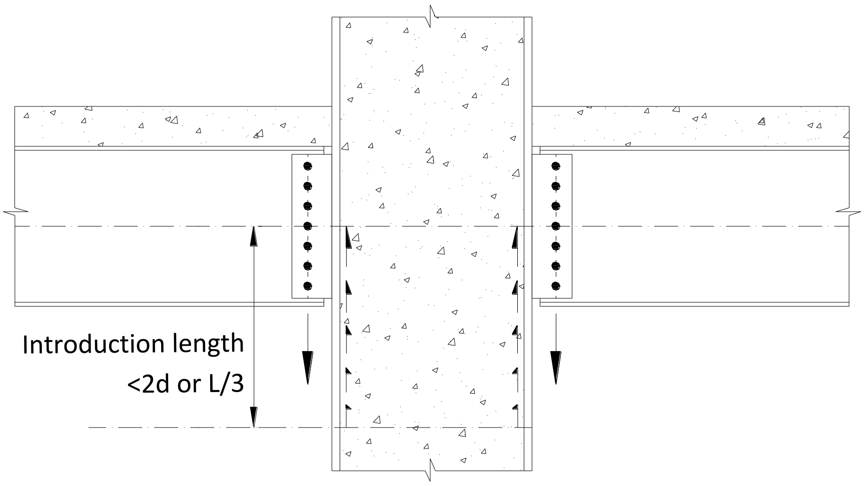


Figure 3.5 Load introduction area

(2) The shear studs may be replaced by welded reinforcements or weld beads as shown in Figure 3.6. The design shear resistance is calculated as:

$$P_{Rw} = \beta_w N_w A_w f_{cN} \tag{3.26}$$

where

β_w	=	$1.54 - 0.0143\, D/t_c$, for $D/t_c > 55$, $\beta_w = 0.7535$
D	is	the column diameter
t_c	is	the thickness of column tube
N_w	is	the number of welded reinforcements or weld beads, ≤ 3
A_w	is	the projected cross-sectional area of welded reinforcement or weld bead
f_{cN}	is	the bearing strength of concrete, $= min(A_c/A_w\, , 5) f_{cd}$
f_{cd}	is	the design strength of concrete
A_c	is	the cross-sectional area of concrete

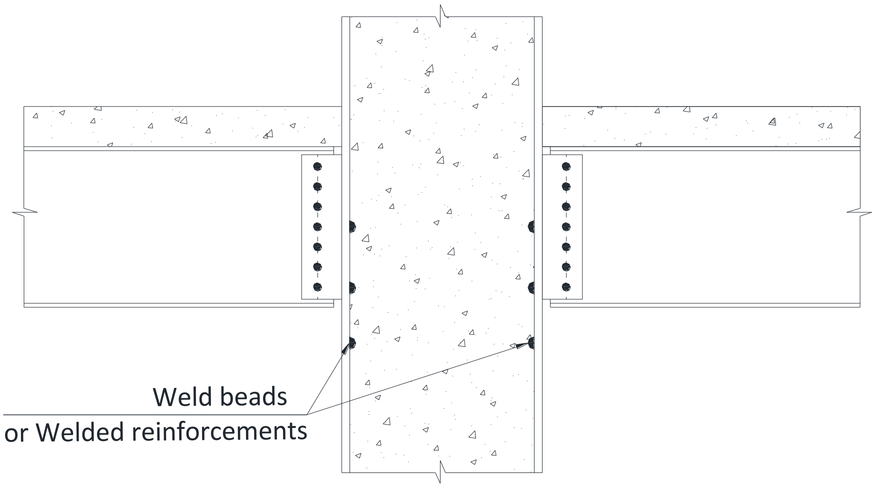


Figure 3.6 Load introduction to steel tubes by weld beads or welded reinforcements

4 Connection Detailing

4.1 General

(1) The joints may be classified into nominally pinned, semi-rigid and rigid. The classification should be in accordance with EN 1993-1-8. Joint behaviour should be taken into account in structural analysis based on each type of joint.

(2) The rules for semi-rigid joints are not applicable for steels with grades greater than S460. The joint of steel sections with high strength grade should be either rigid or pinned, and the resistance of the joint should be determined based on elastic distribution of forces over the components of the joint.

(3) The partial safety factors γ_M for joint design should comply with EN 1993-1-8 as given in Table 4.1, regardless of the steel grade.

Table 4.1 Partial safety factors for joint design

Applications	γ_{M}	Values
Resistance of cross-sections	γ_{M0}	1.0
Resistance of members to instability assessed by member checks	γ_{M1}	1.0
Resistance of cross-sections in tension to fracture	γ_{M2}	1.25
Resistance of bolts Resistance of welds Resistance of plates in bearing	γ_{M2}	1.25
Slip resistance - at ultimate limit state - at serviceability limit state	 γ_{M3} $\gamma_{M3,ser}$	 1.25 1.1
Preload of high strength bolts	γ_{M7}	1.1

4.2 Column Splices

(1) Bolted splices are not recommended unless the leaking of concrete during casting can be effectively prevented.

(2) In case where the bolted splices are used, they should be placed near to the contraflexure point of moment of the column.

(3) Welded splices with either full penetration or partial penetration butt welds are allowed.

(4) For splice joints with full penetration butt welds, it is not necessary to check the joint resistance against the design loads, unless an undermatched electrode is used for the welds. In such cases, the welded joint resistance should be determined by using the strength of the undermatched electrode.

(5) For splices with partial penetration butt welds, the splice joint resistance should be determined on the basis of method for fillet welds with an effective throat thickness equal to the penetration depth.

(6) When the column splice is at the junction with the change of steel section thickness, as shown in Figure 4.1, a gradual change of plate thickness is preferred to avoid stress concentration at the welded joint.

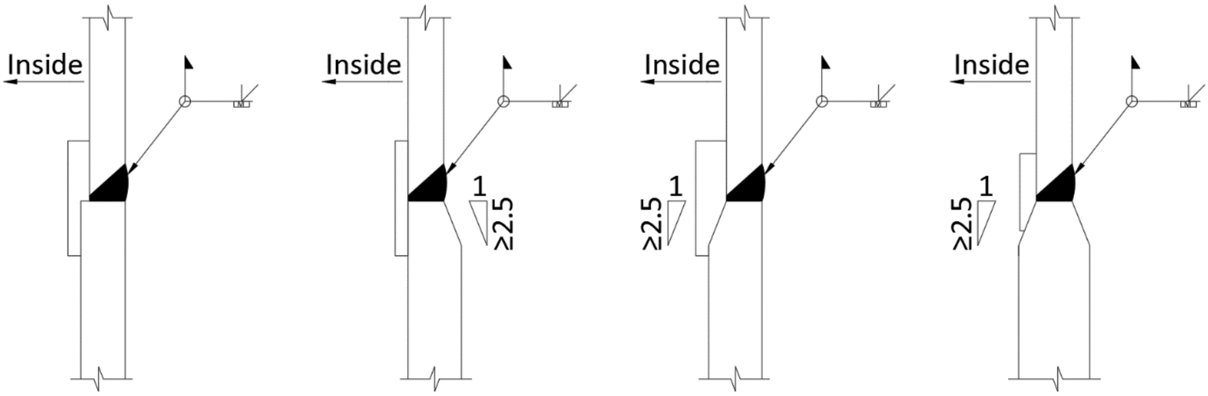
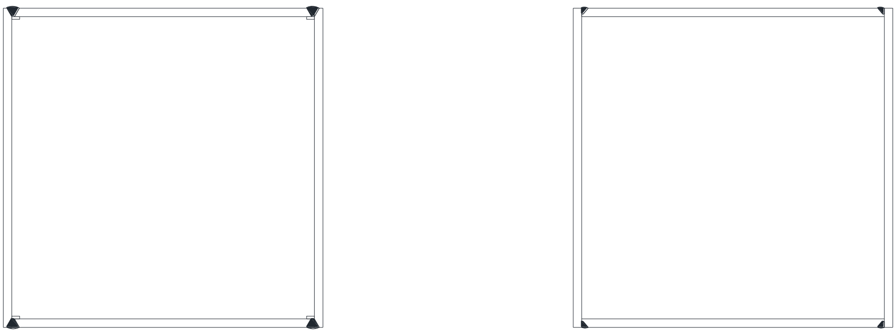


Figure 4.1 Splicing details for change of plate thickness

(7) For boxed column fabricated by four steel plates welded at the corners, full or partial penetration welds may be used as shown in Figure 4.2.



(a) full penetration (b) partial penetration

Figure 4.2 Full or Partial penetration welds for welded sections

4.3 Steel Beam to Column Joints

4.3.1 Simple Connections

(1) Simple connection with fin plate may be used as shown in Figure 4.3 for concrete filled steel tubular columns. A length of plate welded in the workshop to the column to which the supporting beam web is bolted on site. The flanges of the supporting beam are not

connected to the column. The supporting beam and column should be checked for shear and bearing. The design procedure may be referred to SCI Publication P358.

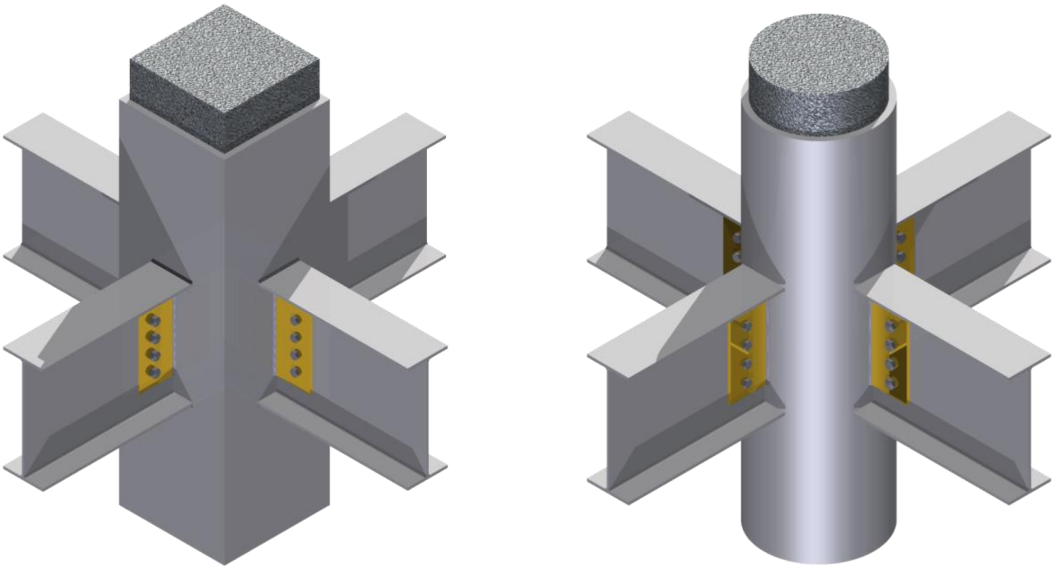


Figure 4.3 Simple connections between steel beams to concrete filled steel tubular column with fin plates

(2) Simple connection with the steel plate passing through the steel tube as shown in Figure 4.4 may be used for concrete filled steel tubular columns. Opening should be cut on the steel tube and all-round fillet welds may be used. The local bearing capacity of concrete under the through pin plate in the column should be checked in accordance with EN 1994-1-1, Clause 6.7.4 (6).

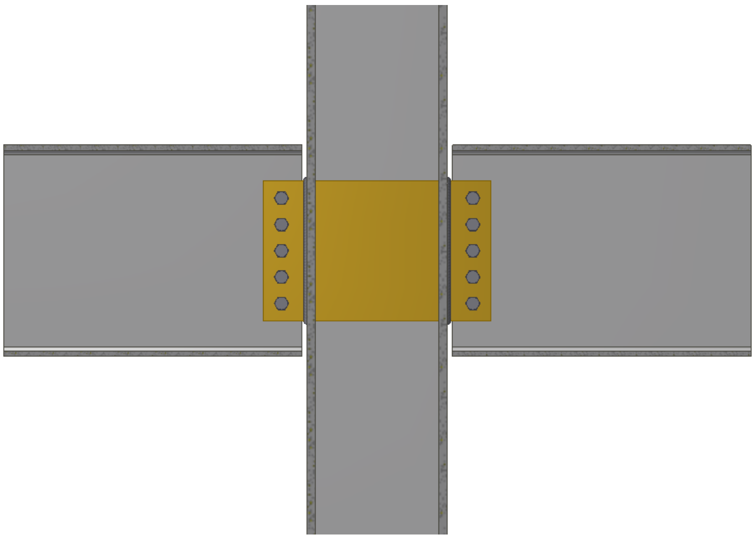


Figure 4.4 Simple connections between steel beams to concrete filled steel tubular column with through fin plate

(3) Simple connection with T-stub may be used as shown in Figure 4.5 for concrete encased steel columns. The T-stub is either welded to flange of the steel section through a fin plate or precast in concrete through bolts in the workshop and the web of the T-stub is bolted to the web of supporting beam. The flanges of the supporting beams are not connected to the column. The fin plate and bolts are designed against combined shear force and bending moment that is introduced by the eccentric shear force. The web of T-stub can be designed as the fin plates shown in Figure 4.3.

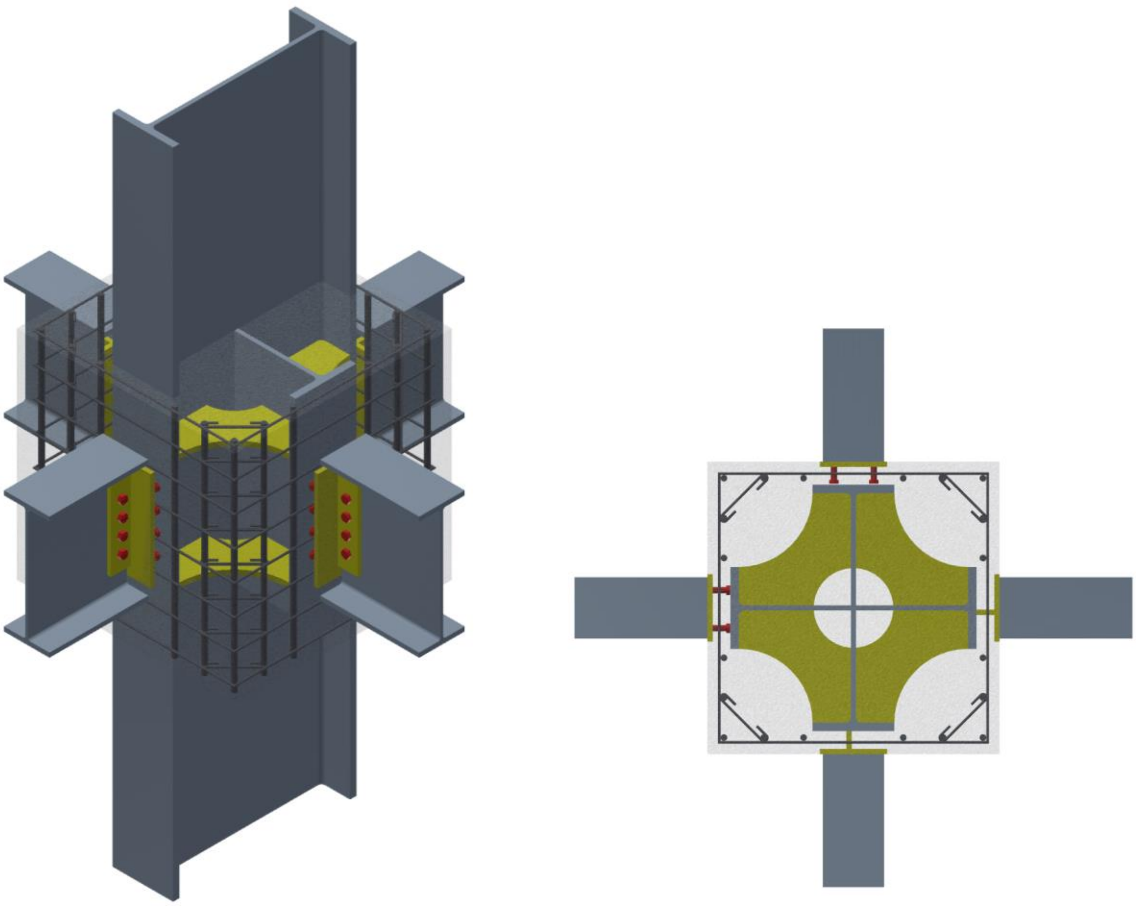


Figure 4.5 Simple connections between steel beams to concrete encased steel column with T-stub

(4) Simple connection with steel bracket (or steel corbel) may be used as shown in Figure 4.6 for concrete filled steel tubular columns and partially encased steel columns. The design procedure requires a check for the corbel subjected to combined shear and bending moment. The beam web at end should be checked for local bearing and shear buckling due to shear in accordance with EN 1993-1-1. Intermediate stiffeners may be used to improve the shear resistance of beam web at end.

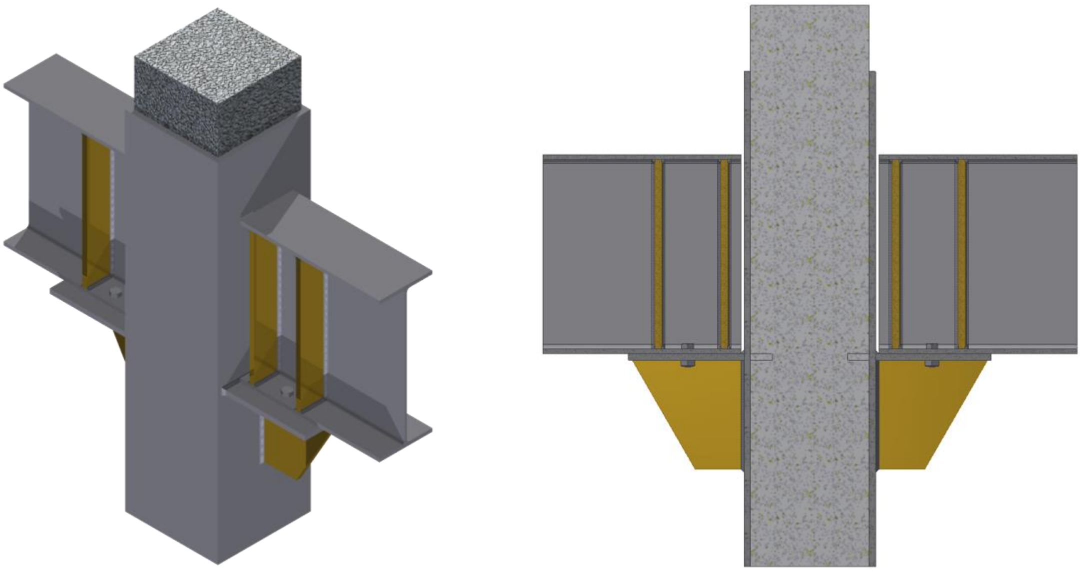


Figure 4.6 Simple beam-column connection with steel corbel

4.3.2 Moment Connections

(1) Moment connection with external diaphragm plate may be used as shown in Figure 4.7 for concrete filled steel tubular columns. The external diaphragm plates are connected with flanges of floor beams to transfer moment, whereas the webs are generally joined by bolts to transfer shear force.

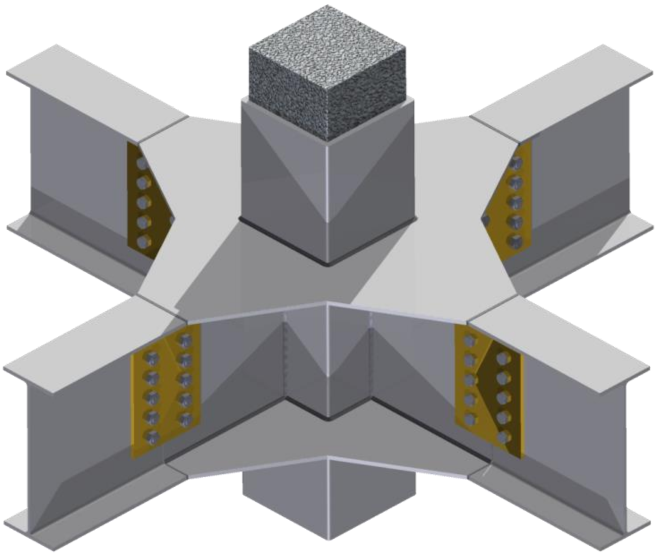


Figure 4.7 Steel beam to column moment connection with external diaphragm plates

(2) The thickness of the external diaphragm plate should be at least 6 mm to 10 mm larger than that of the connected beam flange in case of misalignment during installation.

(3) The minimum width of the external diaphragm plate c_{min}, as shown in Figure 4.8, should be at least $\sqrt{2}/2\ b_{\mathrm{f}}$, where b_f is the width of floor beam flange.

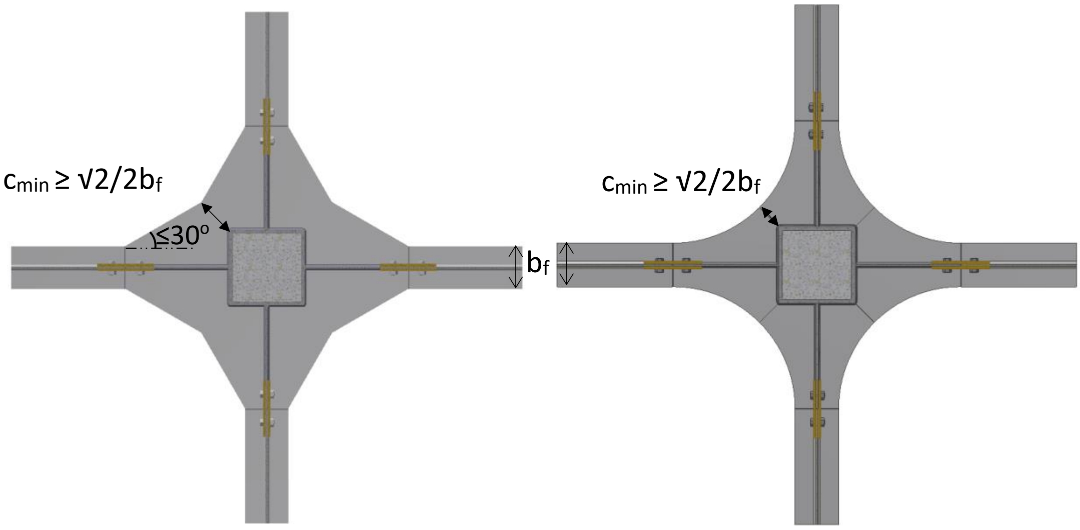


Figure 4.8 Detailing for external diaphragm plates

(4) In case where the external diaphragm plate is used for edge or corner column, the detailing as shown in Figure 4.9 may be adopted

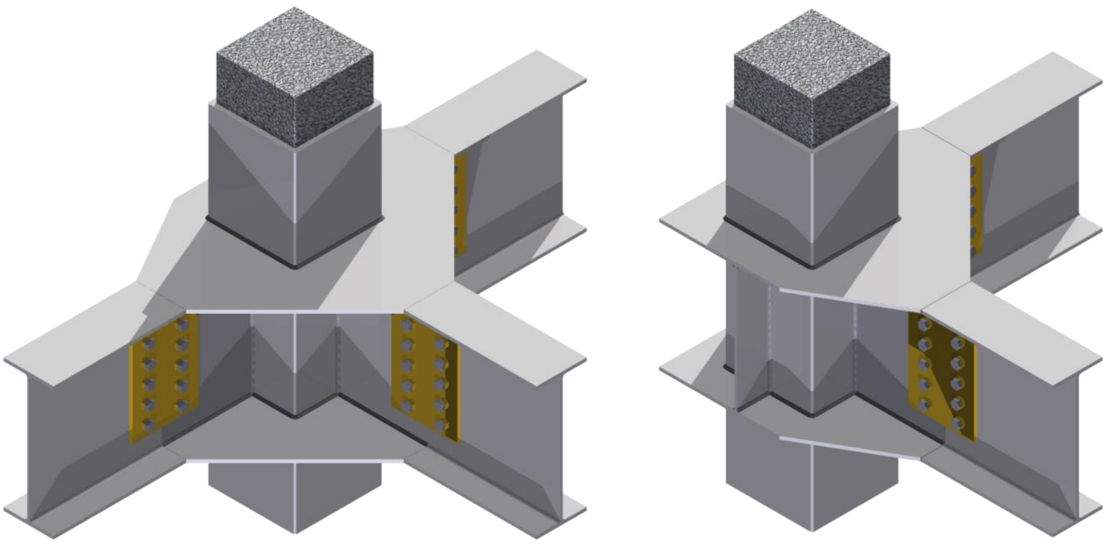


Figure 4.9 Detailing for edge and corner columns with external diaphragm plates

(5) Moment connection with internal diaphragm plate may be used as shown in Figure 4.10 for concrete filled steel tubular columns. Internal diaphragm plates may be used to transfer the moment from the beam to column and they are aligned with the beam flanges.

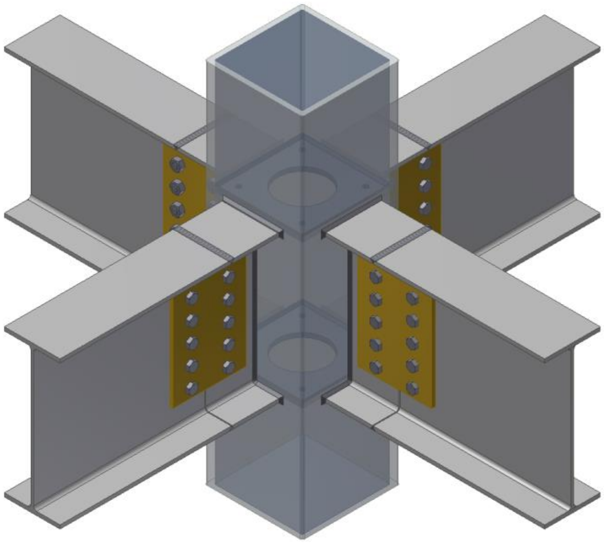


Figure 4.10 Steel beam to column moment connection with internal diaphragm plates

(6) The thickness of internal diaphragm plate should be 3 mm to 5 mm larger than that of the floor beam flanges to avoid misalignment. The width of internal diaphragm plate should be at least equal to $b_{\mathrm{f}}/2$ as shown in Figure 4.11.

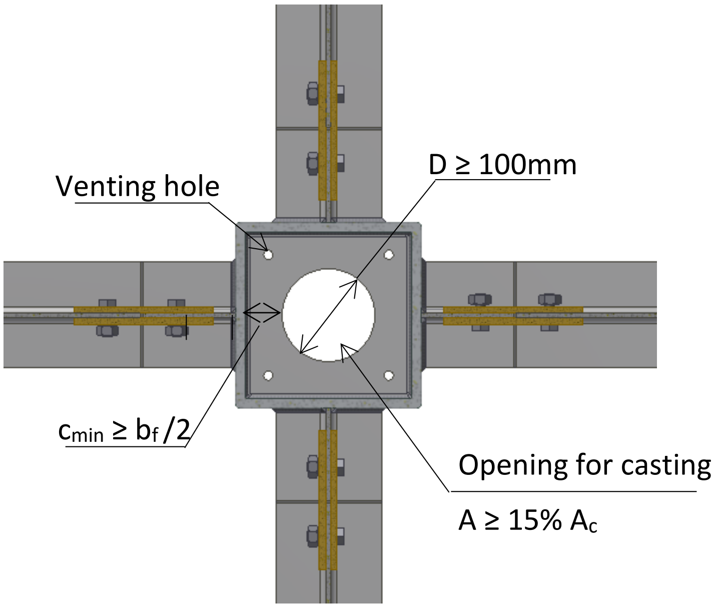


Figure 4.11 Detailing for internal diaphragm plate

(7) Opening area should be prepared for concrete casting as show in Figure 4.11. The opening could be either rectangular or circular. The opening size should be at least 100 mm and the area of opening should be greater than 15% the cross-sectional area of the core concrete. Four venting holes should be provided with diameter of 30 mm but not less than the thickness of the internal diaphragm plate.

(8) Moment connection with through-plate may be used as shown in Figure 4.12 for concrete filled steel tubular columns. The column tubes are discontinuously welded to the through plates with butt welds. The thickness of through-plate should be 6 mm to 10 mm larger than that of flanges of floor beam. Opening and venting holes should be provided on the through-plate for casting of concrete.

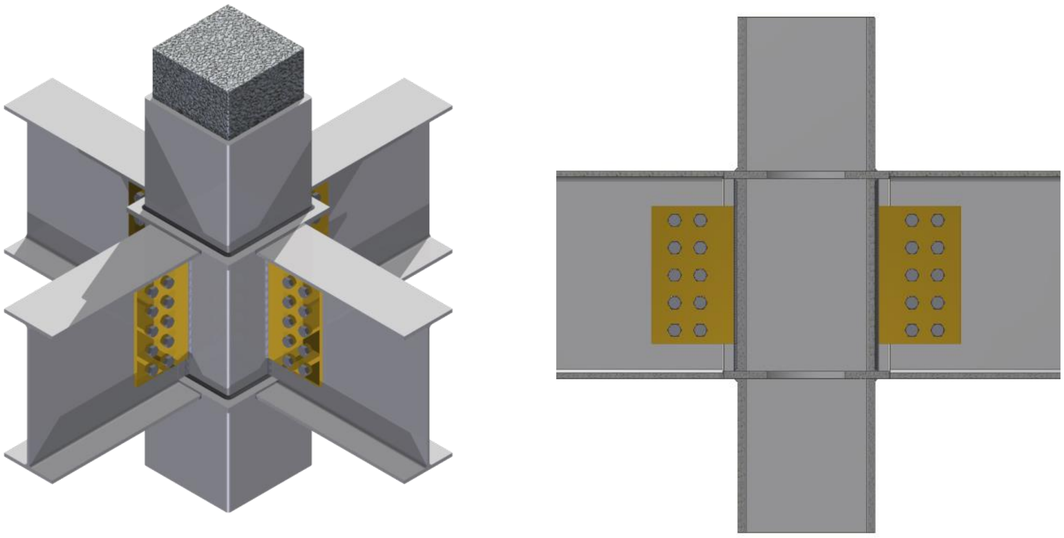


Figure 4.12 Steel beam to column moment connection with through-plate

(9) Moment connection with through beam may be used as shown in Figure 4.13 for concrete filled steel tubular columns. To enhance the bond resistance between the embedded flanges and the core concrete, shear studs may be welded on the flanges.

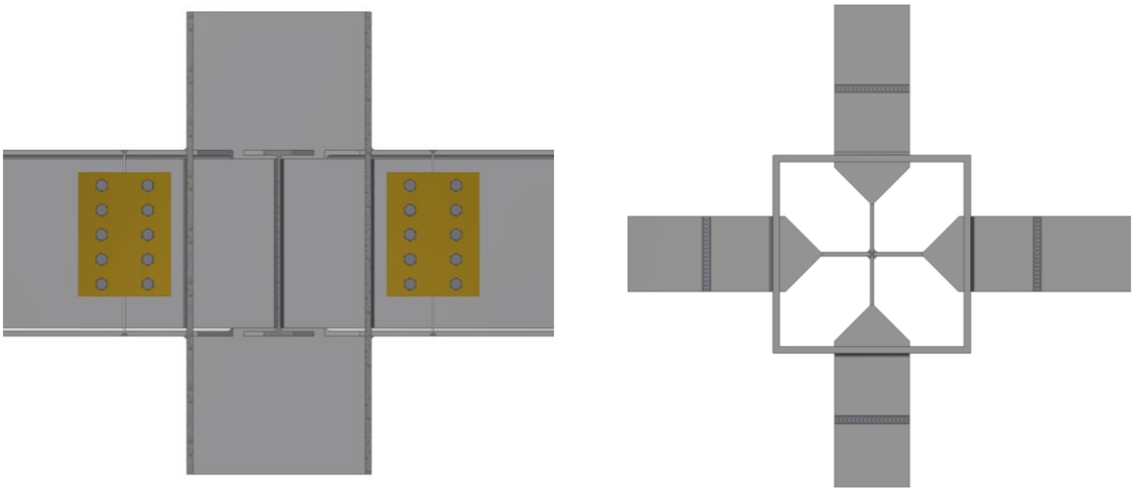


Figure 4.13 Beam to column moment connection with through beam

(10) In case where the connected beams have unequal heights, detailing as shown in Figure 4.14 (a) with oblique plate and Figure 4.14 (b) with additional plates may be adopted.

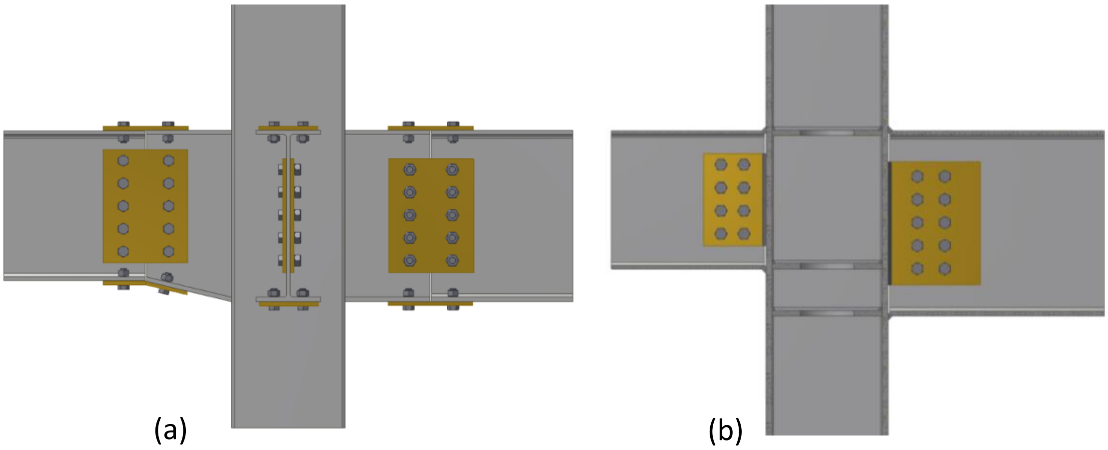


Figure 4.14 Steel beams with unequal depths connected to columns

(11) Moment connection may be achieved by welding the steel beam directly to the encased steel section or connecting it to the pre-fabricated cantilever beam. Internal diaphragm plates with openings are used to transfer moment and aligned with the beam flanges. The shear links of column are welded or pass through the web of beam.

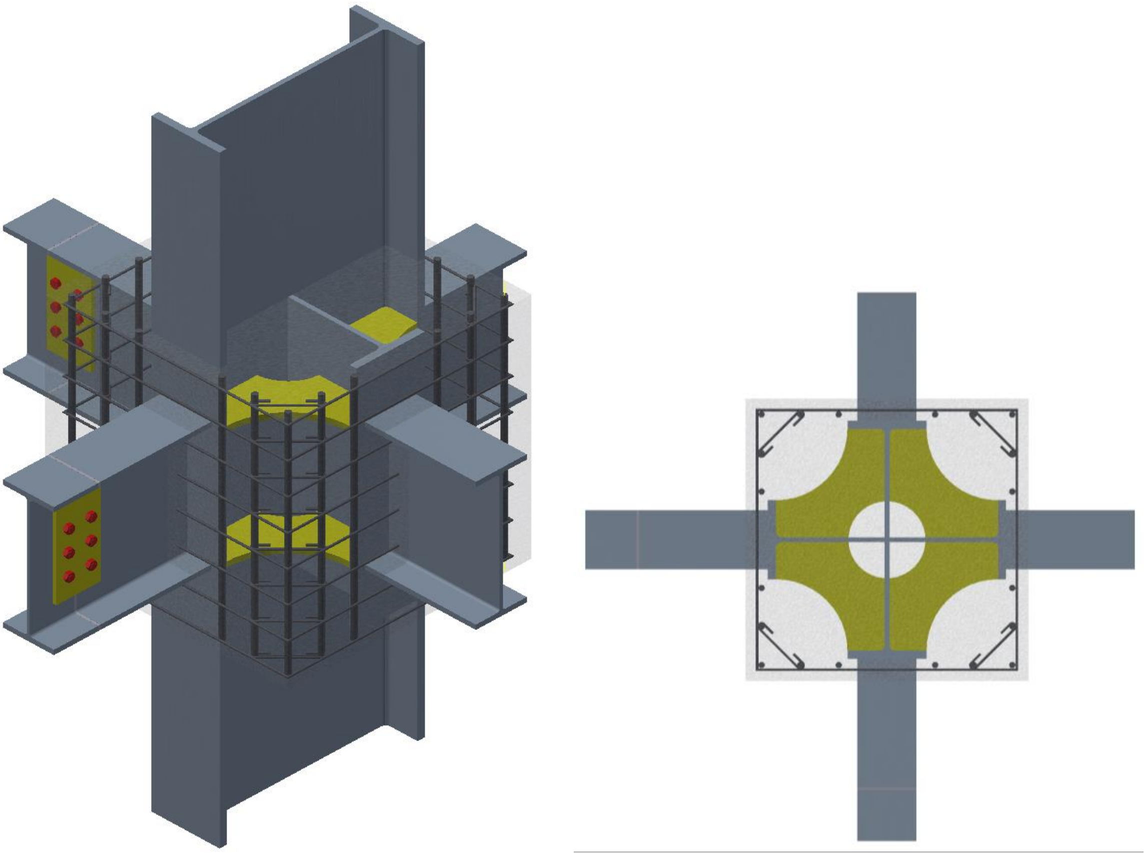


Figure 4.15 Moment connection between steel beam and concrete encased steel column

4.4 Concrete Beam to Column Joints

4.4.1 Simple Connections

(1) Simple connection with the steel corbel may be used as shown in Figure 4.16 for concrete filled steel tubular columns.

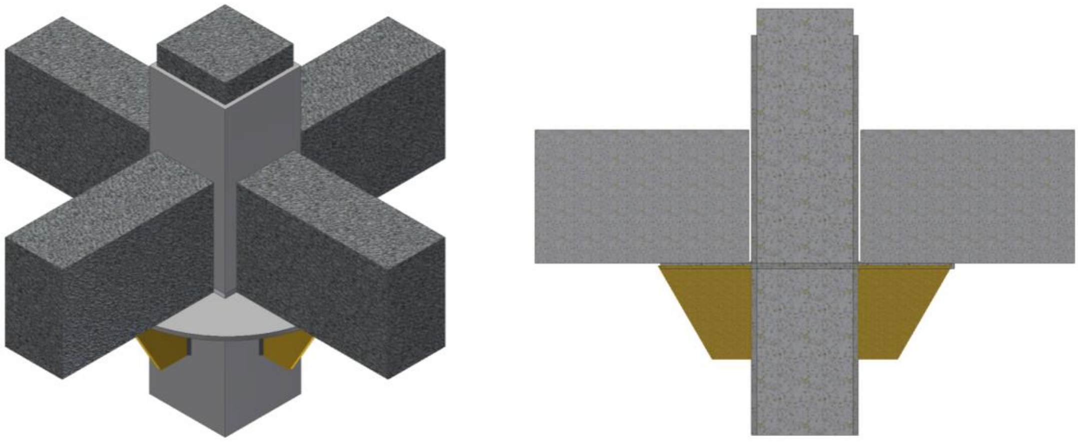


Figure 4.16 Simple connection between concrete beam and concrete filled steel tubular column with supporting corbel

(2) In case where the head room is not enough, the steel corbels may be embedded in the concrete beams as shown in Figure 4.17(a). When the reaction forces from beams are large, the corbels may be continuous through the steel tube as shown in Figure 4.17(b).

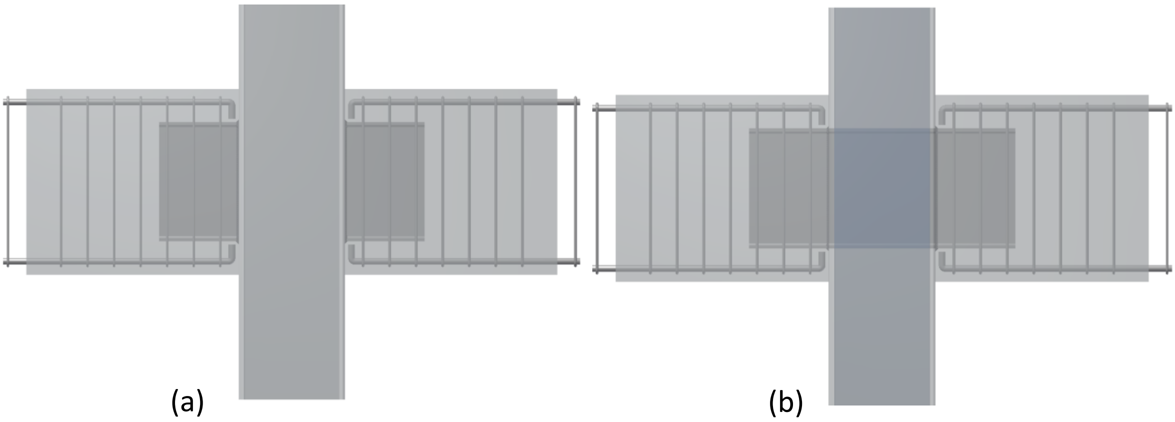


Figure 4.17 Simple connection between concrete beam and concrete filled steel tubular column with embedded corbel

4.4.2 Moment Connections

(1) Moment connection with through reinforcements may be used as shown in Figure 4.18 for concrete filled steel tubular columns. Holes or slot holes may be cut for individual or bundle of reinforcements. Strengthening should be provided for the holes.

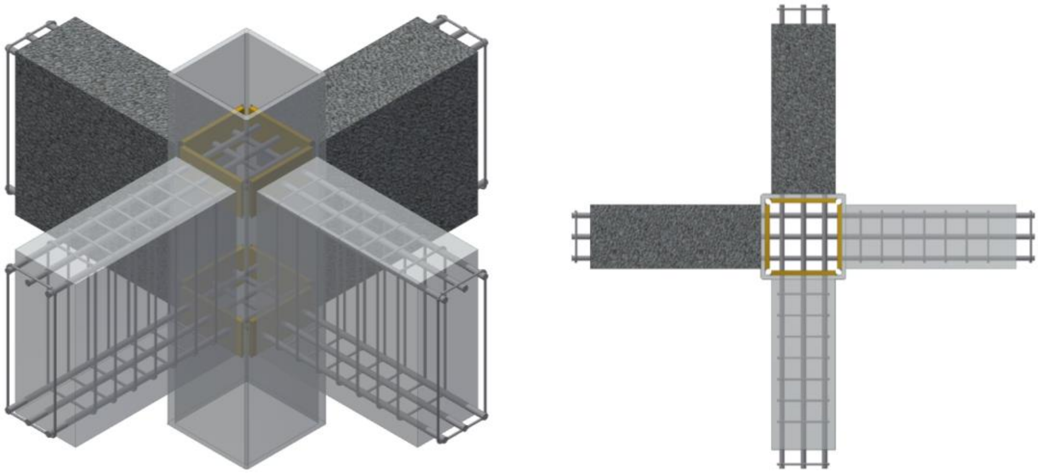


Figure 4.18 Moment connection between concrete beam and concrete filled steel tubular column with through reinforcements

(2) Moment connection with ring corbel may be used as shown in Figure 4.19 for concrete filled steel tubular columns. The longitudinal reinforcements are anchored to the ring corbel by groove welds. The effective throat thickness and length of the groove welds should be determined based on the tensile forces transmitted from the longitudinal reinforcements. The weldability of reinforcements should be checked with BS EN 10080.

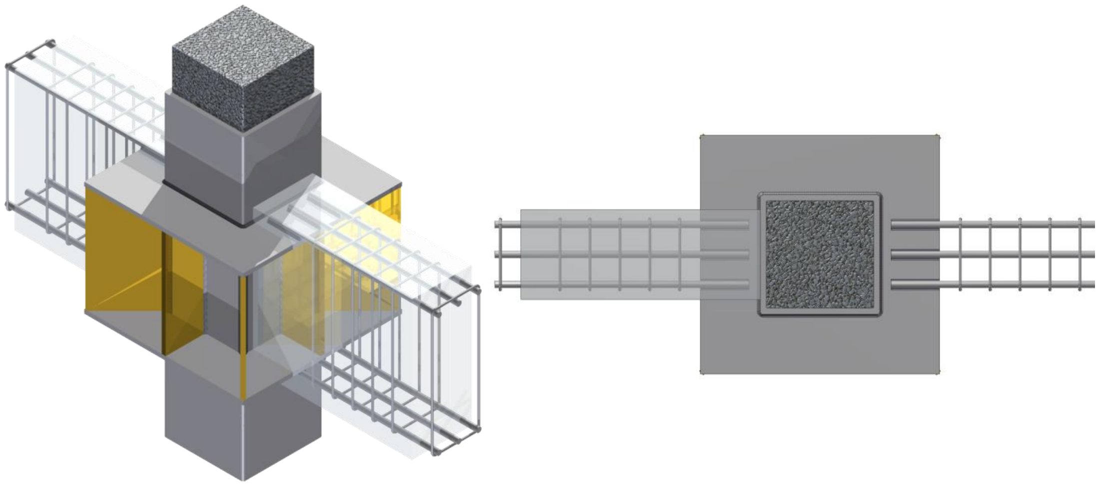


Figure 4.19 Moment connection between concrete beam and concrete filled steel tubular column with ring corbel

(3) Moment connection may be achieved as Figure 4.20 for concrete encased steel columns where the longitudinal reinforcements in one direction are welded to steel corbel and internal diaphragm plate, and the longitudinal reinforcements in the other direction are either welded to a padding plate on steel corbel or pass through the web of encased steel section. Alternatively, the longitudinal reinforcements may be welded to steel corbel and internal diaphragm plate on different sides of them as shown in Figure 4.21.

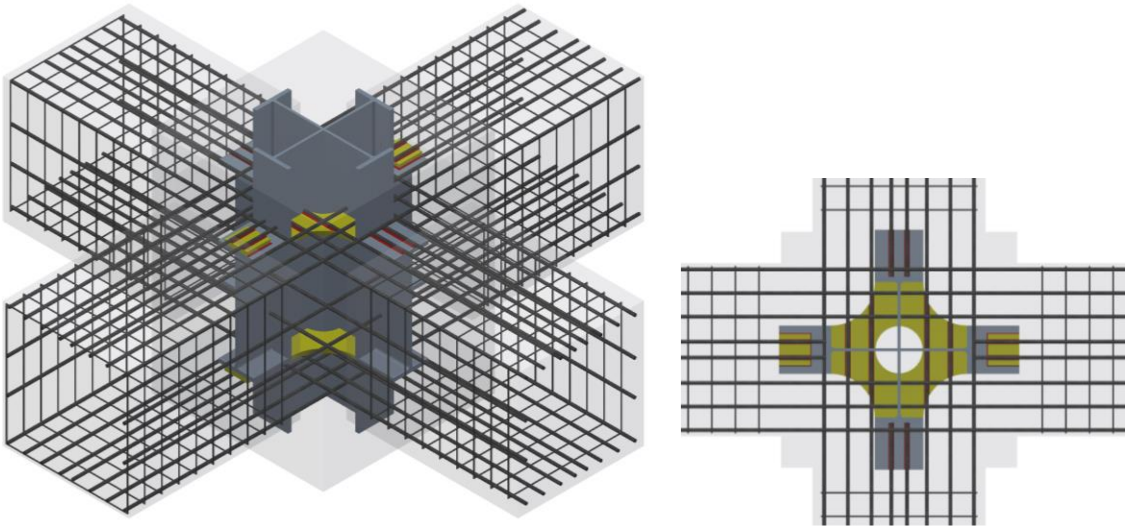


Figure 4.20 Moment connection between concrete beam and concrete encased steel column with bi-directional reinforcements on same sides of internal diaphragm plate and flange of steel corbel

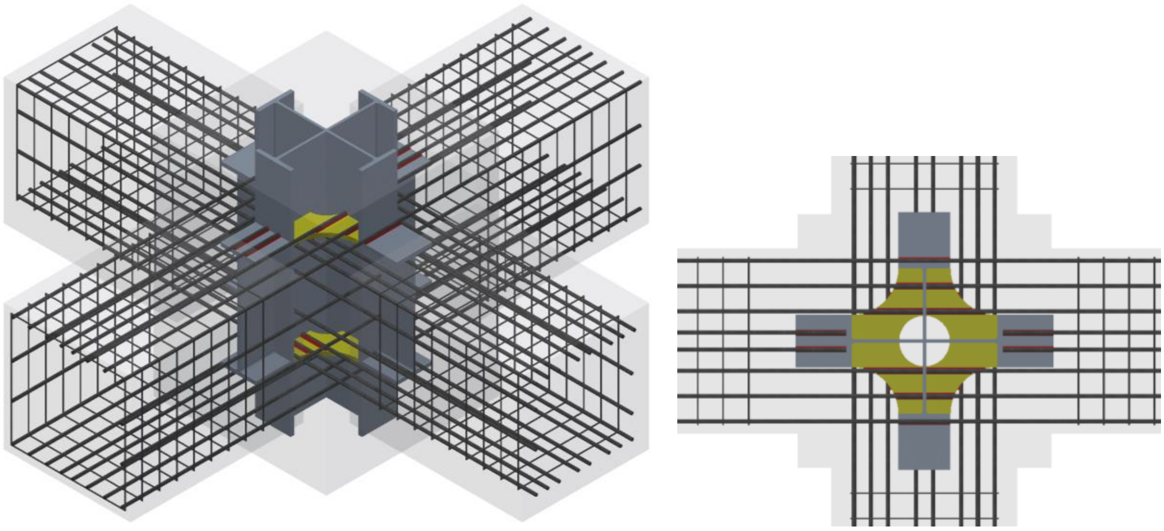


Figure 4.21 Moment connection between concrete beam and concrete encased steel column with bi-directional reinforcements on different sides of internal diaphragm plate and flange of steel corbel

4.5 Column Base

4.5.1 Simple Connections

(1) Column base with holding-down bolts may be used as shown in Figure 4.22 (a) for concrete filled steel tubular columns. Design for column base with pinned connection could be referred to SCI Publication P358.

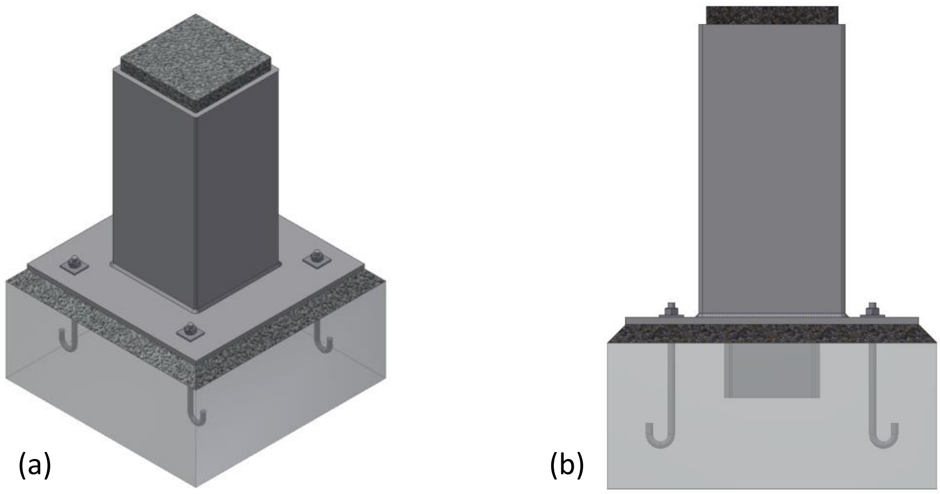


Figure 4.22 Column base with holding-down bolts

(2) The bedding grout shall be of non-shrinkage and at least equal in strength to that of foundation concrete. A bedding spacing of 25 mm to 50 mm is generally adopted, which gives reasonable access for thoroughly filling the space under the base plate.

(3) The bedding grout may be fine concrete with a maximum aggregate of 10 mm. The usual mix is 1:1.25:2 with a water-cement ratio between 0.4 and 0.45.

(4) The use of holding down bolts may be in accordance with BS 7419. The embedded length of the bolt in concrete should be in the range of 16 to 18 bolt diameters.

(5) The use of washers may be referred to BS EN ISO 7091. Alternatively, the washers may be cut from plates.

(6) In case where the transfer of high shear force to the column base is required, a shear stub (shear key) welded to the underside of the base plate may be necessary as shown in Figure 4.22 (b).

4.5.2 Moment-Resisting Column Base Connections

(1) Exposed column bases as shown in Figure 4.23 may be used for concrete filled steel tubular columns. When necessary, double base plates may be used to reduce the force acting on the plate stiffeners, and to reduce the number of stiffeners required.

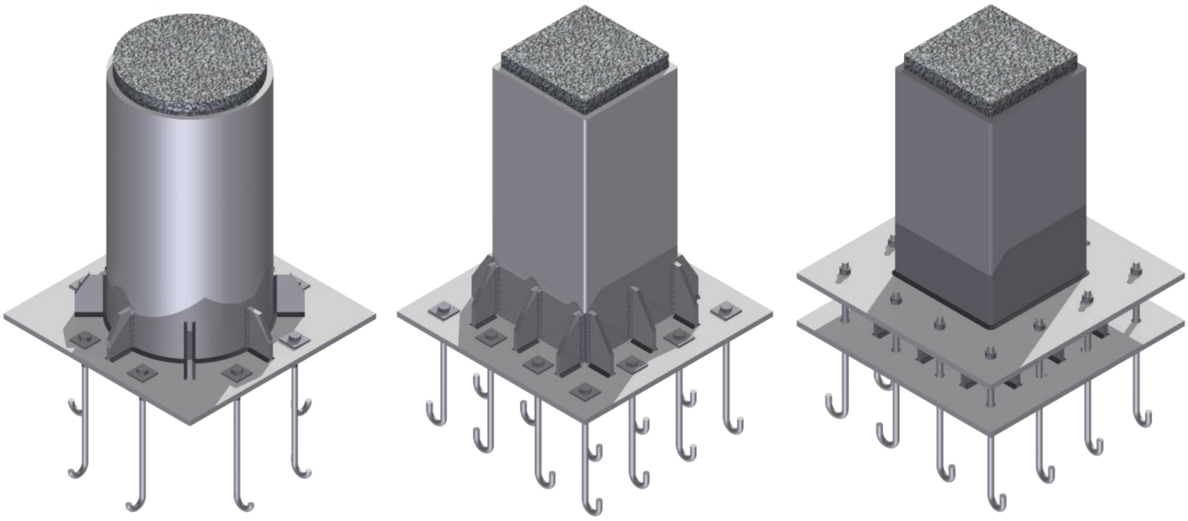


Figure 4.23 Typical exposed column bases for concrete filled steel tubular columns

(2) Exposed column base as shown in Figure 4.24 may be used for concrete encased steel columns. 50 mm bedding grout is needed and holding-down bolts are required for installation and positioning. Shear force is resisted by friction between base plate and the bedding grout and bending moment is resisted by longitudinal reinforcements anchored directly into the concrete base. In cases where said friction is inadequate to resist the shear, a shear key welded to the underside of base plate may be used.

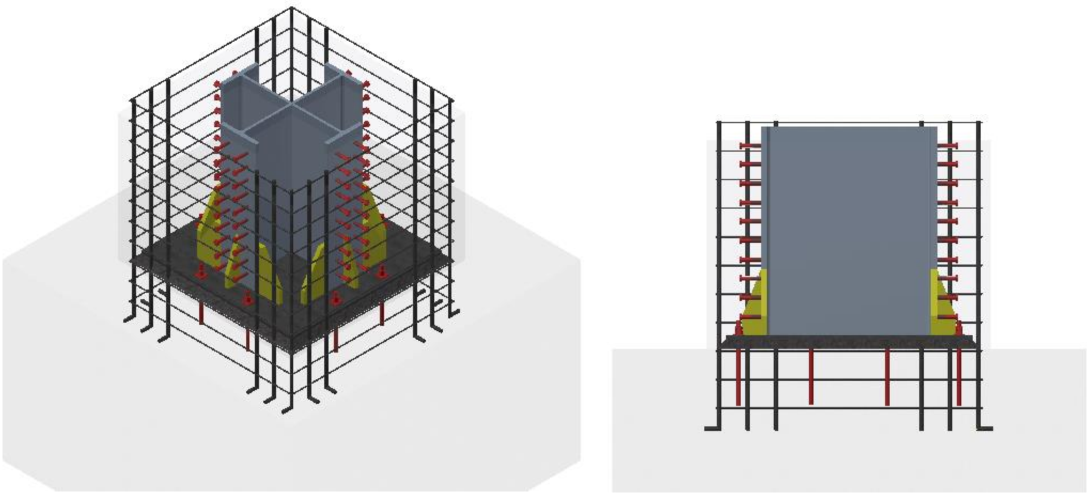


Figure 4.24 Typical exposed column base for concrete encased steel columns

(3) Embedded column base as shown in Figure 4.25 may be used for concrete filled steel tubular columns. Holding-down bolts are required for installation and positioning. The embedded height of the column base should not be less than 3 times the column section height.

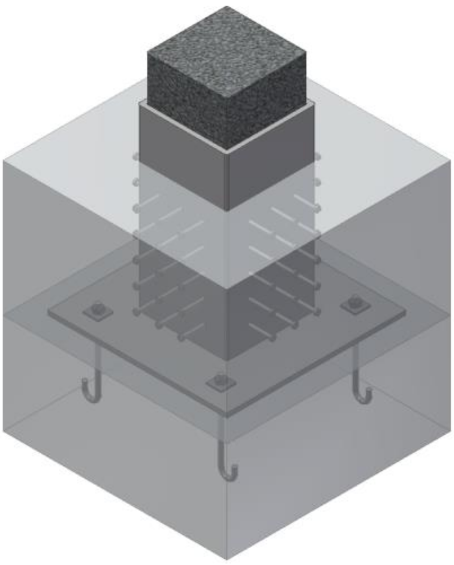


Figure 4.25 Typical embedded column base for concrete filled steel tubular columns

(4) Embedded column base as shown in Figure 4.26 may be used for concrete encased steel columns. Holding-down bolts are required for installation and positioning. The embedded height of the column base should not be less than 3 times the column section height.

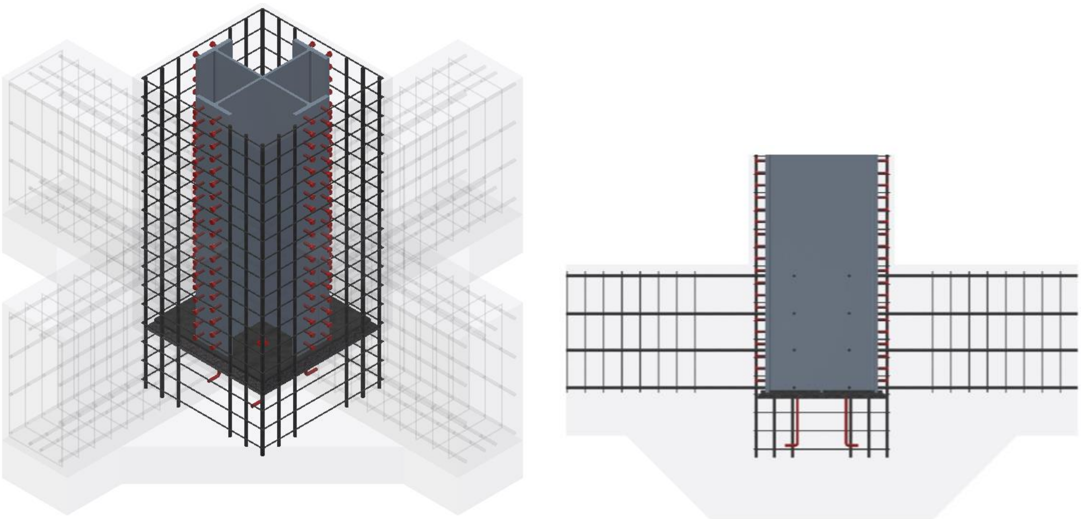


Figure 4.26 Typical embedded column base for concrete encased steel columns

(5) Concrete encased column base as shown in Figure 4.27 may be used for concrete filled steel tubular columns. The encased length should be at least 3 times the column cross-sectional height. In case of local crushing of concrete at top of the encased length, the top shear reinforcements should be strengthened.

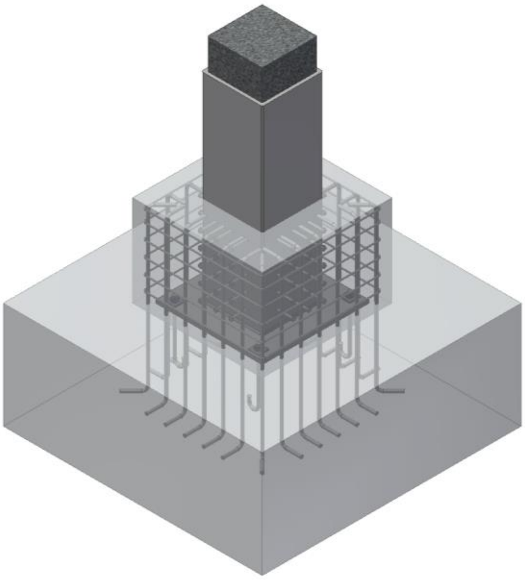


Figure 4.27 Typical concrete encased column base

(6) Concrete filled steel tubular columns at basement level may be fully encased with concrete to resist high axial compression force and accident loads such as car collision (with car park at the basement level), and to protect the steel tube against severe fire.

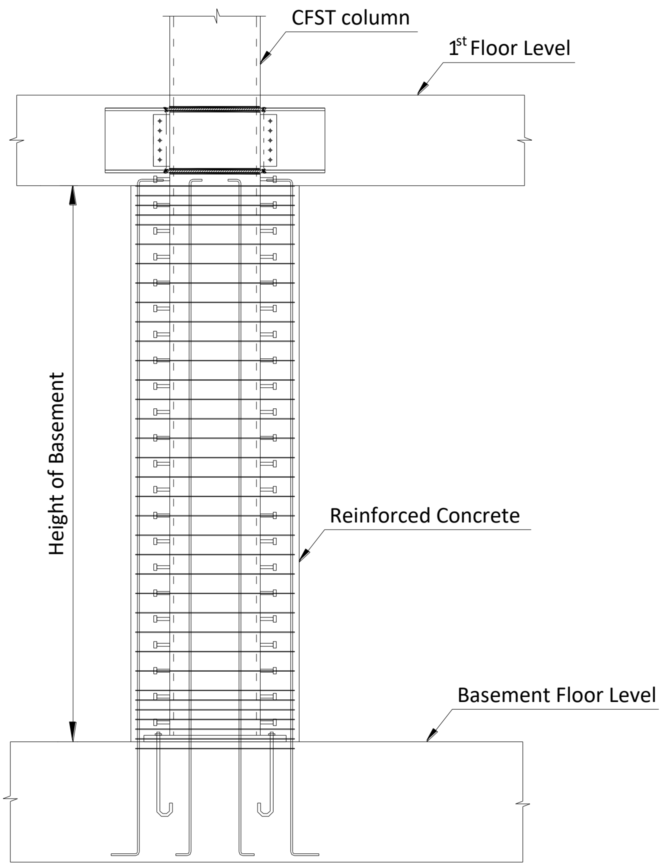


Figure 4.28 Concrete encased concrete filled steel tubular column at basement

5 Special Considerations for High Strength Materials

5.1 Fire Resistance of Steel-Concrete Composite Members

(1) The fire rating should be determined in accordance with the SCDF fire code for various types of occupancy.

(2) To determine the fire resistance of a composite member, the simple calculation models provided in EN 1994-1-2 may be followed.

(3) The advanced calculation models which practically resort to nonlinear finite element analysis are allowed. The advanced calculation models of thermal response shall be based on sound principles and assumptions of the theory of heat transfer.

(4) The risk of concrete spalling should be taken into account. For concrete grades C55/67 to C80/95, the spalling is unlikely to occur when the moisture content is less than 3.0% and if the maximum content of silica fume is less than 6% by weight of cement. Above the limitations, a more accurate assessment of moisture content, silica fume content, type of aggregate, permeability of concrete and heating rate should be taken into account. The assessment could be based on laboratory trial or specialist advices should be sought. Reader should refer to EN 1992-1-2 Clause 6.2 for additional guidance.

(5) For concrete grades higher than C80/95, spalling can occur in any situation for exposure directly to the fire. Thus at least one of the following methods should be provided:

- Include in the concrete mix more than 2 kg/m^3 of monofilament propylene fibres;
- A type of concrete for which it has been demonstrated by local experience or by testing that no spalling occurs under fire exposure.

The following statements (6) To (9) are applicable for concrete filled tubular members:

(6) For a concrete filled steel tubular member with normal strength concrete and mild steel, a complete thermal-stress analysis may not be necessary provided the temperature of the hollow section is lower than 350 ^{o}C.

(7) For a concrete filled steel tubular member with high strength concrete (f_{ck} > 50 N/mm^2) or high tensile steel (f_y > 460 N/mm^2), the fire resistance may be assumed to be satisfied provided the temperature of the hollow section is less than 300 ^{o}C.

(8) The addition of propylene fibres may affect the workability of concrete. In case where the high strength concrete is pumped into concrete filled steel tubular columns, specialist advice should be consulted for the pumpability.

(9) For the release of vapour from the concrete filled steel tubular columns, the hollow steel section shall contain steaming holes with a diameter of not less than 20 mm located at

least one at the top and one at the bottom of the column in every storey. The spacing of these holes should not exceed 5 meters along the column length.

The following statements (10) to (11) is applicable for concrete encased steel composite members:

(10) To determine the fire resistance of concrete encased steel composite column with concrete grade greater than C50/60, the tabulated data in Table 4.4 of EN 1994-1-2 may be followed provided the following requirements are met:

- Include in the concrete mix at least 2 kg/m^3 of monofilament propylene fibres;
- For load level $\eta_{fi,t} \leq 0.5$, the material compatibility requirement as specified in Table 2.6 2.6 should be observed; For load level $\eta_{fi,t} > 0.5$, the simple calculation model in EN1994-1-2 may be used to determine the fire resistance of concrete encased steel composite columns;

(11) For concrete encased steel composite columns subject to combined compression and bending, advanced calculation models in EN1994-1-2 may be used to determine the fire resistance.

5.2 Fabrication of High Tensile Steel Sections

(1) For high tensile steel in the quenched and tempered condition produced in according with EN 10025-6 and high tensile steel in the thermo-mechanically controlled condition in accordance with EN 10149-2, the hot forming is only permitted up to the stress relief annealing temperature. Provided higher temperatures are used for the hot forming, an additional quenching and tempering operation shall be required in which case the manufacturer shall be consulted.

(2) For high tensile steel in the quenched and tempered condition in accordance with EN 10025-6 (2004), the minimum inside bend radii for cold forming without cracks induced should be conformed to the values in Table 5.1.

Table 5.1 Minimum bend radii for cold forming of quenched and tempered steels

Steel class	Minimum inside bend radii ($\mathbf{mm}$)	
	Axis of bend in transverse direction	Axis of bend in longitudinal direction
S500Q/QL/QL1	3t	4t
S550Q/QL/QL1	3t	4t
Note: The values are applicable for bend angles $\leq$ 90° and plate thickness $\mathbf{t} \leq \mathbf{16mm}$.		

Table 5.2 Minimum bend radii for cold forming of thermo-mechanically controlled steels

Steel class	Minimum inside bend radii for nominal thickness in mm		
	$t \leq 3$	$3 < t \leq 6$	$t > 6$
S500MC	1.0t	1.5t	2.0t
S550MC	1.0t	1.5t	2.0t
Note: The values are applicable for bend angles ≤ 90°.			

(3) For high tensile steel in the thermo-mechanically controlled condition in accordance with EN 10149-2 (1996), the minimum inside bend radii for cold forming is given in Table 5.2.

(4) High tensile steel plates can be cold sheared. The maximum thickness of shearing should be determined based on the power available in the shear machine and the material used in the shear blades. The quality of the sheared edge are influenced by the machine setup and therefore the cutting blades should be well maintained.

(5) High tensile steel plates may be cut by oxy-fuel gas flame, abrasive water jet, and plasma techniques. Care should be taken as cutting underwater could result in a high hardness edge owing to the quenching effect.

(6) Hardness of free cut edges should be checked after cutting. For high tensile steels as concerned by EN 10025-6 and EN 10149-2, the permitted maximum hardness (HV 10) at the free cut edge is 450. Hardness testing with a load of HV10 shall be performed in accordance with EN 1043-1 or EN ISO 6507-1.

(7) The execution of bolt holes for high tensile steels may be done by process such as drilling, punching, laser, plasma or other thermal cutting. The local hardness and quality of cut edges around a finished hole should fulfil the requirements as for cutting.

(8) Hydrogen induced cold cracking for welding high tensile steel should be avoided. Appropriate welding procedures may be determined in accordance with Method A and Method B in EN 1011-2, JIS B 8285 and AWS D1.1.

(9) The necessity and requirements for preheat during welding should be consulted with steel manufacturers. In case where it is absent, Method B of EN 1011-2 may be referred to.

(10) Preheat should be extended to a zone of width of at least 4 times the thickness of the plate per side on both sides of the weld seam. For thickness greater than 25 mm, 100 mm adjacent to the seam on both sides is adequate.

(11) In cases where adequate preheat is impracticable, it is advisable to use austenitic or Ni-based welding consumables.

(12) Post-heat may be necessary when there is an increased risk of cold cracking, such as submerged arc weld for high tensile steels and a thickness greater than 30mm. The post-heat can be implemented by means of soaking, such as 2h/250°C, immediately after the welding.

(13) Hydrogen embrittlement induced by hot-dip galvanizing the high tensile steel plates should be avoided. To remove the potential hydrogen embrittlement, heating to 150°C after pickling and before galvanizing may be adopted for expulsion of hydrogen from the grain boundaries of steel. Another way is to use mechanical cleaning, such as shot or sand blasting, to remove the impurities instead of pickling. Nevertheless, a flash pickling after abrasive blast cleaning is required to remove any final traces of blast media before hot-dip galvanization.

(14) Test for the likelihood of hydrogen embrittlement for galvanized high tensile steels may be referred to ASTM A 143/A 143M.

(15) The inspection and testing procedures for welds on high tensile steels are similar as for mild steels, except special attention should be paid on the hydrogen induced cracks. Visual examination and non-destructive testing methods, such as radiographic or ultrasonic inspection, may be adopted.

(16) Due to the risk of delayed cracking of high tensile steel welds, a period of at least 48 hours is required before the inspection. The period shall be stated in the inspection records. For welds with heat-treatment to reduce the hydrogen content, the inspection may be carried out immediately after the heat-treatment.

(17) Personnel performing the visual examination and non-destructive testing shall have documented training and qualifications in accordance with EN ISO 9712.

(18) For direct visual examination, the access shall be sufficient to place the eye within 600 mm of the examined weld at an angle not less than 30° relative to background plane. An additional light source may be necessary to increase the contrast and relief between imperfections and the background. The visual examination shall be done in accordance with EN ISO 17637.

(19) In case where the radiographic inspection is chosen, Class B techniques, being more sensitive to cracks compared with Class A, should be used for high tensile steel welds. The technical requirements for Class B inspection should comply with EN ISO 17636.

(20) Provided the ultrasonic inspection is adopted, special attention should be paid for high tensile steel plates made from thermo-mechanically controlled process. General requirements for ultrasonic inspection shall conform to EN ISO 17640.

5.3 Preparation of High Strength Concrete

(1) High strength concrete may be produced with conventional Portland cement combined with fly ash and ground granulated blast furnace, silica fume slag. High early strength cements should be avoided. To maintain good workability, the cement composition and fineness's and its compatibility with the chemical admixtures should be carefully studied. Experience has shown that low-C_3A cements generally produce concrete with improved rheology.

(2) Care should be taken for the selection of aggregate to avoid the weak links formed on the aggregates. The higher the strength, the smaller the maximum size of coarse aggregate should be used. Up to 70 N/mm^2, compressive strength may be achieved with a good coarse aggregate of a maximum size ranging from 20 to 28 mm. Crushed rock aggregates should preferably be used.

(3) Particle size distribution of fine aggregate should meet the Eurocode specifications. Fine sands should not be used, particularly those with high absorption.

(4) Using supplementary cementitious materials, such as blast furnace slag, fly ash and natural pozzolans, not only reduces the production cost of concrete, but also addresses the slump loss problem. Generally, silica fume is necessary to produce the high strength concrete.

(5) Superplasticizer should be used to achieve maximum water reduction. The compatibility between cement and chemical admixtures and the optimum dosage of an admixture or combination of admixtures should be determined by laboratory experiments.

(6) Basic proportioning of high strength concrete mixture should follow the same method as for normal strength concrete, with the objective of producing a cohesive mix with minimum voids. Theoretical calculations or subjective laboratory trials may be necessary.

(7) The basic strength to water/cement ratio relationships used for producing normal strength concrete are equally valid when applied to high strength concrete, except that the target water/cement ratio can be in the range of 0.18-0.3 or even lower.

(8) High strength concrete containing superplasticizer should be transported, placed and finished prior to the loss of workability.

(9) The same production and quality control techniques for normal strength concrete should be applied to high strength concrete. The importance of strict control over material quality as well as over the production and execution processes should not be over-emphasized for high strength concrete.

(10) More compaction is normally required for high strength concrete than for normal strength concrete of similar slump. As the loss in workability is more rapid, prompt

finishing becomes essential. To avoid plastic shrinkage, the finished concrete surface needs to be covered rapidly with water-retaining curing agents.

5.4 Casting of Concrete in Steel Tubes

(1) Concrete casting in hollow steel tubes may be conducted by means of tremie tube and pumping.

(2) The tremie should be fabricated of heavy-gage steel pipe to withstand all anticipated handling stresses, and should have a diameter large enough to prevent aggregate-induced blockages. Pipes with diameter of 200 to 300 mm are generally recommended. A stable platform should be provided to support the tremie during the placement of concrete. Tremie pipes should be embedded in the fresh concrete with a depth of 1.0 ~ 1.5 m. The embedment depths depend on placement rates and setting time of the concrete. All vertical movements of the tremie pipes should be done slowly and carefully to prevent loss of seal.

(3) The maximum height of pumping should not exceed 60 m. For composite columns with internal diaphragm plates, the pumping rate should not exceed 1 m/min to avoid the air entrapment. The location of pumping inlet should be 300 mm away from the floor level.

5.5 Differential Column Shortening

(1) Differential shortening between columns and core walls due to different material strengths, stress levels, and long-term creep and shrinkage of concrete should be taken into account.

(2) Aggregates play an important role in both creep and shrinkage. Increase of fraction, size and modulus of aggregates may cause decreases of creep and shrinkage. In this aspect, high strength concrete exhibits less creep and shrinkage than normal strength concrete. In situation of large differential column shortening, high strength concrete is recommended.

(3) More accurate analysis may be used to determine the differential shortening between columns and core walls, taking into account for the time dependent creep and shrinkage strains in accordance with EN 1992-1-1. The effect of construction sequence should be taken into account.

(4) Column size, concrete strength and steel contribution ratio etc. may be adjusted to reduce the differential shortening. Alternatively, column length may be corrected based on the calculated differential shortening. The correction may be done after several storeys have been constructed.

(5) Simple connections allowing for vertical slip may be used for the floor beams and outriggers to relief the internal forces induced by the differential shortening. In case

where rigid connections are adopted, the connections may be made simple in the construction stage and become rigid after the creep and shrinkage have sufficiently developed.

References

ASTM A 143/A 143M (2003). Standard practice for safe guarding against embrittlement of hot-dip galvanization structural steel products and procedure for detecting embrittlement.

AWS D1.1/D1.1M (2010). Structural welding code – Steel.

BC 1 (2012). Design guide on use of alternative structural steel to BS 5950 and Eurocode 3. Building and Construction Authority, Singapore.

BC 2 (2008). Design guide of high strength concrete to Singapore standard CP 65. BCA Sustainable Construction Series – 3, BCA Academy, Singapore.

BS 4449 (1997). Specification for carbon steel bars for the reinforcement of concrete.

BS 4449 (2005) + A2 (2009). Steel for the reinforcement of concrete – Weldable reinforcing steel – Bar, coil and decoilded product – Specification.

BS 5950-1 (2000). Structural use of steelwork in building. Part 1: Code of practice for design - Rolled and welded sections.

BS 7419 (2012). Specification for holding down bolts.

BS EN 10080 (2005). Steel for the reinforcement of concrete – Weldable reinforcing steel – General.

BS EN ISO 898-1 (1999). Mechanical properties of fasteners made of carbon steel and alloy steel – Part 1: Bolts, screws and studs.

BS EN ISO 7091 (2000). Plain washers, Normal series, Product Grade C.

BS EN ISO 13918 (2008). Welding – Studs and ceramic ferrules for arc stud welding.

BS EN ISO 14555 (2006). Welding – Arc stud welding of metallic materials.

EN 1011-2 (2001). Welding – Recommendations for welding of metallic materials – Part 2: Arc welding of ferritic steels.

BS EN 1992-1-1:2004+A1:2014 Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings

EN 1992-1-2 (2004). Eurocode 2: Design of concrete structures – Part 1-2: Structural fire design.

EN 1993-1-8 (2005). Eurocode 3 - Design of steel structures – Part 1-8: Design of joints.

EN 1993-1-10 (2005). Eurocode 3 - Design of steel structures – Part 1-10: Materials toughness and through-thickness properties.

EN 1993-1-12 (2007). Eurocode 3 – Design of steel structures – Part 1-12: Additional rules for the extension of EN 1993 up to steel grades S700.

EN 1994-1-1 (2004). Eurocode 4: Design of composite steel and concrete structures – Part 1-1: General rules and rules for buildings.

EN 1994-1-2 (2005). Eurocode 4: Design of composite steel and concrete structures – Part 1-2: General rules – Structural fire design.

EN 10025-6 (2004). Hot-rolled products of structural steels – Part 6: Technical delivery conditions for flat products of high strength structural steels in the quenched and tempered condition.

EN 10149-2 (1996). Hot-rolled flat products made of high strength steels for cold forming – Part 2: Delivery conditions for thermomechanically rolled steels.

EN ISO 1043-1 (2011). Plastics - Symbols and abbreviated terms. Part 1: Basic polymers and their special characteristics.

EN ISO 6507-1 (2005). Metallic materials - Vickers hardness test. Part 1: Test method.

EN ISO 9712 (2012). Non-destructive testing of welds – Qualification and certification of NDT personnel.

EN ISO 17636-1 (2013). Non-destructive testing of welds – Radiographic testing. X- and gamma-ray techniques with film.

EN ISO 17636-2 (2013). Non-destructive testing of welds – Radiographic testing. X- and gamma-ray techniques with digital detectors.

EN ISO 17637 (2011). Non-destructive testing welds – Visual testing of fusion-welded joints.

EN ISO 17640 (2010). Non-destructive testing welds – Ultrasonic testing. Techniques, testing levels, and assessment.

JIS B 8285 (2010). Welding procedure qualification test for pressure vessels.

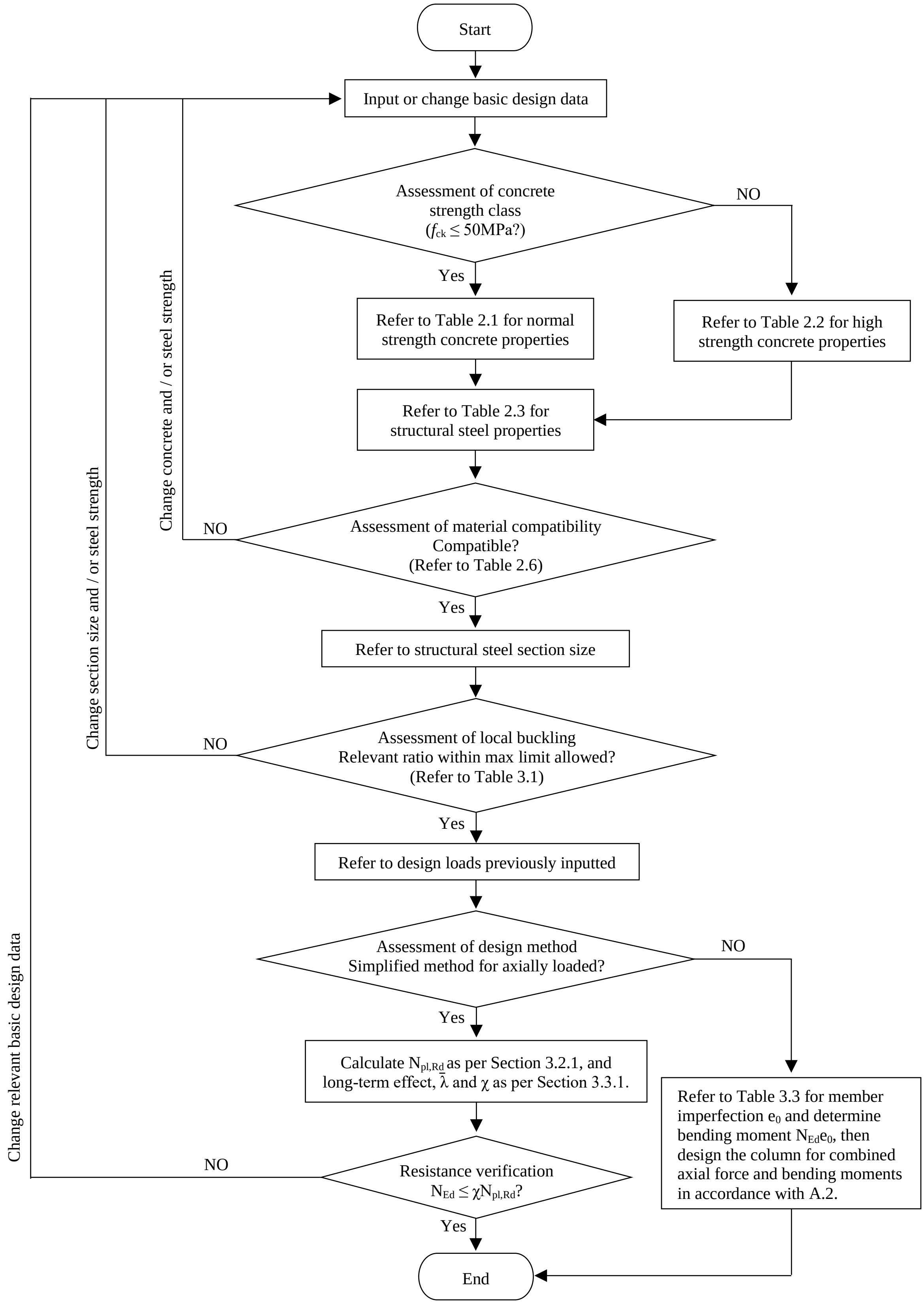
Liew J.Y.R., Xiong M.X. (2015). Design guide for concrete filled tubular members with high strength materials – An extension of Eurocode 4 method to C90/105 concrete and S550 steel. Singapore: Research Publishing Services, ISBN-13: 978-981-09-3267-1, 2015.

Liew, J.Y.R, Xiong M.X. and Lai, B.L. (2021). Design of steel-concrete composite structures – An extension of EN-1994-1-1 to high strength materials, Elsevier.

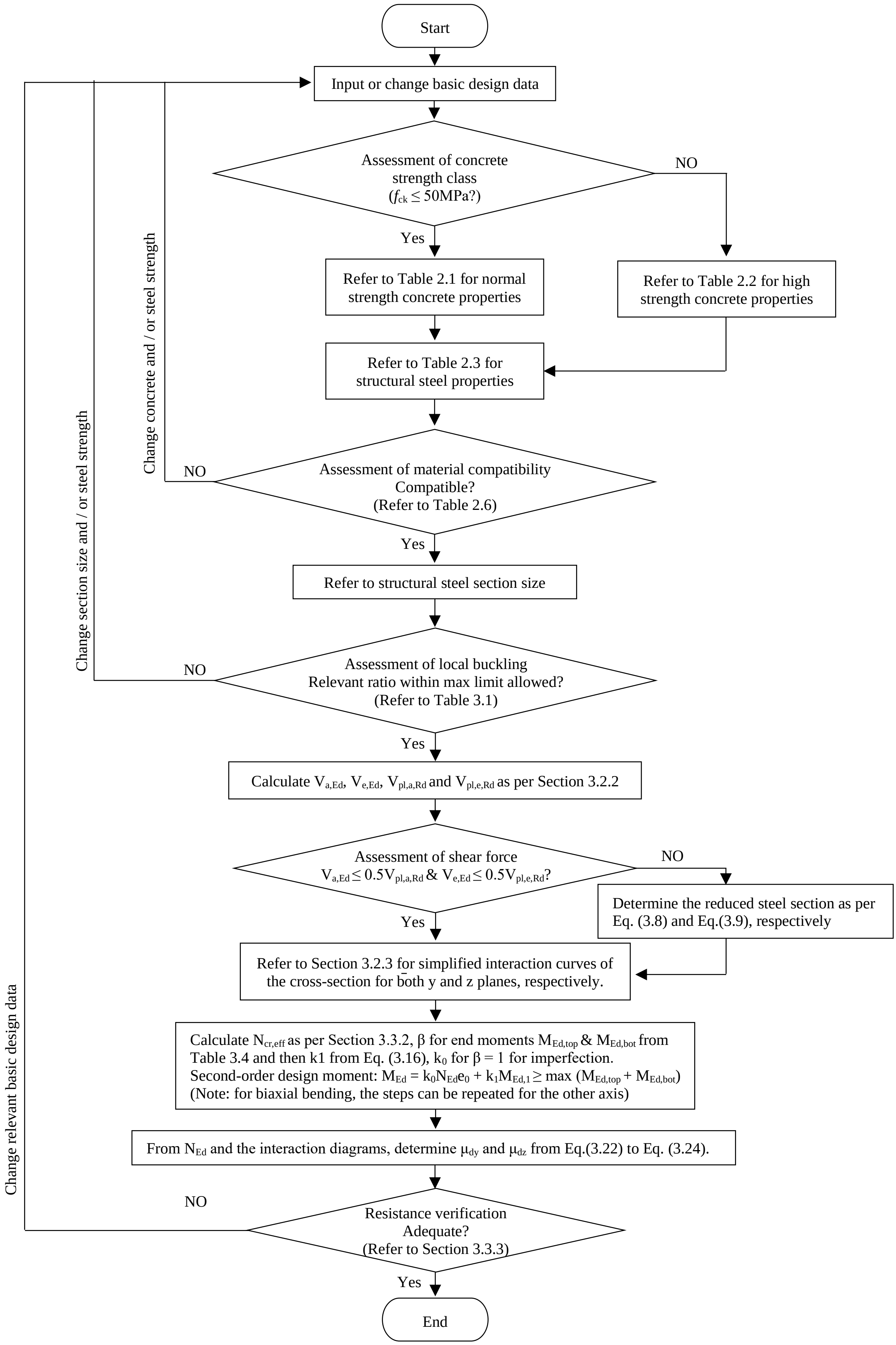
Steel Construction Institute (SCI: P358), (2011). Joints in steel construction: Simple joints to Eurocode 3, Steel Construction Institute, Tata Steel and British Construction Steel Work Association, UK .

Appendix A: Design Flowchart

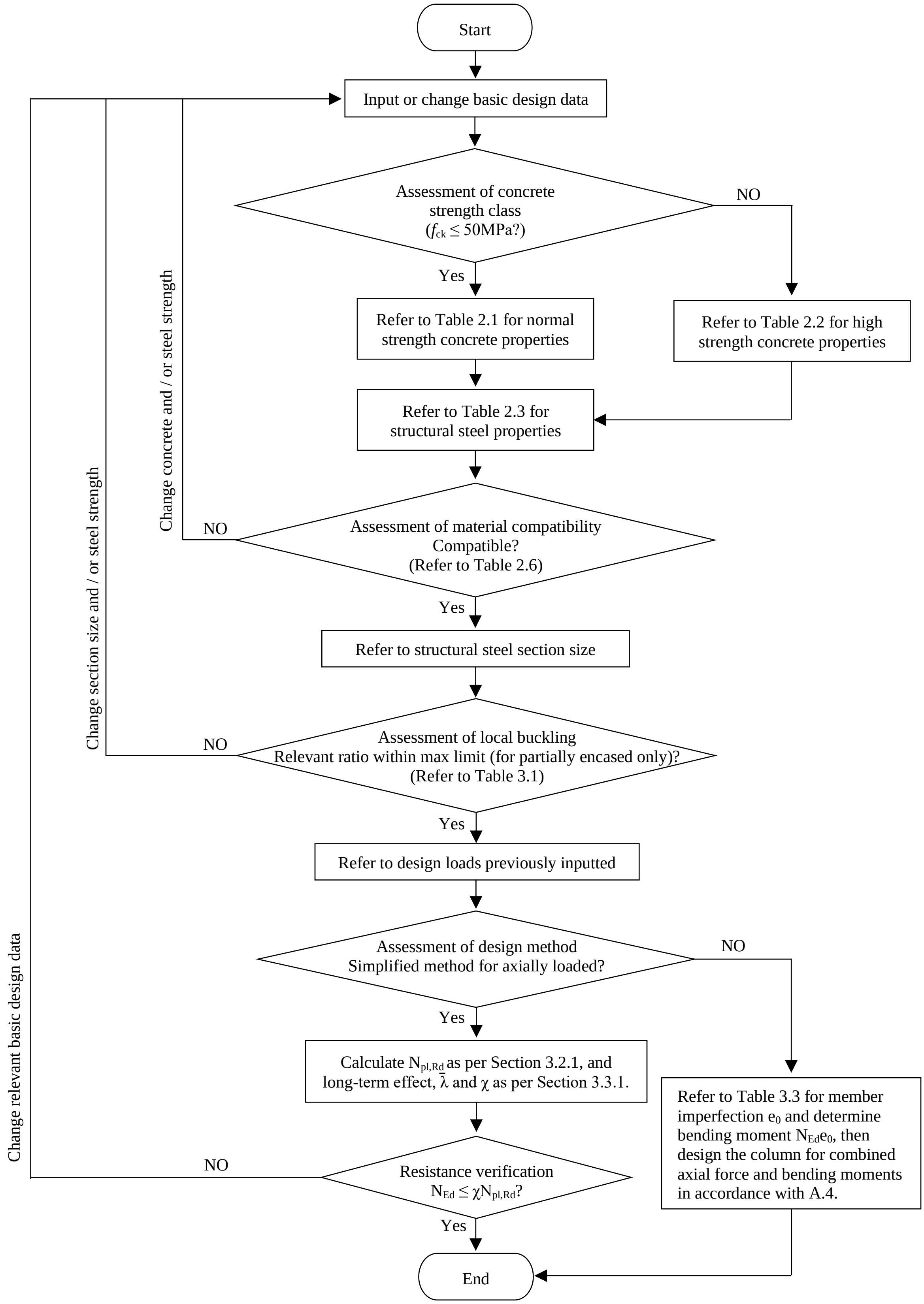
A.1 Concrete Filled Steel Tubular Column Subject to Axial Force Only



A.2 Concrete Filled Steel Tubular Column Subject to Combined Axial Force and Bending Moments



A.3 Concrete Encased Steel Column Subject to Axial Force Only



A4. Concrete Encased Steel Column Subject to Combined Axial Force and Moments